Appendix 12:
AMETS Coastal Process Report

UNIVERSITY COLLEGE CORK

UCC

Coláiste na hOllscoile Corcaigh

Atlantic Marine Energy Test Site (AMETS)

Environmental Impact Assessment

seai SUSTAINABLE ENERGY AUTHORITY OF IRELAND | Ocean Energy Development Unit

Coastal Processes Report

08 December 2011

Hydraulics & Maritime Research Centre
University College Cork
Ireland
Tel: +353 (0)21 4250021
Fax: +353 (0)21-4321003
Email: hmrc@ucc.ie

HMRC HYDRAULICS & MARITIME RESEARCH CENTRE

Table of Contents

1. Introduction

The HMRC has carried out an analysis of the baseline conditions at Belmullet with the view of assessing the potential coastal impacts of limited Wave Energy Convertor (WEC) deployments in the test areas A and B. A breakdown of the work undertaken in this study is provided below.

- Wave Climate Analysis – Analysis of a relevant long term wave record in order to determine the offshore wave conditions
- Analysis of collected wave data and its use in the calibration of the numerical models
- Numerical modelling – refined modelling using MIKE21 SW to determine nearshore wave conditions at the site. This will be used to determine the nature of sediment transport at the site. Wave modelling of various proposed interventions to assess their potential impacts.
- Review and assessment of available current data: The nature of tidal currents in the coastal area will be examined using field and numerical model data. The significance of these will be discussed.
- Review and assessment of historical coastal position. Various Ordnance maps and aerial photographs will be used to show how the coastline position has changed historically.
- Assessment of beach profile data – based on the available profile data the type of beach system can be defined and its typical behaviour patterns explained.
- Sediment Transport Analysis – Examination of existing sediment transport regime and potential changes that are likely to occur as a consequence of the various works.
- Overall discussion on implications of proposed works

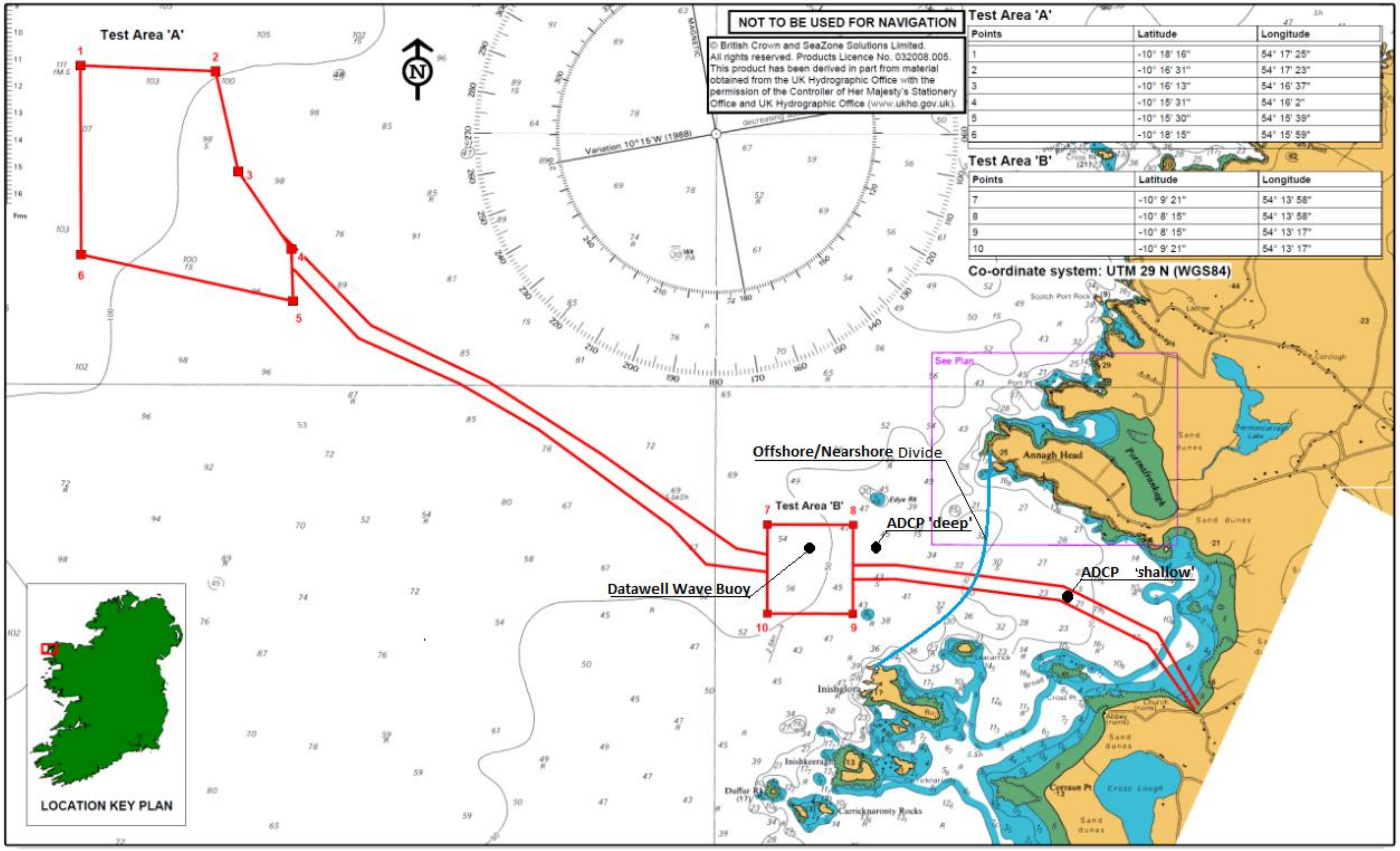
Each of these aspects of work will now be described in detail and conclusions regarding the potential impacts will be presented.

2. Project Methodology

The study relates to undertaking an analysis of all available existing and newly collected data in order to provide where relevant, both a quantitative and qualitative assessment of the coastal processes in the coastal areas related to the test sites. The proposed works to be carried out at the test sites have been generally outlined so it is possible to examine what the impacts of these may be. Wave modelling is considered necessary at this stage but hydrodynamic and sediment transport modelling will not be undertaken due to the scoping review having shown that it may not be fully relevant. However sediment transport will be reviewed more qualitatively by considering the environmental forcings and the sediment characteristics with the expected conclusion that baseline conditions at the site are so extreme that the seabed is naturally mobile.

3. Wave Analysis

This section encompasses various types of analysis and modelling . Both long term and short term wave records will be examined with the view of understanding the wave conditions at the site and also to provide input into the numerical model. The study considers the conditions in Annagh Bay with particular emphasis on Belderra beach, the proposed landfall for the cables. Note that in the text reference will be made to offshore and nearshore conditions. The divide between offshore and nearshore is arbitrary and in this case refers to locations east and west of Annagh Head as indicated in Figure 3.1.



Test Area 'A'

Points	Latitude	Longitude
1	-10° 18' 16"	54° 17' 25"
2	-10° 16' 31"	54° 17' 23"
3	-10° 16' 13"	54° 16' 37"
4	-10° 15' 31"	54° 16' 2"
5	-10° 15' 30"	54° 15' 39"
6	-10° 18' 15"	54° 15' 59"

Test Area 'B'

Points	Latitude	Longitude
7	-10° 9' 21"	54° 13' 58"
8	-10° 8' 15"	54° 13' 58"
9	-10° 8' 15"	54° 13' 17"
10	-10° 9' 21"	54° 13' 17"

Co-ordinate system: UTM 29 N (WGS84)

Figure 3.1 Wave and Current Measurement locations

3.1 Wave Climate Analysis

In is important to establish the wave climate in Annagh Bay and this is achieved by first determining the nature of the offshore wave climate, which can then be transformed using numerical models to the nearshore area. Sixteen years of data as was output from a wave climate analysis of the test sites was obtained from the Numerics Warehouse at a location west of Test Area A (Lat.: -10 29.91, Long.: 54 20.9). This data was first characterised as shown in Figure 3.2 which is a wave rose plot and Figures 3.3 and 3.4 which are scatter diagrams relating significant wave height (mean of highest one third waves, H_s) to peak wave periods and peak wave directions respectively. It is this discretisation of the wave conditions that was used as input to a numerical model required for the transformation/propagation of the offshore wave conditions to closer to the area of interest in Annagh Bay.

From these plots the following general features on the wave conditions can be observed,

- The predominant wave directions range is from the WNW to WSW,
- The Hs value is generally less than 5m but extreme values up to 15m have been modelled. It should be noted that these values are not maximum wave heights. Assuming a Rayleigh distribution of wave heights, which is common in deeper waters, then the maximum wave height can be twice the Hs value. Therefore waves heights of 30m can occur in the 100-200m water depth range off Belmullet.
- Peak wave periods in excess of 20s can occur but the most commonly occurring values range between 8 and 12 seconds. Note that these are peak periods and not mean periods (T_z) which can be considerably lower depending on the nature of the wave conditions.. A typical relationship between T_p and T_z is $T_p = 1.4T_z$ but in reality there is a lot of variability between these two parameters. T_z is the wave period normally used to help characterise WEC performance.

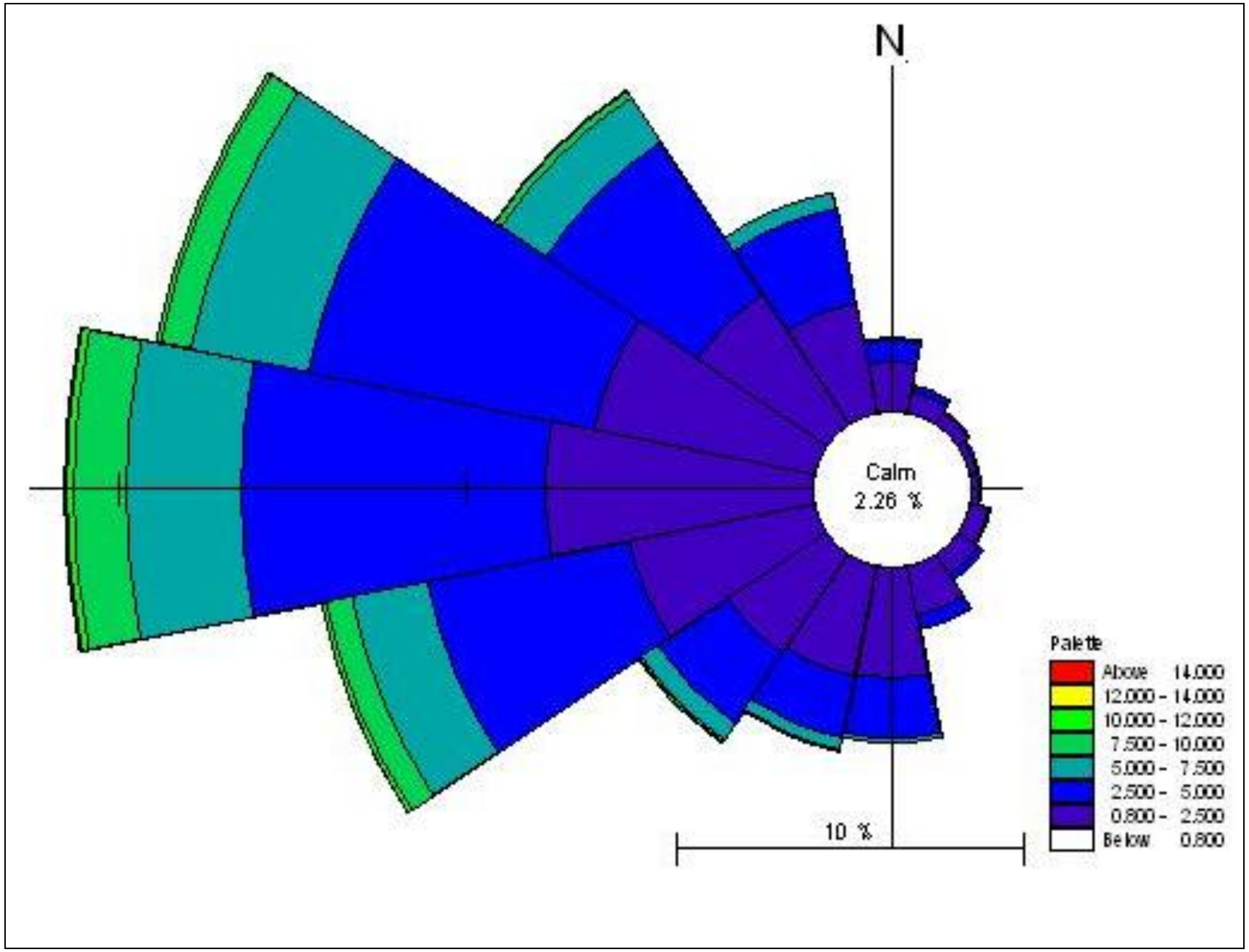


Figure 3.2 Offshore Wave Rose

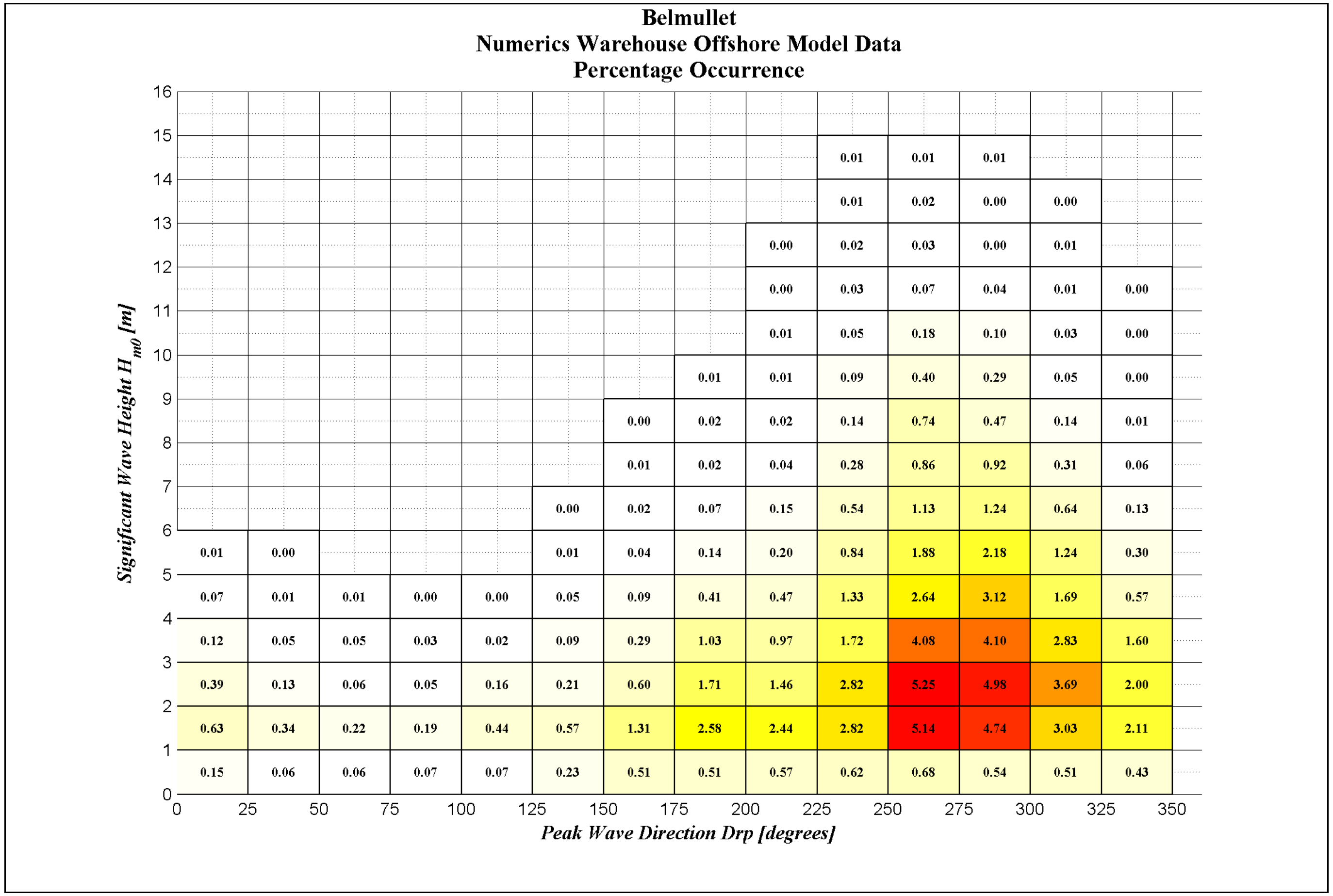


Figure 3.3 Offshore Hs/Wave Direction Scatter Diagram

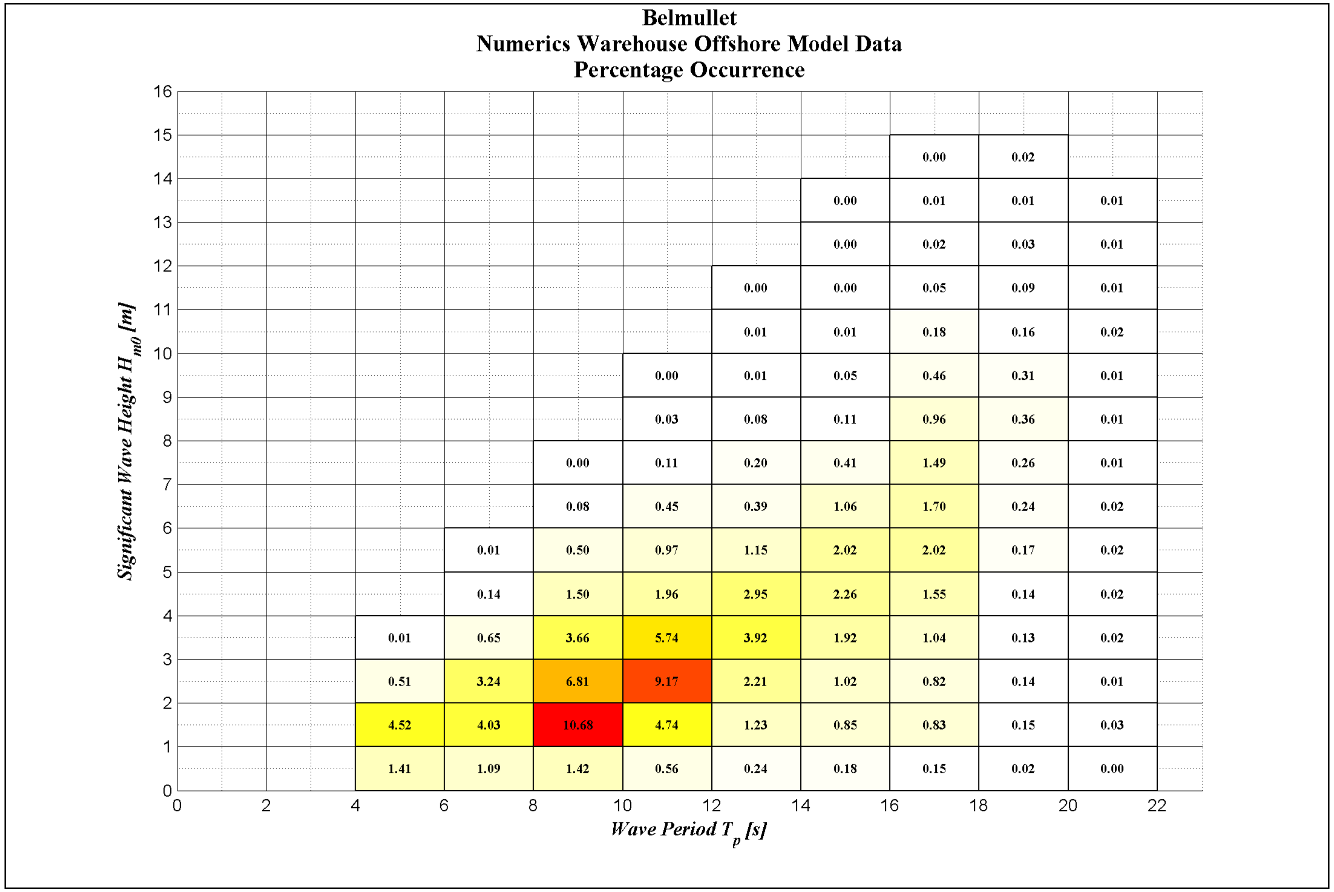


Figure 3.4 Offshore Hs/Tp Scatter Diagram

3.2 Numerical Modelling

Numerical modelling of the wave conditions in the Belmullet area formed a significant part of this study. The MIKE21 software as developed by the Danish Hydraulic Institute (DHI) was used to undertake the modelling work. This is one of the world's leading commercial software in this sector and has a good reputation in terms of flexibility, ease of use and reliability of its output. The MIKE21 SW wave module was used for all modelling in this project and a short description of the model capabilities is given below.

> *MIKE 21 SW is a state-of-the-art numerical tool for prediction and analysis of wave climates in offshore and coastal areas. It includes a new generation spectral wind-wave model based on unstructured meshes. The model simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas. MIKE 21 SW includes two different formulations:*
>
> - *Directional decoupled parametric formulation*
> - *Fully spectral formulation*
>
> *The directional decoupled parametric formulation is based on a parameterization of the wave action conservation equation. The parameterization is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum as dependent variables following Holthuijsen (1989). A similar approximation is used in MIKE 21 NSW Nearshore Spectral Wind-Wave Module.*
>
> *The fully spectral formulation is based on the wave action conservation equation, as described in e.g. Komen et al. (1994) and Young (1999), where the directional-frequency wave action spectrum is the dependent variable.*
>
> *The basic conservation equations are formulated in either Cartesian co-ordinates for small-scale applications and polar spherical co-ordinates for large-scale applications. MIKE 21 SW includes the following physical phenomena:*
>
> - *Wave growth by action of wind*
> - *Non-linear wave-wave interaction*
> - *Dissipation due to white-capping*
> - *Dissipation due to bottom friction*
> - *Dissipation due to depth-induced wave breaking*
> - *Refraction and shoaling due to depth variations*
> - *Wave-current interaction*

- *Effect of time-varying water depth and flooding and drying*

The discretization of the governing equation in geographical and spectral space is performed using cell-centered finite volume method. In the geographical domain, an unstructured mesh technique is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action.

MIKE 21 SW is used for the assessment of wave climates in offshore and coastal areas - in hindcast and forecast mode. A major application area is the design of offshore, coastal and port structures where accurate assessment of wave loads is of utmost importance to the safe and economic design of these structures. Measured data is often not available during periods long enough to allow for the establishment of sufficiently accurate estimates of extreme sea states. In this case, the measured data can then be supplemented with hindcast data through the simulation of wave conditions during historical storms using MIKE 21 SW. MIKE 21 SW is particularly applicable for simultaneous wave prediction and analysis on regional scale like the North Sea and at local scale like in this application. Coarse spatial and temporal resolution is used for the regional part of the mesh and a high-resolution boundary- and depth-adaptive mesh is describing the shallow water environment at the coastline.

The model domain as shown in Figure 3.5 was set up using bathymetry data from both the Admiralty charts and surveys that were carried out for the test sites. The model extends offshore beyond the Test A location in order to ensure that wave propagation characteristics are properly simulated in each of the test areas. The seaward limit of the model acted as the boundary and along which wave conditions were input.

The MIKE 21 SW module was used for a the following tasks in this project

- Calibration using field measurements
- Transformation of wave climate from offshore to nearshore
- Transformation of extreme wave conditions to nearshore area
- Examining the impact of WECs on nearshore wave conditions

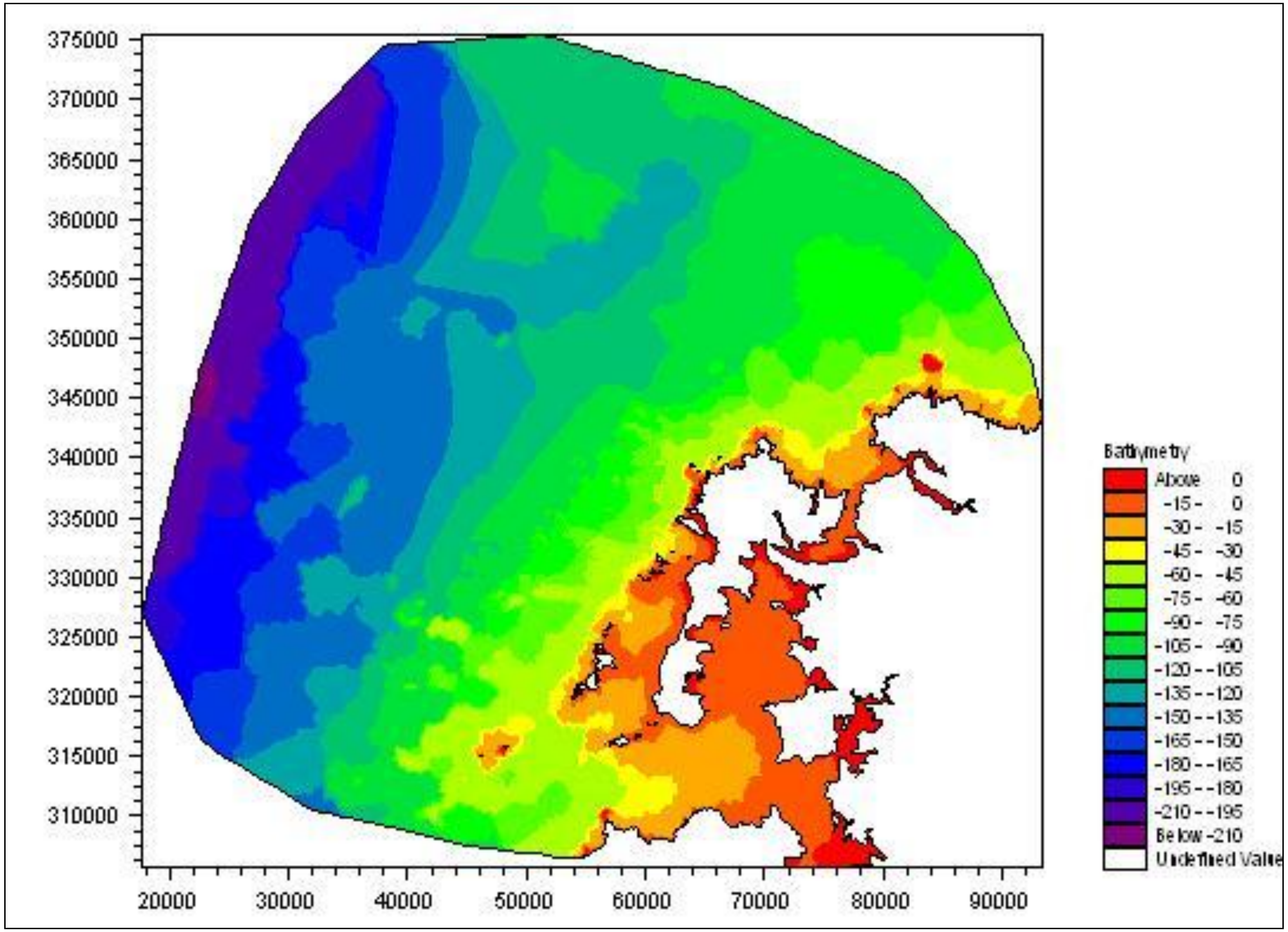


Figure 3.5 MIKE21 SW Model Domain

3.3 Data Collection

Two ADCP wave and current profilers were deployed for a one month period (June/July 2011) and the data collected was important for understanding wave propagation from offshore to nearshore and well as for facilitating the calibration of the numerical model. The gauges were located east of Test Area B area (termed deep) and at the 28m contour in the centre of Annagh Bay (termed shallow). Details of the deployment are provided in Table 3.1 and the location of the gauges is shown in Figure 3.1. Figures 3.6 to 3.9 show the collected statistical summary data and includes a partial overlap with a Datawell wave recorder located in Test Area B (Figure 3.1).

If the wave heights (Figure 3.6) are first considered then it can be seen that because of their proximity there is little change between the Datawell and the 'deep' ADCP but that differences begin to become obvious in the shallow water gauge. Of course there was no major storm events during the deployment periods with the maximum H_s values recorded being of the order of 4m. Larger differences between the two ADCP gauges occur when the waves propagate from a more northerly direction for example between June 21st and 26th. The wave periods are largely the same for the three gauges with the shallow gauge showing disparity for the northerly waves which are more affected by shallow water processes as they pass Annagh Head. Finally it can be seen that there is less directionality in the waves as they come closer to shore and this is important to the understanding of the behaviour of the beach system. Waves tend to travel toward the beaches in Annagh Bay in a W to NW direction especially when the offshore waves are in the W to N quadrant.

Site	Site 1 - Deep Water - TWM01
Date	13/06/2011
Time	09:22
Latitude	54°13'785 N
Longitude	010°07'846 W
Water Depth	53m
Frame #	TWM01
AWAC #	P24208
Acoustic Release #	30732
Acoustic Pinger #	ULB-364/37 SN: DT17080
Battery Pack #	#2
Ballast	450kg - Lead

Site	Site 2 - Shallow Water - TWM02
Date	13/06/2011
Time	14:44
Latitude	54°13'433 N
Longitude	010°05'488 W
Water Depth	28m
Frame #	TWM02
AWAC #	P24209
Acoustic Release #	30731
Acoustic Pinger #	ULB-350/27 SN: DT7398
Battery Pack #	#1
Ballast	450kg - Lead

Table 3.1 Details of Deployment

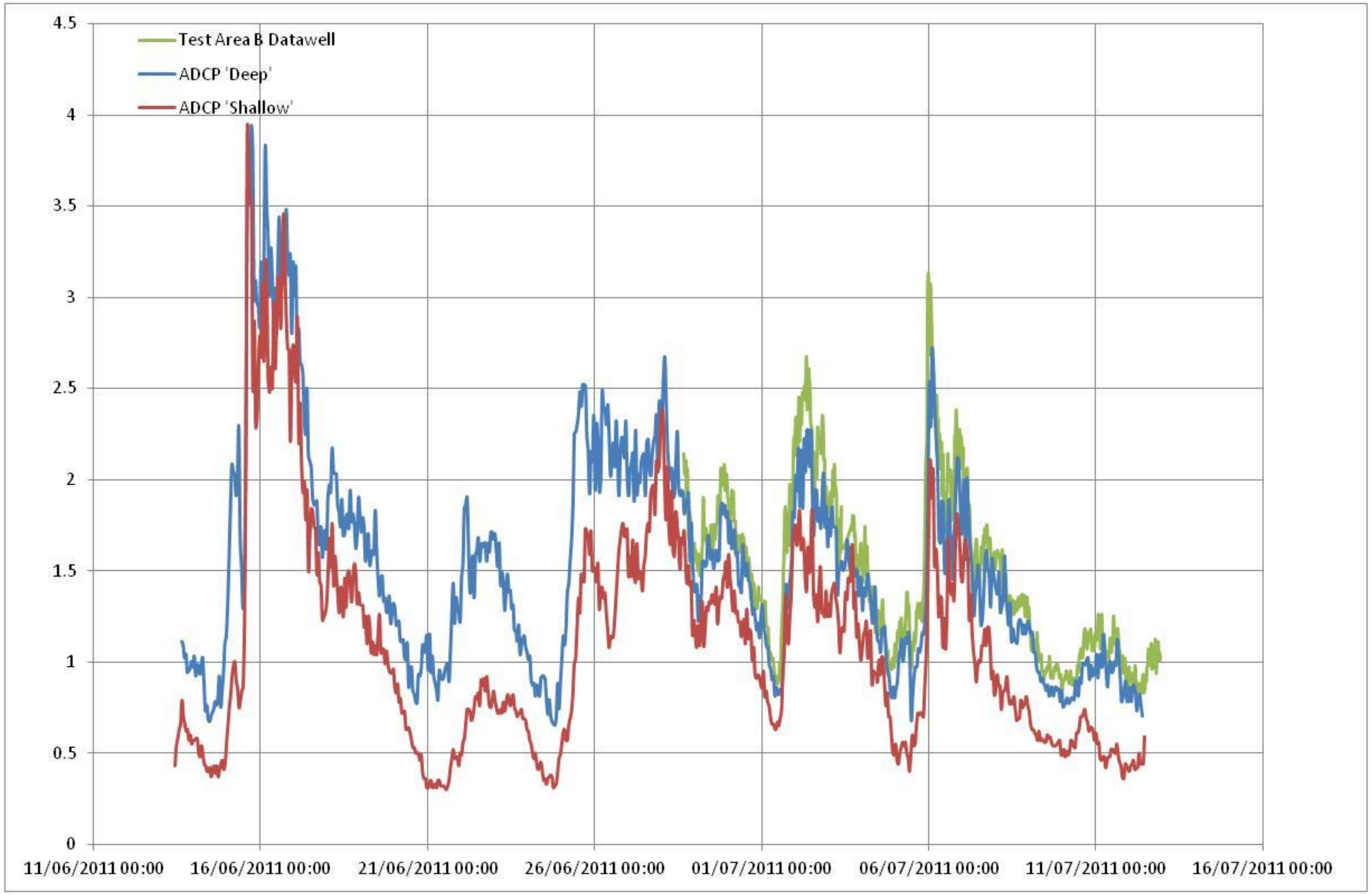


Figure 3.6 Coincident H_s Measurements

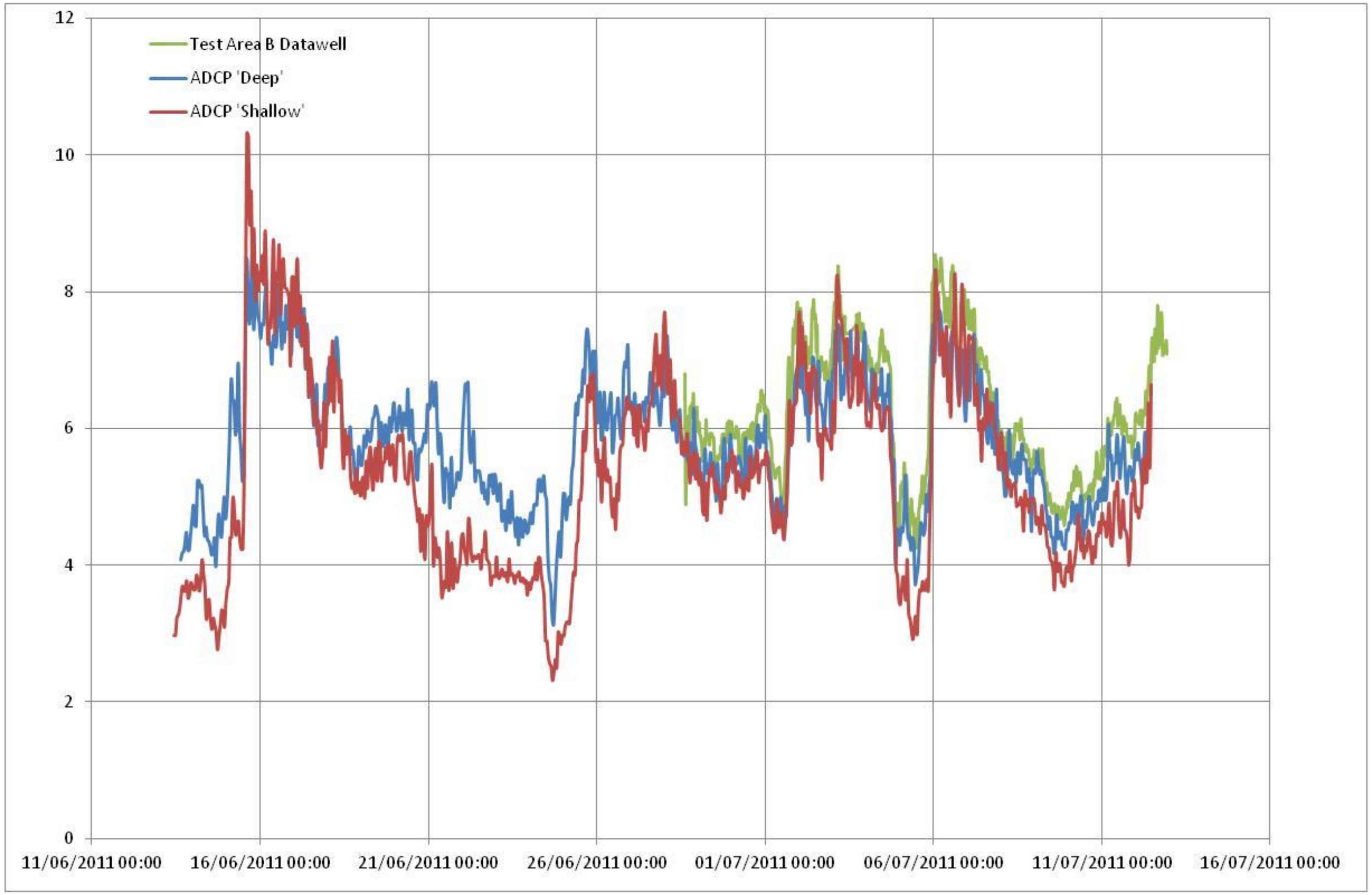


Figure 3.7 Coincident T_z Measurements

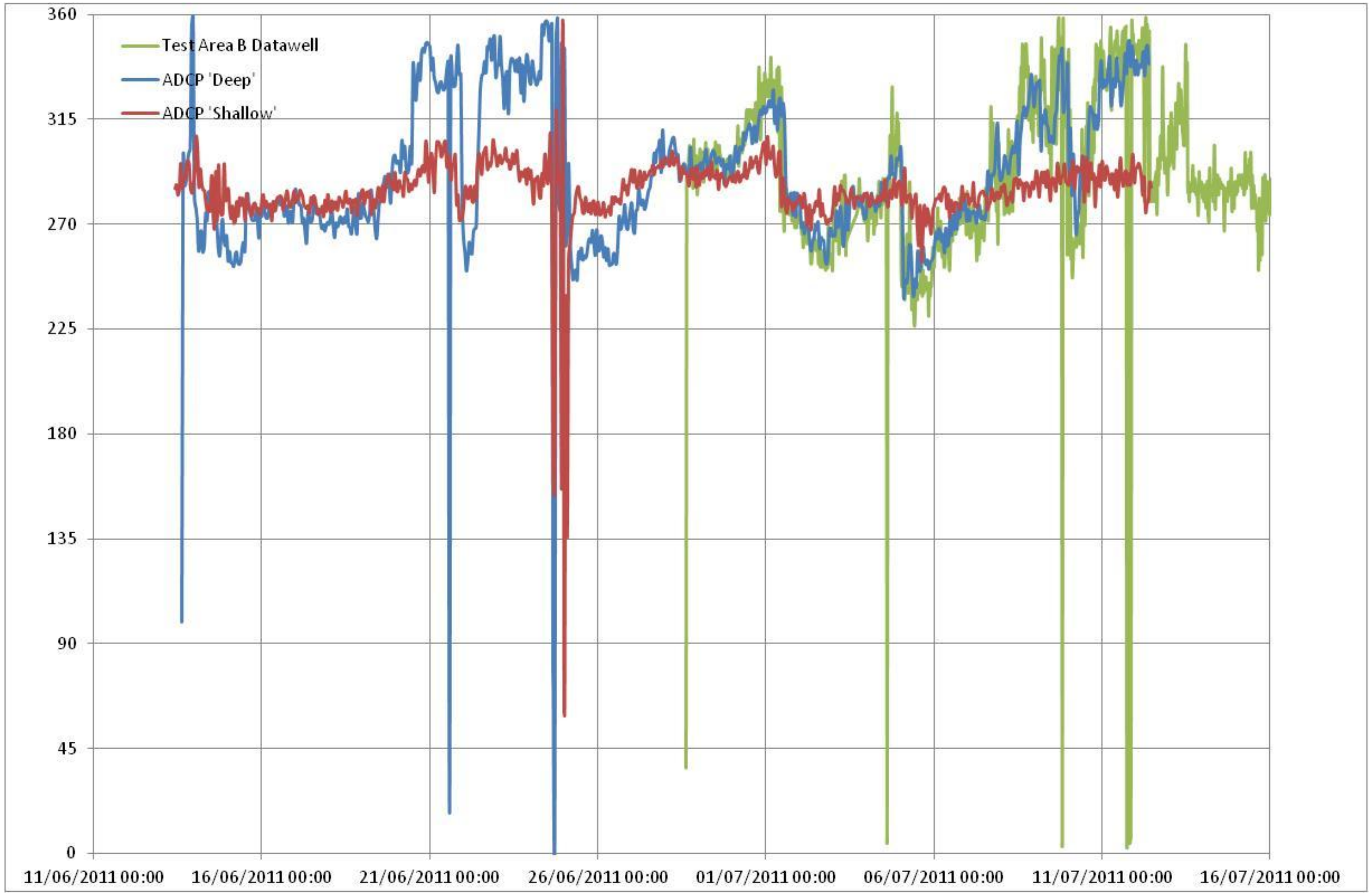


Figure 3.8 Coincident Peak Wave Direction Measurements

3.4 Model Calibration

For the wave model calibration a section of the data from the Datawell buoy was inputed on the boundary of the model. The model then propagated these waves in the input direction where their properties were changed, particularly as they passed Annagh Head. The calibration process had to be repeated a few times in order to get the setup correct and achieve good agreement with the model output and the measured conditions. Figures 3.9 to 3.12 show output from the model calibration process and it can be seen that quite a good calibration was achieved at both ADCP locations and this gives confidence in terms of the accuracy of the remainder of the modelling work. The wave period output is not shown but a similar level of agreement was achieved.

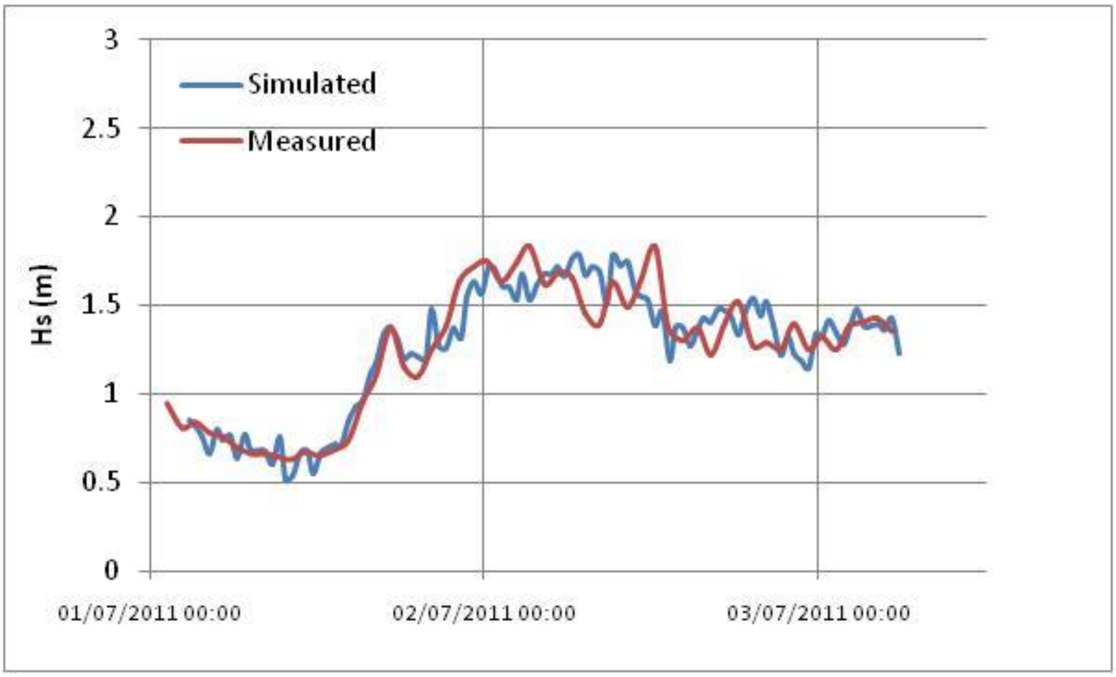


Figure 3.9 H_s calibration curves for Test Area B ADCP location

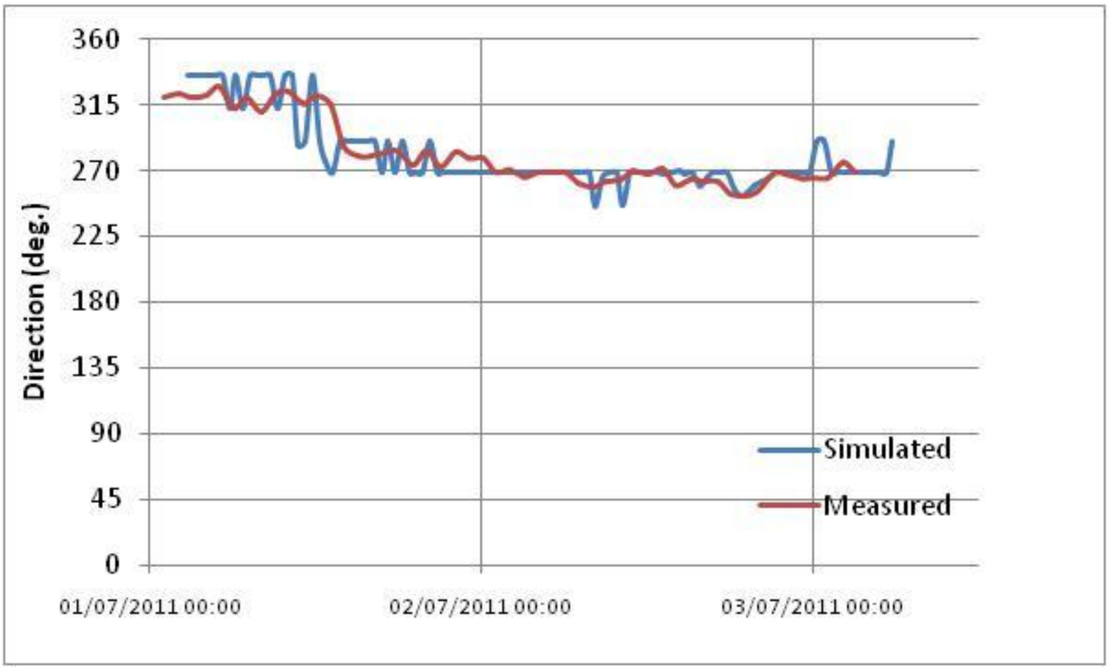


Figure 3.10 Wave Direction calibration curves for Test Area B ADCP location

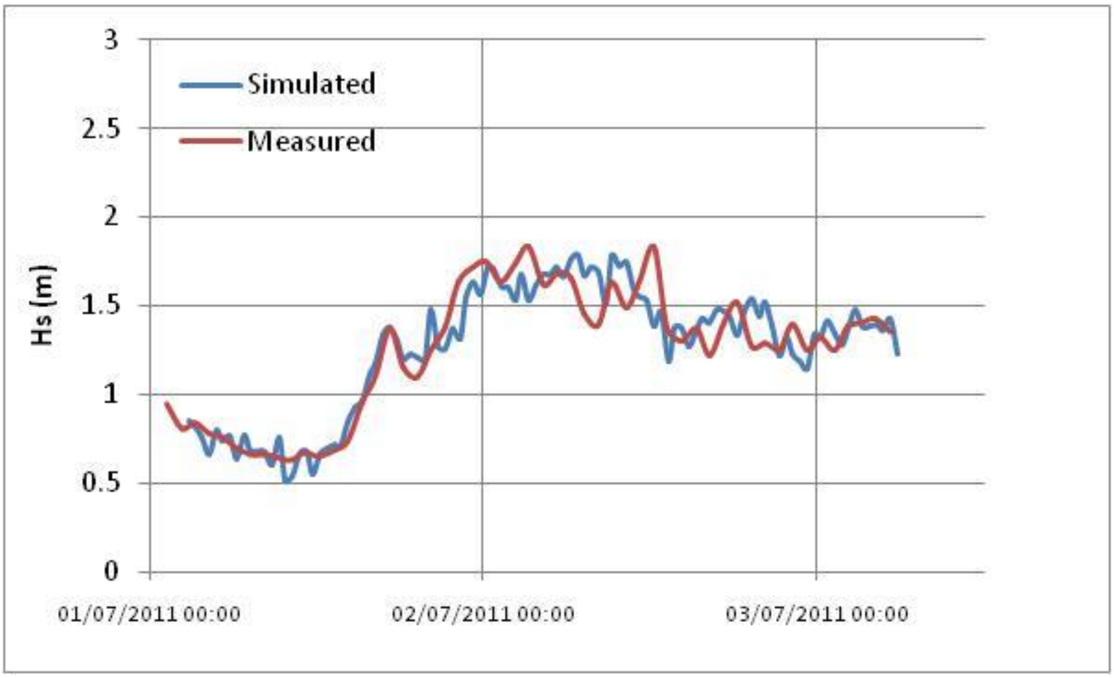


Figure 3.11 H_s calibration curves for 'shallow' ADCP location

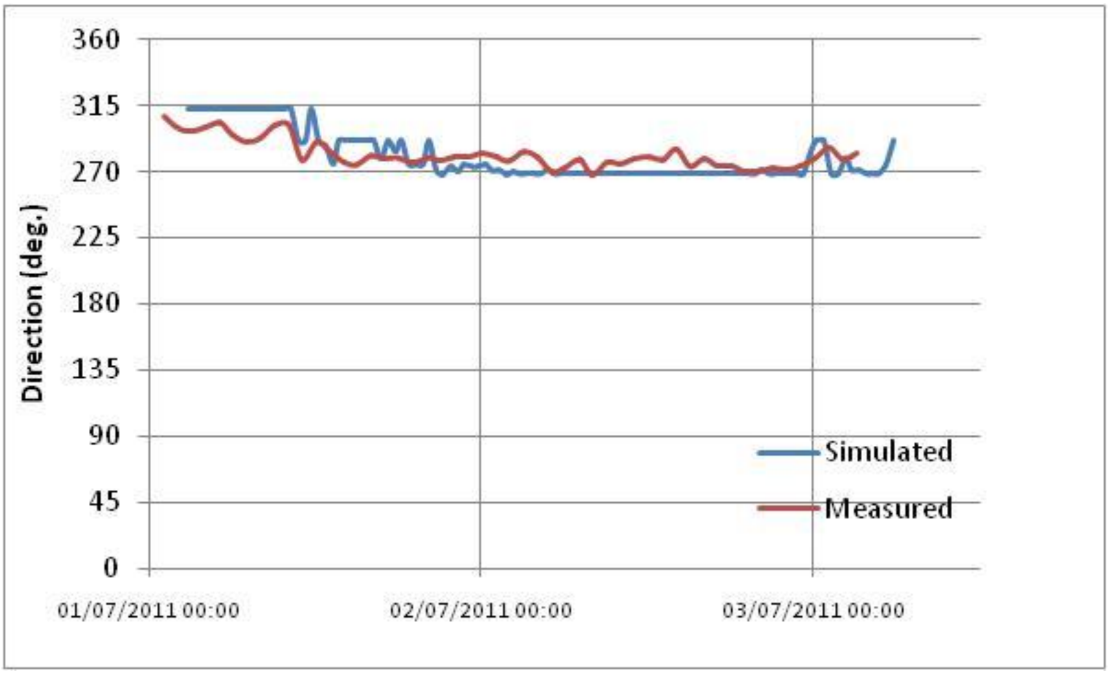


Figure 3.12 Wave Direction calibration curves for 'shallow' ADCP location

3.5 Wave Climate Modelling

The success of the calibration allowed the other aspects of the modelling as outlined in Section 3.2 to proceed. In this case the wave climate as shown in the scatter diagrams (Figure 3.3 and Figure 3.4) was set up as input for the MIKE21 SW model. The simulation was run and the coincident wave climate at two nearshore points as located in Figure 3.13 was determined. For the analysis of the nearshore wave climate the data output point PT2 which is closer to Belderra beach was chosen. The result of this analysis are the scatter diagrams as shown in Figures 3.14 and 3.15.

Figure 3.14 shows that there is a considerable change in the wave conditions from offshore to nearshore both in terms of heights and direction ranges. It can be seen that wide range of offshore directions have been compressed into a mainly WNW propagation direction which is approximately perpendicular to the orientation of Belderra beach. This gives a clear indication as to the nature of wave activity on the beach and the likely sediment transport patterns. Direct wave approach generally result in an onshore/offshore movement of sediment which means that whilst there in variability in the beach profiles, sediment is not lost through longshore movement as would occur for oblique waves. Figure 3.15 shows that the wave dissipation processes cause a general downward shift in the peak wave periods (as compared to their offshore equivalents). Wave breaking and wave to wave interaction processes are the main factors that give rise to this shift.

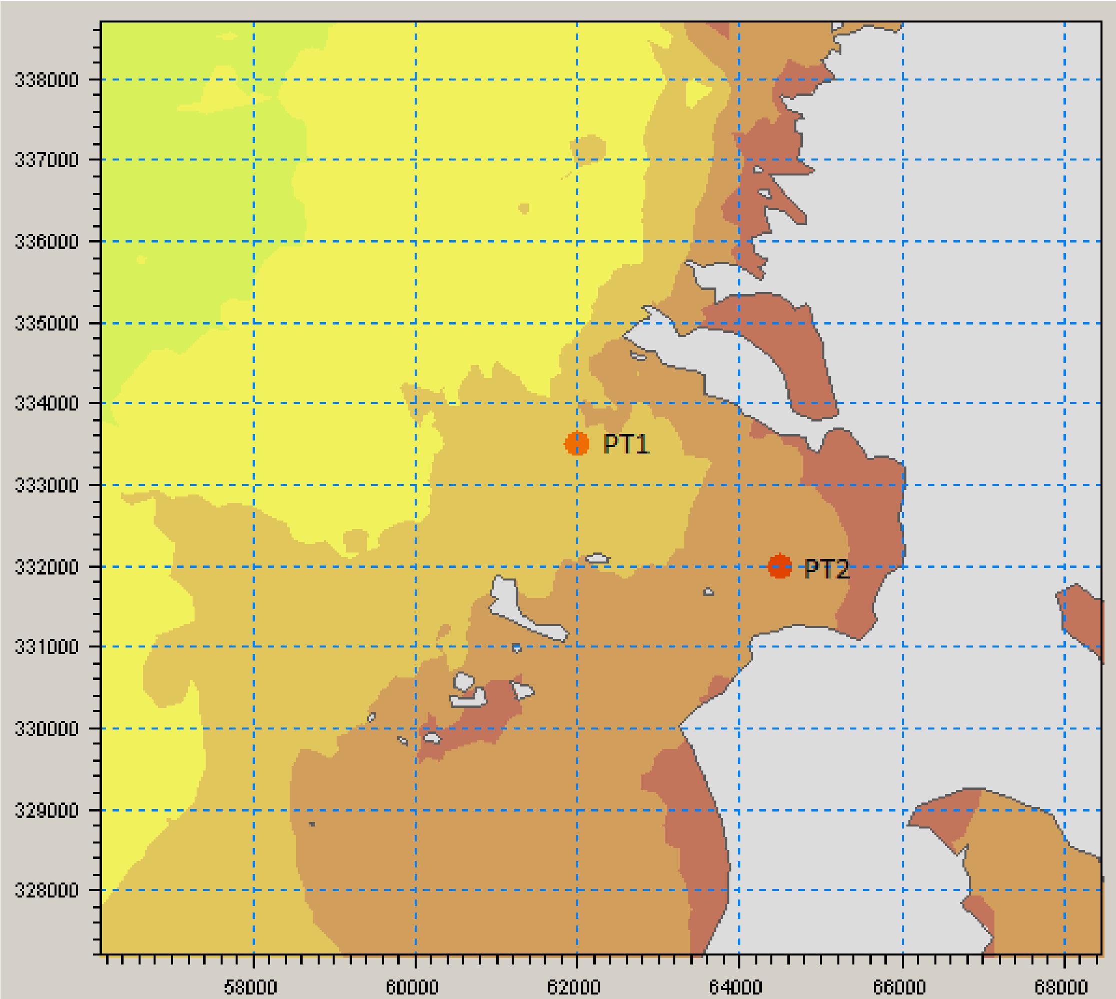


Figure 3.13 Data Extraction Points

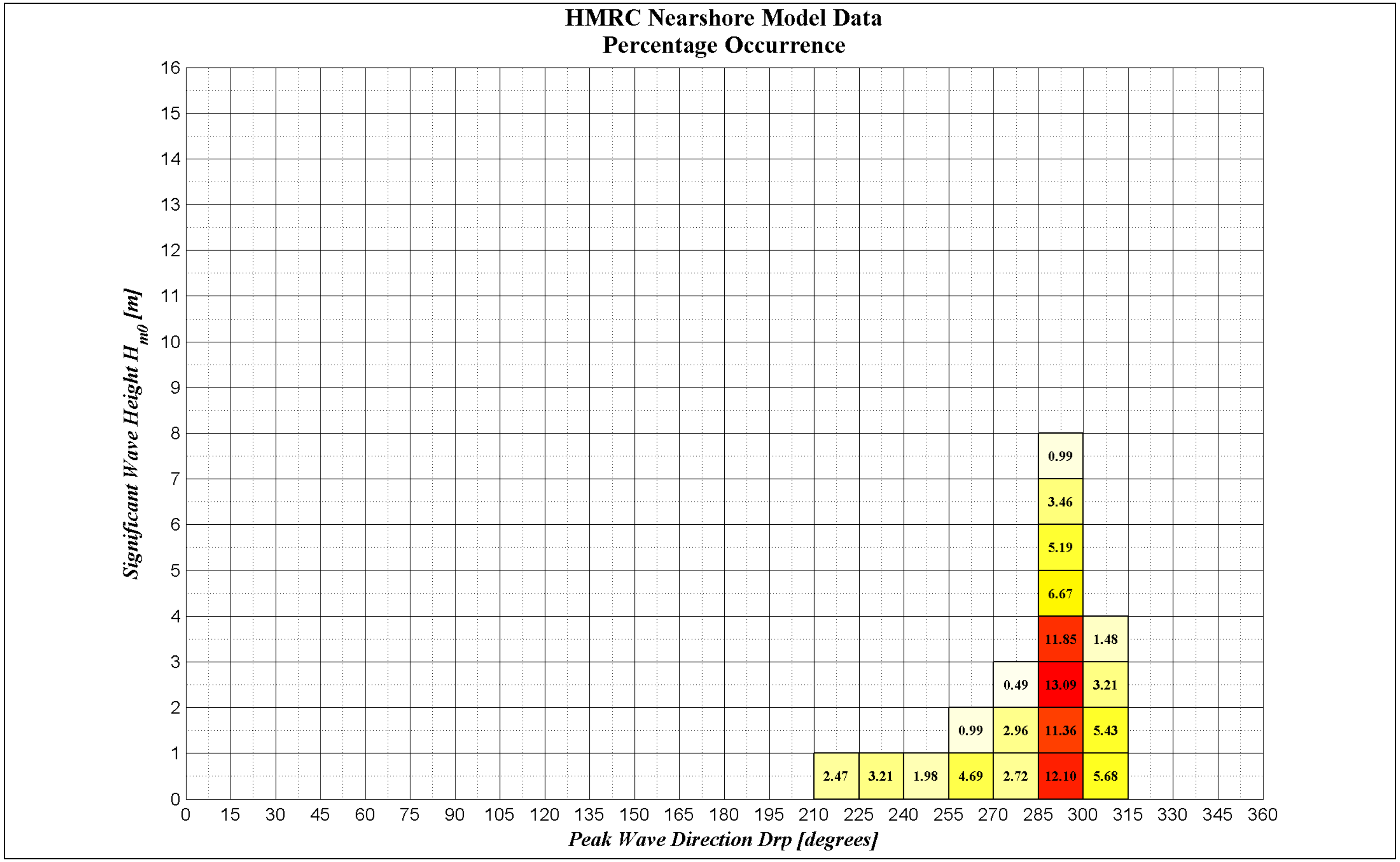


Figure 3.14 PT2 Hs/Wave Direction Scatter Diagram

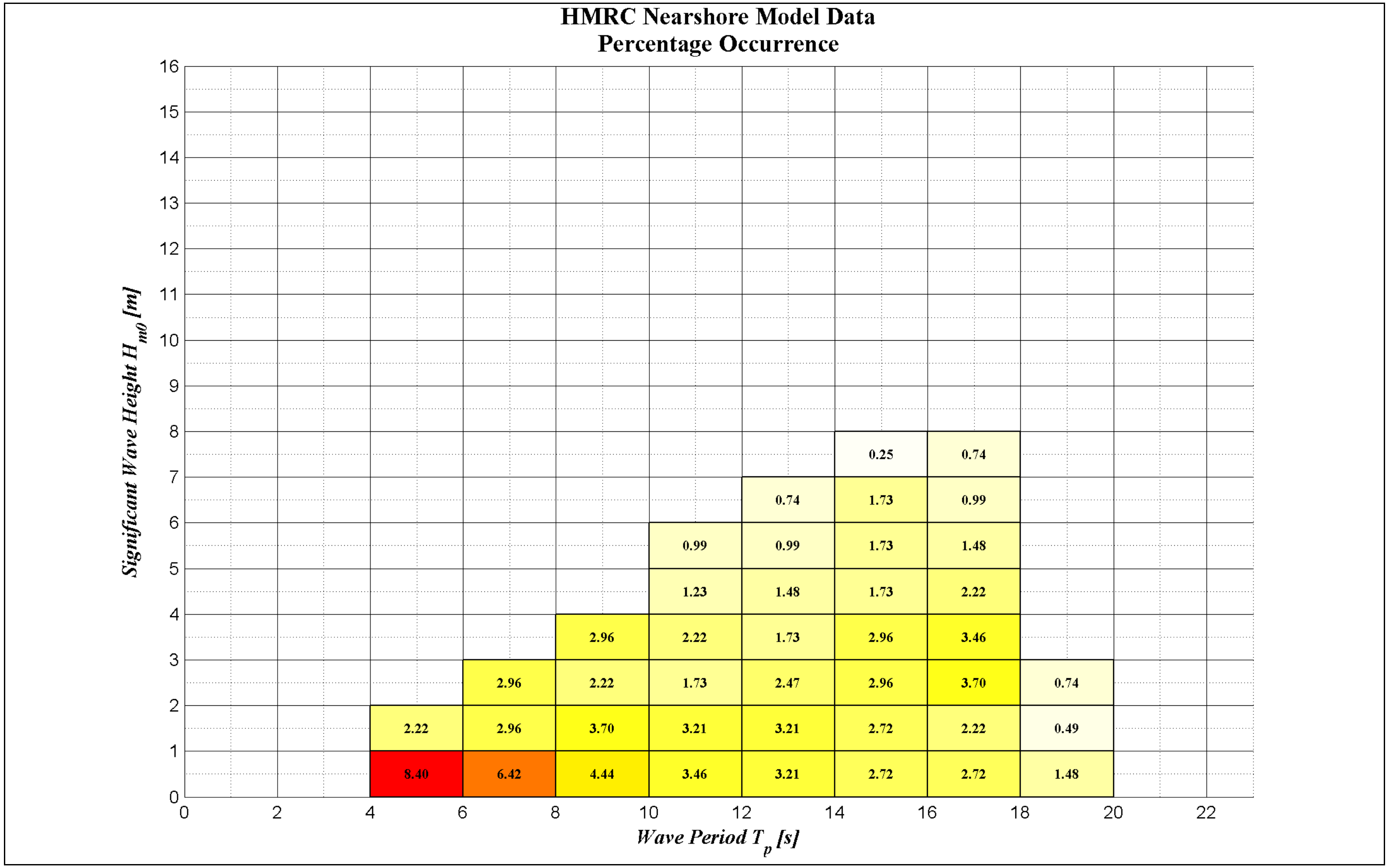


Figure 3.15 PT2 H_s/T_p Scatter Diagram

3.6 Extreme Wave Analysis

The input data for this analysis consisted of the 16 years of simulated output from the Numerics Warehouse study of Belmullet at coordinates latitude: -10 29.91 and longitude: 54 20.9). The time period of the data stretched from January1995 to December 2010 and values wave heights periods and directions were provided at 0.5 hour intervals. The methodology required the identification of storms or large wave events within the wave record. The analysis was carried out on the wave heights and as directionality is important in this analysis the wave record was separated into angular segments of 22.5degrees ranging from S to N.

On each angular segment a partial duration series analysis was carried out. This method is also called the peaks over threshold as only storms with wave heights greater than a specified threshold value are used in the analysis. One value corresponding to the maximum Hs is selected for each storm. The threshold is chosen such that average number of data values in total is approximately 20. Partial duration series are thus considered to be censored as they exclude storms below the threshold value. Once the main storm events have been identified from the data set they must then be fitted to a suitable probability distribution. Unfortunately there is no single generally accepted probability distribution for use in determining extreme wave statistics. In deeper water waves tend to be Rayleigh distributed (this is a form of the Weibull distribution) but as they propagate towards the coastline, shallow water effects can alter their distribution. Therefore other suitable distributions should be examined and these could include the Fisher –Tippett Type 1 (also known as the Gumbel function) and the Fisher-Tippett Type 2 (also known as Frechet function). Once the data fits a distribution then the equations defining this distribution can be used to calculate the required inverse cumulative probabilities. These include the 1 in 1 year, 1 in 10 year, 1 in 50 year and 1 in 100 year wave events.

As shown in Figure 3.16 which is the worksheet of the analysis tool the data was tested against a number of distributions and the final choice as to the most suitable depended on the correlation factor and the judgement of the analyst. The wave periods associated with the wave heights were determined by studying a scatter plot of wave conditions from the 16 years of data. Generally for extreme analysis predictions are not normally provided for return periods beyond 5 times the length of the data record but in this case a 1 in 100 year wave condition is provided in cases when the prediction seemed stable.

These extreme wave conditions, for each direction of approach, were then input on the model boundary and the consequent wave conditions near Belderra beach (at ING 65000 330456) were determined. Both the offshore and the resultant nearshore extreme wave conditions are shown in Table 3.2 below. A number of points can be made regarding this output

- There is a lot of dissipation of the incident wave energy as it approaches the beach area with offshore heights of up to 20m being reduced to about 5m. The maximum wave height at the data extraction point was 5.1m but lesser offshore input wave conditions give similar values which indicate the this size wave is a limiting value for this water depth.
- The inshore area is most significantly affected by waves approaching from directions ranging from SW to NNW
- Waves approaching the beach are essentially uniform in direction, WNW, regardless of the offshore incident direction. This is consistent with the wave climate analysis as was described in the previous section.
- The peak wave periods are generally reduced from their offshore equivalents and this can be attributed to the change in the energy profile of the waves as shallow water wave processes such as breaking, refraction, wave-to-wave interactions occur (as discussed in previous section).

Figures 3.17 to 3.21 show examples for the extreme wave simulations how waves approach the beaches of Annagh Bay from various offshore directions of approach.

Nt =	21		Weibull	Weibull	Weibull	Weibull	FT-II	FT-II	FT-II	FT-II
m	Hs	FT-1	0.75	1	1.4	2	2.5	3.33	5	10
1	14.987	0.973	0.977	0.975	0.973	0.971	0.968	0.965	0.963	0.960
2	14.582	0.926	0.931	0.929	0.927	0.925	0.920	0.918	0.915	0.913
3	14.542	0.879	0.884	0.882	0.880	0.878	0.873	0.870	0.868	0.866
4	13.474	0.831	0.837	0.835	0.833	0.831	0.825	0.823	0.821	0.818
5	13.343	0.784	0.791	0.789	0.786	0.784	0.778	0.776	0.773	0.771
6	13.252	0.737	0.744	0.742	0.740	0.737	0.730	0.728	0.726	0.723
7	12.487	0.689	0.698	0.695	0.693	0.691	0.683	0.681	0.678	0.676
8	12.482	0.642	0.651	0.649	0.646	0.644	0.636	0.633	0.631	0.629
9	11.778	0.595	0.605	0.602	0.599	0.597	0.588	0.586	0.584	0.581
10	11.711	0.547	0.558	0.555	0.553	0.550	0.541	0.538	0.536	0.534
11	11.567	0.500	0.511	0.509	0.506	0.503	0.493	0.491	0.489	0.487
12	11.399	0.453	0.465	0.462	0.459	0.457	0.446	0.444	0.441	0.439
13	10.765	0.405	0.418	0.415	0.412	0.410	0.398	0.396	0.394	0.392
14	10.739	0.358	0.372	0.369	0.366	0.363	0.351	0.349	0.347	0.344
15	10.436	0.311	0.325	0.322	0.319	0.316	0.303	0.301	0.299	0.297
16	10.345	0.263	0.278	0.275	0.272	0.269	0.256	0.254	0.252	0.250
17	9.826	0.216	0.232	0.229	0.225	0.223	0.208	0.206	0.204	0.202
18	9.3386	0.169	0.185	0.182	0.179	0.176	0.161	0.159	0.157	0.155
19	9.2689	0.121	0.139	0.135	0.132	0.129	0.114	0.112	0.110	0.108
20	8.2543	0.074	0.092	0.089	0.085	0.082	0.066	0.064	0.062	0.060
21	7.8195	0.027	0.046	0.042	0.038	0.035	0.019	0.017	0.015	0.013

	ym	Weibull	Weibull	Weibull	Weibull	FT-II	FT-II	FT-II	FT-II
	FT-1	0.75	1	1.4	2	2.5	3.3	5	10
	3.617	5.8954	3.6997	2.5076	1.8861	3.9188	2.7518	1.9236	1.378
	2.567	3.702	2.6395	1.9848	1.6081	2.704	2.1047	1.6243	1.2707
	2.046	2.7833	2.1366	1.7095	1.4501	2.2205	1.819	1.4783	1.2135
	1.690	2.2173	1.8035	1.5157	1.3335	1.935	1.6414	1.3828	1.1742
	1.414	1.817	1.5541	1.3633	1.2384	1.7377	1.5144	1.312	1.1441
	1.186	1.5125	1.3546	1.2361	1.1564	1.5891	1.416	1.2555	1.1194
	0.989	1.2702	1.1884	1.1257	1.0831	1.4706	1.3357	1.2084	1.0983
	0.814	1.0714	1.0459	1.0275	1.016	1.3723	1.2679	1.1678	1.0798
	0.654	0.9049	0.9212	0.9383	0.9533	1.2882	1.2089	1.1317	1.0631
	0.506	0.763	0.8103	0.8559	0.8938	1.2146	1.1564	1.0992	1.0477
	0.367	0.6408	0.7106	0.779	0.8366	1.1488	1.1087	1.0691	1.0333
	0.232	0.5345	0.6198	0.7063	0.7809	1.089	1.0648	1.041	1.0197
	0.102	0.4415	0.5367	0.6368	0.7261	1.0336	1.0236	1.0143	1.0065
	-0.027	0.3599	0.4599	0.5699	0.6715	0.9816	0.9843	0.9885	0.9936
	-0.156	0.288	0.3886	0.5048	0.6164	0.9319	0.9463	0.9631	0.9808
	-0.289	0.2247	0.322	0.4408	0.5602	0.8835	0.9088	0.9377	0.9678
	-0.427	0.1692	0.2596	0.3772	0.5017	0.8354	0.8709	0.9117	0.9542
	-0.577	0.1208	0.2009	0.3132	0.4396	0.7859	0.8315	0.8841	0.9396
	-0.747	0.0792	0.1454	0.2474	0.3715	0.7328	0.7882	0.8533	0.923
	-0.958	0.0443	0.0928	0.1777	0.2927	0.6705	0.7363	0.8152	0.9018
	-1.289	0.0168	0.0429	0.0989	0.1895	0.5755	0.6526	0.75	0.8632
intercept	10.641	10.145	9.573	8.729	7.572	8.422	6.979	3.975	-4.709
slope	1.616	1.181	1.976	3.090	4.482	2.250	3.667	6.674	15.392
R^2	0.935	0.713	0.826	0.915	0.966	0.759	0.825	0.883	0.928
1	12.1	11.484	11.744	12.035	12.27	10.869	10.787	10.578	9.8325
10	16.11	16.184	16.293	16.138	15.838	24.579	28.626	36.267	57.283
50	18.732	20.269	19.473	18.5	17.605	44.488	50.066	61.446	95.33
100	19.855	22.178	20.842	19.448	18.276	57.846	63	74.989	113.6

Figure 3.16 Sample Worksheet for Extreme Wave Analysis

Return Period (yrs)	Offshore				Belderra Beach		
	Hs (m)	Tp (sec)	Dir (deg.)		Hs (m)	Tp (sec)	Dir (deg.)
1 in 1	6.0	12.4	180.0		0.7	10.8	281.3
1 in 10	9.0	15.2	180.0		1.6	13.4	287.6
1 in 50	10.9	16.7	180.0		2.2	14.8	289.3
1 in 100	11.7	17.3	180.0		2.5	15.3	289.7
1 in 1	7.0	13.4	202.5		1.5	12.0	286.9
1 in 10	10.6	16.5	202.5		2.9	14.8	290.1
1 in 50	13.0	18.2	202.5		3.7	16.2	290.7
1 in 100	14.0	18.9	202.5		3.9	16.7	290.8
1 in 1	8.4	14.7	225.0		2.9	13.4	290.0
1 in 10	13.1	18.3	225.0		4.3	16.3	290.8
1 in 50	15.7	20.1	225.0		4.7	17.6	290.5
1 in 100	16.8	20.8	225.0		4.8	18.1	290.4
1 in 1	11.6	17.2	247.5		4.5	15.5	290.8
1 in 10	14.9	19.6	247.5		4.8	17.3	290.8
1 in 50	16.6	20.6	247.5		4.9	18.0	290.8
1 in 100	17.2	21.0	247.5		5.0	18.3	290.8
1 in 1	12.1	17.6	270.0		4.7	15.8	291.3
1 in 10	16.1	20.3	270.0		5.0	17.9	291.0
1 in 50	18.7	21.9	270.0		5.1	19.0	291.1
1 in 100	19.9	22.6	270.0		5.1	19.5	291.1
1 in 1	11.2	16.9	292.5		4.7	15.3	292.0
1 in 10	14.5	19.3	292.5		4.9	17.1	291.7
1 in 50	16.6	20.6	292.5		5.0	18.1	291.7
1 in 100	17.5	21.2	292.5		5.1	18.5	291.7
1 in 1	9.7	15.8	315.0		4.4	14.4	292.9
1 in 10	12.9	18.2	315.0		4.8	16.3	292.5
1 in 50	15.0	19.6	315.0		4.9	17.4	292.4
1 in 100	15.9	20.2	315.0		5.0	17.8	292.3
1 in 1	7.6	13.9	337.5		3.4	12.9	294.5
1 in 10	9.7	15.7	337.5		4.1	14.4	294.0
1 in 50	11.0	16.8	337.5		4.4	15.3	293.8
1 in 100	11.6	17.2	337.5		4.5	15.6	293.7
1 in 1	4.8	11.1	360.0		1.4	10.5	297.8
1 in 10	6.3	12.7	360.0		2.0	11.9	296.9
1 in 50	7.2	13.6	360.0		2.4	12.7	296.5
1 in 100	7.6	14.0	360.0		2.5	13.0	296.3

Table 3.2 Extreme offshore and inshore wave conditions

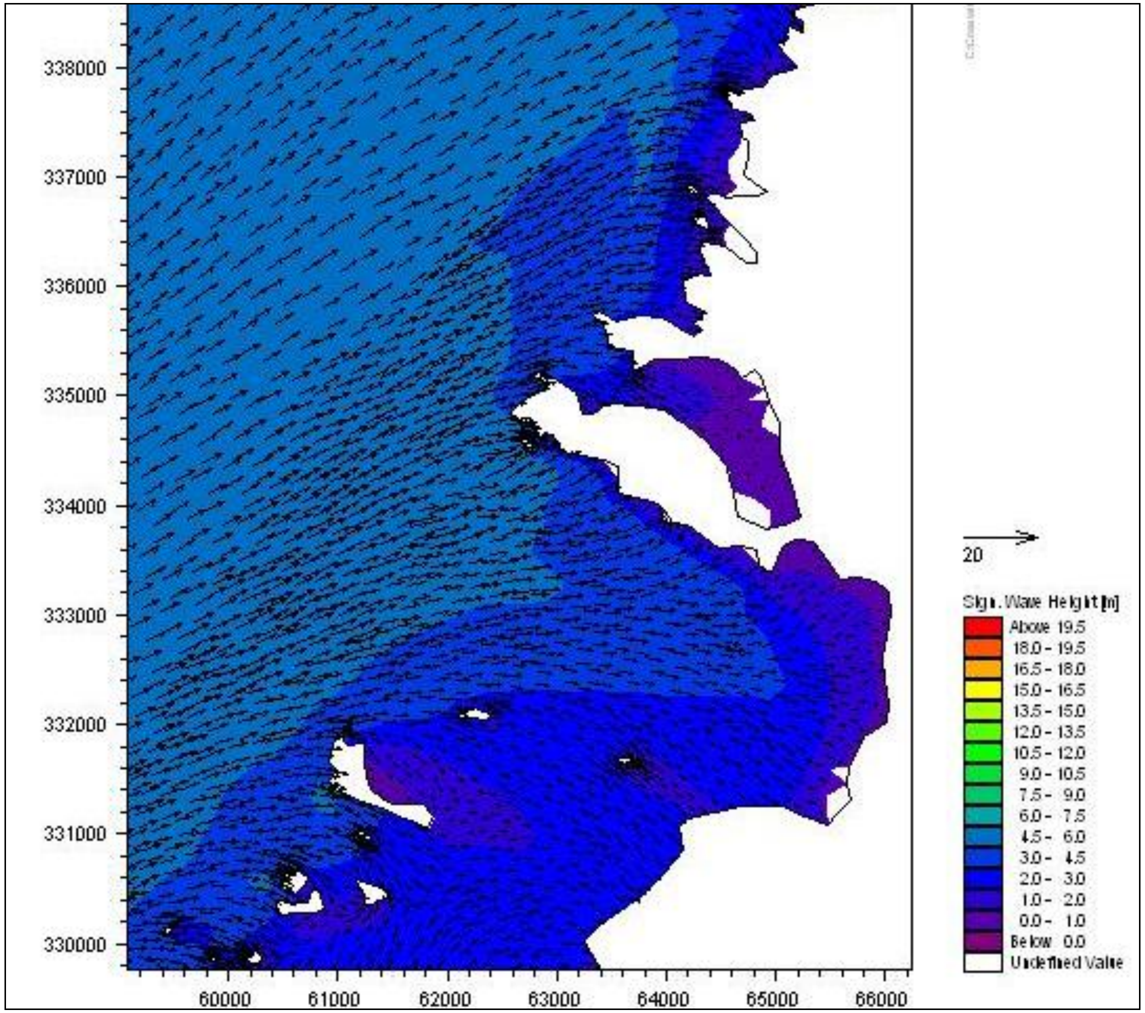


Figure 3.17 S Wave Approach

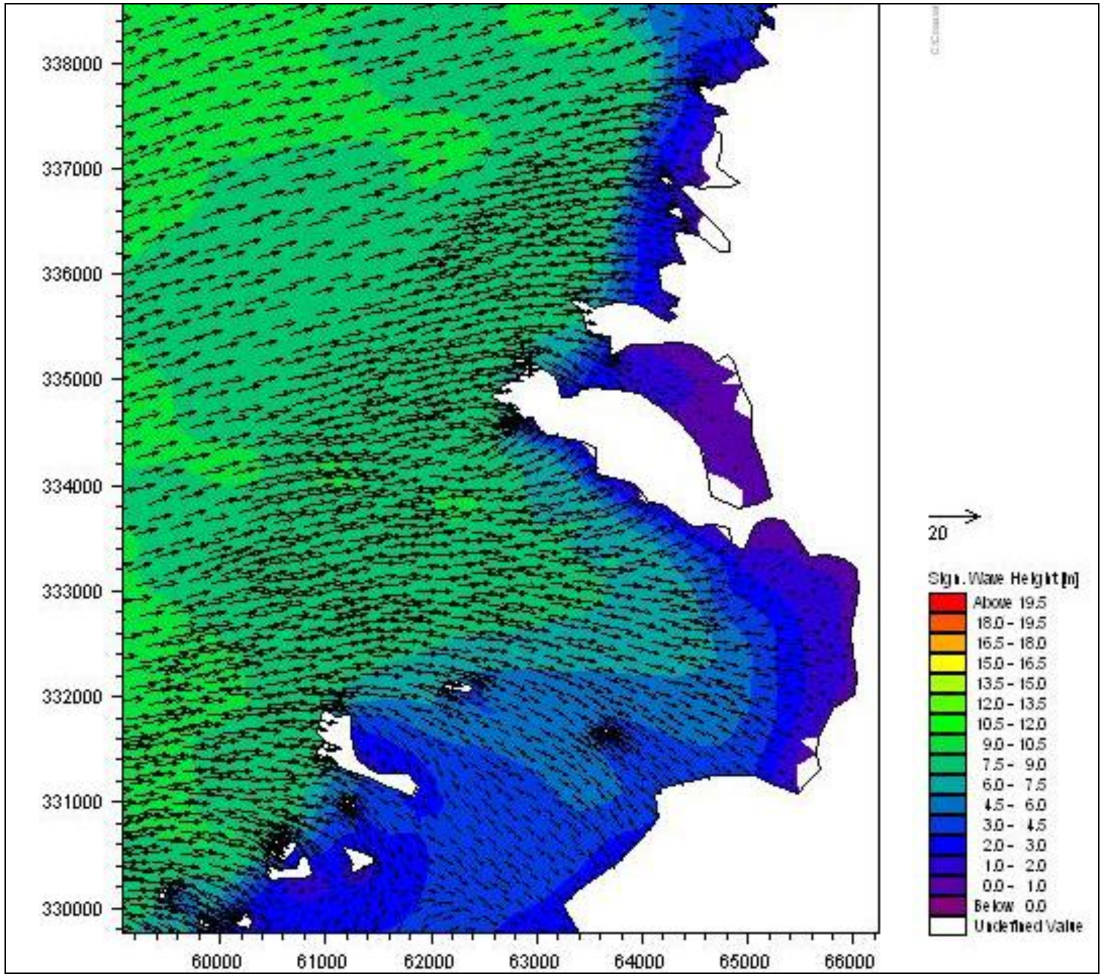


Figure 3.18 SW Wave Approach

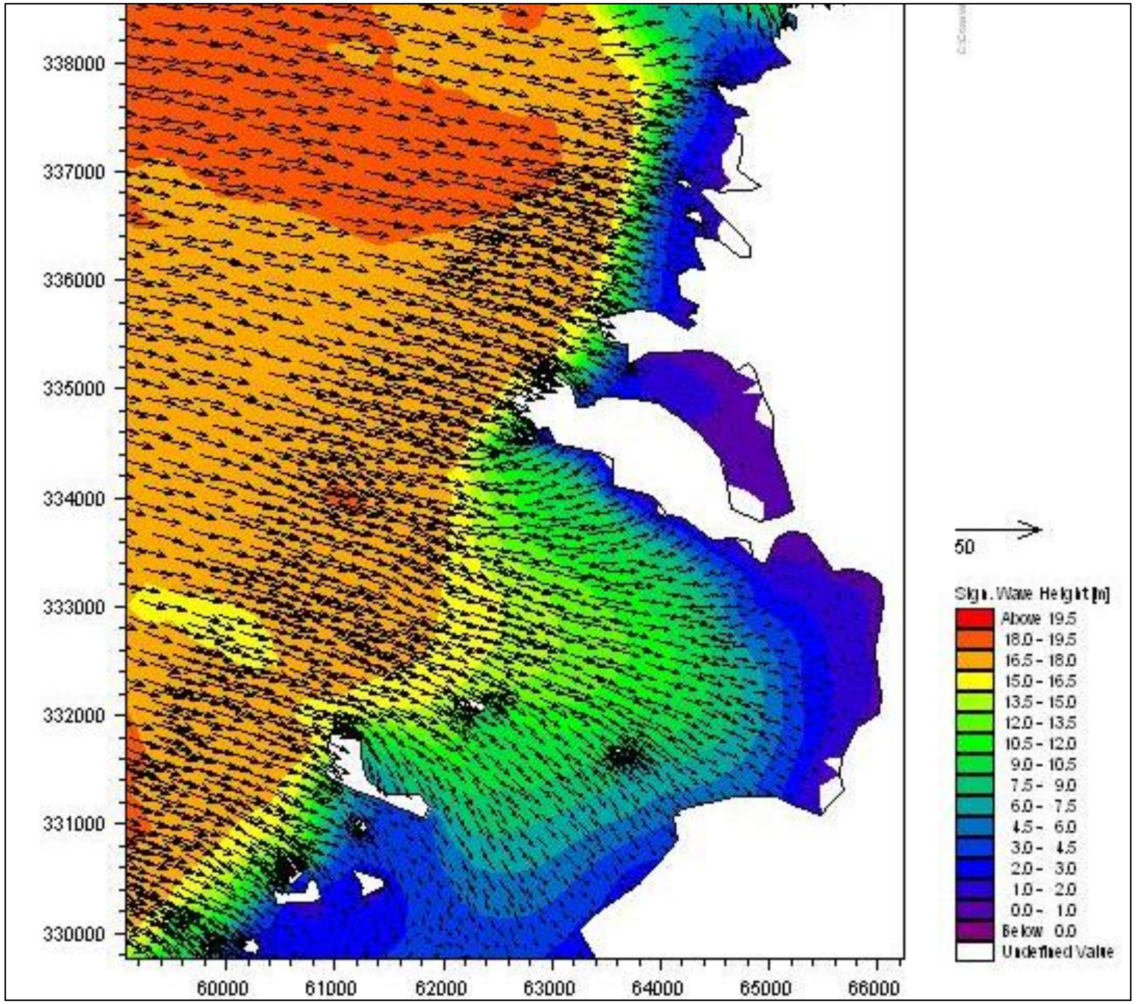


Figure 3.19 WNW Wave Approach

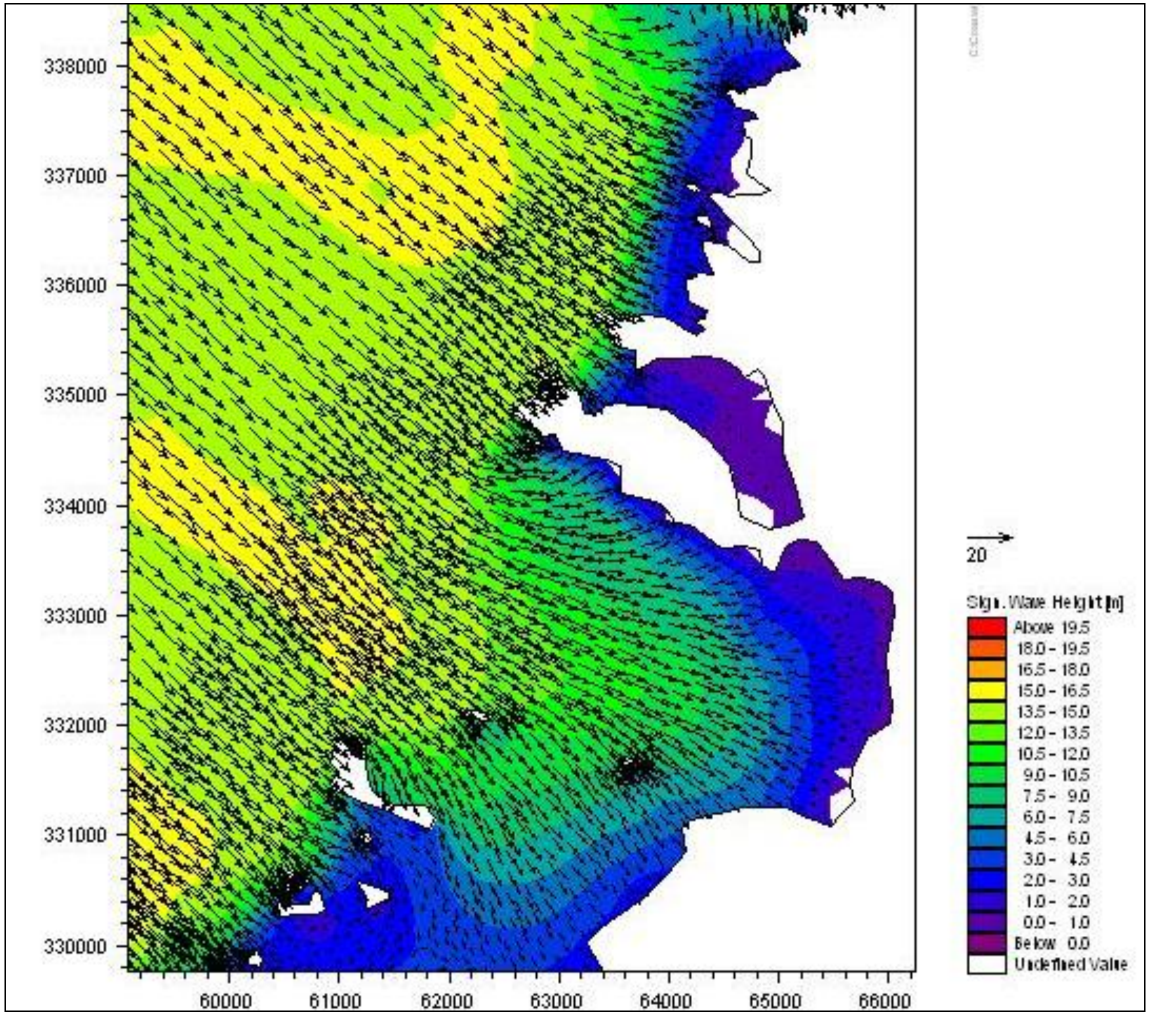


Figure 3.20 NW Wave Approach

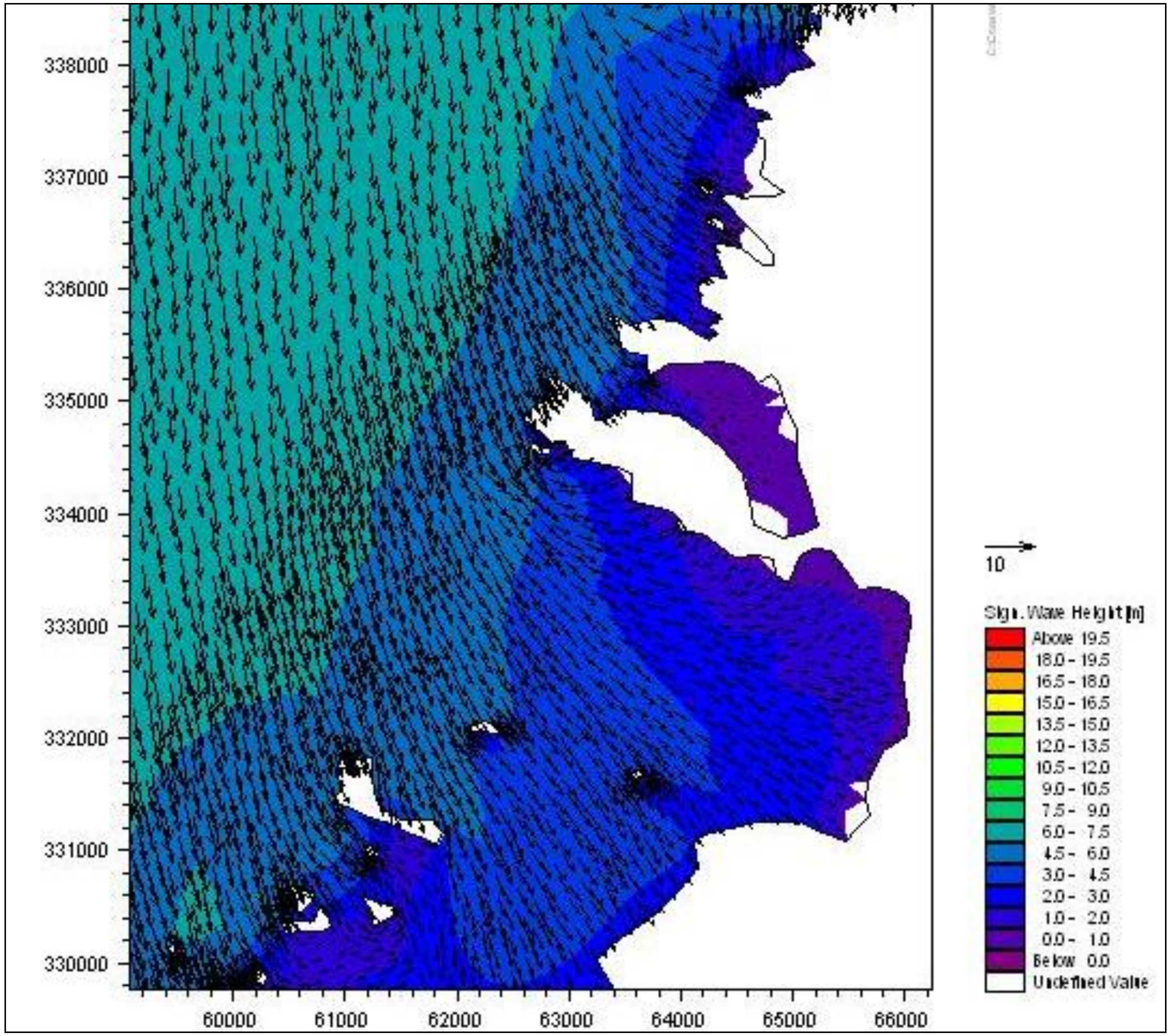


Figure 3.21 N Wave Approach

3.7 Impact of WECs on Nearshore Wave Conditions

The final use of the numerical model involved examining the potential impact of WECs on the nature of nearshore wave conditions. As an introduction WECs normally operate for a certain range of wave heights and periods ideally centred around the mean wave conditions at the site. They extract a portion of the incident energy, normally less than 50%, across the width of the device which is then converted to electricity by means of a power take off system. WECs generally do not extract power from low and high wave conditions and are usually less efficient at frequencies outside the natural resonant frequency of the device. Therefore the proposed WECs for the Belmullet sites will not be extracting energy for a certain portion of the time and so at such times will have negligible impact on nearshore wave conditions and thus coastal processes. As the operational characteristics of each device vary no precise figures can be given on the range of conditions they will not be operational. For the remainder of the time various amounts of energy will be extracted and the methodology proposed seeks to examine their potential effects.

As the MIKE21 SW model, and software in general, has not been designed to represent the mode of operation of the WECs in terms of manner of energy extraction, an alternative approach was adopted to examine their effects on wave propagation. Each WEC was represented as an island with a size 25% larger than the specified device. Therefore instead of a portion of the energy being extracted from the incident waves the WEC as represented in this study provided a barrier to the propagation of the wave. The reason for increasing the size was to ensure that the final results would be conservative. Other studies of impacts of WECs using similar models have used this technique to simulate the energy absorption along with others such as white capping, bottom friction, depth induced wave breaking, wave–wave interaction and diffraction. However, no single method is totally accurate and the best that can be achieved is to be conservativeis Those phenomena are represented by numerical coefficients that need to be tuned to the cases being modelled.

A number of scenarios of device placement were examined and these are summarised in Table 3.3 in terms to the type and number of WECs in Test A and Test B. It should be stated the setup 7 in Table 3.3 represents the projected maximum usage of the test areas. It is unlikely that such a usage levels will be achieved on the site. Test area B has a lower designated density of WECs due to its proximity to the coastline and the potential of higher impacts. Figure 3.22 shows an example of a setup in Test A and Test B.

Simulations were run for a range of wave heights, periods and directions firstly for the baseline case with no WECs and then for each particular setup. These simulations allowed an assessment of the potential impacts to be made. Output was extracted at two data extraction points in the model which have the following ING co-ordinates ((62000 333500) and (64500 332000) which are termed offshore and nearshore points and are shown in Figure 3.13). An example on the localised effect of the WEc on the wave field can be seen in Figure 3.23.

From the simulation output the change in wave height from the baseline (no WECs) condition was determined for each direction of wave approach and the results are displayed in Figures 3.24 and 3.25. These plots show the highest percentage change in wave height for all the wave heights and periods that were simulated from each particular direction.

The following points can be made about the output

- No results are presented from the first two setup situations (Test A) as they showed no discernible change to the wave conditions at the output points. Therefore separate arrays of Wavebob and Pelamis devices at this location will not impact on inshore coastal processes.
- Small changes in incident wave heights may occur as a consequence of the deployment of other arrangements of WECs at Belmullet. These changes are not deemed to be significant on terms of altering the nature of the inshore wave conditions as a most conservative worst case scenario the maximum order of change is only 2.7%. Test area A due to its offshore location has a lesser impact and it requires a significant deployment of WECs to give a wave height change of 0.8%. The magnitude of the wave height change is dependent on wave direction with W to NW waves being most important. Southerly waves obviously do not have any impact on the relevant coastline.
- In the simulations for Test B site, deployments of 3, 2 and 1 WEC were run, however it is planned not to have more than two devices in place at any one time Modelled data output for the 28m water depth location (Figure 3.25 (b)) indicated a maximum wave height change of 1.5% occurs. This is a relatively insignificant change and is likely to reduce further as the waves propagate onwards towards the shore and continue to lose energy.
- For all simulation setups the wave periods and wave directions did not change from the baseline conditions.

Therefore given the conservative nature of the modelling process it can be stated that nearshore waves will essentially be unaffected by the presence of WECs. Studies by Halcrow and Millar et al for the Wave Hub site in Cornwall have indicated significantly higher potential changes then shown in this study. A discussion on acceptable levels of changes is given in Section 6.

Another aspect regarding the work relates to the protection of the cable with rock armouring which will take place from beyond Test area B (50m water depth) in various linear lengths for 4 km creating in total 4 artificial reefs. These reefs will only have an elevation of 1m above the seabed and will be 2m wide. Their very small footprint and low elevation at such a relatively large water depth the overall impact is expected to be negligible. As validation for this reference is made to research carried out by Armono and Hall (2003). They carried out studies on submerged breakwaters/artificial reefs and found that various dimensionless parameters influenced the wave transmission. In relation to the structure geometry it was the ratio of the height of the structure to the water depth (h/D) that was considered most important. Model tests showed that for a structure with a h/D ratio of 0.7 the wave transmission varied between 0.8-1 depending in the incident wave parameters. These results, and there are many other similar publications, indicate that artificial reefs are not effective as coastal protection structures unless the crest level is almost at the water surface. In relation to the current study the h/D ratio will be equal to 0.02 or less so their impact on wave transmission will be negligible.

Setup Number	Test A	Test B
1	Array of five Wavebobs	No Device
2	Array of 5 Pelamis machines	No Device
3	Array of five Wavebobs and five Pelamis machines	No Device
4	No Device	3 OE Buoys
5	No Device	2 OE Buoys
6	No Device	1 OE Buoys
7	Array of five Wavebobs and five Pelamis machines	2 OE Buoys

Table 3.3 WEC model setup configurations

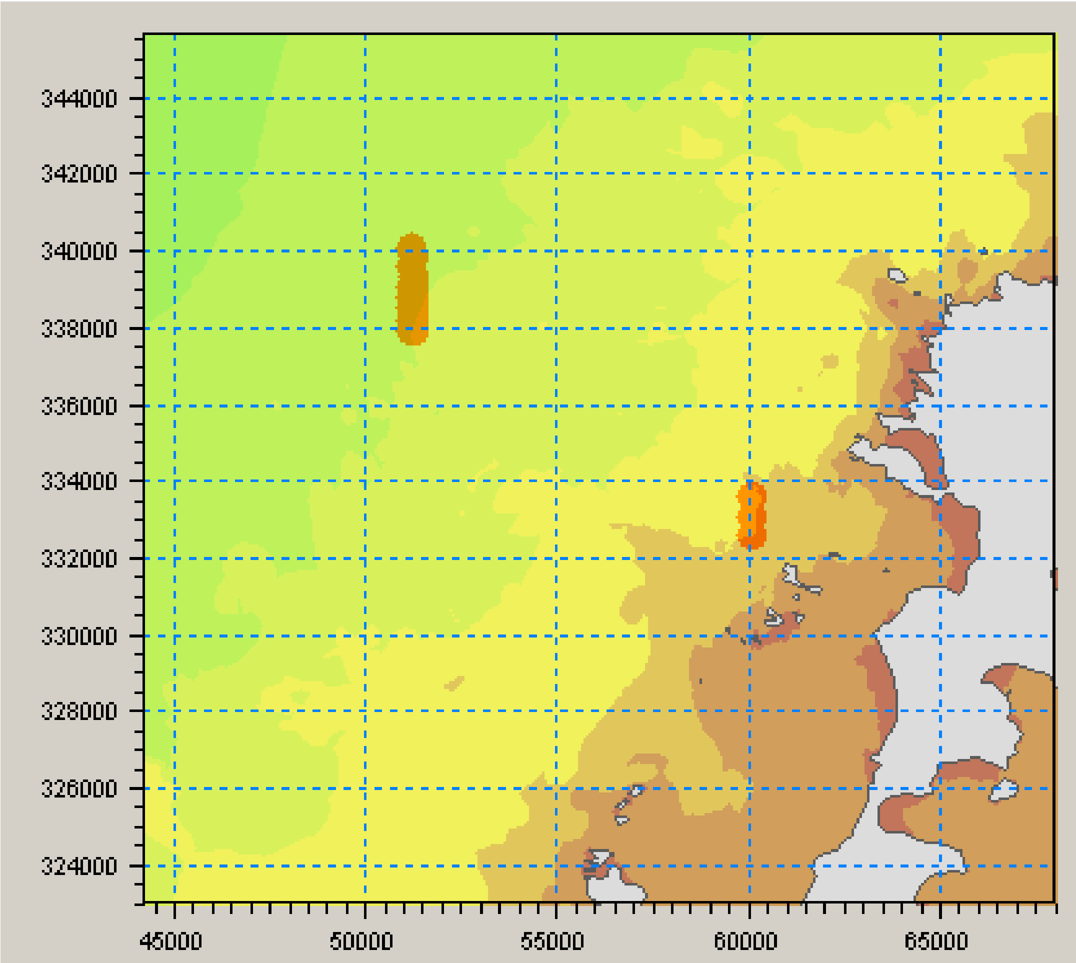


Figure 3.22 WEC locations

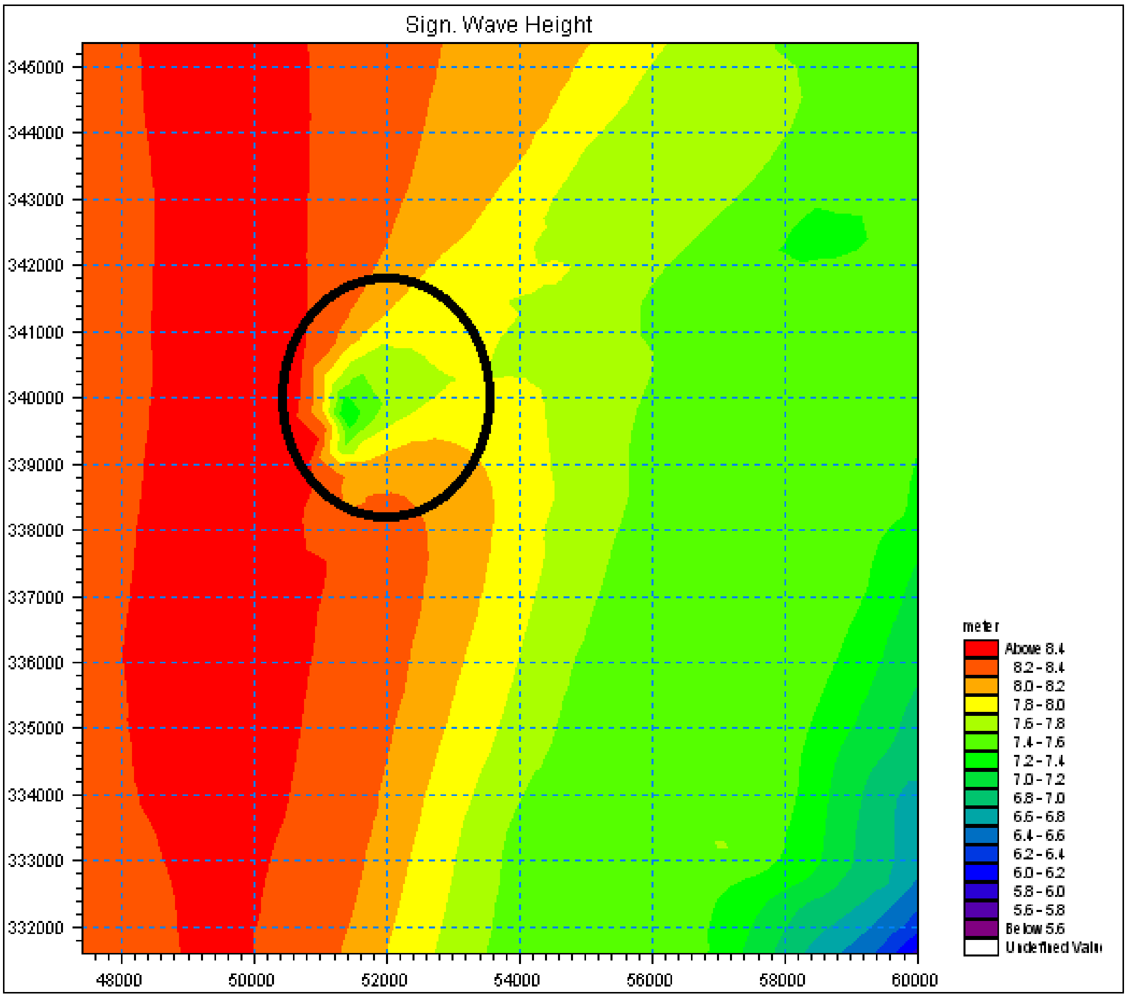


Figure 3.23 Impact of WECs on wave field at Test A

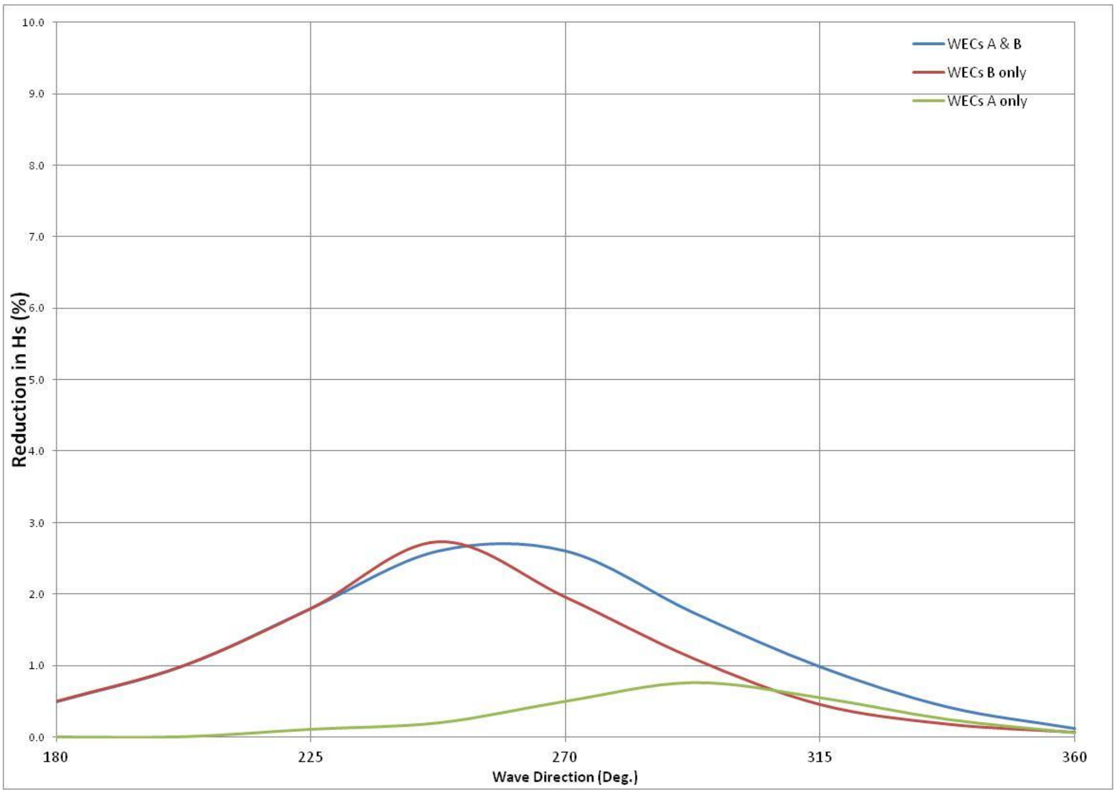


Figure 3.24 (a) PT1 (Figure 3.13) Setups 3,4 &7

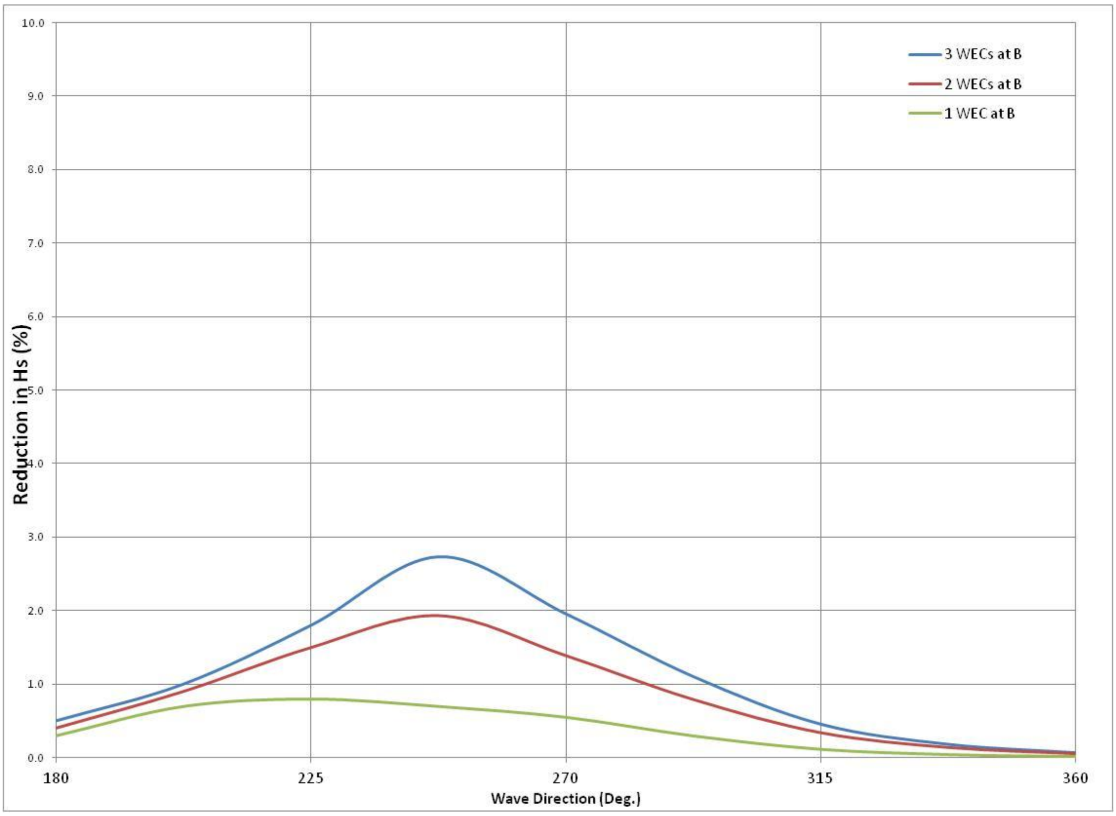


Figure 3.24 (b) PT1 (Figure 3.13) Setups 4,5 &6

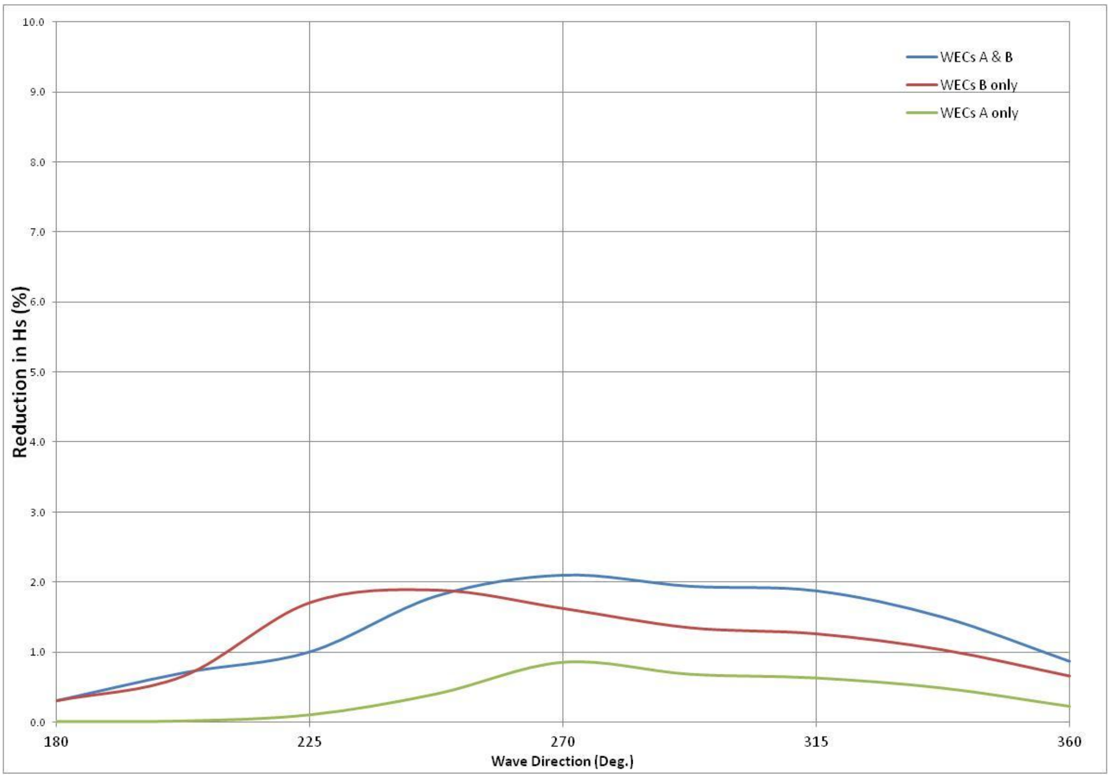


Figure 3.25 (a) PT2 (Figure 3.13) Setups 3,4 &7

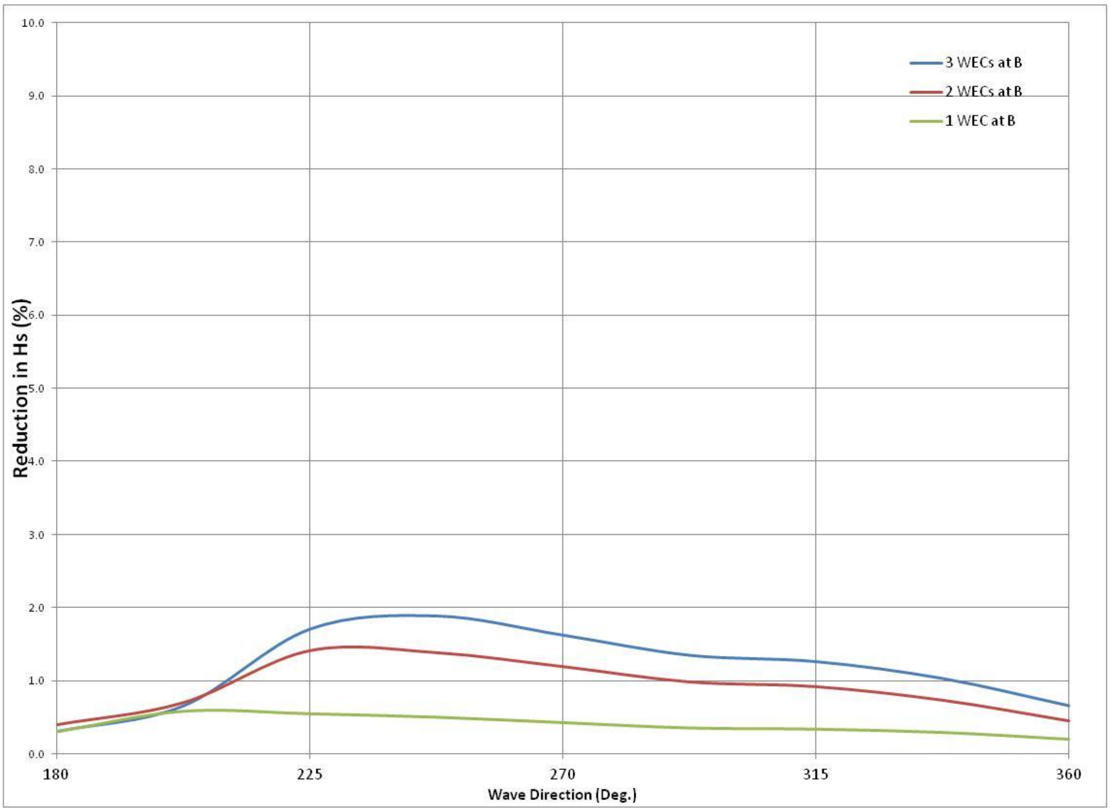


Figure 3.25 (b) PT2 (Figure 3.13) Setups 4,5 &6

4. Current Analysis

The current data as collected by the ADCPs (shown located in Figure 3.1) are shown in Figure 4.1 and 4.2. This data is for a level 5m above the seabed and was used for the following purposes

- Examine the nature of tidal flows
- Derive tidal constituents at each location
- Predict the flow velocities
- Provide input for the calculation of the thresholds for sediment movement along the cable route.

The plots show that the offshore currents are larger than the more onshore currents. This is because there are strong tidal streams off the Belmullet coastline as is also shown by the Marine Institute model output plots (Figures 4.3 and 4.4). These currents reduce in areas away from the main tidal stream for example as they enter the indented coastline past Annagh Head. It is expected that in the vicinity of the various beaches that the astronomically induced current speeds will be low and not important to the overall behaviour. For the ADCP 'Deep' the currents flow in NE and SW directions whilst at the ADCP 'Shallow' location it is difficult to determine coherent flow directions mainly due to the low flow velocities.

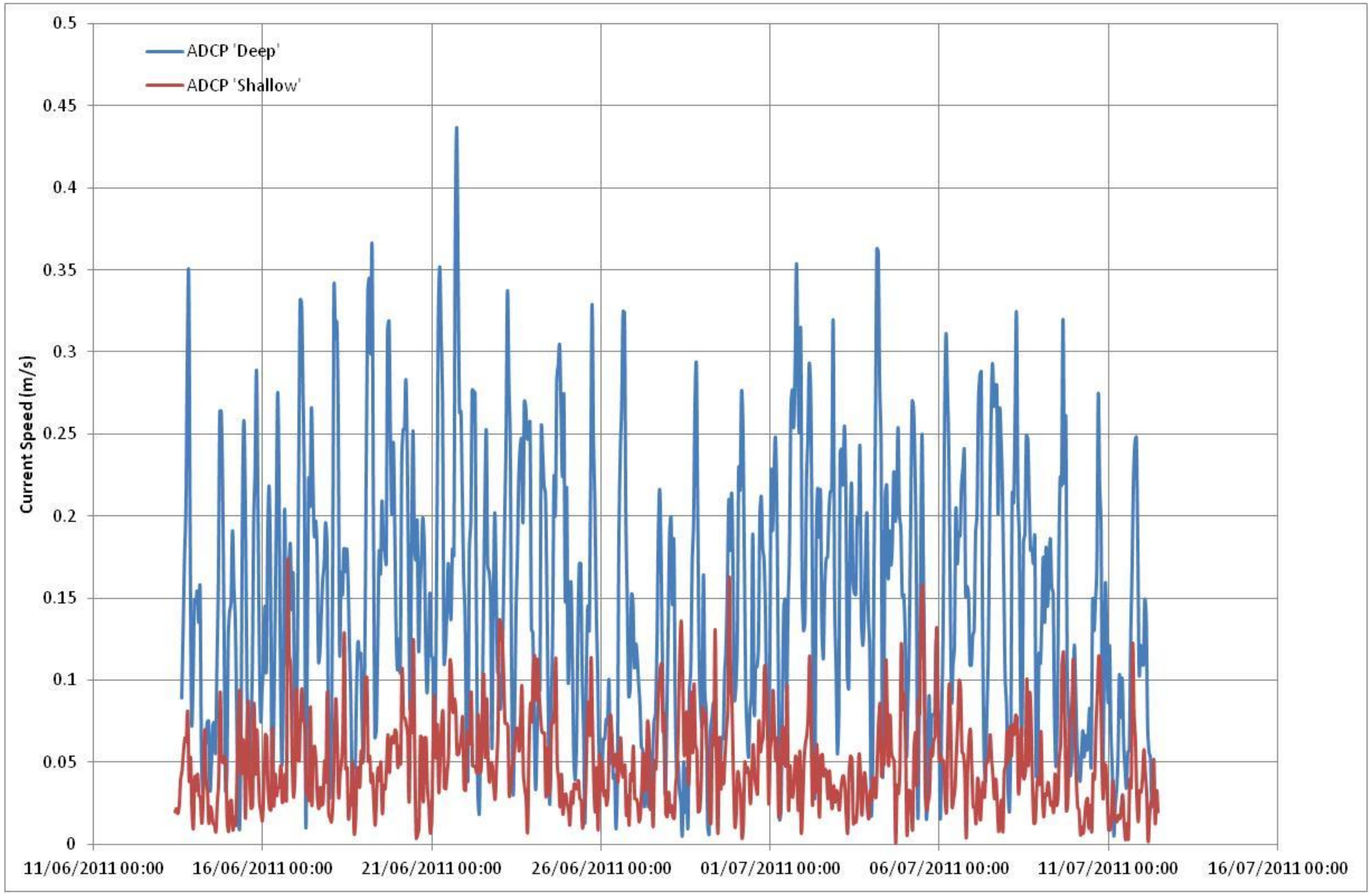


Figure 4.1 ADCP Current Speeds

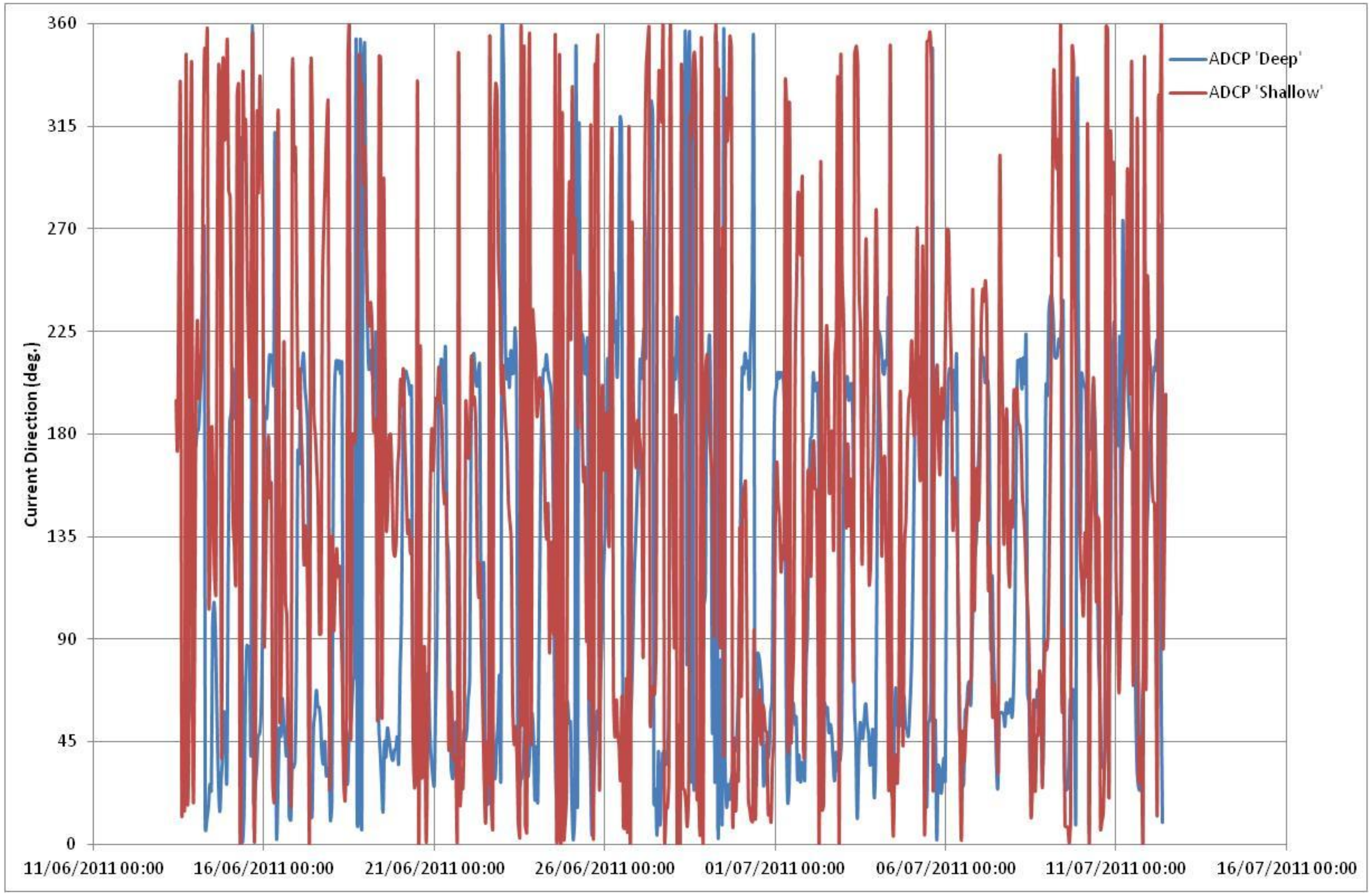


Figure 4.2 ADCP Current Directions

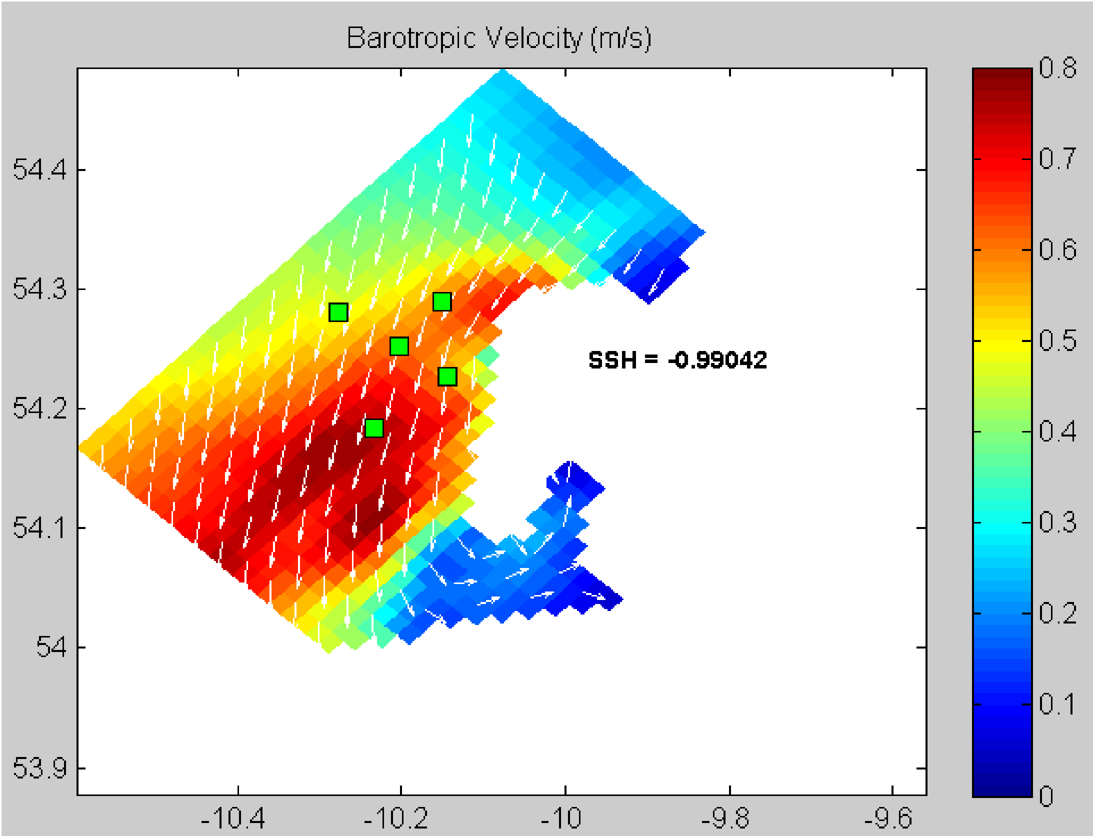


Figure 4.3 Tidal Flows from model output

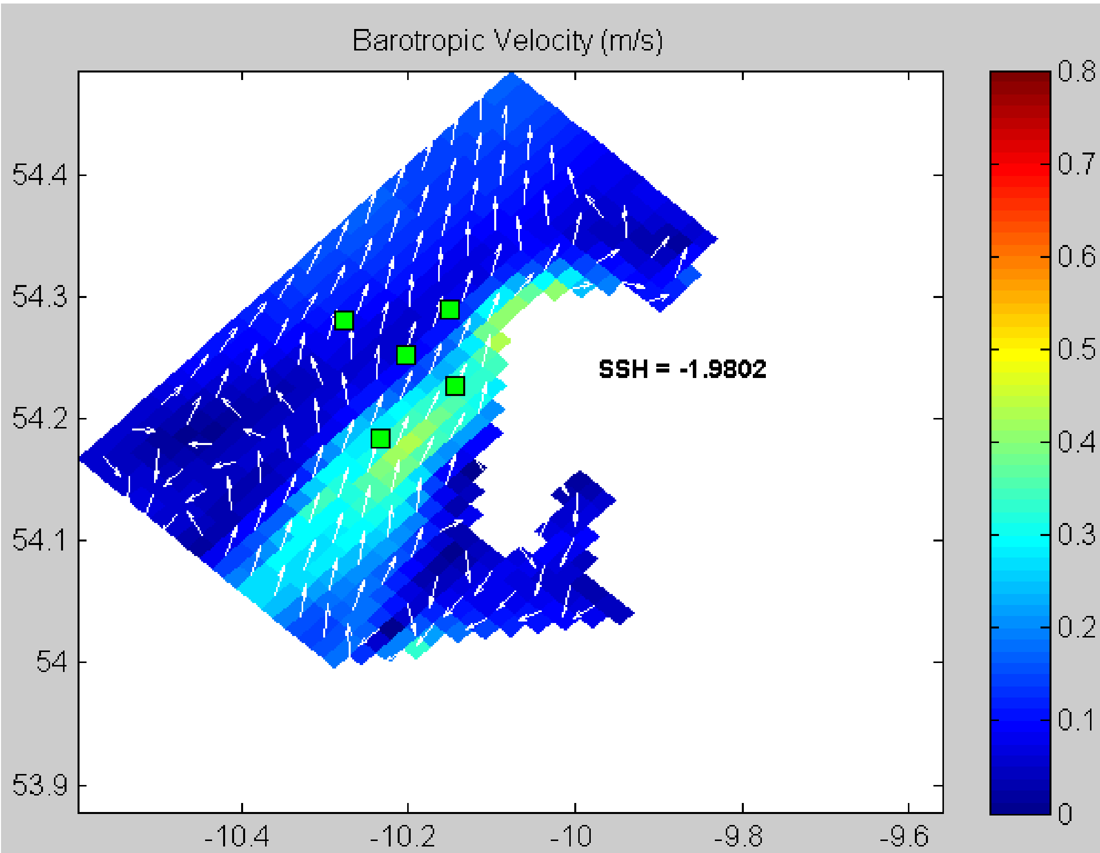


Figure 4.4 Tidal Flows from model output

5. Sediment Transport and Coastal Change

This section considers potential physical changes that could result from the proposed works.

5.1 Mobilisation of seabed sediments by waves and currents

During the cable laying operations it is likely that some sediment will be mobilised so it is important to assess the nature of sediment transport along the cable route based on a knowledge of sediment gradation and environmental forcing from waves and currents. The approach followed considers the data obtained from various core samples provided by Terra Tek Ltd as well as measurements of waves and currents in order to determine the thresholds of motion for the sediment at various points along the cable route to shore. Water depths were estimated from the wave energy test area layout drawing which has core locations identified. Calculations were performed for the locations as indicated in Table 5.1.

It was decided that combined hydrodynamic, wave and sediment transport models would not be necessary based on a fundamental assumption, that given the magnitude of the waves at the site, the sea bed material has high natural mobility (this has been verified anecdotally) and the limited time and space scale of the cable laying operations would have minimal impact on bed movements. In addition the cable laying method will be chosen to give low levels of sediment mobilisation. Therefore the calculations carried out firstly show that the bed material is naturally mobile based on the forcings that result in an exceedance of the threshold of motion and secondly that any sediment that is mobilised by the cable laying operations will quickly settle back on the seabed.

The seabed shear stress was calculated using Fredse's Model of Wave and Current interaction in the boundary layer as outlined in a document prepared by Zhou Liu (2001) of Aalborg University[1] and the critical Shield's parameter was determined using the diagram and method outlined by Madsen et al (1976). The worksheet used for the calculation is shown in Figure 5.1. Generally the theory shows that waves and currents both combine to contribute to the sediment movement with the waves mobilising the sediment and it moves with the current until conditions reduce below the threshold levels and it settles back to the seabed.

The analysis of threshold of motion shows that waves are the dominant driving force for mobilising sediment. The wave period is a critical parameter as in deeper waters which for the Belmullet case is 100m, waves need to have periods of greater than 10s before they have an effect on the seabed. The higher the wave period the more the wave will be affected and energy lost through interaction with the bottom sediments. Table 5.2 and Figure 5.2 show that for each sample analysed the threshold

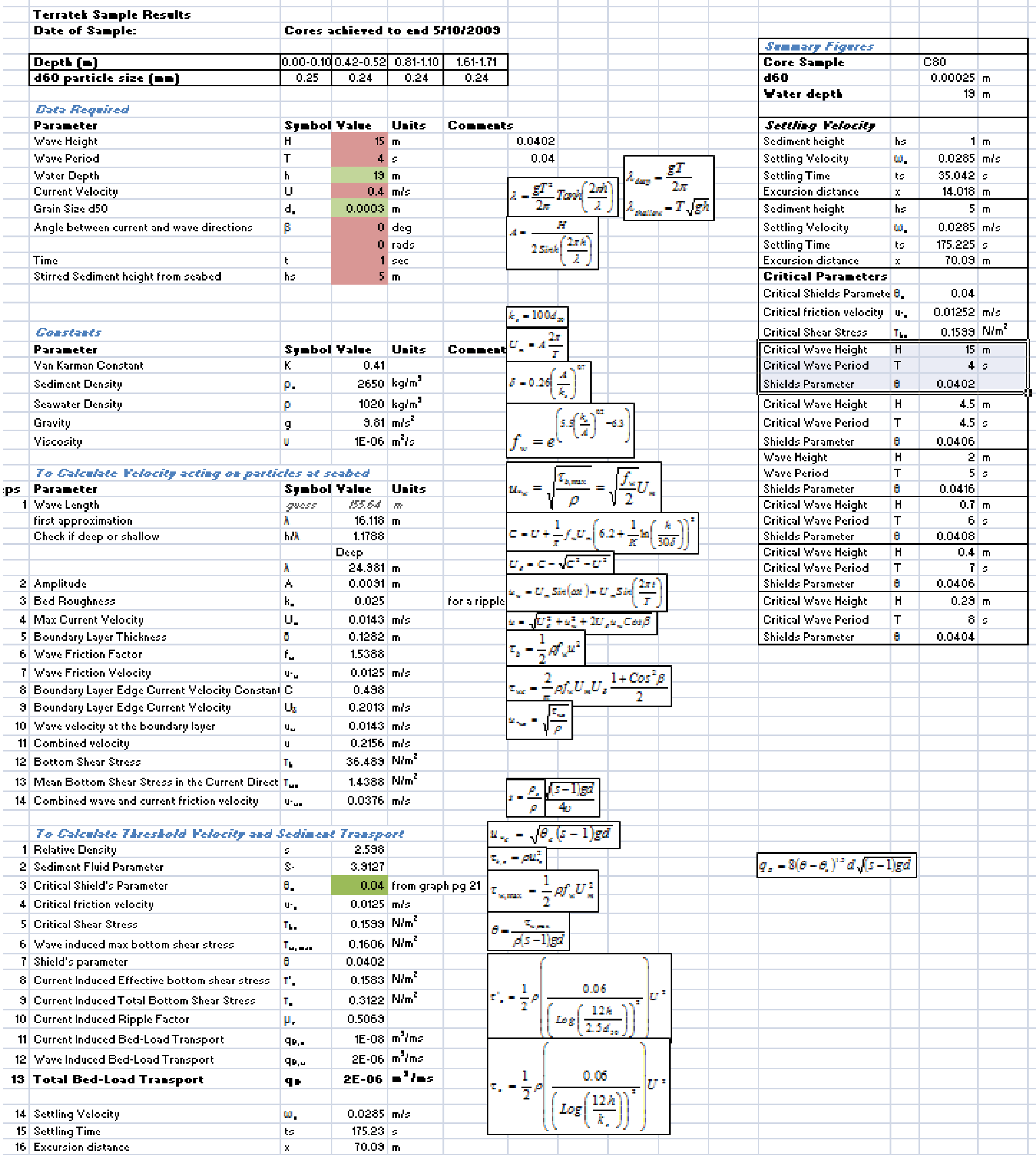
[1] Liu Z., (2001), "Sediment Transfer", 3rd Edition, *Laboratory for Hydraulic and Port Construction, Aalborg University*, January 2001

of motion is exceeded at a certain cut-in wave period which is dependent on the depth. Lower period waves require higher wave heights whilst much reduced heights are sufficient at longer periods. When the heights and periods are considered in conjunction with the scatter diagrams as shown in Figures 3.2 and 3.3 it can be seen that wave conditions that give rise to sediment movement on the seabed occur quite frequently in the Belmullet area. For example at the 100m depth sediment movement will result from wave heights from 0.5m-2m or greater if their wave period exceeds 12s and from Figure 3.3 such conditions occur quite frequently. Therefore in its natural state it is likely that there is a lot of movement of the sand sediment that will obviously be more significant during larger storm events in the test site areas. Determining the scale of this movement is beyond the scope of this study as it would require detailed field measurements and numerical modelling. However it may become important for operational purposes when the site becomes active for instance to examine that the cables do not become exposed or moorings undermined by scouring around anchor blocks. Scouring will depend on the nature of the anchors and the mooring system but given that there will be a large separation between devices the anchors should be sufficiently spaced such that only local scouring will occur. Sediment that will be mobilised due to scouring will settle in the manner as described below. Mitigation measures may be required again depending on the nature of the anchor system.

The second part of the analysis considers the excursion distance of sediment mobilised by cable laying operations such as ploughing or jetting. In this case it is assumed that when this work is taking place wave conditions will be benign and tidal currents will correspond to the maximum near seabed recorded during the ADCP deployments (0.4m/s). This is a conservative assumption as current speeds vary in magnitude over the tidal cycle. The formulation for determining the settlement time is based on calculating the fall velocity of the sand particles. The results as shown in Table 3.3 show that the current speed is not high enough to keep the sediment in suspension and that it quickly settles back to the seabed. Maximum excursion distances range from about 15m when the suspension height is 1m to about 80m for a 5m suspension height. As the quantities of sediment mobilised will be low and given the high natural movement of material it is expected that sediment movement as a result of cable laying operations will be insignificant to the overall morphology of the seabed.

Core Sample	Date of Sample	Estimated Water Depth (m)	Particle Diameter, D60: 60% Passing (mm)
C80	March 2010	19	0.25
C47	Achieved to end 5/10/09 and in March 2010	56	0.28
C73	March 2010	69	0.3
C41	March 2010	80	0.25
C27	March 2010	89	0.23
C10	Achieved to end 5/10/09 and in March 2010	98	0.23
C17	March 2010	100	0.24

Table 5.1 Core Samples used in calculations



Terratek Sample Results
Date of Sample: **Cores achieved to end 5/10/2009**

Depth (m)	0.00-0.10	0.42-0.52	0.81-1.10	1.61-1.71
d60 particle size (mm)	0.25	0.24	0.24	0.24

Data Required

Parameter	Symbol	Value	Units	Comments
Wave Height	H	15	m	0.0402
Wave Period	T	4	s	0.04
Water Depth	h	19	m	
Current Velocity	U	0.4	m/s	
Grain Size d50	d_s	0.0003	m	
Angle between current and wave directions	β	0	deg	
		0	rads	
Time	t	1	sec	
Stirred Sediment height from seabed	hs	5	m	

Constants

Parameter	Symbol	Value	Units	Comments
Van Karman Constant	K	0.41		
Sediment Density	ρ_s	2650	kg/m^3	
Seawater Density	ρ	1020	kg/m^3	
Gravity	g	9.81	m/s^2	
Viscosity	υ	1E-06	m^2/s	

To Calculate Velocity acting on particles at seabed

Steps	Parameter	Symbol	Value	Units	
1	Wave Length	guess	155.64	m	
	first approximation	λ	16.118	m	
	Check if deep or shallow	h/λ	1.1788		
			Deep		
		λ	24.981	m	
2	Amplitude	A	0.0091	m	
3	Bed Roughness	k_s	0.025		for a ripple
4	Max Current Velocity	U_m	0.0143	m/s	
5	Boundary Layer Thickness	δ	0.1282	m	
6	Wave Friction Factor	f_w	1.5388		
7	Wave Friction Velocity	u_{*w}	0.0125	m/s	
8	Boundary Layer Edge Current Velocity Constant	C	0.498		
9	Boundary Layer Edge Current Velocity	U_δ	0.2013	m/s	
10	Wave velocity at the boundary layer	u_w	0.0143	m/s	
11	Combined velocity	u	0.2156	m/s	
12	Bottom Shear Stress	τ_b	36.489	N/m^2	
13	Mean Bottom Shear Stress in the Current Direct	τ_{wc}	1.4388	N/m^2	
14	Combined wave and current friction velocity	u_{*wc}	0.0376	m/s	

To Calculate Threshold Velocity and Sediment Transport

Steps	Parameter	Symbol	Value	Units
1	Relative Density	s	2.598	
2	Sediment Fluid Parameter	S*	3.9127	
3	Critical Shield's Parameter	θ_c	0.04	from graph pg 21
4	Critical friction velocity	u_{*c}	0.0125	m/s
5	Critical Shear Stress	$\tau_{b,c}$	0.1599	N/m^2
6	Wave induced max bottom shear stress	$\tau_{w,max}$	0.1606	N/m^2
7	Shield's parameter	θ	0.0402	
8	Current Induced Effective bottom shear stress	τ'_c	0.1583	N/m^2
9	Current Induced Total Bottom Shear Stress	τ_c	0.3122	N/m^2
10	Current Induced Ripple Factor	μ_c	0.5069	
11	Current Induced Bed-Load Transport	$q_{b,c}$	1E-08	m^3/ms
12	Wave Induced Bed-Load Transport	$q_{b,w}$	2E-06	m^3/ms
13	**Total Bed-Load Transport**	q_b	**2E-06**	m^3/ms
14	Settling Velocity	ω_s	0.0285	m/s
15	Settling Time	ts	175.23	s
16	Excursion distance	x	70.09	m

Summary Figures

Core Sample		C80	
d60		0.00025	m
Water depth		19	m
Settling Velocity			
Sediment height	hs	1	m
Settling Velocity	ω_s	0.0285	m/s
Settling Time	ts	35.042	s
Excursion distance	x	14.018	m
Sediment height	hs	5	m
Settling Velocity	ω_s	0.0285	m/s
Settling Time	ts	175.225	s
Excursion distance	x	70.09	m
Critical Parameters			
Critical Shields Paramete	θ_c	0.04	
Critical friction velocity	u_{*c}	0.01252	m/s
Critical Shear Stress	$\tau_{b,c}$	0.1599	N/m^2
Critical Wave Height	H	15	m
Critical Wave Period	T	4	s
Shields Parameter	θ	0.0402	
Critical Wave Height	H	4.5	m
Critical Wave Period	T	4.5	s
Shields Parameter	θ	0.0406	
Wave Height	H	2	m
Wave Period	T	5	s
Shields Parameter	θ	0.0416	
Critical Wave Height	H	0.7	m
Critical Wave Period	T	6	s
Shields Parameter	θ	0.0408	
Critical Wave Height	H	0.4	m
Critical Wave Period	T	7	s
Shields Parameter	θ	0.0406	
Critical Wave Height	H	0.29	m
Critical Wave Period	T	8	s
Shields Parameter	θ	0.0404	

Figure 5.1 Sediment Movement calculation Worksheet

	Wave Height (m)						
T (sec)	C80	C47	C73	C41	C27	C10	C17
4							
4.5	4.5						
5	2						
6	0.7						
7	0.4	3.7	9.5				
7.5				12.7			
8	0.29	1.61	3.3	7.2	14		
9		0.95	1.67	3.1	5.35	8.3	9.7
10		0.67	1.05	1.8	2.79	4	4.53
11		0.53	0.77	1.2	1.76	2.4	2.65
12			0.62	0.9	1.27	1.65	1.81
13			0.53	0.74	1	1.24	1.36
14					0.83	1.1	1.1
15						0.85	0.93
16						0.76	0.82

Table 5.2 Wave climate parameters meeting critical sediment transport conditions for each core sample

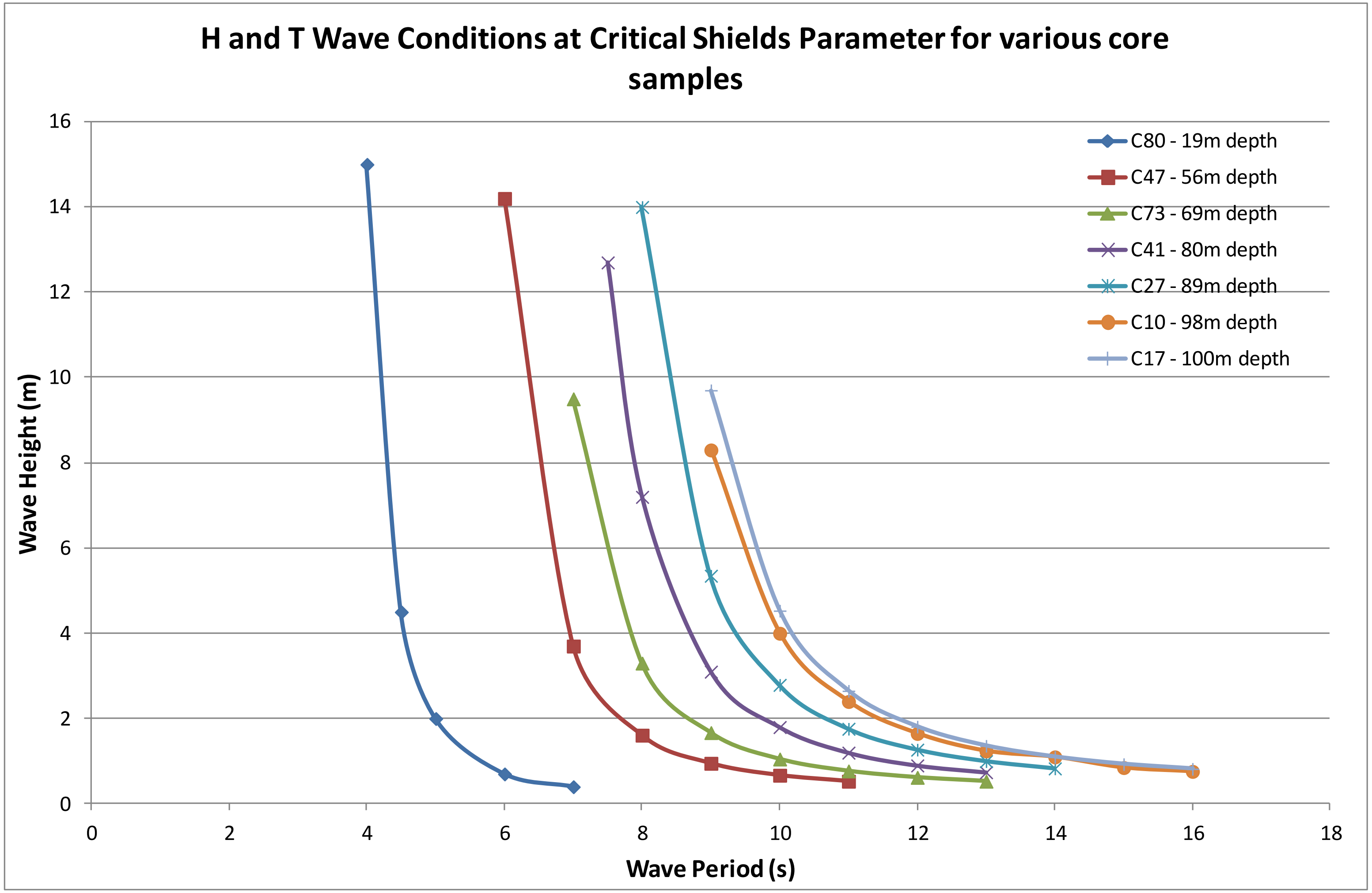


Figure 5.2 Critical Sediment Transport Conditions for each core sample

Core Sample	C80	C47	C73	C41	C27	C10	C17
Water depth (m)	19	56	69	80	89	98	100
Sediment height (m)	1	1	1	1	1	1	1
Excursion distance (m)	14.018	11.94	10.89	14.018	15.885	15.885	15.885
Sediment height (m)	5	5	5	5	5	5	5
Excursion distance (m)	70.09	59.7	54.44	70.09	79.43	79.43	79.43

Table 5.3 Excursion Distances for a given height above seabed at each core sample location

5.2 Coastal Description and Historical change

The section of coastline being considers has three beaches Annagh, Emlybeg and Belderra each separated by rocky outcrops (Figure 5.3). They are relatively short beach systems being only 0.45km, 1.2km and 0.35km in length respectively. It is the nature of wave approach to these beaches and the rocky outcrops that define their behaviour. From the previous section on the nearshore wave climate it was determined that wave approach is generally perpendicular to the beach thus giving rise to a cross shore sediment transport regime. The rocky outcrops are also important in this respect as they help to confine the sediment to the individual beach systems. Another point regarding Belderra beach is that surfers report strong rip currents which is another indication of direct wave attack and anecdotal validation of the numerical modelling work.

The sand on the beaches (Table 5.4) can generally be classified as being fine to medium in relation to the grain sizes. This sediment size has a high mobility especially in relation to the incident wave climate so it would be expected that quite large profile changes occur in the beach during storm events.

If the plan shapes of the beaches is considered it can be seen that they are attempting to adopt a curved equilibrium. This shape occurs for embayed beaches that have a dominant wave direction and is indicative of a highly stable coastline. From Figure 5.3 it can be seen that only Annagh beach achieved a proper curved plan shape whilst Emylbeg is trying to achieve a curved shape but is being restricted by the rocky outcrops. Belderra is such a small beach with rocky outcrop restrictions that it has not the scope to adopt an equilibrium curved shape. However as the subsequent analysis will show it still is a stable beach not subject to progressive erosion. Figure 5.4 shows that soft measures such as grass planting etc. may be necessary to help stabilise areas at the rear of the beach after extreme storm events. A geophysical survey carried out at Belderra Strand indicated deep layers of sand and shingle overlying strong gneissic rock. The maximum depth to strong rock of 15m is reached in the far northeast of the beach site. The survey showed that the wave energy connection seabed cable should be positioned close to the central or eastern part of the beach. This is where the sand and shingle, which could be excavated by digging / ripping, is thickest.

Various Ordnance maps and aerial photographs were used to show past beach position or in this case vegetation lines. This gives an indication of the changes that have occurred to coastline position and provides information on the processes that are occurring. The results of this analysis is shown in Figure 5.5 and it can be seen that there is very little variability in the coastline position. It indicates that this shoreline is very stable and that no significant persistent erosion is occurring. It is possible that certain storms may result in erosion but given that the beaches are swash aligned and the sediment moves in a cross-shore direction they can self repair over time. As already stated some minor remedial measures may be required from time to time.
The stability of the coast means that allowance not having to be made for significant bed level changes or coastal protection works needing to be planned. Finally as the changes to the beach are dictated by storm events and given the fact that WECs become in-operational in those sea states the overall effect of the deployments on beach processes will be negligible.

Figure 5.3 Beach Locations

Transect	Station	Latitude (Decimal degrees)	Longitude (Decimal degrees)
1	Low shore	54.21014	10.064591
	Mid shore	54.209607	10.063546
	Upper shore	54.209201	10.062842
	Strandline	54.209036	10.062636
2	Low shore	54.210991	10.062811
	Mid shore	54.210633	10.062104
	Upper shore	54.210236	10.061233
	Strandline	54.210101	10.06103

			Site					
Sediment type	Size	Phi	T1 LS	T1 MS	T1 US	T2 LS	T2 MS	T2 US
Medium pebble (gravel)	> 8 mm	< -3	0	0	0	0	0	0
Small pebble (gravel)	4-8 mm	-2 to -3	0	0	0	0	0	0
Granule	2-4 mm	-1 to -2	0	0	0	0	0	0
Sand - very coarse	1-2000 µm	0 to -1	0.12	0.11	0	0.08	0.01	0
Sand - coarse	500-1000 µm	1 to 0	3.33	1.74	0.10	0.99	0.02	0.01
Sand - medium	250-500 µm	2 to 1	30.68	45.74	26.01	42.63	14.87	4.38
Sand - fine	125-250 µm	3 to 2	62.95	40.31	70.63	56.17	85.06	95.36
Sand - very fine	63-125 µm	4 to 3	2.91	12.10	3.26	0.11	0.03	0.25
Silt & Clay	< 63 µm	>4	0.01	0.01	0.00	0.01	0.00	0.01
% Organic Matter (LOI)			1.75	1.63	1.42	1.80	0.97	0.99

Table 5.4 Beach sediment grain sizing

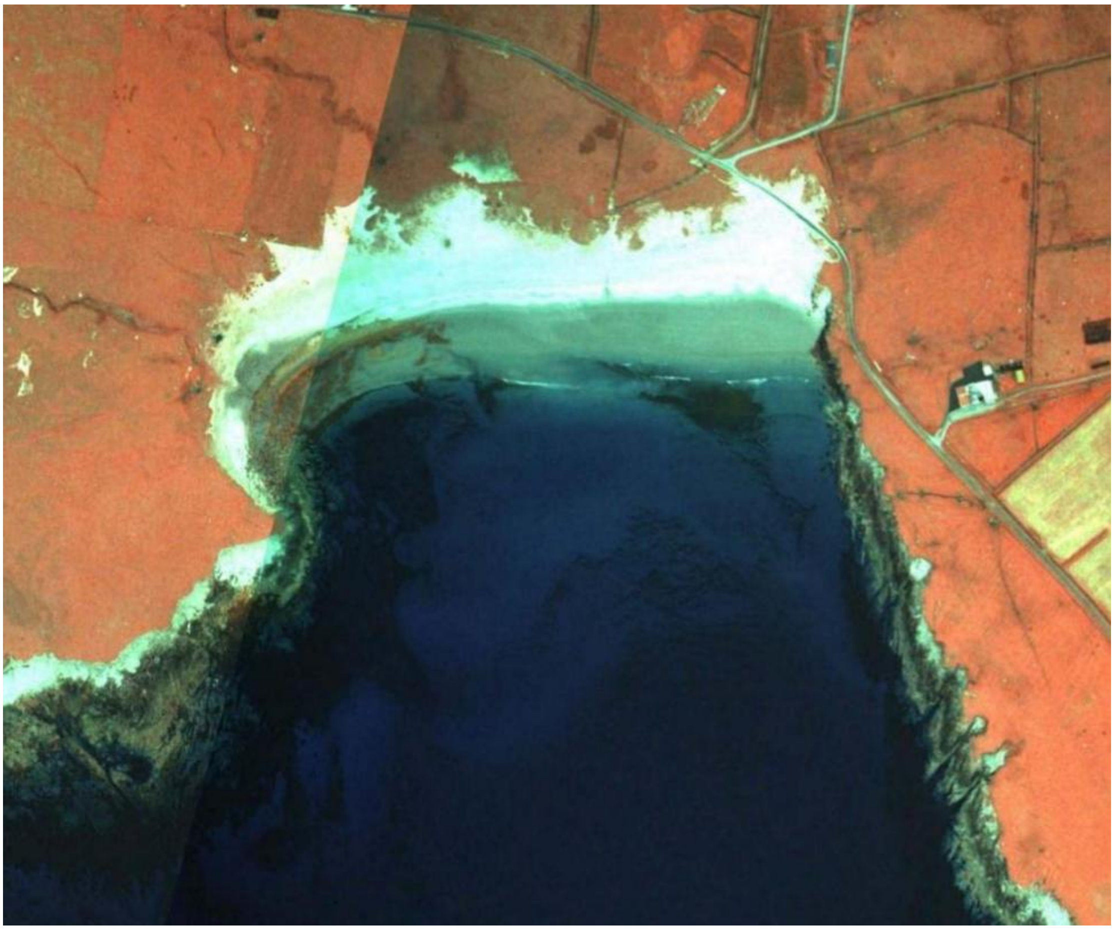


Figure 5.4 Belderra Beach

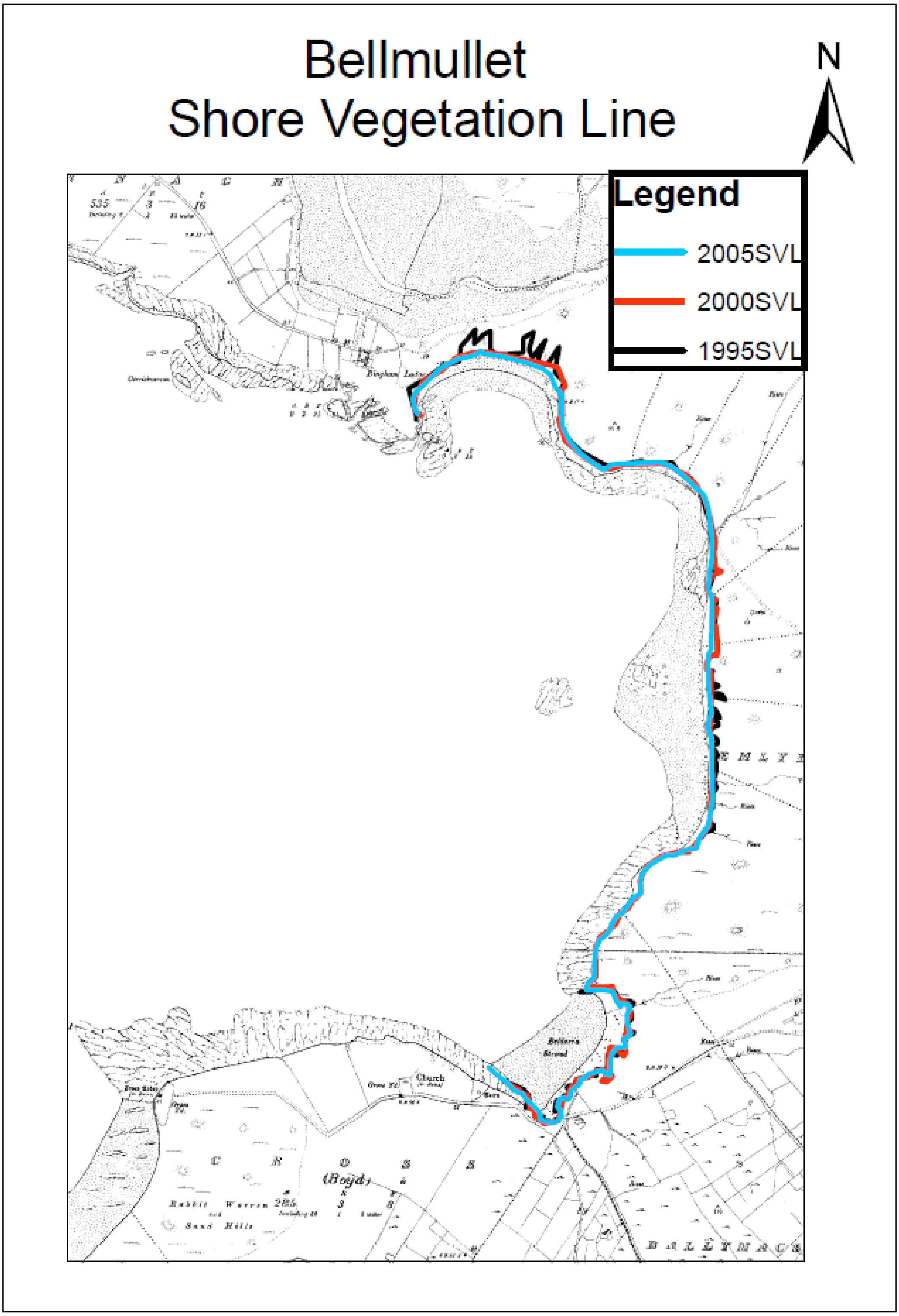


Figure 5.5 Shoreline Position

5.3 Beach Profile Changes

Two topographic surveys of Belderra beach were carried out; the first in September 2009 and the second in August 2010. These surveys only extended to the low water line on the particular day that they were carried out so they do not show the full profile to closure depth. Therefore any comment that is made relates to the visible part of the beach. Figures 5.6 to 5.8 show plan views and sections from the survey data.

The historical examination of coastal position had already shown that this length of coastline is stable and not subject to progressive erosion. The surveys re-affirm this conclusion in so much as the landward limit of the surveys show no differences even though significant changes have occurred further down the beach. As stated previously the beach systems respond to storm events by the drawdown of material from the upper beach to a nearshore location where it forms a bar that dissipates waves and helps protect the beach. This sand can subsequently be returned to the upper beach area by less steep wave conditions that generally occur during the summer period. What is unusual about the surveys is that there has been such a large drawdown of beach levels as recorded in the survey of August 2010 (see Figure 5.8) where levels have dropped by up to 1m from the previous year. A possible explanation for this drawdown at a time when beach regeneration would be expected is related to the wave conditions of July 2010. As can be seen from Figure 5.9 there was a wave event in July where the waves were consistently greater than 3m for an 8 day period and reaching a peak H_s of 8m. This prolonged storm event is likely to have resulted in sediment being pulled off the beaches and resulted in the reduction in levels from the September 2009 values. Thus what the surveys highlight is the natural variability of the beach levels but in effect only give a snapshot of its behaviour. More information is required to get a more complete understanding of the overall magnitude of bed level changes at Belderra beach.

Initially it was indicated that structural interventions in the form of reef type structures may be necessary close to the shoreline to protect the cable but as these are not now required, given the choice of the cable route, the beach profile will continue to respond directly to incoming waves as it has always done. As the level of energy loss caused by the WECs will be negligible so the nearshore wave climate will remain unchanged from the baseline condition. Therefore there will continue to be periods of drawdown and regeneration of the three beach systems. What is important is that even with this variability in the profile levels, the beach is inherently stable with no loss of sediment from the active system and no erosion tendency.

It is unlikely that the cable laying operations will have any influence on the overall behaviour given that the work will be undertaken during calm conditions and provided no sand is removed from the beach in the process. It would be important that the cable is buried deep enough to ensure that it would not be exposed during certain extreme storm events. Additional beach surveys that extend further offshore would help in this respect. If these were carried out after exceedance of some specified wave conditions then it would help with the design of the cable burial depth. There is a deep reservoir of sand on Belderra beach so there should not be any issues in terms of achieving a bigger burial depth.

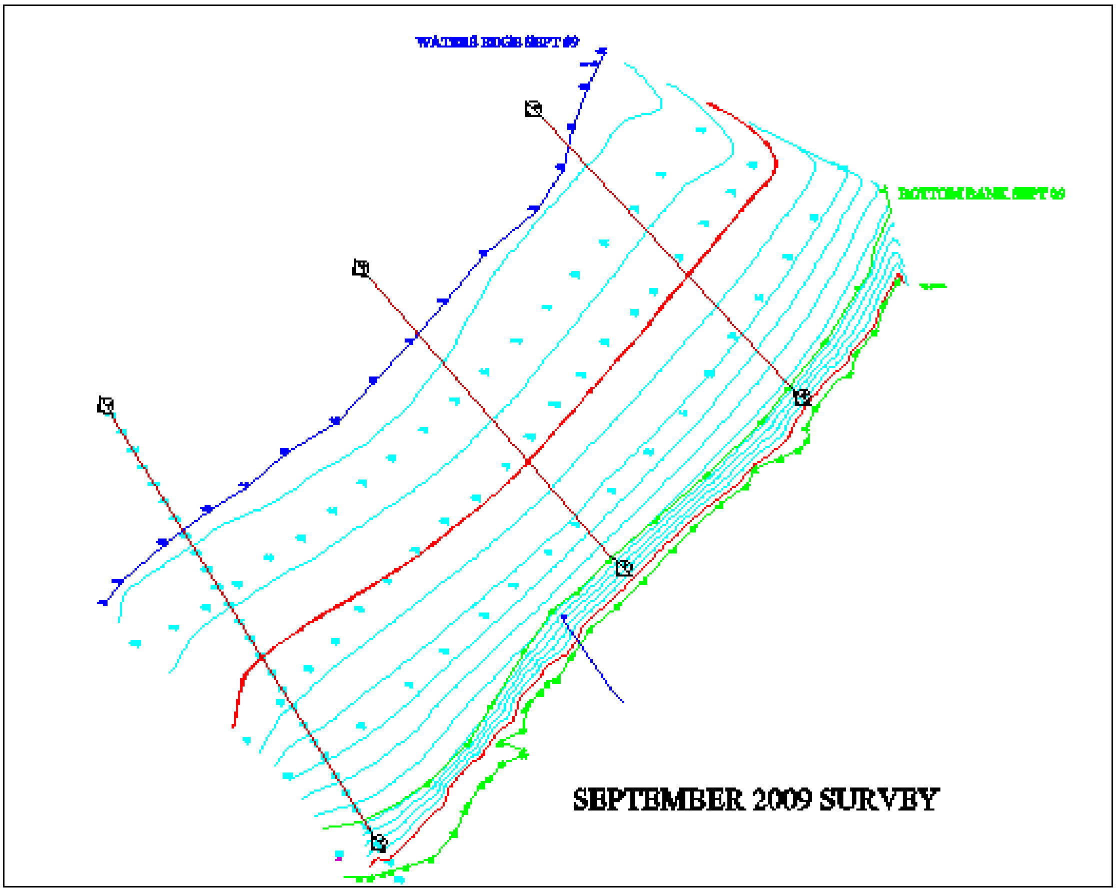


Figure 5.6 September 2009 Survey of Belderra Beach

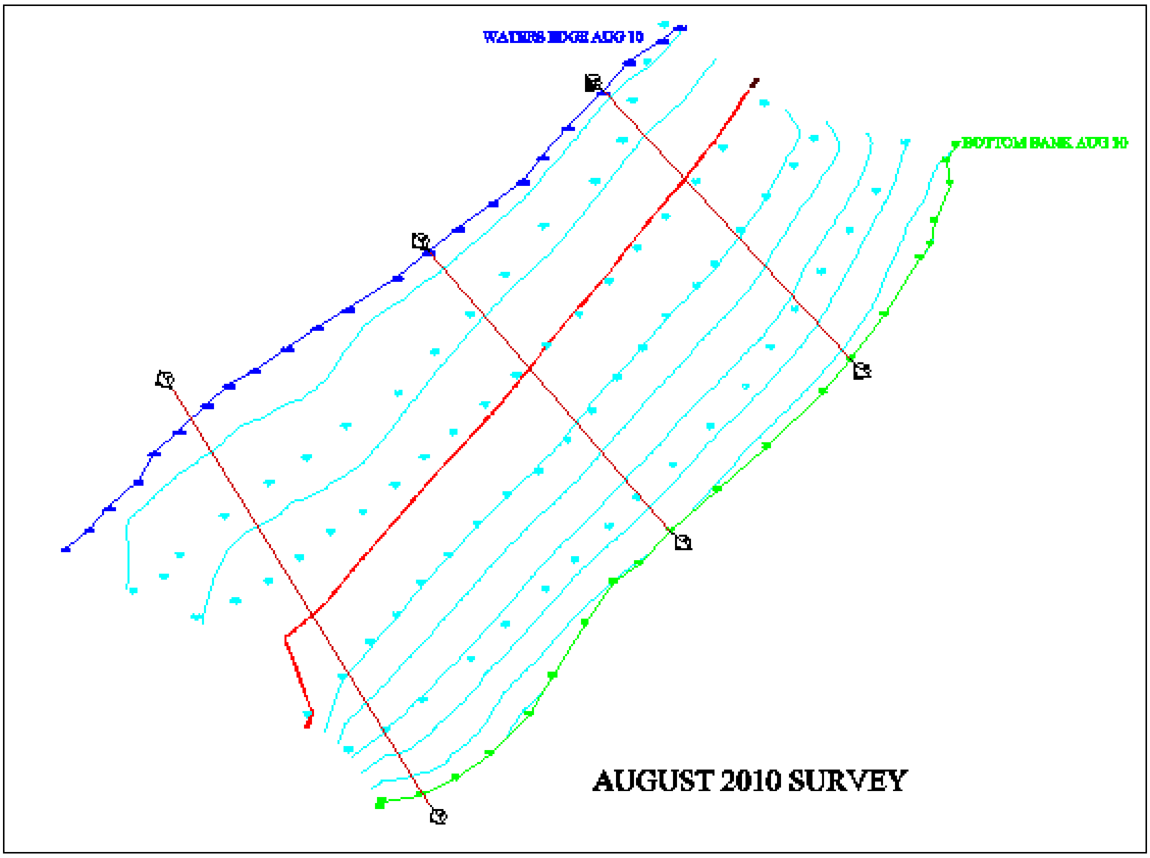


Figure 5.7 August 2010 Survey of Belderra Beach

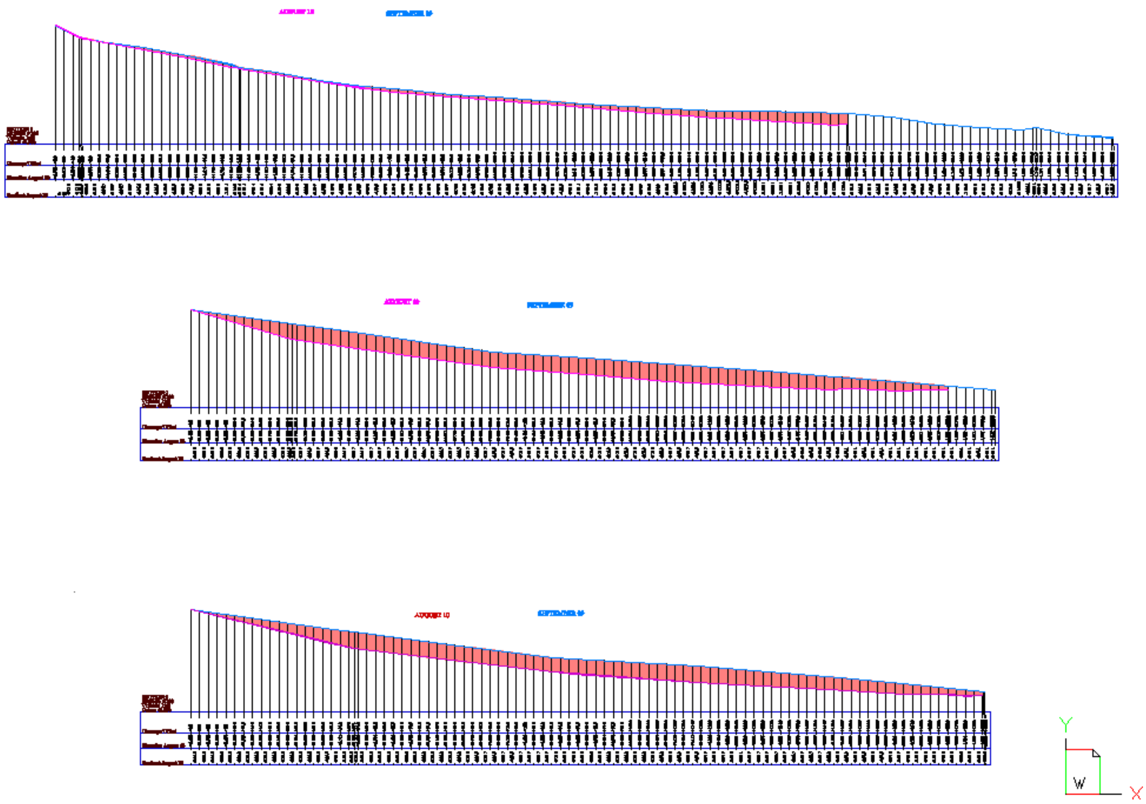


Figure 5.8 Beach Sections showing difference between surveys

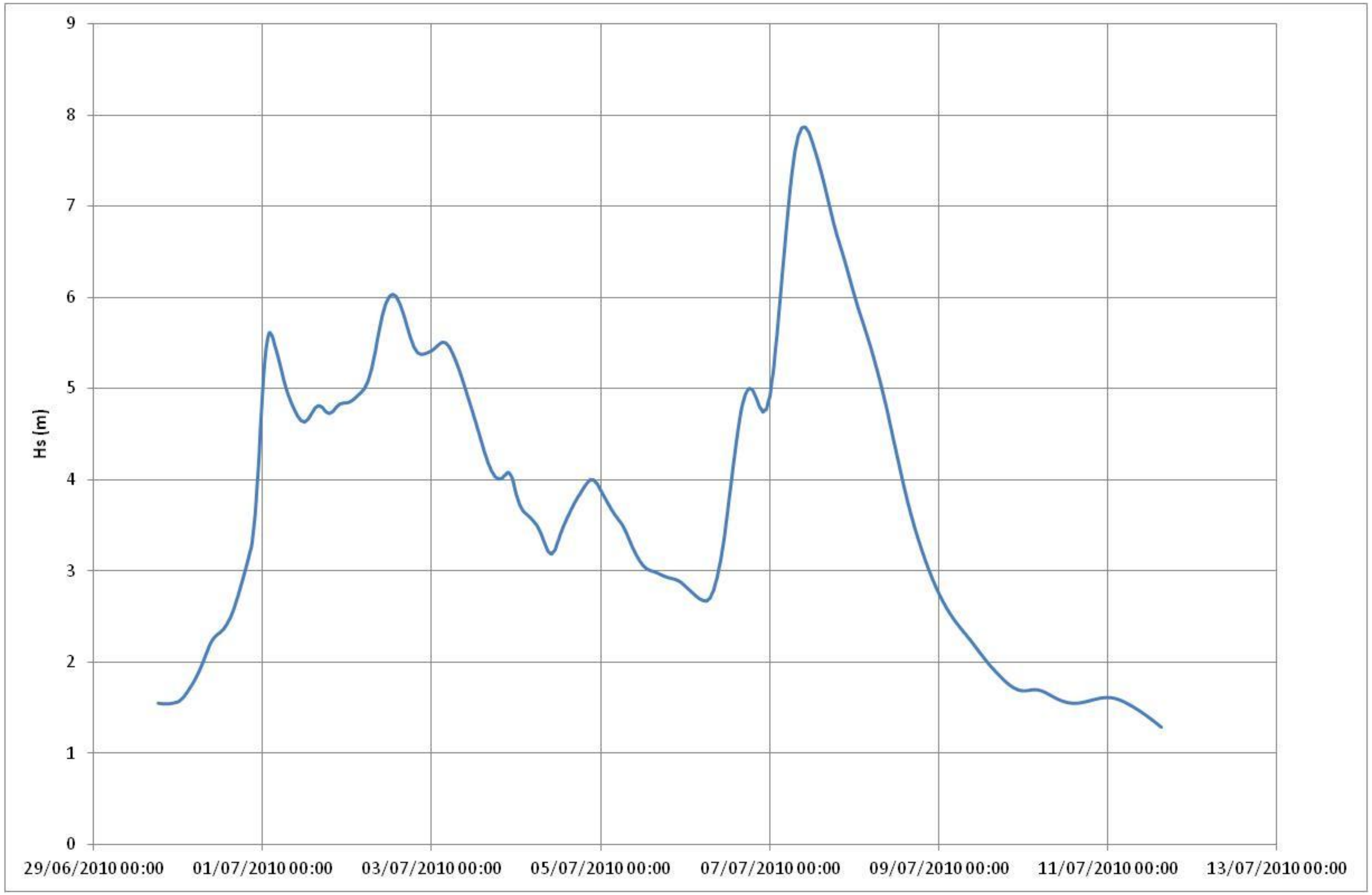


Figure 5.9 Offshore Wave heights July 2010

6. Surfing Impacts

Finally as there can be concerns from the surfing community regarding potential changes to the wave climate, this is now considered. In Annagh Bay surfing has been specified in the locations as shown in Figure 6.1. Before discussing this site reference is first made to various studies carried out in the UK for the Wave Hub site. These indicated that in a worst case scenario the wave conditions may be altered by up to 11%. Surfers objected to this as being excessive which lead to an important question being raised as to what is the threshold percentage wave height reduction at which it does make a difference to surfing. There seems to be no definitive answer to this with the view commonly taken being from a surfing point of view that any change is bad. In this study general points will be made in relation to potential impacts.

At a particular location good surfing conditions result from suitable wave, wind and tidal conditions. Longer swell wave periods (Tz >10s) are usually more favoured by surfers and for these periods WECs only extract small amounts of energy so offshore devices will have no impact on these waves.

Given that waves breaking in shallow water is related to wave height and water depth (which in turn is related to tidal elevation) there is a lot of natural variability in terms of the breaking process. In addition in a normal wave spectrum there can be a wide range of wave heights and periods so minor changes, as indicated in this study, to the incoming waves would be lost in this variability. Waves usually break when they reach a critical water depth. For regular/monochromatic wave conditions the ratio of H_b (breaking wave height) to d_b (breaking water depth) ranges from 0.78 to about 1.3 depending in part on the seabed slope. For irregular waves given that there is a spread of heights and periods the $H_{rms,b}$ is normally used to define breaking with a ratio with d_b of 0.42 is normally specified. Therefore a small change in wave height might slightly move the point of breaking but this point is continually moving in any case with changes to wave heights, periods, directions and water levels. So unless there was a large shift in the wave heights there should not be an observable difference in the wave breaking characteristics of the site.

An analysis was carried out on the wave conditions close to the beach similar to that undertaken in section 3.7. Only setup 7 was considered and the wave height change was determined at the locations as shown in Figure 6.2. The results of this analysis is shown in Figure 6.3 and it can be seen that changes of less than 1% occur. Obviously the impacts of WECs become less as energy is being lost be the waves by the various shallow water wave processes (breaking, refraction etc.). Also given the manner in which the WECs were modelled it is believed that these results mean no change to the wave conditions at these two locations and that the surf conditions will remain unaltered.

Finally another concern that surfers have is that changes to wave conditions can lead to changes in seabed morphology. However that argument does not really apply in this case. The beach profiles have been shown to have high variability which will alter the breaking characteristics significantly between the typical winter profile (where sand is moved to an offshore bar) to the summer profile (sand moved back onto the intertidal area). Therefore natural changes to seabed levels which are dependent on storm activity will have a potential much greater impact than possibly small wave height changes.

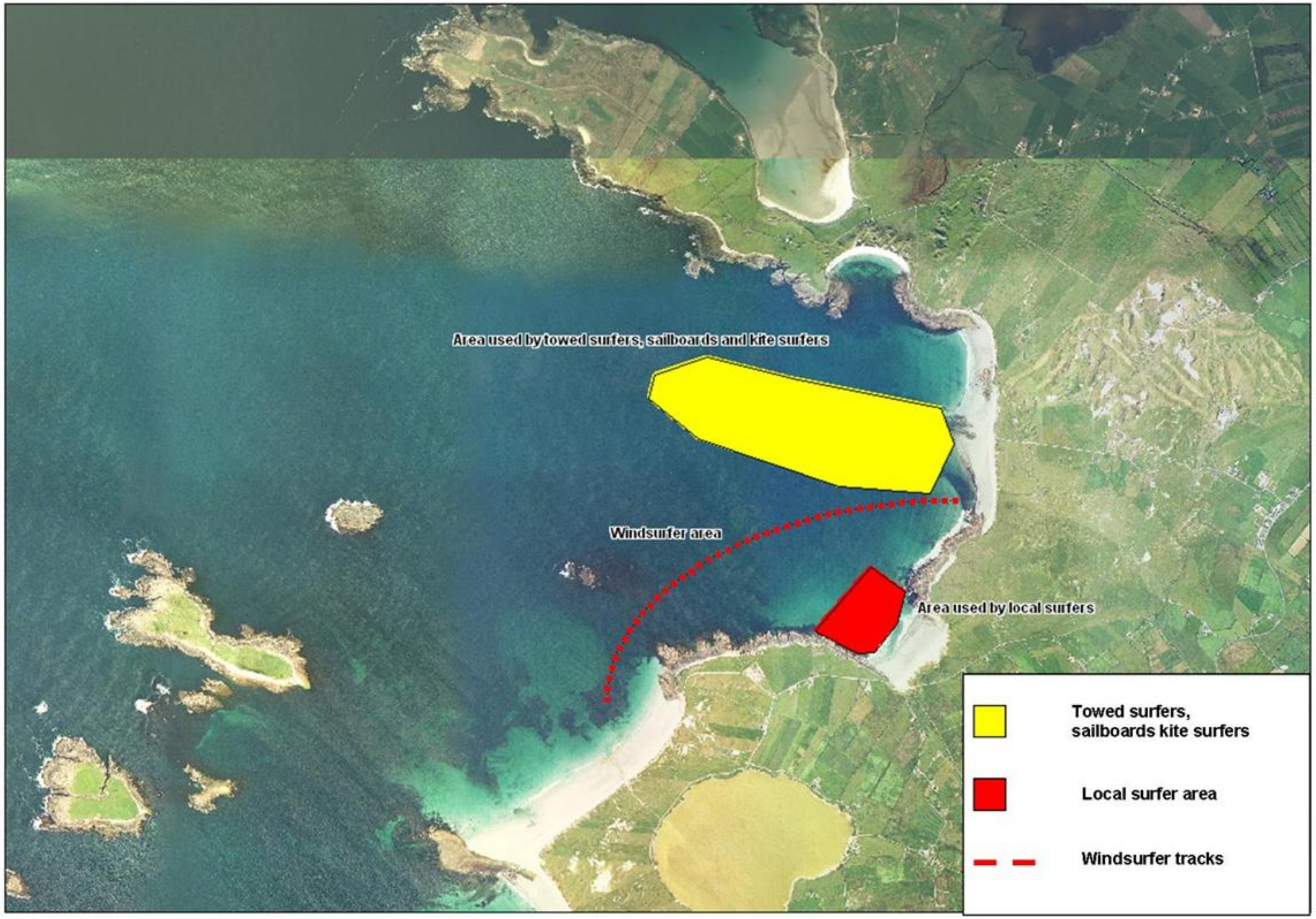


Figure 6.1 Surfing locations in Annagh Bay

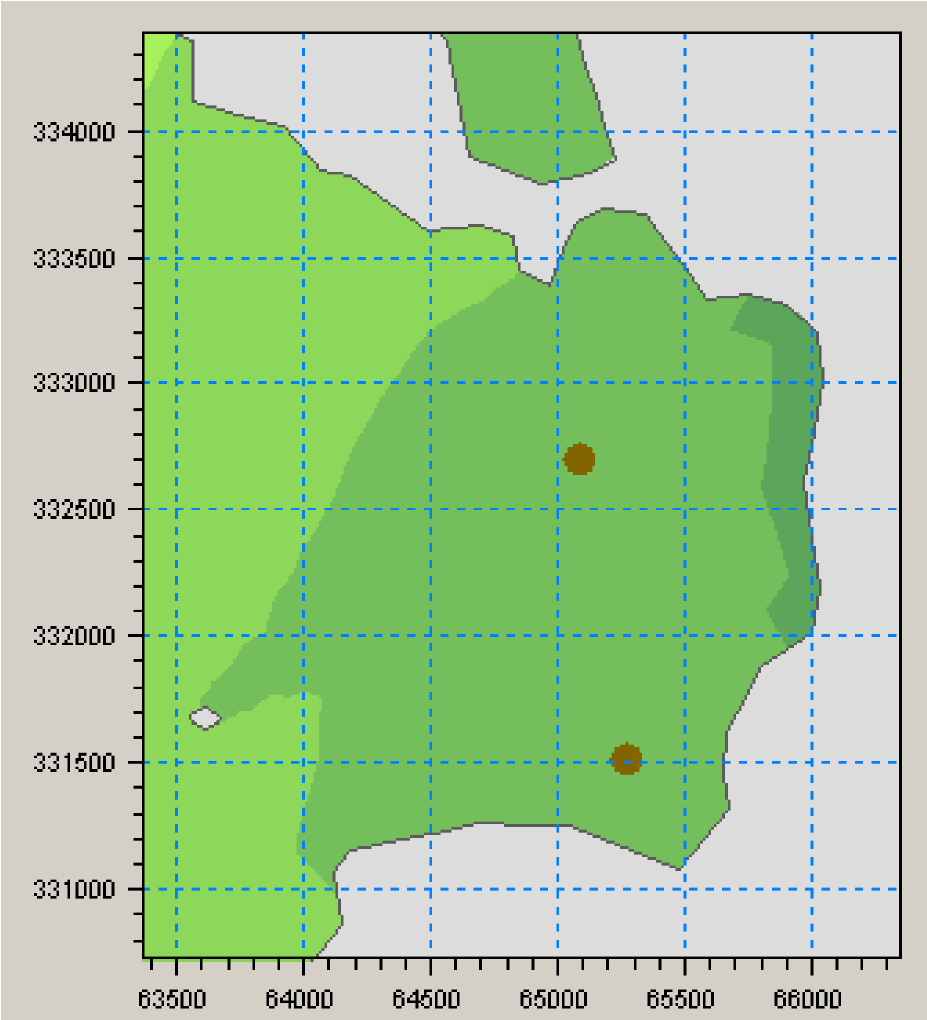


Figure 6.2 Model Data Extraction Locations

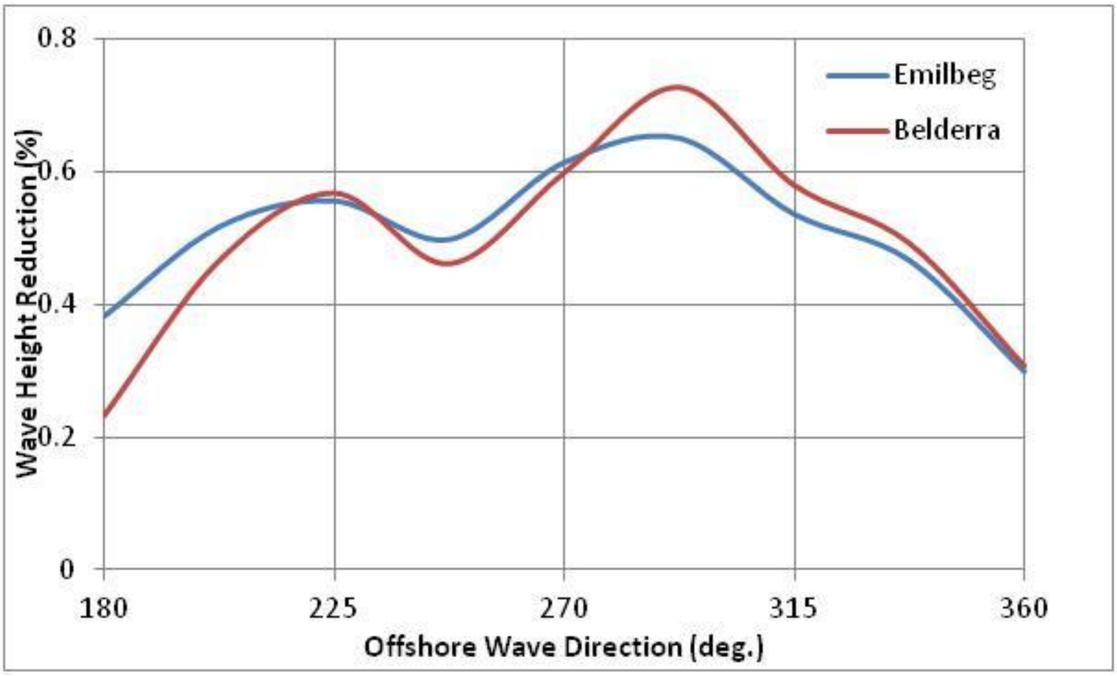


Figure 6.3 Wave height changes for Setup 7

7. Conclusions

This study considered wave, currents and sediment transport processes in the region of the Belmullet wave energy test site. The general conclusion is that neither the construction works or the operation of the wave energy convertors will have an impact on coastal processes at the relevant locations of interest. If the various elements of the work undertaken are considered the following specific conclusions can be made.

- There may be minor changes to wave heights under certain wave conditions after the deployment of WECs. These if they occur are not expected to have any impact on coastal processes or surfing activity.
- Wave periods and directions will not be affected by the presence of WECs.
- Waves approach Belderra beach primarily from a direction perpendicular to the beach orientation and so induce a cross shore sediment transport.
- Tidal currents reduce in magnitude east of Annagh head and are not important in terms of coastal behaviour
- The laying of the cables is likely to mobilise limited sediment but the analysis has shown that it will not impact on seabed morphology and that sediment mobilised naturally by waves and currents is far more significant.
- The landfall for the cable is on a relatively stable section of coastline as demonstrated by the historical review of coastal position
- The three beaches respond to storm events by adjusting their profiles such that they are in equilibrium with the waves. As such there can be significant variability in beach levels. Pre-construction surveys will thus be required to optimise the depth of burial of the cable.
- The study indicates that surfing activity will not be affected by the development of the AMETS site.

8. References

H D. Armono, K.R. Hall (2003). *Wave transmission on submerged breakwaters made of hollow hemispherical shape artificial reefs* Canadian Coastal Conference (http://www.reefbeach.com/Armono%20and%20Hall.pdf)