Numerical Modelling Report

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Dawlish Warren and Exmouth Beach Recharge
Technical Appraisal Study

Teignbridge District Council

June 2013

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Teignbridge District Council

June 2013

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Document history

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Teignbridge District Council

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Executive Summary

This report describes the coastal modelling work carried out for the Dawlish Warren and Exmouth Beach Recharge Technical Appraisal Study. The modelling work is comprised of four complementary studies; wave transformation modelling, shoreline evolution modelling, cross-shore modelling and detailed coastal area modelling.

A long time-series of wave data suitable for determining long term wave climate at Dawlish Warren and Exmouth was obtained from the Met Office European Wave model. The wave data is located offshore from the study area to the west of Lyme Bay. Wave transformation modelling has been undertaken to transform this data inshore for use in the other components of the modelling work, using Halcrow's existing calibrated MIKE 21 regional wave model. For this study the model was updated with the latest bathymetric data from the Channel Coastal Observatory and verified against the observed nearshore wave data used for the original calibration. The calibrated model was then used to establish a look up table to transform the complete time-series of 20 years of wave data inshore.

Shoreline evolution modelling, using Halcrow's COASTLINE model, has been undertaken to look at the evolution of the coastline over the next 20 years due to longshore transport. Shoreline models are one dimensional and may omit important two dimensional sediment transport mechanisms but are computationally inexpensive and can be used to assess the performance of management options over a long timescale where longshore transport is the dominant process. Models of Dawlish Warren and Exmouth Beach have been calibrated using observed shoreline positions and show good agreement with the data. These calibrated models have been used to assess a number of options for recharging the beaches and improving groynes along the beaches.

Cross-shore modelling, using Halcrow's cross-shore beach profile models COSMOS and SHINGLE, has been undertaken to determine the risk of significant erosion due to cross-shore transport during severe storm events at two locations on Dawlish Warren and Exmouth Beach. A joint probability analysis for extreme wave conditions at the inshore wave points and still water level at Exmouth Dock has been carried out to generate boundary conditions for use in the cross-shore model. The model has been verified against pre and post storm profiles for Dawlish Warren, and generally shows good agreement with observed data. However, no data for model verification was available for Exmouth Beach.

A two dimensional coastal area model of Dawlish Warren, Exmouth Beach, Pole Sand and the Exe Estuary has been developed to aid understanding of present day sediment pathways. The model has been constructed using MIKE 21 and includes tide and wave driven currents, surface waves and sand transport. A hydrodynamic model of the English Channel has been constructed to provide tidal boundary conditions for the local model. Both models are calibrated to within Environment Agency guidelines.

The models have been used to develop a comprehensive understanding of the sediment transport regime around Exmouth Beach and Dawlish Warren, and a summary of these processes is shown in Figure 8-1 at the end of this Executive Summary. Key findings of the modelling are:

Pole Sand and Bull Hill

- The flood and ebb deltas of Bull Hill and Pole Sand are increasing due to transport of sediment by tidal currents. The coastal area model shows a net deposition of 80 000 m^3/year of sediment over Bull Hill and 200 000 m^3/year over Pole Sand. The model also shows that Pole Sand is increasing in extent due to a net deposition of 300 000 m^3/year along both the southern and south-eastern edges of Pole Sand.

- The coastal area model shows westwards wave driven currents are set up along the southern edge of Pole Sand at low tide and across the central section of Pole Sand at the mid-point in the tide. These currents lead to the formation of accretional features offshore from Dawlish Warren and are a potential mechanism for supply of sediment from Pole Sand to Dawlish Warren.

Dawlish Warren

- The coastal area model shows north eastwards wave driven transport dominates between the south west end of Dawlish Warren and 7/8th of the way along the spit. At the north-west tip of Dawlish Warren tidal currents are dominant and the nearshore tidal sand movement is north or north-eastwards into the estuary. Sediment removed from the end of the spit by tidal currents enters the main flow out of the estuary and is eventually deposited over Pole Sand.

- The COASTLINE model shows erosion is greatest in the central section of the spit, in agreement with observed rates of change, and that without groynes the spit will be breached after twenty years as occurred in the 1940s. The coastal area model underestimates the rate of erosion over the south-western and central sections of Dawlish Warren and overestimates erosion at the north-eastern end.

- Cross-shore modelling shows that during a 100 year return period storm event the dune crest may be eroded by as much as 2 m. The spit may therefore be breached in an extreme storm event following gradual erosion due to longshore transport.

- The coastal area model shows that westwards currents set up along Pole Sand are a potential mechanism for supplying sediment to the south-western and central section of the spit.

Exmouth Beach

- **Section of beach between the mouth of the estuary and the cricket ground:** The COASTLINE model shows that this area is largely stable. The coastal area model shows that transport along this section is tidally dominated.

- **West of the cricket ground to Maer Rocks:** The coastal area model shows south eastwards transport (mainly due to tidal currents) along this section. This leads to accretion to the west of Maer Rocks due to a tidal eddy. This is not shown in the COASTLINE model as it does not include tidal currents.

- **In front of Queen's Drive between Orcombe point and Maer Rocks:** The coastal area model shows westwards transport along this section of the beach. The westward transport close to the beach (i.e. in the nearshore) is mainly due to wave driven currents. Further offshore, interactions with rocks and strong tidal currents lead to variable flow directions. The shoreline model shows severe erosion in this location.
- Cross-shore modelling shows that the beach in front of Queen's Drive is eroded by as much 0.6 m in a 100 year return period storm event and may retreat by 10 m.

The COASTLINE model has been used to assess options for reducing erosion Dawlish Warren and Exmouth Beach. Seven options were tested for Dawlish Warren and five options for Exmouth Beach.

- For Dawlish Warren the most beneficial/cost-effective option to hold the existing defence line in the short-term without compromising the long-term management objectives of the frontage is to replace all existing groynes, extend the three north-eastern groynes (numbers 12, 13 and 14), recharge the section of the beach between Groynes 11 to 14, and recharge the south-western section of the beach between Groynes 1 to 5. This option is expected to stabilise the Dawlish Warren coast for 20 years.
- For Exmouth Beach the most beneficial/cost-effective option to hold the existing defence line in the short-term is to install two new groynes at the eastern end of Queen's Drive and repair Groynes 5 and 6 at the western end of Queen's Drive, and to recharge the beach from Orcombe Rocks to Groyne 3.

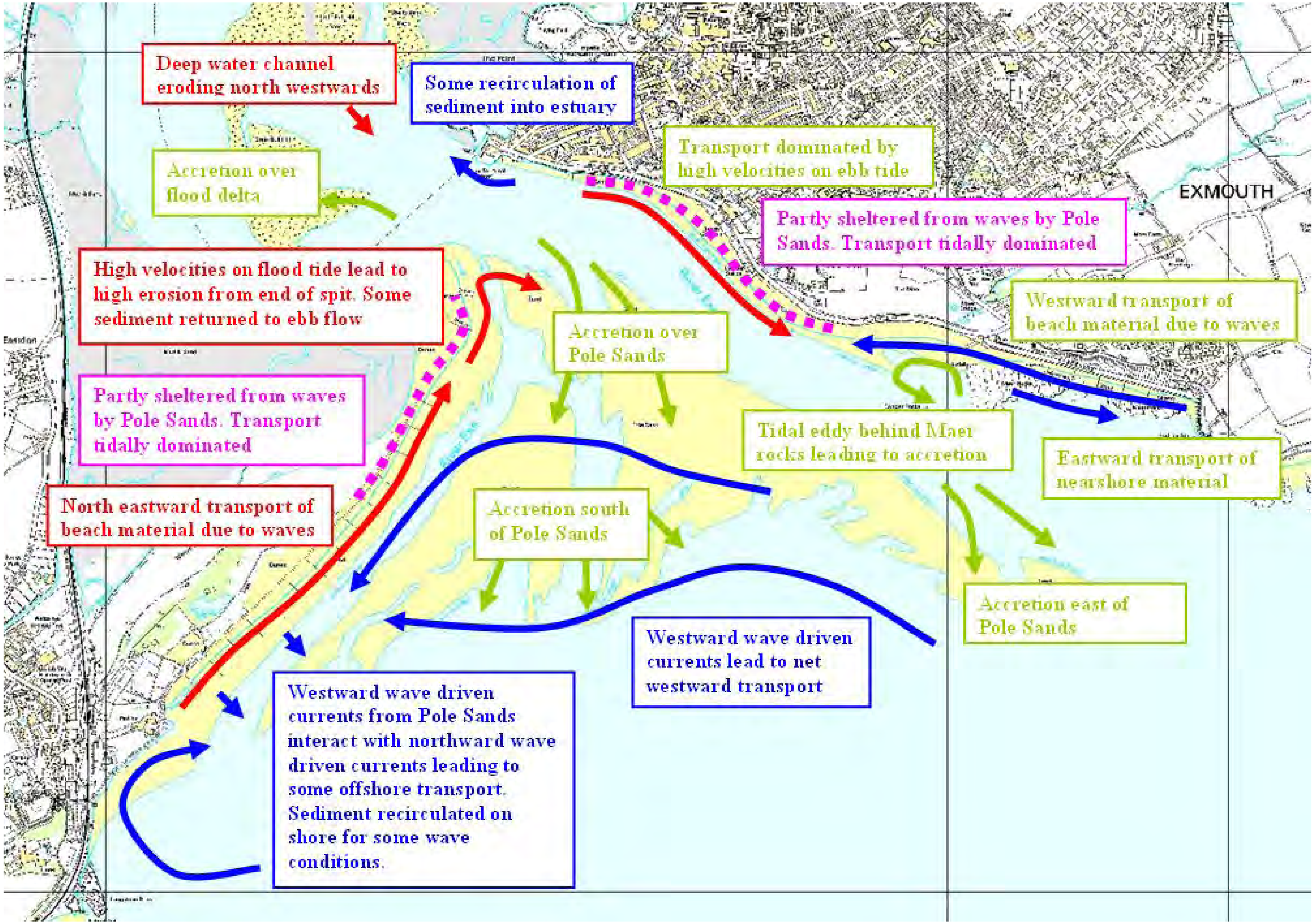


Figure 8-1: Summary of sediment transport processes around the mouth of the Exe Estuary.

1 Introduction

Halcrow Group Ltd (Halcrow) has been commissioned to undertake a Technical Appraisal Study for Recharge of Dawlish Warren and Exmouth Beach.

The aim of this project is to investigate the sediment transport processes and associated erosion and deposition affecting Dawlish Warren and Exmouth Beach, in order to develop a sustainable beach recharge or recycling option to safeguard these assets and their flood and coastal erosion defence functions in the short term (0-20 years).

Specifically, this will be achieved by:

- Assessing beach volume movements along the frontages using regional coastal monitoring programme data;
- Undertaking numerical modelling of the sediment transport processes and corresponding beach/shoreline evolution in the area;
- Investigating the potential sediment site in the area offshore of the eastern part of Exmouth;
- Identifying a preferred option for future management intervention; and
- Developing Beach Management Plans for both Dawlish Warren and Exmouth Beach in line with CIRIA Beach Management Manual (2010) best practice template to guide future management of the beach in line with the findings of this project.

This technical report presents the numerical modelling work undertaken for the project. The remainder of this report is structured as follows:

- **Section 2** describes the project data, including bathymetry data, wave data, tidal data and sediment data.
- **Section 3** presents the wave modelling.
- **Section 4** presents the extreme value analysis of wave heights water levels.
- **Section 5** presents the longshore transport and coastline evolution modelling.
- **Section 6** presents the cross-shore transport and beach profile evolution modelling.
- **Section 7** presents the two-dimensional sediment modelling.
- **Section 8** provides a summary of the modelling work and presents the conclusions from the study.

2 Data Review

2.1 Bathymetric data

Three sources of bathymetric data are available for the study area, local bathymetric survey, LiDAR and digitised admiralty chart data in the DHI C-MAP database.

2.1.1 LiDAR and local bathymetry surveys

LiDAR data for 2010 and 2011 and swath bathymetry for 2010 was obtained from the Channel Coastal Observatory. Bathymetric survey for 2005 was extracted from the Exe Estuary strategy model and reused for this project. Both LiDAR and bathymetric surveys are to Ordnance Datum Newlyn (mODN).

The extent of the different data sets is shown in Figure 2-1. Both bathymetric and LiDAR data is available for Pole Sand, and the difference between the two data sets over Pole Sand is shown in Figure 2-2. Over most of the area differences between the two data sets are below 0.25 m. Larger differences occur around the edges of Pole Sand. This may be due to these areas being partially covered by water during the LiDAR survey as LiDAR cannot penetrate the water surface, or it may be due to changes in the shape and extent of Pole Sand between the two surveys. Bathymetric survey has therefore been used in preference to LiDAR in overlapping areas to ensure the use of a consistent data set from the same date, and to avoid having to filter the water surface from the LiDAR data. Within the estuary cross-sections across the boundary of the two datasets show no steps in gradient, or level at the boundary within the estuary, and use of the LiDAR data for the intertidal areas is considered appropriate. The extent of LiDAR and bathymetric data is greater than that used in the Exe Estuary Flood and Coastal Risk Management Strategy modelling (Halcrow, 2010) and increases the accuracy of the model bathymetry over the intertidal areas. Where there is no bathymetric survey or LiDAR available in the estuary the low water channel has been taken as -1.83 mAOD based on Admiralty Charts for the Exe, this is consistent with bed levels at the upstream extent of the 2005 bathymetric survey which are between -1.5 and -2.25 mAOD.

2.1.2 C-MAP

In offshore areas where no high resolution LiDAR or bathymetric survey is available, data was extracted from the DHI C-MAP electronic chart database and converted to ODN.

The model mesh and bathymetry can be seen in Figure 2-3.

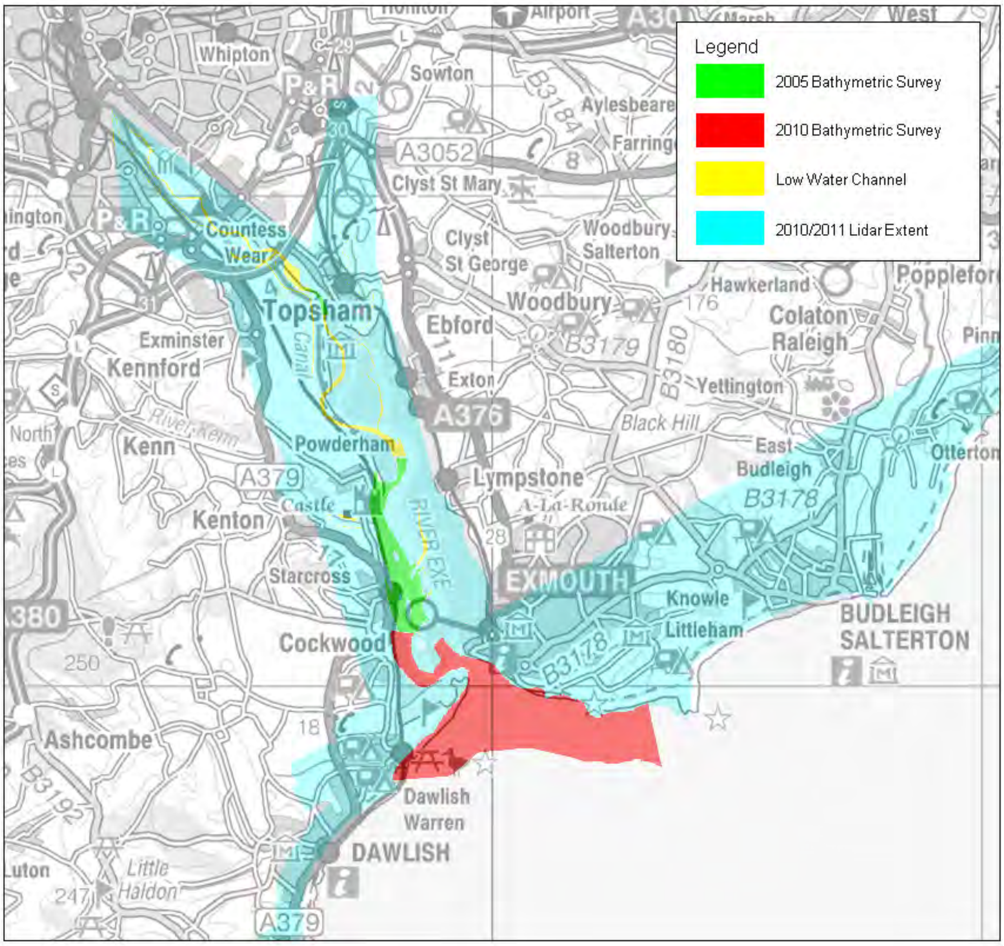


Figure 2-1: Extent of high resolution LiDAR and bathymetric survey used in model construction. Admiralty Chart data from C-MAP has been used in the white areas.

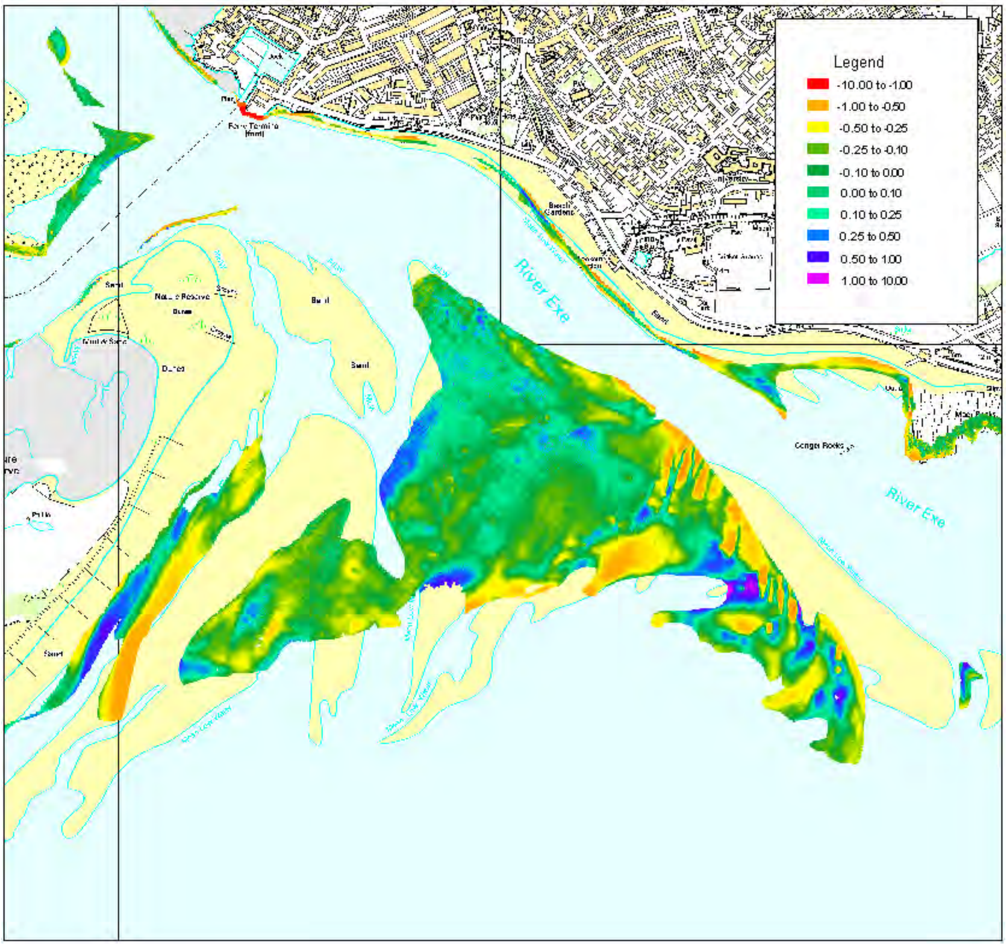


Figure 2-2: 2010 Bathymetric survey minus 2010/2011 LiDAR. There is no overlap between the data sets in the un-shaded areas.

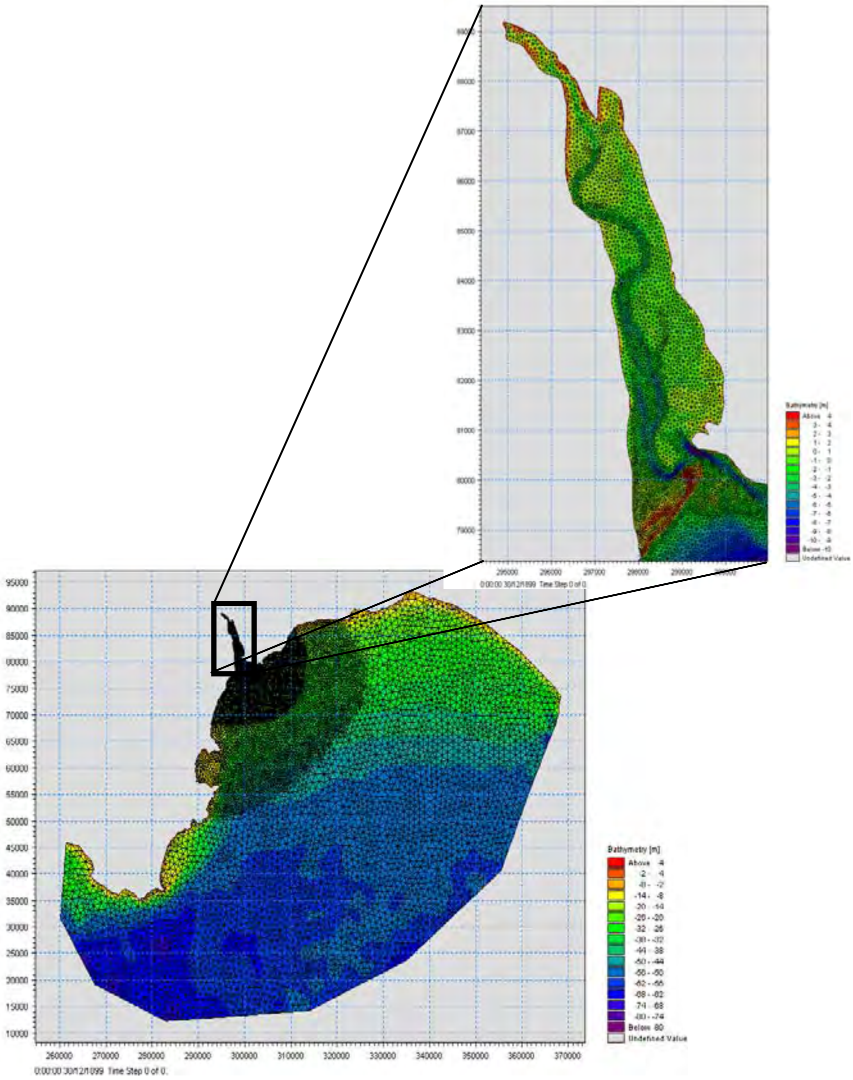


Figure 2-3: Regional Wave model mesh and bathymetry with a close up view of the Exe Estuary.

2.2 Wind and Wave Data

2.2.1 Nearshore Measured Wave Data

Measured wave data is available for two points outside the entrance to the Exe Estuary, the location of these is shown in Figure 2-4. Station 5 is located at Easting 471832.44m and Northing 5605296.95m with the water depth of 4.89m below ODN, while Station 6 is located at Easting 473027.06m and Northing 5603635.06m with the water depth of 16.18m below ODN. Two complete months of data are available for both stations from February 2008 to March 2008. This data is suitable for model calibration but is too short to be used for analysing the wave climate. Figure 2-5 and Figure 2-6 present the time-series of measured wave data.

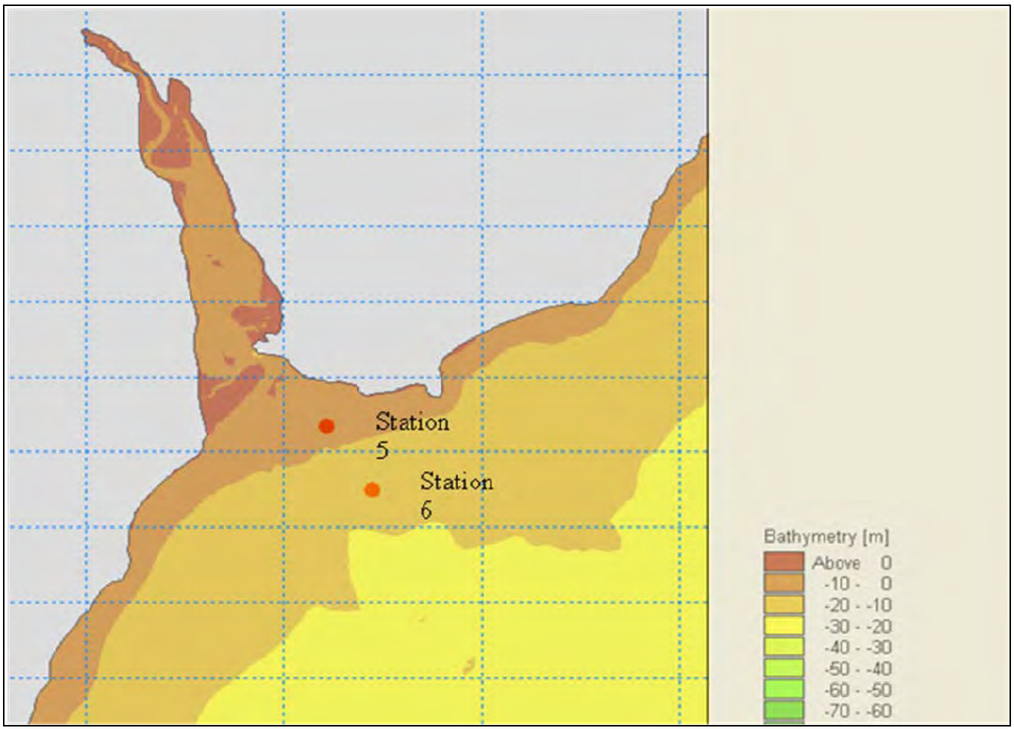


Figure 2-4: Measured wave data locations for wave model calibration.

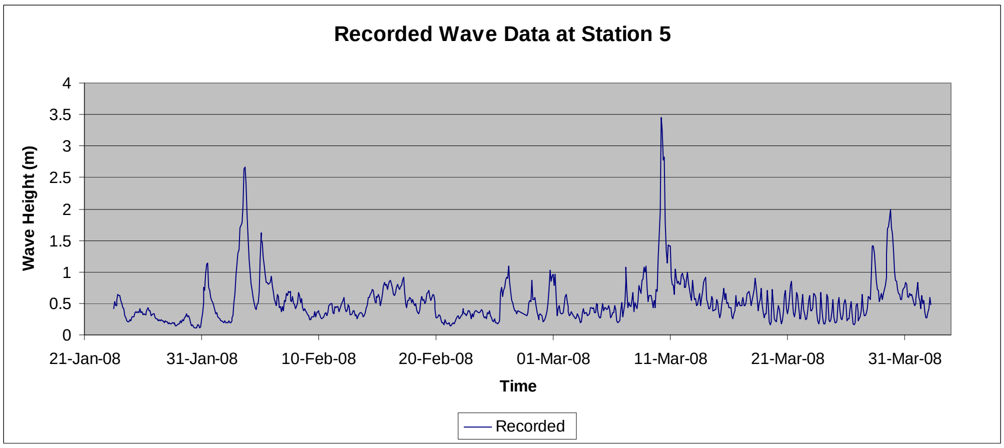


Figure 2-5: Measured wave data locations for wave model calibration.

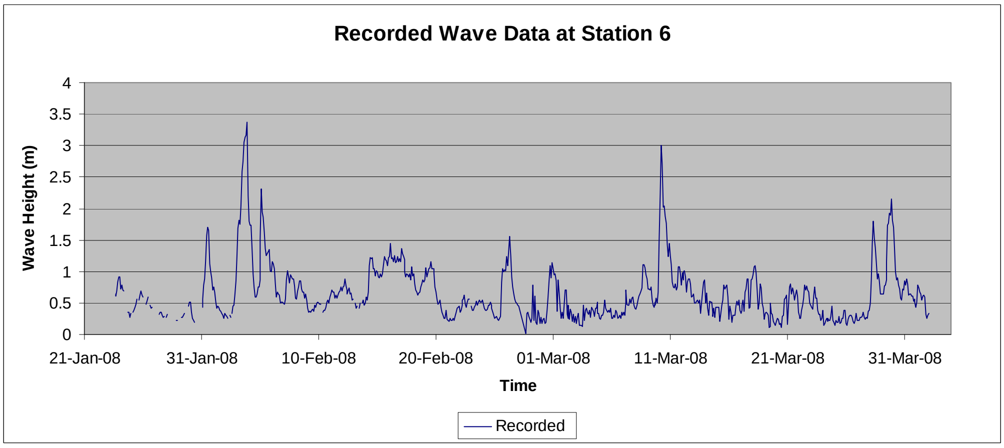


Figure 2-6: Measured wave data locations for wave model calibration.

2.2.2 Modelled Offshore Wind and Wave Data

Wind and wave data from the Met Office European Wave Model are available for 22 years, from 15/10/1986 to 25/11/2008, at 50° N, 3.66° W offshore from Lyme Bay. The wind rose at this location is shown in Figure 2-7, while the corresponding wave rose is shown in Figure 2-8.

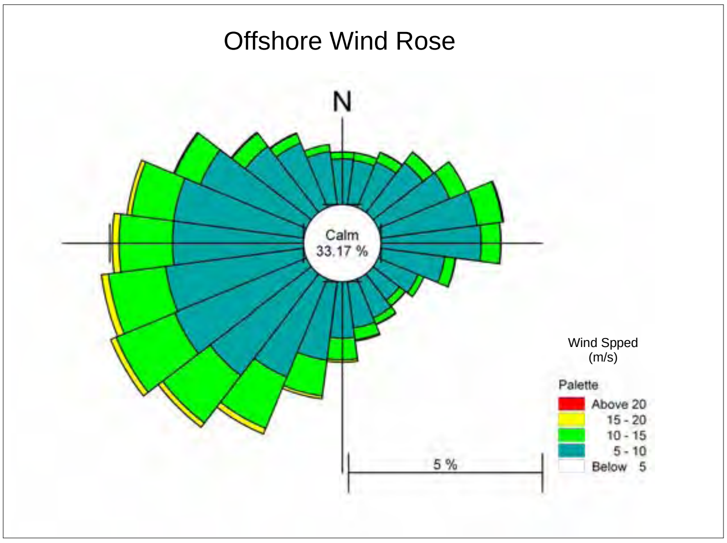


Figure 2-7: Offshore wind rose at 50° N, 3.66° W from the Met Office European Wave Model.

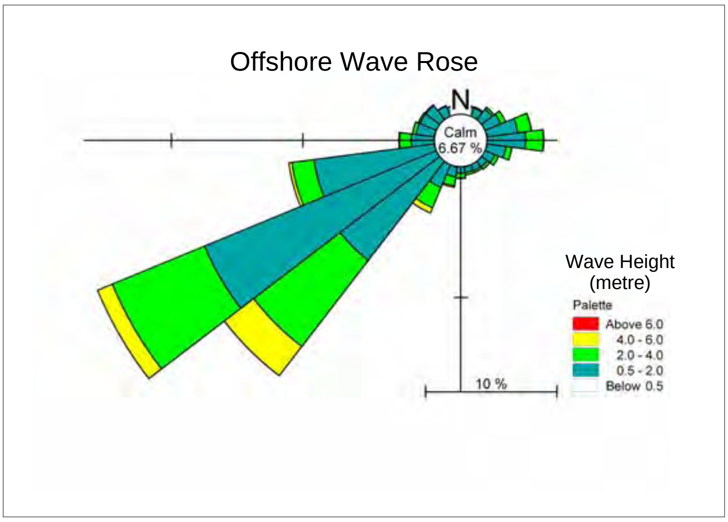


Figure 2-8: Offshore wave rose at 50° N, 3.66° W from the Met Office European Wave Model.

The Met Office wind and wave data have been analysed and the relationship between the wave height Hm0 and the wind speed can be seen in Figure 2-9.

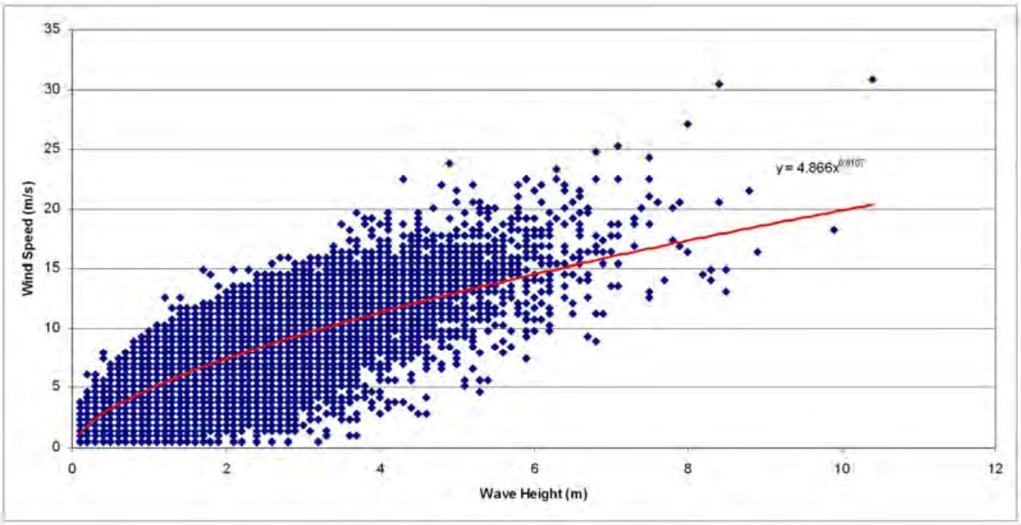


Figure 2-9: Relationship between the wave height and the wind speed based on the Met Office data from 1986 to 2008.

The extreme wind and wave conditions at this offshore point 50° N, 3.66° W were analysed as part of the Exe Estuary Flood and Coastal Risk Management Strategy Study (Halcrow, 2010) and are shown in Table 2-1 below for all directions.

Table 2-1: Tabulated extreme significant wave heights (Hm0), wind speeds and peak wave periods (Tp) for each sector.

Extreme waves, Directions 0~360 (All Directions)			
Return Period (years)	Hm0(m)	Wind Speed (m/s)	Period Tp (s)
1	7.26	24.25	11.94
5	9.10	27.14	13.36
10	9.90	28.32	13.94
25	10.99	29.83	14.69
50	11.82	30.94	15.23
100	12.65	32.01	15.76
200	13.49	33.06	16.27

2.3 Water Level Data

2.3.1 Water level data

2.3.1.1 Tide Gauge Information

Five tide gauges are available around Lyme Bay. The data availability for these stations is given in Table 2-2, and the locations of the gauges are shown in Figure 2-10 and Figure 2-11.

Table 2-2: Available water level data.

Gauge	Easting (OSGB m)	Northing (OSGB m)	Information Source	Data Availability
Weymouth	368400	78850	BODC Class 'A' Tide gauge	1991-2011
Exmouth	299308	80598	Environment Agency	1/03/2007-1/03/2008
Exmouth Dock	299787	80684	Environment Agency	10/11/2000-31/12/2009
Teignmouth Pier	294584	72927	Channel Coastal Observatory (South West Strategic Regional Coastal Monitoring Programme)	04/07/2008-30/06/2009 26/09/2009-31/12/2009
West Bay Harbour	346225	90300		1/02/2008-31/12/2009

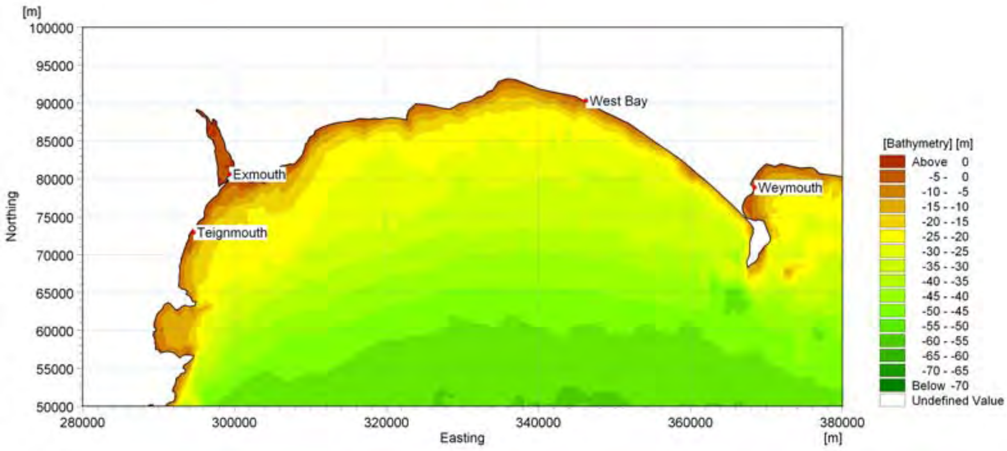


Figure 2-10: Location of observed water levels around Lyme Bay.

2.3.1.2 C-MAP

Tidal predictions from various locations within the model area not covered by tide gauges were extracted from the C-MAP database. Water levels were extracted with a time step of 30 minutes. The location of the C-MAP stations is shown in Figure 2-11.

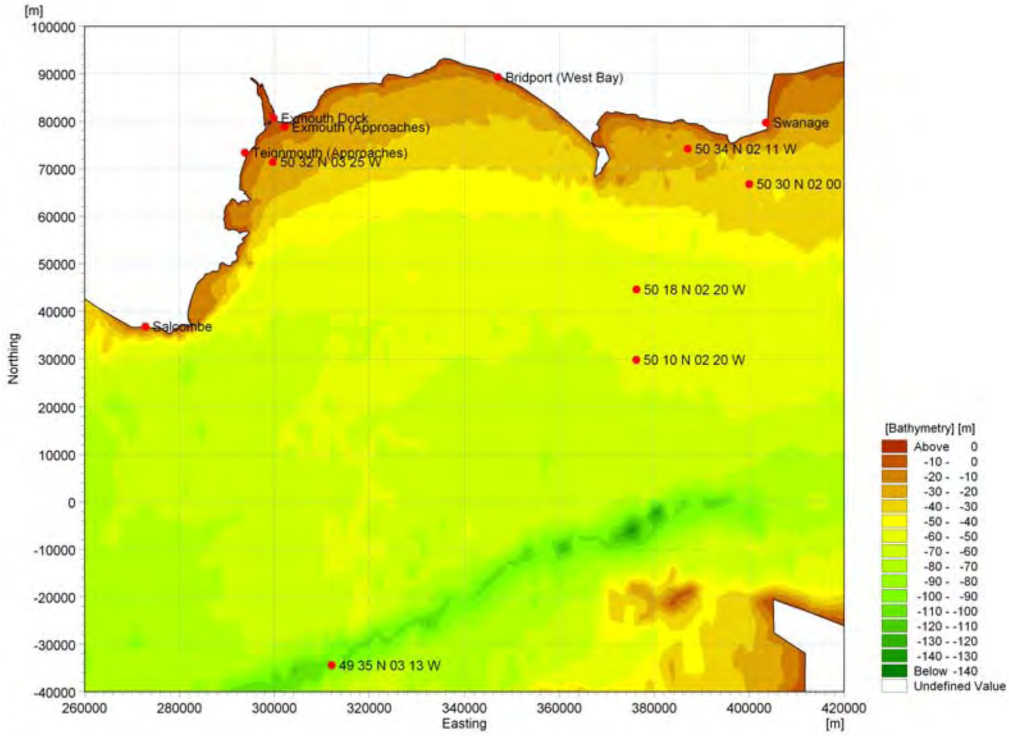


Figure 2-11: Location of C-Map stations used in model calibration.

2.4 Current Data

2.4.1.1 ADCP data

Acoustic Doppler Current Profilers (ADCPs) were deployed around the Exe Estuary in late 2007/early 2008. Locations of the ADCP stations are given in Figure 2-12 and Table 2-3. ADCP Stations 5 and 6 at the mouth of the estuary were considered most suitable for calibrating the regional hydrodynamic model, as the model resolution within the estuary in the regional hydrodynamic model is not sufficient to correctly resolve flows at Stations 2 and 4.

During the neap tides from 31/01/2008 to 02/02/2008 there is no clear tidal signal in the measured speeds at either ADCP Station 5 or ADCP Station 6. There is a surge event during this time, shown by the difference between the predicted astronomical and observed tides at Weymouth (Figure 2-13). Measured speeds during this time are very high, and it is possible that the amount of sediment in the water at ADCP Stations 5 and 6 was also higher than normal during this period affecting readings from the ADCPs, or that strong wave driven currents may have been setup in this region. The aim of the regional model is to provide tidal boundary conditions for the coastal area model, and so this data period which appears to be dominated by meteorological events has not been analysed. ADCP readings at Station 5 appear noisy over the entire observation interval. ADCP Station 5 is located at the edge of the ebb delta, and higher quantities of suspended sediment or wave driven currents at this location may also affect the measurements at this location during normal tidal conditions.

There is no information available regarding reference datums for the ADCP measurements. Depth measurements at the ADCP sites were therefore converted to water levels by shifting measured depth time-series vertically so that the root mean squared (RMS) error between the modelled water level and the measured depth was minimised.

Table 2-3: Available current data. Note stations 1 and 3 did not yield any data due to sediment within the estuary.

ADCP Station	Easting (OSGB m)	Northing (OSGB m)	Observation Interval
2	296509	85741	15/12/2007-20/01/2008
4	299602	80475	15/12/2007-16/12/2007
5	301100	78600	23/01/2008-02/04/2008
6	302300	76900	23/01/2008-27/02/2008

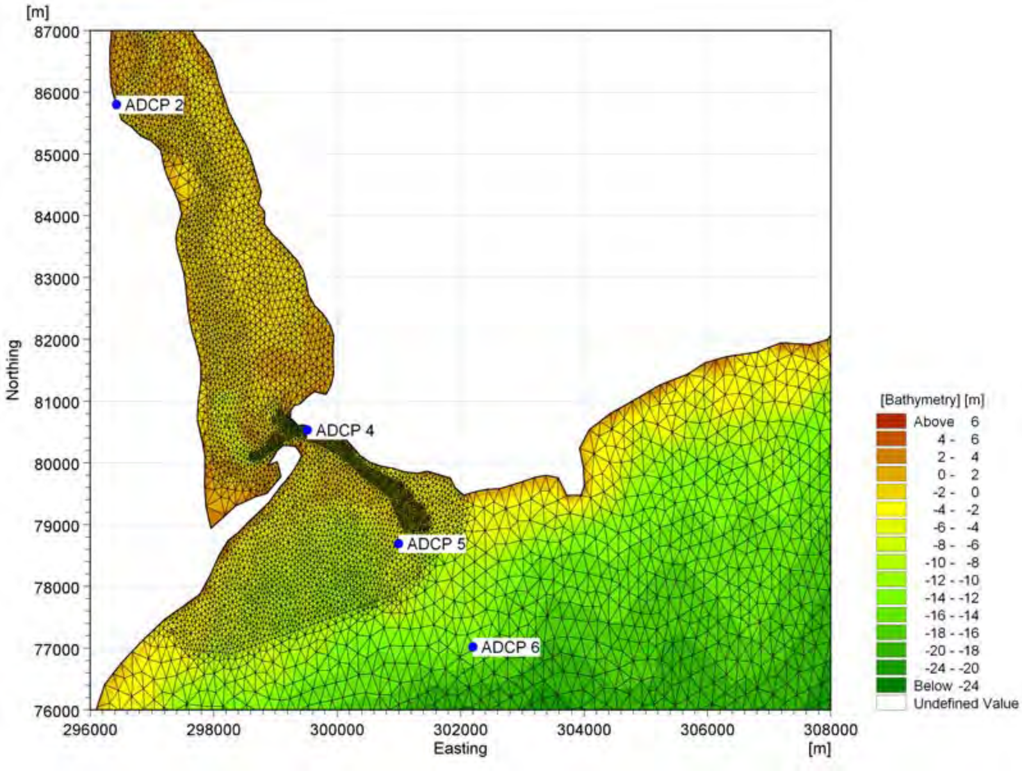


Figure 2-12: Location of ADCP measurement stations, also shown is the higher resolution mesh around the study area in the regional hydrodynamic model.

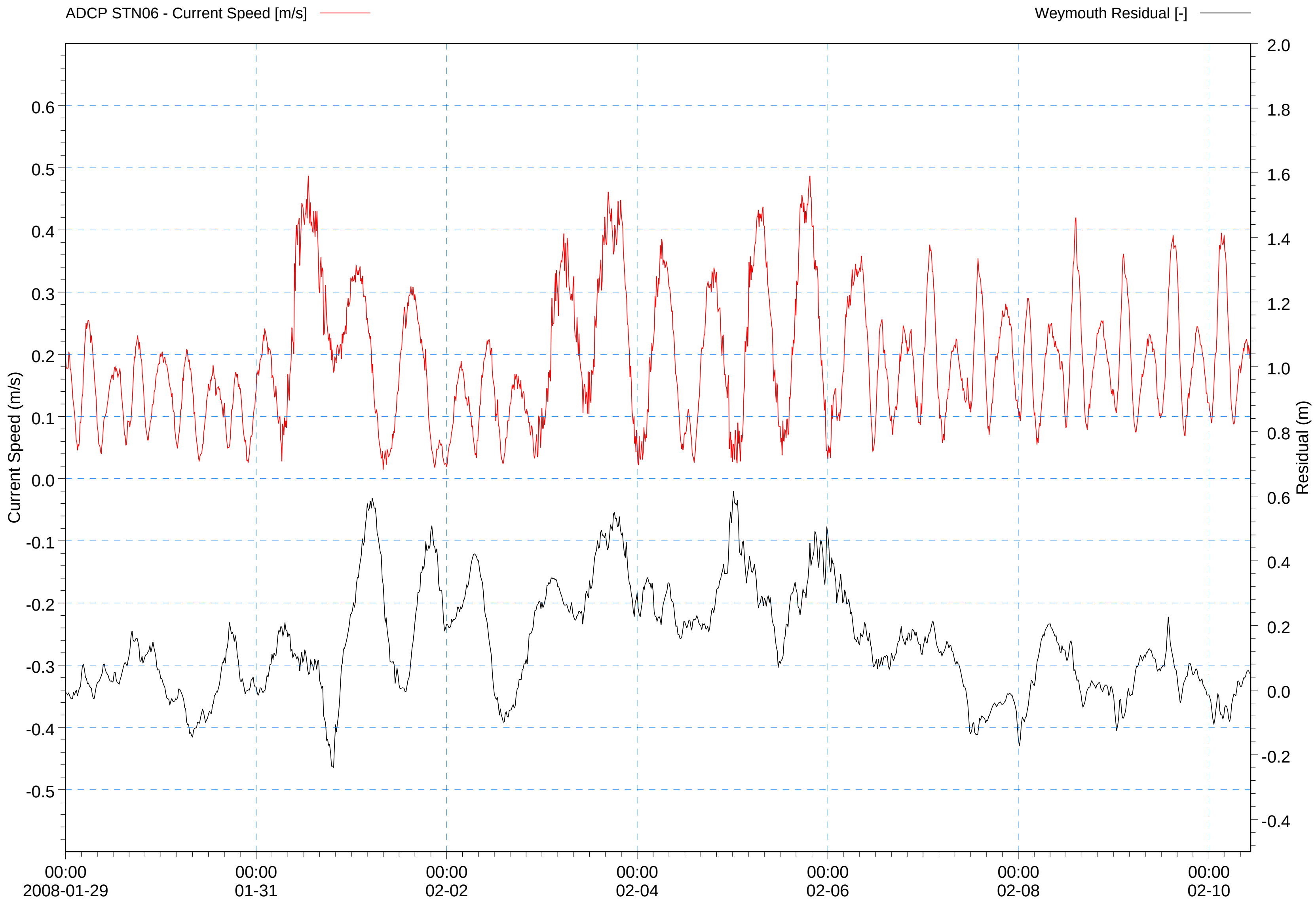


Figure 2-13: Current Speed at ADCP Station 6 and observed minus predicted (residual) tide at Weymouth.

2.5 Beach Profile and Sediment data

2.5.1 Beach Profile Data

Beach profile data from a number of sources was collated as part of the Exe Estuary Coastal Management Study (Halcrow, 2008). Additional profiles were downloaded from the Channel Coastal Observatory website on Friday 4th June 2010. Data is available from two sources:

- LiDAR profiles for May 1998 and December 2005; and
- Ground based survey from the South-West Regional Coastal Monitoring Programme (SWRCMP from Spring 2007 up to (in some places but not all) Spring 2010.

The coastal process units and the beach profile locations for Dawlish Warren and Exmouth are shown in Figure 2-14 and Figure 2-15 respectively.

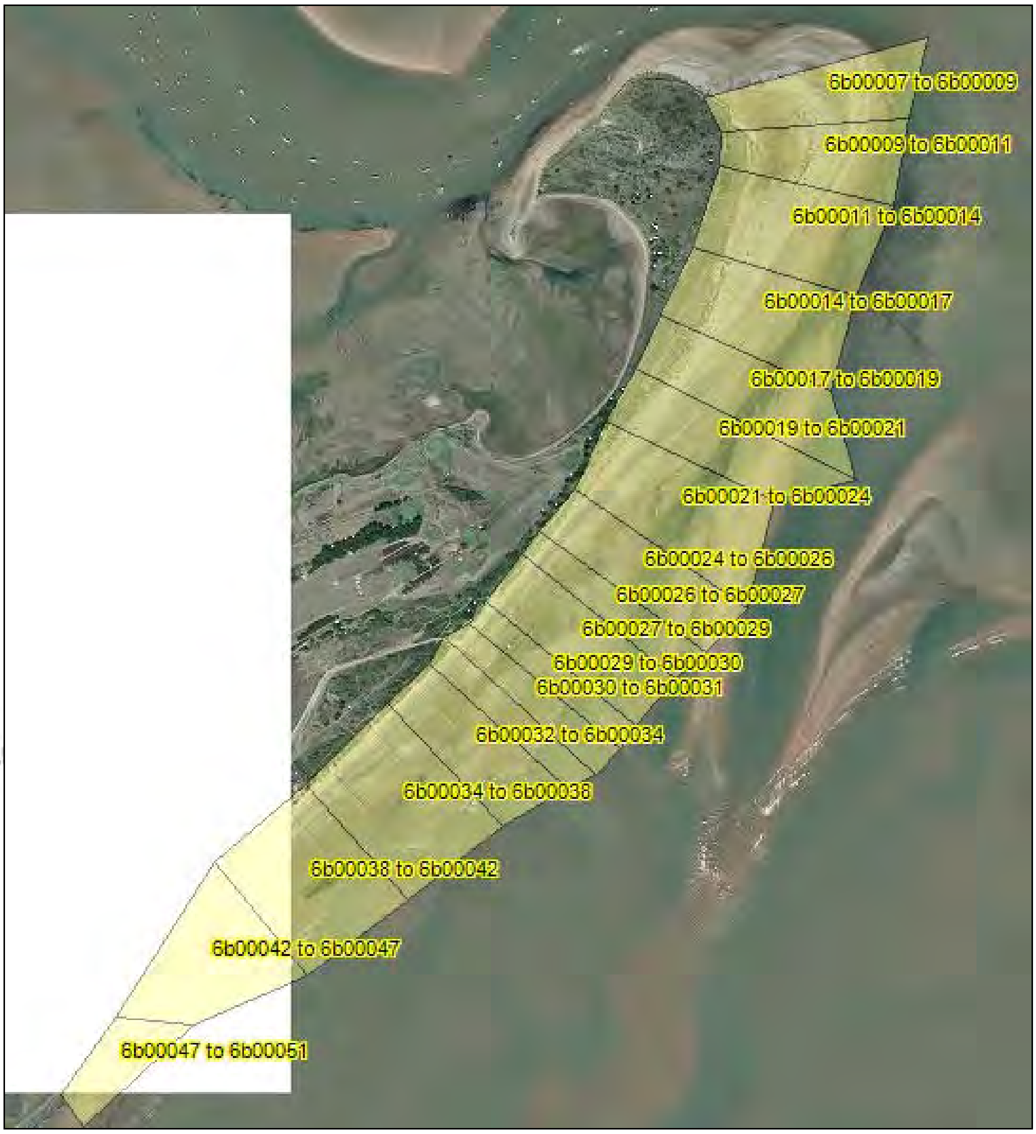


Figure 2-14: Dawlish Warren Coastal Process Units and Beach Profile Locations.

Figure 2-15: Exmouth Coastal Process Units and Beach Profile Locations.

2.5.2 Beach Sediment Data

The measured sediment sizes are available for the different beach profiles of the Dawlish Warren and Exmouth. Table 2-4 shows the sediment sizes at different locations of the beach profiles of Dawlish Warren (see Figure 2-14), while Table 2-5 shows the sediment sizes at different locations of the beach profiles of Exmouth (see Figure 2-15).

Table 2-4: Sediment sizes along Dawlish Warren frontage (See Figure 2-14 for locations).

Location	Nearest sample location	D50 (mm)
6b00007	DW10	4.1
6b00009	average of DW 9 and DW10	1.15
6b00011	DW9	0.9
6b00014	average of DW 9 and DWA	2.4
6b00017	DWA	3.9
6b00019	DW8	3
6b00021	average of DW 7 and DW8	4.4
6b00024	DW7	5.8
6b00026	average of DW 6 and DW7	4.4
6b00030	average of DW 5 and DW6	2.3
6b00034	average of DW 3 and DW 5	3.3
6b00038	average of DW 2 and DW 3	5.5
6b00042	average of DW 2 and DW 1	5.6
6b00047	DW1	5.2
6b00051	DW1	5.2

Table 2-5: Sediment sizes along Exmouth frontage (See Figure 2-15 for locations).

Location	Nearest sample location	D50 (mm)
6a01767	EB10	0.54
6a01772	EB9	0.54
6a01776	EB9	0.54
6a01780	EB8	0.34
6a01784	EB8	0.34
6a01788	average of EB7 and EB8	0.53
6a01792	EB7	0.72
6a01796	average of EB 6 and EB7	0.55
6a01800	average of EB5 and EB6	0.365
6a01804	average of EB4 and EB5	0.365
6a01808	average of EB 3 and EB4	3.25
6a01812	EB2	3.9
6a01816	EB1	0.42
6a01820	EB1	0.42

2.5.3 Offshore Sediment Data

Sediment sampling was carried out at Pole Sand as part of this study. A large variation in the sediment size and grading over Pole Sand can be seen in Figure 2-16, with larger sediment sizes observed adjacent to the deep water channels at the mouth of the estuary where there are high velocities.

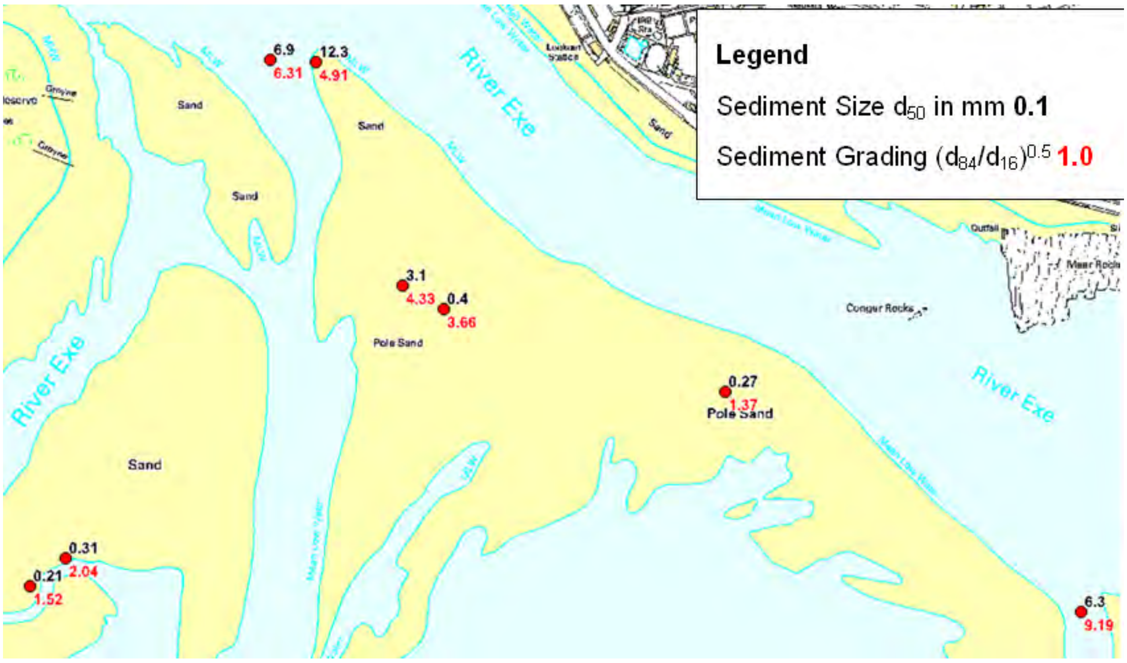


Figure 2-16: Sediment size and grading over Pole Sand.

3 Wave Modelling

3.1 Introduction

In order to investigate the potential evolution of the coastline over decadal timescales, several years of wave and wind data is required at near shore locations along the Dawlish and Exmouth coasts. Several years of data are also required in order to carry out a joint probability analysis for wave heights and water levels in order to investigate the beach profile response during extreme events. Nearshore observations are only available for a period of two months so it is necessary to use offshore data from the Met Office European wave model and transform this data inshore using a calibrated regional wave model. This section provides details of the wave model set-up, calibration and transformation of offshore wave data to inshore locations.

3.2 Wave Model

The wave model used for this project is the MIKE 21-SW spectral wind-wave model developed by DHI. This model works on a flexible mesh using triangular elements, which allows a variable resolution to be prescribed. This model is a third generation spectral wave model capable of simulating the transformation of waves as they propagate from offshore to nearshore waters. The model includes wave processes such as refraction, shoaling, energy dissipation due to bottom friction and wave breaking, local wind-wave generation and wave reflection from structures that are important for this application.

An existing wave model of Lyme Bay was set up as part of the Exe Estuary Coastal Management Study (Halcrow, 2008) and the calibration was further improved for the Dawlish to Teignmouth Seawall Study (Halcrow, 2009). Further improvements to the wave model were made for the Exe Estuary Flood and Coastal Risk Management Strategy (Halcrow, 2010). For this study the wave model has been updated with the latest bathymetric and LIDAR data. The model mesh with the latest bathymetry is shown Figure 3-1.

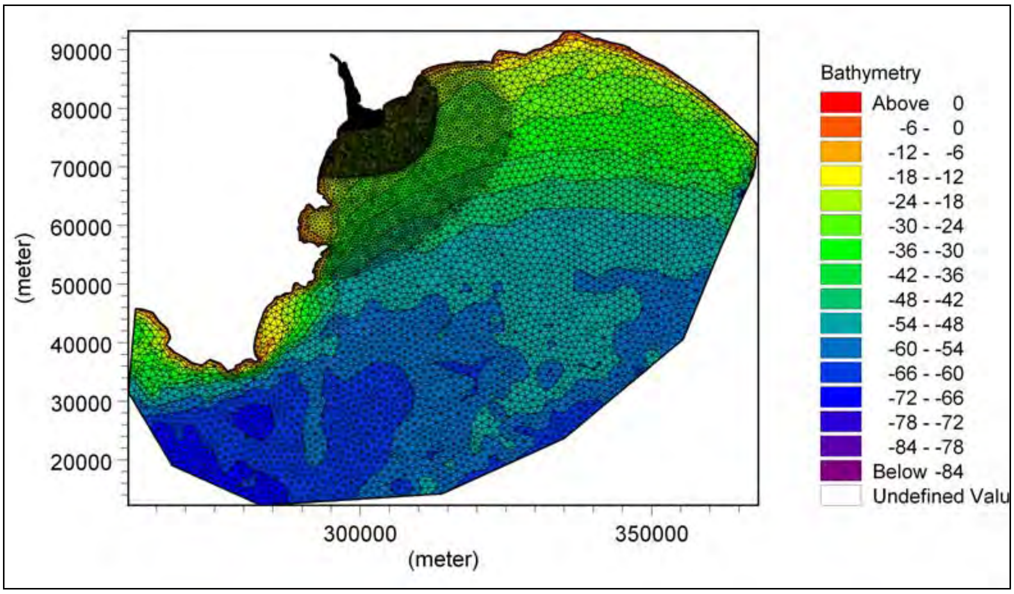


Figure 3-1: Regional wave model bathymetry.

3.3 Model Verification

Model verification was carried out using the same boundary conditions as used for the Exe Estuary Flood Risk Management Strategy (Halcrow, 2010). Waves were prescribed along the entire length of the offshore boundary and winds were assumed to be spatially constant. There are two calibration points outside of the entrance to the Exe Estuary at which significant wave height data was available, the location of these can be seen in Figure 2-4 above. The model has been run for the same period as the available data for the purposes of comparison. Model Station 6 is further offshore than model Station 5 and therefore larger waves are expected at this location.

Figure 3-2 and Figure 3-3 shows the measured data and the model results for a time period of January 2008 to March 2008. ADCP Station 6 (16.18m below ODN) is further offshore and located in deeper water than ADCP Station 5 (4.89m below ODN). From wave refraction theory smaller wave heights would be expected at ADCP Station 5 than at ADCP Station 6.

In order to quantify the level of agreement between the measured and simulated wave heights at these two stations the root mean square (RMS) error for the significant wave height was calculated from two datasets. For Station 6 the RMS error for wave height is 0.29m and for Station 5 (further inshore) the RMS error for wave height is 0.25m. There is no significant difference in wave heights at Stations 5 and 6 due to the addition of the new bathymetry, and so model parameters have not been changed from those used in calibrated Exe Estuary Flood and Coastal Risk Management Strategy model (Halcrow, 2010). The level of agreement is reasonable and the model is suitable for transforming offshore wave data inshore.

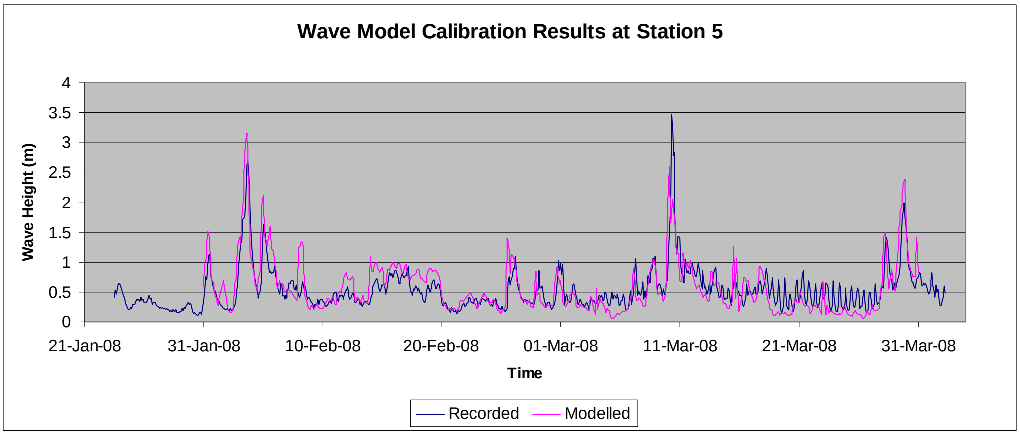


Figure 3-2: Comparison of regional wave model results with measured wave data at Station 5.

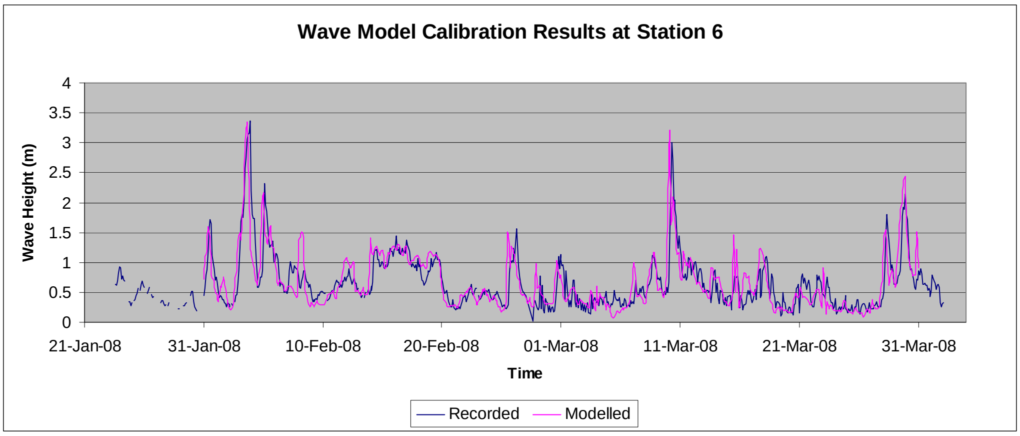


Figure 3-3: Comparison of regional wave model results with measured wave data at Station 6.

3.4 Transformation of Offshore Wave Data Inshore

It is too computationally expensive to run the regional wave model for the entire 22 years of data from the Met Office European Wave model, so the calibrated regional wave model has been used to generate a look up table to transform the offshore data inshore for 23 inshore locations shown in Figure 3-4. The bins of significant wave height, period, mean wave direction and still water level used in the look up table are shown in Table 3-1. Resulting inshore wave roses are shown in Figure 3-5a to Figure 3-5e for the Dawlish Warren coast and in Figure 3-6a to Figure 3-6g for the Exmouth coast.

Table 3-1: Bins of significant wave height, period and mean wave direction used for the onshore wave transformation. Tide levels are for Exmouth Approaches from the Admiralty Tide Tables.

Wave Parameter	Min	Max	Interval
Hm0(m)	0	6	1
Tp(s)	0	12	2
MWD(°)	0	360	20
Water Level Parameter	**MLWS**	**MHWS**	**MSL**
Water Level(m)	-1.94OD	2.16OD	0.07OD

Figure 3-4: Location of the 23 inshore wave data locations along Dawlish and Exmouth coasts.

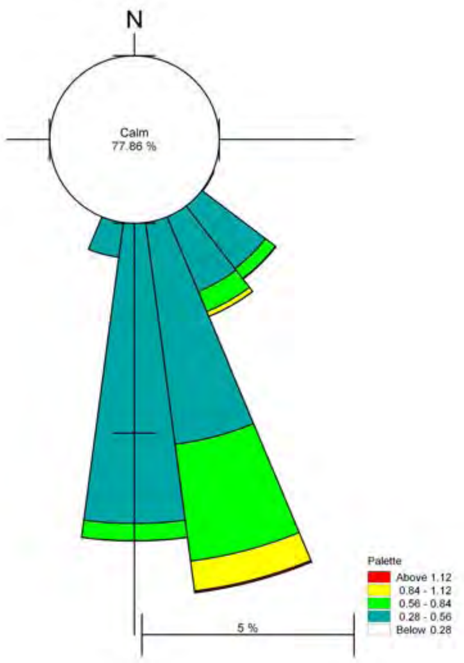


Figure 3-5a: Nearshore wave rose for the Dawlish Warren coast, Point 12.

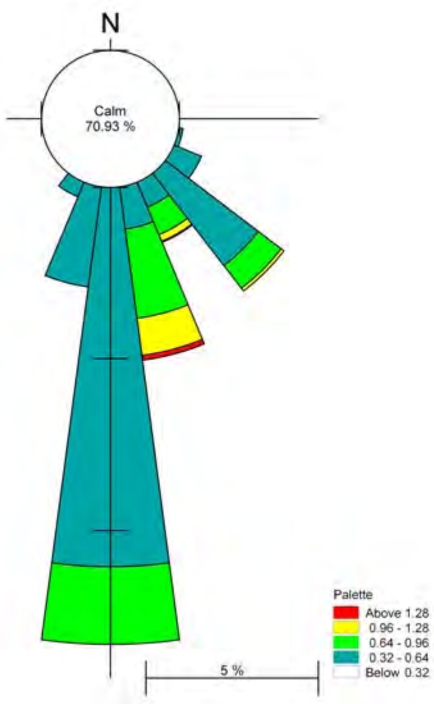


Figure 3-5b: Nearshore wave rose for the Dawlish Warren coast, Point 13.

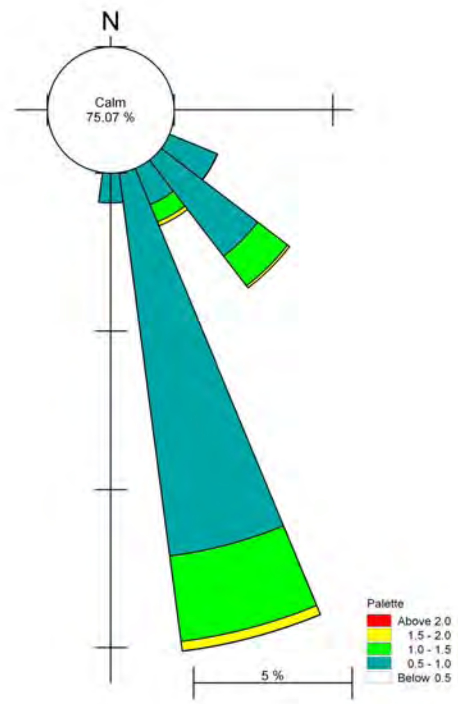


Figure 3-5c: Nearshore wave rose for the Dawlish Warren coast, Point 14.

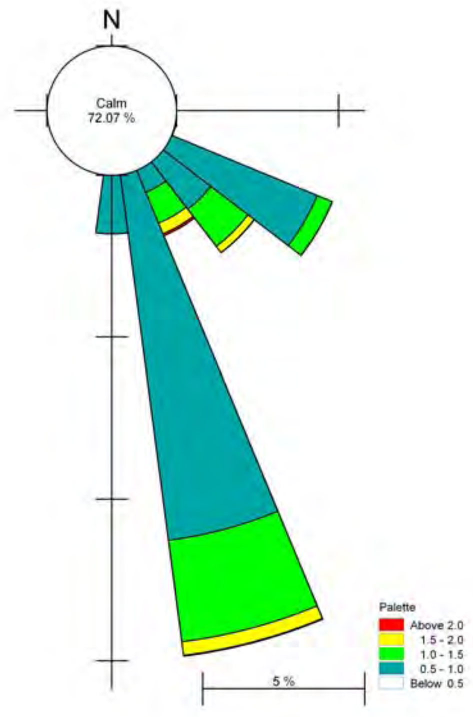


Figure 3-5d: Nearshore wave rose for the Dawlish Warren coast, Point 15.

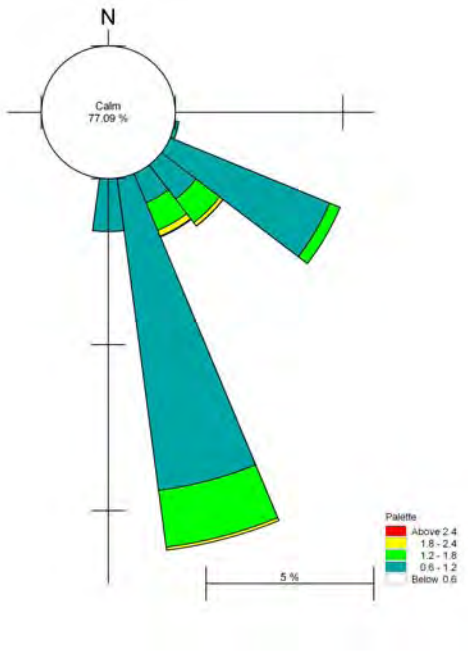


Figure 3-5e: Nearshore wave rose for the Dawlish Warren coast, Point 16.

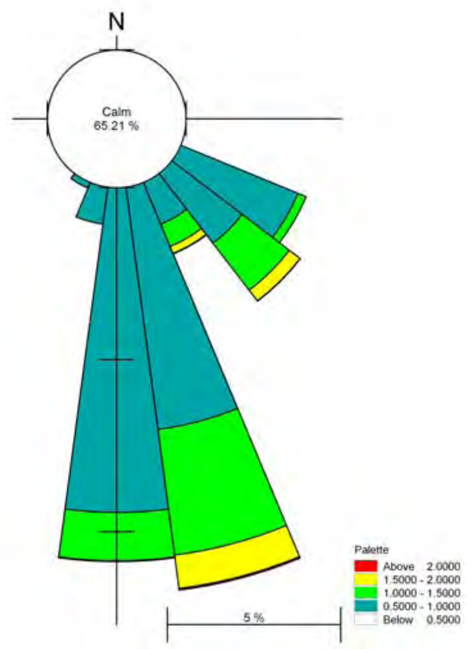


Figure 3-6a: Nearshore wave rose for the Exmouth coast, Point 5.

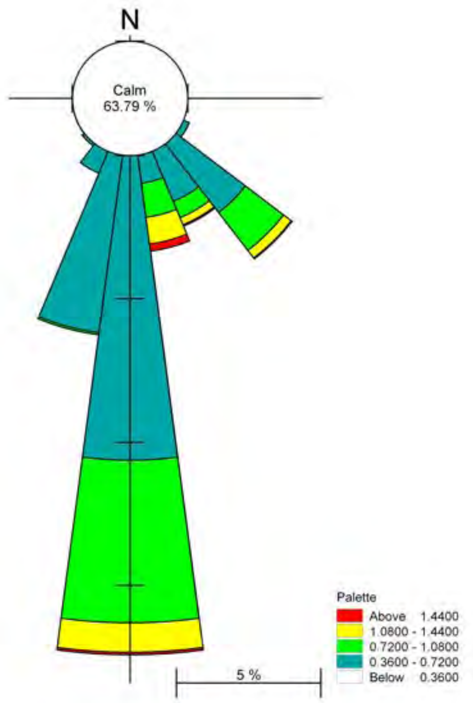


Figure 3-6b: Nearshore wave rose for the Exmouth coast, Point 6.

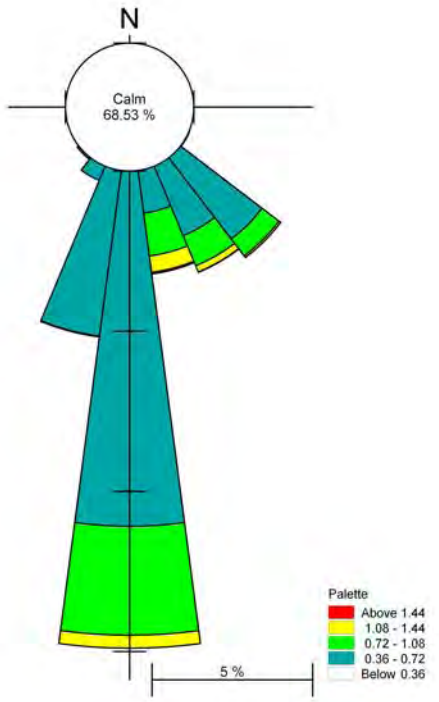


Figure 3-6c: Nearshore wave rose for the Exmouth coast, Point 7.

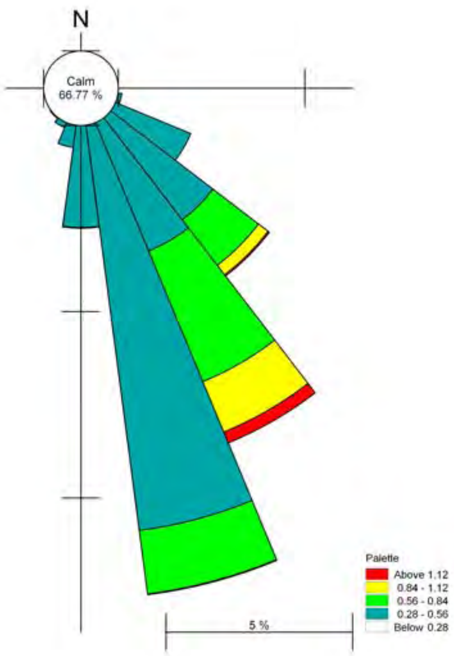


Figure 3-6d: Nearshore wave rose for the Exmouth coast, Point 8.

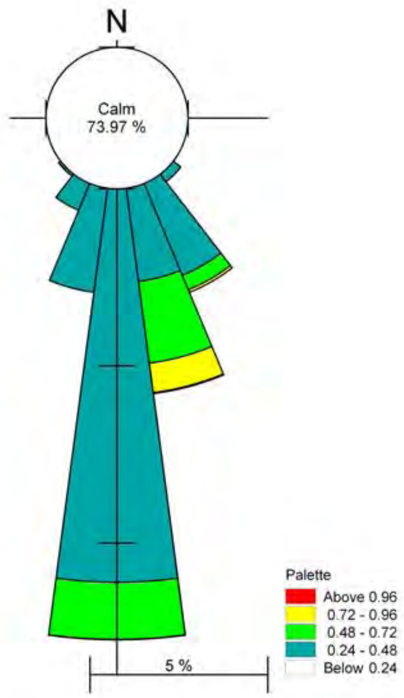


Figure 3-6e: Nearshore wave rose for the Exmouth coast, Point 9.

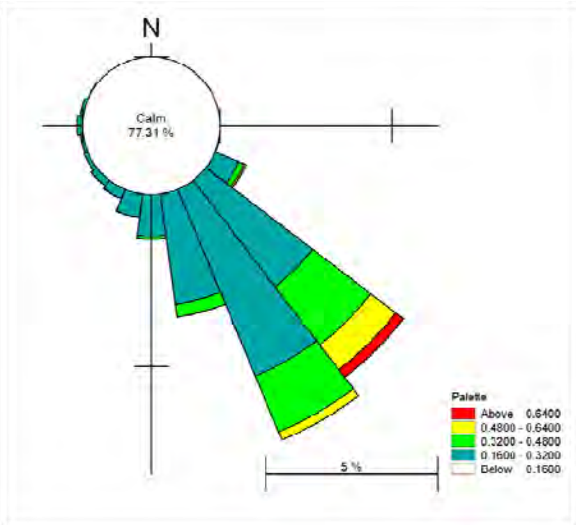


Figure 3-6f: Nearshore wave rose for the Exmouth coast, Point 10.

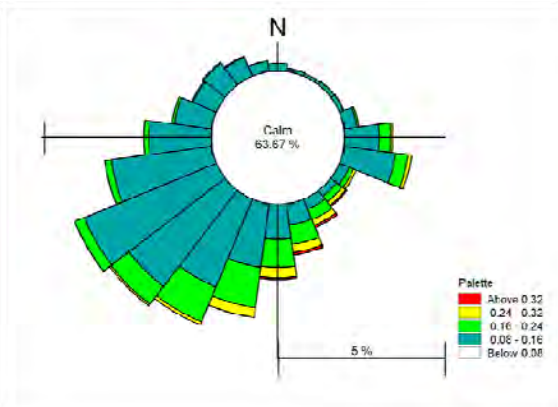


Figure 3-6g: Nearshore wave rose for the Exmouth coast, Point 11.

4 Extreme Value Analysis

Extreme value analysis (EVA) has been carried out for inshore points using the transformed wave data and water level data. A joint probability analysis between waves and water levels has been carried out using the JOIN-SEA software. In the JOIN-SEA model pairs of data are extracted from the time-series of waves and water levels at each nearshore location; for example, the highest wave occurring around the highest water level for each tide. The dependence/correlation coefficient is then derived for each location and a statistical model is fitted. This information is then used in a Monte-Carlo style simulation to create a very long time-series of the order of 10,000 years which is then used to calculate joint probability curves for extreme waves and water levels. It is this data that is used in the cross-shore modelling described in Section 6.4 and Section 6.5.

Before carrying out the joint probability analysis, a marginal extreme analysis for wave data and water levels is performed to generate the boundary conditions for the JOIN-SEA model.

4.1 Marginal Extreme Wave Conditions

The MIKE EVA tool has been used to investigate various extreme value distributions from which the best fit distribution has been used to determine the extreme significant wave height values for the required return periods. The predicted extreme wave heights for nearshore points 6, 7, 8, 9, 12, 13, 14 and 15 in Figure 3-4 have been generated using the transformed wave data described in Section 3.4. A summary of the generated extreme wave heights for the eight nearshore points is shown in Table 4-1.

For each of the extreme wave heights in Table 4-1, the associated wave period was determined using a wave steepness calculation. The square root of the peak wave height was plotted against the corresponding peak wave period and a least squares trend line was then fitted through the data. The relationship of the wave heights and wave periods is $T_P = 4.43 \times \sqrt{Hs}$. (Halcrow, 2010). The results are based on the existing data and the reliability is dependent on the length of the data records.

Table 4-1: Summary of extreme wave heights (m) for the eight nearshore points.

Return Period	Point 6	Point 7	Point 8	Point 9	Point 12	Point 13	Point 14	Point 15
0.2	1.55	1.29	1.18	0.88	1.03	1.28	1.63	1.70
0.3	1.59	1.35	1.21	0.90	1.06	1.31	1.68	1.82
0.5	1.63	1.40	1.24	0.92	1.09	1.35	1.74	1.93
1	1.67	1.45	1.26	0.94	1.12	1.39	1.81	2.06
2	1.70	1.50	1.29	0.96	1.15	1.43	1.88	2.18
3	1.72	1.53	1.30	0.97	1.17	1.45	1.92	2.25
5	1.75	1.56	1.32	0.99	1.18	1.48	1.96	2.33
10	1.78	1.60	1.34	1.01	1.21	1.51	2.02	2.44
20	1.81	1.64	1.36	1.02	1.23	1.54	2.08	2.54
30	1.82	1.66	1.37	1.03	1.24	1.56	2.12	2.60
50	1.84	1.69	1.38	1.05	1.25	1.59	2.16	2.68
100	1.87	1.72	1.40	1.06	1.27	1.62	2.22	2.78
200	1.89	1.76	1.42	1.08	1.29	1.64	2.27	2.87
300	1.91	1.78	1.43	1.09	1.30	1.66	2.31	2.93
500	1.93	1.80	1.44	1.10	1.31	1.68	2.35	3.00
1000	1.95	1.83	1.45	1.11	1.33	1.71	2.40	3.09
2000	1.97	1.86	1.47	1.13	1.35	1.74	2.46	3.18
3000	1.98	1.88	1.48	1.14	1.36	1.75	2.49	3.24
5000	2.00	1.90	1.49	1.15	1.37	1.77	2.53	3.30
10000	2.02	1.93	1.50	1.16	1.39	1.80	2.58	3.39

4.2 Extreme Water Level Conditions

Almost 10 years of continuous data are available for Exmouth Dock (location shown in Figure 2-101). The quality of the data has been checked rigorously and bad data have been removed. Extreme value distributions have been fitted to the data using Halcrow's in house extreme value fitting software MWAV_FIT results are shown in Table 4-2.

Table 4-2: Predicted extreme water levels at Exmouth.

Return Period (years)	Extreme Water Level (mODN)
1 (ARI)	2.98
2	3.09
3	3.16
5	3.24
10	3.35
20	3.47
30	3.54
50	3.63
100	3.75
200	3.87
300	3.95
500	4.04
1000	4.17

4.3 Joint Extreme Wave and Water Level Conditions

Joint probability of wave and water level conditions are required for assessment of standards of protection of the defences and also as input data for the cross-shore modelling of beach profiles under extreme conditions. In this project the joint probability is carried out using JOIN-SEA model. The joint probability model JOIN-SEA has been developed by HR Wallingford and represents a rigorous approach to joint probability (JP) determination. In order to carry out the JOIN-SEA analysis, a list of wave parameters and the corresponding water level records are required. Water levels are similar along the frontage of interest and wave heights vary along the frontage according to their direction of origin. For this project the measured water levels at the Exmouth Dock are used as input data and combined with the time-series of transformed inshore wave data. From this data, high water levels and the corresponding wave heights (or maximum wave height near each high water) are extracted and used in the JOIN-SEA analysis. The marginal extreme wave heights and water levels, presented in Section 4.1 and Section 4.2 respectively, are used as input data for the JOIN-SEA model.

The plots of data fitting curves from the joint probability analysis for inshore positions 6, 8, 12, and 14 in Figure 3-4 are shown in Figure 4-1 to Figure 4-4. The model outputs from the JOIN-SEA model for 1 year, 100 year and 500 year recurrence intervals for the eight inshore positions are shown in Table 4-3, Table 4-4 and Table 4-5. Those joint extremes are used as boundary conditions for the cross-shore modelling.

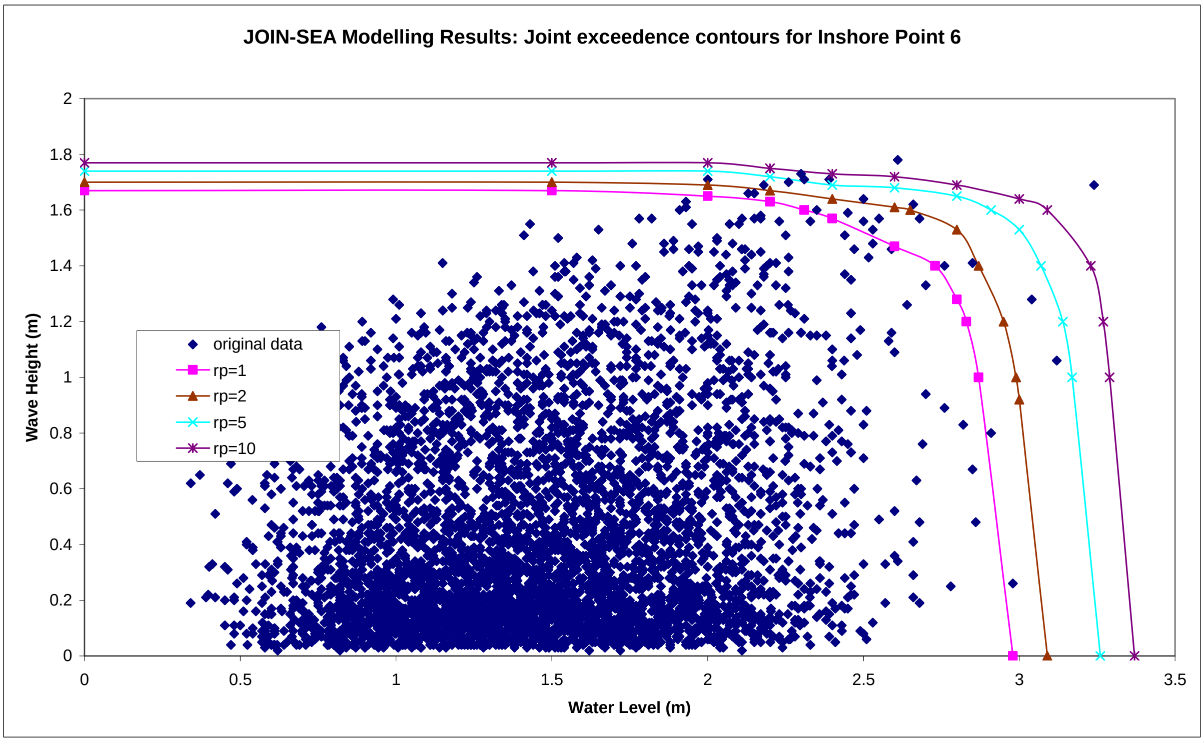


Figure 4-1: Joint probability results for inshore position 6 of Exmouth coast.

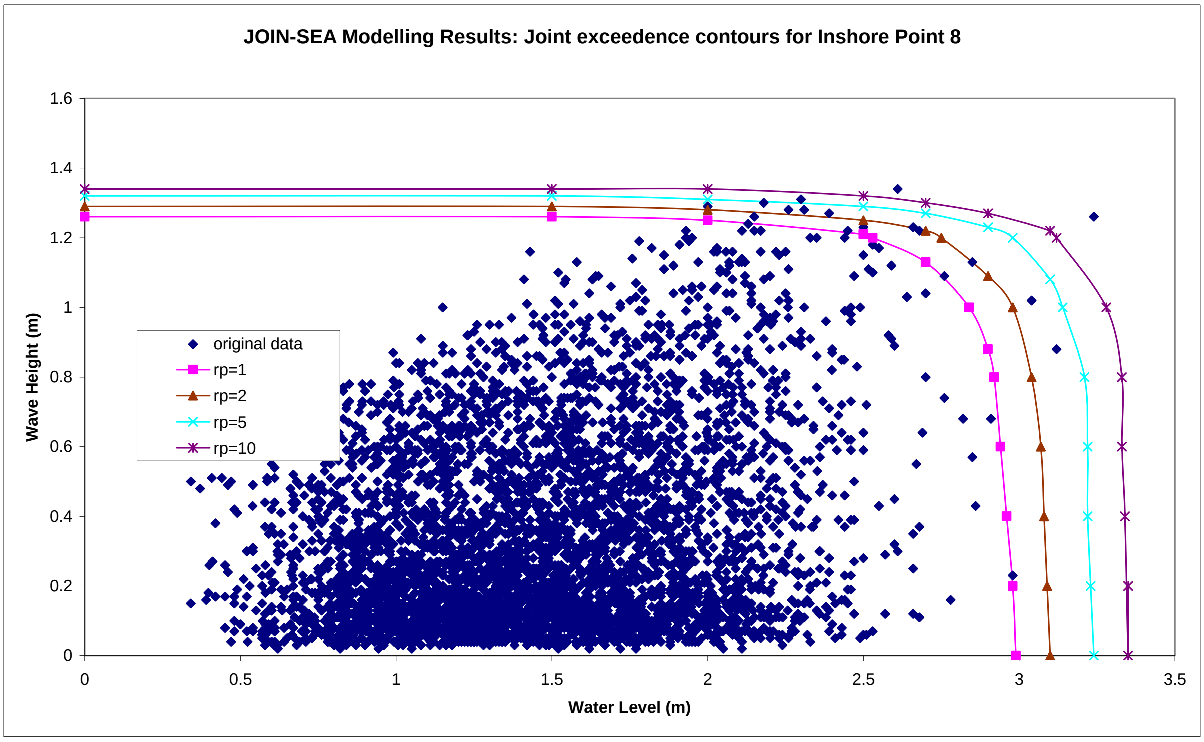


Figure 4-2: Joint probability results for inshore position 8 of Exmouth coast.

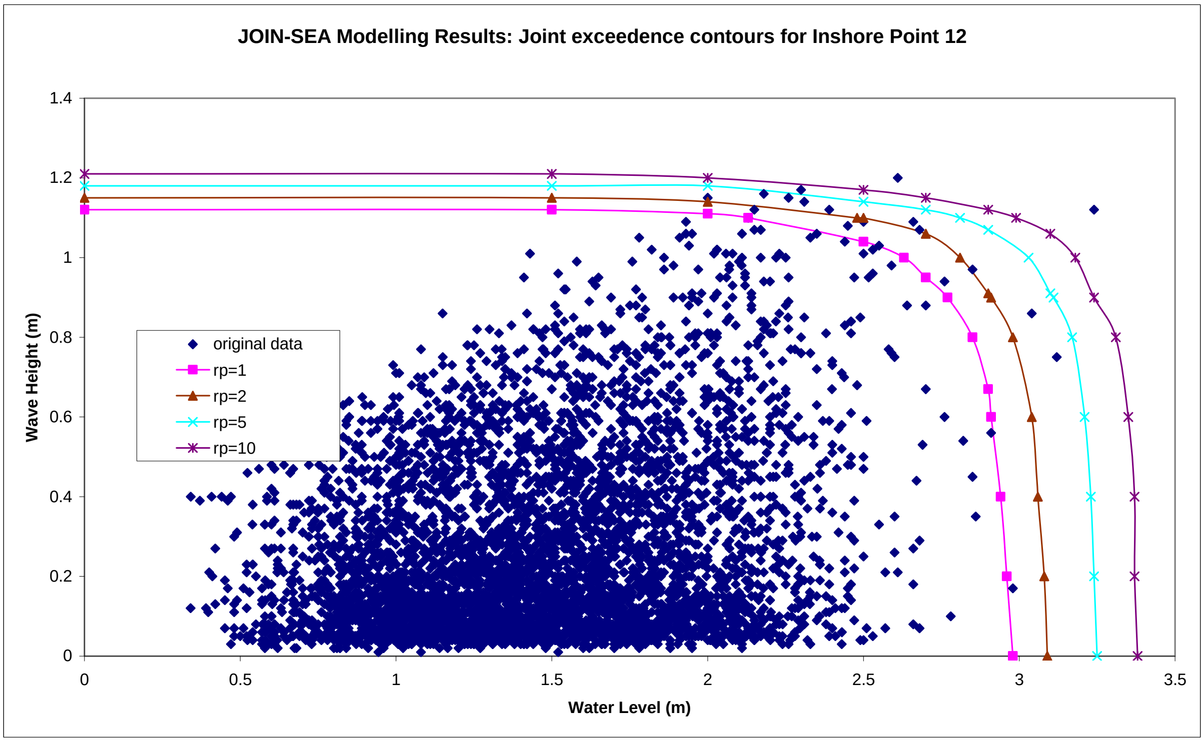


Figure 4-3: Joint probability results for inshore position 12 of Dawlish Warren coast.

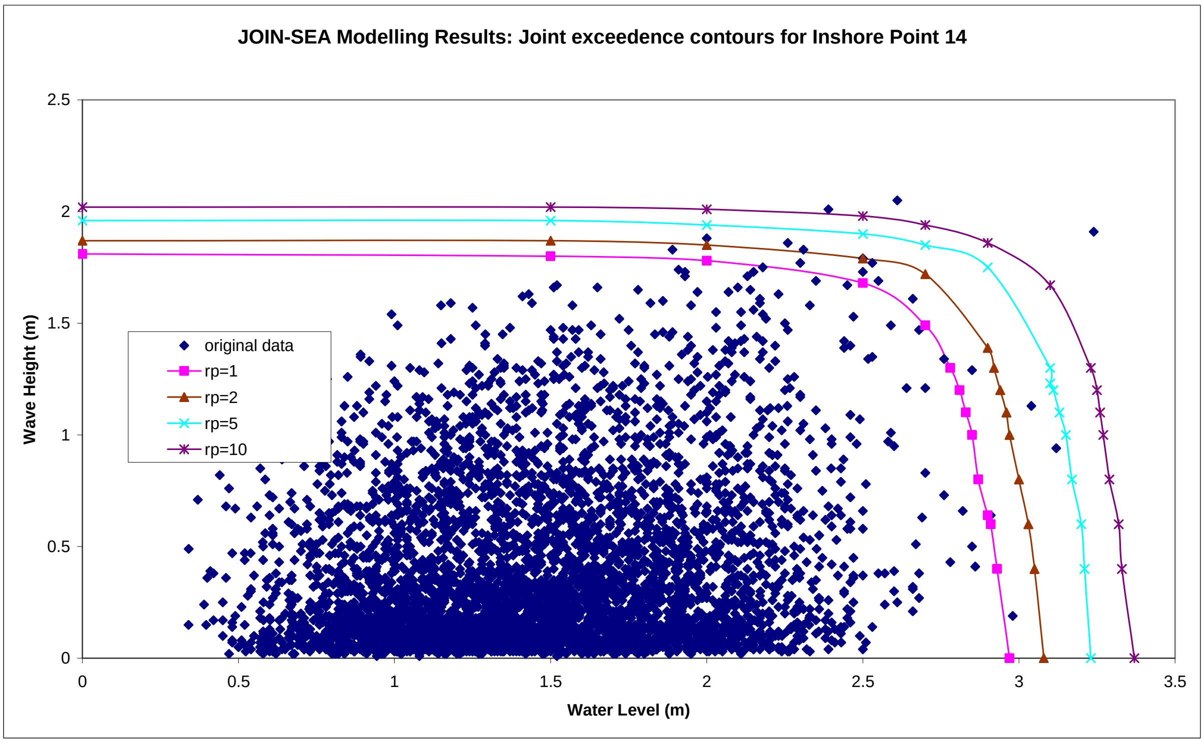


Figure 4-4: Joint probability results for inshore position 14 of Dawlish Warren coast.

Table 4-3: Joint extreme water levels and wave heights for a 1 year recurrence interval.

Nearshore Point	Joint extreme values of water levels and wave heights 1 in 1 year return period														
6	WL(m)	0	1.5	2	2.2	2.31	2.4	2.6	2.73	2.8	2.83	2.87	2.98		
	Hmo(m)	1.67	1.67	1.65	1.63	1.6	1.57	1.47	1.4	1.28	1.2	1	0		
7	WL(m)	0	1.5	2	2.17	2.2	2.4	2.6	2.72	2.8	2.82	2.86	2.9	2.92	2.97
	Hmo(m)	1.45	1.45	1.43	1.4	1.39	1.3	1.25	1.2	1.08	1	0.8	0.6	0.4	0
8	WL(m)	0	1.5	2	2.5	2.53	2.7	2.84	2.9	2.92	2.94	2.96	2.98	2.99	
	Hmo(m)	1.26	1.26	1.25	1.21	1.2	1.13	1	0.88	0.8	0.6	0.4	0.2	0	
9	WL(m)	0	1.5	2	2.48	2.5	2.7	2.75	2.9	2.91	2.94	2.96	2.98		
	Hmo(m)	0.94	0.94	0.93	0.9	0.9	0.83	0.8	0.64	0.6	0.4	0.2	0		
12	WL(m)	0	1.5	2	2.13	2.5	2.63	2.7	2.77	2.85	2.9	2.91	2.94	2.96	2.98
	Hmo(m)	1.12	1.12	1.11	1.1	1.04	1	0.95	0.9	0.8	0.67	0.6	0.4	0.2	0
13	WL(m)	0	1.5	2	2.42	2.5	2.67	2.7	2.77	2.8	2.85	2.89	2.9	2.92	2.98
	Hmo(m)	1.39	1.39	1.37	1.3	1.27	1.2	1.18	1.1	1	0.8	0.6	0.54	0.4	0
14	WL(m)	0	1.5	2	2.5	2.7	2.78	2.81	2.83	2.85	2.87	2.9	2.91	2.93	2.97
	Hmo(m)	1.81	1.8	1.78	1.68	1.49	1.3	1.2	1.1	1	0.8	0.64	0.6	0.4	0
15	WL(m)	0	1.5	1.96	2	2.2	2.3	2.4	2.46	2.6	2.69	2.78	2.8	2.83	2.98
	Hmo(m)	2.06	2.05	2.0	1.99	1.9	1.8	1.68	1.6	1.48	1.4	1.2	1.13	1.0	0.0

Table 4-4: Joint extreme water levels and wave heights for return period of 1 in 100 years.

Nearshore Point	Joint extreme values of water levels and wave heights 1 in 100 year return period																
6	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	2.85	3	3.59	3.89	3.89	3.89	3.89		
	Hmo(m)	1.87	1.87	1.85	1.84	1.83	1.82	1.81	1.8	1.77	1.6	1.4	1.2	1	0		
7	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	3	3.07	3.49	3.7	3.7	3.71	3.71	3.71	3.71
	Hmo(m)	1.71	1.71	1.71	1.71	1.68	1.65	1.65	1.62	1.6	1.4	1.2	1	0.8	0.6	0.4	0
8	WL(m)	0	1.5	2	2.11	2.5	2.7	2.9	3.1	3.5	3.59	3.79	3.79	3.79	3.79	3.81	3.81
	Hmo(m)	1.41	1.41	1.41	1.4	1.39	1.38	1.35	1.32	1.28	1.2	1	0.8	0.6	0.4	0.2	0
9	WL(m)	0	1.5	2	2.5	2.7	2.9	3.03	3.1	3.5	3.58	3.75	3.8	3.81	3.81	3.81	
	Hmo(m)	1.07	1.07	1.07	1.06	1.05	1.02	1	0.98	0.93	0.9	0.8	0.6	0.4	0.2	0	
12	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.51	3.62	3.65	3.75	3.76	3.76	3.76	3.76
	Hmo(m)	1.27	1.27	1.27	1.24	1.24	1.23	1.22	1.12	1.1	1	0.9	0.8	0.6	0.4	0.2	0
13	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.54	3.63	3.66	3.66	3.66	3.67	3.73	3.74
	Hmo(m)	1.61	1.61	1.6	1.56	1.53	1.53	1.47	1.34	1.3	1.2	1.1	1	0.8	0.6	0.4	0
14	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.64	3.64	3.65	3.65	3.66	3.69	3.69	3.69
	Hmo(m)	2.22	2.22	2.22	2.2	2.19	2.07	2	1.79	1.3	1.2	1.1	1	0.8	0.6	0.4	0
15	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	3	3.29	3.52	3.55	3.62	3.65	3.66	3.72	3.77
	Hmo(m)	2.77	2.77	2.76	2.75	2.75	2.7	2.55	2.44	2.2	2	1.8	1.6	1.4	1.2	1	0

Table 4-5: Joint extreme water levels and wave heights for return period of 1 in 500 years

Nearshore Point	Joint extreme values of water levels and wave heights 1 in 500 year return period																
6	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	3	3.24	4.33	4.37	4.37	4.37	4.37		
	Hmo(m)	1.97	1.97	1.96	1.96	1.86	1.86	1.86	1.81	1.8	1.6	1.4	1.2	1	0		
7	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	3	3.28	3.81	4.08	4.08	4.08	4.08	4.08	4.08
	Hmo(m)	1.76	1.76	1.76	1.75	1.73	1.71	1.71	1.71	1.6	1.4	1.2	1	0.8	0.6	0.4	0
8	WL(m)	0	1.5	2	2.5	2.7	2.9	2.93	3.1	3.5	3.83	4.05	4.05	4.05	4.05	4.05	4.05
	Hmo(m)	1.43	1.43	1.43	1.43	1.43	1.42	1.4	1.36	1.31	1.2	1	0.8	0.6	0.4	0.2	0
9	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.44	3.5	3.85	3.98	4.41	4.41	4.41	4.41	
	Hmo(m)	1.11	1.11	1.11	1.1	1.1	1.08	1.06	1	1	0.9	0.8	0.6	0.4	0.2	0	
12	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.84	3.84	3.96	3.97	3.97	3.97	3.97	3.97
	Hmo(m)	1.32	1.32	1.32	1.32	1.32	1.32	1.3	1.22	1.1	1	0.9	0.8	0.6	0.4	0.2	0
13	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.67	3.84	3.84	3.84	3.91	3.91	3.91	3.97
	Hmo(m)	1.65	1.65	1.65	1.63	1.63	1.6	1.6	1.52	1.3	1.2	1.1	1	0.8	0.6	0.4	0
14	WL(m)	0	1.5	2	2.5	2.7	2.9	3.1	3.5	3.97	3.97	3.97	3.97	3.97	4.12	4.12	4.12
	Hmo(m)	2.51	2.51	2.51	2.51	2.51	2.32	2.1	2	1.3	1.2	1.1	1	0.8	0.6	0.4	0
15	WL(m)	0	1.5	2	2.2	2.4	2.6	2.8	3	3.52	3.87	3.87	3.96	3.96	3.96	3.96	3.96
	Hmo(m)	3.02	3.02	3.02	3.02	3.02	2.89	2.84	2.74	2.2	2	1.8	1.6	1.4	1.2	1	0

5 Shoreline Evolution Modelling

5.1 COASTLINE Model

The Halcrow-developed longshore sediment transport modelling software package, COASTLINE, was used in this study. COASTLINE is a wave energy driven littoral sediment transport and shoreline evolution (1-line) model. This model assumes that a beach profile of constant shape slides along a horizontal base located at the closure depth in response to gradients in the longshore sediment transport rate. Erosion causes the profile to move landward and accretion moves it seaward. With the constant profile and assuming that there is conservation of sediment, all the contours move the same distance and therefore the movement of a single contour line can present the movement of the complete beach profile. Thus the model is known as a single line model. The COASTLINE model is based on two basic equations: one to describe the relationship between the incident waves and littoral drift (sediment transport equation) and another one to describe the relationship between littoral drift and beach line movement (sediment continuity equation). The sediment transport equation (Kamphuis, 1991), that includes the wave height gradient term (Ozasa and Brampton, 1980) is shown as follows:

$$Q = 7.3 H_{sb}^{2} T_{p}^{1.5} m_{b}^{0.75} D^{-0.25} \{ \frac{(\sin 2\alpha \cos 2\beta) - (2\cos 2\alpha \cos^{2}\beta)\frac{\partial y}{\partial x}}{\sin^{0.4}(2\alpha - 2\beta)} - \frac{3.24}{m_b}\cos(\alpha - \beta)\frac{dH_{sb}}{dx} \}$$

Where Q is the longshore transport rate in m^3 per year, H_{sb} is the breaking wave height, T_p is the peak wave period, m_b is the beach slope in the breaking zone, D is the nominal grain size (D_{50}), α is the breaking wave angle (measured from the beach normal) and β is the angle between the alignment of the beach and an arbitrary horizontal baseline.

The sediment continuity equation assumes that there is conservation of sediment for an infinitely small length of shoreline Δx (See Figure 5-1):

$$\frac{\partial Q}{\partial x} + d_p \frac{\partial y}{\partial t} = 0$$

where y is the shoreline position, x is the longshore coordinate, t is the time, Q is the longshore sediment transport, q represents the average on-offshore transport rate (taken to be zero at the closure depth) and d_P is the profile depth which equals the closure depth d_c plus the beach berm height d_B.

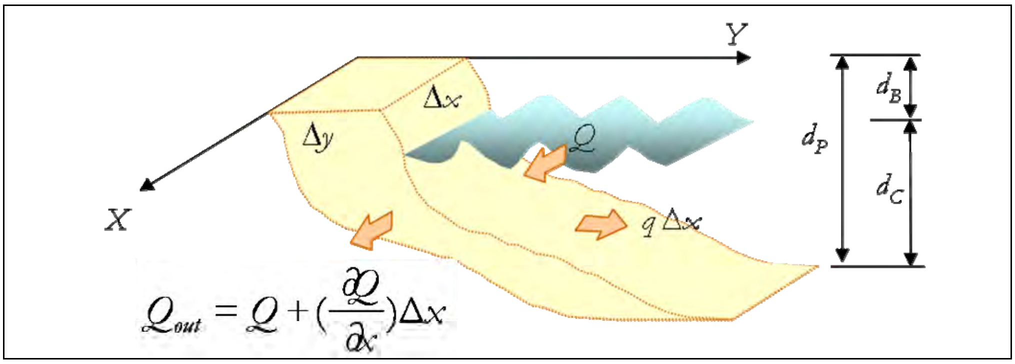


Figure 5-1: Definition Sketch of Conservation of Sediment.

The sediment transport equation is a bulk sediment transport expression. This equation integrates all pertinent fluid flow and sediment entrainment properties into simple sediment transport expressions involving a few wave and beach parameters. With the simplicity of this expression, many calculations can be made without involving large computation times. Many years of wave data can be introduced to the model. The calibration of this model is also relatively simple. No other assumptions have been imposed on the model equations used. The model is a one-dimensional model and it assumes a groyne will have sufficient length extended landward direction and will perform during the study period. Thus the studied groyne should have minimum length of five metres. The input data to COASTLINE include the initial shoreline position and a time-series of wave data comprising all transformed inshore wave data of many years which were used to obtain averaged annual wave conditions and repeated every year, the beach profile and sediment size.

5.2 Dawlish Warren COASTLINE Model Calibration

Two measured coastline positions are used for calibrating the COASTLINE model of Dawlish Warren. The measured coastline position from 3/5/1998, is used as the initial input data, and the most recent measured profile from 22/1/2011, which is used as the target coastline position. The coastline position has been defined from the mean sea level position at each surveyed section. Initial model parameters are based on available data and published literature and are given in Table 5-1. The boundary condition at the south-west is set to be the fixed point assumed to be a closed boundary (i.e. no sediment enters or leaves via this boundary) and at the north-east is set to be the zero gradient of the sediment transport rate.

A site investigation was carried out to assess the condition of groynes along Dawlish Warren. The position of groynes is shown in Figure 5-2 and details of the groyne condition/performance survey are given in Appendix A. The assessment shows that not all groynes are fully functional, some groynes are completely buried by sand, some groynes have missing boards and some groynes only project a small above the beach level allowing sand to pass over the groynes. For calibration purposes it has been assumed that the active length of a groyne is the distance from the groyne root to the mean sea level mark surveyed in 2011 as if this were not the case the beach would either have eroded or accreted to the active length of the groyne.

The model is run using the initial parameters stated in Table 5-1 along with assumed groyne lengths, and is calibrated by adjusting a comprehensive factor representing local variations in sediment size, beach slope and beach for each modelled section. This ensures that the transport rates give the correct shoreline evolution from the initial to the target profile. Calibration of the model is shown in Figure 5-2, and the model results show good agreement with the surveyed coastline positions. The sediment size in the study area is at the limit of validity of the Kamphuis equation for sediment transport used in the COASTLINE model. However, the good calibration provides confidence in the results.

Table 5-1: Input Parameters of COASTLINE Model for Dawlish Warren.

Input parameters	Value used	Remarks
Water level	0.07mOD N	Mean sea level at Exmouth (Approaches)
Sediment size (D_{50})	3.72mm	Mean D_{50} for sediment samples for Dawlish Warren (section 2.5.2)
Sediment specific gravity	2.65	Typical value for sediment (Soulsby, 1997)
Sediment porosity	35%	Typical value for sediment (Soulsby, 1997)
Beach height	4.1m	Difference between MHWS and MLWS at Exmouth Approaches.
Beach slope	0.038	Average slope between MHWS and MLWS for each of the beach profiles.

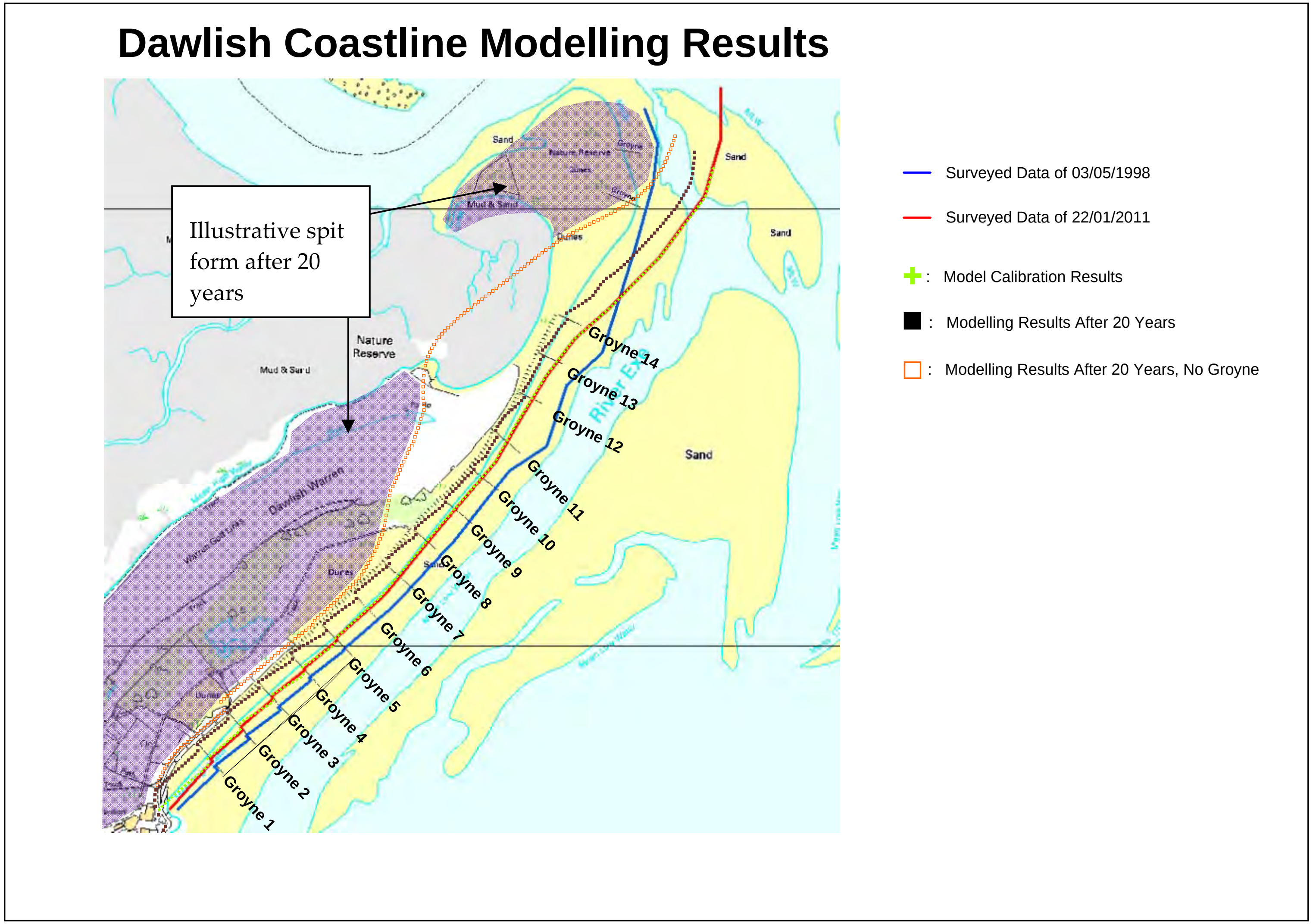


Figure 5-2: Dawlish coastline model calibration and prediction results.

5.3 Dawlish Warren COASTLINE Model Do Nothing Scenario

The calibrated model has been used to predict the future evolution of Dawlish Warren over the next 20 years under the "do-nothing" scenario. Between 1998 and 2011 there has been deterioration in the groyne condition. Over the next 20 years the groyne condition is assumed to deteriorate further so that the functional groyne length will be 10m from the root of the groynes. The model predicts that the beach will retreat to the groyne roots after 20 years if groyne conditions are not improved and there is no recharge of the beach.

A sensitivity test on the assumption that the groynes will remain on the beach in any form in future has been tested by removing groynes from the model entirely. The results in Figure 5-2 show that the central section of Dawlish Warren will be eroded away without groynes. This was observed to have occurred in the 1940s as shown by aerial photography from 1945, Figure 5-3, and Admiralty Charts from 1949, Figure 5-4.

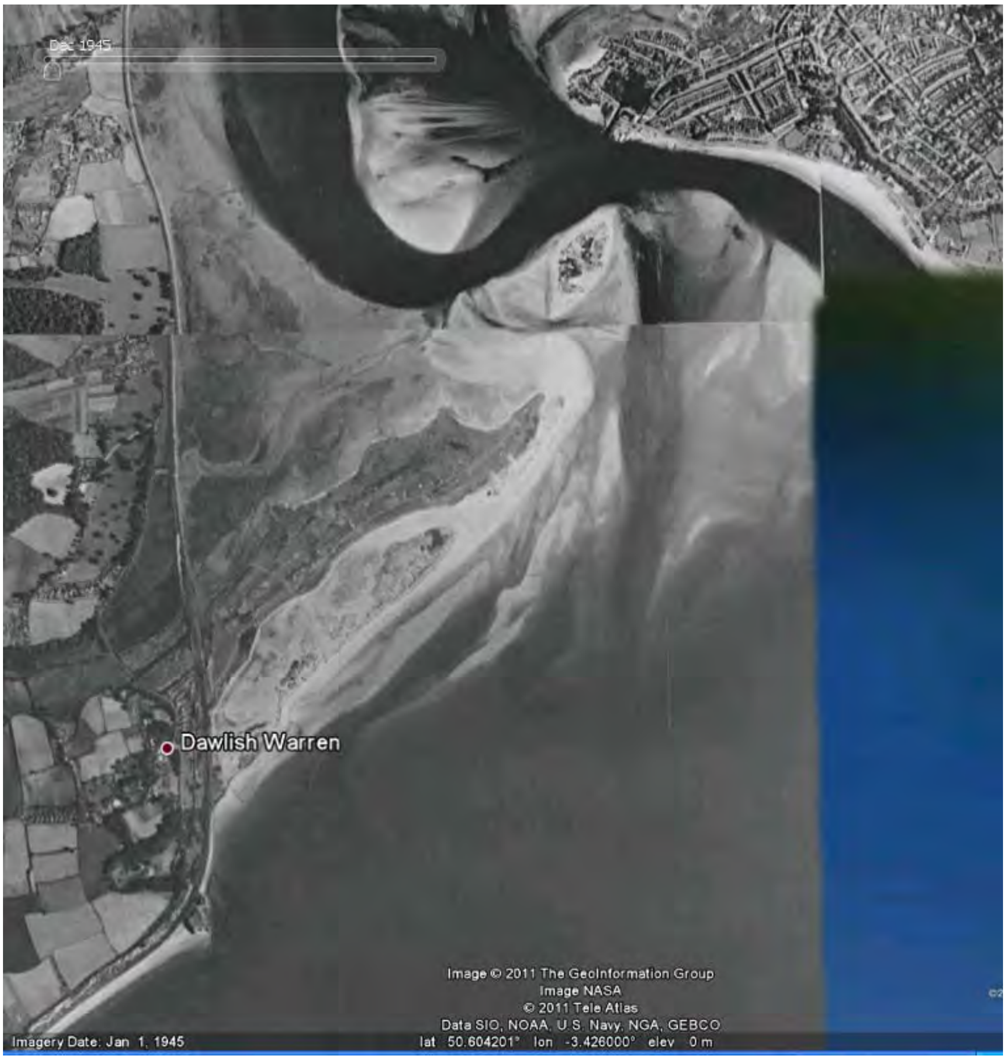


Figure 5-3: Dawlish coastal layout of 1945 from Google Earth.

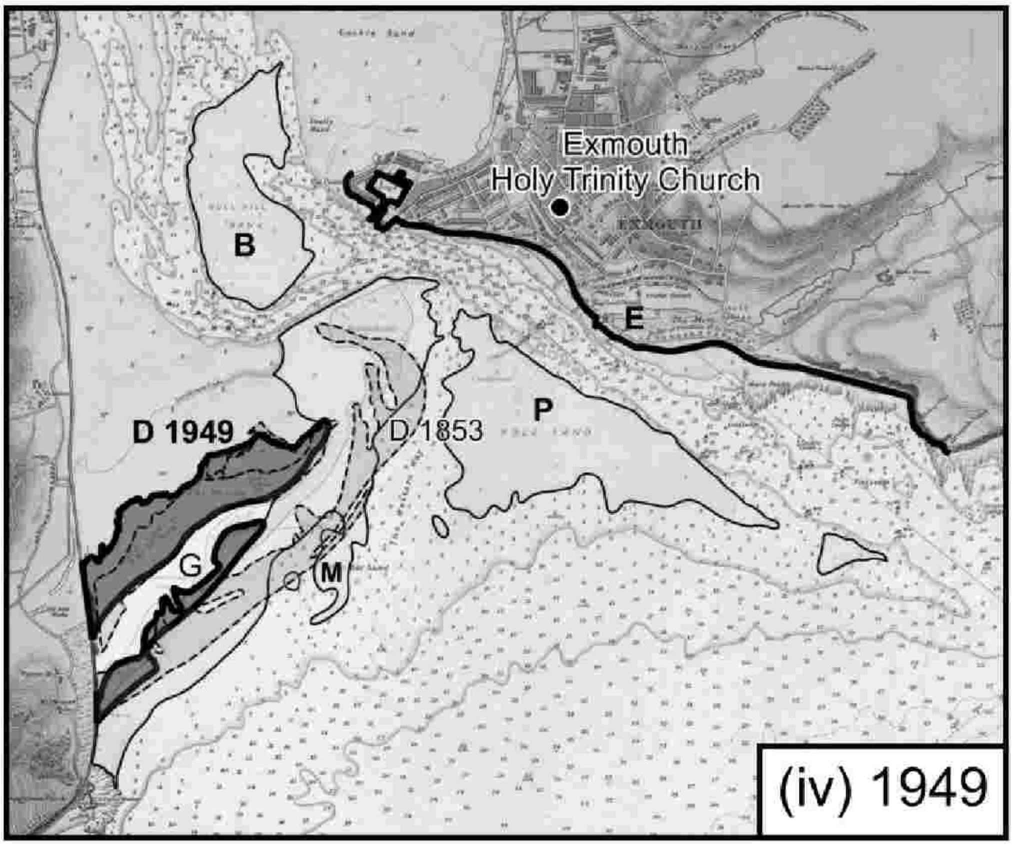


Figure 5-4: Dawlish coastal layout of 1949.

5.4 Exmouth Beach COASTLINE Model Calibration

Two measured coastline positions are used for calibrating the COASTLINE model of Exmouth Beach. The measured coastline position from 20/04/2007 is used as the initial input data, and the most recent measured profile from 23/01/2011, is used as the target coastline position. The coastline position has been defined from the mean sea level position at each surveyed section. The boundary condition at the south-east is set to be the fixed point closed boundary (i.e. no sediment enters or leaves via this boundary) and at the north-west is set to be the zero gradient of the sediment transport rate. Initial model parameters are based on available data and published literature and are given in Table 5-2 for the Exmouth coast.

Table 5-2: Input Parameters of COASTLINE Model for Exmouth.

Input parameters	Value used	Remarks
Water level	0.07mODN	Mean sea level at Exmouth (Approaches)
Sediment size (D_{50})	1.01mm	Mean D_{50} for sediment samples for Exmouth beach (section 2.5.2)
Sediment specific gravity	2.65	Typical value for sediment (Soulsby, 1997)
Sediment porosity	35%	Typical value for sediment (Soulsby, 1997)
Beach height	4.1m	Difference between MHWS and MLWS at Exmouth Approaches.
Beach slope	0.0304	Average slope between MHWS and MLWS for each of the beach profiles.

A site investigation was carried out to assess the condition of groynes along the Exmouth coast. Details of the groyne condition/performance survey are given in Appendix A. For calibration purposes it has been assumed that the active length of a groyne is the distance from the groyne root to the mean sea level mark surveyed in 2011 as if this were not the case the beach would either have eroded or accreted to the active length of the groyne. It can be see that Groyne 6, location shown in Figure 5-6, is almost completely covered by sand. There is an approximately 1 m diameter outfall adjacent to Groyne 6 which has been treated as a sand barrier which reduces sediment transport rates, but does not stop all transport across the barrier, during model calibration.

The model is run using the initial parameters stated in table 5-2 along with assumed groyne lengths, and is calibrated by adjusting a comprehensive factor representing local variations in sediment size, beach slope and beach for each modelled section. This ensures that the transport rates give the correct evolution from the initial to the target profile. Calibration of the model is shown in Figure 5-5 and Figure 5-6, and the model results show good agreement with the surveyed coastline positions.

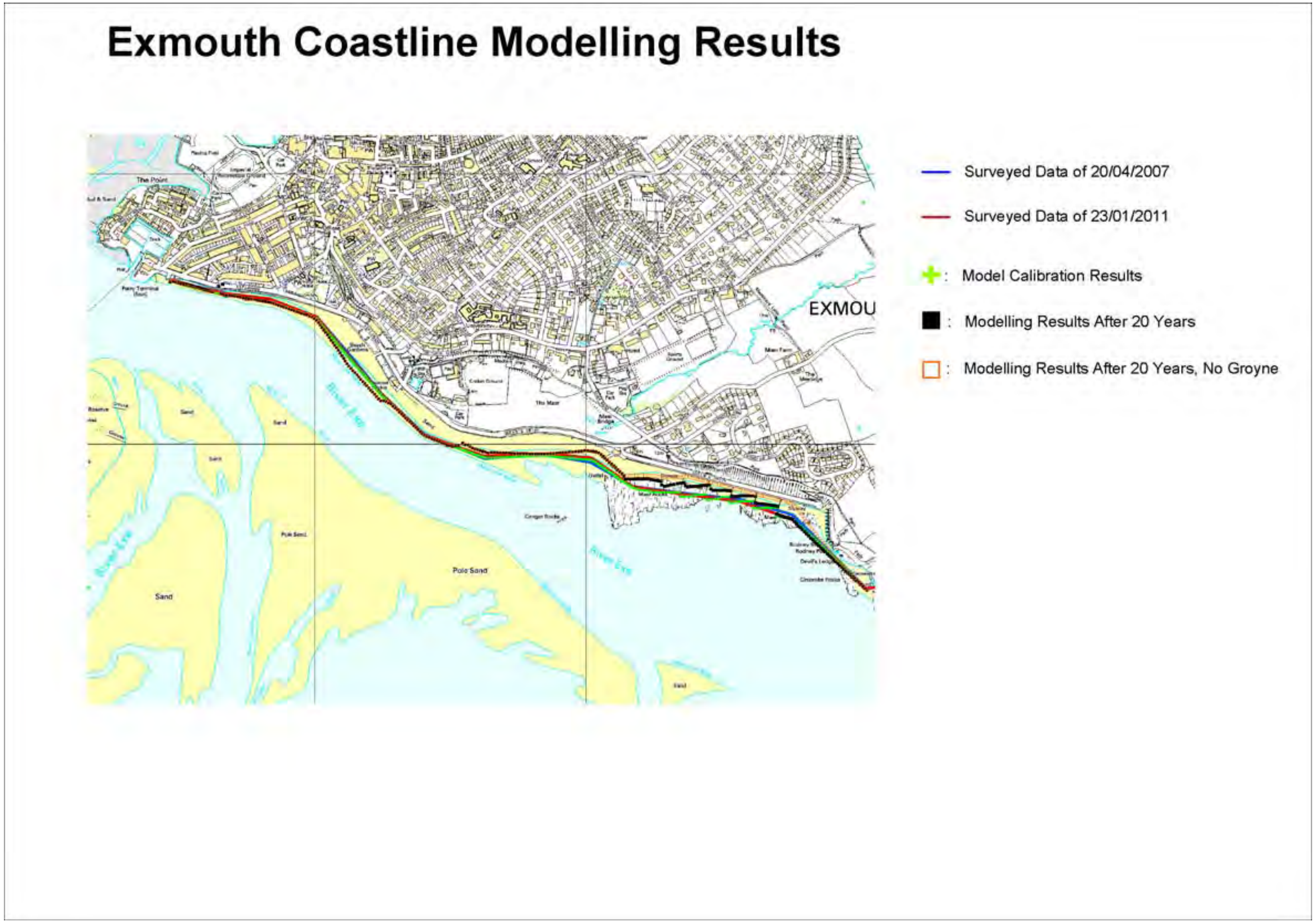


Figure 5-5: Coastline model calibration results for Exmouth.

Halcrow

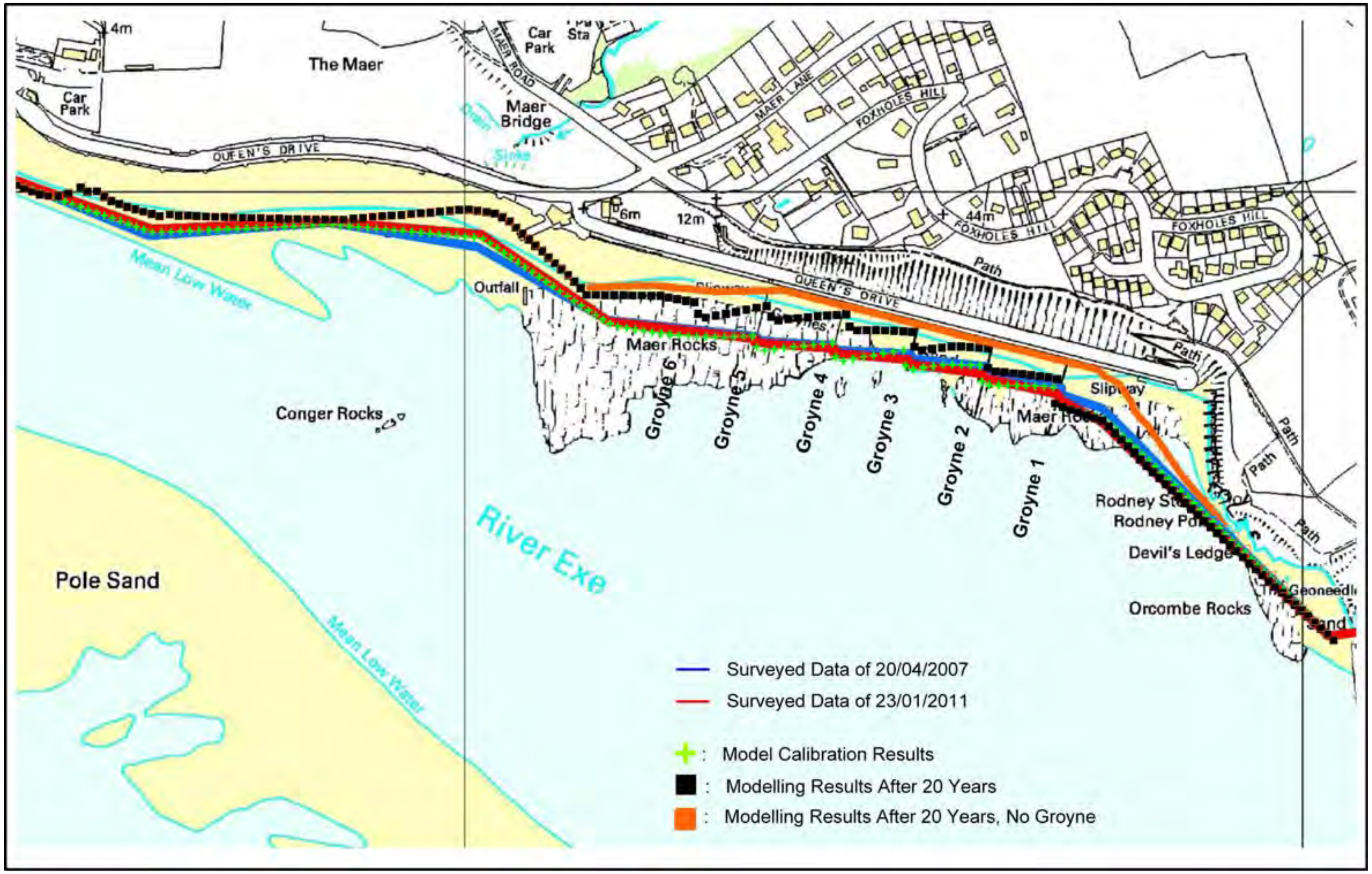


Figure 5-6: Coastline model calibration results for the section of Exmouth Beach along Queen's Drive.

Halcrow

5.5 Exmouth Beach COASTLINE Model Do Nothing Scenario

For modelling the evolution of the coastline over the next 20 years it has been assumed that the groyne condition will deteriorate further, and the functional groyne length will be 10m from the groyne root. From Google Earth (Figure 5-7) it can clearly be seen that the functional lengths of these groynes is already shorter than their actual length. In this scenario the model predicts that the eastern end of the beach will retreat to the groyne root positions within 20 years if groyne conditions are not improved and the beach is not recharged.

A sensitivity test on the assumption that the groynes will remain on the beach in any form in future has been tested by removing groynes from the model entirely. The results in Figure 5-5 show that in this scenario the eastern part of the Exmouth beach will be eroded back to the seawall.

Figure 5-7: Eastern part of Exmouth coast from Google Earth

5.6 Dawlish Warren COASTLINE Model "Do Something" Options

The calibrated coastline model is used to study different options to protect Dawlish Warren from further erosion due to long-shore drift from the west of Dawlish Warren to the east. In general there are two methods which can be used to reduce the erosion problem, improving some or all of the existing groynes which are in poor condition or recharging the beach with sediment from elsewhere. A combination of these options is also possible. A total of seven options have been studied and those options are listed in Table 5-3.

Table 5-3: Summary of different modelling options for Dawlish Warren.

Run No.	Option No.	Groynes	Recharge
0	D0	All groynes existing condition	No recharge.
1	D1	All groynes existing condition	Recharge between groynes 0-5 (115,000m^3)
2	D2	All groynes fully functional	Recharge between groynes 0-5 (115,000m^3)
3	D3	All groynes existing condition	Recharge between groynes 0-14 (250,000m^3)
4	D4	All groynes fully functional	Recharge between groynes 0-14 (250,000m^3)
5	D5	Groynes 12 13 & 14 are fully functional Remaining groynes existing condition	Recharge between groynes 0-5, 12-14 (130,000m^3)
6	D6	Increase length of groynes 12 13 & 14 by 50% and make them fully functional Remaining groynes existing condition	Recharge between groynes 0-5, 12-14 (145,000m^3)
7	D7	Increase length of groynes 12 13 & 14 by 50% Remaining groynes fully functional	Recharge between groynes 0-5, 11-14 (160,000m^3)

For Option D1 the beach is recharged between Groynes 1 and 5, and groynes are assumed to remain in their existing condition. Results for this case are shown in Figure 5-8 together with the D0 baseline case for comparison. The results for Option D1 show that the beach section from Groyne 1 to Groyne 9 will be improved relative to the baseline scenario as the recharged sediment drifts from the south-west towards the north-east due to wave action. The results show no improvement relative to the baseline case between Groynes 9 and 14.

For Option D2 the beach is recharged between Groynes 1 and 5, and all groynes are repaired or re-installed to a fully functional condition. The initial coastline and the projected coastline after 20 years are shown Figure 5-9. The beach shape between groynes will be adjusted responding to the incident dominate wave direction. It has been found that the beach between Groyne 11 and Groyne 13 will retreat to the groyne roots. Thus this option is still not ideal for the Dawlish Warren coast and further improvement is required for the beach between Groynes 11 and 14.

For Option D3 the beach is recharged between Groynes 1 and 14 and all groynes are assumed to remain in their existing condition. Figure 5-10 shows the initial coastline and the projected coastline after 20 years. The modelling results show that the beach sections between Groynes 1 and 9 and between Groynes 12 and 14 will be improved relative to the do nothing case. However, the improvement between Groynes 9 and 12 is marginal. The recharged sediment is transported from the south-west to the north-east by wave actions. This also appears to result in a large increase in the eastern extent of the beach, which is greatest around Groyne 14; however this is symptomatic of one-line shoreline evolution models and in reality this increase would be transported beyond the distal end (to the offshore banks) by nearshore currents.

For Option D4 the beach is recharged between Groynes 1 and 14 and all groynes are repaired or re-installed to a fully functional condition. Figure 5-11 shows the initial coastline and the projected coastline after 20 years. The results show the Dawlish beach with this option will be stable over the next twenty years; however it is also the most expensive option modelled. In practice it is unlikely that groynes will remain in good condition over 20 years and some deterioration in the groyne condition is likely. Inspection of groyne conditions would be required in the future.

For Option D5 the beach is recharged between Groynes 1 and 5 and between Groynes 12 and 14. Groynes 12, 13 and 14 are repaired or reinstalled to a fully functional condition. Figure 5-12 shows the initial coastline and the projected coastline after 20 years. By recharging the south-west section of the beach and repairing the groynes in the north-east section the recharged sediment is transported by wave action from the south-west section to the north-east section and trapped by the repaired groynes. The modelling shows that most of the Dawlish Warren beach is improved relative to the baseline D0 case, but there is only marginal improvement at Groyne 11.

For Option D6 the beach is recharged between Groynes 1 and 5 and between Groynes 12 and 14, and Groynes 12, 13 and 14 are repaired or reinstalled to fully functional conditions and extended seawards by 50% (or 30m). Figure 5-13 shows the initial coastline and the projected coastline after 20 years. The modelling results show improvement relative to the baseline D0 case for the entire of Dawlish Warren beach. The potential capital costs for this option may be lower than for recharging the entire beach and repairing all existing groynes. Compared to Option D5 there is improvement at Groyne 11 as more sand accumulates to the west of Groyne 12 due to the extension of the groynes.

Option D7 is a combination of Option D2 and Option D6 and involves extending Groynes 12, 13 and 14 by thirty metres or 50%, recharging between Groynes 1 and 5 and between Groynes 11 and 14, and repairing or re-installing all groynes to a fully functional condition. Figure 5-14 shows the initial coastline and the projected coastline after 20 years. The model results show the Dawlish Warren coast should be able to stand for another twenty years without breach. However, in practice it is unlikely that groynes will remain in good condition over 20 years and some deterioration in the groyne condition is likely, and it is recommended to inspect all groynes from time to time in the future.

In summary, it is concluded that Option D7 should be considered as the preferred option due to the relatively lower capital costs and maximum benefits to Dawlish Warren beach.

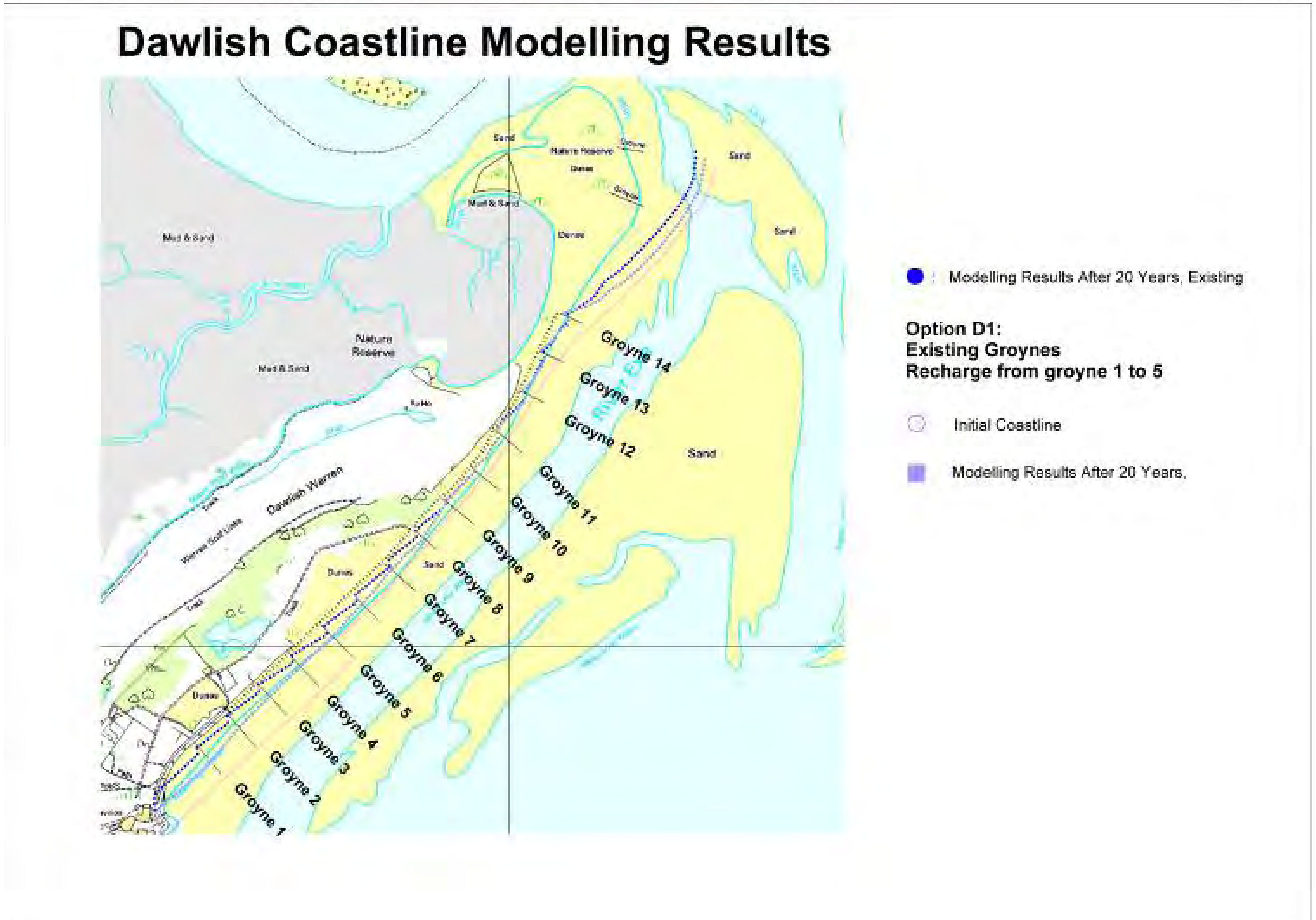


Figure 5-8: Option D1 modelling results for Dawlish Warren.

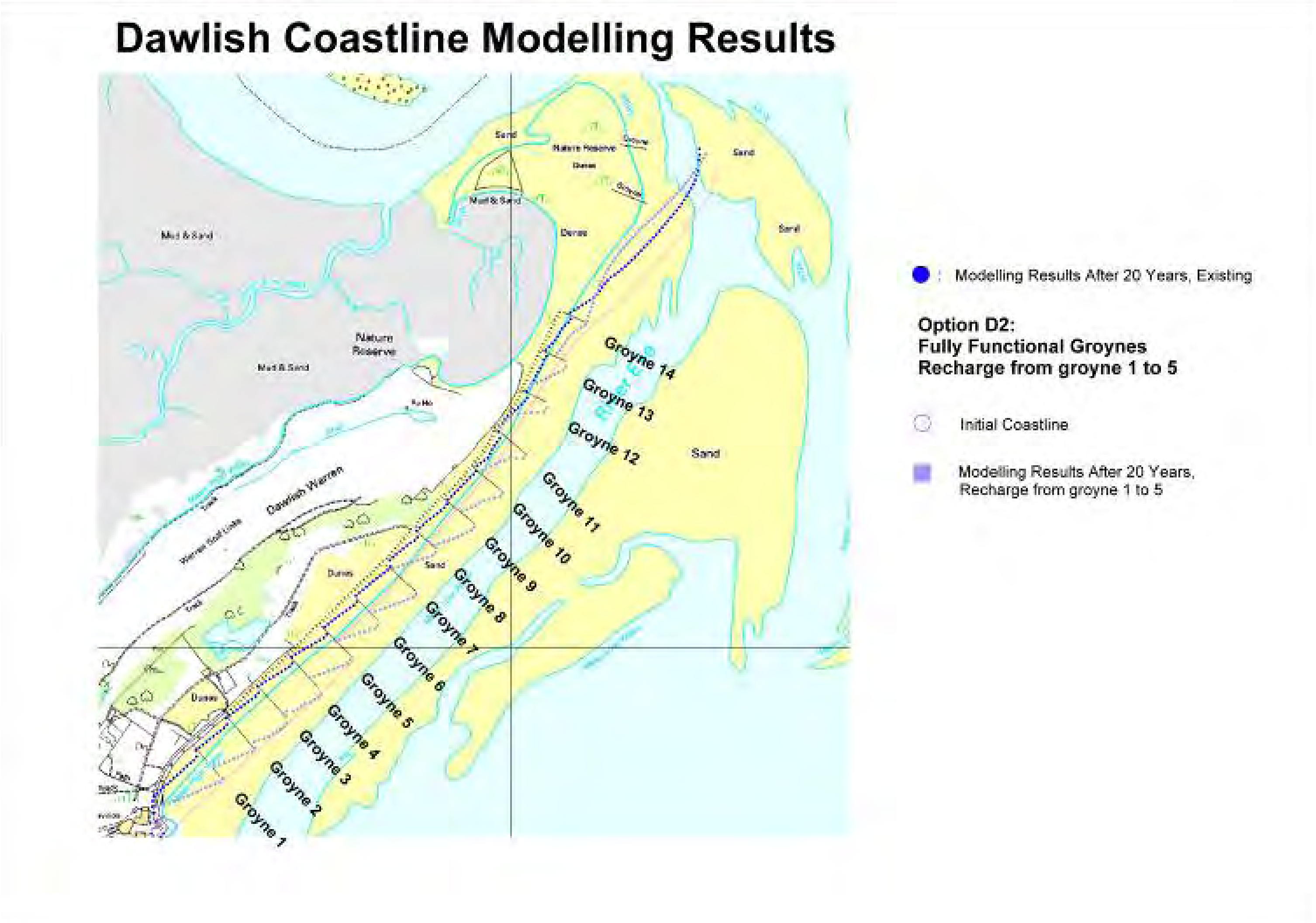


Figure 5-9: Option D2 modelling results for Dawlish Warren.

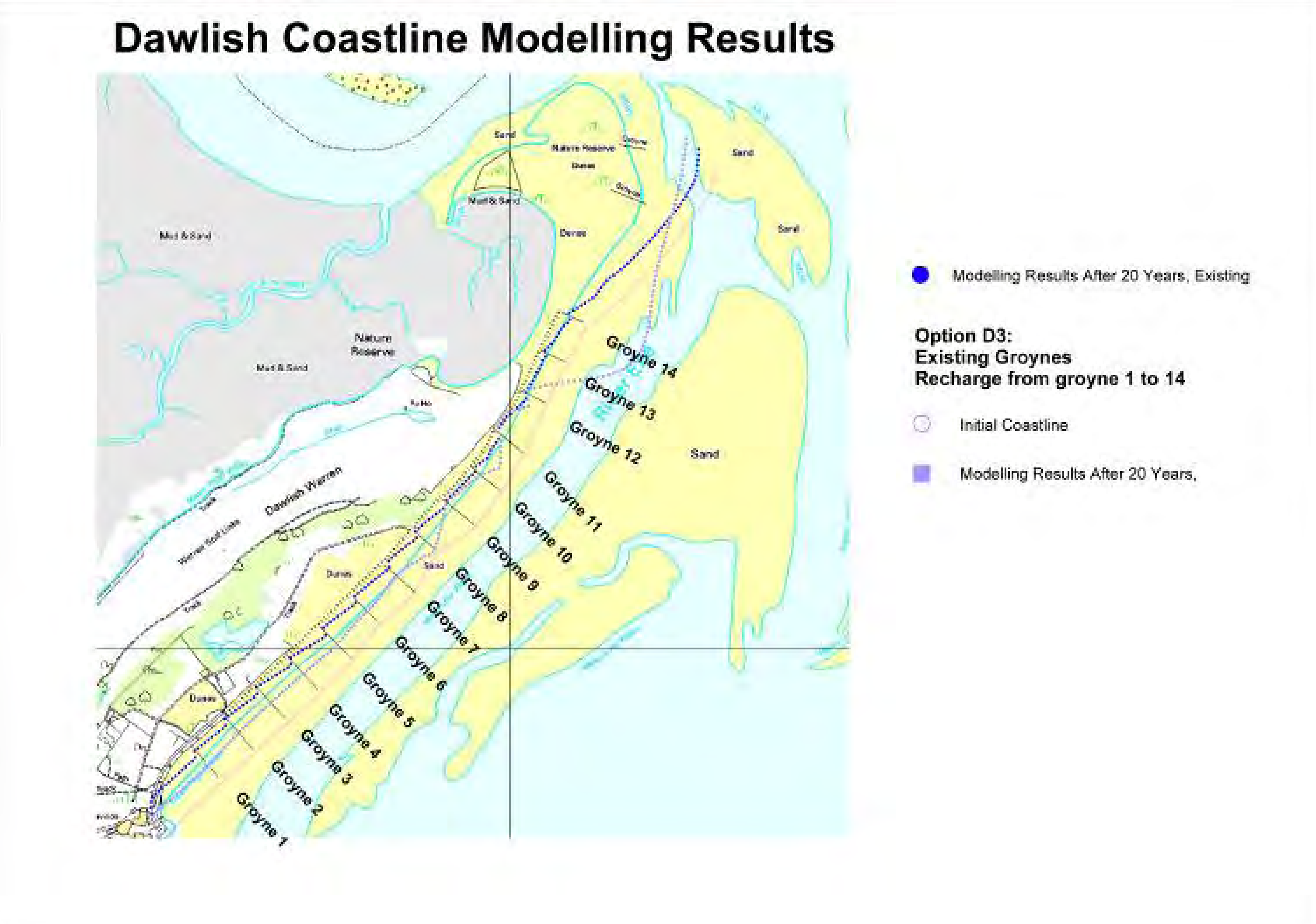


Figure 5-10: Option D3 modelling results for Dawlish Warren.

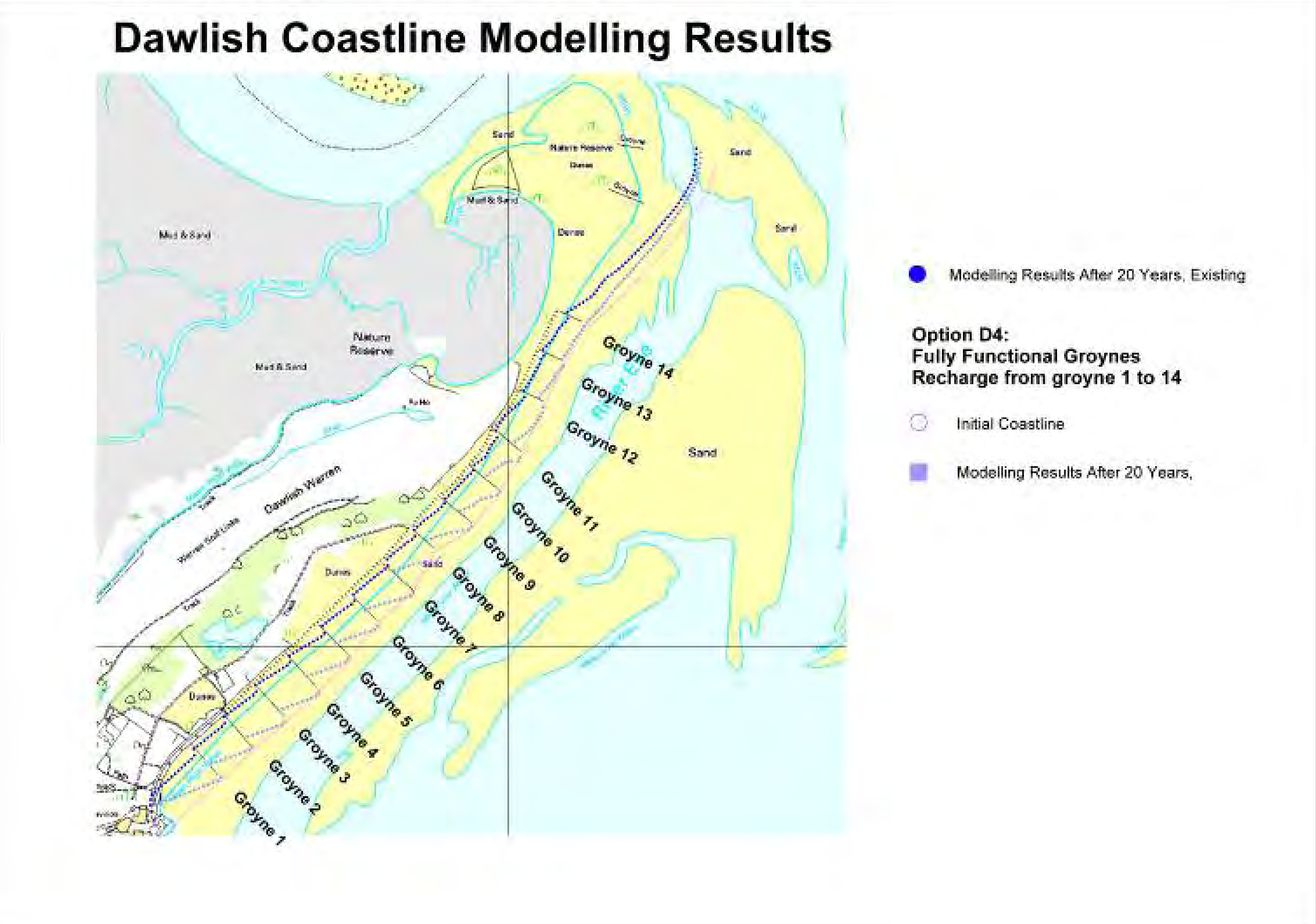


Figure 5-11: Option D4 modelling results for Dawlish Warren.

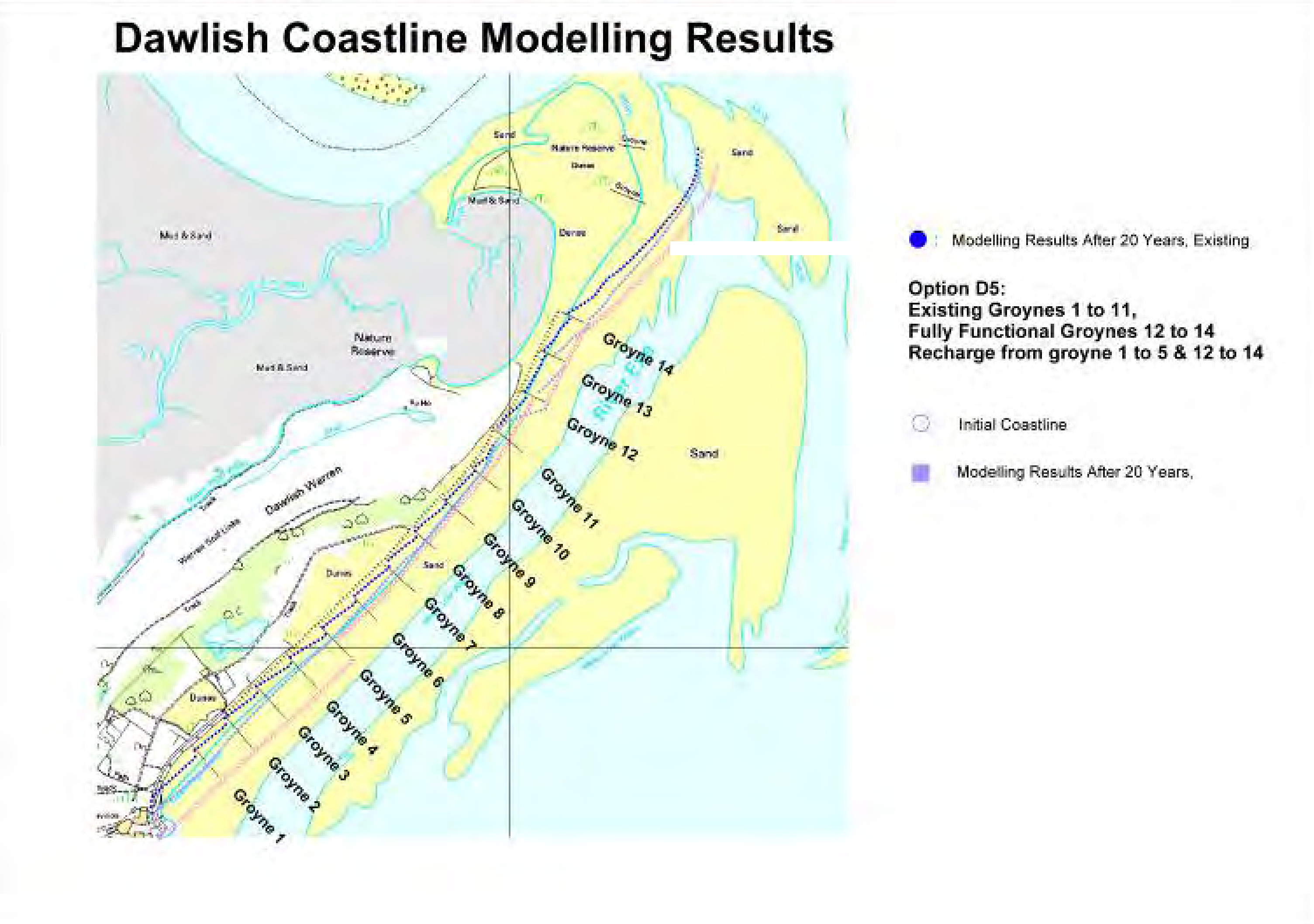


Figure 5-12: Option D5 modelling results for Dawlish Warren.

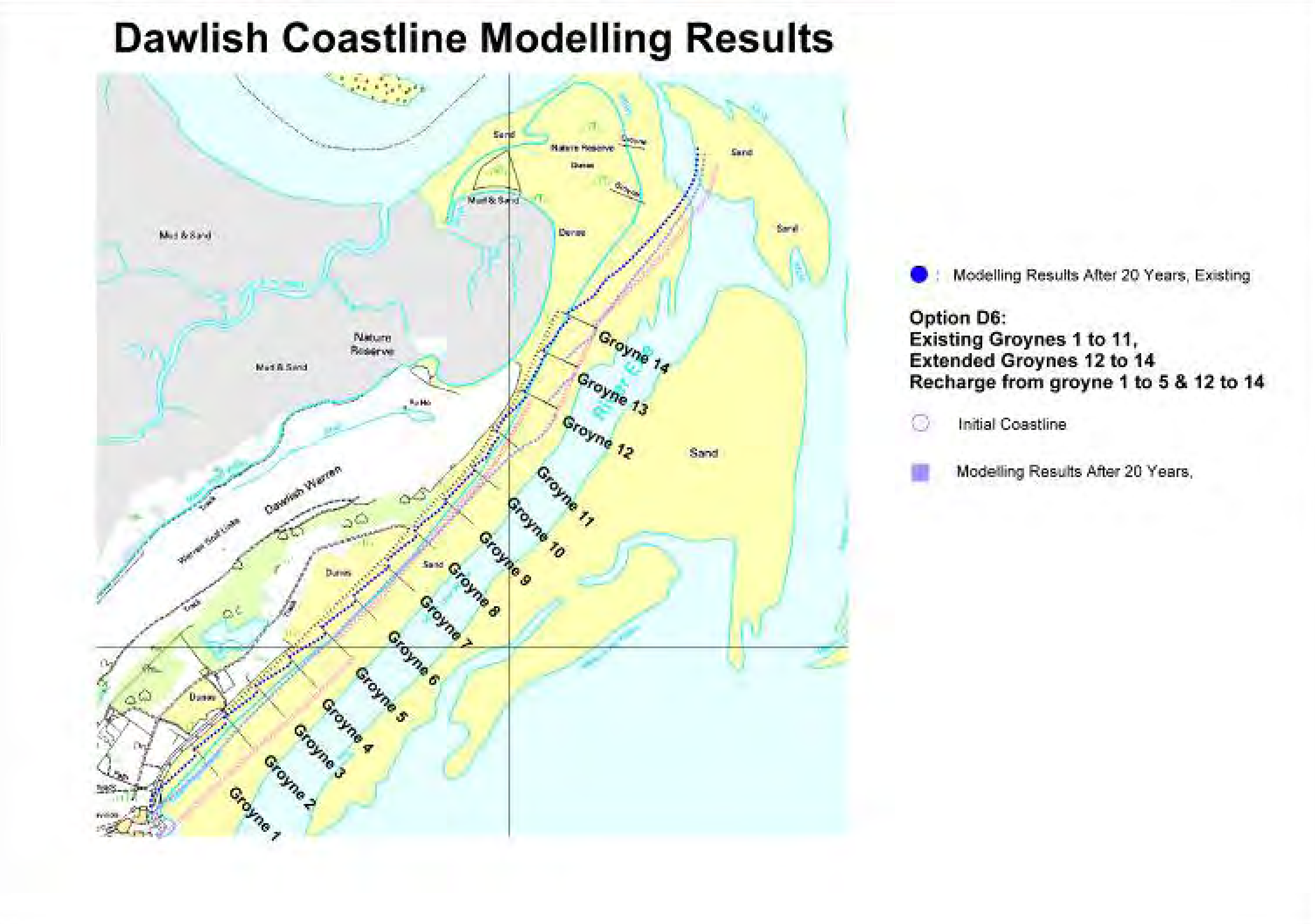


Figure 5-13: Option D6 modelling results for Dawlish Warren.

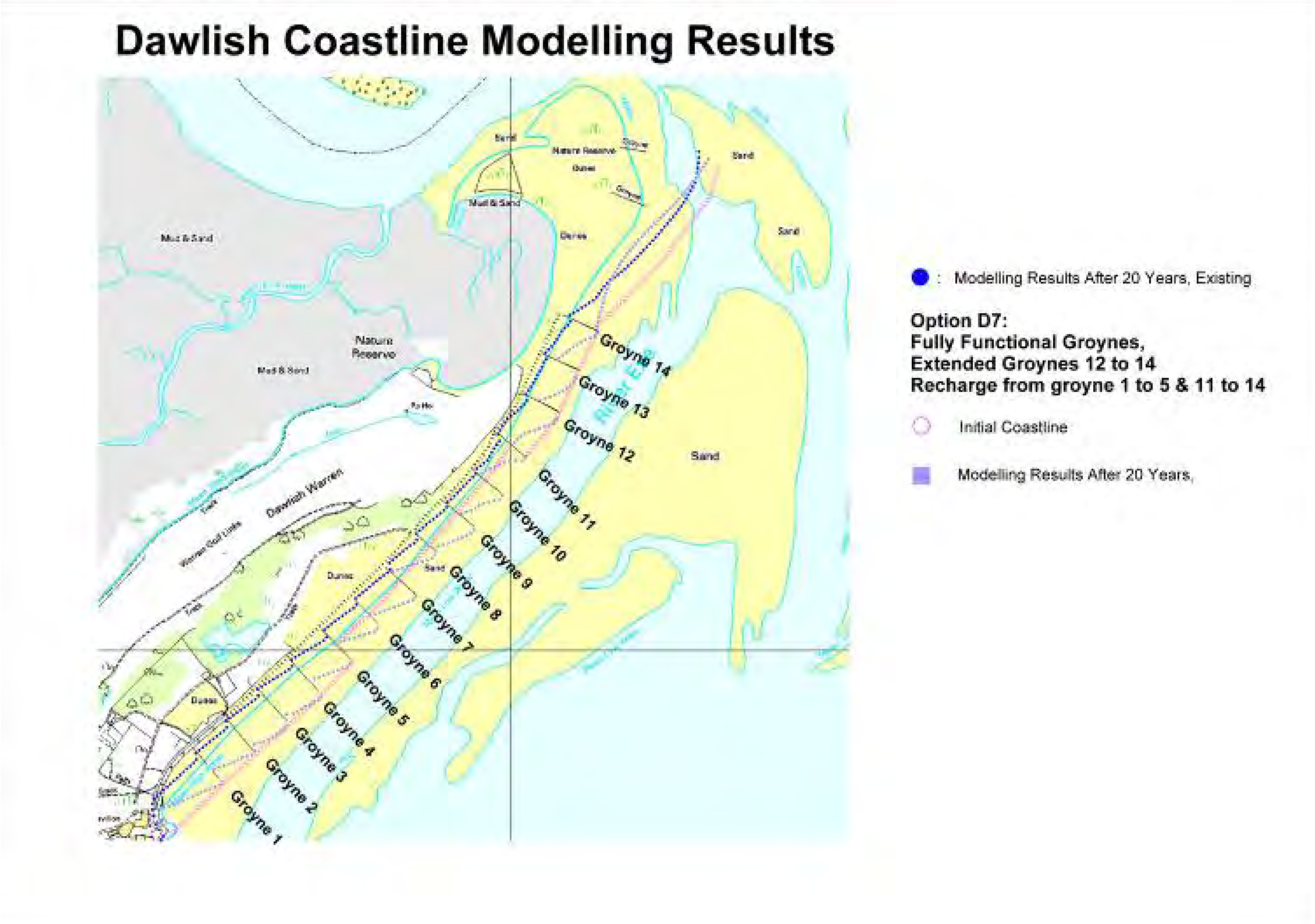


Figure 5-14: Option D7 modelling results for Dawlish Warren.

5.7 Exmouth Beach COASTLINE Model "Do Something" Options

Exmouth Beach is suffering from erosion which is most severe at the eastern section. The calibrated coastline model is used to study different options to protect Exmouth beach from further erosion due to the longshore drift from the south-east towards the north-west. The options assessed are similar to the options assessed for Dawlish Warren (Section 5.6), i.e. improving the existing groynes and at recharging part of the beach. The option of installing new groynes where erosion is most severe has also been assessed. For this project, five options have been studied and those options are listed in Table 5-4.

Table 5-4: Summary of different modelling options for Exmouth Beach.

Run No.	Option No.	Groynes	Recharge
0	E0	All groynes existing condition.	No recharge
1	E1	All groynes existing condition	Recharge along eastern section 1 to 3 (80,000m^3)
2	E2	Fully functional groyne 5, others are existing condition	Recharge along eastern section 1 to 3 (80,000m^3)
3	E3	Fully functional groyne 4 and 5, others are existing condition	Recharge along eastern section 1 to 3 (80,000m^3)
4	E4	Fully functional groyne 4 and 5, others are existing condition; Add a new groyne at Queen's Drive	Recharge along eastern section 1 to 3 (80,000m^3)
5	E5	Fully functional groyne 4 and 5, others are existing condition; Add two new groynes at Queen's Drive	Recharge along eastern section 1 to 3 (80,000m^3)

For Option E1 the beach is recharged between Groyne 1 and 3 and there is no improvement to the existing groynes. Results are shown in Figure 5-15 and Figure 5-16 together with the baseline case (E0) for comparison. **Error! Reference source not found.** shows a smaller scale view of the section between Maer Rocks and Orcombe Point where erosion is highest. The results show that the beach between Groynes 1 and 3 will be improved compared to the existing baseline case, however there will be little improvement between Groynes 3 and 6.

Option E2 is the same as Option E1 except that Groyne 5 is re-installed or repaired to a fully functional condition. Figure 5-17 and Figure 5-18 show the initial coastline and projected coastline after 20 years for Option E2, together with the baseline E0 case for comparison. Beach conditions are improved significantly compared to the baseline case except at Groyne 3.

Option E3 is the same as Option E2 except that Groyne 4 is also re-installed or repaired to a fully functional condition. Results for this option are shown in Figure 5-19 and Figure 5-20. The beach condition is improved between Groynes 1 to 6 relative to the baseline E0 case. However, it is unlikely that Groynes 4 and 5 will remain in good condition during over 20 years.

In Option E4, Groynes 4 and 5 are repaired and an additional groyne is installed in front of Queen's Drive, and the beach is recharged between Groynes 1 and 3. During severe storms it has been observed that sand is transported up the beach in this location and the COASTLINE model shows that there will be beach erosion to the west of the outfall and slipway. Installing a new groyne (New Groyne 1) in this section should increase the stability of this part of the beach. The initial coastline and projected coastline after 20 years are shown in Figure 5-21 and Figure 5-22 for Option E4 and show an improvement compared to the baseline E0 case.

For Option E5, the New Groyne 1 is moved further from the outfall and slipway and an additional groyne (New Groyne 2) is installed west of the New Groyne 1 in order to improve beach stability across the entire Queen's Drive frontage. Figure 5-23 and Figure 5-24 show the initial coastline and the projected coastline after 20 years. The results clearly indicate that both new groynes are necessary to protect this section of beach.

Overall, Exmouth Beach is more stable than the Dawlish Warren beach, and capital costs for the beach stabilisation work are not likely to be as high as for the Dawlish Warren beach. Option E5, where two new groynes are installed in front of Queen's Drive provides the best protection to this section of coast.

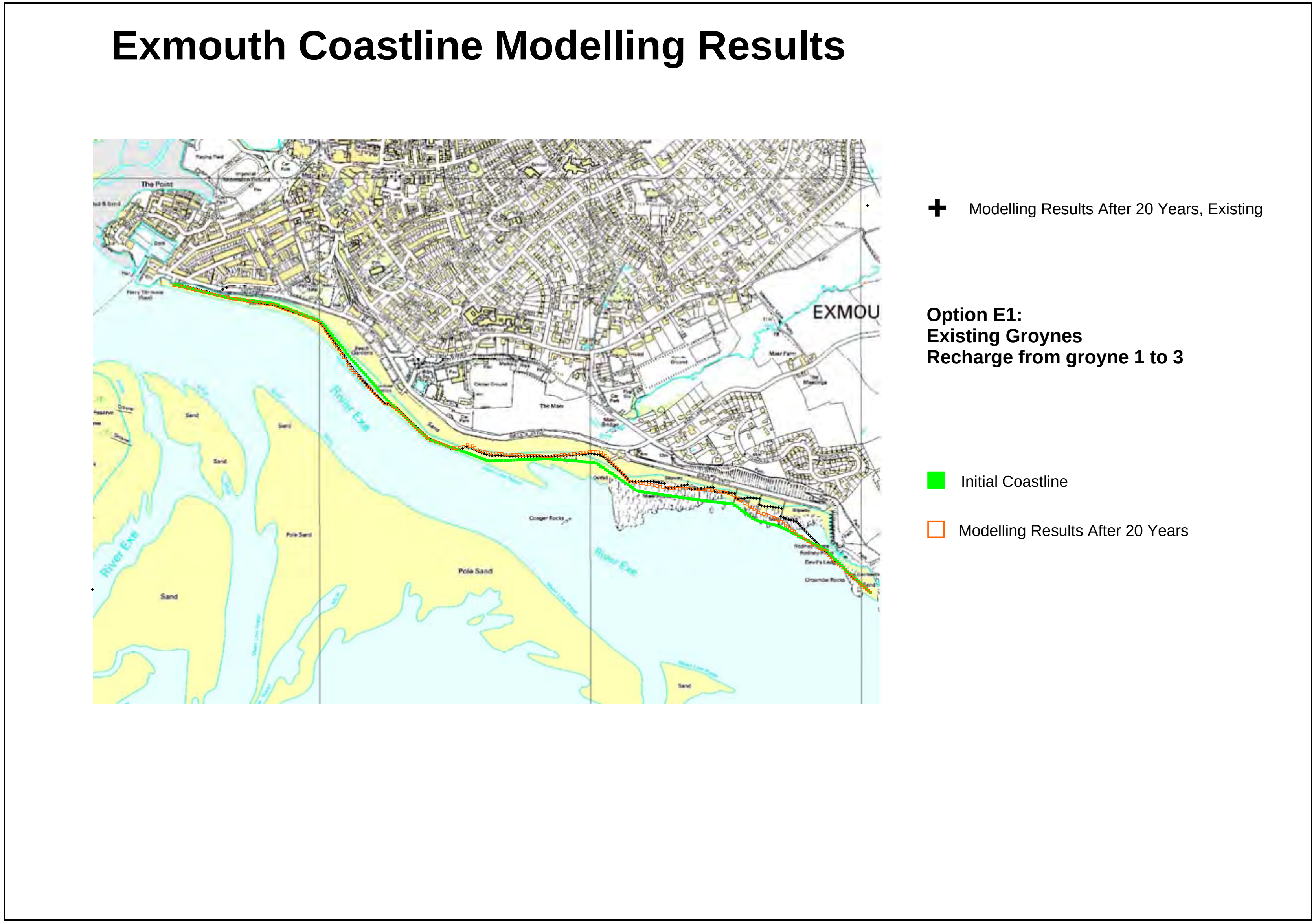


Figure 5-15: Option E1 modelling results for Exmouth coast.

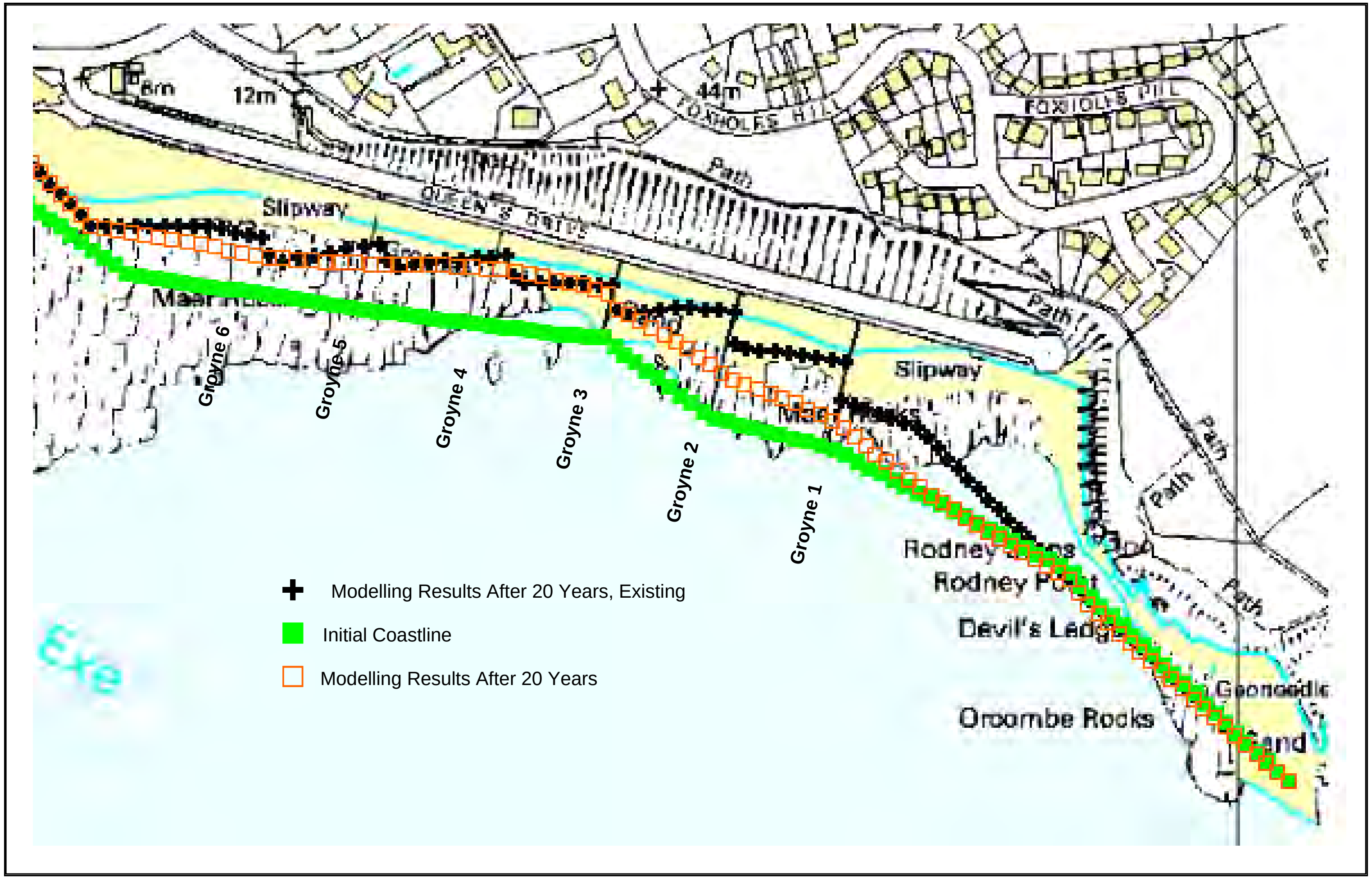


Figure 5-16: Zoomed in plot of Option E1 modelling results for Exmouth coast.

Exmouth Coastline Modelling Results

✚ Modelling Results After 20 Years, Existing

Option E2:
Existing Groynes 1 to 3
Recharge from groyne 1 to 3
Fully Functional Groyne 5

■ Initial Coastline

□ Modelling Results After 20 Years

Figure 5-17: Option E2 modelling results for Exmouth coast.

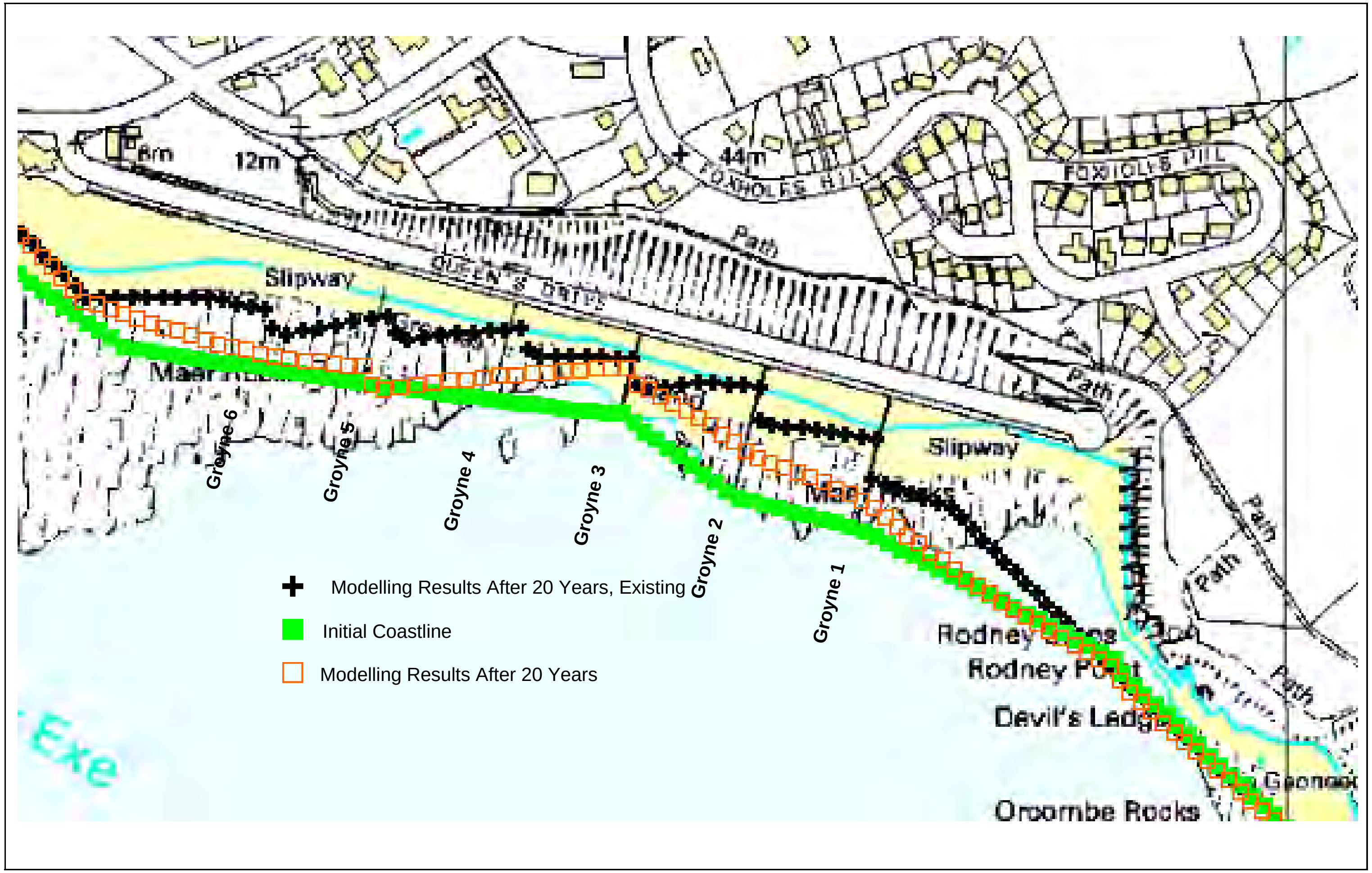


Figure 5-18: Zoomed-in plot of Option E2 modelling results for Exmouth coast.

Exmouth Coastline Modelling Results

✚ Modelling Results After 20 Years, Existing

Option E3:
Existing Groynes 1 to 3
Recharge from groyne 1 to 3
Fully Functional Groyne 4 and 5

Initial Coastline

Modelling Results After 20 Years

Figure 5-19: Option E3 modelling results for Exmouth coast.

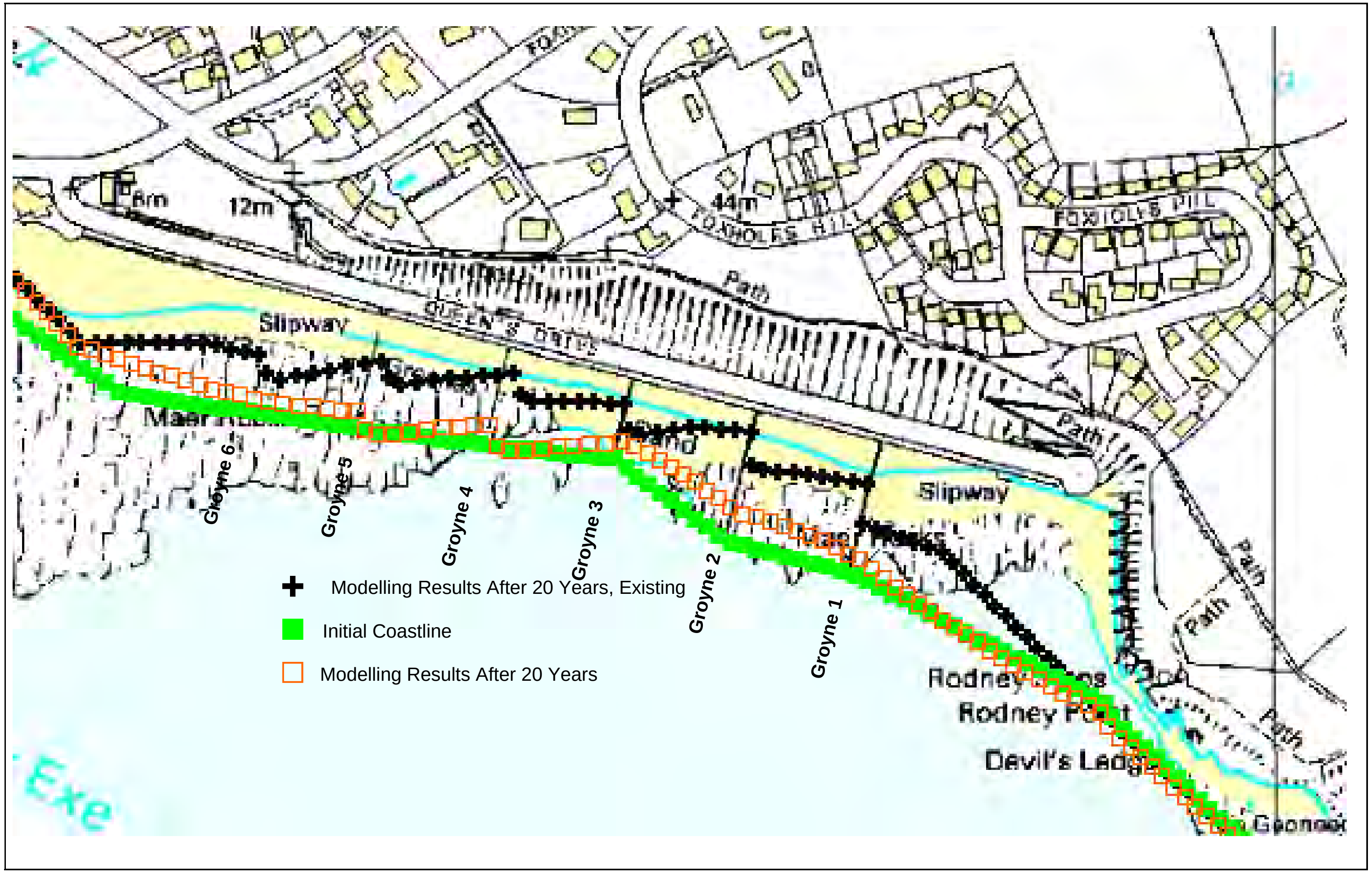


Figure 5-20: Zoomed in plot of Option E3 modelling results for Exmouth coast.

Exmouth Coastline Modelling Results

✚ Modelling Results After 20 Years, Existing

Option E4:
Existing Groynes 1 to 3
Recharge from groyne 1 to 3
Fully Functional Groyne 4 and 5
Add A New Groyne

■ Initial Coastline

□ Modelling Results After 20 Years

Figure 5-21: Option E4 modelling results for Exmouth coast.

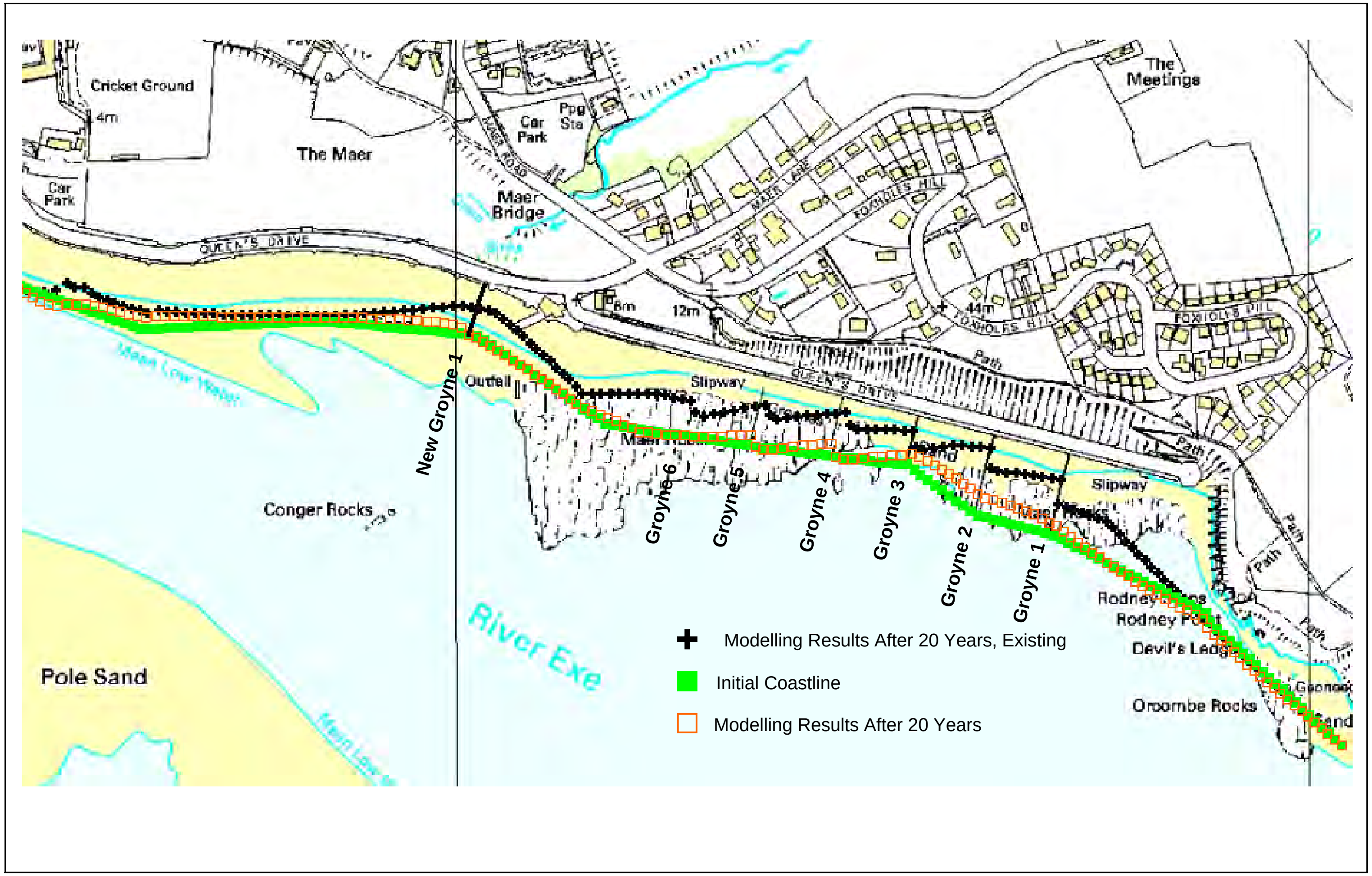


Figure 5-22: Zoomed in plot of Option E4 modelling results for Exmouth coast.

Exmouth Coastline Modelling Results

✚ Modelling Results After 20 Years, Existing

Option E5:
Existing Groynes 1 to 3
Recharge from groyne 1 to 3
Fully Functional Groyne 4 and 5
Add Two New Groynes

Initial Coastline

Modelling Results After 20 Years

Figure 5-23: Option E5 modelling results for Exmouth coast.

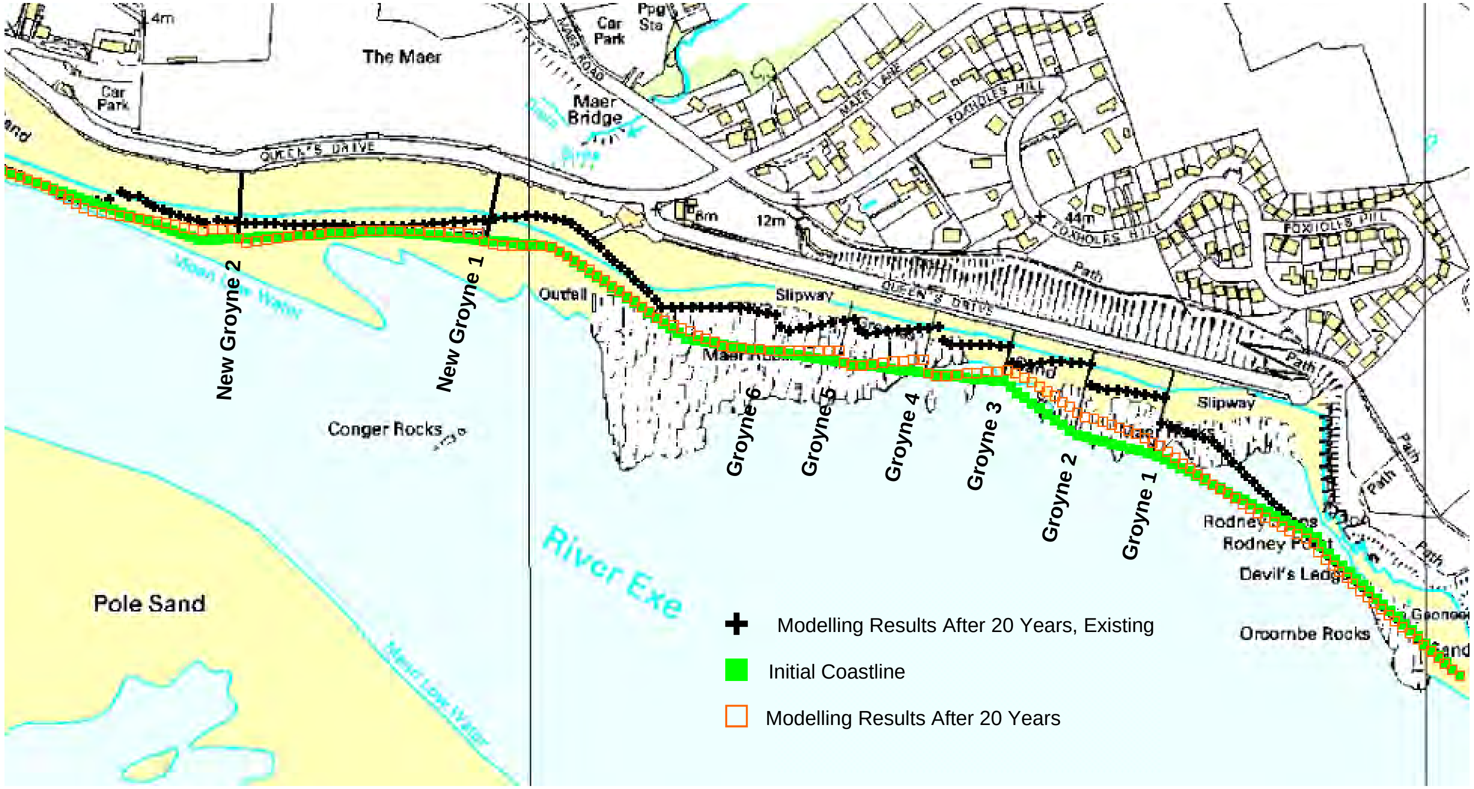


Figure 5-24: Zoomed in plot of Option E5 modelling results for Exmouth coast.

6 Cross-Shore Beach Profile Modelling

6.1 Introduction

In this section, the approach used to determine short-term shoreline changes due to cross-shore transport during storms is described, and the model results are discussed.

For extreme storm events (typically over durations of hours to days), breaking waves on the beach create offshore directed flow at the beach (so-called undertow), which carries significant sediment offshore during storm events. This cross-shore sediment transport typically results in the erosion of the upper part of beach profile or lowering of the beach levels. Halcrow's in-house cross-shore transport and profile evolution model COSMOS is used for modelling of historical beach profiles under simulated storm conditions to assess the resilience of the beach system. The model is used to study two beach profiles along the Exmouth coast and two profiles along the Dawlish Warren coast. The model is calibrated using observed pre and post storm beach profiles for the 23 October 2009, 3 March 2010 and 9 October 2010 events. The extra wave data covering a period of 25/11/2008 to 31/10/2011 have been obtained from the Met Office through the client. The calibrated model is then used to predict beach profile changes under different extreme wave and water level conditions for the 1 year average recurrence interval and 1 in 100 year return period events.

6.2 COSMOS Model

COSMOS-2D is a two-dimensional vertical plane sediment transport model, built to simulate the wave transformation and sediment transport along a cross-shore beach profile (i.e. normal to the shoreline). The model is formulated to include both suspended and bed sediment loads under the action of the oscillatory flow associated with breaking waves on a beach. Details of the model can be found in Nairn and Southgate (1993). The model assumes a straight coastline with parallel depth contours, and is intended for investigation of cross-shore beach stability under specified wave conditions. The COSMOS model includes the following physical processes:

- Wave transformation by refraction (by depth variations and currents), shoaling, Doppler shifting, bottom-friction and wave breaking. For random waves, the Battjes and Janssen (1978) theory is used for determining the distribution of wave height and the fraction of time that waves are breaking at any point.
- Wave set-up determined from the gradient of wave radiation stress.
- Driving forces for longshore wave-induced currents, determined directly from the spatial rate of wave energy dissipation.
- Long-shore currents from pressure-driven tidal forces and wave-induced forces, and the interaction between the two types of current.
- Cross-shore undertow velocities, using a three-layer model of the vertical distribution of cross-shore currents.
- Transition zone effects (the transition zone is the distance between where a wave starts to break and where turbulence becomes fully developed).

- Cross-shore and longshore sediment transport rates using an 'energetics' approach.
- Seabed level changes due to cross-shore sediment transport using a Lax-Wendroff scheme.
- Down-cutting of a cohesive profile due to abrasion by a covering layer of sand.

The initial cross-shore profile and time-series of wave height and direction are specified by the user, and the model determines wave, current and sediment transport parameters at each grid point. The model was developed jointly by Halcrow, HR Wallingford and Imperial College.

6.3 Model Verification

6.3.1 Beach Profile Data and Sediment Boundary Conditions

Cross-shore modelling has been undertaken for nearshore wave points 12 and 14 for Dawlish Warren, and nearshore wave points 6 and 8 for Exmouth Beach. The location of these points is shown in Figure 3-4.

For Dawlish Warren, beach profiles suitable for calibrating the COSMOS models were taken immediately before and after three storm events as shown in Table 6-1**Error! Reference source not found.**. No pre- and post-storm profiles are available for Exmouth Beach so the Exmouth model has been verified using the same storm events as for the Dawlish Warren, and a comparison has been made against the observed profiles which are closest in time. The dates of the profiles used at Exmouth are also shown in Table 6-1, along with the sediment size at the profiles.

Table 6-1: Summary of surveyed beach profile and sediment data.

Data for storm modelled using COSMOS					
Nearshore Point	**Sediment D50**	**Description**	**Storm 1**	**Storm 2**	**Storm 3**
6 (Exmouth)	0.54mm	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
		Pre-Storm Beach Profile	20/10/2009	1/2/2010	8/9/2010
		Post-Storm Beach Profile	1/2/2010	8/9/2010	7/11/2010
8 (Exmouth)	0.72mm	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
		Pre-Storm Beach Profile	20/10/2009	1/2/2010	8/9/2010
		Post-Storm Beach Profile	1/2/2010	8/9/2010	7/11/2010
12 (Dawlish)	0.90mm	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
		Pre-Storm Beach Profile	19/10/2009	2/3/2010	12/9/2010
		Post-Storm Beach Profile	23/10/2009	6/3/2010	14/10/2010
14 (Dawlish)	1.60mm	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
		Pre-Storm Beach Profile	19/10/2009	2/3/2010	12/9/2010
		Post-Storm Beach Profile	23/10/2009	6/3/2010	14/10/2010

6.3.2 Wave and Water Level Boundary Conditions

Inshore time-series of wave and water level data are required for setting up the COSMOS model. The inshore wave data were obtained from the wave transformation modelling as described in 3.4 for the extra wave data covering a period of 25/11/2008 to 31/10/2011 obtained from the Met Office through the client. These inshore wave data are then combined with the measured water level data at Exmouth Dock and used as the input wave and water level data for COSMOS model. It is assumed that each storm studied would last four days, and the time periods simulated for those three storms are shown in Table 6-2. All storm conditions of wave and water level are listed in Appendix B.

Table 6-2: Summary of model simulation period for each storm.

Wave and water level data used for storm modelling				
Nearshore	**Description**	**Storm 1**	**Storm 2**	**Storm 3**
6 (Exmouth)	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
	Start of simulation	20/10/2009	2/3/2010	7/10/2010
	End of simulation	23/10/2009	5/3/2010	10/10/2010
8 (Exmouth)	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
	Start of simulation	20/10/2009	2/3/2010	7/10/2010
	End of simulation	23/10/2009	5/3/2010	10/10/2010
12 (Dawlish)	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
	Start of simulation	20/10/2009	2/3/2010	7/10/2010
	End of simulation	23/10/2009	5/3/2010	10/10/2010
14 (Dawlish)	Storm Modelled	23/10/2009	3/3/2010	9/10/2010
	Start of simulation	20/10/2009	2/3/2010	7/10/2010
	End of simulation	23/10/2009	5/3/2010	10/10/2010

6.3.3 Model Parameters

A summary of the key parameters and their adopted values, as used in the model, are summarised in Table 6-3 for the beach profiles at nearshore points 6, 8, 12 and 14,. The default parameter settings are also listed for comparison. A full description of each parameter is given in Nairn and Southgate (1993).

Table 6-3: COSMOS Model Parameters for cross-shore modelling at nearshore points 6, 8, 12 and 14.

Parameter	Description	Default	Nearshore Point			
			6	8	12	14
rho	the density of the fluid in kg/m^3	1030	1030	1030	1030	1030
brcoef	coefficient for dissipation of broken wave energy	1	1	1	1	1
bbb	shallow water breaking criterion	-1	-1	-1	-1	-1
dfrac	wave breaking water depth	0.6	0.6	0.6	0.6	0.6
irebr	flag for re-breaking of random waves	1	1	1	1	1
ihs	flag for input and output of RMS or significant wave heights	0	0	0	0	0
datum	depth datum used within the model	0	0	0	0	0
sllen	distance of over which the seabed slope is calculated	5	5	5	5	5
smlen	distance over which the radiation stress is calculated	5	5	5	5	5
yton	angle, in degrees, from North to the y-axis	0	0	0	0	0
maxd	number of grid points	none	58	50	89	65
sedsiz	sediment size read in as constant across the profile.	none	540	720	900	1600
rninit	initial seabed roughness height in m	0.016	0.016	0.016	0.016	0.016
rhosed	density of the sediment, assumed constant, in $kglm^3$	none	2650	2650	2650	2650
maxds	grid points at which the sediment grain size is specified	none	58	50	89	65
itz	switches on transition zone energy dissipation if = 1	1	1	1	1	1
ubarO	constant (i.e. non-sinusoidal) part of the offshore velocity, in rn/s	0	0	0	0	0
cfO	the current and sea bed friction factor at an offshore point	0.0065	0.0065	0.0065	0.0065	0.0065
ho	offshore depth at which cfO and ubarO are specified	dep(1)	10	10	10	10

6.3.4 Cross-Shore Model Verification - Dawlish Warren

Model results at nearshore point 12 are presented for the three storm events in Figure 6-1, Figure 6-2 and Figure 6-3 together with the observed pre- and post-storm profiles. Observed and modelled erosion depths between the pre- and post-storm profiles are small for all three storms. Figure 6-1 shows there was net accretion between 2 mAOD and 3 mAOD during the storm event of 23/10/2009 which is reproduced well by the model. For the storm of 3/3/2010, Figure 6-2 shows little difference between the measured pre-storm and post-storm profiles, which is again reproduced by the model. Figure 6-3 shows net beach creation during the storm event of 9/10/2010. This is not reproduced by the model which assumes that sediment along the beach profile in conserved. For this storm event there has been a net sediment input to the beach profile and it may be caused by the longshore sediment transport.

Model results for the three storm events at nearshore point 14 are shown in Figure 6-4, Figure 6-5 and Figure 6-6 together with the observed pre and post-storm profiles. Figure 6-4 and Figure 6-5 show little change to the measured beach profiles during the storms of 23/10/2009 and 3/3/2010, and this is replicated by the model. Figure 6-6 shows that for the storm of 9/10/2010 there was net accretion over the entire profile. This net gain of sands is not reproduced by the model which assumes sediment along the beach profile is conserved. It is possible that the net gain of sands is from the longshore sediment transport.

In summary, the COSMOS modelling results of the beach responses are in reasonable agreement with the surveyed data when considering the Dawlish Warren coast.

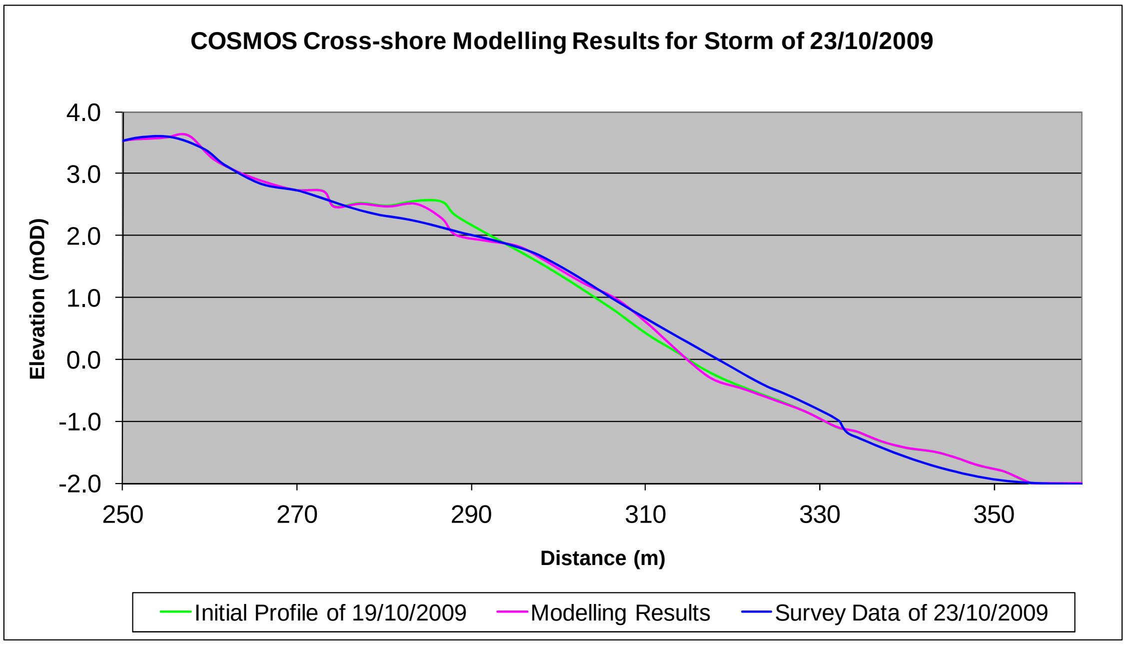


Figure 6-1: Dawlish Warren cross-shore modelling results for profile at inshore point 12.

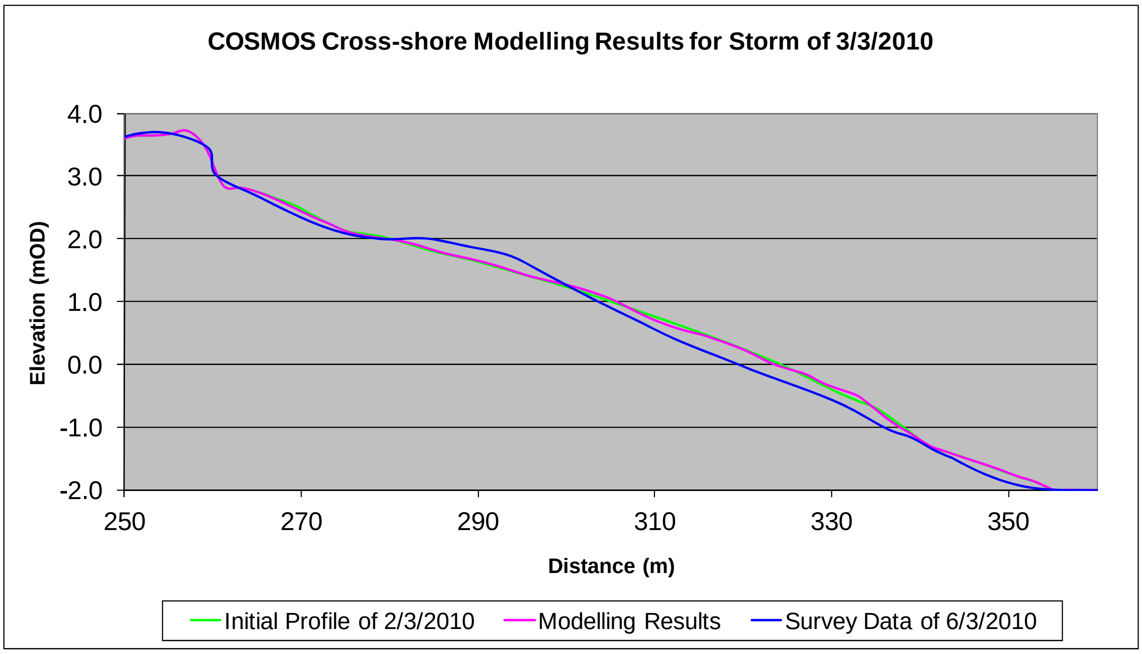


Figure 6-2: Dawlish Warren cross-shore modelling results for profile at inshore point 12.

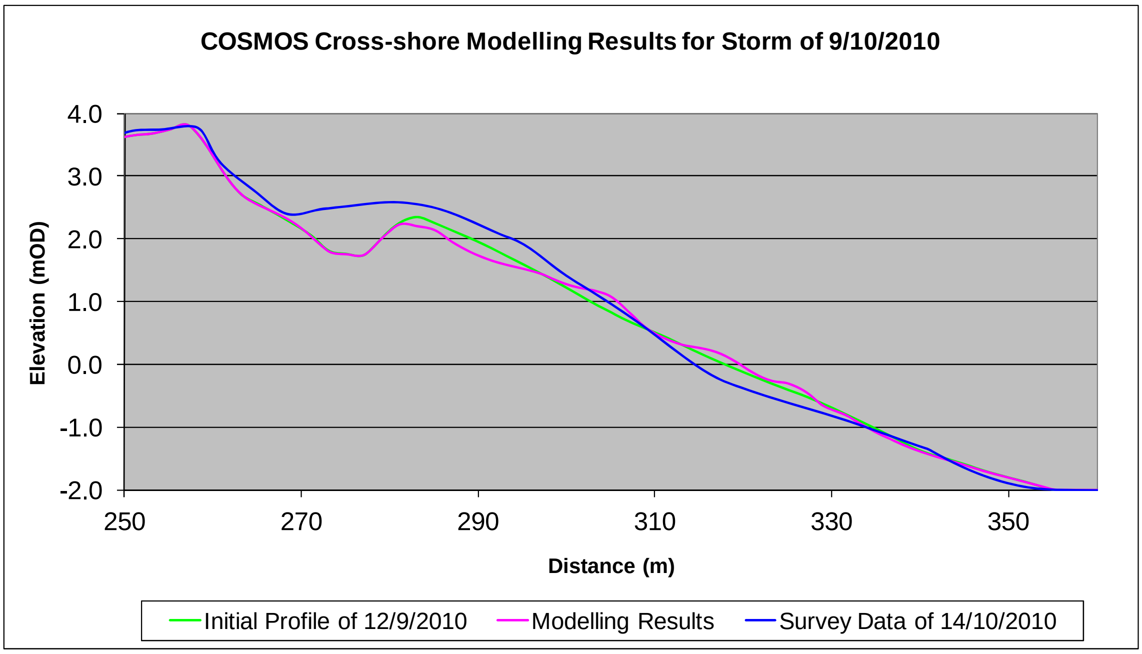


Figure 6-3: Dawlish Warren cross-shore modelling results for profile at inshore point 12.

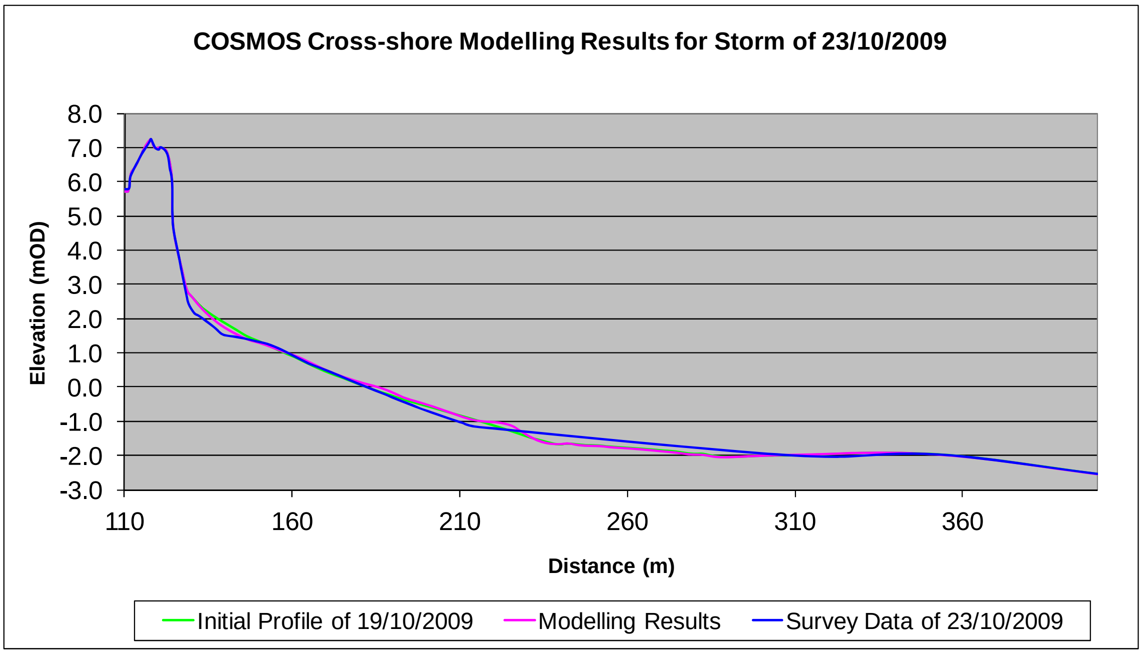


Figure 6-4: Dawlish Warren cross-shore modelling results for profile at inshore point 14.

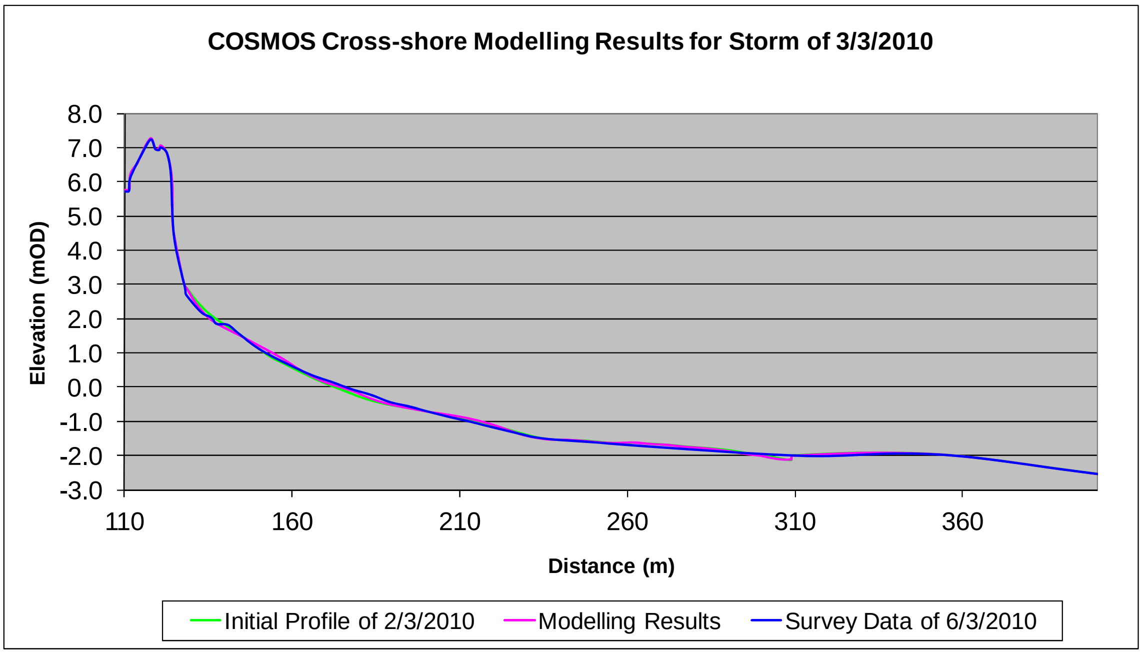


Figure 6-5: Dawlish Warren cross-shore modelling results for profile at inshore point 14.

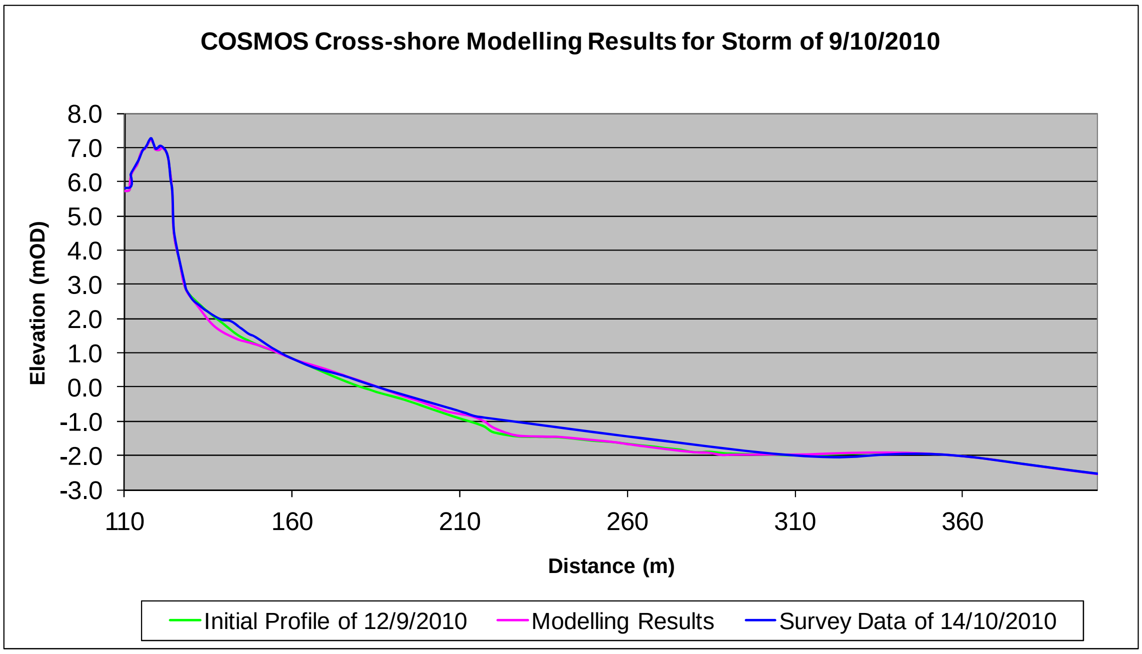


Figure 6-6: Dawlish Warren cross-shore modelling results for profile at inshore point 14.

6.3.5 Cross-Shore Model Verification – Exmouth Beach

For the Exmouth coast, there is only one of surveyed beach profile data available just before the storm of 23/10/2009. No other pre- and post-storm beach profile data close to the storm periods for model calibration. Thus the COSMOS model is used for this case to study the potential beach changes during identified storms of 23/10/2009, 3/3/2010 and 9/10/2010. For comparison, the surveyed beach profile at the nearest observed time after the identified storm is also plotted. This is for illustration only and not for the model calibration.

The modelling results for cross-shore profile at nearshore point 6 reveal that there is no substantial erosion and/or accretion caused by all three storms. The maximum erosion of less than 0.4m occurred during the storm of 9/10/2010, which is shown in Figure 6-9. The model results are close to the surveyed data of 7/11/2010 a month later than the storm. The gap between the time of surveyed data and the storm is too large to judge the modelling accuracy.

Considering the beach profile at nearshore point 8, the modelling results indicate that there is little change of the beach during the storms of 23/10/2009, 3/3/2010 and 9/10/2010, which can be seen in Figure 6-10, 6-11 and 6-12, respectively. Again the post-storm profile data are plotted for illustration only as the survey times were not close to the storm periods.

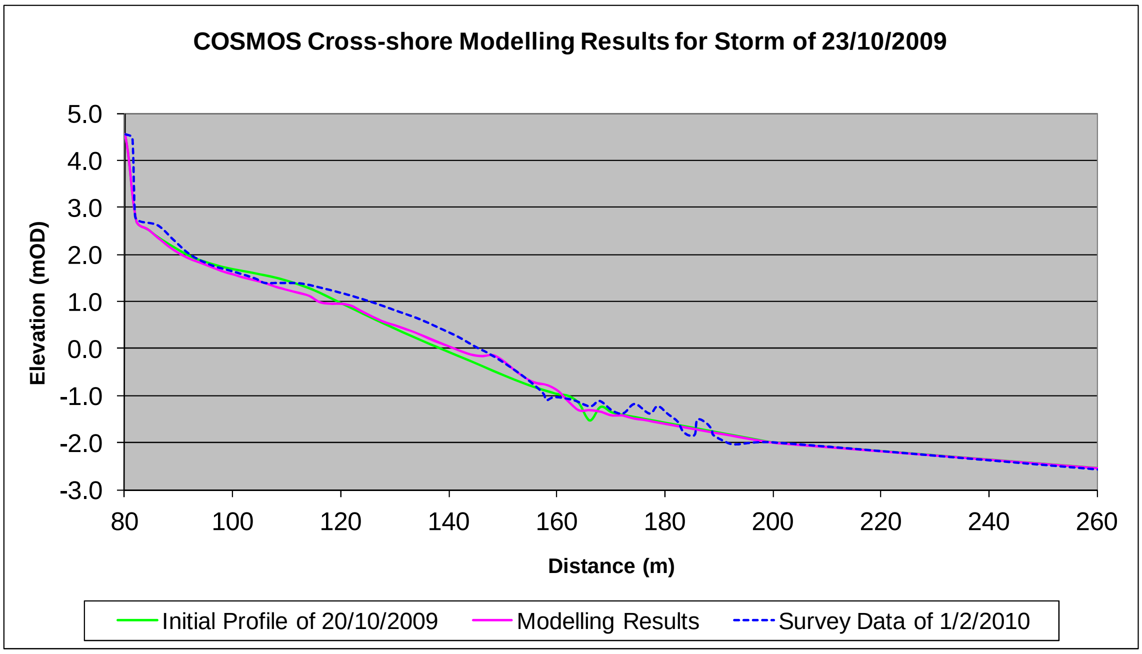


Figure 6-7: Exmouth cross-shore modelling results for profile at inshore point 6.

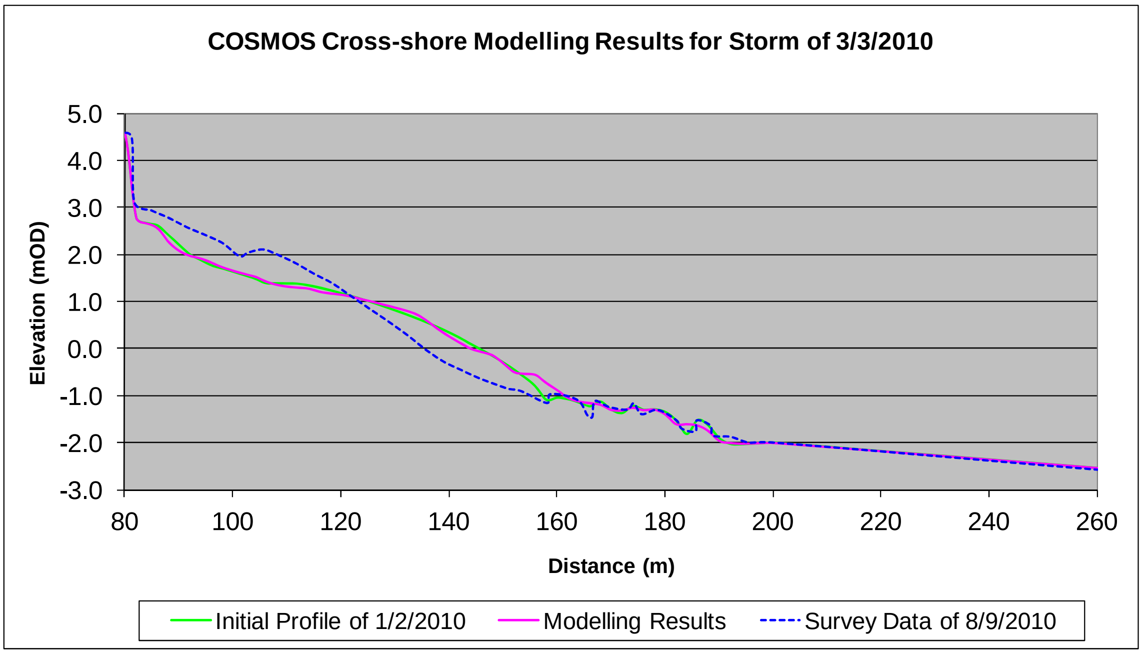


Figure 6-8: Exmouth cross-shore modelling results for profile at inshore point 6.

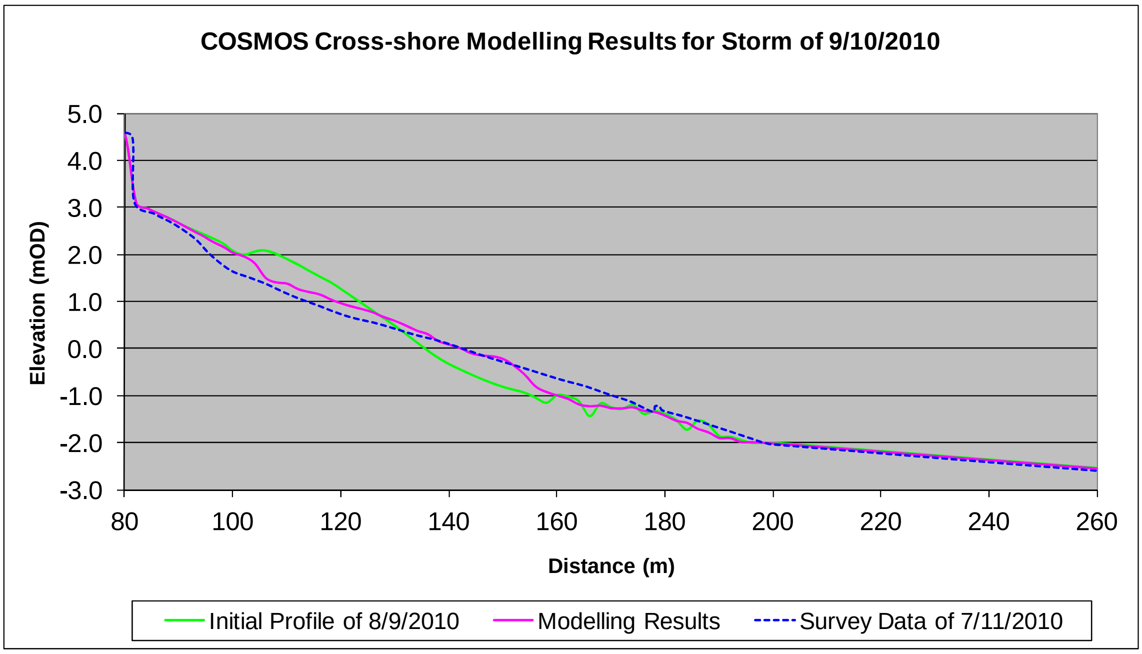


Figure 6-9: Exmouth cross-shore modelling results for profile at inshore point 6.

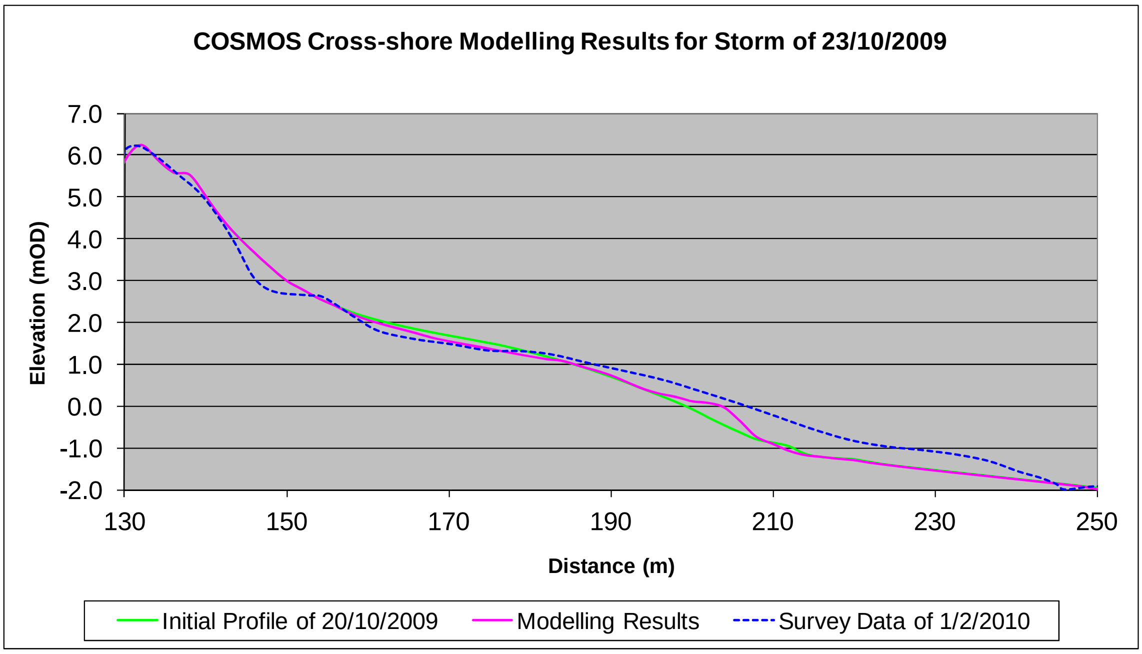


Figure 6-10: Exmouth cross-shore modelling results for profile at inshore point 8.

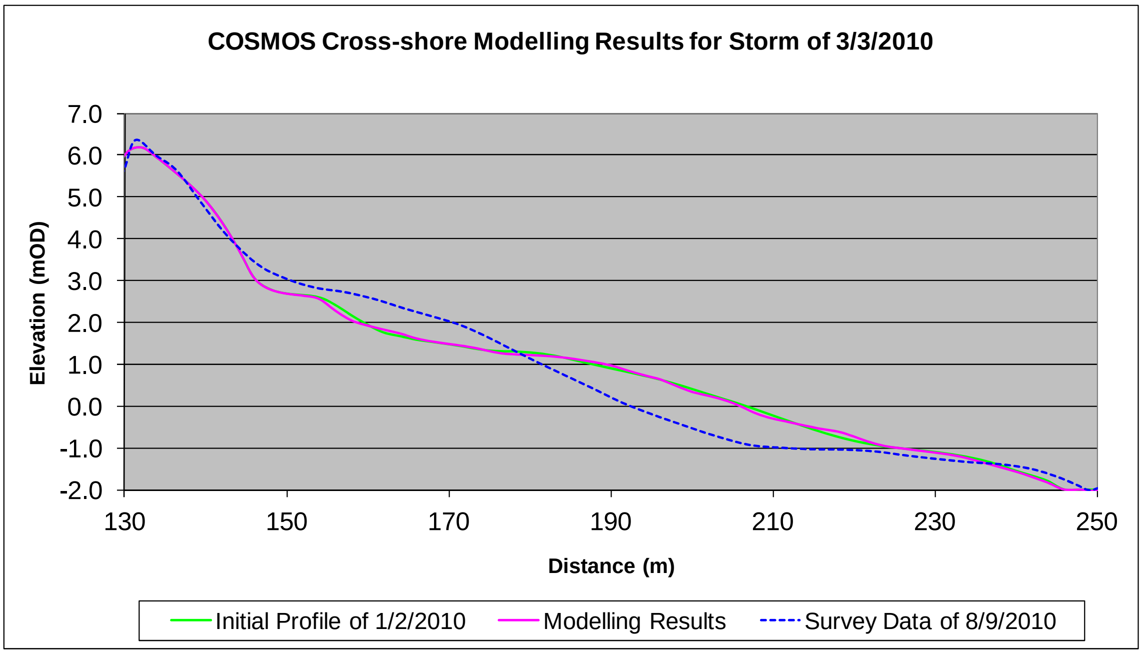


Figure 6-11: Exmouth cross-shore modelling results for profile at inshore point 8.

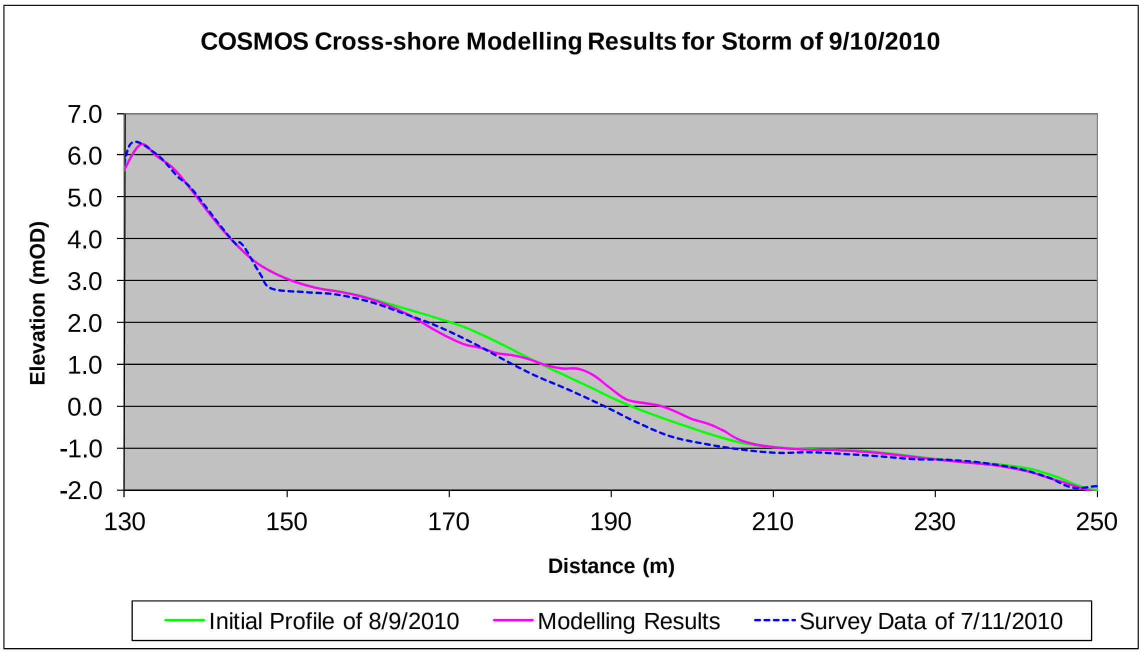


Figure 6-12: Exmouth cross-shore modelling results for profile at inshore point 8.

6.4 Cross-Shore Modelling for Extreme Conditions

6.4.1 Wave and Water Level Boundary Conditions

For the four beach profiles at the nearshore points, i.e. nearshore point 6, 8, 12 and 14, the joint probability analysis results reported in Section 4.3 have been used as input data for the COSMOS model. The joint extreme conditions of wave and water levels are presented in Table 6-5: Extreme wave and water level conditions of 1 in 100 year return period.for an average recurrence interval of 1 year, and in for a return period of 1 in 100 years. These extreme wave and water level conditions are then used to generate synthetic storms as input data for the cross-shore sediment modelling.

Table 6-4: Extreme wave and water level conditions of 1 in 1 year return period.

Nearshore Point	Joint extreme values of water levels and wave heights 1 in 1 average recurrence interval					
		Condition 1	Condition 2	Condition 3	Condition 4	Condition 5
6	WL(m)	1.5	2.2	2.4	2.73	2.83
	Hmo(m)	1.67	1.63	1.57	1.4	1.2
8	WL(m)	1.5	2.0	2.53	2.84	2.92
	Hmo(m)	1.26	1.26	1.2	1.0	0.8
12	WL(m)	1.5	2.0	2.5	2.7	2.85
	Hmo(m)	1.12	1.11	1.04	0.95	0.8
14	WL(m)	1.5	2.5	2.78	2.83	2.87
	Hmo(m)	1.8	1.68	1.3	1.1	0.8

Table 6-5: Extreme wave and water level conditions of 1 in 100 year return period.

Nearshore Point	Joint extreme values of water levels and wave heights 1 in 100 year return period					
		Condition 1	Condition 2	Condition 3	Condition 4	Condition 5
6	WL(m)	1.5	2.2	2.8	3.59	3.89
	Hmo(m)	1.87	1.84	1.81	1.6	1.4
8	WL(m)	2.0	2.5	2.9	3.5	3.79
	Hmo(m)	1.41	1..39	1.35	1.28	1.0
12	WL(m)	2.0	2.7	3.1	3.62	3.75
	Hmo(m)	1.27	1.24	1.22	1.0	0.8
14	WL(m)	2.5	2.9	3.5	3.64	3.65
	Hmo(m)	2.2	2.07	1.79	1.3	1.1

The idealised storm profile is obtained from analysing the 22 year nearshore wave time-series from the wave transformation modelling work in Section 3.4. The top ten largest storms associated with the top ten highest waves during the 22 years have been identified and are plotted in Figure 6-13 to Figure 6-16 for the four nearshore wave locations. The plotted wave heights are relative wave height ratios. The relative wave height ratio is defined as the real wave height is divided by the peak wave height during a storm. The idealised storm shape of wave heights has then obtained by averaging the data from the top 10 highest waves. The wave height plots indicate that the water level has a significant effect on the wave height. At low water levels, wave refraction is greater due to the lower water depths. Wave breaking also reduces wave height when waves propagate from offshore towards inshore.

The storm shape is used to generate a time-series of wave heights associated with a given extreme wave condition for each nearshore location.

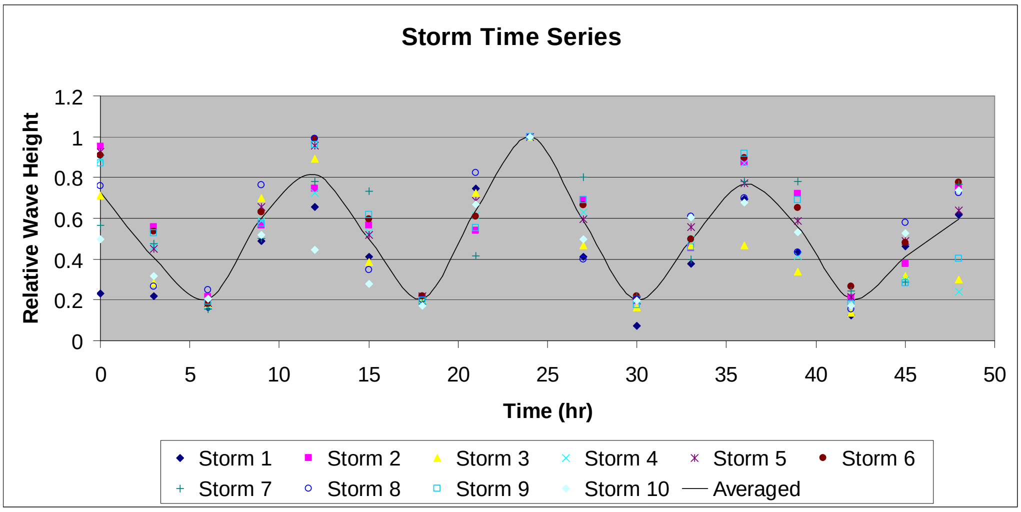


Figure 6-13: Wave shape from the top ten highest waves of 22 years for point 6.

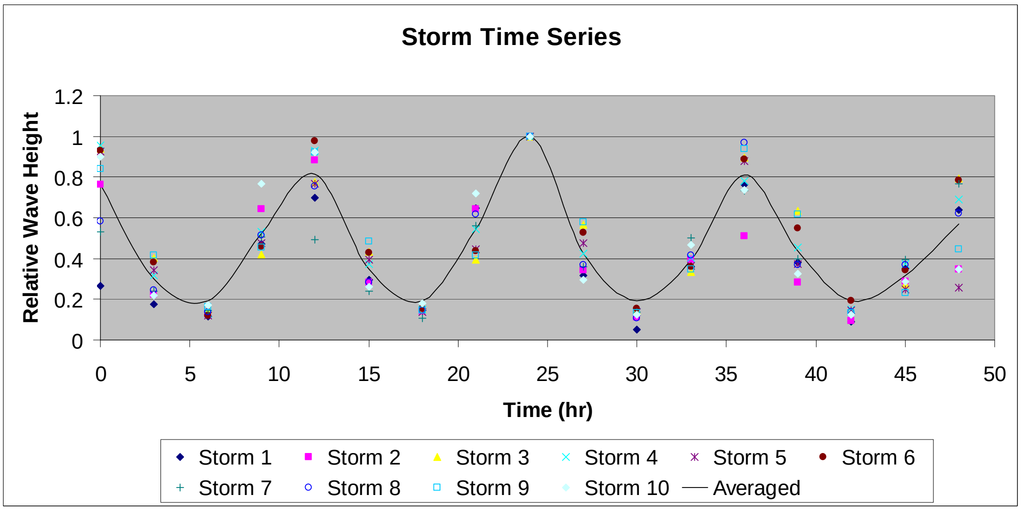


Figure 6-14: Wave shape from the top ten highest waves of 22 years for point 8.

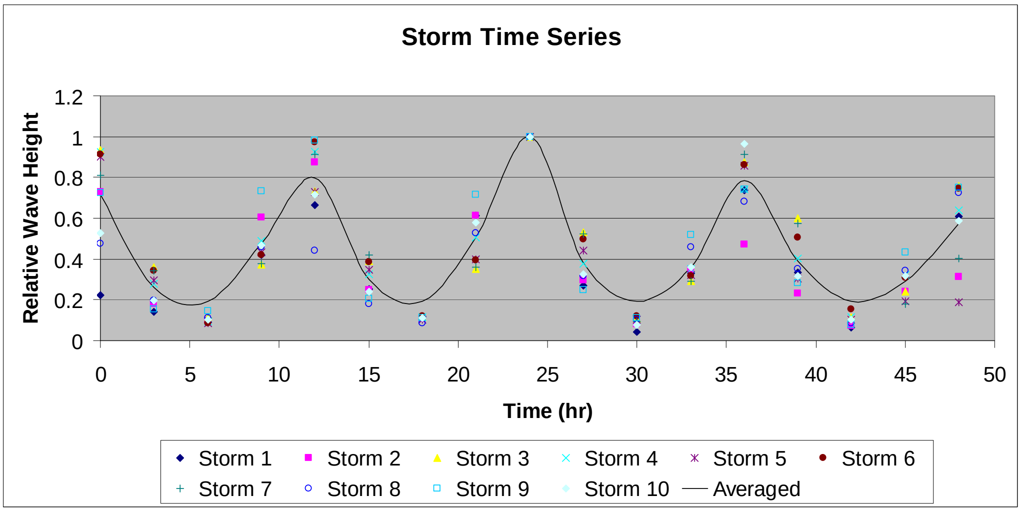


Figure 6-15: Wave shape from the top ten highest waves of 22 years for point 12.

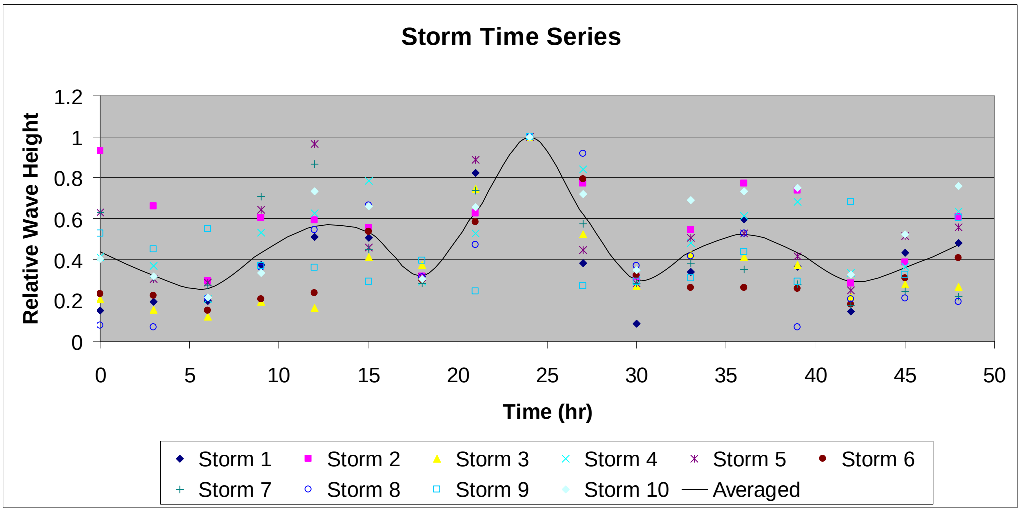


Figure 6-16: Wave shape from the top ten highest waves of 22 years for point 14.

Measured water level data is available for Exmouth Dock from 10/11/2000 to 31/12/2009. Storms associated with high water levels have been identified and analysed. The records for many severe storms are incomplete with long periods of missing data and the eight storms with the highest still water level and complete records have been identified.

The results are shown in Figure 6-17 together with the average water level. It is reasonable to use the averaged storm curve as the representative storm profile shape for a given extreme water level so that a time-series of water level data can be generated as input data for the cross-shore sediment modelling.

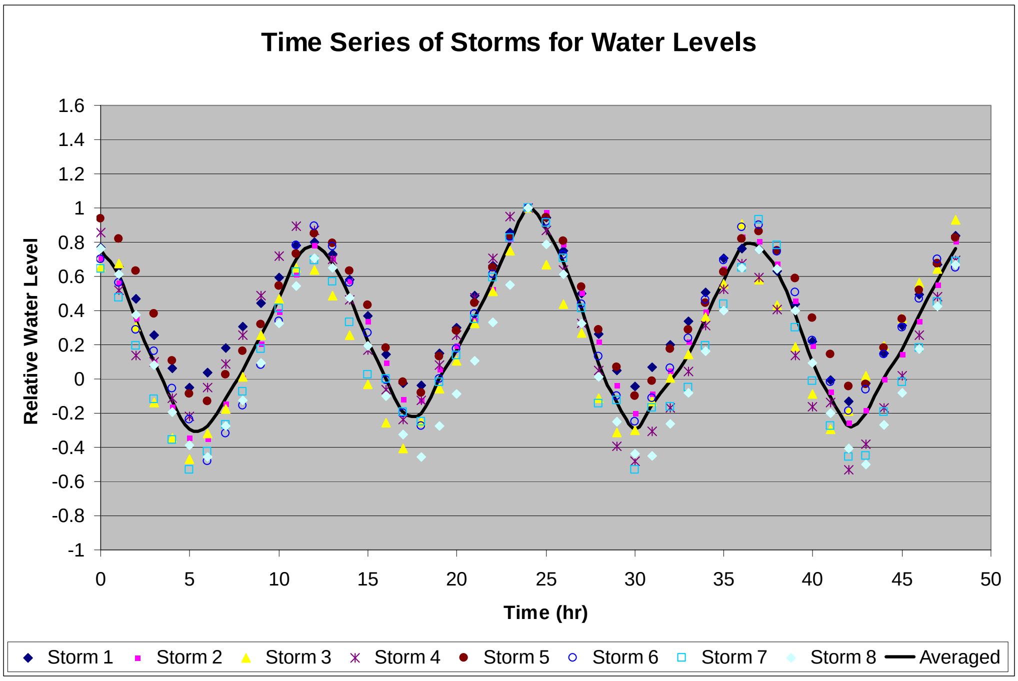


Figure 6-17: Water level shape from the top eight highest water levels of 10 years.

6.4.2 Cross-shore Modelling Results for the Dawlish Warren Coast

The wave and water level time-series described in Section 6.4.1 are used as input data for COSMOS model. The initial beach profile and modelled beach profiles under extreme wave and water level conditions for the 1 year ARI and 100 year return period events are presented in Figure 6-18 and Figure 6-19 for nearshore point 12, and Figure 6-20 and Figure 6-21 for nearshore point 14.

For nearshore point 12, the results show little erosion at the upper beach for the 100 year return period event as the beach slope is small and the wave energy is low. For the nearshore point 14, the results show that for a 1 year ARI event, the maximum beach retreat distance is 4.3 m and the maximum eroded depth is 0.23 m. For a 100 year return period event, the beach retreat distance is 6.7 m, and the maximum depth of erosion 0.51m. The model shows that offshore bars are generated under all conditions for a 1 year ARI and 1 in 100 year return period events.

The cross-shore profile at point 14 is more active than at point 12, due to the greater wave energy at this location; Table 6-4 and Tale 6-5 shows that wave heights at point 14 are generally higher than wave heights at point 12.

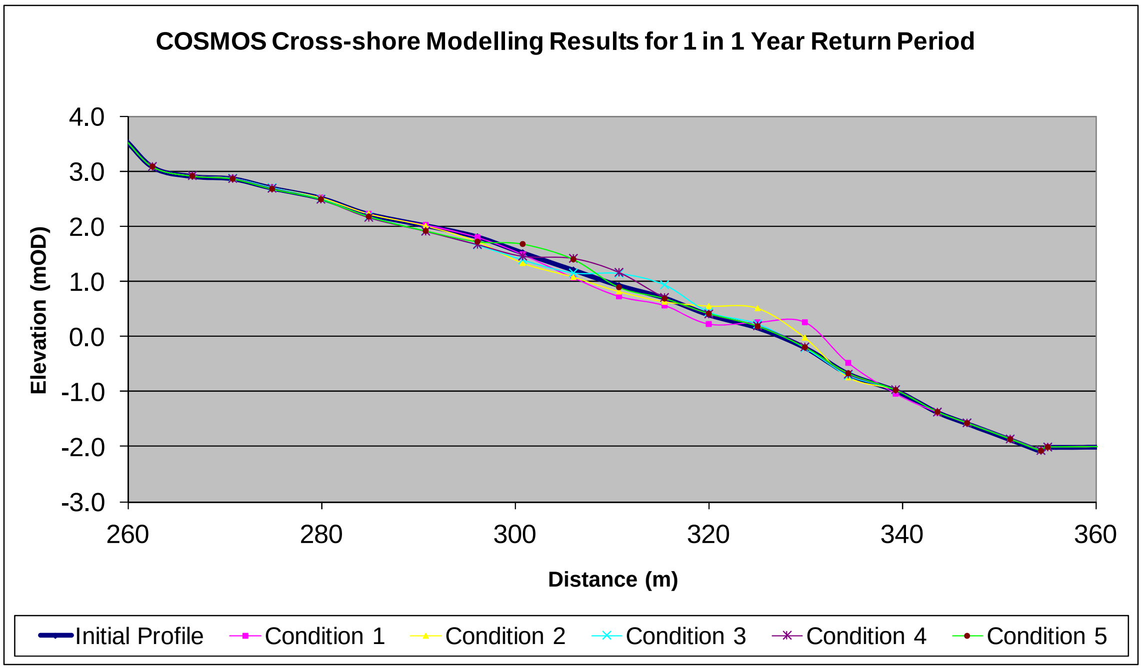


Figure 6-18: Dawlish Warren modelling results for profile at inshore point 12 for 1 in 1 year return period.

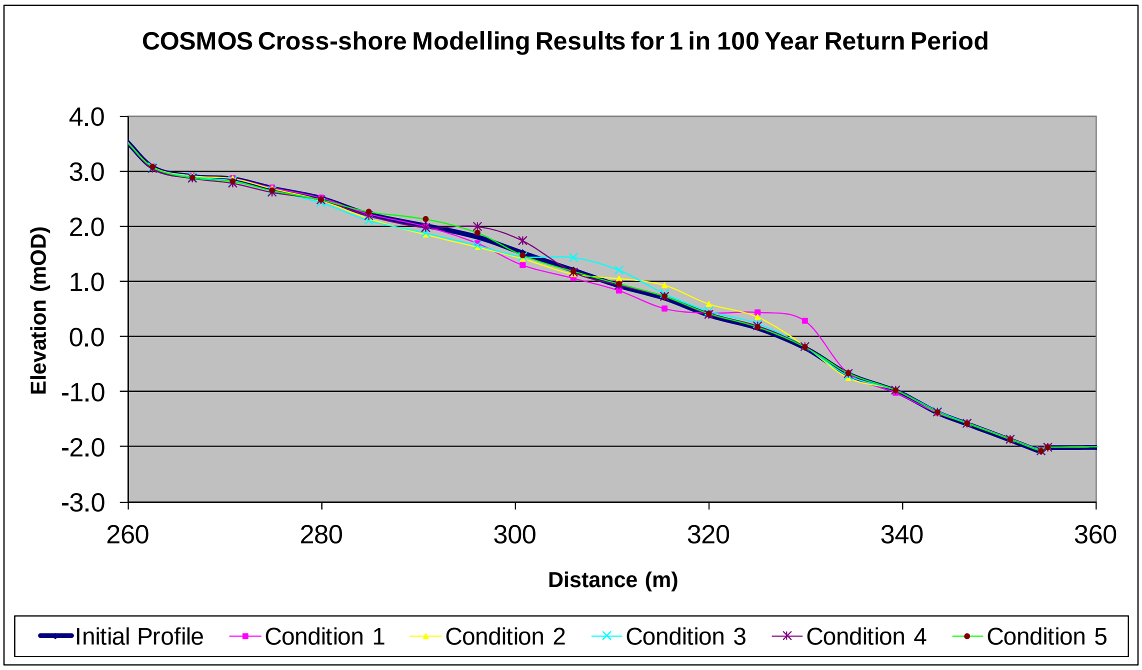


Figure 6-19: Dawlish Warren modelling results for profile at inshore point 12 for 1 in 100 years return period.

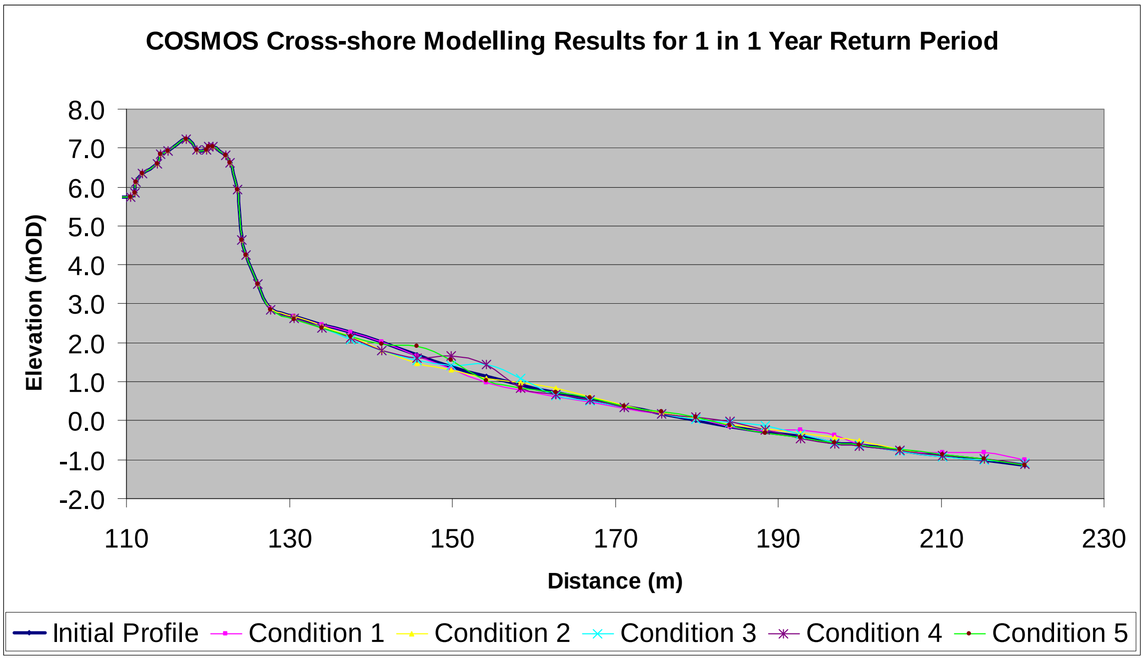


Figure 6-20: Dawlish Warren modelling results for profile at inshore point 14 for 1 in 1 year return period.

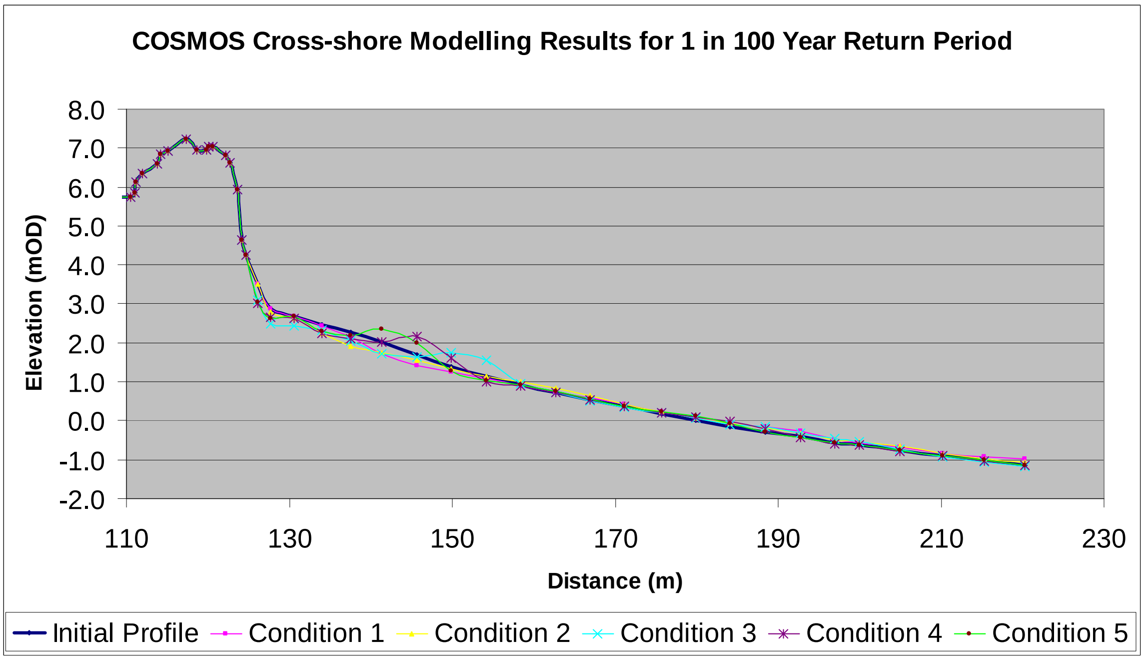


Figure 6-21: Dawlish Warren modelling results for profile at inshore point 14 for 1 in 100 years return period

6.4.3 Cross-Shore Modelling Results for the Exmouth Coast

The wave and water level time-series described in section 6.4.1 are used as input data for the COSMOS model. The initial beach profile and modelled beach profiles for 1 year ARI and 100 year return period events are presented in Figure 6-22 and Figure 6-23 for nearshore point 6, and Figure 6-24 and Figure 6-25 for nearshore point 8.

For nearshore point 6, Figure 6-22 shows that there is no significant erosion of the upper beach for a 1 year ARI event. For the 1 in 100 year return period event, Figure 6-23 shows that the beach retreat distance 18.5m, and the maximum depth of erosion is 1.55 m in front of the sea wall.

For nearshore point 8, Figure 6-24 shows that the maximum beach retreat distance is 6.5 m and the maximum eroded depth is 0.49 m for a 1 year ARI event. For a 100 year return period event, Figure 6-25 shows that the maximum beach retreat distance is 9.7m, and the maximum depth of erosion is 0.60 m. Offshore bars are generated in the model for all joint wave and water level conditions modelled.

In general wave heights at point 6 are higher than wave heights at point 8. But the beach slope at the point 6 is 1 in 28, compared to 1 in 12 at point 8. Thus the beach response to the extreme storms at both points is the same order of magnitude.

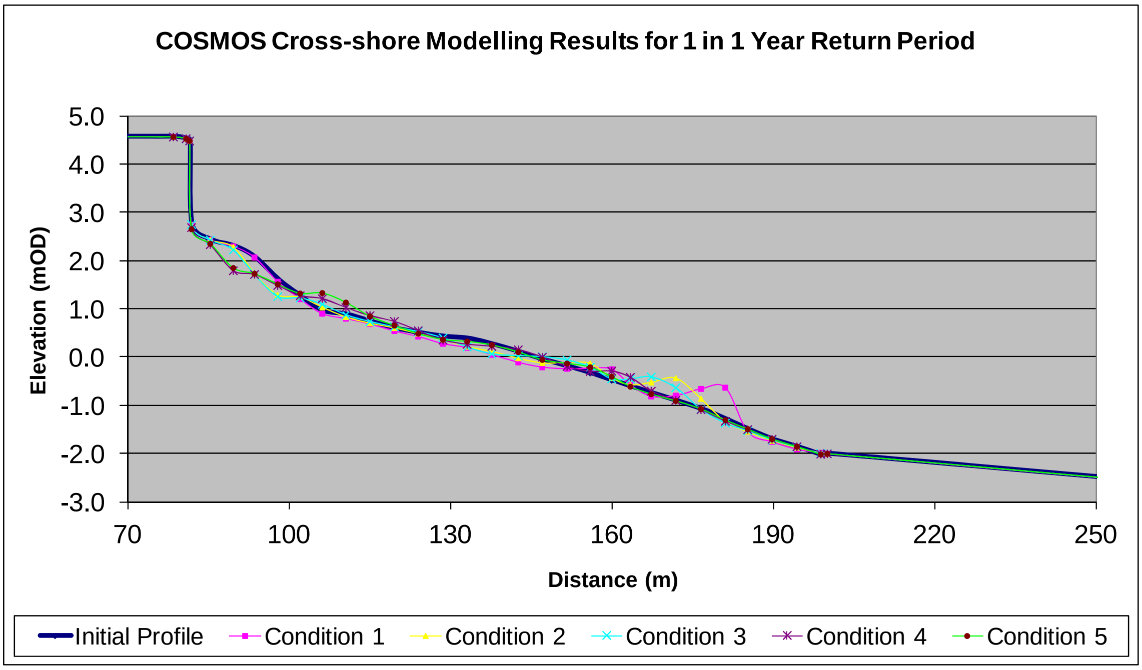


Figure 6-22: Exmouth modelling results for profile at inshore point 6 for 1 in 1 year return period.

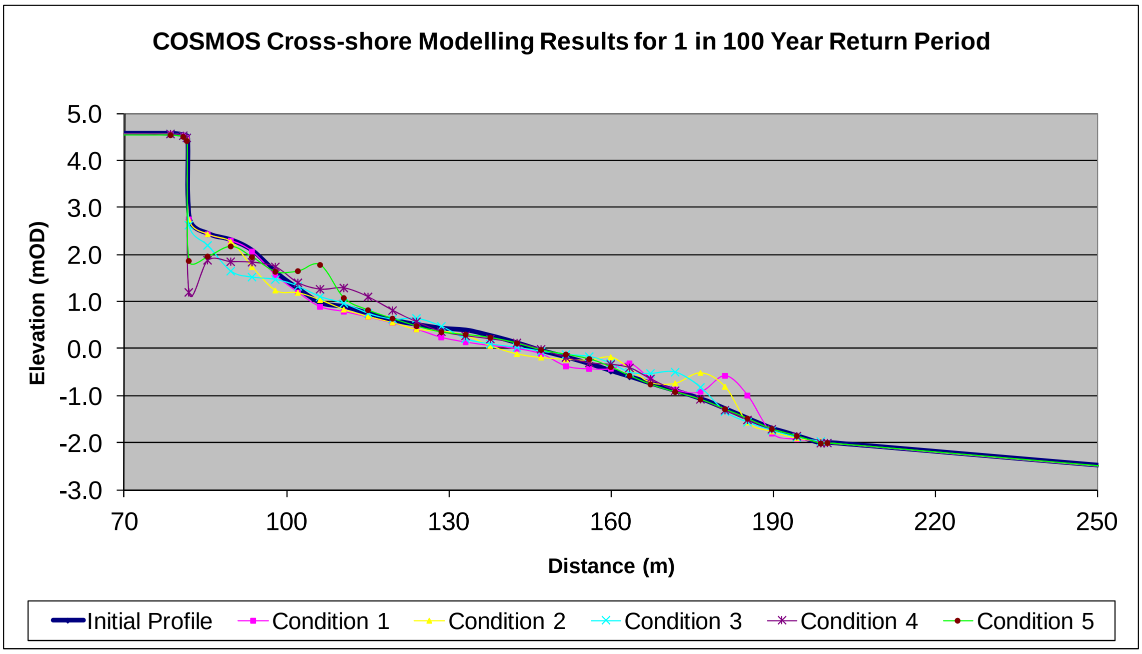


Figure 6-23: Exmouth modelling results for profile at inshore point 6 for 1 in 100 years return period.

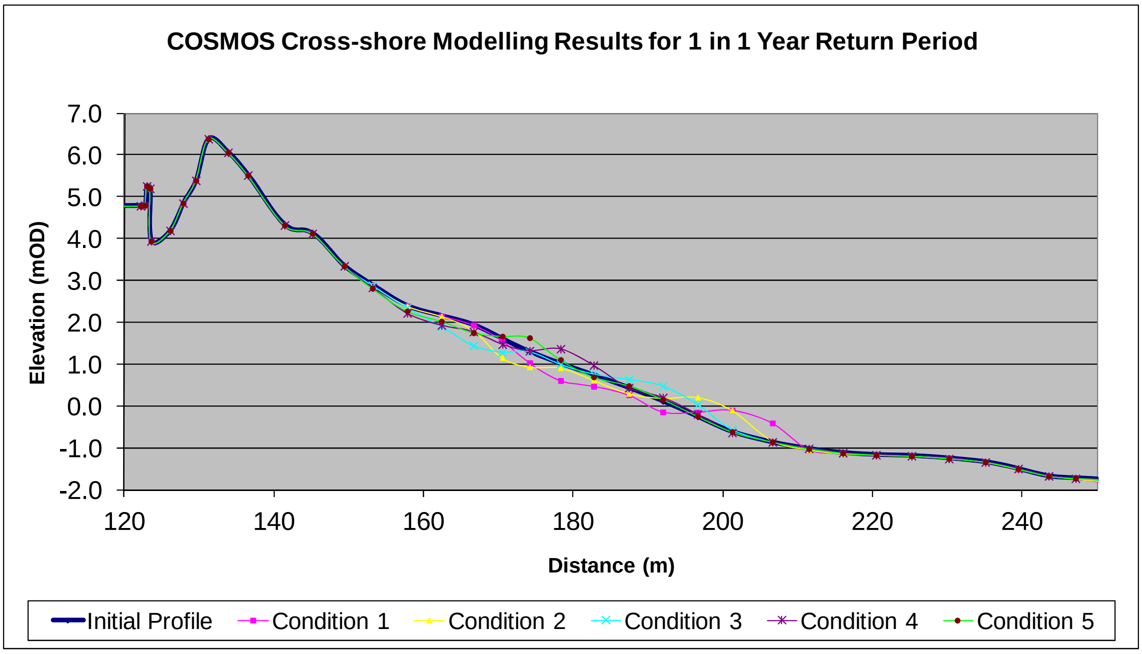


Figure 6-24: Exmouth modelling results for profile at inshore point 8 for 1 in 1 year return period.

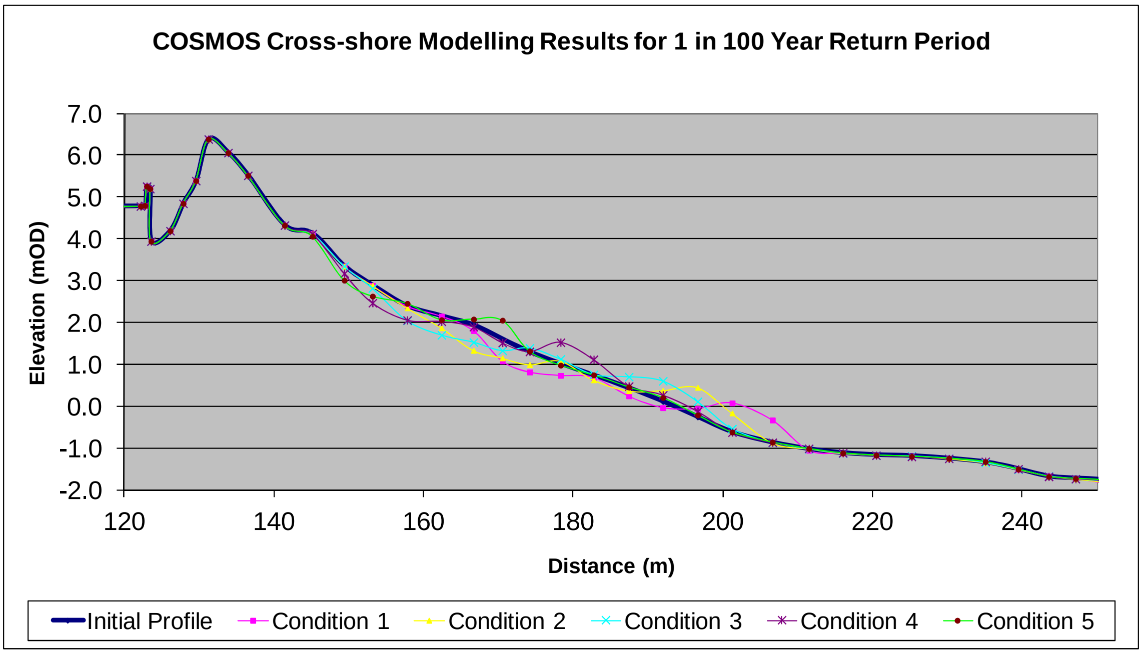


Figure 6-25: Exmouth modelling results for profile at inshore point 8 for 1 in 100 years return period.

6.5 Cross-Shore Modelling for Dawlish Warren - SHINGLE Model

The COSMOS modelling presented in Section 6.4.2 shows that the changes to cross-shore beach profiles during storms are not significant at nearshore points 12 and 14 along Dawlish Warren. However, Figure 6-26 shows that at some locations along Dawlish Warren there was significant erosion to the upper beach during a storm event in October 2010. Erosion to the dune crest cannot be modelled using COSMOS because COSMOS assumes that waves break when they propagate towards the shallow water region, and there is not enough wave energy reaching the dune crest level. Instead Halcrow's SHINGLE modelling software is used to investigate the potential erosion of the coarser grained upper section of beach along Dawlish Warren which, if eroded would in turn lead to undermining and erosion of the dune crest under extreme wave and water level conditions.

Figure 6-26: Erosion of the Dawlish dunes observed after the severe storm event on 9/10/2010.

Halcrow's SHINGLE model is a two-dimensional parametric shingle beach model based on equations derived by HR Wallingford (HR Wallingford, 1990). SHINGLE relates the development of various features of shingle beaches directly to the incident wave conditions and beach material characteristics, instead of the underlying physical processes.

SHINGLE is used to investigate the potential erosion of the coarser grained upper section of beach along Dawlish Warren (i.e. the toe of the sand dunes) which, if eroded, can be inferred to lead to undermining and erosion of the Dawlish Warren dune crest at nearshore point 14 for 1 year ARI and 100 year and 500 year return period events. The beach section at the nearshore point 14 is covered with shingles and gravels and is therefore suitable for study using the SHINGLE model. It is intended to investigate the mechanism of the sand dune erosion under very severe weather conditions. Results are shown in Figure 6-27 together with the initial measured beach profile. The model shows no significant change of the beach for the 1 year ARI event which can be compared to the COSMOS model results which showed maximum erosion of 0.23 m for this event. In the 100 year return period event the dune crest is eroded by 1 m and in the 500 year return period event the dune crest is eroded by and 2 m. It should, however, be noted that the model does not take into account the protective function of the gabion mattresses and as such, actual erosion is likely to be less. A single severe storm is therefore unlikely to cause a breach of Dawlish Warren such as that observed in the 1940s, and a number of extreme storms following a long period of net erosion to the beach would be required.

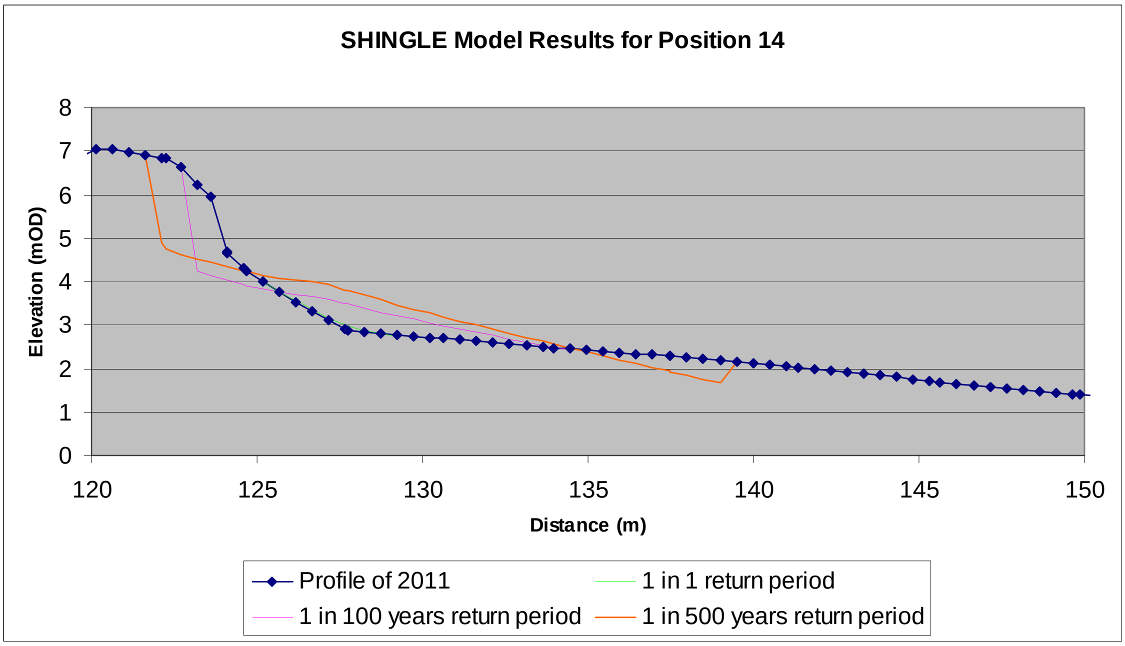


Figure 6-27: SHINGLE modelling results for Dawlish Warren.

7 Two-Dimensional Sediment Transport Modelling

A coastal area model has been set up to model the typical annual sediment transport regime around the mouth of the Exe Estuary, to aid understanding of the present day sediment transport regime in this area. The model is a two dimensional model and includes the exchange of sediment between the estuary, Dawlish Warren and Exmouth Beach and the ebb and flood deltas. The modelling looks at the annual average sediment transport, around Dawlish Warren and Exmouth beaches, but does not explicitly model changes in the bed level. The modelling is therefore complementary to the COASTLINE modelling (Section 5) which models the continuous evolution of Dawlish Warren and Exmouth beaches over a period of 20 years, but did not include the two dimensional flow patterns, which are important in this area and thus investigated in this section. It is therefore important to understand not only the long term evolution of Dawlish Warren and Exmouth Beach due to local longshore and cross-shore transport but also the 2D interactions of the ebb and flood deltas with Exmouth and Dawlish Warren beaches.

The model has been set up in MIKE 21, commercial software developed by DHI in Denmark, and consists of three components; a hydrodynamic model of the tidal and wave driven currents, a spectral wave model of the surface waves, and a sand transport model. MIKE 21 allows these components to be run and calibrated separately or to be coupled so that feedback processes between the three elements are included. The hydrodynamic and wave models have been coupled so that the wave field is applied to the hydrodynamic model and wave driven currents develop, and the wave model uses the water level variation from the hydrodynamic model so that areas of wave shoaling and breaking vary over a tidal cycle. Both the spectral wave model and hydrodynamic model have been used to force the sand transport model. The modelling therefore addresses issues identified with the Exe Estuary Flood and Coastal Risk Management Strategy model (Halcrow, 2010) where sediment transport was not modelled explicitly and wave driven currents were not modelled. All three components of the model use the same unstructured mesh, which allows resolution to be increased where processes such as wave breaking and wave driven currents need to be resolved over a small spatial scale.

In order to model typical annual sediment transport using continuous simulations the model would have to be run over several years of tide and wave data. Due to the high resolution and inclusion of multiple processes, model run times are approximately half real time making this length of simulation impractical. Instead the method of representative waves and tides has been used, where wave and tide conditions are chosen which will reproduce the patterns of annual average sediment transport. This method and the selection of the representative waves and tides used here are discussed under boundary conditions for the wave and hydrodynamic models retrospectively. There is insufficient wave and current data to use as boundary conditions for the coastal area wave and hydrodynamic models so regional wave and hydrodynamic models have been used to provide boundary conditions for these models.

The two-dimensional net sediment transport results have been analysed to determine general patterns of sediment transport and areas of net accretion and erosion. To simplify interpretation of the results, cross-sections have been extracted along

Dawlish Warren and Exmouth Beaches, and sediment budgets for the flood and ebb deltas have been calculated.

The bathymetry common to all three coastal area model components is described in Section 7.1. The coastal area wave model and the regional wave model used to derive the boundary conditions are described in Section 7.2 the coastal area hydrodynamic model and regional hydrodynamic model used to derive the tidal boundary conditions are described in Section 7.3. The sediment transport model is described in Section 7.4, and results from the modelling work are discussed in Section 7.5.

7.1 Model Bathymetry and Model Mesh

Model extents for the coastal area models are shown in Figure 7-1. The upstream extent of the model is Topsham Weir in Exeter, which is the tidal limit of the Exe. The bathymetry has been constructed from a combination of bathymetric survey and LiDAR in the near shore area, and digitised Admiralty Charts contained within the C-MAP database in the offshore area. A review of bathymetric data used is given in Section 2.1.

The model uses an unstructured mesh which allows different resolutions to be used in offshore and inshore areas. In the wave breaking zone, over Pole Sand and along Dawlish Warren and Exmouth beaches the minimum resolution is 10 m. This provides a resolution of at least 10 grid cells over the breaking zone. Within the Exe Estuary the low water channels have a minimum resolution of 40 m which is sufficient to allow water to drain out of the estuary at low tide. Offshore the resolution is around 150 m. The model mesh is shown in Figure 7-1.

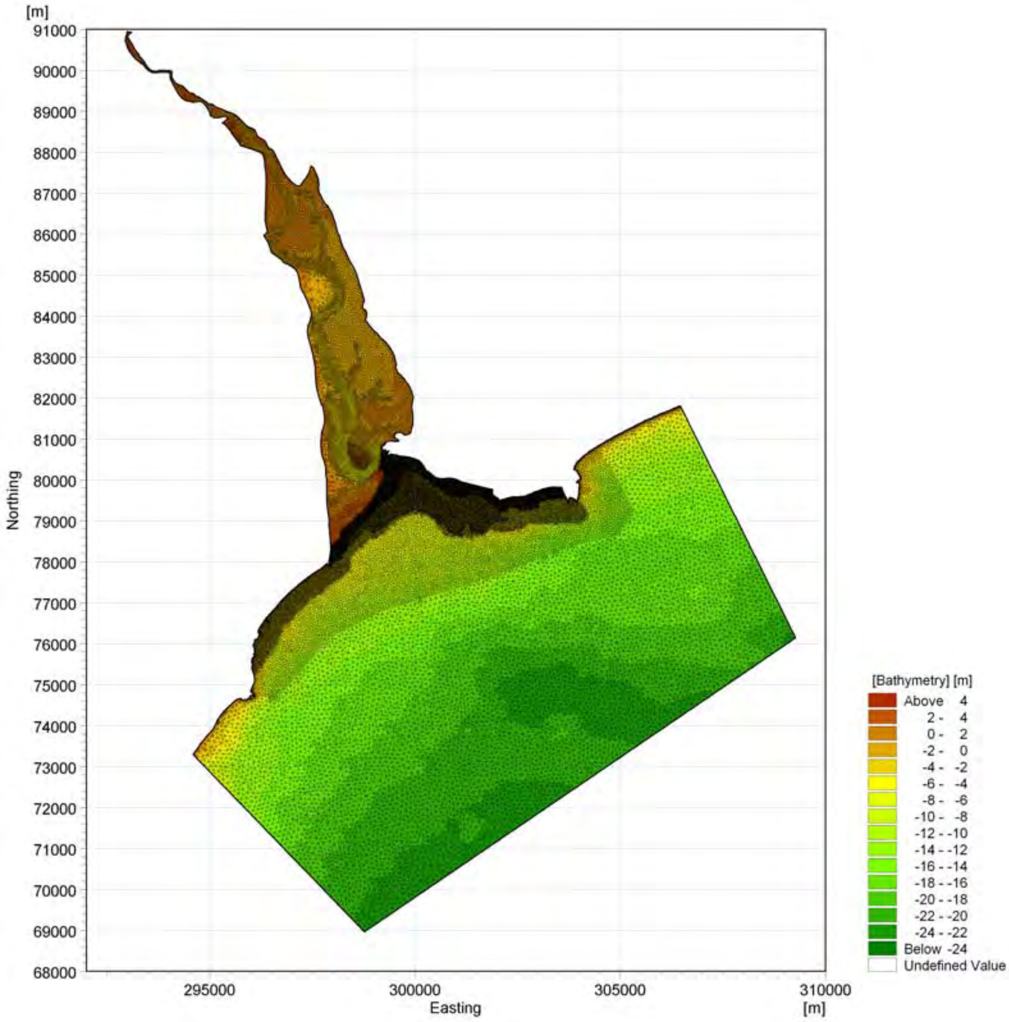


Figure 7-1: Model mesh and bathymetry.

Groyne condition surveys were carried out by Halcrow for Exmouth Beach and Dawlish Warren in 2011 (Appendix A). Groynes on both sides of the estuary were generally found to be in poor condition. Where groynes were still acting to retain sediment, beach levels on the non sheltered side of the groynes were similar to the top of the groynes, and resulting steps in beach level were detected in the LiDAR data. Groynes have therefore not been explicitly included in the coastal area model bathymetry; however any steps in the beach level due to the presence of groynes are included in the LiDAR and have been incorporated into the bathymetry.

7.2 Coastal Area Wave Model

7.2.1 General Approach

Waves have been modelled using the MIKE 21 Spectral Wave (SW) model. MIKE 21 SW is a third generation spectral wind-wave model that simulates the growth, decay and transformation of wind-generated waves and swells in offshore and coastal areas. The model includes wave growth by action of wind, non-linear wave-wave interaction, dissipation by white-capping, dissipation by wave breaking, dissipation due to bottom friction, refraction due to depth variations, and wave-current interaction. The model includes the effects of time-varying water levels, including flooding and drying of low-lying land surfaces.

To determine annual sediment transport rates with the model the typical annual wave climate needs to be taken into account in the model. Due to the high resolution the run time for several years of observed wave conditions would be prohibitively long, so instead the model has been run for a series of representative waves which produce the average sediment transport in each direction sector. Exmouth is sheltered from waves from the west and east due to the shape of Lyme Bay. Long time-series of wave data suitable for climatic analysis are not available within Lyme Bay, so available wave data has therefore been analysed for an offshore location and the regional wave model has been used to transform the waves inshore. Boundaries for the coastal area wave model have then been taken from the regional wave model results.

7.2.2 Representative Waves

It is impractical to run a coastal area model for all wave conditions throughout a year due to long simulation times. Therefore an approach is required where the yearly wave climate is reduced to a small number of representative or characteristic wave conditions which when simulated with the sediment transport model can be combined together to form a representation of the yearly sediment transport. The method employed in this study is described below.

The characteristic wave conditions are determined based on the contributions of different wave events in the long (e.g. 20-years) time-series to the annual energy flux (Johnson, 2011). In this approach, it is considered that the sediment transport rate from a given direction is related to the energy flux from that direction.

A brief description of the method is given as follows:

- First, data in the wave time-series at the model boundary are grouped into bins with intervals of 0.5m for H_{m0}, 1s for T_p and 10^o for wave directions.
- Next, the energy flux contribution for each bin is calculated as:

$$E_f = H_{m0}^2 \times T_p \times \textit{percent occurence}. \qquad (7.1)$$

- Next, the characteristic wave conditions (Hm0 and Tp) for each 10^o sector are determined as the centroid (first moment) of the E_f versus Hm0 and E_f versus Tp curves as shown in *Eqs 7.2* and *7.3* below, where the subscript "i" refers to (H, T) elements in each direction sector:

$$H_{rep} = \frac{\int H * E_f * dH}{\int E_f * dH} \cong \frac{\Delta H \sum H_i * E_{fi}}{\Delta H \sum E_{fi}} = \frac{\sum H_i * E_{fi}}{\sum E_{fi}} \qquad (7.2)$$

$$T_{rep} = \frac{\int T * E_f * dT}{\int E_f * dT} \cong \frac{\Delta T \sum T_i * E_{fi}}{\Delta T \sum E_{fi}} = \frac{\sum T_i * E_{fi}}{\sum E_{fi}} \qquad (7.3)$$

- Lastly, the associated frequency of occurrence is calculated such that the characteristic event for a given 10^o sector gives the same energy flux contribution as the sum of energy flux contributions for all bins in that sector:

$$f_{rep}(\%) = \frac{\sum_{sector} E_{fi}}{H_{rep}^2 * T_{rep}} \quad (7.4)$$

In this study representative waves have been calculated using 10 years of 3 hourly wave data from the Met Office European Wave Model at 50° N, 3.66° W which is offshore from Lyme Bay. The resulting representative waves are given in Figure 7-2 together with the associated frequencies for the wave conditions. Model results for individual representative wave conditions are weighted using these relative frequencies to give the annual average transport. The number of tide only or calm days is then chosen so that the total number of days modelled is 365 to ensure that the transport due to the tide is included 365 days a year.

Table 7-1: Significant waves used to force the regional wave model.

Direction (degrees)	Significant Wave Height (m)	Peak Wave period (s)	Equivalent Days
30-60	2.12	6.53	4.2
60-90	2.59	7.14	12.3
90-120	2.39	6.87	8.5
120-150	2.38	6.86	3.1
150-180	2.72	7.26	2.5
180-210	3.66	8.40	5.8
210-240	4.06	9.08	52.8
240-270	3.23	8.24	26.7
270-300	2.54	7.35	5.8
300-330	2.00	6.39	3.2
330-30	1.76	5.92	4.9

Figure 7-2 shows that the maximum offshore energy flux is due to waves from 210°N to 240°N, which is expected both due to the longer fetch over the Atlantic Ocean for waves from this direction, and prevailing westerly winds. There is a smaller peak between 60°N and 90°N corresponding to waves from the North Sea.

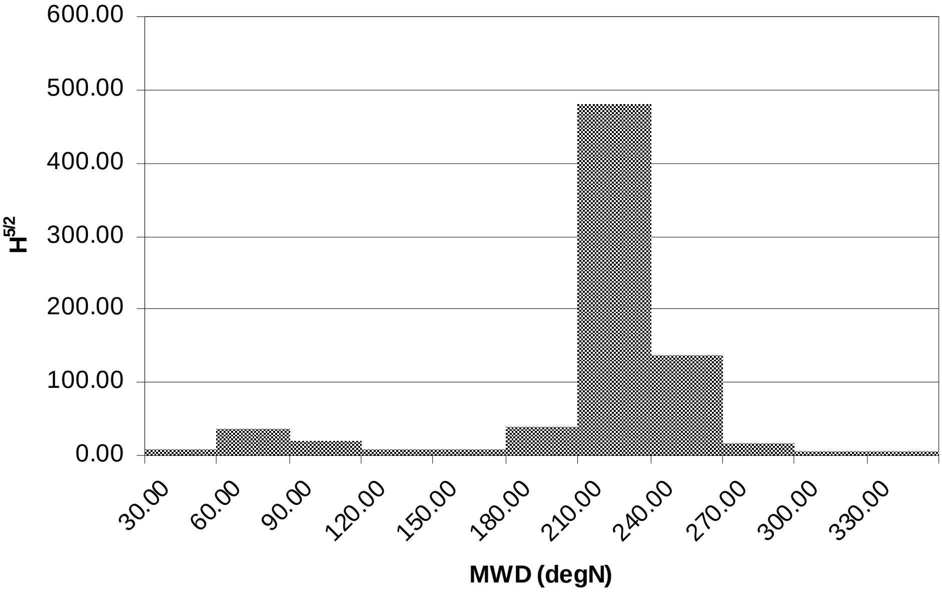


Figure 7-2: Predicted sediment flux due to waves for 30° direction sectors.

7.2.3 Regional Wave Model Results for Representative Wave Conditions

The regional wave model and calibration is described in Section 3. The model has been run using the calculated representative waves and Figure 7-2 to generate boundary conditions for the coastal area wave model. A constant uniform wind field has been applied to the model using the speed and direction relationship described in Section 2.2.2.

The regional wave model results for the dominant direction of 225°N are shown in **Error! Reference source not found.**, and clearly show a reduction in wave height within Lyme Bay due to the headlands to the south west.

For directions between 270°N and 30°N significant wave heights around the mouth of the Exe Estuary are less than 0.25 m so these wave conditions have been discarded for the coupled coastal area model runs and days corresponding to these events have been considered as calm days.

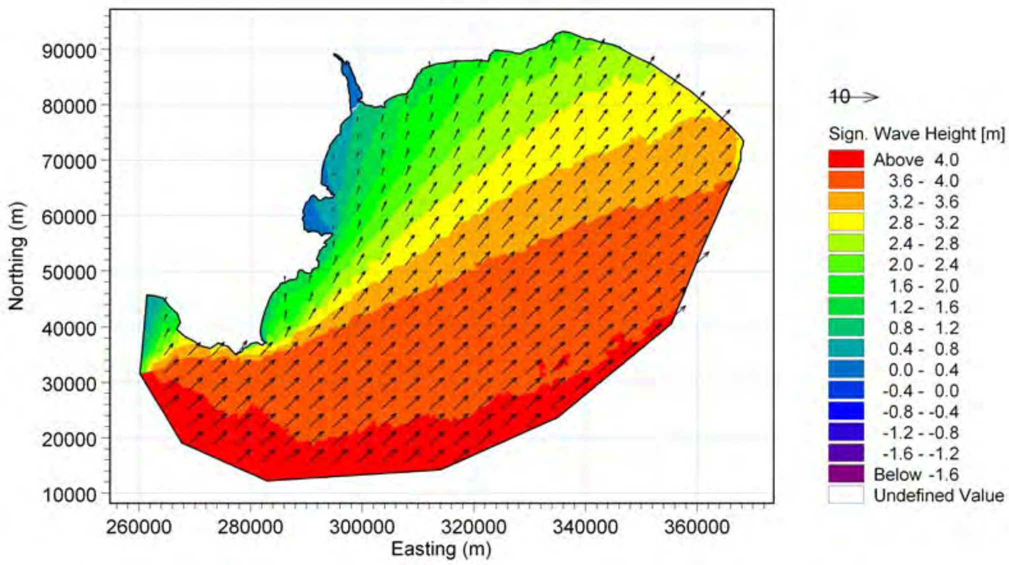


Figure 7-2: Significant wave height in the regional wave model for a MHWS water level and waves from 225°N.

7.2.4 Model Calibration

The coastal area model was run using boundaries derived from the regional wave model for each of the representative wave conditions in Table 7-1 for both a constant MHWS and a constant MLWS water level. A constant wind stress was applied using the values in Table 7-1. All other parameters were chosen to be the same as in the regional wave model. Results from the coastal area model for a constant MHWS level of 2.16 mAOD are shown in Figure 7-3.

The coastal area model results were compared with the results from the calibrated regional wave model for MHWS and MLWS conditions. Away from the breaking zone maximum differences of 0.1 m were found. The breaking zone in the coastal area model is sufficient for accurate resolution of the breaking zone, while the resolution in the regional wave model is insufficient for accurate resolution in this area. Significant differences in the results can therefore be expected in this area.

7.2.5 Model Coupling

The water level variation from the hydrodynamic model has been applied to the spectral wave model, and the wave field was recalculated every hour. Currents from the hydrodynamic model were not applied to the spectral wave model as this prevented the wave model from converging to a solution. This is unlikely to have a significant effect on the results, as the area in the model with strong currents at the mouth of the estuary is largely sheltered by Pole Sand and has small wave heights.

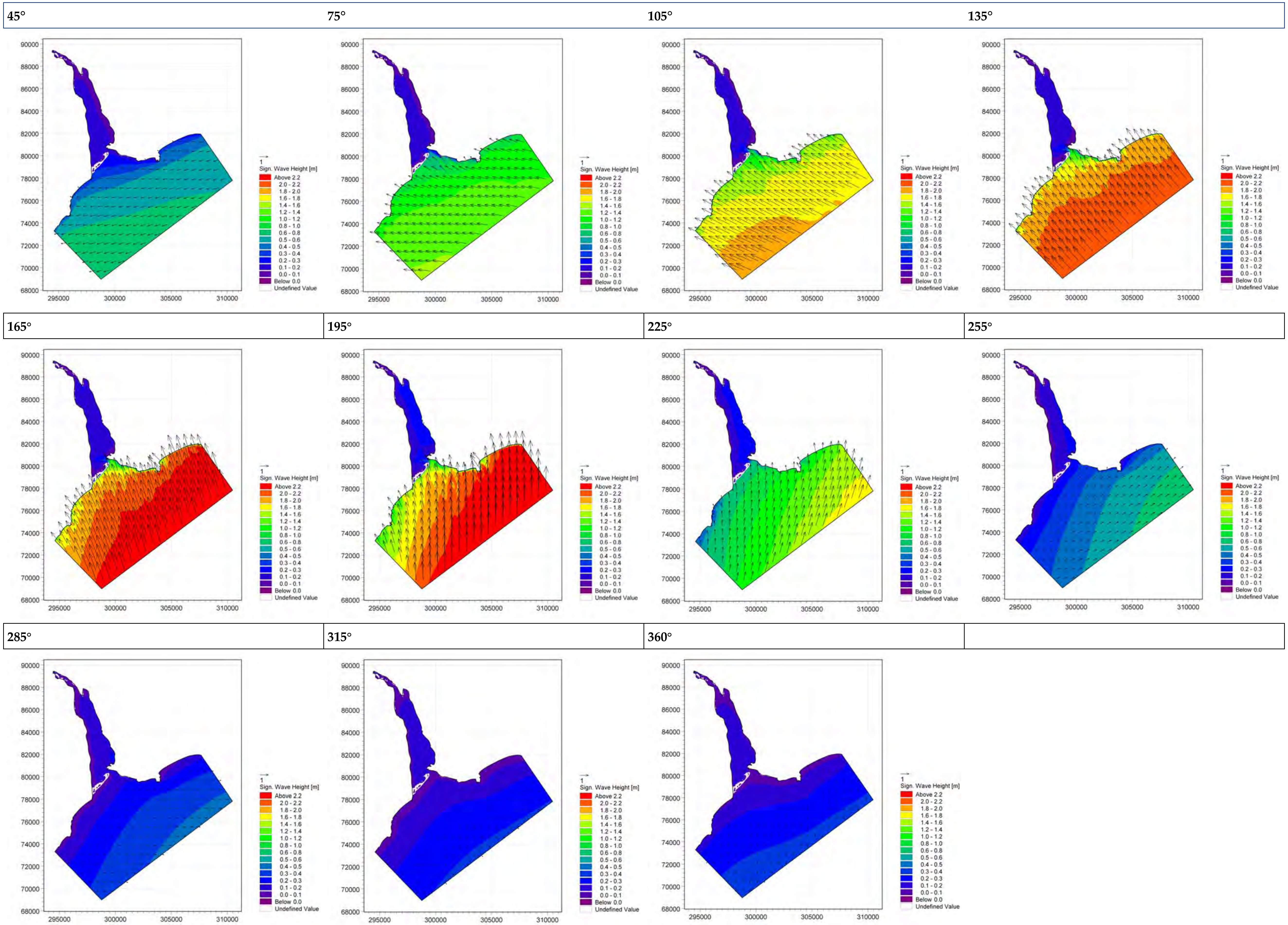


Figure 7-2: Inshore wave conditions for offshore representative waves. MHWS conditions.

7.3 Hydrodynamic Modelling

7.3.1 General Approach

A hydrodynamic model of the coastal area has been set up using the MIKE 21 Flow Model (MIKE 21 FM). MIKE 21 FM uses an unstructured grid allowing higher resolution to be used in areas of interest, and lower resolution to be used elsewhere so that run times are minimised. The hydrodynamic module of MIKE 21 FM solves the 2d shallow water equations and can be used to simulate water level variations and flow in response to a variety of forcing functions in lakes, estuaries and coastal regions. The effects and facilities include bottom shear stress, wind shear stress, barometric pressure gradients, Coriolis force, momentum dispersion, sources and sinks, evaporation, flooding and drying and wave radiation stresses. The hydrodynamic module can be easily coupled to other modules in the MIKE 21 suite which are required for the coastal area modelling. These include the spectral wave model (MIKE 21 SW) and the sand transport model (MIKE 21 ST).

There are insufficient velocity and water level observations available to use as boundary conditions for a coastal area model so a lower resolution regional hydrodynamic model of the circulation patterns within Lyme Bay and the wider English Channel has been set up, also using MIKE 21 FM, and used to produce suitable boundary conditions. The setup and calibration of this regional hydrodynamic model is discussed in Section 7.3.2, while the set up and calibration of the coastal area hydrodynamic model is discussed in Section 7.3.3. The data available for calibration of both models is discussed in Section 2.

7.3.2 Regional Hydrodynamic model

7.3.2.1 General Approach

The model covers the entire English Channel with higher resolution in the study area around Lyme Bay. Bathymetry was created from available bathymetric data amalgamated from C-MAP bathymetry and LiDAR data. The model has been forced using predicted tidal boundary conditions only, so simulates the behaviour of the English Channel in the absence of wind and waves. Calibration simulations were carried out for the period 23/01/2008 to 09/02/2008, to give a good overlap with observations at ADCP Station 6. Principal tide levels for this are given in Table 7-2.

There are spring tides at the start and end of the simulation period. The spring tide on 24/02/2008 has a peak water level of 1.85 mAOD at Exmouth Dock while those at the end of the period are higher with a high spring tide of 2.25mAOD on 9/02/2008. Neap tides occur around 3/02/2008 however there is missing data at Exmouth Dock over this period.

Table 7-2: Principal tide levels for Exmouth Dock and Approaches from the Admiralty Tide Tables.

Tide Levels (m ODN)	MHWS	MHWN	MSL	MLWN	MLWS
Exmouth Dock	2.17	0.97	0.27	-0.53	-1.63
Exmouth Approaches	2.16	0.96	0.07	-0.74	-1.94

The main parameters within the hydrodynamic model that have been modified during the calibration exercise are:

- bathymetry and mesh resolution;
- model boundaries;
- eddy viscosity; and
- bed roughness.

7.3.2.2 Bathymetry and model resolution

Offshore bathymetric data have been obtained using MIKE C-MAP, an electronic chart database (CM-93 Edition 3.0) obtained through DHI software. These data have been used outside of the Exe Estuary. The data have been converted from Chart Datum to Mean Sea Level (MSL). Values of MSL are only given at isolated locations throughout the model domain, elsewhere they have been interpolated between adjacent points using the method of nearest neighbours.

Local LiDAR and bathymetry survey data for the Exe Estuary was obtained from the Channel Coastal Observatory. The bathymetric surveys are from 2010, and are to ODN Newlyn. The difference between ODN and MSL for Exmouth Approaches is 0.07 m, and is not considered significant in a regional model. Therefore no correction has been made to convert the LiDAR and bathymetric data to MSL.

The computational mesh (unstructured mesh) uses a coarse mesh size in offshore areas and finer mesh in near shore areas in order to optimize run times whilst reasonably resolving important features on a regional scale. The bathymetry and modelling mesh is shown in Figure 7-3. The detailed bathymetry and higher resolution mesh around the Exe Estuary is shown in Figure 7-3. The model resolution is in the order of 5 km in the offshore region and 35 m for the low water channels at the mouth of the Exe Estuary. This is sufficient to capture the large scale tidal currents in the English Channel and around ADCP Station 6 in Lyme Bay. Within the Exe Estuary not all low water channels are resolved so calibration at ADCP Stations 4 and 5 is expected to be less reliable that at ADCP Station 6, as the water exchange through the inlet may not be accurate due to poor resolution in the estuary. Due to the large model domain increasing the resolution to improve calibration within the Exe Estuary is not feasible as it would lead to very long run times.

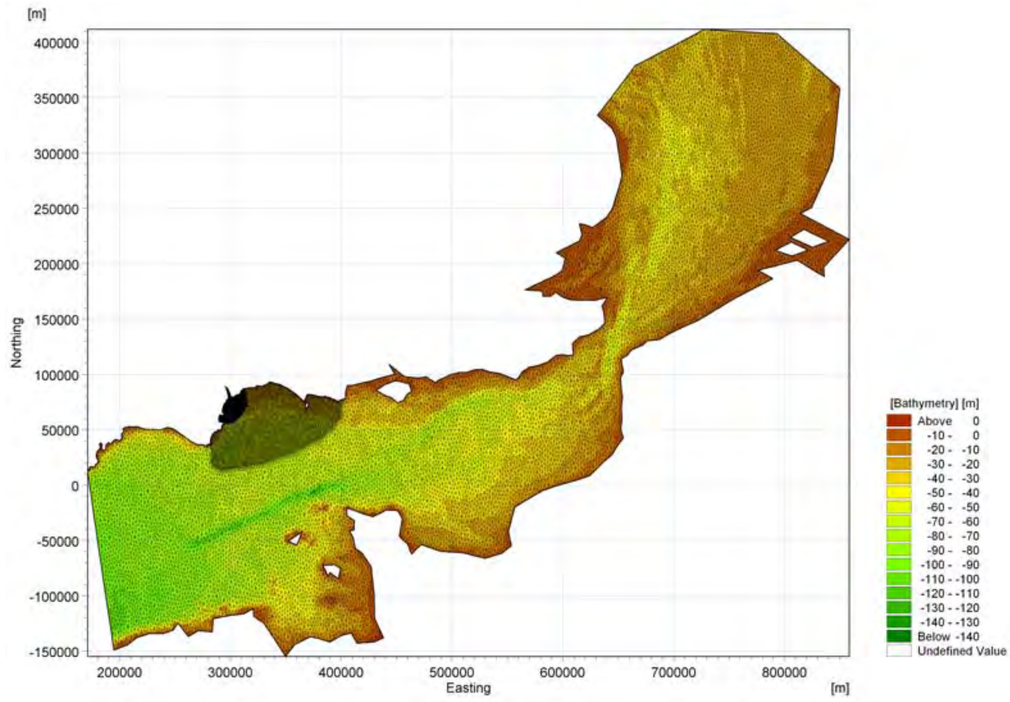


Figure 7-3: Regional Hydrodynamic Model mesh and bathymetry.

7.3.2.3 Model Boundaries

Model boundaries have been generated using the global tide model contained within MIKE 21. The model is based on TOPEX/POSEIDON satellite altimetry data and the constituents are at ¼° resolution and only applicable in depths greater than 20 m, further details are available in the MIKE 21 User Manual, and Andersen 1995. Tides to the east of Lyme Bay are complex with a double high water at Swanage and a double low water at 50 30N 02 00W (Figure 7-4). On the eastern side of Lyme Bay at West Bay the tidal shape is distorted with a period of slower rise between two and four hours after low tide (Figure 7-5). The global tide model has insufficient resolution to correctly capture the complexity around Lyme Bay, and the limited observations and C-MAP points in this area means that tidal boundaries around Lyme Bay cannot be derived from observed time-series. By modelling the entire English Channel the boundaries can be located in areas where the MIKE 21 global model has sufficient resolution to represent the tides.

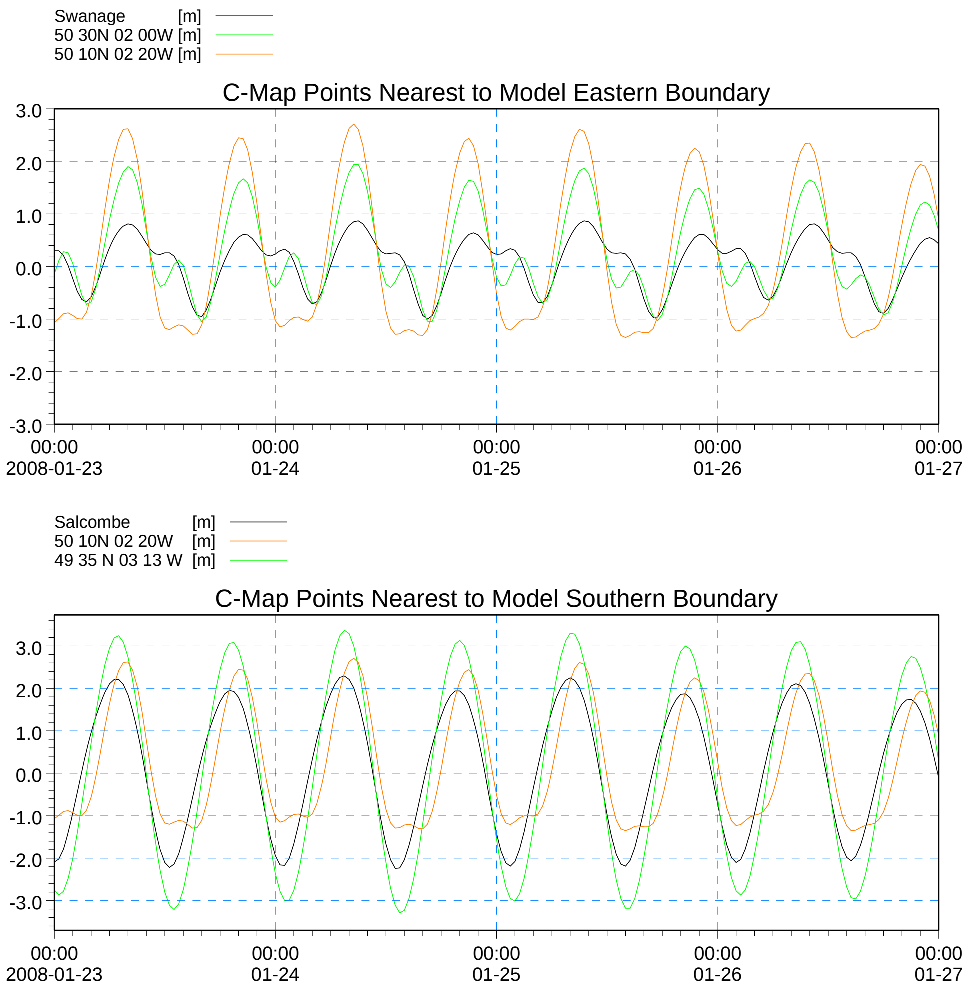


Figure 7-4: C-Map points closes to the southern and eastern model boundaries.

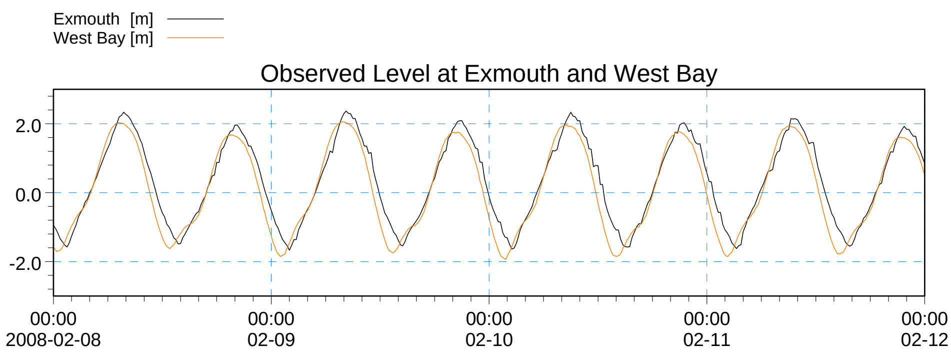


Figure 7-5: Observed Tide levels at Exmouth and West Bay.

7.3.2.4 Roughness (Manning)

The general Manning (M) number in the model domain is M=50. During model calibration, sensitivity tests to M=60 and M=40 were carried out. The tidal range at ADCP Station 6 is insensitive to the roughness (Figure 7-8). The RMS error was calculated for different lag times between the model results and observations in order to determine the phase error. The phase error between observed and modelled levels is less than 15 minutes for all Manning numbers tested (Figure 7-7). High water occurs earlier for the case with higher roughness M=40 than M=60.

Figure 7-8 shows that current speed is more sensitive to roughness than water level, with peak speeds increasing with decreasing roughness. The RMS error was calculated for the period 23/01/2008 -30/01/2008, and found to be similar for all three roughness values, as the model underestimates speeds on some tides and overestimates speeds on others with no consistent pattern. The calibration for current direction is better for M=50 and M=60 than M=40.

Manning Roughness	RMS
40	0.03
50	0.03
60	0.04

Figure 7-6: RMS error in the modelled speed in the regional hydrodynamic model at ADCP Station 6 for the period 23/01/2008 - 30/01/2008.

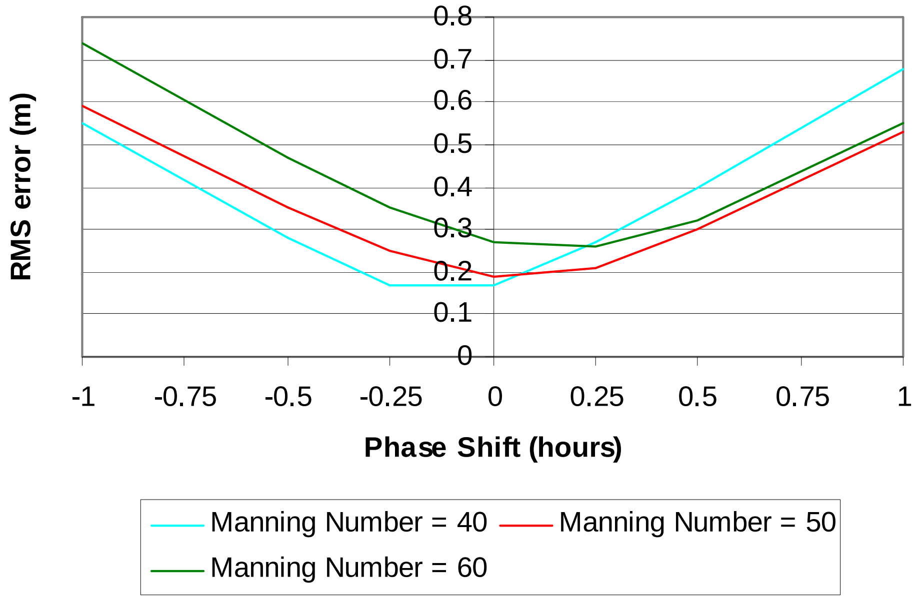


Figure 7-7: RMS error in the modelled level in the regional hydrodynamic model at ADCP Station 6 for different phase shifts to the model results.

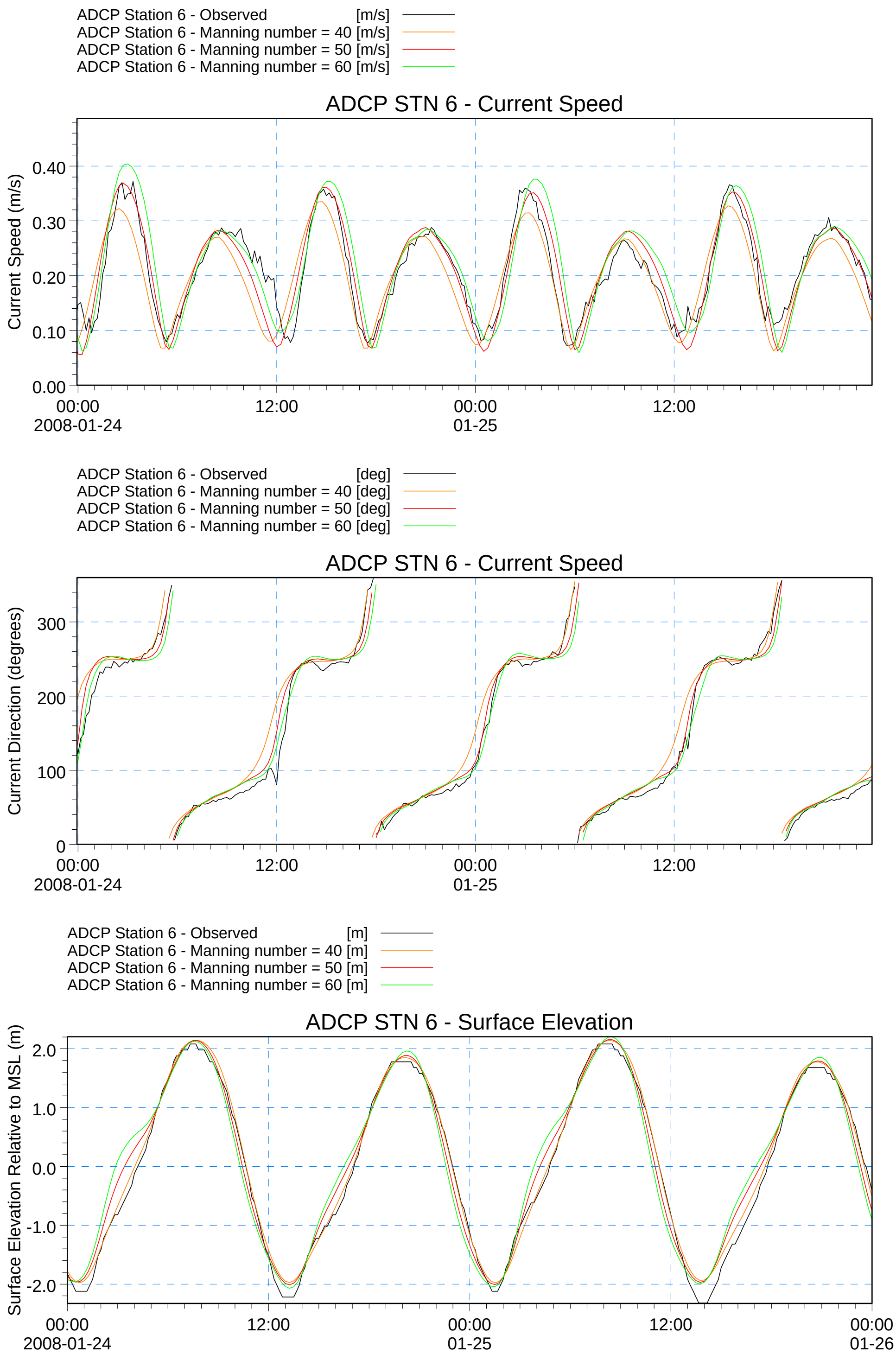


Figure 7-8: Sensitivity to surface roughness in the regional hydrodynamic model at ADCP Station 6.

7.3.2.5 Turbulence (Eddy Viscosity)

The eddy viscosity formulation used in the model is Smagorinsky with a constant coefficient value of 0.28. Coefficient values of 0.2 and 0.4 was tested during model calibration, and found to make little or no difference to model water levels, current speeds and current direction at ADCP Stations 5 and 6.

7.3.2.6 Model calibration results

7.3.2.6.1 Environment Agency Calibration Guidelines

Environment Agency Guidelines for calibration of coastal area models are given in Technical Report W168, Quality Control for Computational Estuarine Modelling, Environment Agency 1998, and are reproduced in Table 7-3.

Table 7-3: Environment Agency Guidelines for Model Calibration from Technical Report, W168. These criteria are expected to hold for 90% of the observations where the model performance is evaluated.

Parameter	Coastal Area	Within Estuaries
Level	± 0.1m or 10% of spring ranges or 15% of Neap ranges	± 0.1m at the mouth of the estuary ± 0.3 m at the head of the estuary or 15% of Spring Tidal ranges or 20% of neap tidal ranges
Speed	± 0.1m/s or ±10-20% of observed speed	± 0.2m/s or ±10-20% of observed speed
Direction	± 10 degrees	± 20 degrees
Timing of High Water	± 15 minutes	± 15 minutes at the mouth of the estuary ± 25 minutes at the head of the estuary.

7.3.2.6.2 Water level calibration results

The only tidal level observations available during either simulation period are at Exmouth and Weymouth. RMS errors have been calculated for Exmouth and Weymouth, and are given in Table 7-4. In addition to the observed levels, C-MAP data is available at Teignmouth and Bridport.

Table 7-4: RMS error in regional hydrodynamic model water level at Exmouth and Teignmouth.

Gauge	RMS error
Exmouth	0.36
Weymouth	0.23

Figure 7-11 shows that the model slightly overestimates the tidal range at all locations other than Teignmouth. Calibration at Teignmouth is poor as the Teign Estuary is not included in the model.

At Exmouth, there is a phase lag of around one hour on the flood tide and 0.5 hours on the ebb tide, although the average phase lag is less than 15 minutes at high water. The RMS error at Exmouth is greater than the Environment Agency modelling guidelines for coastal area models that levels should be correct to within 0.1 m, however much of this discrepancy can be attributed to the phase lag; the RMS error in spring water level is 0.13 m. The gauge at Exmouth is located within the estuary and it is possible that lack of model resolution in this area may affect calibration.

At Weymouth the model overestimates water levels on alternate high tides by approximately 0.15 m. The model correctly reproduces the first low water, however the second low water present in the model results it is higher than in the observed. The RMS error at Weymouth is greater than in the Environment Agency modelling guidelines, however much of this can be attributable to surge (Figure 7-9) and given the complicated tidal shape at Weymouth the calibration is acceptable.

At Bridport the tide rises slowly between one and six hours after low water, and this behaviour is not captured in the model as the rate of rise in the model is faster than observed.

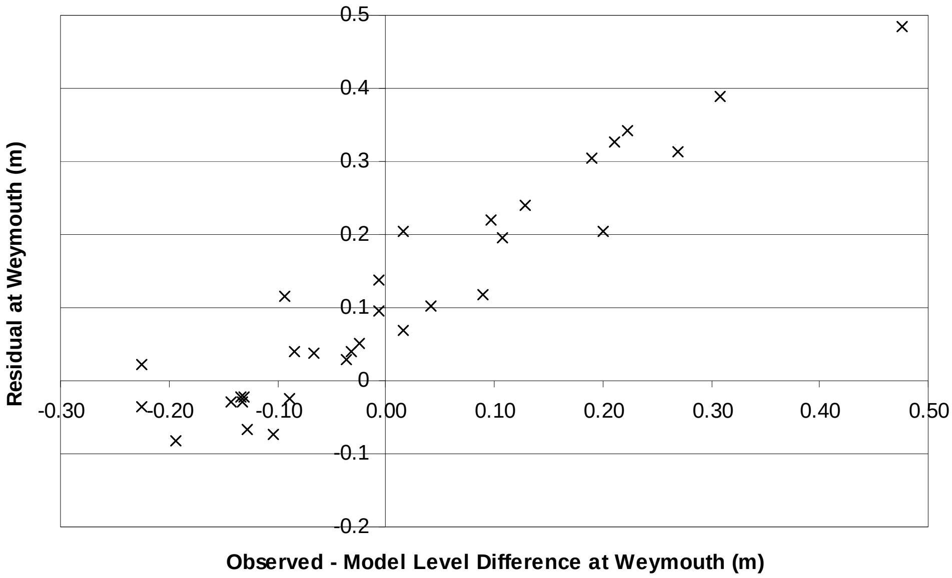


Figure 7-9: Observed-Model level difference at high tide at Weymouth and residual between predicted and astronomical tide at the BODC gauge at Weymouth.

7.3.2.6.3 Current calibration results

Model results at ADCP Stations 5 and 6 are shown in Figure 7-13 to Figure 7-16. Calibration at ADCP Station 6 is generally good (Figure 7-13 and Figure 7-14). The observations show strong flood ebb asymmetry with higher speeds on the flood than the ebb tide, and this behaviour is correctly captured by the model. Flow is in the correct direction on both the flood and ebb tides; however there is a slight overestimation of peak speeds on the ebb tide of around 0.05 m/s. Figure 7-6 shows that the RMS error in and phase lag in the speed at ADCP Station 6 are both well within the Environment Agency guidelines that speeds should be correct to within 0.1m/s and that the phasing should be correct to within 15 minutes.

ADCP Station 5 is located slightly to the west of the main ebb flow out of the Exe Estuary (Figure 7-10) and a result calibration at ADCP Station 5 is sensitive both to small errors in the location of the model time-series and to changes in the bathymetry which may have occurred between January 2008 when the ADCP was deployed and 2010 when the bathymetric survey was taken. The current speed at various points around ADCP Station 5 was sampled and found to be consistently too high over too long a duration on both the flood and the ebb tides. The model shows that there is flow out of the Exe Estuary over the Pole Sands and through two distinct low water channels, to the east and west of Pole Sands. The model resolution may be insufficient to partition flow correctly between these flow paths.

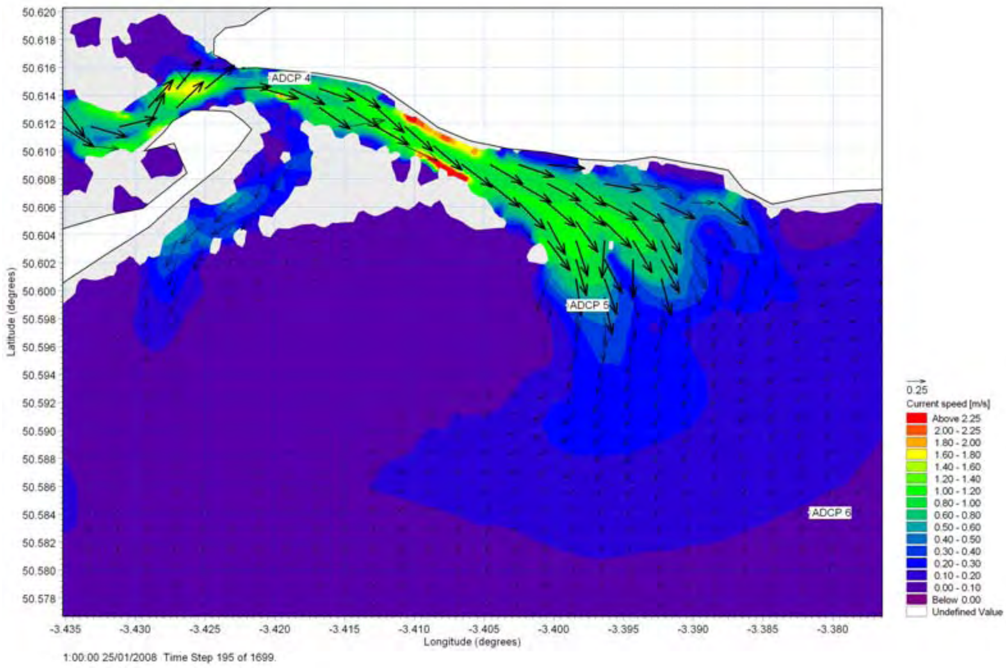


Figure 7-10: Flow at the mouth of the estuary in the regional hydrodynamic model on a spring Ebb tide. The time shown is 5 hours after high water at Exmouth and corresponds to the peak velocity at ADCP Station 5.

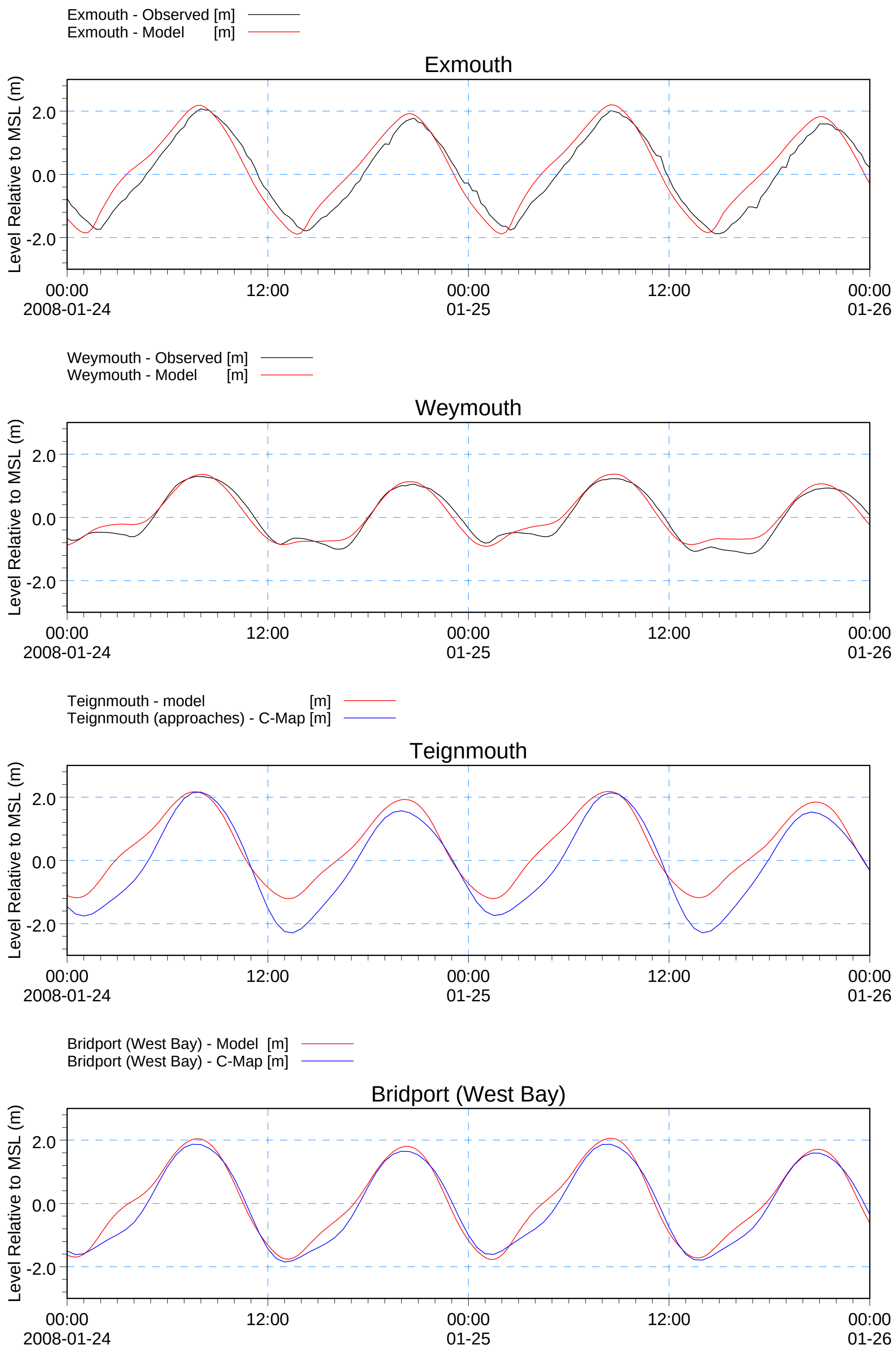


Figure 7-11: Water level calibration at for the regional hydrodynamic model at Exmouth, Weymouth, Teignmouth and Bridport during spring tides.

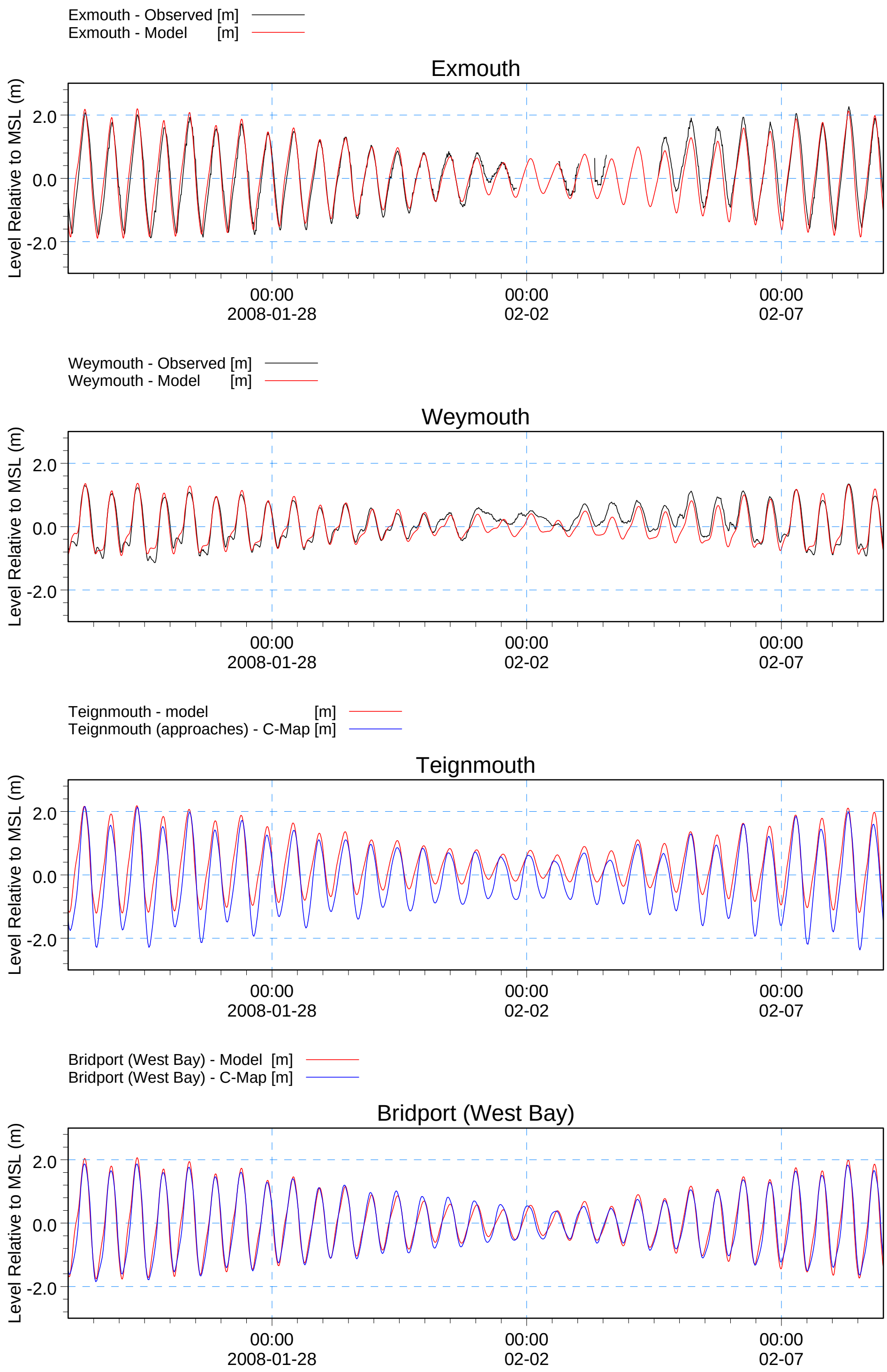


Figure 7-12: Water level calibration for the regional hydrodynamic model at Exmouth, Weymouth, Teignmouth and Bridport over the entire simulation interval.

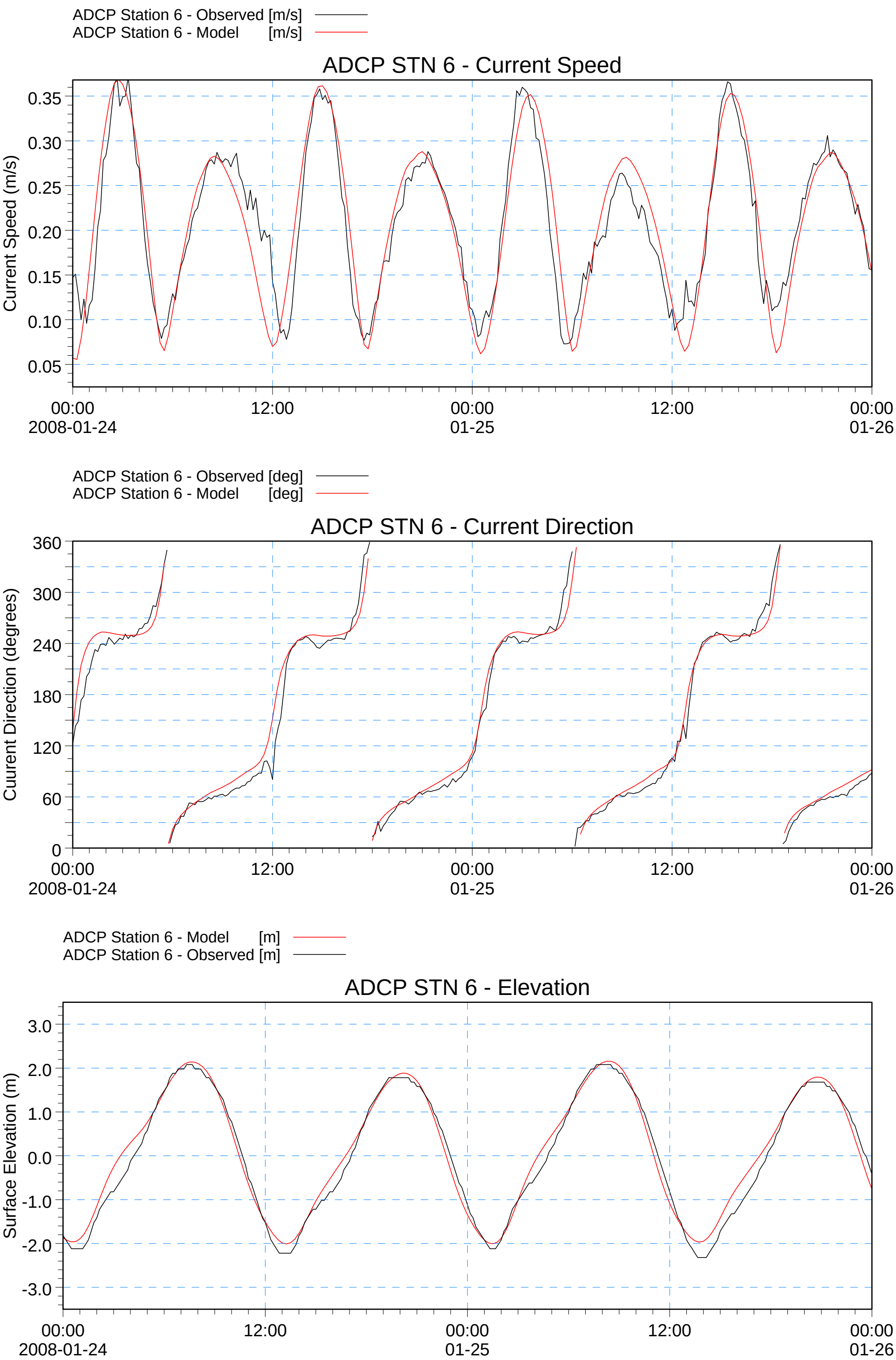


Figure 7-13: Model calibration at ADCP Station 6 for the regional hydrodynamic model during a spring tide.

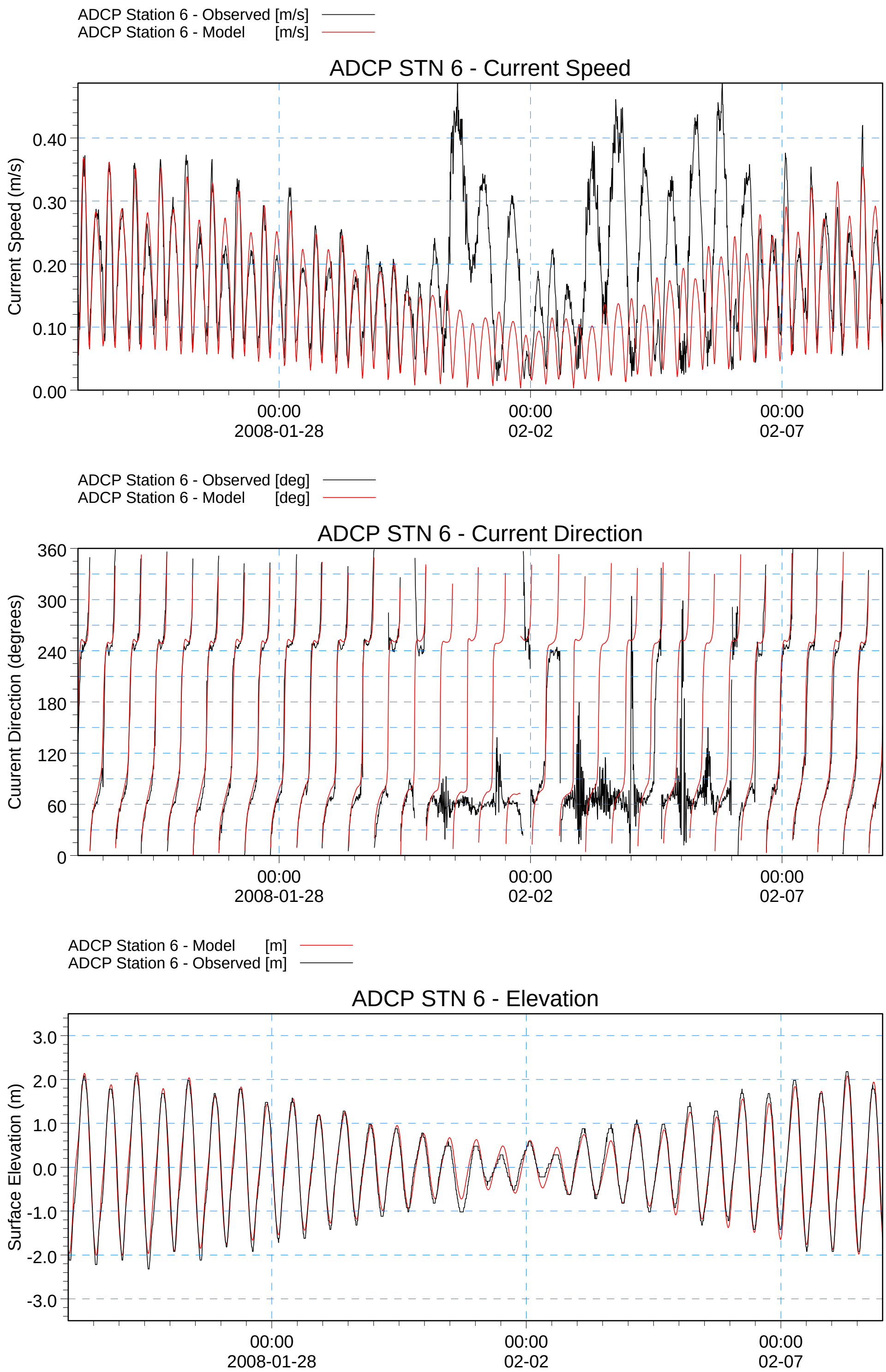


Figure 7-14: Model calibration at ADCP Station 6 for the regional hydrodynamic model over the entire simulation interval.

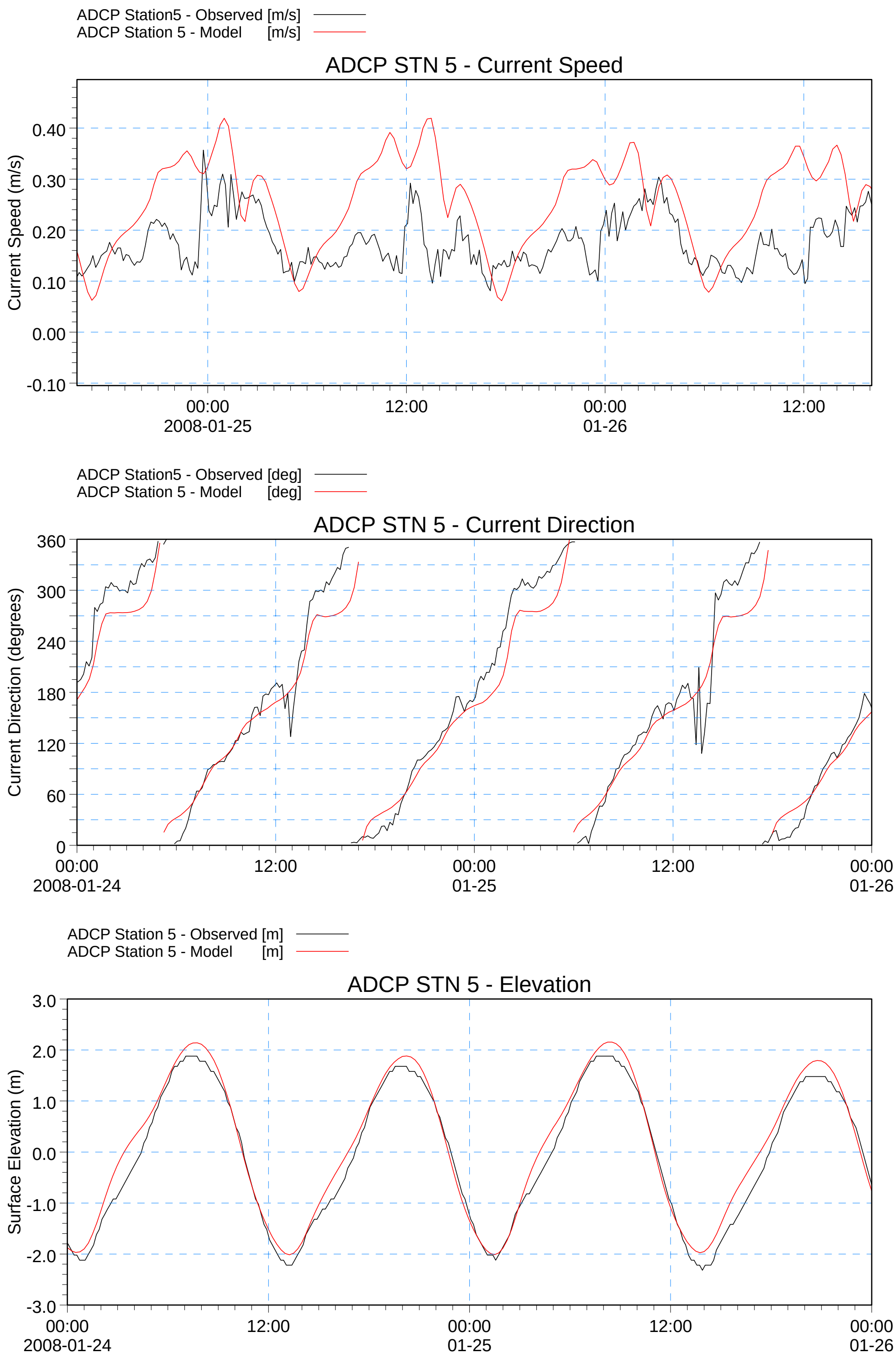
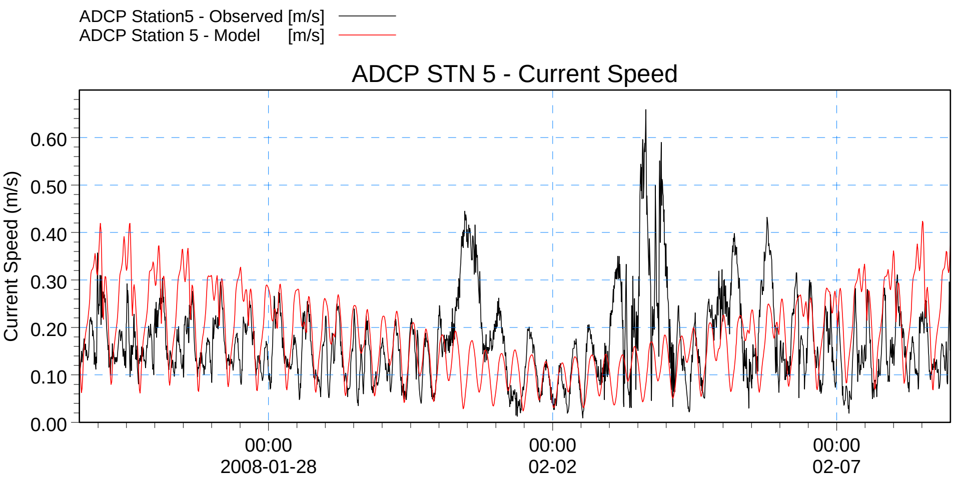
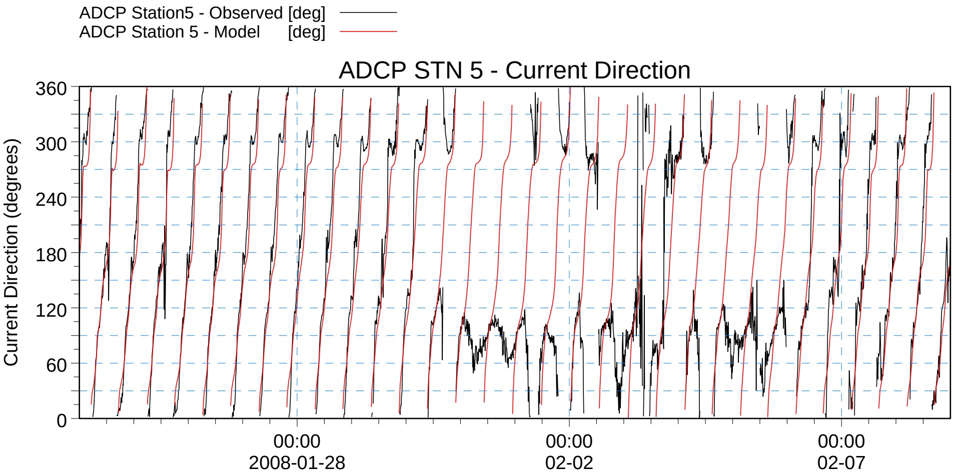


Figure 7-15: Model calibration for the regional hydrodynamic model at ADCP Station 5 during a spring tide.





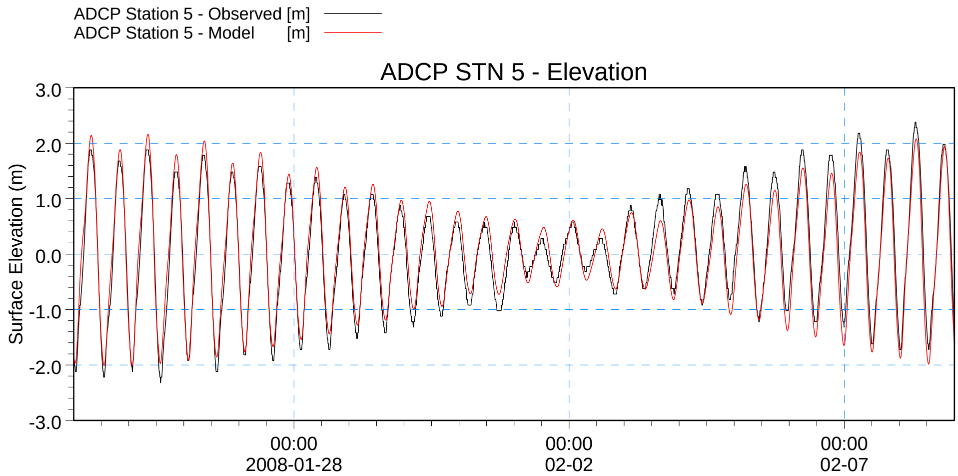


Figure 7-16: Model calibration for the regional hydrodynamic model at ADCP Station 5 over the entire simulation interval.

7.3.2.7 Summary

A regional hydrodynamic model of the English Channel has been constructed in order to provide flow and level boundary conditions for a coastal area model of Dawlish Warren, Pole Sands and Exmouth Beach. The model shows good calibration for current speeds against ADCP measurements in deep water within Lyme Bay, and is able to capture most features of the complex tidal shape to the east of Lyme Bay. For the low water channel to the east of Pole Sands calibration is poorer due to insufficient model resolution, and noise in the observed current speeds. The regional model is therefore sufficient to produce boundary conditions for the coastal area model, provided that the boundaries for the coastal area model are located in deep water away from Pole Sands and the mouth of the Exe Estuary.

7.3.3 Coastal Area Hydrodynamic Model

7.3.3.1 Boundary Conditions

Boundary conditions for the hydrodynamic model are taken from the regional model of the English Channel constructed for this project and described in Section 7.3.2. Both velocity and level have been imposed at all boundaries, using the Flather condition. The English Channel model has been built using MSL as a datum rather than ODN used in the coastal area model so the level from the English Channel model has been adjusted from MSL to ODN by adding 0.07 m the difference between MSL and ODN at Exmouth Approaches.

The coupled hydrodynamic and wave model runs in approximately ½ real time, and needs to be run for waves from several different direction sectors. This makes it impractical to run the hydrodynamic model over a several months of tidal cycles. Instead it is necessary to choose a representative tide which reproduces the average annual tidal sediment transport. The mean spring tide is a conservative assumption for this tide (Cayocca, 2001). This tide is used for all wave conditions modelled (see Section 7.2.2).

The hydrodynamic model is run over the period 24/01/2008 17:30 to 26/01/2008 01:30 and the sediment transport is analysed over the two tides from 25/01/2008 00:30 to 26/01/2008 01:30; which allows the hydrodynamic model ½ tide to spin up. The period chosen corresponds to a mean spring tide.

7.3.3.2 Calibration for Tidal Conditions

A description of the data available for calibration of the hydrodynamic model is given in Section 2. ADCP Stations 4, 5 and 6 and the level gauge at Exmouth Dock are within the area of the coastal area model. There is no data available for ADCP Station 4 for the simulation period chosen, so a period with a similar range tide has been used for comparison at ADCP Station 4 instead. These observations have been plotted using dotted lines to highlight the lower confidence in using these results for calibration due to the time difference.

The sensitivity to bed roughness and eddy viscosity has been tested during model calibration.

7.3.3.2.1 Model Roughness

A Manning number of M=50 has been used in Lyme Bay and a Manning number of M=40 has been used within the estuary and over rocky areas. The distribution of the Manning number used within the model is shown in Figure 7-18. The sensitivity to a uniform roughness of M=60 throughout the model domain, and a uniform roughness of M=50 throughout the model domain was tested during model calibration. At ADCP Station 6 there was little or no difference in current speed and direction between the simulations with different bed roughness. Observations at ADCP Station

5 are noisy and do not have a clear tidal signal making detailed comparison with model results difficult. For M=60, peak velocities at ADCP Station 5 are higher than observed and may show some numerical noise (Figure 7-17). Reducing the Manning number in the estuary to M=40 slightly improves the phase lag on the ebb tide at Exmouth Dock (Figure 7-19). It should be noted that the comparisons with current speed and direction at ADCP Station 4 are less reliable than those for level, as the tide compared is of a similar range but from a different time period.

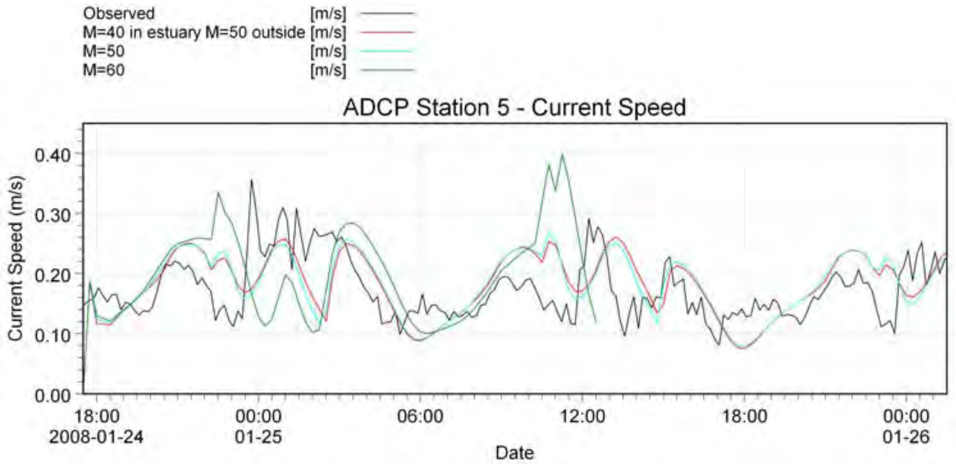


Figure 7-17: Sensitivity of current speed to bed roughness in the hydrodynamic coastal area model at ADCP Station 5.

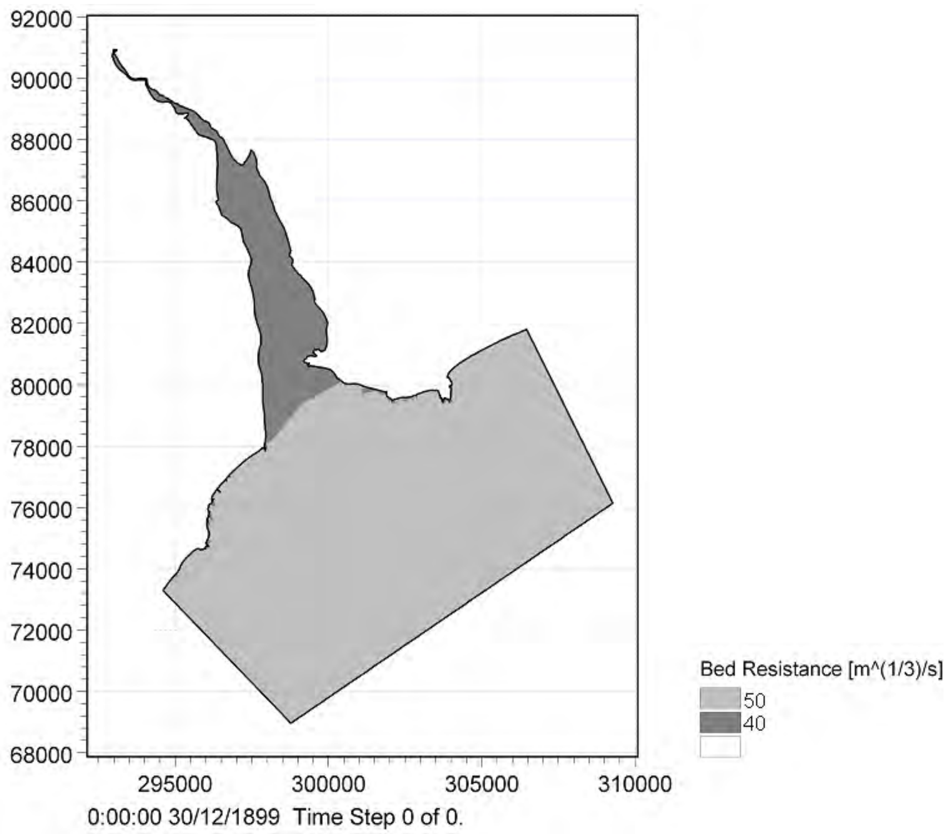


Figure 7-18: Bed resistance used in the coastal area hydrodynamic model.

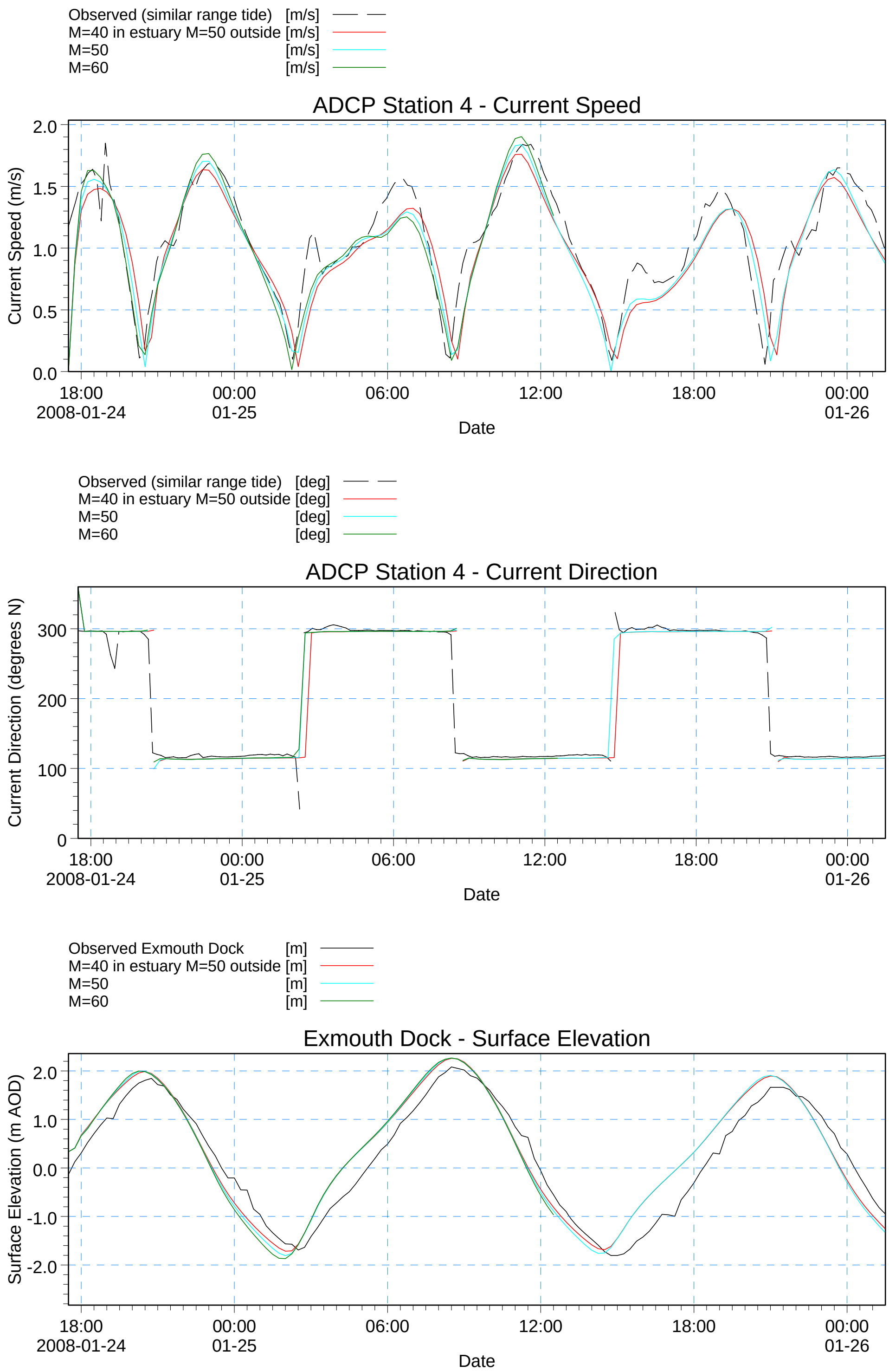


Figure 7-19: Sensitivity to bed roughness at ADCP Station 4 for the coastal area hydrodynamic model.

7.3.3.2.2 Eddy Viscosity

The eddy viscosity formulation used in the model is Smagorinsky with a constant coefficient value of 0.28. Coefficient values of 0.2 and 0.4 was tested during model construction and found to make no difference to the results.

7.3.3.2.3 Calibration Performance

Environment Agency guidelines for model calibration in coastal areas are given in Table 7-3 and have been used in assessing model calibration performance.

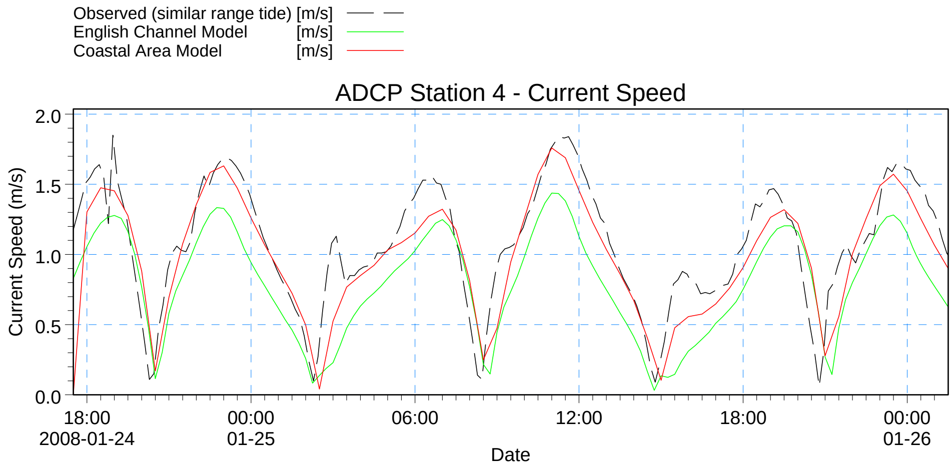
Calibration results from the hydrodynamic model are shown in Figure 7-20 to Figure 7-22 together with the English Channel model used to generate the boundaries.

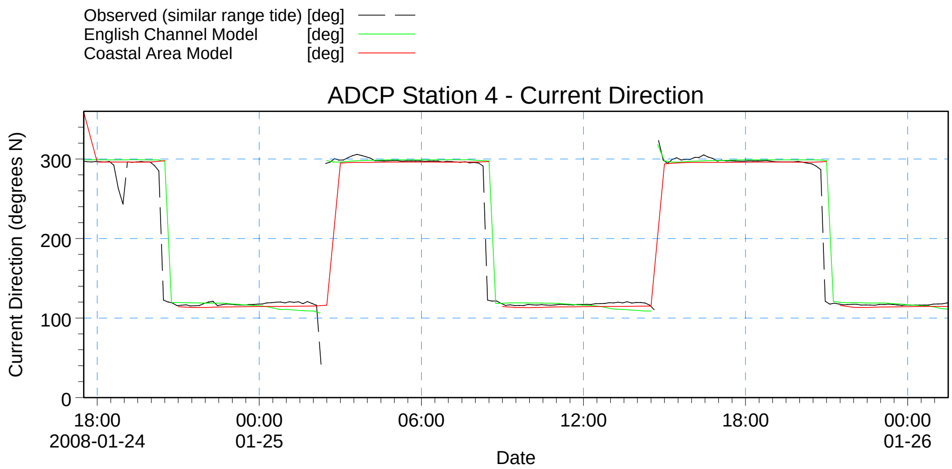
Calibration for current speeds at ADCP Station 4 is improved relative to the regional hydrodynamic model due to the higher resolution bathymetry (Figure 7-20). However, it should be noted that there are no measurements at ADCP Station 4 over the simulation period used, so observations from a similar range tide have been used for comparison. Over this period the velocity on the flood tide is lower than in the observations and outside the Environment Agency guidance; however this may be due to differences in the tides over the simulation and data periods. This underestimation means that the model has greater flood ebb asymmetry than observed.

Level measurements within the model area are only available at Exmouth Dock. Levels are similar to those in the English Channel model with a phase lag of around one hour on the flood tide and 0.5 hours on the ebb tide (Figure 7-20; bottom panel). The RMS error at Exmouth Dock is greater than the Environment Agency modelling guidelines for coastal area models that levels should be correct to within 0.1 m, however much of this discrepancy can be attributed to the phase lag. Sensitivity tests to the roughness within the estuary showed a slight improvement to the phase lag on the ebb tide from increasing the roughness within the estuary.

Figure 7-22 shows that over the interval shown, peak speeds at ADCP Station 6 are lower in the Exmouth coastal area model than in the English Channel model, and the calibration is poorer than in the English Channel model. However calibration at ADCP Stations 5 and 6 is within the Environment Agency guidelines for coastal area models that current speeds should be correct to within 0.1 m/s or to within 10-20% of observed speeds. The model also captures some of the flood ebb asymmetry at ADCP Station 6 with higher velocities on the flood than the ebb tide; however under-prediction on the flood tide is greater than on the ebb tide so that the asymmetry is reduced.

In summary, calibration for current speed and direction is within the Environment Agency guidelines at all locations and key features such as the flood ebb asymmetry are captured. Calibration for level at Exmouth Dock is outside the Environment Agency guidelines however given the complicated nature tides within Lyme Bay and the wider English Channel the model calibration is considered acceptable.





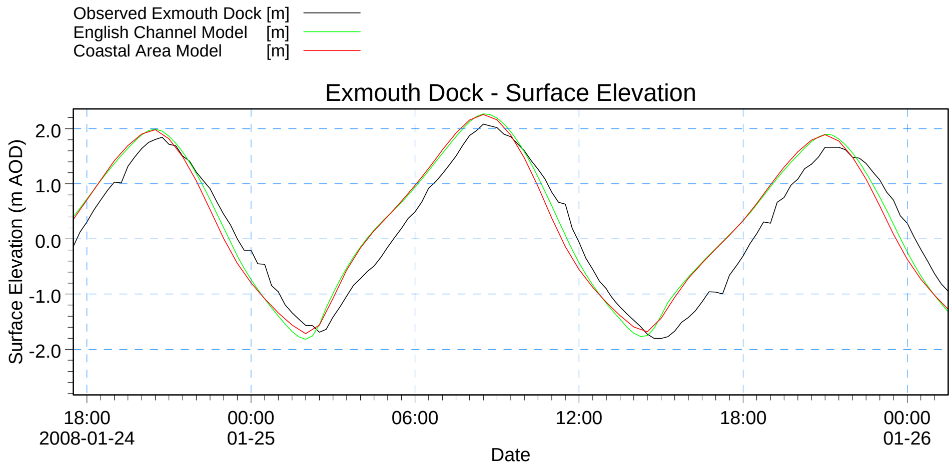


Figure 7-20: Hydrodynamic coastal area model calibration at ADCP Station 4. There are no observations at ADCP Station 4 for the simulation period so a similar range tide is shown. ADCP Station 4 is adjacent the entrance to Exmouth Dock, and level at Exmouth Dock is shown because the datum for ADCP Station 4 is not given.

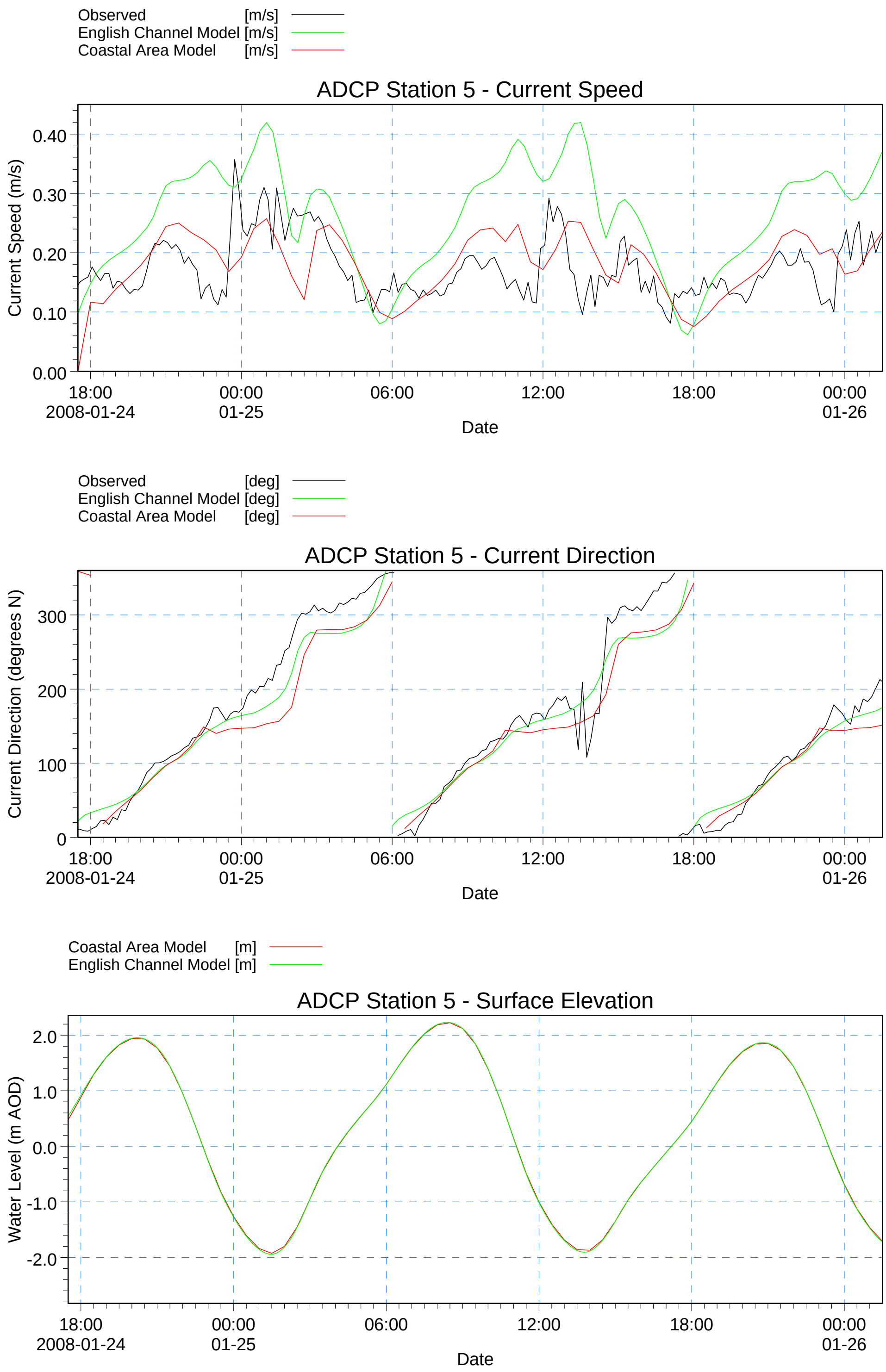
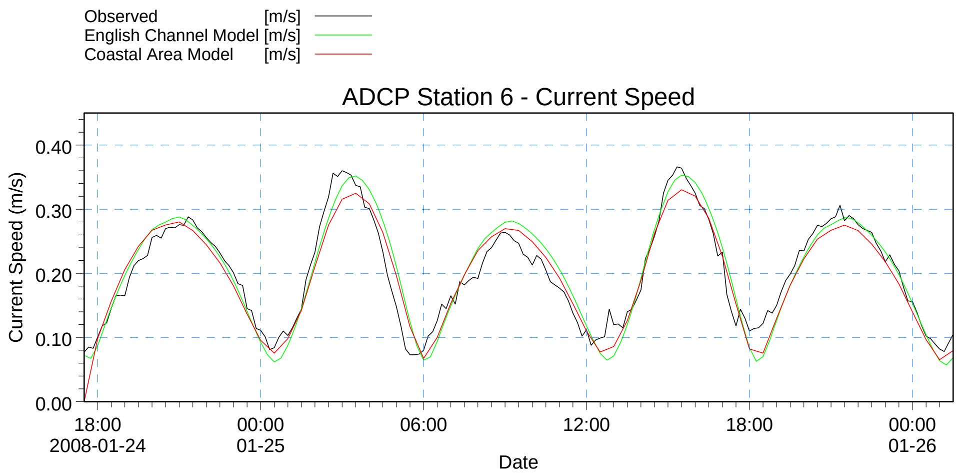
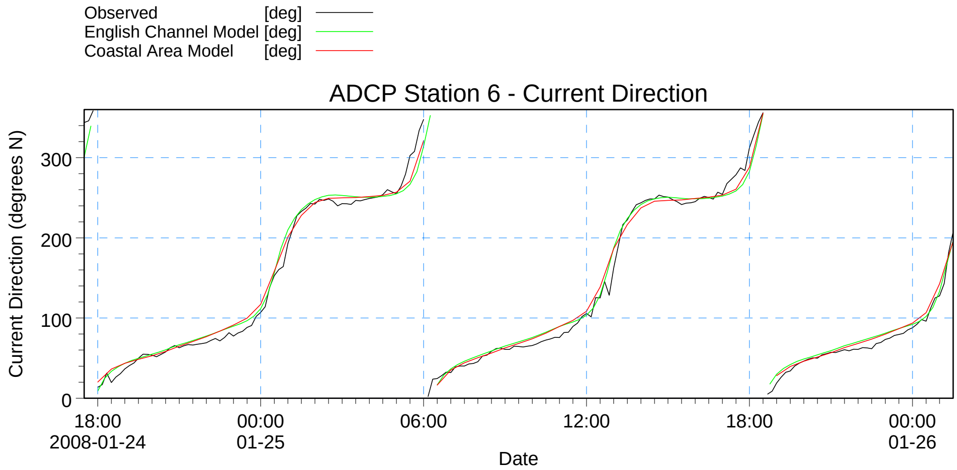


Figure 7-21: Hydrodynamic coastal area model calibration at ADCP Station 5.





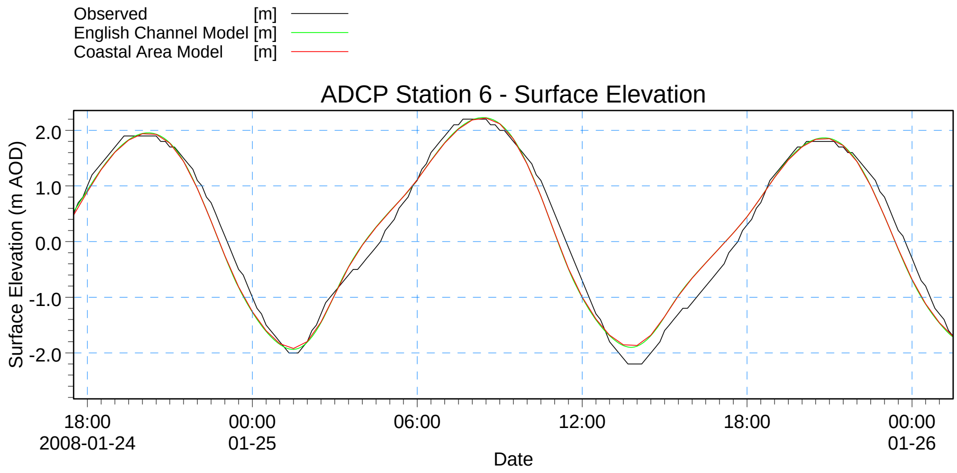


Figure 7-22: Hydrodynamic coastal area model calibration at ADCP Station 6. Only depth measurements are available at ADCP Station 6, so the elevation has been adjusted to give the best fit with the model.

7.3.3.3 Coupled Model Flow Results

Velocity in the tide only hydrodynamic model is shown for four states of the tide in the Figure 7-24. The velocity in the cases with representative waves from 45° to 255° North is shown in the bottom four panels of Figure 7-25 to Figure 7-32. All velocity snapshots are from 25/01/2008; 02:30 corresponds to low water at Exmouth Dock and peak flood at ADCP Station 6 in Lyme Bay; 04:30 corresponds to the stand on the rising tide at Exmouth Dock; 08:30 corresponds to high water at Exmouth Dock; and 11:30 corresponds to the peak ebb tide at Exmouth Dock. For convenience a map showing key locations referred to in the text is shown in Figure 7-23.

Figure 7-23: Key locations referred to in the text.

Results for representative waves from 45°N, Figure 7-25, 225°N, Figure 7-31 and 255°N, Figure 7-32, are similar to calm conditions due to the small waves from these directions.

For representative waves from 75° and 105°N (Figure 7-26 and Figure 7-27), south westwards wave driven currents are set up along Dawlish Warren at south of Pole Sand. At low water this causes a clockwise eddy to develop off Langstone Rock as the currents are deflected around the rock. Pole Sand partially shelters Dawlish Warren from waves from 75° and 105°N so shorewards of Pole Sand wave driven currents are reduced and the tidal flow through the low water channel adjacent to

Dawlish Warren is dominant. Along Exmouth Beach, east of Orcombe Rocks, westwards wave driven currents are set up for these representative wave conditions. At low water (02:30) westwards wave driven currents are also set up along the southern edge of Pole Sand. For waves from 105° this results in continuous westwards flow along from east of Orcombe Rocks, to Dawlish Warren at low water (02:30 on Figure 7-27). On the ebb tide, convergence of the westwards wave driven currents from east of Orcombe Rocks with the main ebb flow over Pole Sand leads to southwards flows at the eastern edge of Dawlish Warren are increased (11:30 on Figure 7-27) .

For representative waves from 135°N to 195°N, north-eastwards wave driven currents are set up along Dawlish Warren (Figure 7-28 to Figure 7-30). At low water westwards wave driven currents are setup along the southern edge of Pole Sand (02:30 on Figure 7-28 to Figure 7-30). For the representative waves from 135 °N, westwards wave driven currents are also set up across the middle of Pole Sand at high water (08:30 on Figure 7-28). The westwards wave driven currents south of Pole Sand interact with the north-eastwards wave driven currents along Dawlish Warren. At low water, this causes an intensification of the eddy off Langstone Rock and areas of convergent flow along the Dawlish Warren. For representative waves from 135°N, at 4:30 the eddy off Langstone Rock re-circulates the flow from across Pole Sand back onto Dawlish Warren. For representative waves from 135°N to 195°N, westwards wave driven currents are set up along Exmouth Beach at high water (8:30 on Figure 7-28 to Figure 7-30) . For representative waves from 165°N and 195°N, flow patterns at low water around Orcombe Rocks are highly two-dimensional (2:30 and 4:30 on Figure 7-29 and Figure 7-30 respectively).

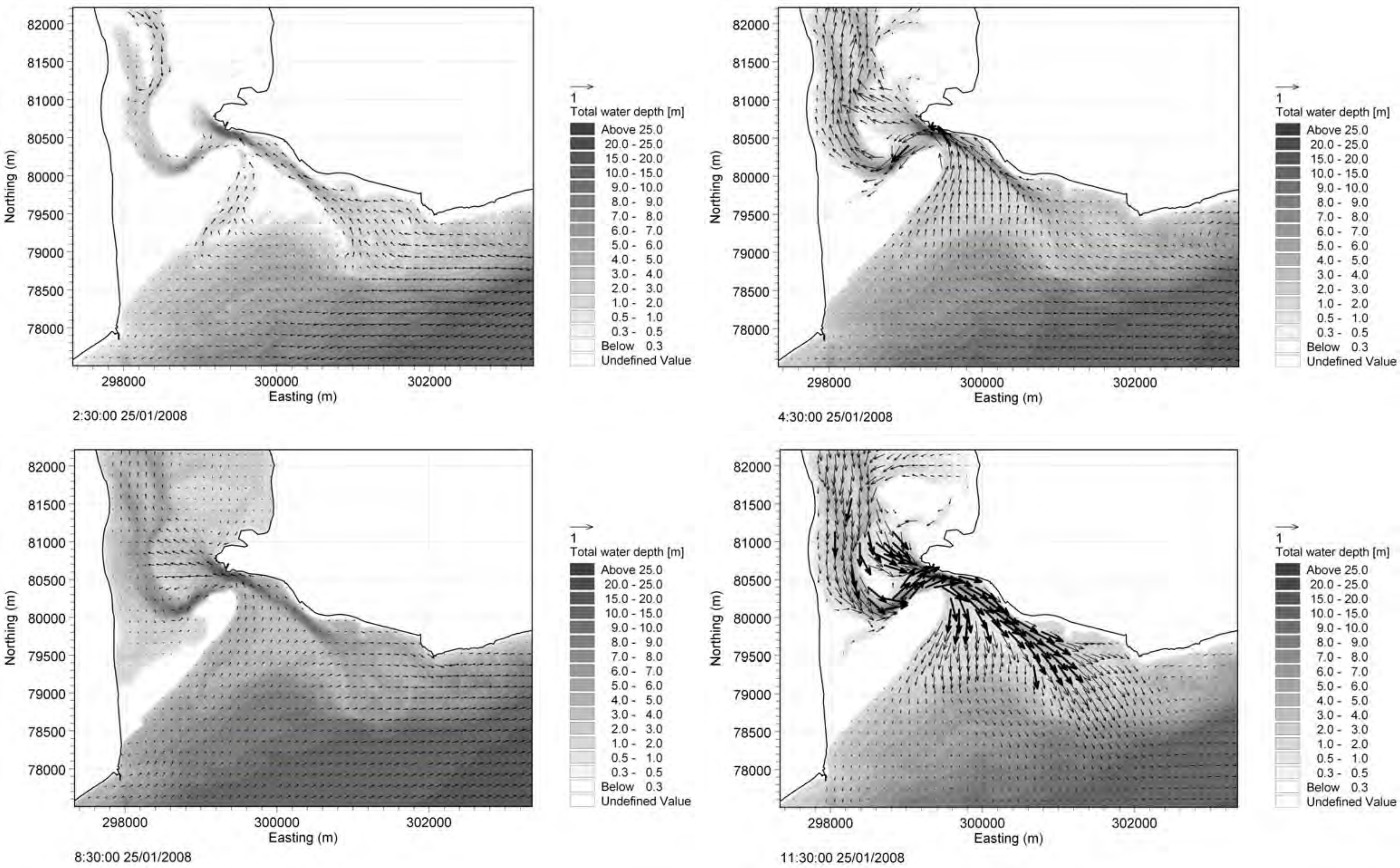


Figure 7-24: Velocity fields for calm conditions, 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.

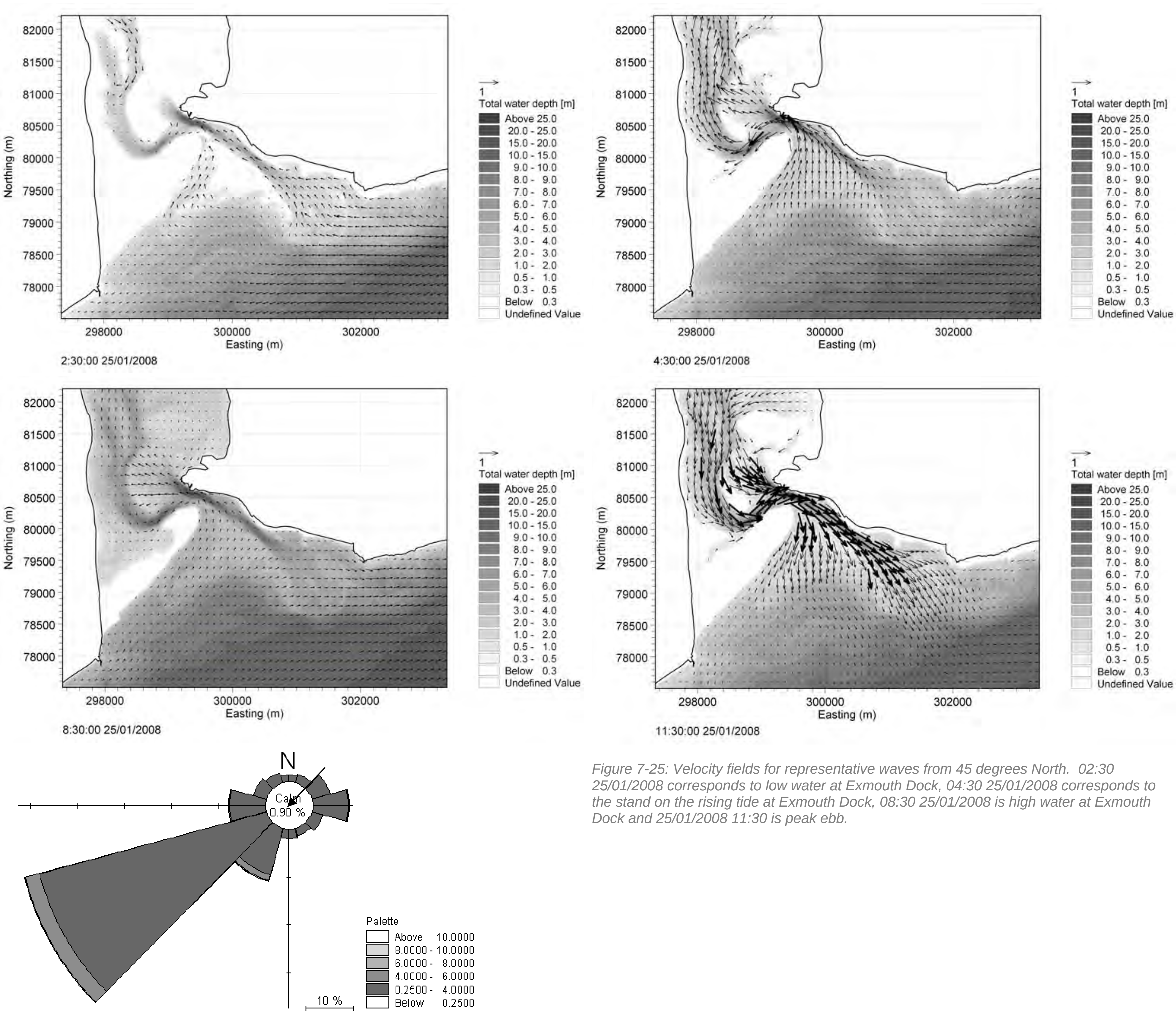
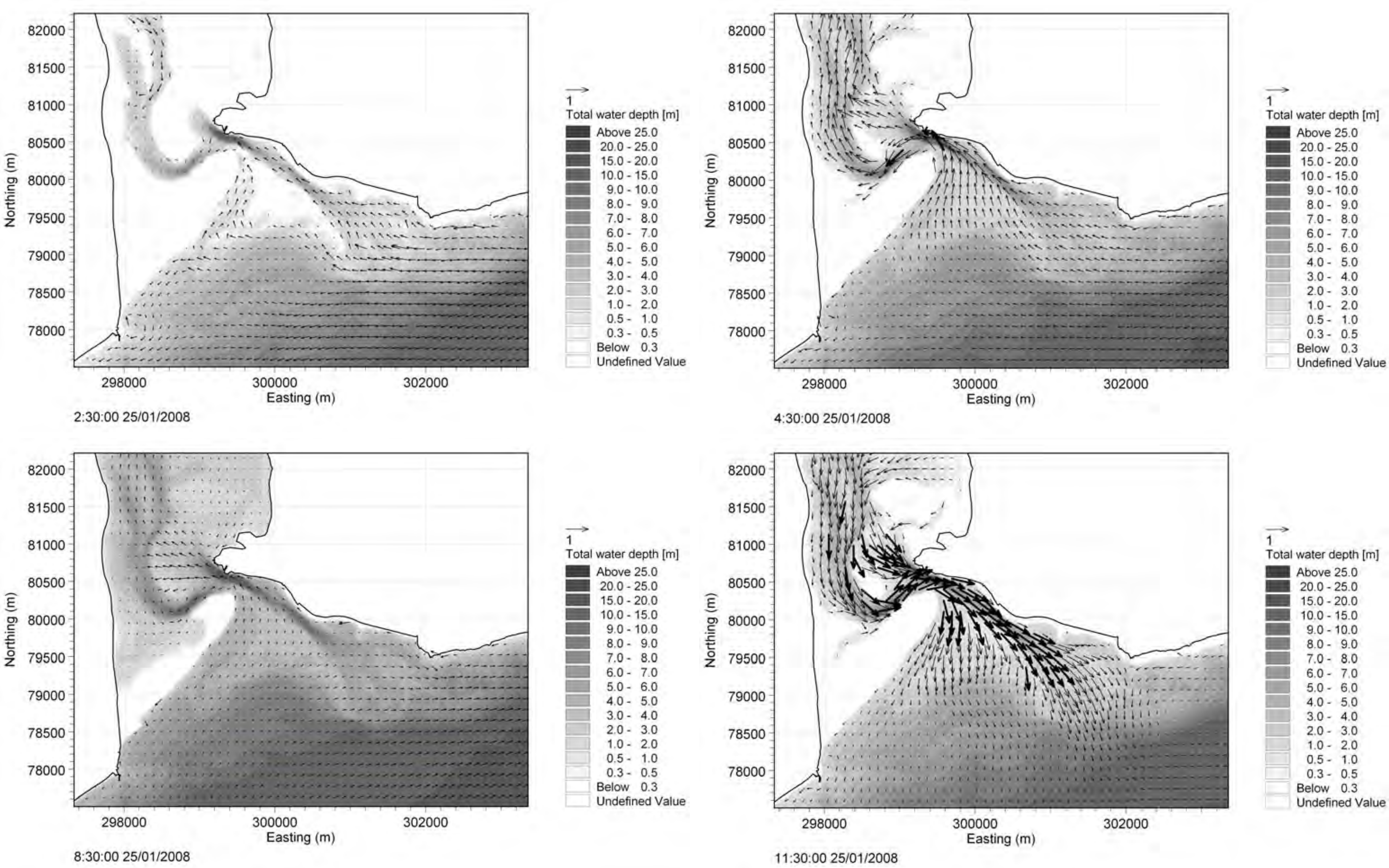


Figure 7-25: Velocity fields for representative waves from 45 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



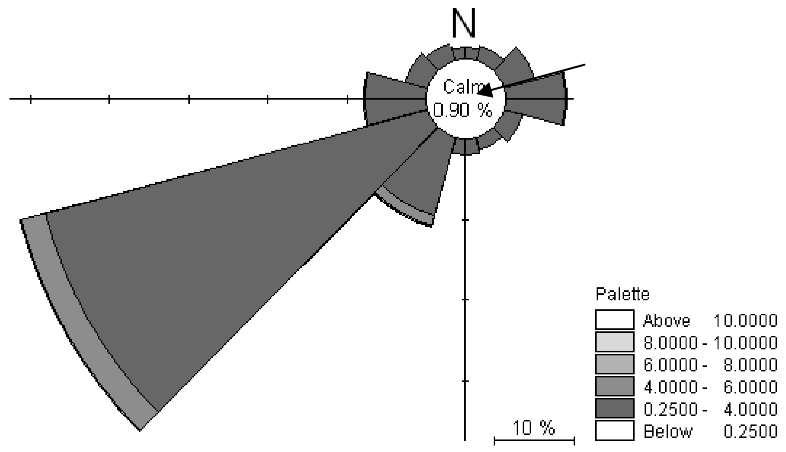


Figure 7-26: Velocity fields for representative waves from 75 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.

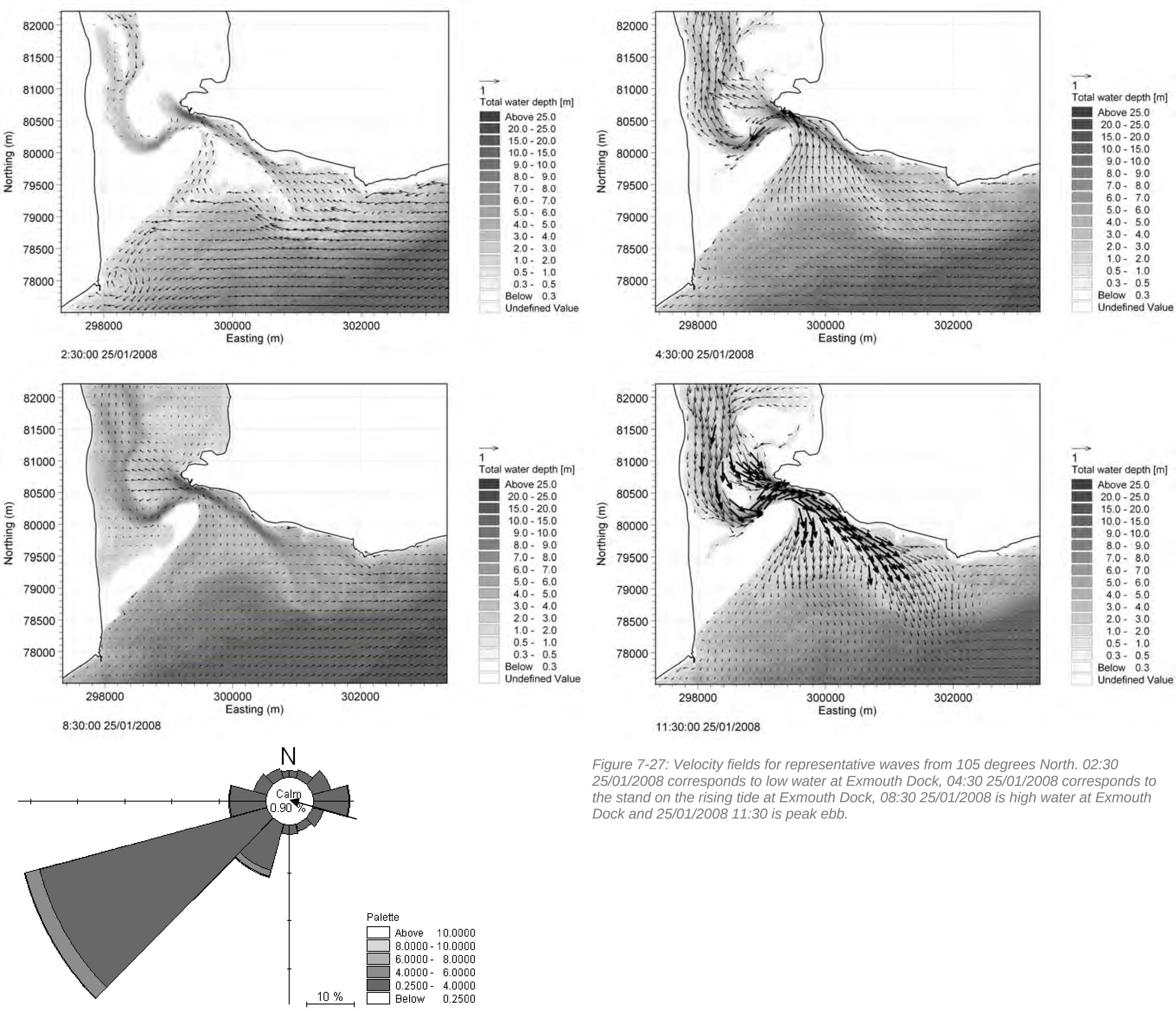
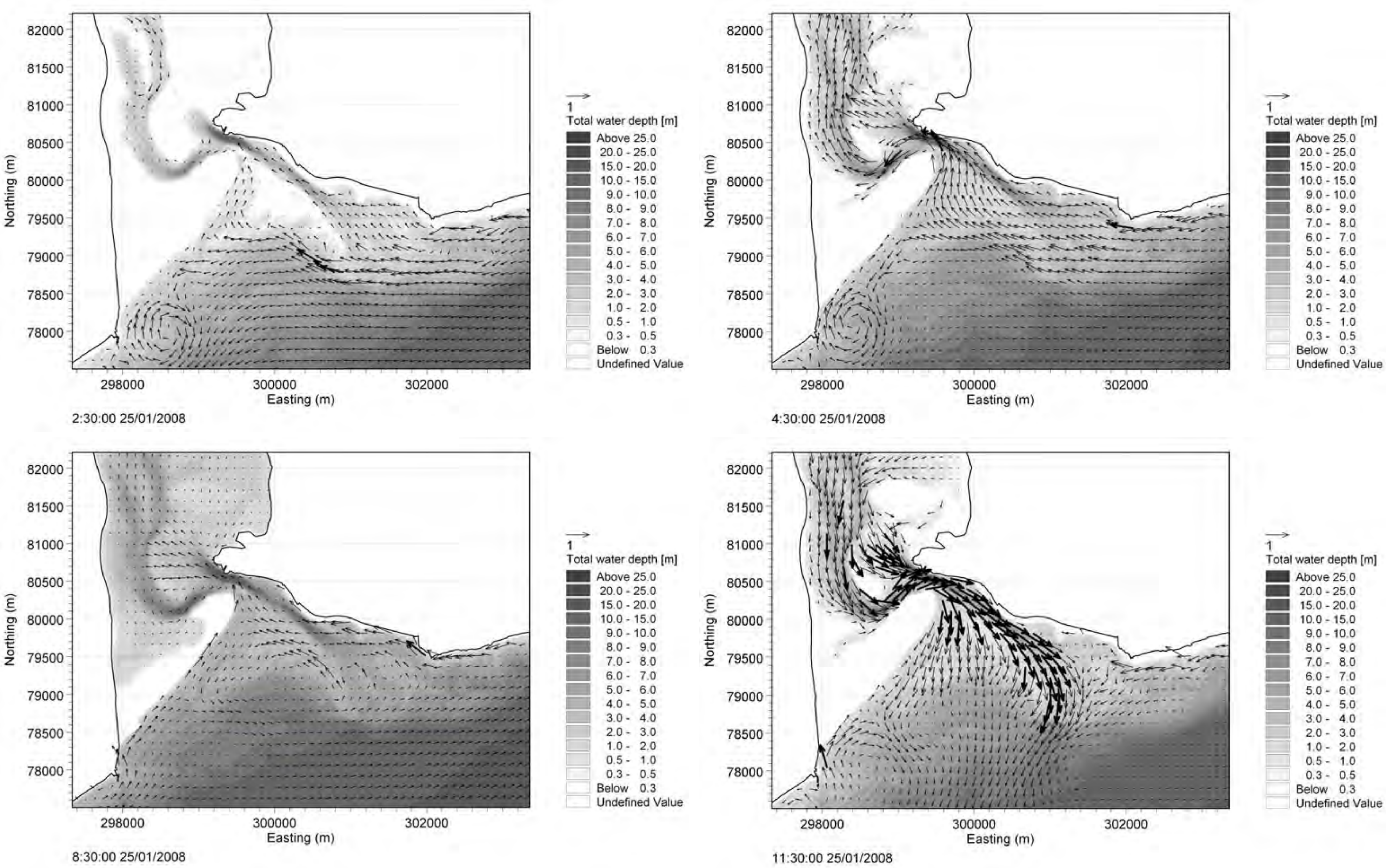


Figure 7-27: Velocity fields for representative waves from 105 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



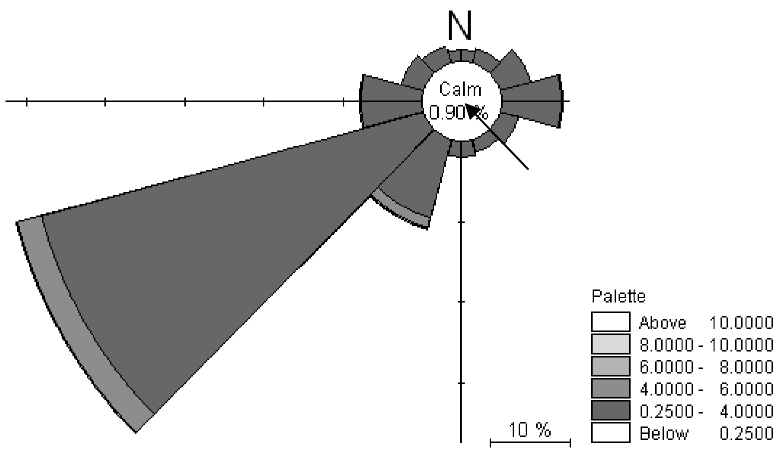
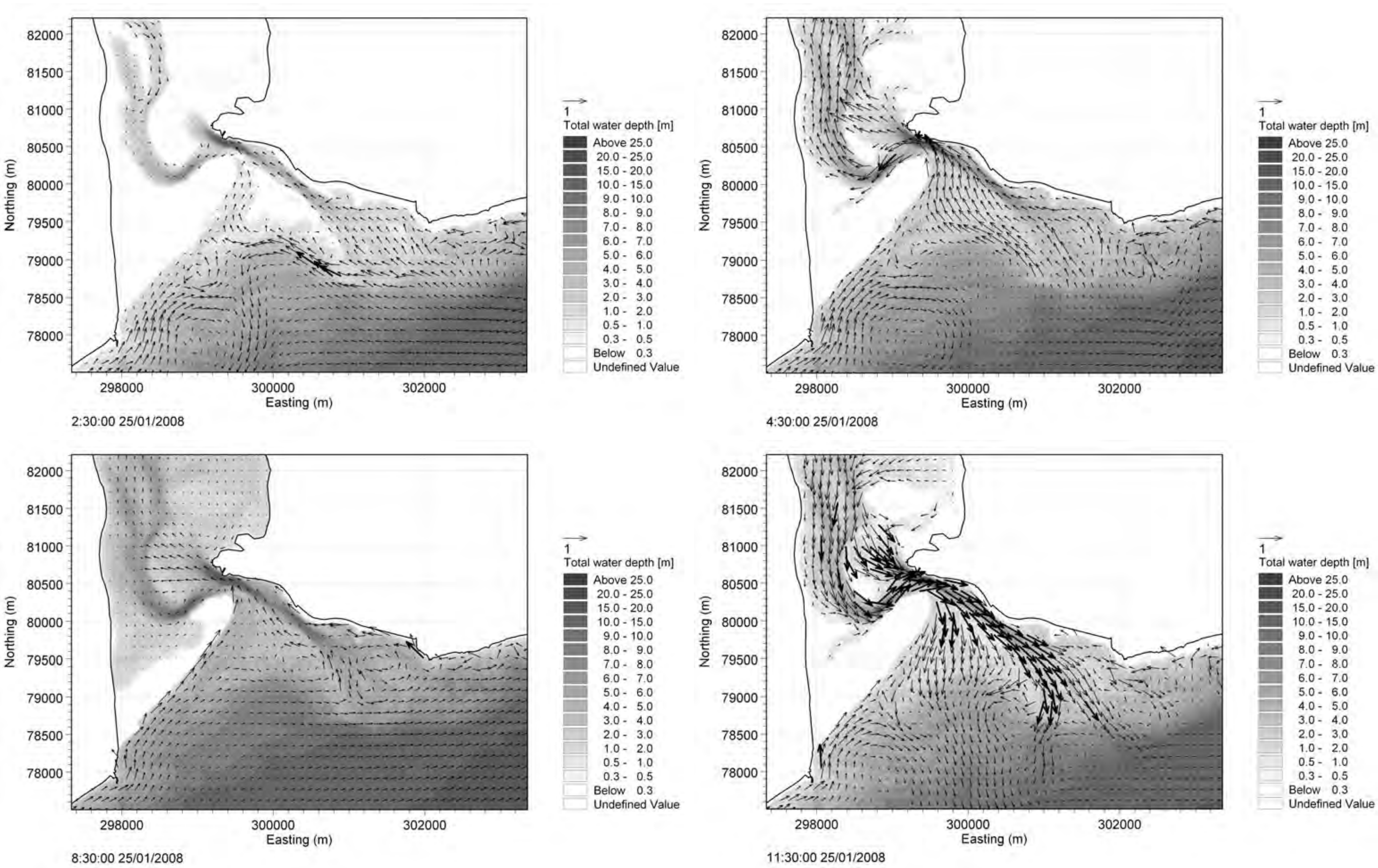


Figure 7-28: Velocity fields for representative waves from 135 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



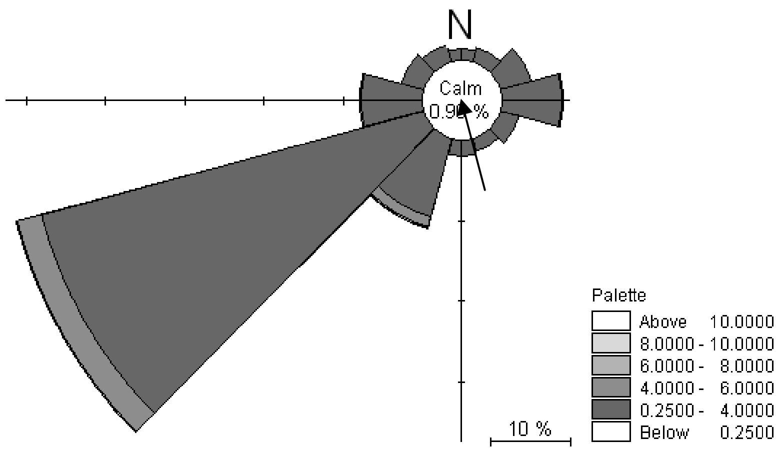
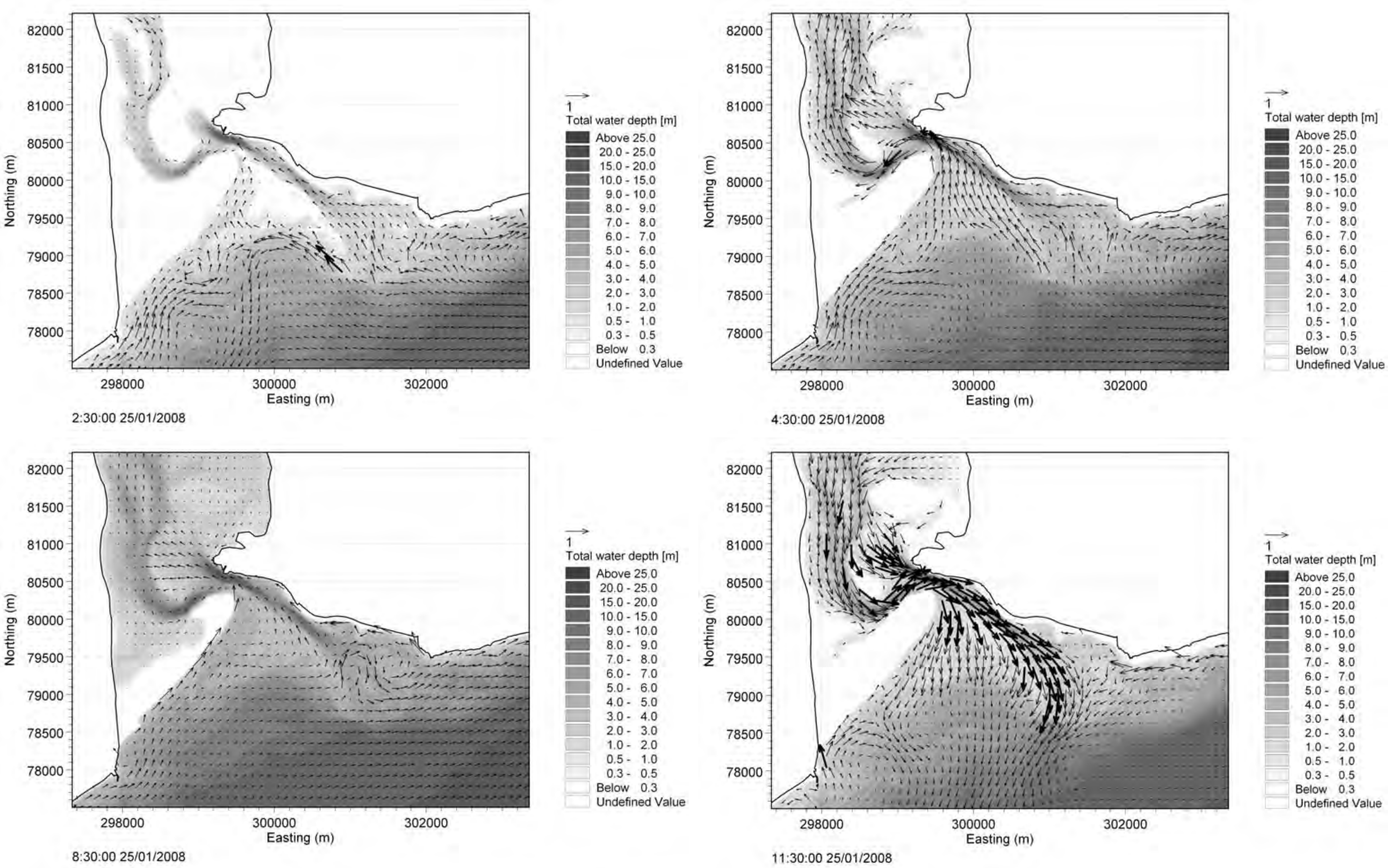


Figure 7-29: Velocity fields for representative waves from 165 degrees North. The bottom four panels show velocity at 4 states of the tide. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



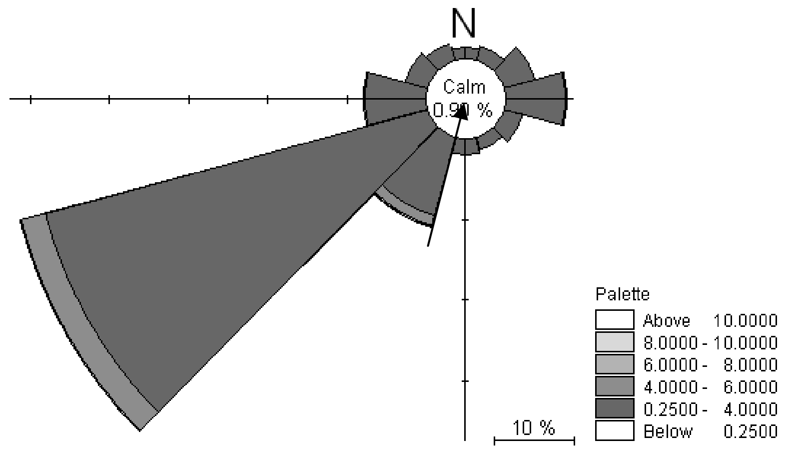
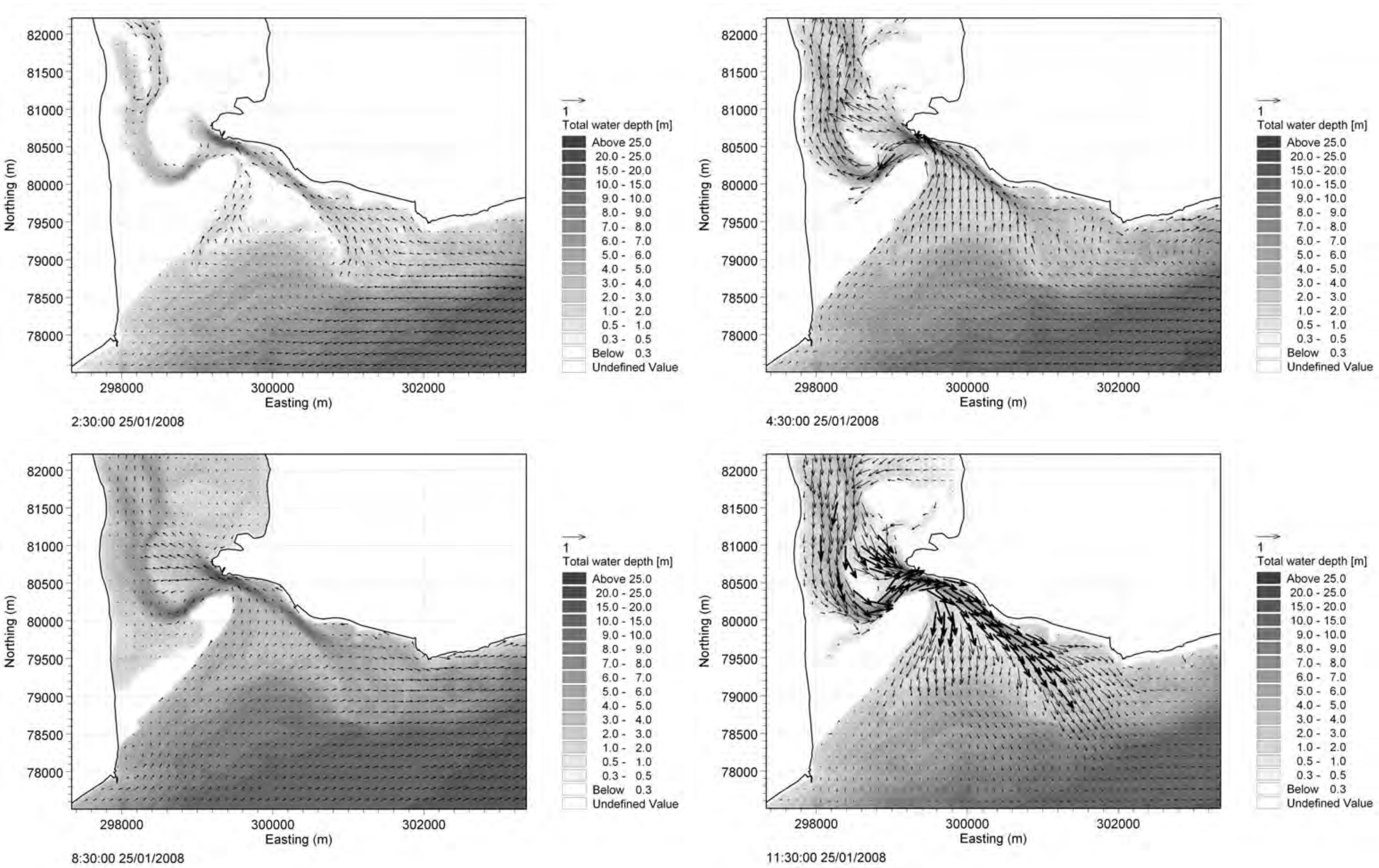


Figure 7-30: Velocity fields for representative waves from 195 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



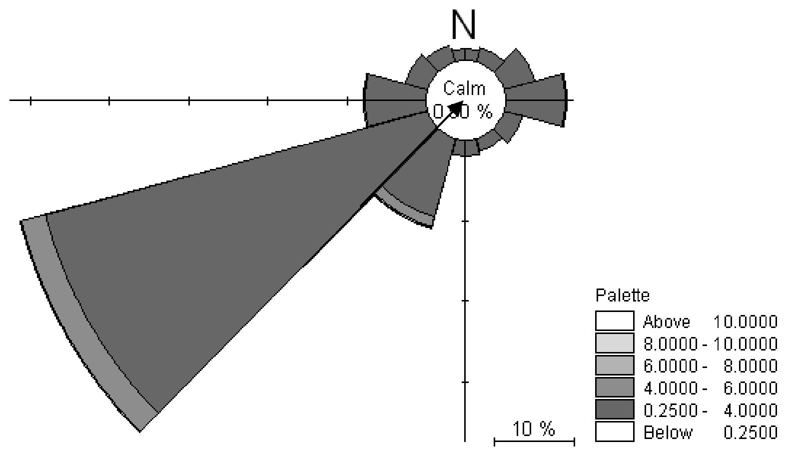
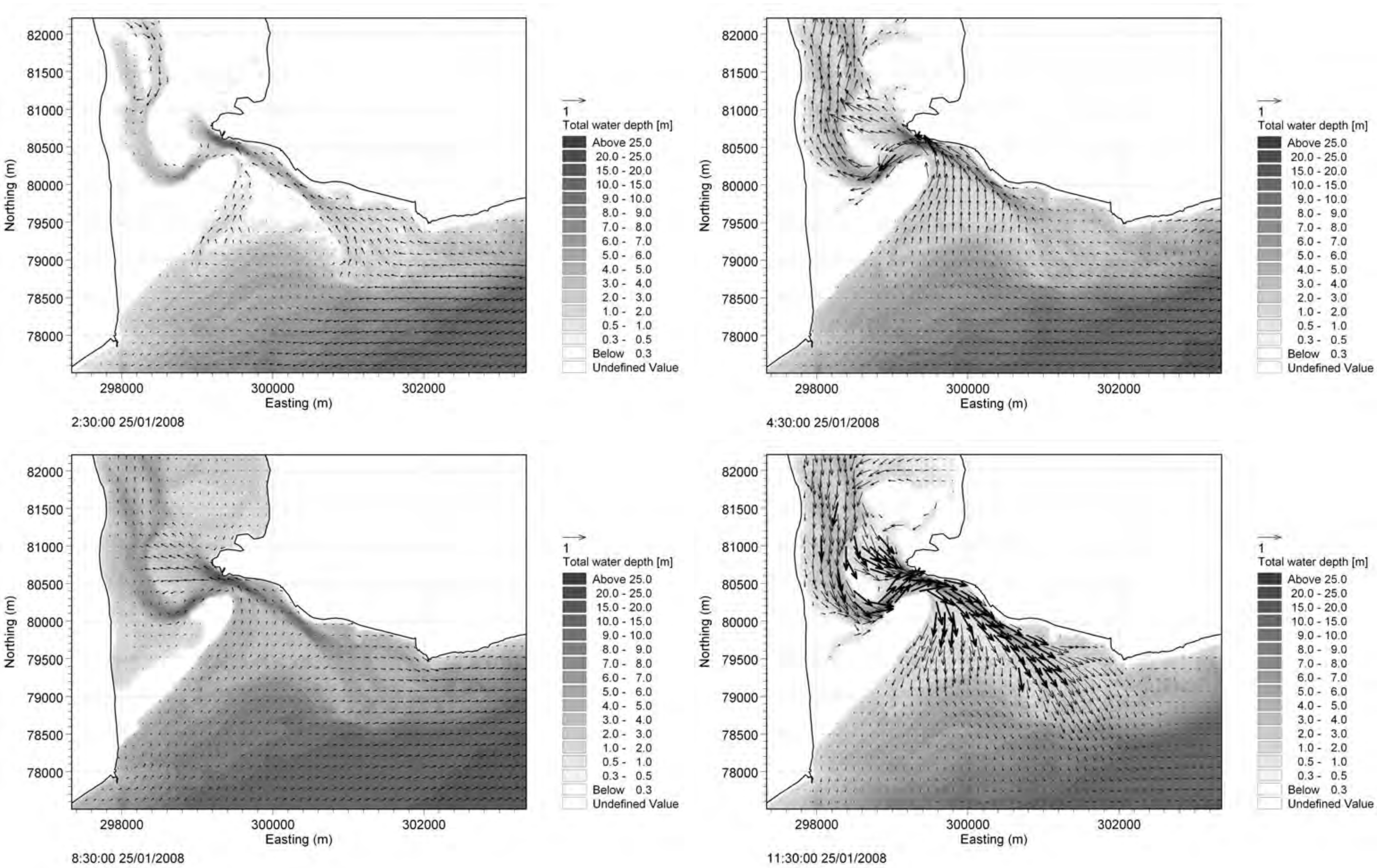


Figure 7-31: Velocity fields for representative waves from 225 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.



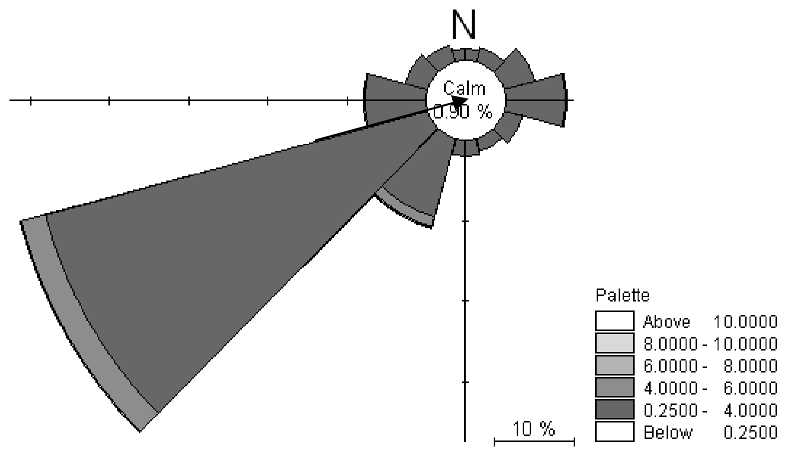


Figure 7-32: Net sediment transport and velocity fields for representative waves from 255 degrees North. 02:30 25/01/2008 corresponds to low water at Exmouth Dock, 04:30 25/01/2008 corresponds to the stand on the rising tide at Exmouth Dock, 08:30 25/01/2008 is high water at Exmouth Dock and 25/01/2008 11:30 is peak ebb.

7.4 Sediment Transport Model

7.4.1 General Approach

Potential sediment transport has been modelled using the MIKE 21 Non-Cohesive Sediment Transport (ST) model. This calculates the sediment transport rates and initial rates of bed level change for non-cohesive sediment resulting from currents or combined wave-current flows. The model is two-dimensional but contains parameterisations of three-dimensional processes.

7.4.2 Sediment Size and Distribution

Sediment sampling has been carried out for Dawlish Warren, Exmouth Beach and Pole Sands (Section 2.5.2 and Section 2.5.3). A constant sediment size of 0.3 mm and a grading coefficient of 1.5 has been used for the sediment transport model as this is thought to be most representative of the sediment size on the intertidal areas. Sediment samples for Exmouth Beach and Dawlish Warren show that the sediment size for the beaches is larger than over Pole Sands, a larger sediment size (1-3 mm) has therefore been used for the shoreline evolution modelling in Section 5.

Areas marked as rocks on the OS 1:10000 maps have been assigned a zero sediment depth, elsewhere a constant depth of 10m has been assumed due to lack of information about the spatial distribution of sediment depth over the area. Velocities on the ebb tide through the low water channel at the mouth of the Exe Estuary are high, it is possible that sediment depths in this area are reduced and that the seabed sediment size is coarser that the 0.3 mm assumed elsewhere. As a result potential sediment transport in this area may be overestimated.

7.4.3 Model Definition

The sediment transport model has been forced with both wave and current data from the spectral wave model and hydrodynamic model. In order to reduce the computational time involved with solving the complicated transport equations at each time-step sediment transport lookup tables have been pre-calculated for a range of current and wave conditions and used to calculate the transport.

The model uses a quasi-three-dimensional representation of sediment dynamics which uses the depth averaged velocity from the hydrodynamic model but includes parameterisations of three-dimensional transport processes. Using the representative waves described in Section 7.2.2 only the large wave events which are responsible for the majority of longshore transport are modelled. These wave events cause significant erosion due to cross-shore transport which is included in the quasi-three-dimensional representation of sediment transport. Smaller wave events which play an important role in beach building due to cross-shore transport are not included in the representative waves modelled. As a result erosion due to cross-shore transport is overestimated. Furthermore, the sediment size in the upper part of the beach is likely to be significantly coarser than the 0.3 mm assumed; this will overestimate sediment transport close to the shoreline. The results are analysed by taking cross-sections perpendicular to the shore and resolving transport into longshore and cross-shore components which can be analysed separately. Only the longshore sediment transport results have been used in inferring potential shoreline changes.

The model has been run for a period of two tides only and the bed level has not been allowed to evolve, there is therefore no feedback from the sediment transport model to the hydrodynamic model.

7.5 Results

7.5.1 Average Annual Transport

The average transport over two tides for each representative wave event is shown in Figure 7-33 to Figure 7-41. The annual average transport has been calculated by summing the two tide average transport for each representative wave event shown in using the relative frequencies in Table 7-1. These have been combined with the number of calm days to give the annual transport pattern.

7.5.2 General Features

Net annual sediment changes for areas within the estuary are shown in Figure 7-42, together with arrows showing the direction of the annual average sediment transport. Areas of net sediment loss are shown in pink, areas of net sediment gain are shown in green. On this scale the transport is generally tidally dominated with a net gain in sediment over the flood (Bull Hill) and ebb delta (Pole Sand), and a loss of sediment from the low water channels.

The model shows large cross-shore transport causing a net loss of sediment from Dawlish Warren and Exmouth Beach. As remarked in Section 7.4.3, cross-shore transport in the coastal area model is overestimated, because the model has only been run for larger wave events which will remove sediment from the beach, and simulations with the smaller beach building waves have not been run. Quantitative results for the annual cross-shore transport cannot be derived from the coastal area model results. Cross-shore transport for individual storm events is also likely to be higher than in the cross-shore modelling in Section 6, due to the smaller sediment size used in the coastal area model, which is more representative for offshore areas.

As discussed in Section 7.3.3.3 for representative wave conditions, wave driven currents are set up along the south of Pole Sand and along Exmouth Beach and Dawlish Warren and these lead to a highly two-dimensional flow patterns in this area. These flow patterns in turn lead to a highly two-dimensional sediment transport regime around Dawlish Warren and Exmouth Beach.

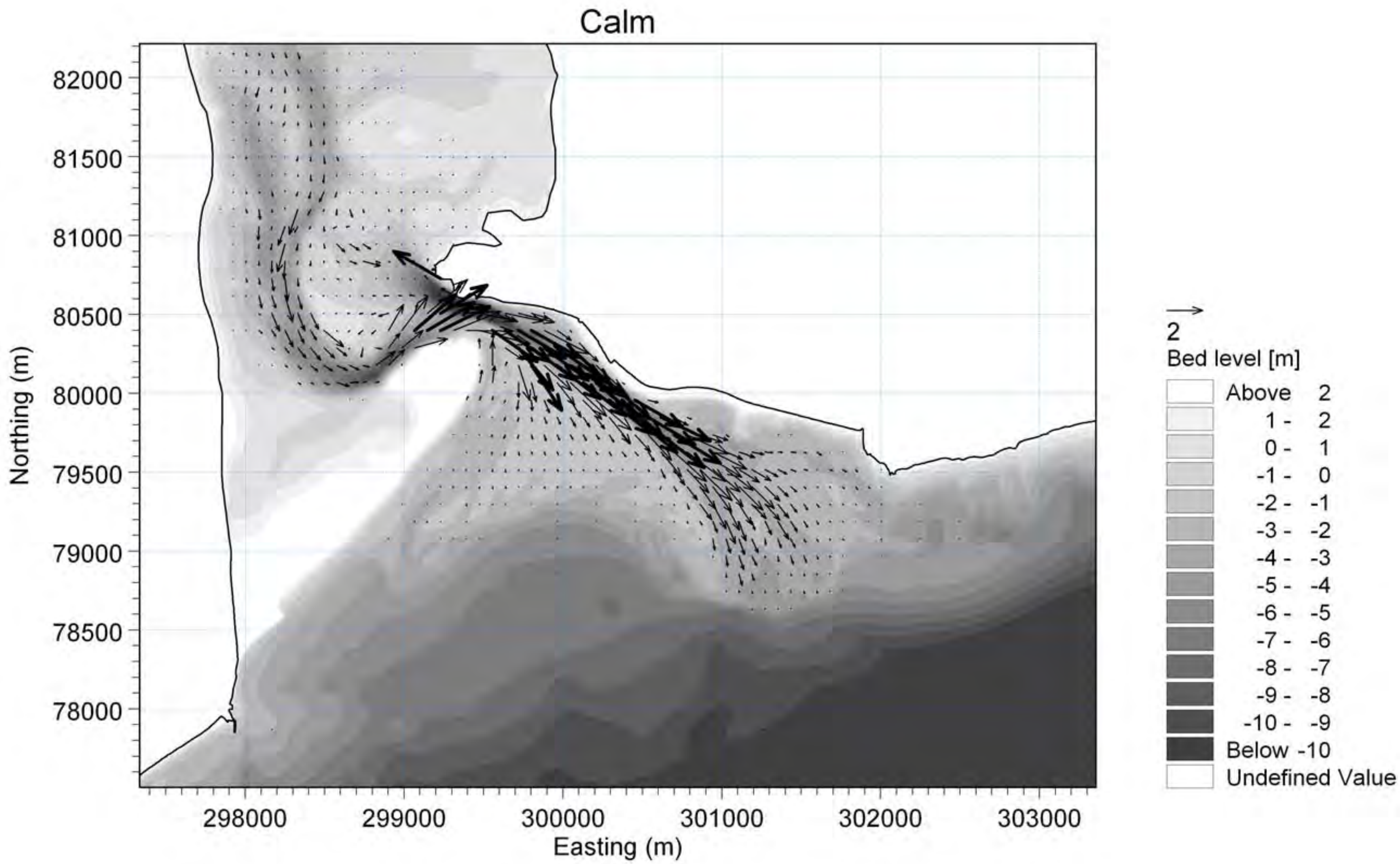


Figure 7-33: Net sediment transport for calm conditions.

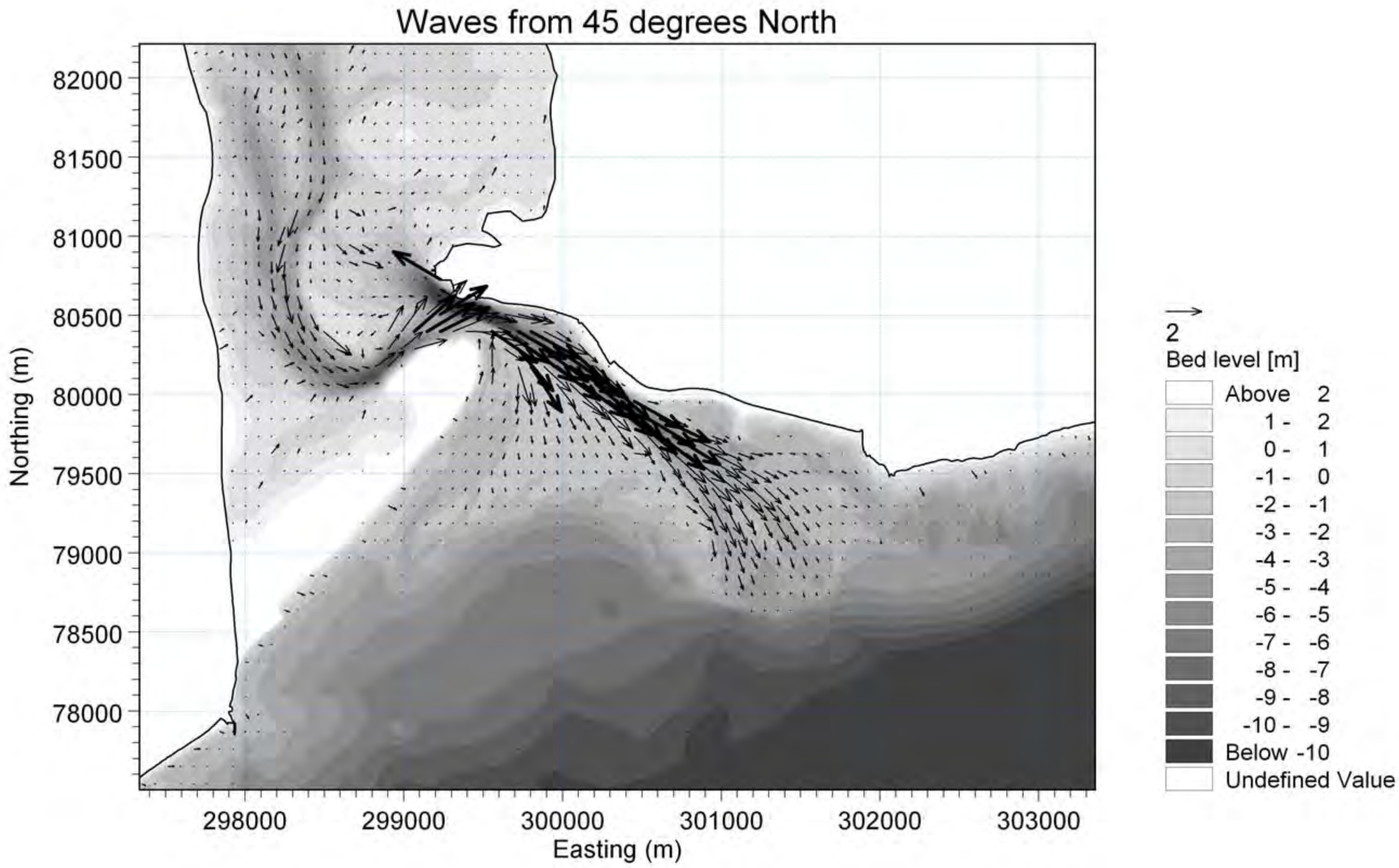


Figure 7-34: Net sediment transport for waves from 45°N.

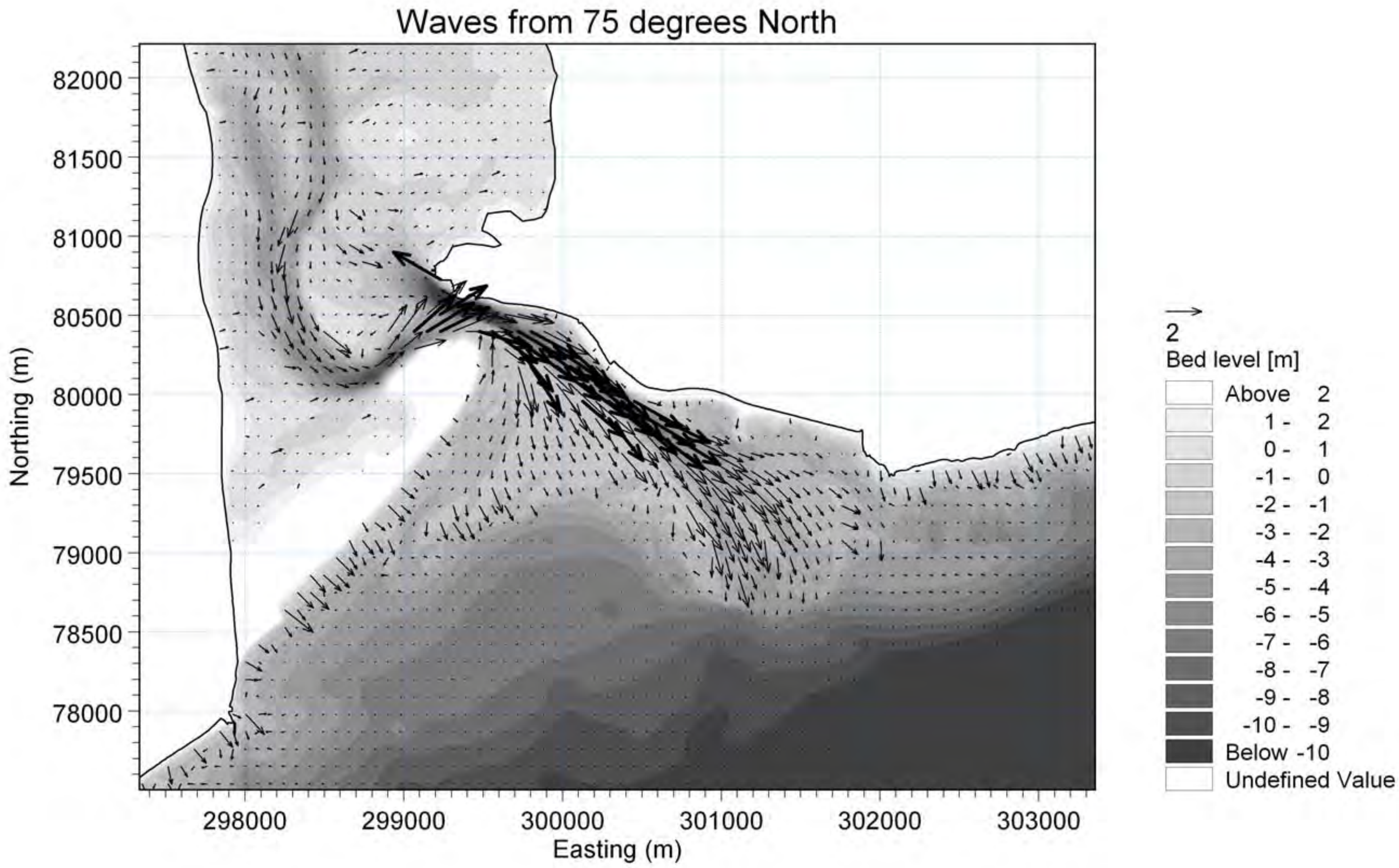


Figure 7-35: Net sediment transport for waves from 75°N.

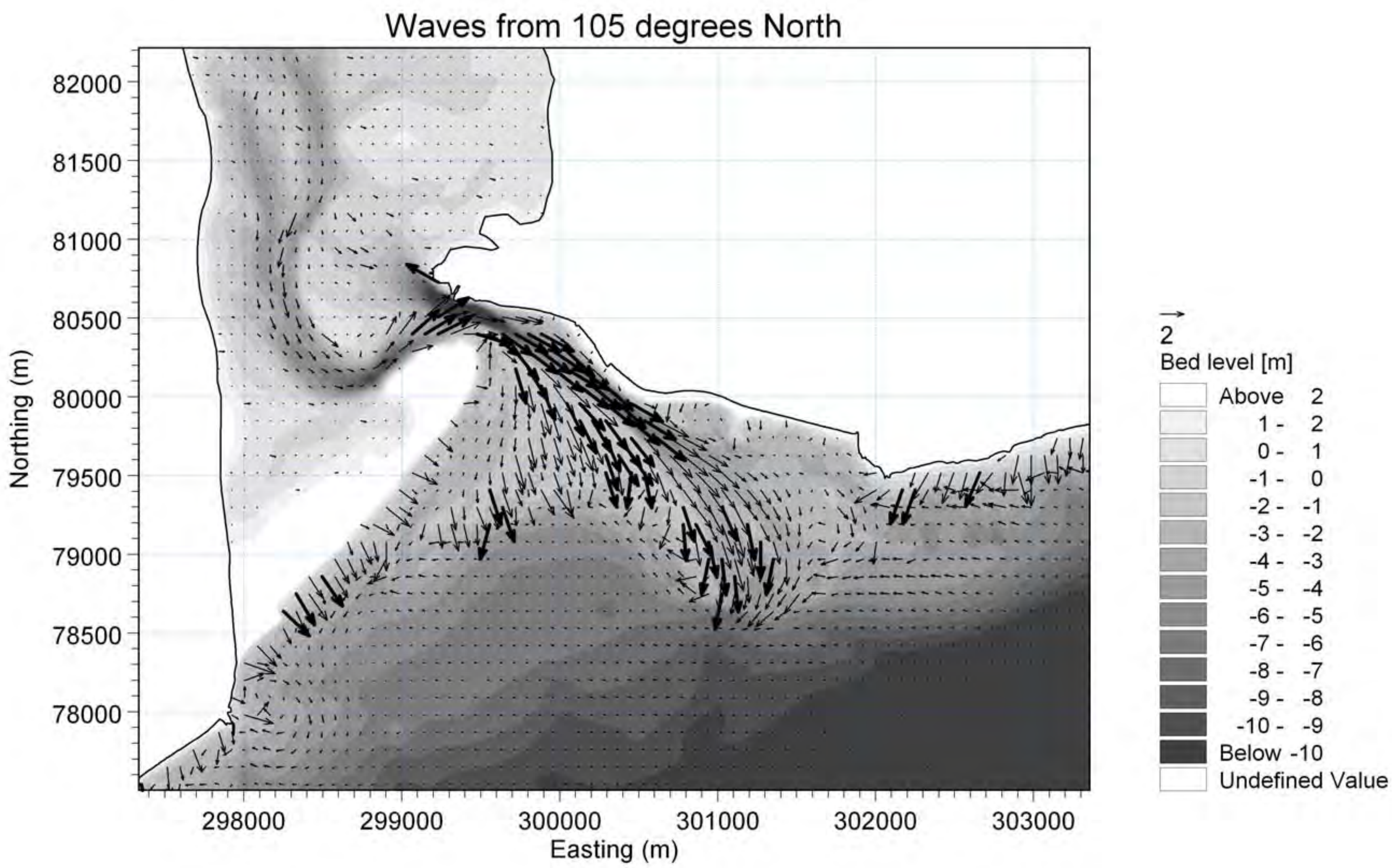


Figure 7-36: Net sediment transport for waves from 105°N.

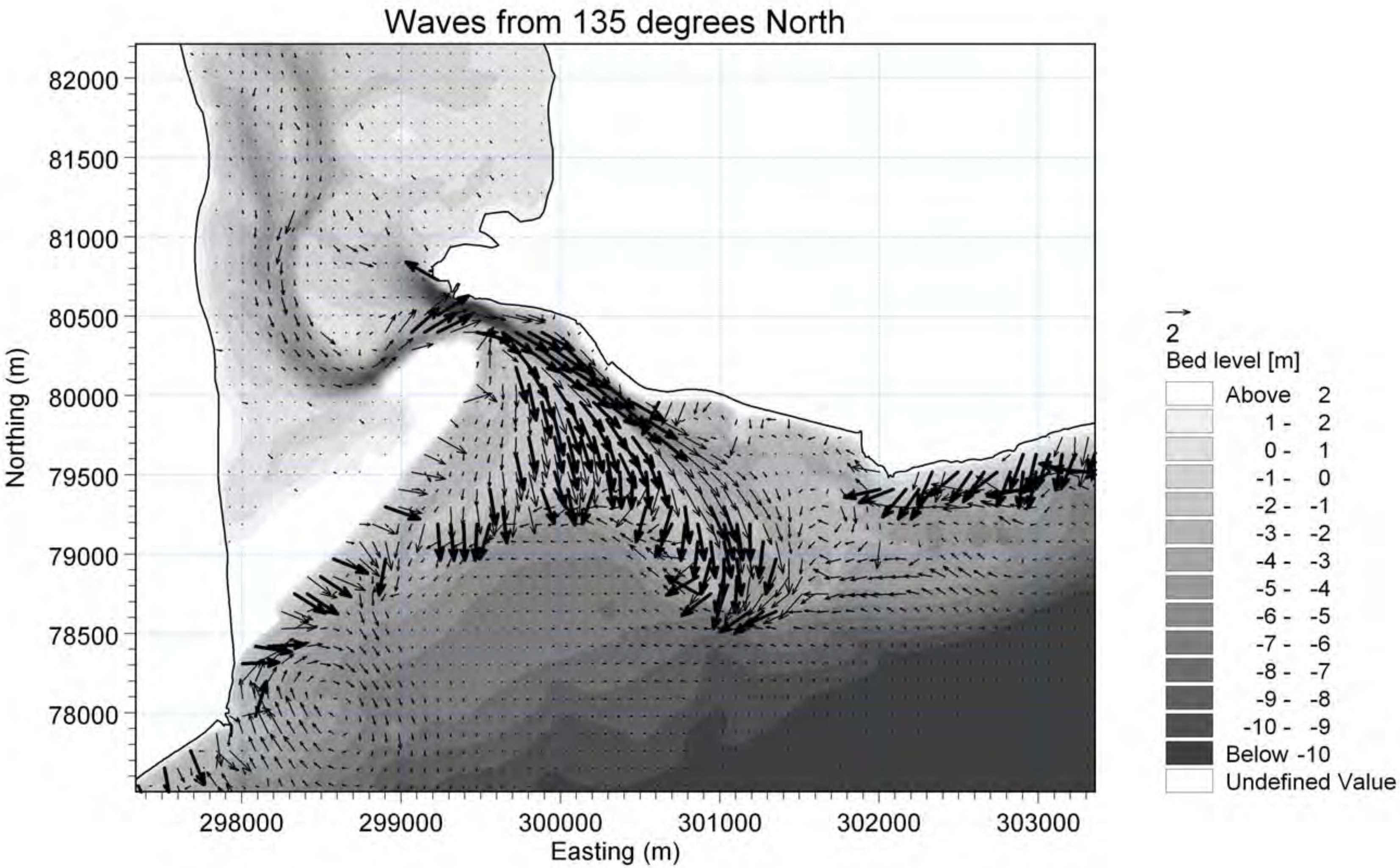


Figure 7-37: Net sediment transport for waves from 135°N.

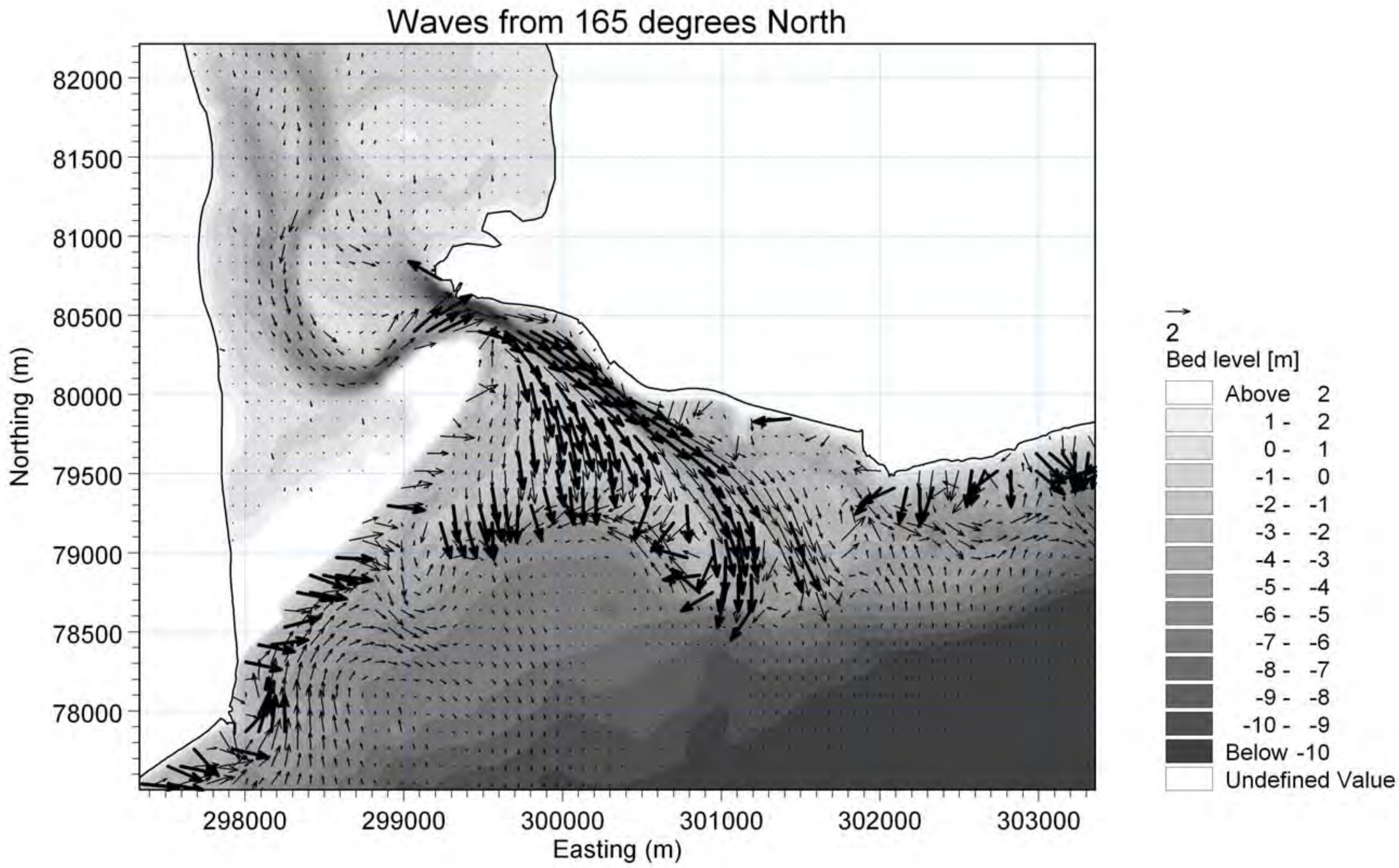


Figure 7-38: Net sediment transport for waves from 165°N.

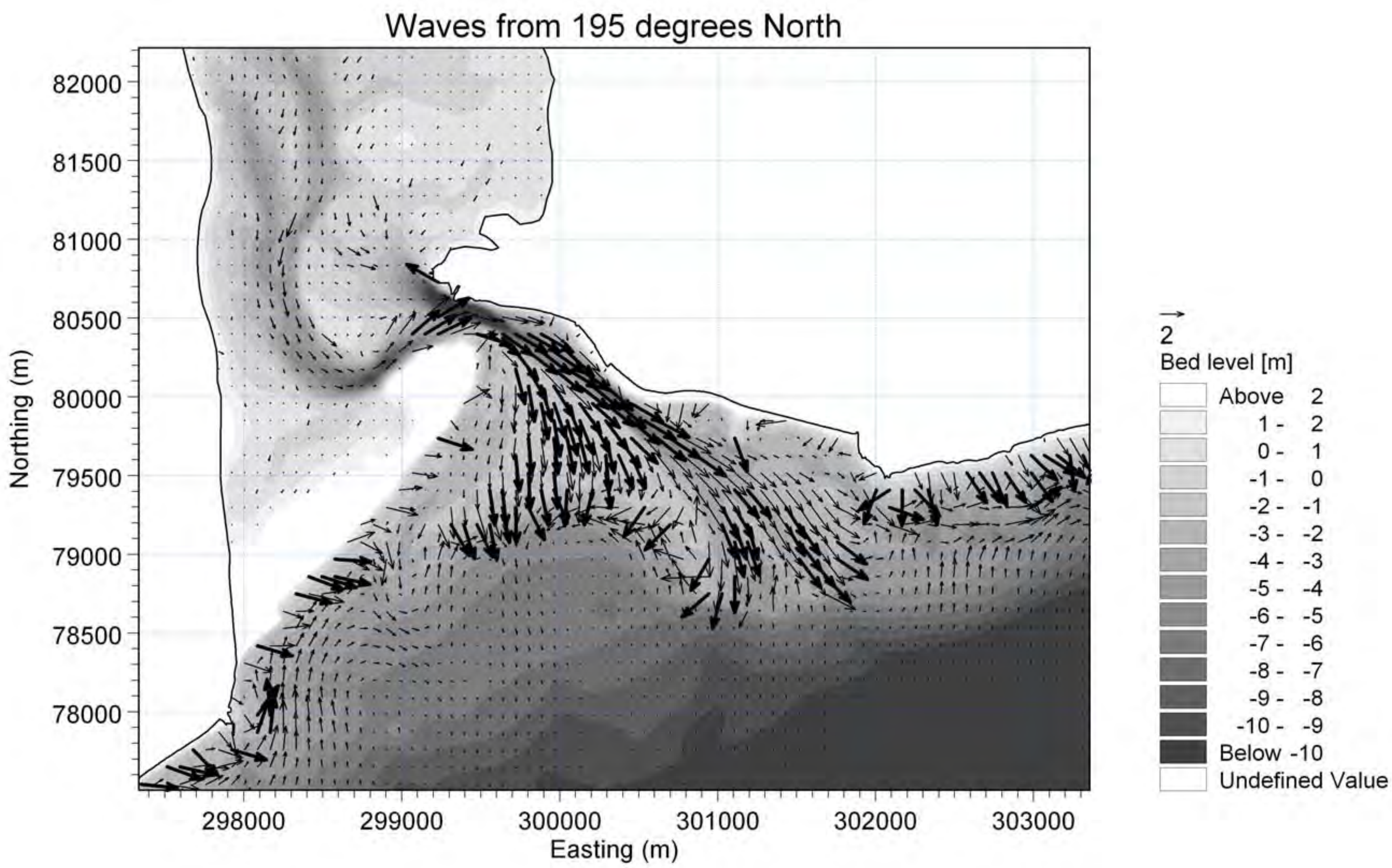


Figure 7-39: Net sediment transport for waves from 195°N.

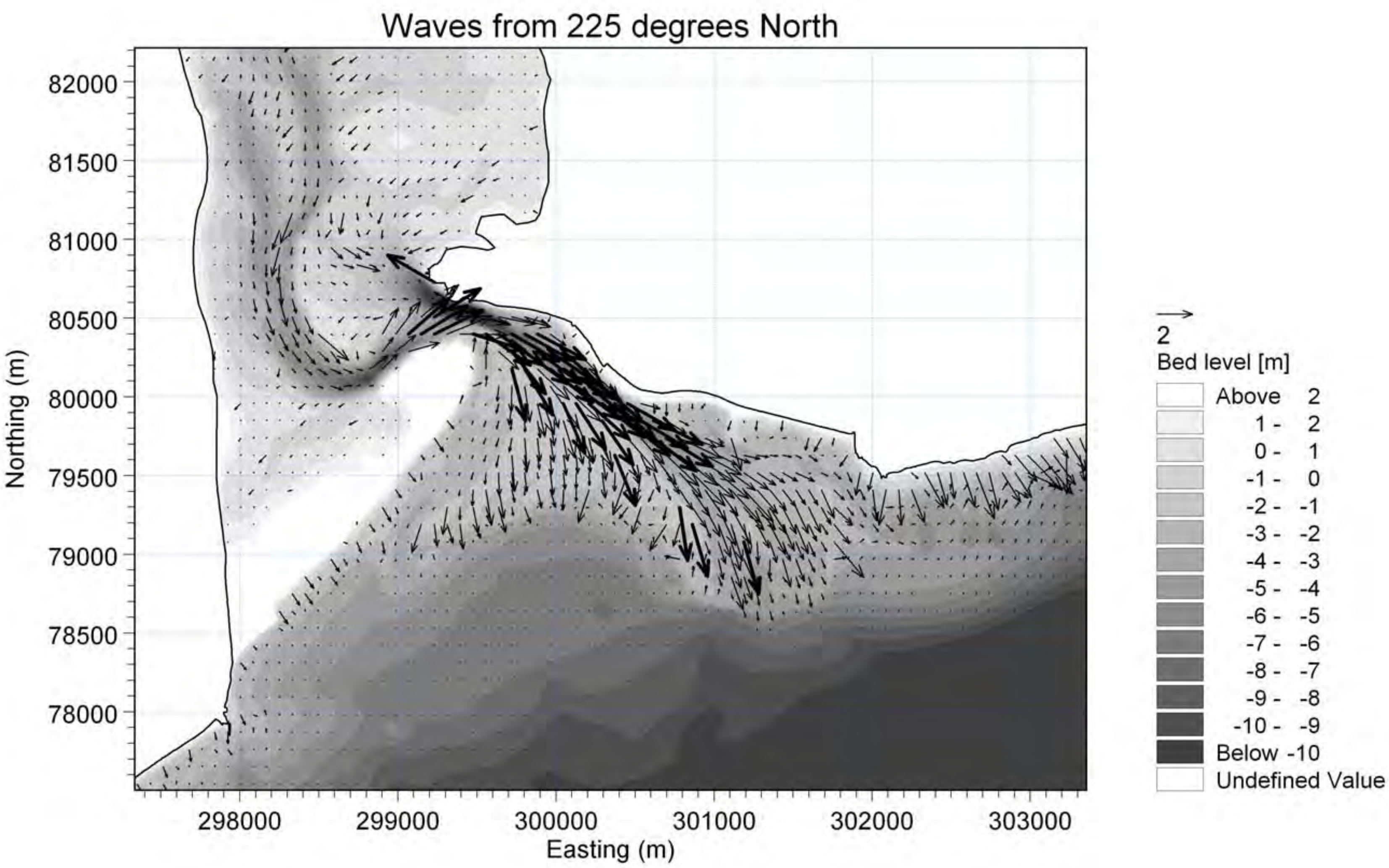


Figure 7-40: Net sediment transport for waves from 225°N.

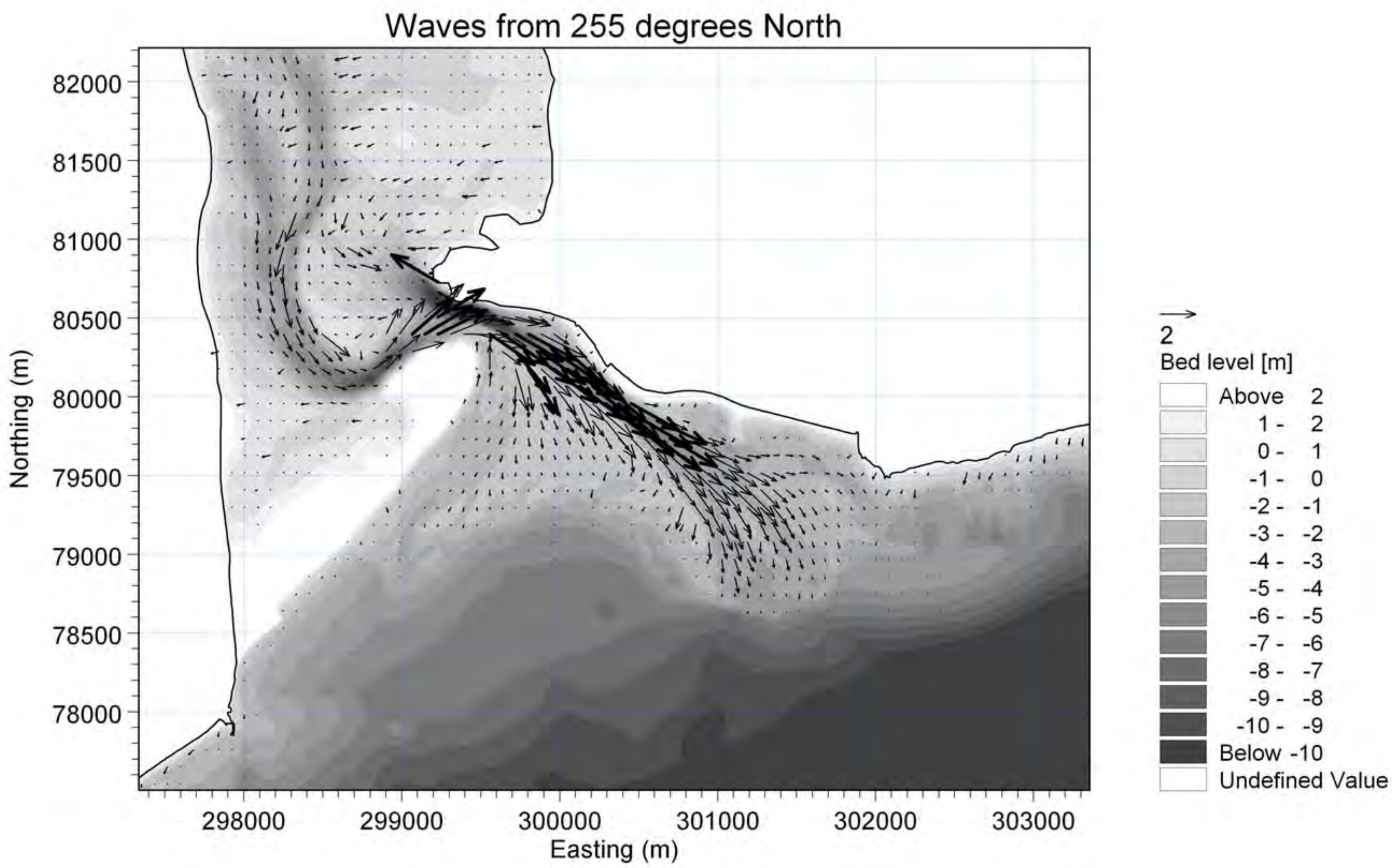


Figure 7-41: Net sediment transport for waves from 255°N.

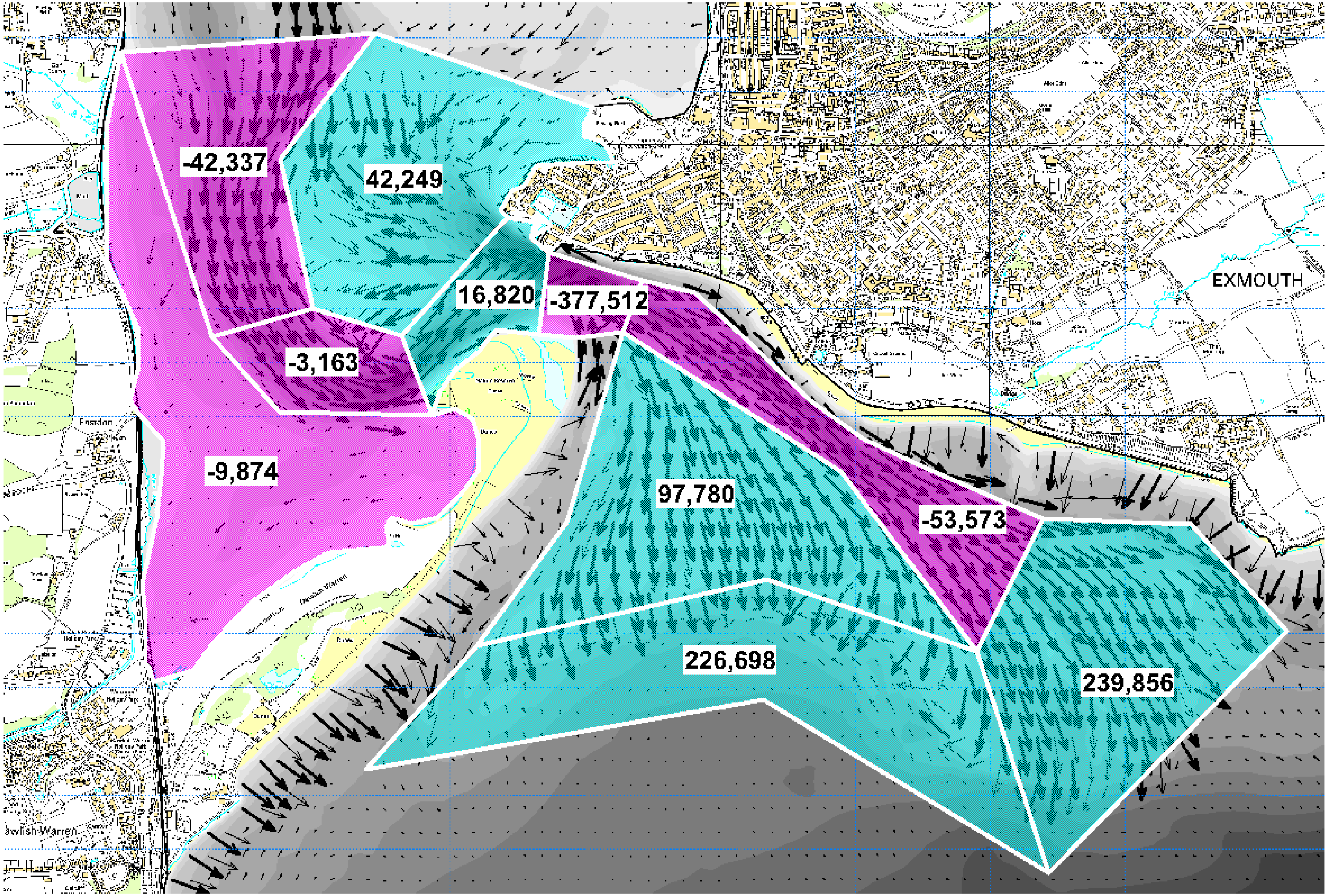


Figure 7-42: Simulated annual changes in sediment volume (m^3) for areas within the Exe Estuary. Areas of net sediment Loss are shown in pink; areas of net sediment gain are shown in blue.

7.5.3 Dawlish Warren

The results show annual average longshore drift along Dawlish Warren from combining results from all wave conditions. At the north-east end of Dawlish Warren there is also a component of transport due to the tide only. For wave conditions from 135°N to 195°N, the sediment transport patterns are highly two-dimensional due to westward wave driven currents set up along Pole Sand. Key features of the sediment transport around Dawlish Warren, and net sediment transport through cross-sections perpendicular to the shore, are shown in Figure 7-43. The annual average transport across odd numbered sections is shown in Figure 7-44, and the variation in the transport rate along the shore is shown in Figure 7-45.

Figure 7-43 shows that annual average transport is highest through sections D12 and D13 at the north-eastern end of the spit. Figure 7-46 shows the transport over two tides through section D13. The tide only transport is of a similar magnitude to the transport with waves and tides in this location, so that the annual average transport is dominated by tidal rather than wave driven currents as there are more calm days. Flow through the low water channel adjacent to Dawlish Warren is greater on the flood tide than the ebb tide, where a larger component of the flow is through the channel to the north-east of Pole Sand. Maximum nearshore velocities around cross-section D13 are 1.5 m/s on the flood tide and 0.6 m/s on the ebb tide so that more sediment is transported north-east on the flood tide than south-west on the ebb tide, and the tidal currents elongate the spit. Figure 7-43 shows that sediment removed from the end of the spit is incorporated into the main flow out of the estuary and eventually deposited over Pole Sand. Actual tidal transport through these sections may be lower than the modelled transport due to the use of a mean spring tide to model the tide only transport. The low water channel adjacent to Dawlish Warren is shallow at the north of Dawlish Warren and flows through the channel will be low for smaller range tides.

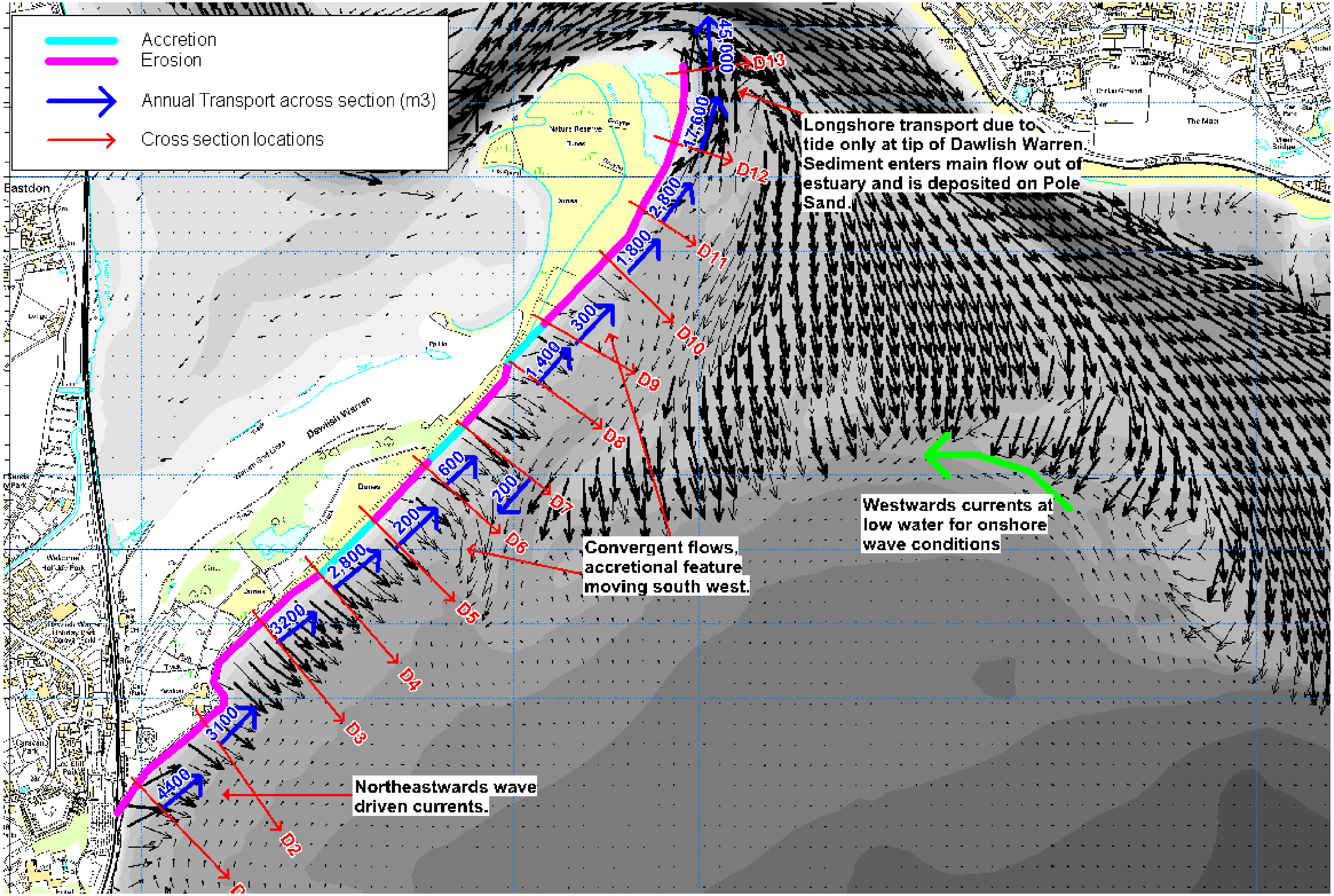


Figure 7-43: Key Features of the annual average sediment transport regime around Dawlish Warren. Results from all representative wave conditions have been combined.

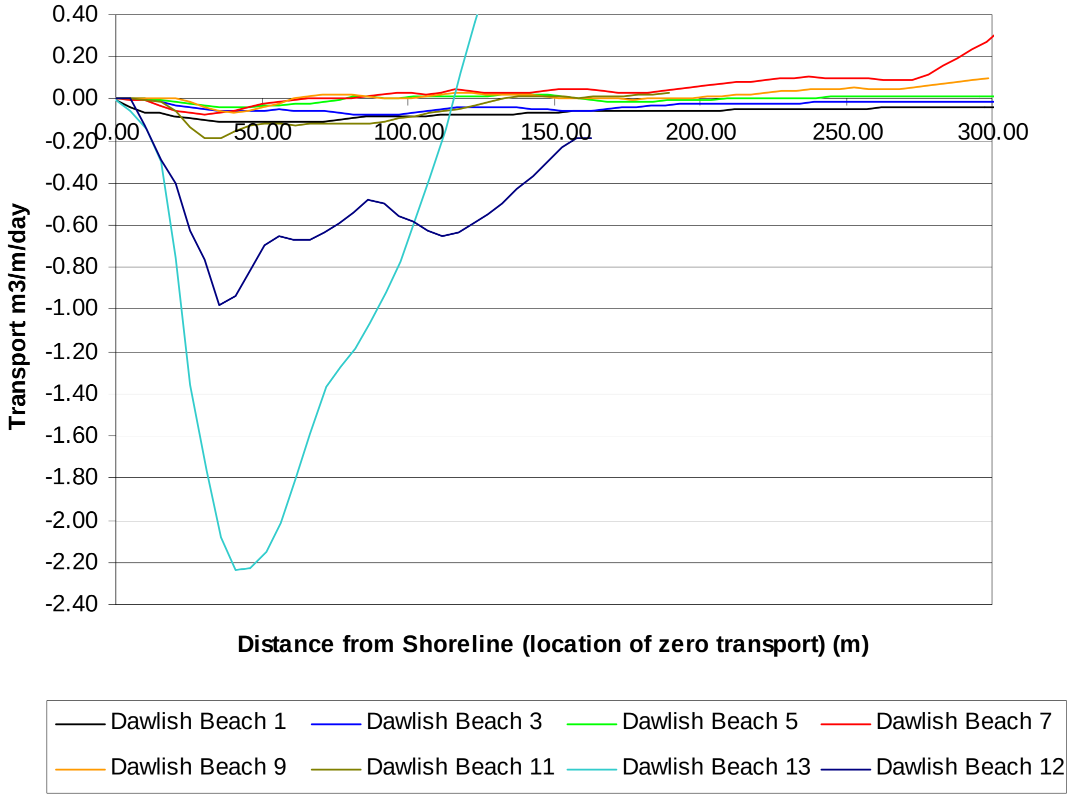


Figure 7-44: Annual average of daily transport across the sections shown in Figure 7-43., calculated by combining model runs from the representative wave conditions in Figure 7-2.

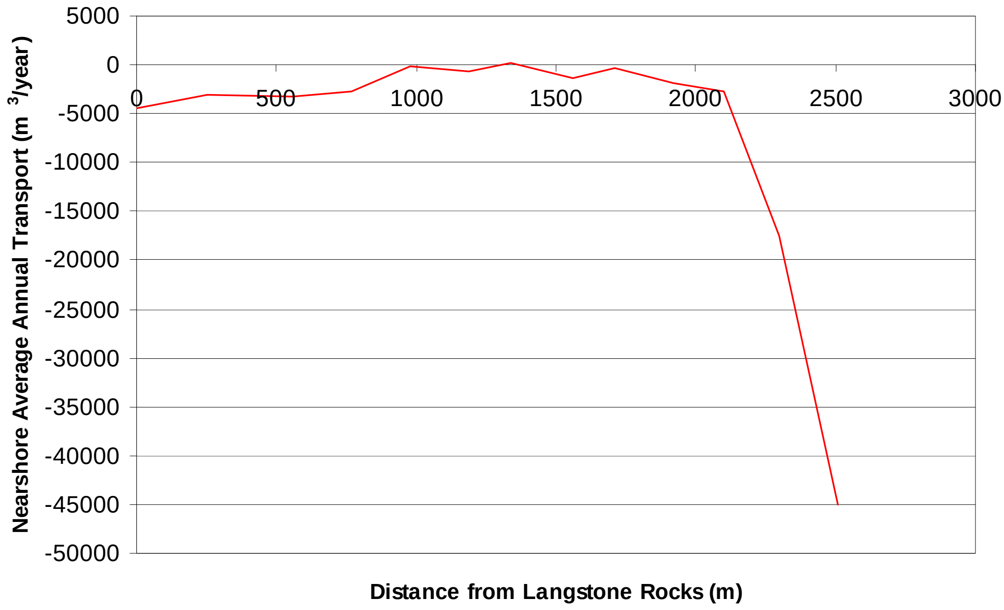


Figure 7-45 :Modelled annual average transport along Dawlish Warren as a function of distance from Langstone Rocks.

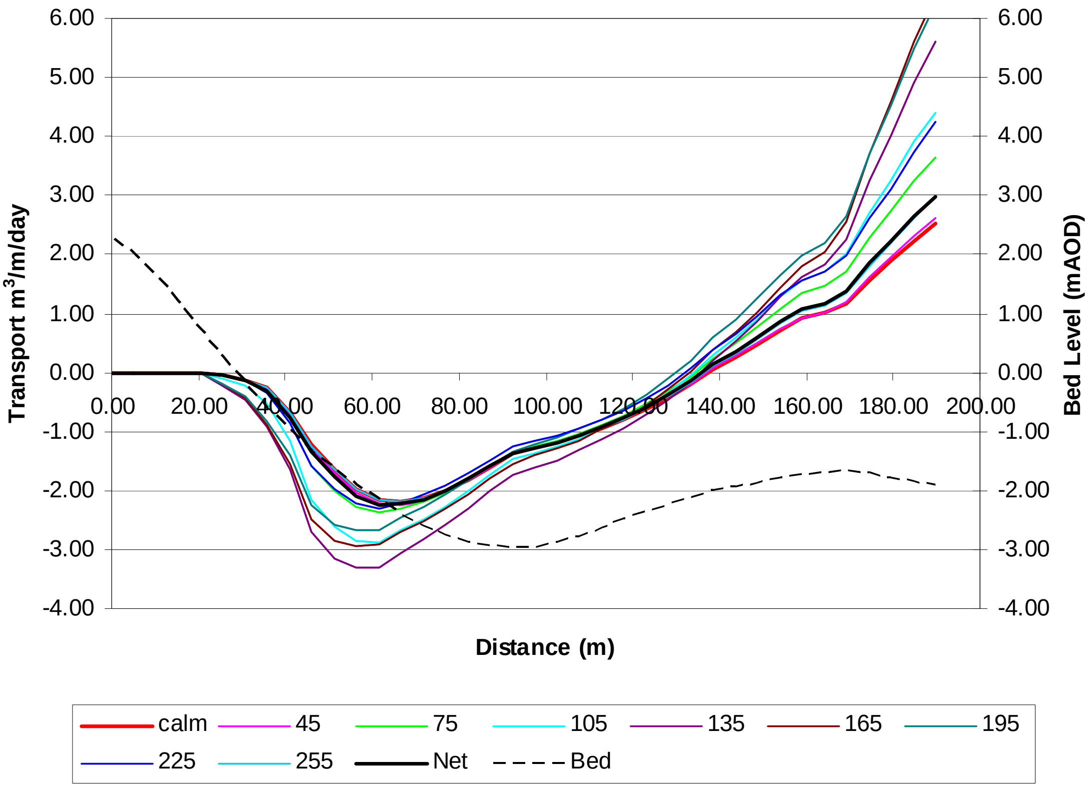


Figure 7-46: Sediment transport over 2 tides through cross-section D13 in Figure 7-43. The net transport is the weighted sum of all wave conditions including the calm condition. The bed level across this section is shown by the dotted line. Northwards flows are negative.

For representative waves from 135°N to 195°N there are north-eastwards wave driven currents along the Dawlish Warren (Section 7.3.3.3). This results in north-eastwards transport along the entire length of the spit for these wave directions. The cross-shore distribution of sediment transport through cross-section D3 (south-west of Pole Sand) and cross-section D10 (north west of Pole Sand) for these wave directions is shown in Figure 7-47 and Figure 7-48. For waves from 75°N and 105°N there are south-westwards wave driven currents along the spit south of Pole Sand only (Pole Sand partially shelters Dawlish Warren from this direction). This results in south-westwards longshore transport south of Pole Sand (Figure 7-47) and minimal wave driven transport north of Pole Sand (Figure 7-48) for these wave directions. Representative waves from 75°N and 105°N and the associated wave driven currents are smaller than for representative waves from 135°N and 195°N and the result is net north-eastwards nearshore longshore transport along the spit through all cross-sections (Figure 7-44).

Fox *et al.* (2008) analysed historic spit profiles and found that there was net transport of sediment from the south-west of the spit to the north-east of the spit. Figure 7-43 shows sediment transport through sections D1 to D5 is approximately two-times larger than transport through sections D8-D11; however the model does not show consistent transport from the south-west to the north-east of spit. Implied model changes in MSL position (calculated by assuming that the difference in transport between adjacent cross-sections is spread evenly over the existing beach profile between the sections) are shown in Figure 7-49. The model does not show the loss of sediment between 1500m and 2500m (measured from section D1), where the model and bathymetry show growth of an offshore accretion feature which may not be captured in the observed cross-sections. The observations show a gain in sediment at the tip of Dawlish Warren where the model shows a loss due to the tidal currents. There is a potential that tidal transport may be overestimated here due to the conservative use of a mean spring tide for the representative tide. Also changes in position in the low water channel will have a large impact on the location of MSL along Dawlish Warren.

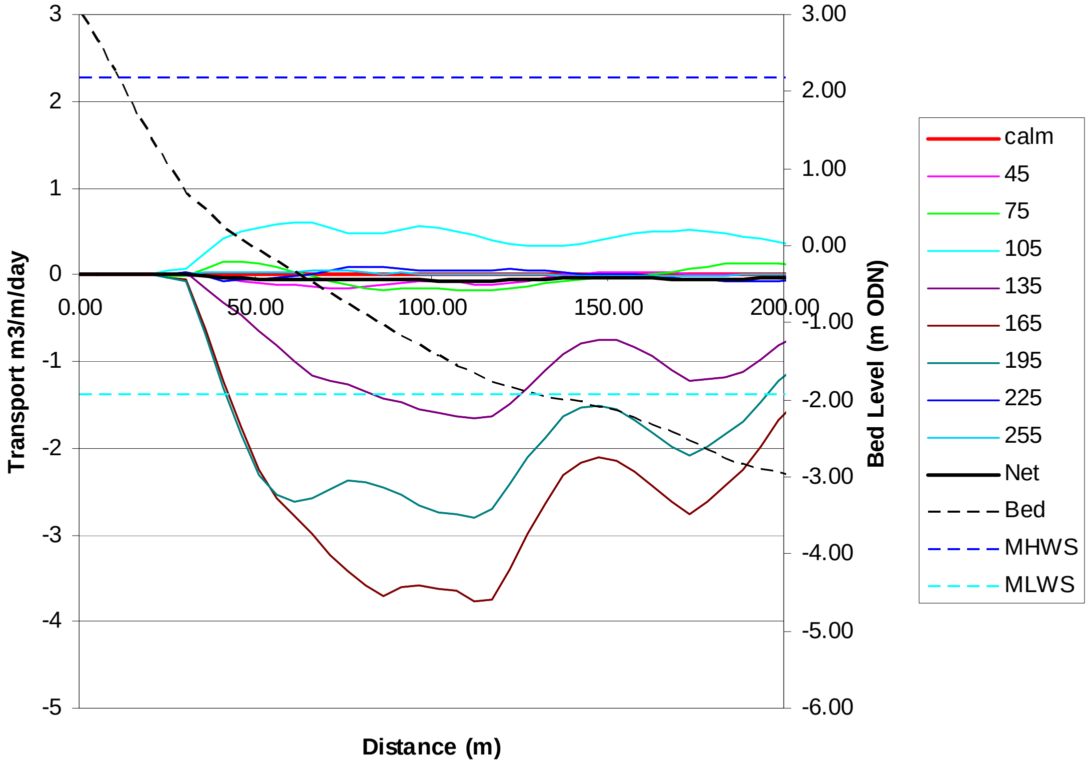


Figure 7-47: Transport through section D3 for all representative wave conditions. The section is south west of Pole Sand and south-westwards (+ve) transport occurs for representative waves from 75°N and 105 °N, and north-eastwards (-ve) transport occurs for representative waves from 135°N to 195 °N.

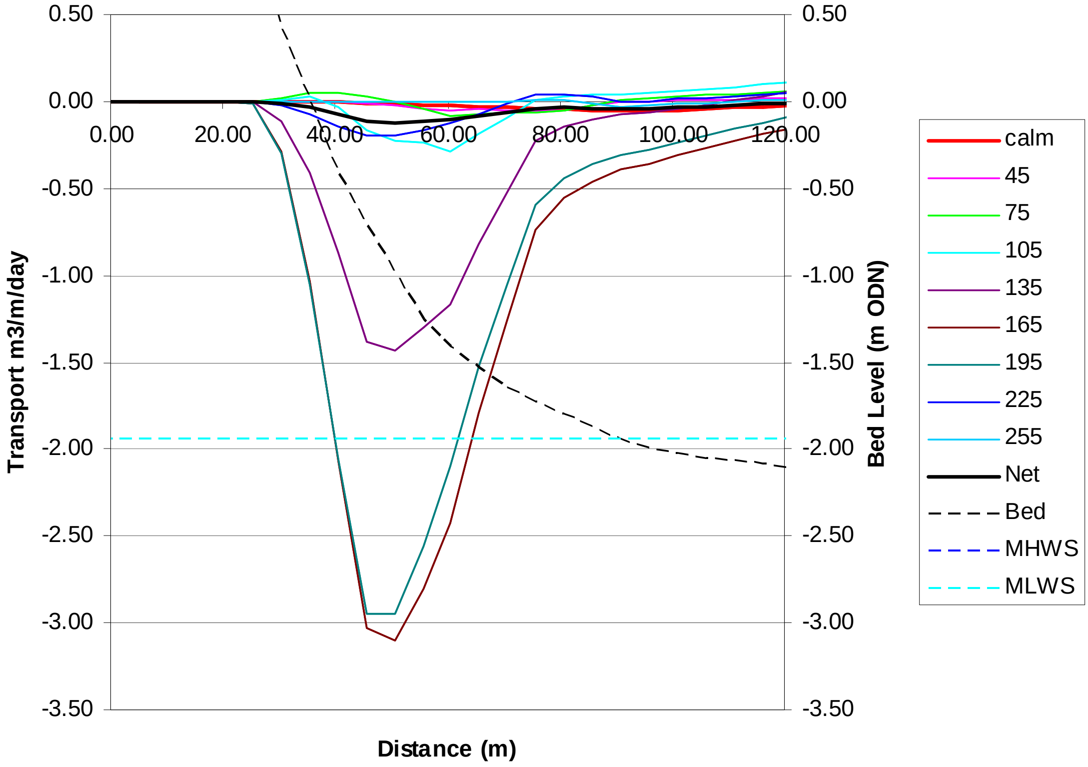


Figure 7-48: Transport through section D10 for all representative wave conditions. The section is north east of Pole Sand there is minimal transport for representative waves from 75°N and 105 °N, and north-eastwards (-ve) transport) occurs for representative waves from 135°N to 195 °N.

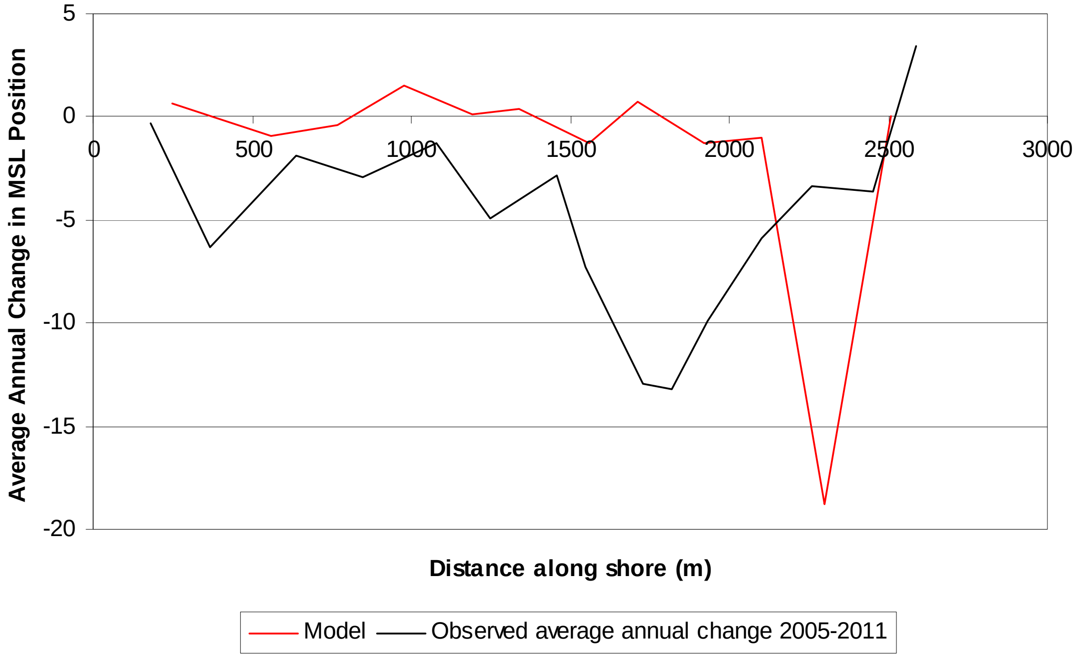


Figure 7-49: Horizontal change in MSL position calculated from cross-sections from 2005 -2011 and model results.

For representative waves, the sediment transport regime around Dawlish Warren is highly two-dimensional due to the interaction with Pole Sand. For representative waves from 105°N to 225°N, westwards wave driven currents are set up at low water along the southern edge of Pole Sand and an eddy develops north-east of Langstone Rock (Figure 7-27 to Figure 7-30; Section 7.3.3.3). This results in convergent flow patterns around an accretional area mid-way along Dawlish Warren as the north-eastwards nearshore currents along Dawlish Warren interact with the westwards currents along Pole Sand. Cross-sections 5, 6 (shown in Figure 7-50) and 7 through this area show that although the nearshore transport through these sections is north-eastwards, there is south-westwards transport over the accretional area. There is a smaller accretional feature further north along Dawlish Warren, caused by westwards wave driven currents set up over the centre of Pole Sand (Section 7.3.3.3; Figure 7-28) for larger northwards representative waves. There is a potential that recharge of the beach south-west of these accretional features may lead to additional growth of the feature with not all of the recharged material being transported north-east along Dawlish Warren. These westwards currents also provide a potential mechanism for supply of sediment to Dawlish Warren from Pole Sand. As these areas of accretion are due to two-dimensional interactions with Pole Sand they are not represented in the shoreline evolution model in Section 5.

The spatial distribution of cross-shore transport along Dawlish Warren is broadly consistent with that reported in Section 6, with higher cross-shore transport at the mid-point of Dawlish Warren (nearshore point 14 in Figure 3-4) than at the Exmouth end (nearshore point 12 Figure 3-4) due to the sheltering provided by Pole Sand.

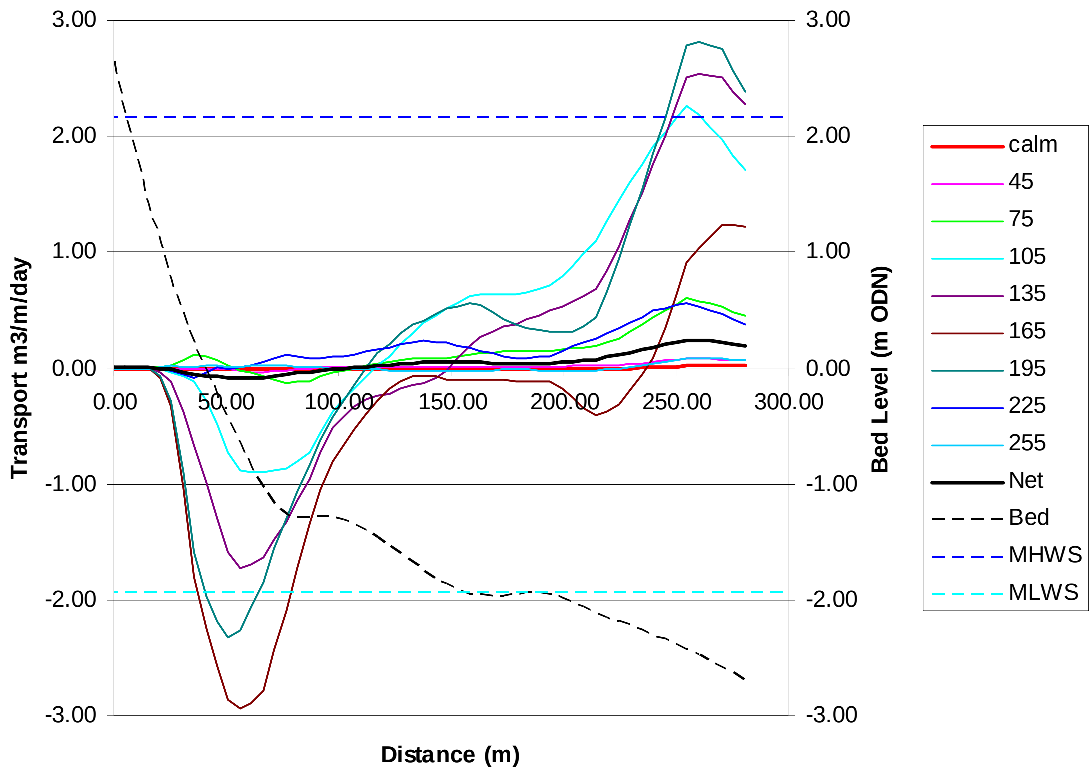


Figure 7-50: Transport through section D6 for all representative wave conditions. Nearshore transport is north-eastwards (-ve) while offshore transport is south-westwards (+ve) due to the interaction with the westwards currents along the south of Pole Sand.

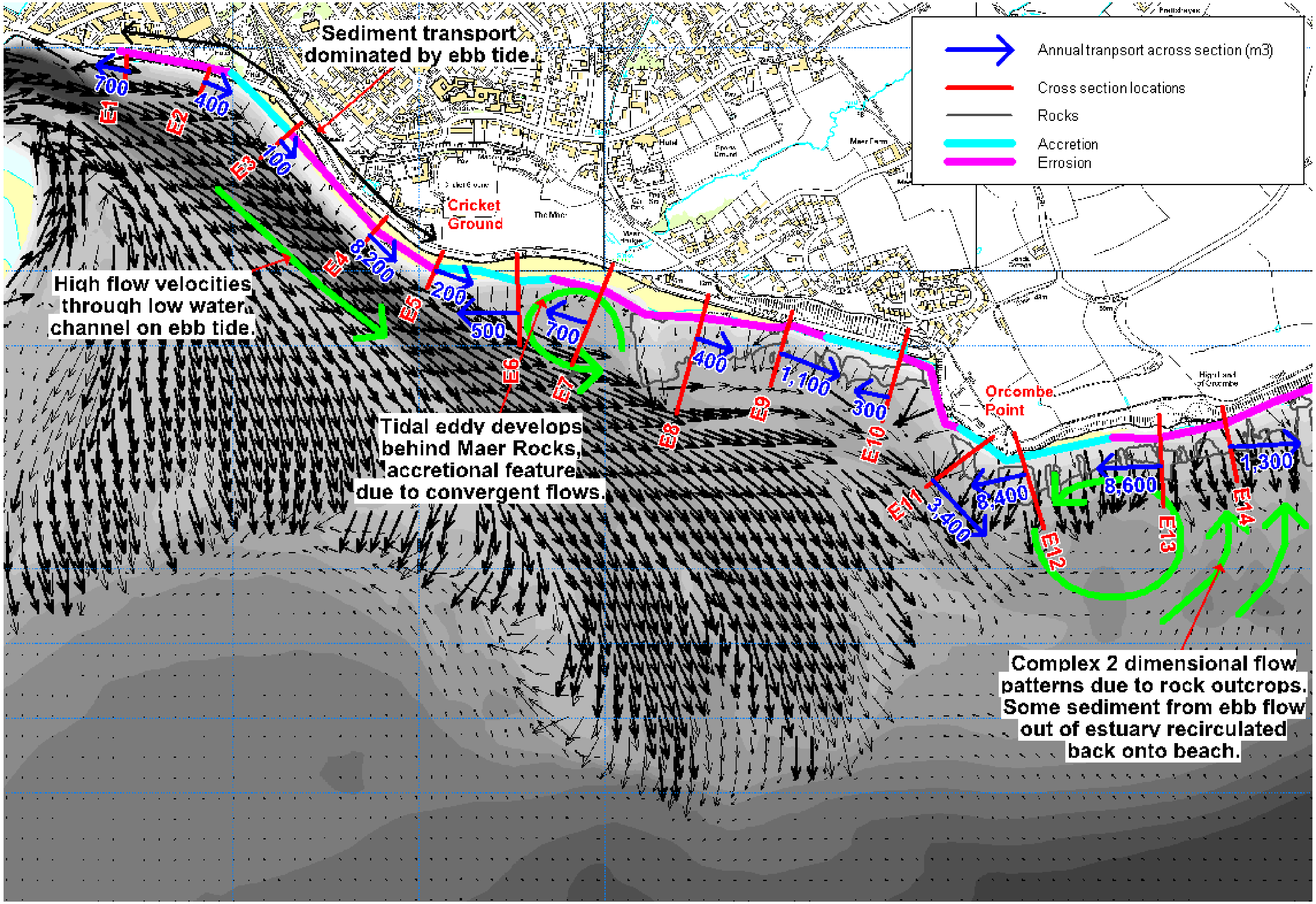


Figure 7-51: key features of the annual average sediment transport regime around Exmouth beach. Results from all representative wave conditions have been combined.

7.5.4 Exmouth Beach

Key features of the transport around Exmouth Beach are shown on Figure 7-51. Between the mouth of the estuary and the cricket ground the sediment transport along Exmouth Beach is tidally dominated. Ebb flows through the low water channel to the north-east of Pole Sand are higher than the flood flows so that for all wave conditions there is a net transport south eastwards over the length of beach. The southwards transport is higher for the cases with larger waves due to increased mixing. Net sediment transport through cross-sections E1 to E3 is small due to the small width of the beach and there is little net erosion or accretion between these sections. The shoreline modelling also shows that the beach is largely stable along this reach (Section 5.5). Between cross-sections E3 and E4 the coastal area model shows erosion while the shoreline modelling shows net accretion. Longshore transport through these sections in the tide only simulations is a similar magnitude to the case with representative waves and due to the larger number of calm days tidal transport dominates in this area. Transport due to the tide only is not included in the shoreline modelling and accretion in this area is unlikely due to the strong ebb currents through the low water channel.

Due to the tidal currents an eddy develops west of Maer Rocks leading to the formation of an accretional feature west of the rocks due to convergent flows. This is not clearly shown in Figure 7-24 due to the scale of currents out of the estuary. As this feature is due to two-dimensional flow patterns it is not included in the shoreline model which shows slight net erosion in this area (Section 5.5).

For representative waves from 105° to 195°, westwards wave driven currents are set along Exmouth Beach, while eastwards currents are set up for waves from 225°N. **Error! Reference source not found.** shows that sheltering leads to much smaller significant wave heights inshore for representative waves from 225° than from 105° to 195° so that at most sections north-west of Orcombe Rocks (cross-sections E5 to E10) the westward longshore transport for the individual wave conditions is higher for representative waves between 105° to 195° than the eastwards transport for 225° (Figure 7-52). The result is a wave driven longshore transport westwards through cross-sections E5 to E10, between 0 and 75m offshore. Further offshore the transport direction varies due to complex interactions with rocks projecting out from the shore and tidal currents, the direction of net transport through cross-sections E5 to E10 is highly variable. Results are also sensitive to the weighting given to the representative wave conditions and areas of net accretion and erosion along this reach are difficult to determine.

East of Orcombe Rocks sheltering from the dominant south westerly waves is reduced and there is a net eastwards transport at some cross-sections for some wave directions. Figure 7-53 shows the sediment transport for individual representative wave conditions at cross-section E14; at this section the net transport is small and eastwards due to the weighted contributions from different wave directions cancelling out. The situation in this area is further complicated by Orcombe Rocks which extend some distance from the coast. These rocks cause two-dimensional flow patterns in this area which vary with the state of the tide and wave direction and no clear conclusion can be drawn as to the net direction of sediment transport in this area. However, it is likely that the two-dimensional flow patterns lead to some recirculation of sediment from the main ebb flow out of the estuary onto the beach.

The coastal area model shows cross-shore transport is lower between Maer Rocks and Orcombe Point, corresponding to nearshore point 6 in Section 6 and between the cricket ground and Maer Rocks corresponding nearshore point 8 in Section 6 than to the east of Orcombe Point. Tidal currents are dominant in both of these locations as Pole Sand and the Dawlish coast provide sheltering from onshore waves. However the cross-shore modelling shows that during large storm events there can be a significant loss of beach in both of these areas due to cross-shore transport.

7.5.5 Flood Delta

There is a net transport of sediment out of the estuary. However there is some re-circulation of sediment from the main channel on to the flood delta around the dock entrance and north-west of the tip of the Dawlish Warren causing accretion of the flood delta. The low water channel at the mouth of the dock is eroding north westwards; these features are shown in Figure 7-54.

7.5.6 Pole Sand

Key features of the sediment transport regime over Pole Sand are shown in Figure 7-55. Sediment transport due to tidal currents leads to an accumulation over the centre of Pole Sand and increase in the extent of Pole Sand to the south and east. For onshore waves westwards wave driven currents are set up along Pole Sand and lead to an increase in the extent of Pole Sand to the west. Current speeds on the ebb tide through the low water channel along the north-east edge of Pole Sand are high, and there is no evidence of an increase in the extent of Pole Sand to the north-east reducing the width of the low water channel.

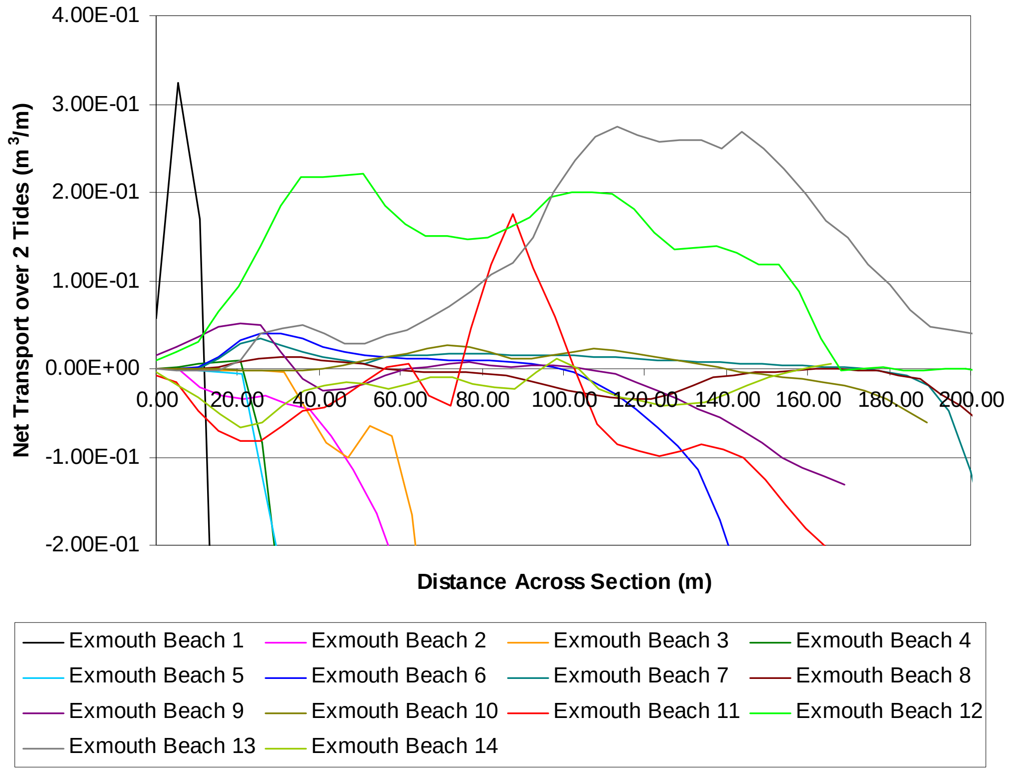


Figure 7-52: Annual average of daily transport across the sections shown in Figure 7-51, calculated by combining model runs from the representative wave conditions in Table 7-1. Sections are numbered from 1 in the north-west; and north-westwards flows are positive. The vertical scale shows the wave driven transport in the near shore. Transport in the main ebb flow out of the channel is higher.

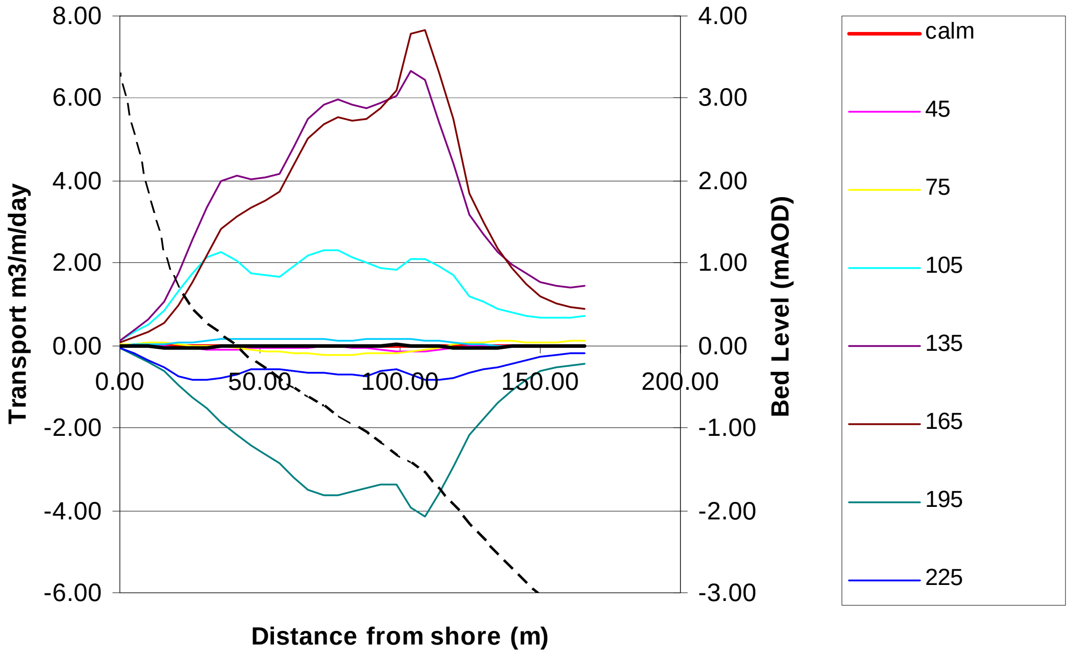


Figure 7-53: Sediment transport over two tides for different representative wave conditions through cross-section 14 in Figure 7-51. The annual average transport is small as contributions from different wave directions are in opposite directions.

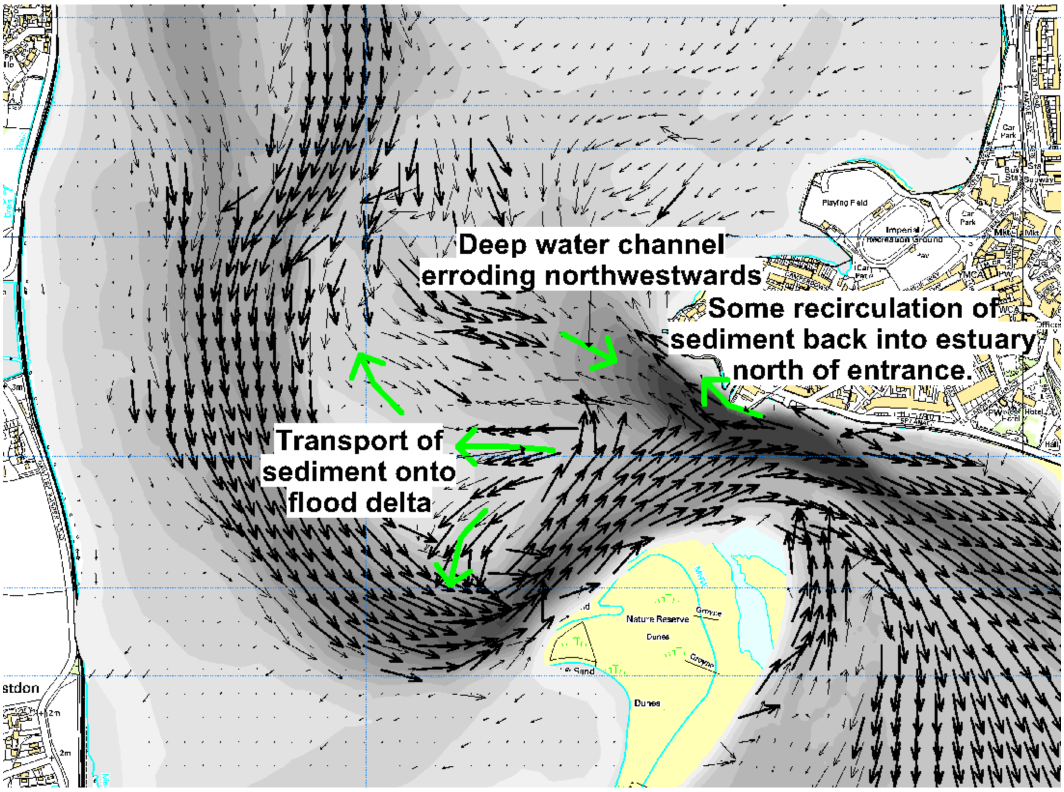


Figure 7-54: Key features of the annual average sediment transport regime within the estuary. Results from all representative wave conditions have been combined.

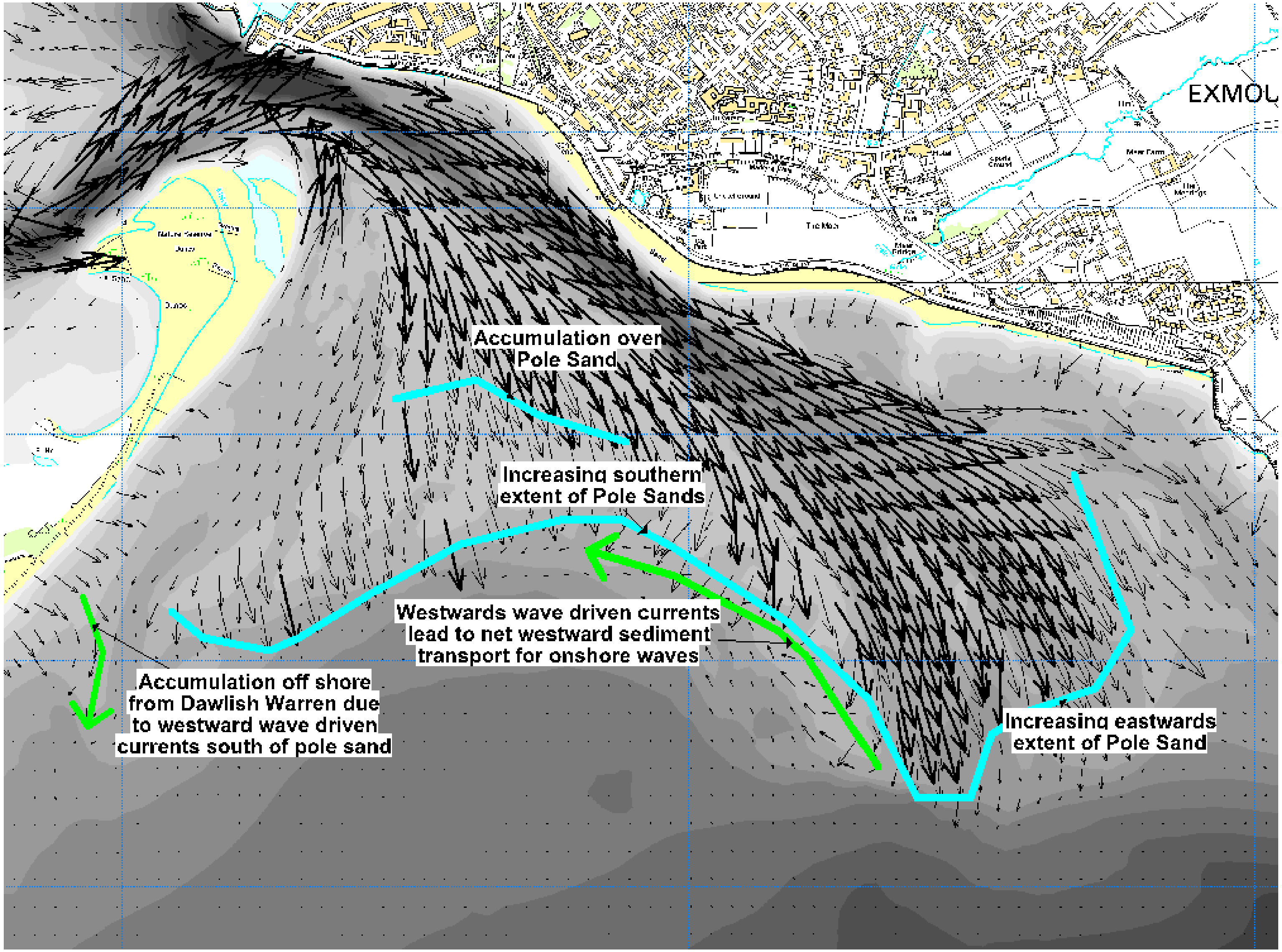


Figure 7-55: Key features of the annual average sediment transport regime over Pole Sand. Results from all representative wave conditions have been combined

8 Summary and Conclusions

8.1.1 Wave Data

- For the purpose of this modelling study, the wind and wave data were obtained from UK Met Office covering the period from 15/10/1986 to 25/11/2008 inclusive, representing a 22.08yr data set. This data set was from an offshore location located at 50° N, 3.66° W.
- Measured nearshore wave data for the calibration and validation of the nearshore wave model were collated. There are two measured wave points outside of the entrance to the Exe Estuary at which significant wave height data are available. The measured data cover the period between 21 January 2008 and 21 March 2008.

8.1.2 Water Level Data

- Four tide gauges are available around Lyme Bay. The data availability for these stations is presented in Table 2-2, and the locations of the gauges are shown in Figure 2-10.

8.1.3 Beach Profile and Bathymetry Data

- The beach profile data have come from a number of sources, much of which was collated as part of the Exe Estuary Coastal Management Study (Halcrow, 2008). The main data are LiDAR profiles for May 1998 and December 2005, and South-West Regional Coastal Monitoring Programme (SWRCMP) profiles from Spring 2007 up to (in some places but not all) Spring 2010.
- The LiDAR profiles were updated with the SWRCMP profile data that were down-loaded as part of this analysis from the Channel Coastal Observatory website on Friday 4th June 2010.
- The latest survey was carried out in December 2011 and the surveyed bathymetry data are available for updating the wave model bathymetry. Thus the latest bathymetry data have been used in the wave modelling work of this project.

8.1.4 Sediment Data

- The measured sediment sizes are available for the different beach profiles of the Dawlish Warren and Exmouth. Table 2-4 shows the sediment sizes at different locations (Figure 2-4) of the Dawlish beach profiles, while Table 2-5 shows the sediment sizes at different locations (see Figure 2-5) of the beach profiles of Exmouth.

8.2 Wave Modelling

- The wave model used for this project is the MIKE 21 SW spectral wind-wave model developed by DHI based on a flexible mesh using triangular elements, which allows a variable resolution to be prescribed. The wave model has been calibrated using measured wave data from two points outside of the entrance to the Exe Estuary at which significant wave height data are available. The model has been run for the same period as the available data for the purposes of comparison.

- The comparison has been made between the measured data and the simulated model results for a time period of January 2008 to March 2008. The agreement is remarkably good.

- The nearshore wave transformation has been carried out using the calibrated MIKE 21 SW model. The time-series of offshore wave data were transformed into corresponding time-series at the nearshore positions by establishing wave transfer look-up table including varying water levels. The wave transfer lookup table relates the offshore wave conditions to the corresponding nearshore waves for several combinations of offshore waves that propagate towards the shoreline in the 22-year time-series.

8.3 Extreme Value Analysis

- The marginal extreme analysis for the transformed nearshore wave data and also for the measured water levels has been performed to generate the boundary conditions for the joint probability model JOIN-SEA. The MIKE EVA tool has been used to produce various extreme value distributions (Weibull and Gumbel) from which the best fit distribution has been used to determine the extreme significant wave heights and water level values for the required return periods.

- The joint probability analysis between waves and water levels was carried out by making use of the JOIN-SEA software based on transformed nearshore wave data and measured water level data. From the JOIN-SEA model the pairs of data were extracted from the time-series of waves and water levels at each nearshore location.

- Those joint extreme results were used as parameters for study of beach cross-shore modelling and also the beach overtopping calculations.

8.4 Shoreline Evolution Modelling

- The Halcrow's COASTLINE model has been used to simulate the effects of wave action over several years along the Dawlish and Exmouth coasts, for the existing coastline and for the proposed options.

8.4.1 Shoreline Evolution Modelling for Dawlish Warren Coast

- For the Dawlish Warren coast, the shoreline evolution model was calibrated using the surveyed coastline position of 3/5/1998, which was used as the initial input data, and another surveyed data set of 22/1/2011, which was used as the target coastline position of the model calibration. For Exmouth coast, the surveyed coastline positions of 20/4/2007 and 23/1/2011 were used as initial and target coastline positions.

- Based on the existing coastal conditions and assuming the removal of all existing groynes, the COASTLINE model shows that there will be a breach on the Dawlish Warren spit after twenty years, and the existing coastline along the Exmouth coast will retreat to the existing seawall location. The historic evidence indicates that the Dawlish Warren spit was breached between 1945 and 1949.

- A total of seven options to enhance/extend the existing groynes and to recharge the beach were studied for the Dawlish Warren coast. It has been concluded from the modelling work that the Dawlish Warren beach could be improved by making all groynes fully functional and extending Groynes 12, 13 and 14 by 30m, whilst at the same time recharging the sections of beach between Groynes 1 to 5 and Groynes 11 to 14.

8.4.2 Shoreline Evolution Modelling for Exmouth Coast

- For the Exmouth coast, a total of five options were simulated in order to stop the beach erosion. The same strategy as applied to Dawlish Warren was also applied to the Exmouth; that is to recharge the south-east section of beach and to repair the groynes along the north-west section of beach. Thus it is recommended to recharge the beach between Groynes 1 to 3, and enhance the Groynes 4 and 5. The modelling results reveal that the beach section at Queen's Drive can be stabilized by installing two new groynes.

8.5 Cross-Shore Beach Profile Modelling

8.5.1 Modelling of Three Storms

- Halcrow was commissioned to study three specific storms that occurred on 23/10/2009, 3/3/2010 and 9/10/2010 along the Dawlish Warren and Exmouth coasts. Halcrow's cross-shore beach profile model COSMOS was applied to investigate the beach behaviours during these three storm events.

8.5.1.1 Modelling of Three Storms for Dawlish Warren Coast

- For the Dawlish Warren coast there are pre-storm and post-storm beach profiles available for these three storms which were used to calibrate the COSMOS model. However, such profile data just before and after the identified storms is not available for Exmouth beach.

- The modelling results for cross-shore profile at nearshore point 12 along the Dawlish Warren coast revealed that the erosion volumes and maximum erosion depths caused by all three storms are quite small, which is in agreement with the surveyed data.

- For the nearshore point 14 along the Dawlish Warren coast, the modelling results indicate that there should be little change of the beach during the storms of 23/10/2009 and 3/3/2010. The measured beach profile data confirms the same trend as predicted by the model. For the storm of 9/10/2010, the measured post-storm beach profile is consistently higher than the pre-storm measured beach profile. This net gain of sand cannot be modelled by any beach profile model which should be based on the mass conservative principle (i.e. the volume of gained sand should be balanced by the lost sand).

8.5.1.2 Modelling of Three Storms for Exmouth Coast

- For the Exmouth coast, there is only one of surveyed beach profile data available just before the storm of 23/10/2009. No other pre- and post-storm beach profile data is available for model calibration. Thus the COSMOS model was used to study the potential beach changes during identified storms of 23/10/2009, 3/3/2010 and 9/10/2010.

- The modelling results for cross-shore profile at nearshore point 6 along the Exmouth coast revealed that there is no substantial erosion and accretion caused by all three storms. The maximum erosion depth is less than 0.4m and occurred during the storm of 9/10/2010.
- For the beach profile at nearshore point 8 along the Exmouth coast, the modelling results indicate that there should be little change of the beach during the storms of 23/10/2009, 3/3/2010 and 9/10/2010. In general the modelling results are in the same order as the predicted results for the Dawlish Warren coast.

8.5.2 Modelling of Beach Profiles for Extreme Wave and Water Level Conditions

- The joint probability analysis results were used to generate the synthetic storms as input data for the cross-shore sediment modelling.
- The idealised storm profile of waves were obtained from analysing nearshore wave data of time-series which were obtained from the wave modelling work for the period 15/10/1986 to 25/11/2008 inclusive, representing a 22.08yr data set. The top ten largest storms associated with the top ten highest waves during the 22 years were identified and the idealised storm shape of wave heights was obtained by averaging the data of the top highest waves.
- The idealised storm profile of water levels was obtained from the measured water levels at Exmouth Dock from 10/11/2000 to 31/12/2009. The top largest storms associated the highest water levels with complete records were identified and the averaged water levels from those largest storms were calculated. The averaged storm curve was used as the representative storm profile shape for a given extreme water level so that a time-series of water level data was generated as input data for the cross-shore sediment modelling.
- The idealised storm wave and water level time-series were used as input data for COSMOS modelling and the model was used to simulate the beach profiles under extreme wave and water level conditions of 1 in 1 and 1 in 100 year return period.

8.5.2.1 Modelling of Beach Profiles for Dawlish Warren Coast

- For the nearshore point 12 along the Dawlish Warren coast, the results show there should be no significant erosion of the upper beach. This is because the beach slope of this profile is very small, and the wave energy is not high.
- For the nearshore point 14 of Dawlish Warren coast, however, the results revealed that for a normal annual storm which should be close to the extreme event of 1 in 1 year return period, the maximum beach retreat distance could be about 4.3m and the maximum eroded depth could be about 0.23m. For an extreme event of 1 in 100 year return period, the beach retreat distance could be about 6.7m, and the maximum erosion depth of beach could be about 0.51m. The modelling results demonstrated that there would be offshore bars generated under all conditions of 1 in 1 year return period and 1 in 100 year return period.
- From comparison of both cross-shore modelling results at point 12 and point 14, it can be concluded that the cross-shore profile at point 14 could be much more

active than that at point 12. This is due to the wave heights at point 14 are generally much higher than those wave heights at point 12.

8.5.2.2 Modelling of Beach Profiles for Exmouth Coast

- For the nearshore point 6 along the Exmouth coast, the results show there should be no significant erosion at the upper beach for the extreme event of 1 in 1 year return period. But for conditions of 1 in 100 year return period, the beach retreat distance could be about 18.5m, and the maximum erosion depth of beach could be about 1.55m. The place of the maximum erosion should be very close to the top of the beach.
- For the nearshore point 8 along the Exmouth coast, the results indicate that for a normal annual storm which would be close to the extreme event of 1 in 1 year return period, the maximum beach retreat distance could be about 6.5m and the maximum eroded depth would be about 0.49m. For an extreme event of 1 in 100 year return period, the beach retreat distance could be about 9.7m, and the maximum erosion depth of beach could be about 0.60m. The offshore bars were generated from the model for all joint wave and water level conditions used for the COSMOS modelling.
- Comparing point 6 and 8, the wave heights at point 6 are normally higher than those wave heights at point 8. But the upper beach slope at point 6 is about 1 in 28, while the upper beach slope at point 8 is about 1 in 12. Thus the beach responses to the extreme storms at both points could be in the same order.

8.5.3 Modelling of Beach Profiles Using SHINGLE Model

- Halcrow's SHINGLE model is used to investigate the potential erosion of the coarser grained upper section of beach along Dawlish Warren (i.e. the toe of the sand dunes) which, if eroded, can be inferred to lead to undermining and erosion of the Dawlish Warren dune crest. This is because the studied beach section is covered with shingle and gravel which is suitable for using the SHINGLE model. The modelling results reveal that there should be no significant change of the beach for the extreme condition of 1 in 1 year return period. This finding is very similar to the surveyed beach profile data of pre and post storm of October 2010.
- The results of 1 in 500 years return period show that the dune crest could retreat further inland compared with the results of 1 in 100 years return period. An erosion of more than two metres could be expected at the crest of the Dawlish Warren dune for both extremes.
- The Dawlish Warren dunes should be able to stand for a single severe storm without breach due to the width of the crest. However, under a number of extreme severe storms a breach of the dunes is likely to occur.

8.6 Two-Dimensional Sediment Transport Modelling

A coastal area model of Dawlish Warren, Exmouth Beach, Pole Sand and the Exe Estuary has been developed. The model includes tide and wave driven currents, surface waves and sand transport. The model is calibrated to within Environment Agency guidelines and is a suitable tool for studying sediment transport around the Exe Estuary. The model shows that sediment transport around Exmouth Beach and

Dawlish Warren is highly two-dimensional, due interactions of the beaches with the offshore sand banks. Over most of the area transport is tidally dominated, but wave driven currents are important along Dawlish Warren. Key features of the sediment transport regime are shown by coastal area model:

- The flood and ebb deltas of Bull Hill and Pole Sand are increasing due to transport of sediment by tidal currents. The model shows a net deposition of 80 000 m^3/year of sediment over Bull Hill and 200 000 m^3/year over Pole Sand. The southern of extent of Pole Sand is increasing due to a net deposition of 300 000 m^3/year along the southern edge of Pole Sand. The eastern extent is increasing due to a net deposition of 300 000 m^3/year along the south-eastern edge of Pole Sand.

- There is nearshore north-eastwards transport along the entire length of Dawlish Warren for representative wave conditions. Wave driven currents dominate transport between the south-west end of Dawlish Warren and 7/8th of the way along the spit.

- At the north-west tip of Dawlish Warren tidal currents are dominant and there is nearshore north/north-eastwards transport at the north-west tip of Dawlish Warren due to tidal currents. The model shows the annual transport due to the tide at the end of Dawlish Warren is 40 000m^3/year. However, this is a conservative estimate due to the use of a mean spring tide for the tide only simulations, and the actual transport is likely to be lower. Sediment removed from the end of the spit enters the main flow out of the estuary and is eventually deposited over Pole Sand.

- Accretional features forming along Dawlish Warren due to westwards wave driven currents set up at low water along the southern edge and centre of Pole Sand. These westwards currents provide a possible mechanism for transport of sediment from Pole Sand back onto Dawlish Warren

- Tidal currents dominate transport along Exmouth Beach west of Maer Rocks. There is nearshore south-eastwards transport along Exmouth Beach west of the cricket ground due to tidal currents and growth of an accretional feature west of Maer Rocks due to a tidal eddy to the west of Maer Rocks.

- East of Maer Rocks, transport between 0 and 75m from the shore is dominated by wave driven currents. There is nearshore westwards transport along Exmouth Beach between Orcombe Point and Maer Rocks due wave driven currents. Further offshore, interactions with rocks and strong tidal currents lead to variable flow directions.

The model cannot be used to analyse annual cross-shore transport as smaller waves which would add sediment back onto the beaches have not been included in the representative wave conditions run.

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Appendix A

Groyne Inspections

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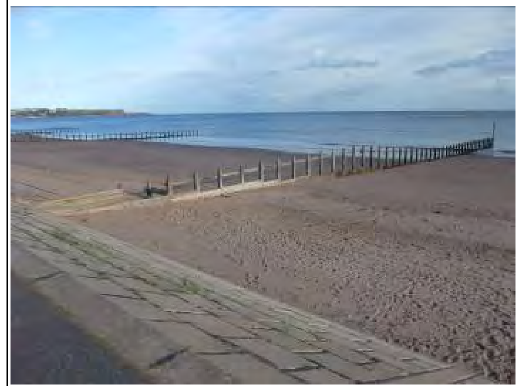
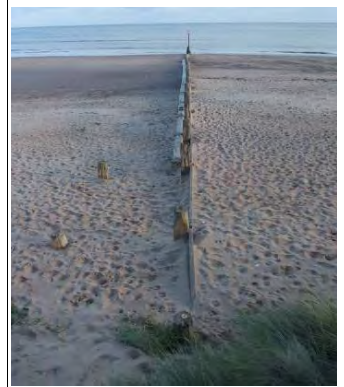
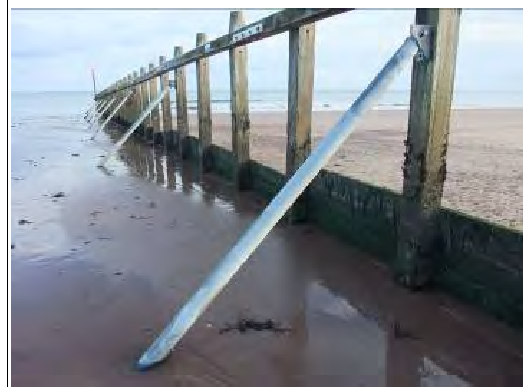
Appendix A Groyne Inspections

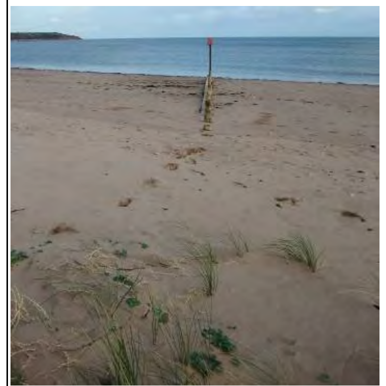
The existing groynes along both the Dawlish Warren and Exmouth Beach coasts have been inspected and details of these inspections are provided in this appendix.

A.1 Groyne Conditions on Dawlish Warren Coast

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile)			Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
		Landward	Central	Seaward			
1	80m (80m)	1.5m (2m)	0.5m (2m)	0.3m (2m)			West end (start) of seawall, groyne in good condition, good differential build-up of shingle on west side in upper beach, full height plants at landward end, planks removed in central/ seaward sections.
2	80m (50m)	0m (1.7m)	0.6m (2.7m)	0.2m (2m)			East end (end) of seawall, start of gabions, groyne in good condition, some differential build-up of material on up-drift (west) side at central section, planks removed at landward, central and seaward sections
3	60m (40m)	0.1m (1.5m)	0.2m (2.5m)	0.2m (2m)			Gabions present at root of groyne, groyne in good condition, very little differential build-up of material on updrift (west) side, planks removed at landward, central and seaward sections.
4	70m (50m)	0.6m (1.5m)	0.2m (3m)	0.1m (2m)			Groyne in good condition, gabions present at root of groyne, some differential build-up of sand on up-drift (west) west side of landward section, some gaps in planks at landward section.

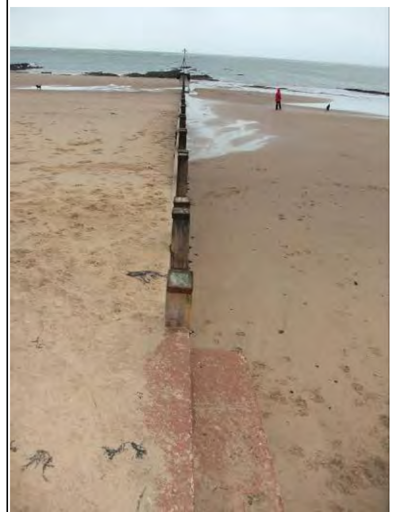
No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile)			Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
		Landward	Central	Seaward			
5	70m (70m)	0m (1.5m)	0.4m (2m)	0.2m (2m)			Groyne in good condition, landward section of groyne almost buried, very little differential build-up of material on updrfit (west) side, planks removed at central and seaward sections, some gaps in plant at landward section.
6	65m (50m)	0.8m (1.7m)	0.3m (1.8m)	0.1m (1.5m)			Some piles leaning in central section, gabions at root of groyne, some differential build-up of sand at landward section of groyne on west side, planks removed at central and seaward sections.
7	70m (70m)	0.1m (1.5m)	0.3m (2.7m)	0m (3m)			Recent repairs undertaken to brace the groyne, groyne now vertical, very little differential build-up of sand on west side, rock armour present at root of groyne, planks removed at landward, central and seaward sections, gaps beneath planks at central section
8	50m (20m – landward section)	0.2m (1.8m)	0.1m (3m)	0m (3.5m)			Rock armour at root, some lower boards missing at seaward end, gabions finish, concrete revetment continues to east, planks removed at landward, central and seaward sections, gaps beneath planks at seaward section,

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile)			Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
		Landward	Central	Seaward			
9	60m (30m – central section)	0m (1.3m)	0.2m (2.5m)	0m (2.5m)			Rock armour at root of groyne, east end of concrete matting, some slight leaning of piles at seaward end, planks removed at landward, central and seaward sections. No noticeable differential build-up on sand on up-drift (west) side.
10	80m (40m – seaward end)	0.0m (1.2m)	0.0m (2.5m)	0.4m (2.5m)			Some leaning of piles at seaward end, planks removed at central and seaward sections, no noticeable differential build-up of sand on up-drift (west) side, gaps beneath planks at central and seaward sections, some gaps beneath planks at seaward end,
11	70m (0m)	0.1m (1.7m)	0m (1.8m)	0m (2m)			Gabions at root of groyne, large gaps in lower planks at seaward end, planks removed at landward, central and seaward sections.
12	60m (0m)	0.7 m (1.5m)	0.9m (1.3m)	0.2m (1.3m)			Some leaning of piles at central section, some build-up of sand on up-drift (west) side at landward and central sections, planks removed at landward, central and seaward sections.

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile) Landward	Central	Seaward	Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
13	50m (50m)	0.1m (0.6m)	0.5m (1.3m)	0.2m (1.5m)			Completely buried at landward end, some differential build-up of sand on up-drift (west) side at central and seaward sections, no planks appear to have been removed,
14	25m (0m)	0m (0.9m)	0m (0.5m)	0m (0m)			Completely buried at landward and central sections, only a few top planks visible on seaward section
15	-	-	-	-	Not located	Not located	The spacing between groyne 14 and 17 suggests that this groyne may have existing but no evidence was found. Healthy beach levels suggest this groyne has been completely buried and the nav beacon removed.
16	-	-	-	-	Not located	Not located	The spacing between groyne 14 and 17 suggests that this groyne may have existing but no evidence was found. Healthy beach levels suggest this groyne has been completely buried and the nav beacon removed.
17	-	-	-	-			Buried

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile) Landward	Central	Seaward	Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
18	-	-	-	-			Buried

A.2 Groyne Conditions on Exmouth Coast

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile) Landward	Central	Seaward	Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
1 (East)	50m (40m)	0.4m	0.2m	0m			Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, beach crest level with top plank.
2	50m (25m)	1m	0.3m	0.5m			Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, beach crest level with top plank.
3	50m (20m)	0.6m	0.2m	0.2m			Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, 2 planks visible at beach crest.

<table>
<tr><th>No.</th><th>Exposed Length (active length – where material is being retained)</th><th colspan="3">Height of retained material (max height to top of pile)</th><th colspan="2">Photos
oblique, linear</th><th>Comments
leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.</th></tr>
<tr><th></th><th></th><th>Landward</th><th>Central</th><th>Seaward</th><th></th><th></th><th></th></tr>
<tr><td>4</td><td>50m (20m)</td><td>0.7m</td><td>0.1m</td><td>0m</td><td></td><td>[Image not available]</td><td>Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, beach crest level with top plank.</td></tr>
<tr><td>5</td><td>50m (25m)</td><td>0.3m</td><td>0.1m</td><td>0m</td><td></td><td></td><td>Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, beach crest level with top plank.</td></tr>
<tr><td>6</td><td>50m (0m)</td><td>0m</td><td>0m</td><td>0m</td><td></td><td></td><td>Groyne piles upright, no visible repairs, all planks in place, no visible gaps between planks, beach crest level with top plank.</td></tr>
</table>

No.	Exposed Length (active length – where material is being retained)	Height of retained material (max height to top of pile) Landward	Central	Seaward	Photos oblique, linear		Comments leaning piles, repairs, gaps beneath planks, build-up of material on updrift side etc.
Outfall		-	1m	-			Outfall appears to be acting as groyne to retain beach material

Appendix B

Storm Wave and Water Level Conditions

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Appendix B Storm Wave and Water Level Conditions

Wave and Water Level Conditions of Storm 23/10/2009, for Nearshore Point 6				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
20/10/2009 00:00	2.12	173.59	0.36	-0.58
20/10/2009 03:00	3.00	166.83	0.62	-0.23
20/10/2009 06:00	5.22	157.76	1.38	1.65
20/10/2009 09:00	4.97	160.15	1.26	2.23
20/10/2009 12:00	2.33	174.98	0.41	-0.29
20/10/2009 15:00	2.64	177.25	0.49	-0.42
20/10/2009 18:00	4.92	172.29	0.88	1.34
20/10/2009 21:00	5.48	176.01	0.66	2.08
21/10/2009 00:00	3.50	192.86	0.29	-0.30
21/10/2009 03:00	3.06	202.22	0.27	-0.49
21/10/2009 06:00	6.45	177.08	0.63	1.23
21/10/2009 09:00	6.78	176.12	0.76	2.23
21/10/2009 12:00	3.76	189.08	0.46	0.02
21/10/2009 15:00	2.63	197.12	0.31	-0.59
21/10/2009 18:00	6.58	173.40	0.86	1.00
21/10/2009 21:00	7.07	174.55	0.93	2.17
22/10/2009 00:00	4.36	182.69	0.61	0.22
22/10/2009 03:00	2.57	193.86	0.32	-0.65
22/10/2009 06:00	6.57	171.67	0.94	0.93
22/10/2009 09:00	6.94	174.94	0.95	2.19
22/10/2009 12:00	4.64	179.86	0.61	0.41
22/10/2009 15:00	2.15	202.96	0.22	-0.82
22/10/2009 18:00	4.91	185.09	0.49	0.62
22/10/2009 21:00	4.62	194.50	0.48	1.90
23/10/2009 00:00	3.88	197.20	0.37	0.33
23/10/2009 03:00	1.95	219.49	0.18	-0.99
23/10/2009 06:00	3.95	197.37	0.35	0.60
23/10/2009 09:00	4.10	199.16	0.35	1.87
23/10/2009 12:00	4.31	190.39	0.33	0.53
23/10/2009 15:00	2.08	205.18	0.15	-0.94
23/10/2009 18:00	4.23	186.33	0.31	0.36
23/10/2009 21:00	4.72	187.76	0.37	1.56

Wave and Water Level Conditions of Storm 3/3/2010, for Nearshore Point 6				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
02/03/2010 06:00	3.44	198.68	0.06	1.35
02/03/2010 09:00	4.67	181.97	0.07	2.37
02/03/2010 12:00	6.98	156.22	0.12	-0.33
02/03/2010 15:00	6.77	165.75	0.30	-1.34
02/03/2010 18:00	2.91	180.18	0.21	0.81
02/03/2010 21:00	3.11	180.86	0.22	2.32
03/03/2010 00:00	2.50	154.35	0.42	-0.14
03/03/2010 03:00	2.37	156.94	0.38	-1.28
03/03/2010 06:00	4.90	137.26	1.05	0.84
03/03/2010 09:00	5.22	134.93	1.08	2.71
03/03/2010 12:00	2.91	150.91	0.49	0.29
03/03/2010 15:00	2.07	158.25	0.28	-1.45
03/03/2010 18:00	4.82	134.75	0.85	0.33
03/03/2010 21:00	5.42	132.86	0.95	2.43
04/03/2010 00:00	3.31	146.53	0.54	0.50
04/03/2010 03:00	2.04	159.15	0.27	-1.51
04/03/2010 06:00	4.83	137.04	0.95	0.26
04/03/2010 09:00	5.68	134.05	1.18	2.38
04/03/2010 12:00	3.83	143.40	0.71	0.73
04/03/2010 15:00	1.96	157.73	0.27	-1.42
04/03/2010 18:00	4.48	136.98	0.82	-0.16
04/03/2010 21:00	5.29	132.26	0.85	1.77
05/03/2010 00:00	3.86	138.42	0.59	0.77
05/03/2010 03:00	1.96	163.41	0.20	-1.43
05/03/2010 06:00	3.91	134.17	0.51	-0.19
05/03/2010 09:00	4.63	128.73	0.46	1.62
05/03/2010 12:00	4.06	141.09	0.36	0.92
05/03/2010 15:00	2.89	180.46	0.14	-1.30
05/03/2010 18:00	5.03	163.98	0.33	-0.28
05/03/2010 21:00	8.11	159.76	0.23	1.39

Wave and Water Level Conditions of Storm 9/10/2010, for Nearshore Point 6				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
07/10/2010 00:00	2.56	216.35	0.20	-1.22
07/10/2010 03:00	5.05	187.78	0.37	0.43
07/10/2010 06:00	4.81	193.37	0.35	2.41
07/10/2010 09:00	5.16	185.77	0.36	0.66
07/10/2010 12:00	2.95	190.66	0.25	-1.29
07/10/2010 15:00	6.50	166.51	0.85	0.34
07/10/2010 18:00	5.01	140.34	1.15	2.55
07/10/2010 21:00	3.98	139.71	0.80	1.17
08/10/2010 00:00	1.96	159.10	0.25	-1.15
08/10/2010 03:00	3.51	141.52	0.64	-0.12
08/10/2010 06:00	4.52	134.13	0.76	2.17
08/10/2010 09:00	3.59	135.19	0.60	1.49
08/10/2010 12:00	1.91	153.42	0.21	-0.99
08/10/2010 15:00	3.17	139.56	0.50	-0.21
08/10/2010 18:00	4.86	131.67	0.79	2.02
08/10/2010 21:00	6.46	154.71	1.21	1.87
09/10/2010 00:00	2.54	192.09	0.25	-0.79
09/10/2010 03:00	4.35	183.29	0.45	-0.64
09/10/2010 06:00	7.21	172.22	0.92	1.63
09/10/2010 09:00	6.38	162.08	1.33	2.05
09/10/2010 12:00	2.62	185.50	0.36	-0.61
09/10/2010 15:00	3.90	185.10	0.55	-0.69
09/10/2010 18:00	7.42	171.26	1.11	1.46
09/10/2010 21:00	7.00	166.59	1.32	2.33
10/10/2010 00:00	2.82	175.89	0.44	-0.36
10/10/2010 03:00	3.05	171.73	0.50	-0.90
10/10/2010 06:00	6.70	158.23	1.38	1.12
10/10/2010 09:00	6.64	146.55	1.35	2.28
10/10/2010 12:00	3.18	158.07	0.47	-0.20
10/10/2010 15:00	2.79	159.86	0.38	-1.01
10/10/2010 18:00	6.40	142.10	1.17	0.88
10/10/2010 21:00	6.81	141.24	1.24	2.33

Wave and Water Level Conditions of Storm 23/10/2009, for Nearshore Point 8				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
20/10/2009 00:00	1.78	133.93	0.20	-0.58
20/10/2009 03:00	2.48	138.77	0.36	-0.23
20/10/2009 06:00	4.67	147.02	1.09	1.65
20/10/2009 09:00	4.35	148.27	0.94	2.23
20/10/2009 12:00	1.88	141.15	0.23	-0.29
20/10/2009 15:00	2.13	146.14	0.28	-0.42
20/10/2009 18:00	4.41	156.87	0.73	1.34
20/10/2009 21:00	4.95	155.05	0.54	2.08
21/10/2009 00:00	2.57	165.00	0.17	-0.30
21/10/2009 03:00	2.21	190.74	0.16	-0.49
21/10/2009 06:00	5.81	152.87	0.48	1.23
21/10/2009 09:00	6.26	152.24	0.61	2.23
21/10/2009 12:00	2.79	176.27	0.26	0.02
21/10/2009 15:00	1.87	188.31	0.17	-0.59
21/10/2009 18:00	5.66	150.37	0.61	1.00
21/10/2009 21:00	6.50	151.16	0.74	2.17
22/10/2009 00:00	3.33	165.62	0.34	0.22
22/10/2009 03:00	1.83	180.33	0.17	-0.65
22/10/2009 06:00	5.58	148.95	0.64	0.93
22/10/2009 09:00	6.42	151.94	0.78	2.19
22/10/2009 12:00	3.56	160.73	0.36	0.41
22/10/2009 15:00	1.49	197.10	0.12	-0.82
22/10/2009 18:00	3.94	163.39	0.33	0.62
22/10/2009 21:00	4.37	173.21	0.39	1.90
23/10/2009 00:00	3.04	182.10	0.25	0.33
23/10/2009 03:00	1.38	228.02	0.10	-0.99
23/10/2009 06:00	3.25	178.08	0.24	0.60
23/10/2009 09:00	3.99	178.58	0.30	1.87
23/10/2009 12:00	3.55	167.23	0.23	0.53
23/10/2009 15:00	1.45	261.69	0.08	-0.94
23/10/2009 18:00	3.44	158.82	0.21	0.36
23/10/2009 21:00	4.56	163.50	0.31	1.56

Wave and Water Level Conditions of Storm 3/3/2010, for Nearshore Point 8				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
02/03/2010 06:00	1.44	69.58	0.03	1.35
02/03/2010 09:00	2.63	80.88	0.04	2.37
02/03/2010 12:00	6.00	87.61	0.09	-0.33
02/03/2010 15:00	5.86	120.43	0.24	-1.34
02/03/2010 18:00	2.14	137.36	0.12	0.81
02/03/2010 21:00	2.31	140.06	0.13	2.32
03/03/2010 00:00	2.23	117.85	0.26	-0.14
03/03/2010 03:00	2.07	113.49	0.23	-1.28
03/03/2010 06:00	4.41	131.93	0.77	0.84
03/03/2010 09:00	4.74	130.64	0.82	2.71
03/03/2010 12:00	2.49	113.83	0.30	0.29
03/03/2010 15:00	1.78	99.27	0.17	-1.45
03/03/2010 18:00	4.20	130.33	0.59	0.33
03/03/2010 21:00	4.89	129.63	0.72	2.43
04/03/2010 00:00	2.77	116.47	0.33	0.50
04/03/2010 03:00	1.76	98.86	0.17	-1.51
04/03/2010 06:00	4.20	131.35	0.64	0.26
04/03/2010 09:00	5.16	130.20	0.88	2.38
04/03/2010 12:00	3.21	123.53	0.43	0.73
04/03/2010 15:00	1.68	98.08	0.17	-1.42
04/03/2010 18:00	3.82	131.55	0.52	-0.16
04/03/2010 21:00	4.80	129.45	0.65	1.77
05/03/2010 00:00	3.23	128.06	0.37	0.77
05/03/2010 03:00	1.79	102.27	0.12	-1.43
05/03/2010 06:00	3.29	132.98	0.34	-0.19
05/03/2010 09:00	4.25	127.72	0.38	1.62
05/03/2010 12:00	3.41	127.25	0.26	0.92
05/03/2010 15:00	1.94	72.61	0.07	-1.30
05/03/2010 18:00	3.80	108.98	0.20	-0.28
05/03/2010 21:00	7.06	96.39	0.18	1.39

Wave and Water Level Conditions of Storm 9/10/2010, for Nearshore Point 8				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
07/10/2010 00:00	1.86	214.40	0.12	-1.22
07/10/2010 03:00	4.05	164.17	0.25	0.43
07/10/2010 06:00	4.58	168.42	0.27	2.41
07/10/2010 09:00	3.94	163.02	0.23	0.66
07/10/2010 12:00	2.17	154.14	0.14	-1.29
07/10/2010 15:00	5.28	145.41	0.51	0.34
07/10/2010 18:00	4.55	135.99	0.92	2.55
07/10/2010 21:00	3.43	132.98	0.50	1.17
08/10/2010 00:00	1.80	108.49	0.15	-1.15
08/10/2010 03:00	3.04	131.06	0.41	-0.12
08/10/2010 06:00	4.15	131.92	0.62	2.17
08/10/2010 09:00	3.20	132.20	0.43	1.49
08/10/2010 12:00	1.79	109.42	0.13	-0.99
08/10/2010 15:00	2.81	125.18	0.34	-0.21
08/10/2010 18:00	4.47	129.03	0.63	2.02
08/10/2010 21:00	5.46	140.90	0.75	1.87
09/10/2010 00:00	1.83	163.72	0.13	-0.79
09/10/2010 03:00	3.29	161.40	0.27	-0.64
09/10/2010 06:00	6.54	149.94	0.74	1.63
09/10/2010 09:00	5.46	145.16	0.90	2.05
09/10/2010 12:00	1.89	166.04	0.19	-0.61
09/10/2010 15:00	2.96	168.53	0.30	-0.69
09/10/2010 18:00	6.67	148.72	0.88	1.46
09/10/2010 21:00	6.10	146.65	0.95	2.33
10/10/2010 00:00	2.15	145.86	0.23	-0.36
10/10/2010 03:00	2.44	137.61	0.27	-0.90
10/10/2010 06:00	5.88	143.59	1.00	1.12
10/10/2010 09:00	5.85	136.79	0.95	2.28
10/10/2010 12:00	2.65	113.17	0.26	-0.20
10/10/2010 15:00	2.34	107.62	0.21	-1.01
10/10/2010 18:00	5.59	133.84	0.79	0.88
10/10/2010 21:00	6.04	133.77	0.89	2.33

Wave and Water Level Conditions of Storm 23/10/2009, for Nearshore Point 12				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
20/10/2009 00:00	1.07	163.53	0.14	-0.58
20/10/2009 03:00	1.76	164.18	0.27	-0.23
20/10/2009 06:00	4.11	159.93	0.94	1.65
20/10/2009 09:00	3.75	164.17	0.79	2.23
20/10/2009 12:00	1.22	170.07	0.16	-0.29
20/10/2009 15:00	1.49	181.13	0.21	-0.42
20/10/2009 18:00	3.95	167.78	0.61	1.34
20/10/2009 21:00	4.48	166.46	0.44	2.08
21/10/2009 00:00	1.87	208.06	0.12	-0.30
21/10/2009 03:00	1.63	214.86	0.11	-0.49
21/10/2009 06:00	5.46	163.61	0.40	1.23
21/10/2009 09:00	5.85	162.59	0.51	2.23
21/10/2009 12:00	2.08	200.47	0.20	0.02
21/10/2009 15:00	1.30	213.90	0.12	-0.59
21/10/2009 18:00	4.91	167.07	0.50	1.00
21/10/2009 21:00	5.92	162.63	0.63	2.17
22/10/2009 00:00	2.51	192.24	0.27	0.22
22/10/2009 03:00	1.26	210.68	0.13	-0.65
22/10/2009 06:00	4.66	169.33	0.53	0.93
22/10/2009 09:00	5.86	162.17	0.67	2.19
22/10/2009 12:00	2.77	186.86	0.28	0.41
22/10/2009 15:00	1.00	223.63	0.08	-0.82
22/10/2009 18:00	3.65	174.18	0.25	0.62
22/10/2009 21:00	5.04	167.06	0.30	1.90
23/10/2009 00:00	2.81	187.11	0.17	0.33
23/10/2009 03:00	1.06	232.56	0.06	-0.99
23/10/2009 06:00	3.31	179.80	0.17	0.60
23/10/2009 09:00	4.87	170.59	0.23	1.87
23/10/2009 12:00	3.40	176.77	0.17	0.53
23/10/2009 15:00	1.08	227.09	0.05	-0.94
23/10/2009 18:00	3.02	182.53	0.15	0.36
23/10/2009 21:00	5.00	167.74	0.25	1.56

Wave and Water Level Conditions of Storm 3/3/2010, for Nearshore Point 12				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
02/03/2010 06:00	1.38	213.32	0.02	1.35
02/03/2010 09:00	2.62	181.09	0.03	2.37
02/03/2010 12:00	5.95	139.32	0.08	-0.33
02/03/2010 15:00	5.26	152.81	0.20	-1.34
02/03/2010 18:00	1.49	199.15	0.09	0.81
02/03/2010 21:00	1.67	197.93	0.09	2.32
03/03/2010 00:00	1.38	133.60	0.18	-0.14
03/03/2010 03:00	1.25	126.82	0.15	-1.28
03/03/2010 06:00	3.64	144.66	0.63	0.84
03/03/2010 09:00	3.95	140.02	0.69	2.71
03/03/2010 12:00	1.71	126.36	0.20	0.29
03/03/2010 15:00	1.01	103.56	0.10	-1.45
03/03/2010 18:00	3.39	143.66	0.46	0.33
03/03/2010 21:00	4.04	136.51	0.60	2.43
04/03/2010 00:00	1.97	128.55	0.22	0.50
04/03/2010 03:00	0.98	103.22	0.09	-1.51
04/03/2010 06:00	3.35	147.55	0.50	0.26
04/03/2010 09:00	4.28	137.55	0.75	2.38
04/03/2010 12:00	2.38	141.07	0.30	0.73
04/03/2010 15:00	0.92	103.20	0.09	-1.42
04/03/2010 18:00	2.99	149.16	0.39	-0.16
04/03/2010 21:00	3.99	135.23	0.55	1.77
05/03/2010 00:00	2.48	143.35	0.25	0.77
05/03/2010 03:00	0.89	103.76	0.07	-1.43
05/03/2010 06:00	2.64	146.53	0.24	-0.19
05/03/2010 09:00	3.64	134.03	0.34	1.62
05/03/2010 12:00	2.80	142.24	0.18	0.92
05/03/2010 15:00	1.04	114.50	0.05	-1.30
05/03/2010 18:00	3.10	150.73	0.15	-0.28
05/03/2010 21:00	6.29	133.51	0.17	1.39

Wave and Water Level Conditions of Storm 9/10/2010, for Nearshore Point 12				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
07/10/2010 00:00	1.41	224.41	0.08	-1.22
07/10/2010 03:00	3.94	166.22	0.19	0.43
07/10/2010 06:00	5.62	161.49	0.21	2.41
07/10/2010 09:00	3.36	176.15	0.18	0.66
07/10/2010 12:00	1.46	212.17	0.10	-1.29
07/10/2010 15:00	4.10	171.05	0.42	0.34
07/10/2010 18:00	3.89	144.44	0.79	2.55
07/10/2010 21:00	2.69	153.33	0.37	1.17
08/10/2010 00:00	0.90	118.20	0.09	-1.15
08/10/2010 03:00	2.32	147.61	0.30	-0.12
08/10/2010 06:00	3.57	138.72	0.53	2.17
08/10/2010 09:00	2.60	145.65	0.33	1.49
08/10/2010 12:00	0.88	113.29	0.08	-0.99
08/10/2010 15:00	2.09	138.33	0.23	-0.21
08/10/2010 18:00	3.78	135.77	0.54	2.02
08/10/2010 21:00	4.36	164.31	0.65	1.87
09/10/2010 00:00	1.20	214.60	0.09	-0.79
09/10/2010 03:00	2.51	192.49	0.20	-0.64
09/10/2010 06:00	5.77	162.09	0.64	1.63
09/10/2010 09:00	4.51	167.22	0.78	2.05
09/10/2010 12:00	1.30	198.11	0.14	-0.61
09/10/2010 15:00	2.19	196.74	0.24	-0.69
09/10/2010 18:00	5.77	163.81	0.75	1.46
09/10/2010 21:00	5.12	166.50	0.83	2.33
10/10/2010 00:00	1.45	178.81	0.17	-0.36
10/10/2010 03:00	1.63	169.71	0.20	-0.90
10/10/2010 06:00	4.96	160.92	0.89	1.12
10/10/2010 09:00	4.86	153.27	0.83	2.28
10/10/2010 12:00	1.68	131.61	0.19	-0.20
10/10/2010 15:00	1.39	122.44	0.14	-1.01
10/10/2010 18:00	4.54	150.96	0.67	0.88
10/10/2010 21:00	5.01	146.58	0.77	2.33

Wave and Water Level Conditions of Storm 23/10/2009, for Nearshore Point 14				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
20/10/2009 00:00	3.41	152.34	0.61	-0.58
20/10/2009 03:00	4.15	151.67	0.87	-0.23
20/10/2009 06:00	5.75	152.88	1.42	1.65
20/10/2009 09:00	5.62	154.38	1.36	2.23
20/10/2009 12:00	3.66	154.10	0.67	-0.29
20/10/2009 15:00	3.83	158.27	0.67	-0.42
20/10/2009 18:00	5.34	165.40	0.80	1.34
20/10/2009 21:00	5.83	167.93	0.56	2.08
21/10/2009 00:00	5.42	161.46	0.36	-0.30
21/10/2009 03:00	5.04	162.35	0.35	-0.49
21/10/2009 06:00	6.95	167.11	0.52	1.23
21/10/2009 09:00	7.27	166.41	0.65	2.23
21/10/2009 12:00	5.58	161.08	0.56	0.02
21/10/2009 15:00	4.91	159.66	0.48	-0.59
21/10/2009 18:00	7.24	164.69	0.77	1.00
21/10/2009 21:00	7.54	165.39	0.80	2.17
22/10/2009 00:00	6.00	161.08	0.69	0.22
22/10/2009 03:00	4.87	158.97	0.52	-0.65
22/10/2009 06:00	7.37	163.44	0.87	0.93
22/10/2009 09:00	7.31	165.98	0.82	2.19
22/10/2009 12:00	5.88	163.19	0.63	0.41
22/10/2009 15:00	4.34	161.24	0.36	-0.82
22/10/2009 18:00	5.26	171.99	0.39	0.62
22/10/2009 21:00	4.79	178.91	0.36	1.90
23/10/2009 00:00	4.35	175.29	0.30	0.33
23/10/2009 03:00	3.42	170.52	0.22	-0.99
23/10/2009 06:00	4.19	179.81	0.26	0.60
23/10/2009 09:00	4.21	183.77	0.26	1.87
23/10/2009 12:00	4.55	176.26	0.26	0.53
23/10/2009 15:00	3.76	167.06	0.20	-0.94
23/10/2009 18:00	4.70	171.46	0.26	0.36
23/10/2009 21:00	4.85	176.13	0.29	1.56

Wave and Water Level Conditions of Storm 3/3/2010, for Nearshore Point 14				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
02/03/2010 06:00	5.19	152.74	0.07	1.35
02/03/2010 09:00	6.05	151.05	0.07	2.37
02/03/2010 12:00	7.31	151.34	0.10	-0.33
02/03/2010 15:00	7.19	158.43	0.27	-1.34
02/03/2010 18:00	4.63	160.62	0.29	0.81
02/03/2010 21:00	4.69	161.46	0.28	2.32
03/03/2010 00:00	3.57	139.33	0.63	-0.14
03/03/2010 03:00	3.53	139.26	0.61	-1.28
03/03/2010 06:00	5.54	132.42	1.13	0.84
03/03/2010 09:00	5.75	130.15	1.13	2.71
03/03/2010 12:00	4.08	135.26	0.70	0.29
03/03/2010 15:00	3.42	136.11	0.50	-1.45
03/03/2010 18:00	5.53	128.95	0.95	0.33
03/03/2010 21:00	5.92	127.55	0.99	2.43
04/03/2010 00:00	4.45	133.19	0.72	0.50
04/03/2010 03:00	3.42	136.47	0.49	-1.51
04/03/2010 06:00	5.62	131.31	1.10	0.26
04/03/2010 09:00	6.17	129.14	1.21	2.38
04/03/2010 12:00	4.92	133.81	0.94	0.73
04/03/2010 15:00	3.42	137.59	0.51	-1.42
04/03/2010 18:00	5.35	131.14	0.96	-0.16
04/03/2010 21:00	5.74	127.05	0.87	1.77
05/03/2010 00:00	4.79	130.04	0.69	0.77
05/03/2010 03:00	3.16	135.31	0.38	-1.43
05/03/2010 06:00	4.74	128.23	0.58	-0.19
05/03/2010 09:00	5.01	123.57	0.49	1.62
05/03/2010 12:00	4.79	130.76	0.40	0.92
05/03/2010 15:00	4.68	141.39	0.24	-1.30
05/03/2010 18:00	5.95	143.62	0.34	-0.28
05/03/2010 21:00	8.56	151.03	0.20	1.39

Wave and Water Level Conditions of Storm 9/10/2010, for Nearshore Point 14				
Time	Tp(s)	Direction(°)	Hm0 (m)	Water Level(m)
07/10/2010 00:00	4.53	164.14	0.25	-1.22
07/10/2010 03:00	5.44	170.79	0.30	0.43
07/10/2010 06:00	5.39	174.47	0.26	2.41
07/10/2010 09:00	5.76	167.10	0.31	0.66
07/10/2010 12:00	5.21	159.93	0.37	-1.29
07/10/2010 15:00	7.59	159.93	0.84	0.34
07/10/2010 18:00	5.50	136.63	1.17	2.55
07/10/2010 21:00	4.90	134.12	0.95	1.17
08/10/2010 00:00	3.08	138.21	0.47	-1.15
08/10/2010 03:00	4.39	134.27	0.77	-0.12
08/10/2010 06:00	4.94	129.98	0.79	2.17
08/10/2010 09:00	4.27	130.10	0.69	1.49
08/10/2010 12:00	2.82	133.98	0.40	-0.99
08/10/2010 15:00	3.99	130.32	0.63	-0.21
08/10/2010 18:00	5.29	126.83	0.82	2.02
08/10/2010 21:00	7.49	150.00	1.37	1.87
09/10/2010 00:00	4.88	159.49	0.40	-0.79
09/10/2010 03:00	5.73	162.69	0.48	-0.64
09/10/2010 06:00	7.56	164.31	0.80	1.63
09/10/2010 09:00	7.23	155.82	1.52	2.05
09/10/2010 12:00	4.67	157.86	0.60	-0.61
09/10/2010 15:00	5.73	160.33	0.67	-0.69
09/10/2010 18:00	7.91	163.26	1.00	1.46
09/10/2010 21:00	7.76	159.48	1.36	2.33
10/10/2010 00:00	4.57	155.51	0.71	-0.36
10/10/2010 03:00	4.67	153.66	0.78	-0.90
10/10/2010 06:00	7.42	153.38	1.43	1.12
10/10/2010 09:00	7.41	142.78	1.38	2.28
10/10/2010 12:00	4.89	141.99	0.71	-0.20
10/10/2010 15:00	4.61	141.48	0.61	-1.01
10/10/2010 18:00	7.24	137.70	1.18	0.88
10/10/2010 21:00	7.49	137.49	1.19	2.33

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