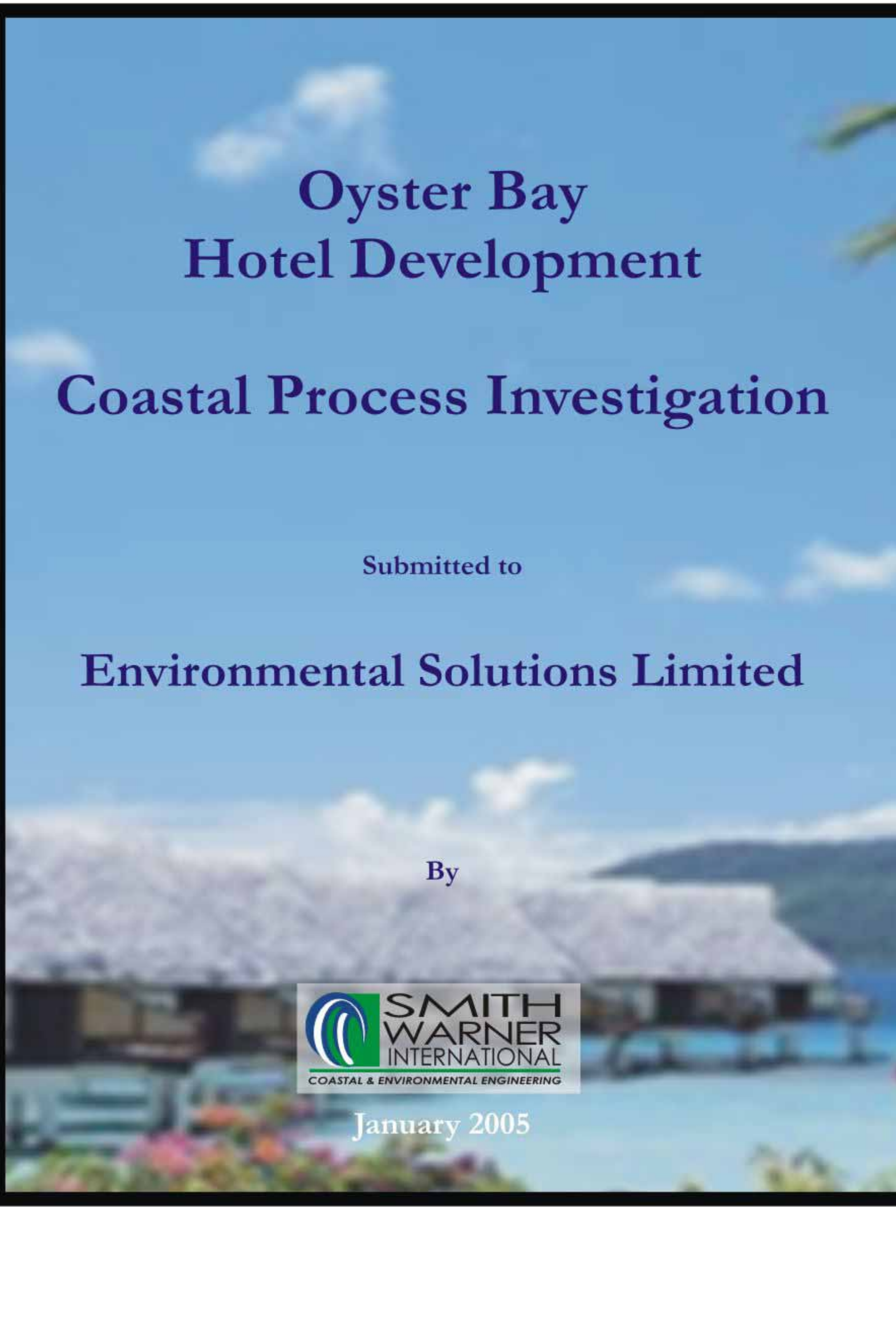

Oyster Bay Hotel Development

Coastal Process Investigation

Submitted to

Environmental Solutions Limited

By

SMITH WARNER INTERNATIONAL
COASTAL & ENVIRONMENTAL ENGINEERING

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Table of Contents

List of Figures

List of Tables

1. Project Background

Oyster Bay is located on the north coast of Jamaica, just east of Falmouth in the parish of Trelawny. The site of the proposed development lies on the exposed north coast and is therefore subject to strong waves, currents and winds. The offshore protective coral reef will reduce the wave heights, but the site remains at risk to several hazards that should be quantified. The primary hazards at this site would be due to extreme wind and wave conditions, which could result in high waves, storm surge, and strong currents. As such, an investigation quantifying these hazards has been carried out, in order that risks can be reduced to acceptable levels. In addition, there is the potential that the use of pile clusters as proposed by the developers may have impacts on the environment. These also must be quantified and mitigation measures developed as part of the NEPA approvals process. Smith Warner International was contracted to carry out the investigations required to quantify the hazards, and to investigate the use of pile clusters in the development.

This report describes the investigations that were carried out, and presents in detail the findings thereof.

2. Site Investigations

2.1 Bathymetry

A bathymetric survey of the nearshore region was carried out, mapping the topography of the existing seabed. Bathymetric data collection is essential to the prediction of the nearshore wave conditions and the alongshore sediment transport. The data was collected using a boat to traverse the study area, ensuring that the details of the shallow reef system parallel to the shoreline were captured.

The bathymetric data was logged to a hand-held computer at one-second intervals using an Autohelm ST60 depth gauge. Simultaneously, positions were logged using a Magellan ProMark X ten-channel GPS receiver. Tides and waves were not accounted for in the determination of the water depth, due to minimal tidal fluctuations (0.3m) and wave heights (0.5m) at the time of the survey. The required accuracy for the computations to be carried out does not exceed these levels, making the exclusion of this data acceptable. Data quality was checked as data from different survey runs were compared at data intersections. The coastal outline was demarcated by walking following along the waterline with the handheld GPS and logging the locations into the hand-held computer. The results of the bathymetric survey and coastline data are plotted in three-dimensional form in Figure 2.1

The bathymetry of the site is an undulating one. There is a major reef system central to the site, about 50m from the shoreline and spanning approximately 1600m. Smaller pockets of reefs can be found at the eastern and western ends of the larger reef. Seaward of the reefs the depth contours are generally in alignment with the shoreline. The slope of the seabed steepens beyond the reef, with a depth ranging from 5m to 20m over a distance of approximately 300m.

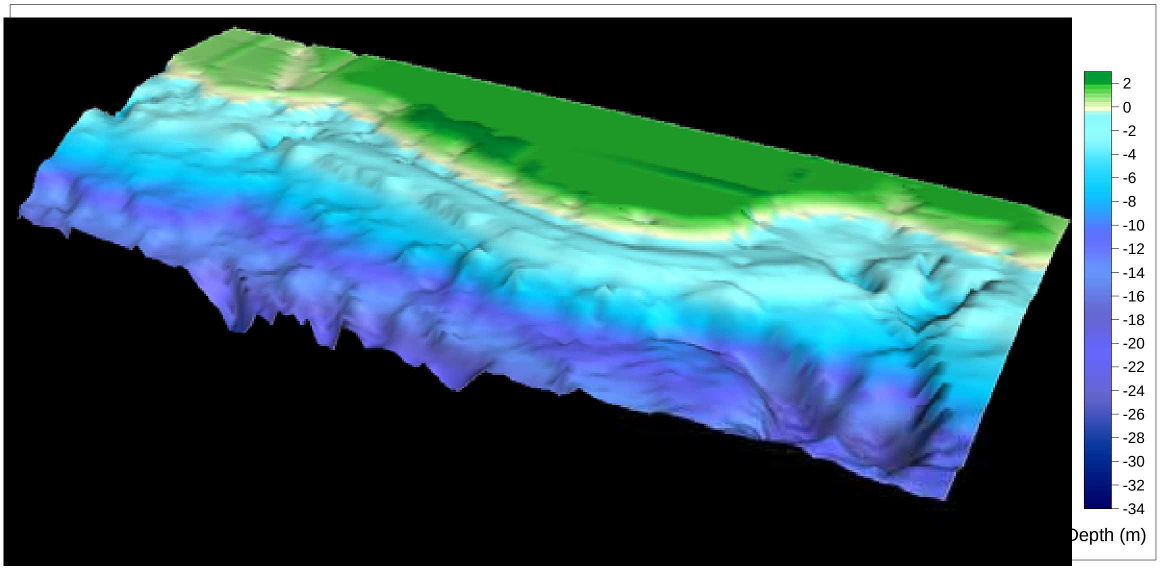

Figure 2.1 Existing bathymetry of Project Area

2.2 Currents

A series of drogue tracking exercises were conducted, capturing periods of rising and falling tides. Figure 2.2 presents a graphical summary of the stages of the tide examined. The drogues were allowed to drift unobstructed with the tidal current, and GPS readings of their locations and times were taken at intervals. This process provided a procedure from which the speeds that the drogues were transported by the currents could be calculated. The drogues were placed at various strategic locations in order capture the variation of current speeds within the study area.

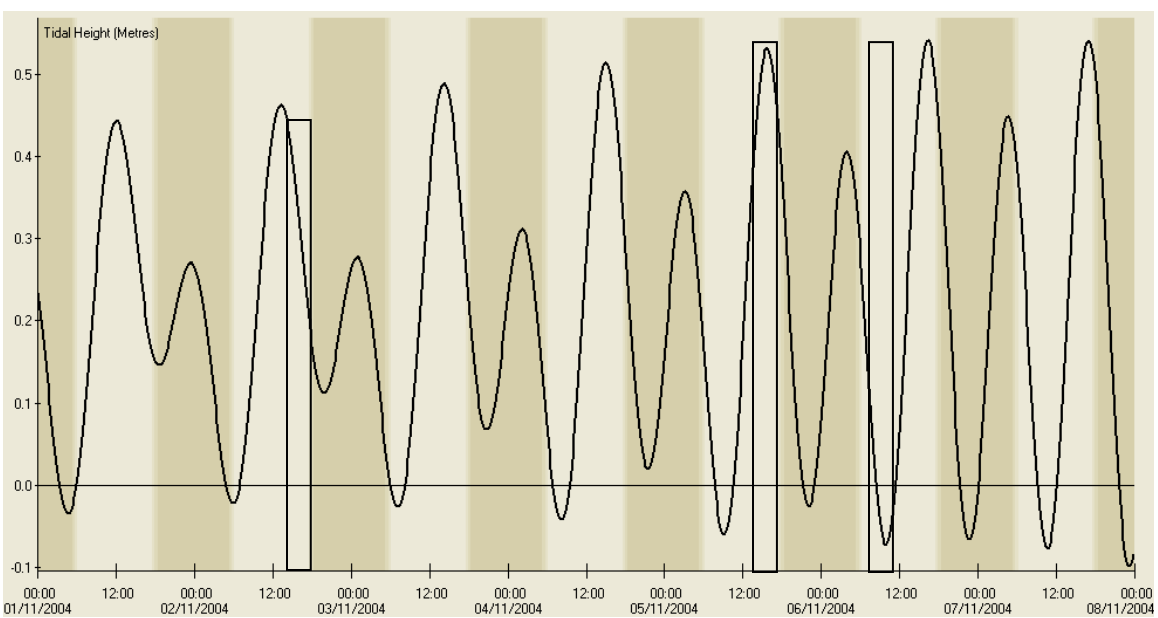

Figure 2.2 Tide stages during drogue tracking

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Figure 2.3 shows the directions and speeds at which the drogues moved. The currents generally move in a northeast direction on the rising tide and a southeast direction on the falling tide. The current speed at a given location in the project area is dependent on the stage of the tide and the local bathymetry. Current speeds computed from the drogue tracking exercise ranged from speeds as low as 0.032 ms^{-1} to speeds as high as 0.909 ms^{-1}.

Figure 2.3 Drogues tracks and speeds within the study area

When waves break on the reef, there is a collection of water between the reef and the shoreline, which runs alongshore current. At several locations, these are then transformed into a rip current as water escapes through the gaps in the reef. Rips currents poses a safety issue for users of the beach, as they can be swept offshore by the force of the current.

It is recommended that designated swimming areas be identified along the shoreline where the swimming hazard that the rip current poses are non-existent. As an alternative, artificial reefs could be created in the reef gaps to reduce the velocity of the rip currents. Re-profiling of the shoreline with the intention of reducing longshore currents and in turn reducing the rip currents is also another possible option. Finally, it is possible to construct deepened swimming areas along with cross-shore barriers to the alongshore current

2.3 Sediment Collection and Sieve Analysis

Establishing the properties of sediments in the study area is essential to the assessment of coastal processes such as sediment transport. Sediment samples were collected at four locations along the shoreline of the project site at Oyster Bay. Samples were collected on the upper face of the beach and in the swash zone. The samples collected were visually inspected, air dried and then subjected to a standard dry sieve analysis to determine the grain size distribution as well as other characteristic parameters. The parameters derived from this analysis are given in terms of both millimetres and Phi units (ϕ), where (ϕ) = $-\log_2$ (sediment diameter in mm). The total suite of parameters obtained from the sieve analysis procedure included size, sorting, skewness and kurtosis, descriptions of which follows.

- The characteristic size of a sand sample is defined by the *mean diameter*, d_{50}.

- The *degree of sorting* measured in the sample gives an overall indication of the gradation present in the sediment. A poorly sorted sample is one that contains a wide range of sediment sizes, whereas a well-sorted sample has an abundance of one size of sediment in the sample.
- The *skewness* gives the departure from symmetry of the analysed sand sample and indicates whether or not the sample is coarsely or finely skewed.
- *Kurtosis*, as measured by most sedimentologists, examines the ratio of sorting in the tails of the distribution, compared to sorting in the central portion. It is essentially a test of normality of a distribution. A curve with Kurtosis (K_G) = 2.00 is leptokurtic, or excessively peaked (relatively better sorting in the central area than in the tails). If the K_G = 0.70, then the sample is platykurtic or deficiently peaked, and there is better sorting in the tails than in the central portion of the curve. This information also provides some inference as to the type of wave processes to which the sediments are typically exposed. A summary of the sieve analysis results is given in Table 2-1 below. The analysis revealed the presence of finer sediments in the upper beach face (mean diameter of approximately 0.20mm) and coarser sand in the swash zone (mean diameter of approximately 0.68mm). The sediments in the upper beach face are moderately well sorted while those in the swash zone are poorly sorted.

Table 2-1 Grain Size Characteristics (from sieve analysis)

Sample	Mean Diameter (phi) / (mm)	Stand. Dev. (Sorting)	Skewness (Departure from Symmetry)	Kurtosis (Ratio of Sorting,)	Description (Based on Phi statistics)
Time and Place	2.35 / 0.20	0.55	-0.93	5.59	Fine sand, moderately well sorted, very coarse-skewed, extremely leptokurtic.
Bay 2 – Upper Beach	2.22 / 0.21	0.66	-0.52	6.73	Fine sand, moderately well sorted, coarse-skewed, very leptokurtic
Bay 2 – Swash Zone	0.52 / 0.70	1.56	0.18	2.51	Coarse sand, poorly sorted, fine-skewed, very leptokurtic
Bay 3 – Swash Zone	0.58 / 0.67	1.05	0.16	3.05	Coarse sand, poorly sorted, fine-skewed, very leptokurtic

Note: All parameter values are in Phi units unless stated otherwise.

3. Wave Climate

The proposed development of the Florida Lands for the creation of "Bora Bora" style clusters of suites over the water requires in-depth knowledge of a range of prevailing natural conditions. The definition of wave conditions in shallow water is an essential aspect in the design and operation of a wide variety of coastal facilities and for the safe performance of human activities in coastal areas. These conditions include wave characteristics during both daily (operational), severe (extreme) conditions and design water level data. The daily wave conditions will determine the maximum wave characteristics for which the facilities in and around the shoreline can be used, while the extreme conditions determine the maximum wave characteristics which any coastal structures will have to endure. The characteristics of the incoming waves in both cases are represented by a significant wave height, period and direction for which an acceptable chance of exceedance must be defined.

Most ocean deep water waves are generated by the winds in deep sea and propagate towards the shore. Deep water wave climates were investigated for both operational and extreme wave conditions. As the waves propagate from deep to shallow water, they experience changes in height and direction due to the presence of uneven bathymetry, islands, headlands and structures. The wave transformation processes comprise shoaling, refraction, diffraction, wave breaking, friction, etc. which will change the waves' heights and directions, however, the wave period is unaffected by these processes. The following sections address the simulation methods and provide a summary of the results for wave conditions in deep water and in the nearshore zone.

3.1 Operational Wave Climate

3.1.1 Deep Water Waves

Operational waves were obtained from the Alkyon Hydrobase, a long-term database of statistical data on waves derived from visual observations taken from ships. The ship observations were made from 1960 to 1997, north of the project site in an area defined by 18.5 to 19 degrees latitude and 77.5 to 78 degrees longitude. The analysis of this data employed directional filtering of the offshore waves from 270^0-90^0 through 0^0N, such that only waves reaching the study area were considered in the investigation. The data was filtered into the following four directional bins:

Sector 1 – Waves from the west to northwest (270-315°)

Sector 2 – Waves from the northwest to north (315-360°)

Sector 3 – Waves from the north to northeast (0-45°)

Sector 4 – Waves from the northeast to east (45-90°)

Figure 3.1 presents a graphical summary of the directional distribution of wave heights for the study area. The plot clearly demonstrates that waves are approaching the site predominantly from the 45^0 -90^0 (Sector 4) direction.

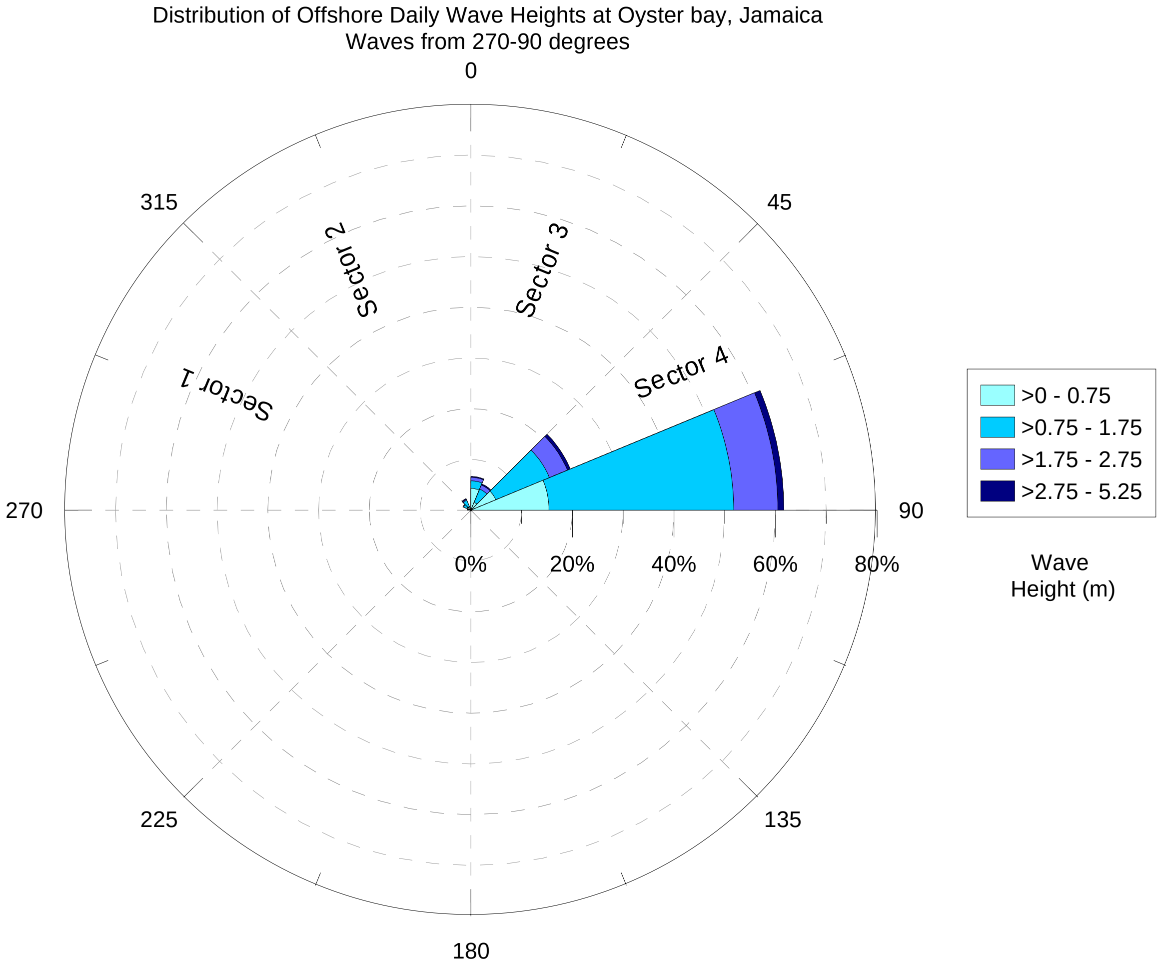


Figure 3.1 Directional and Quantitative Distributions of Deep Water Daily Wave Heights

An average wave height and period was computed for each directional sector (270^0-90^0) (see Table 3-1). The results of the computations were then used as the average condition for deep water waves in each directional sector that was transformed to the nearshore.

3.1.2 Wind data

Measured wind data from the Sangster International Airport was used in the transformation of waves from deep water to nearshore. Similarly as with the wave data, directional filtering was applied to the wind data, such that only winds from directions affecting the site were considered in the analysis. This information is presented in Figure 3.2, which shows that the wind approaches the site predominantly from the northeast-east direction. To obtain the design wind speeds, the wind observations in each directional sector were averaged.

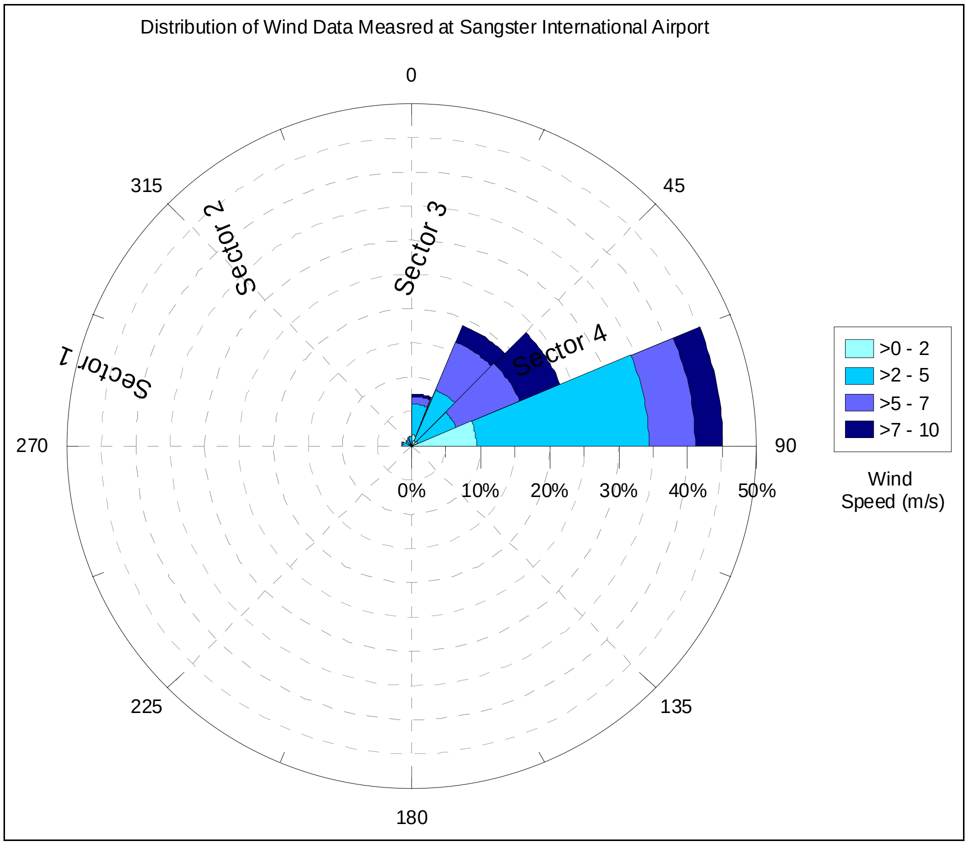


Figure 3.2 Directional and Quantitative Distribution of Wind Data

Table 3-1 shows the deep water design conditions, including wave height, period, direction and wind speed.

Table 3-1 Daily Deep Water Wave Climate

Waves	Hs (m)	Period (s)	Wave Direction from North (°)	Wind Velocity (m/s)
Sector 1 W-NW waves	0.95	5.11	292.5	3.04
Sector 2 NW-N waves	1.19	5.75	337.5	3.14
Sector 3 N-NE waves	0.95	5.32	22.5	4.76
Sector 4 NE-E waves	1.2	5.06	67.5	4.46

3.2 Extreme (Hurricane) Wave Climate

3.2.1 Deep Water Waves

Property damage and loss from hurricanes have increased with population growth in coastal areas, and climatic factors point to more frequent and intense hurricanes in the future. As a result, there is an increased awareness of the likelihood of catastrophic hurricane damage to developments in hurricane-prone areas. A common image of the damaging effects of hurricanes is that of storm-driven waves crashing against a shoreline, destroying fishing piers and coastal homes. Storm surge ahead of the hurricane raises the sea level and carries damaging waves onshore, which crash against any structure in their path with tremendous force and cause flooding in areas normally well above the high tide line. Structures that are to be implemented along this shoreline must be designed to withstand these hurricane waves, therefore the estimated storm surge levels along the shoreline of the project site must also be determined.

A hindcast analysis was carried out, which involved the use of parametric wave models to derive a data series of wave heights from hurricanes. The NOAA database of hurricane records, which dates back to the 19th century, was used in this analysis. The analysis incorporates, from the database, all hurricanes passing within a 400 km radius of the Oyster Bay shoreline, and wave heights computed for those selected occurrences.

As with the operational wave data, the extreme wave data was filtered into four directional bins: W-NW, NW-N, N-NE, and NE-E. An extremal analysis was carried out for each data series of wave heights. The following plots (Figure 3.3 to Figure 3.6) show the fit of the extremal distribution (Weibull) to the data series of wave heights. Coming out of the extremal analysis, a number of return period events were identified. The results of this hurricane analysis are shown in Table 3-2 to Table 3-5 for waves from the W-NW, NW-N, N-NE, and NE-E, respectively. The exceedance probability shown in the table represents the chance that the event will occur at least once within the next fifty years. For marine infrastructure in the Caribbean, a 50-year return period design condition is recommended.

Table 3-2 Results of Statistical Hurricane Analysis for Directional Sector 1, W-NW Waves

Weibull Distribution, k =	1.4		
Correlation Coefficient =	0.916		
Return Period (years)	**Significant Wave Height H_s (m)**	**Wave Period T_p (s)**	**Exceedance Probability (%) for 50 yrs**
2	0.0	0.0	100.0
5	0.8	3.0	100.0
10	2.9	6.5	99.5
20	4.3	8.2	92.3
25	4.6	8.7	87.0
50	5.7	9.8	63.6
100	6.6	10.8	39.5

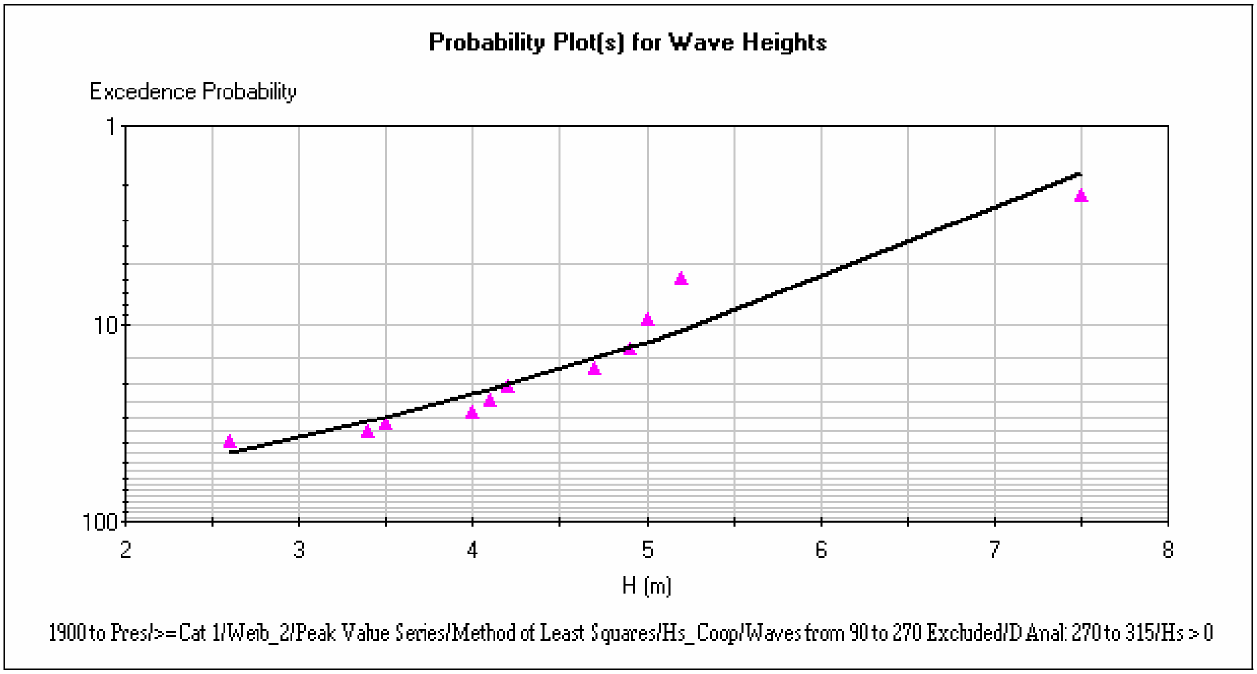


Figure 3.3 Hurricane Wave Extremal Analysis for Directional Sector 1, W-NW Waves

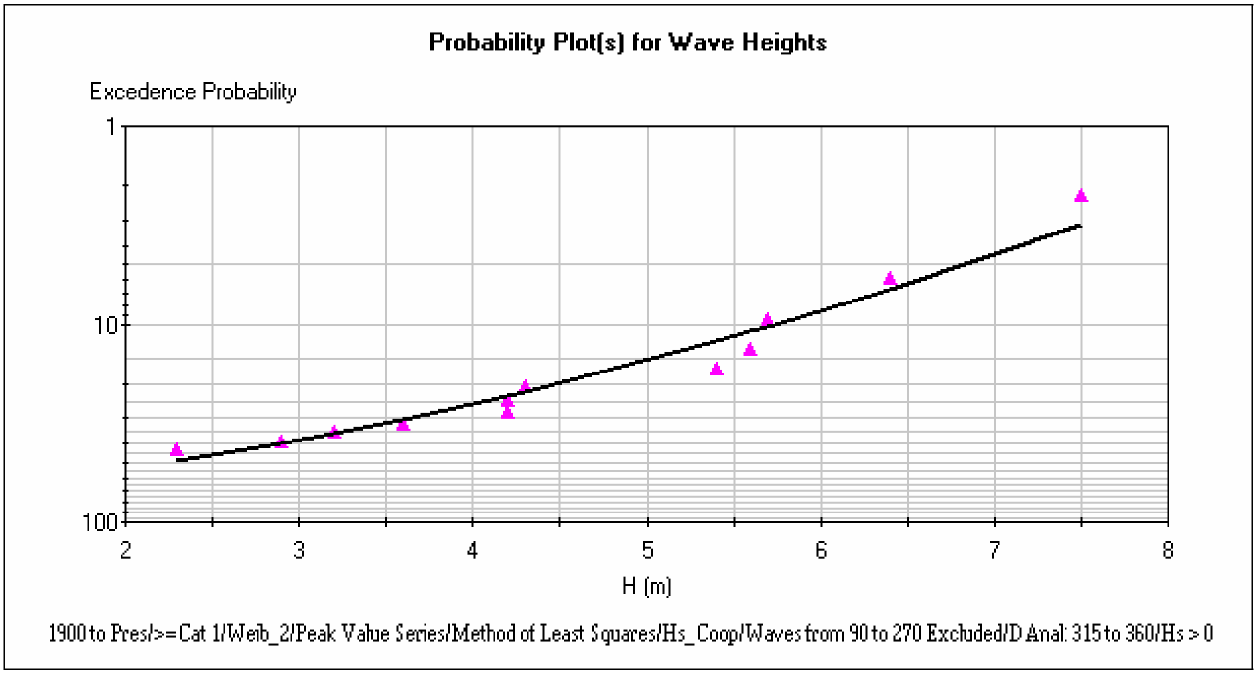


Figure 3.4 Hurricane Wave Extremal Analysis for Directional Sector 2, NW-N Waves

Table 3-3 Results of Statistical Hurricane Analysis for Directional Sector 2, NW-N Waves

Weibull Distribution, k =	2.0		
Correlation Coefficient =	0.943		
Return Period (years)	**Significant Wave Height H_s (m)**	**Wave Period T_p (s)**	**Exceedance Probability (%) for 50 yrs**
2	0.0	0.0	100.0
5	0.6	2.4	100.0
10	3.0	6.6	99.5
20	4.6	8.6	92.3
25	5.0	9.1	87.0
50	6.2	10.4	63.6
100	7.2	11.4	39.5

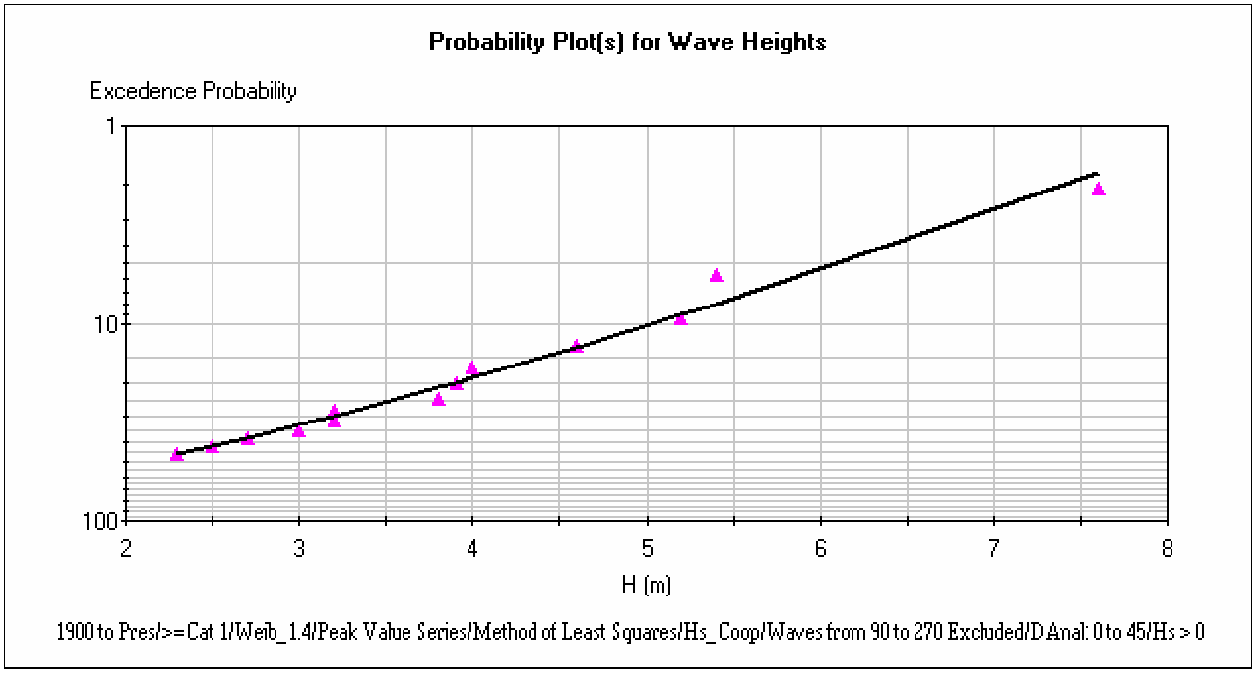


Figure 3.5 Hurricane Wave Extremal Analysis for Directional Sector 3, N-NE Waves

Table 3-4 Results of Statistical Hurricane Analysis for Directional Sector 3, N-NE Waves

Weibull Distribution, k =	1.4		
Correlation Coefficient =	0.987		
Return Period (years)	**Significant Wave Height H_s (m)**	**Wave Period T_p (s)**	**Exceedance Probability (%) for 50 yrs**
2	0.0	0.0	100.0
5	1.0	3.3	100.0
10	2.7	6.1	99.5
20	4.0	7.8	92.3
25	4.3	8.3	87.0
50	5.5	9.6	63.6
100	6.5	10.7	39.5

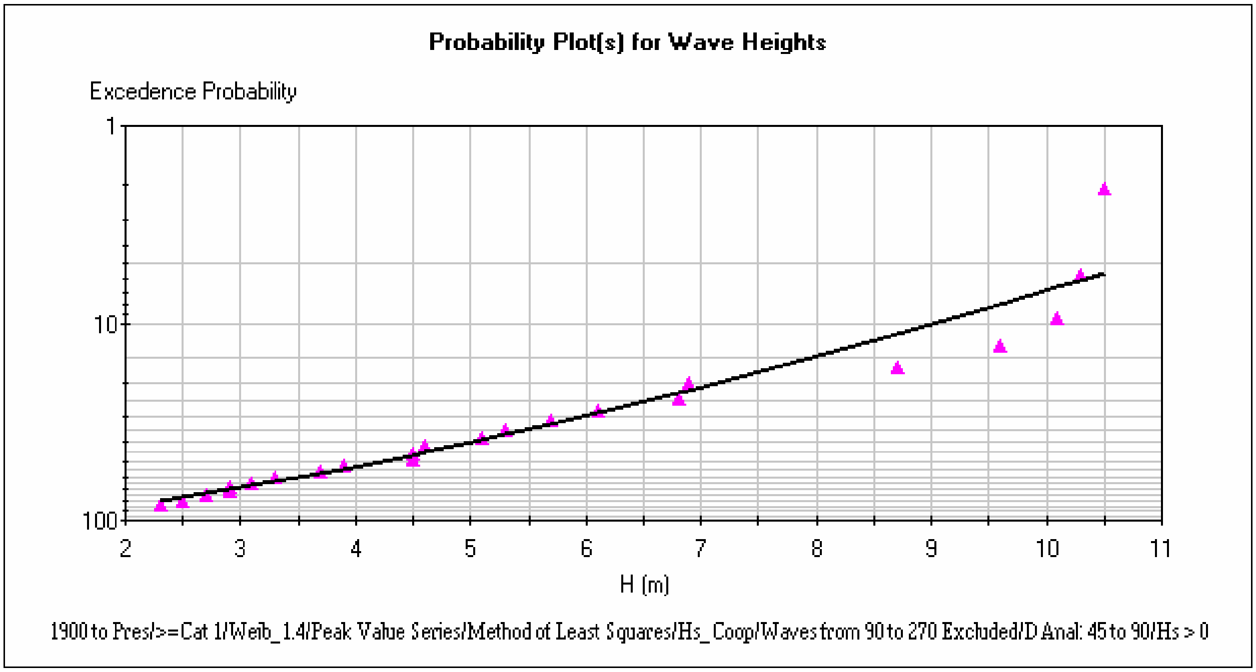


Figure 3.6 Hurricane Wave Extremal Analysis for Directional Sector 4, NE-E Waves

Table 3-5 Results of Statistical Hurricane Analysis for Directional Sector 4, NE-E Waves

Weibull Distribution, k =	1.4		
Correlation Coefficient =	0.989		
Return Period (years)	**Significant Wave Height H_s (m)**	**Wave Period T_p (s)**	**Exceedance Probability (%) for 50 yrs**
2	0.7	2.6	100.0
5	2.4	5.7	100.0
10	5.1	9.2	99.5
20	7.3	11.5	92.3
25	7.9	12.1	87.0
50	9.7	13.8	63.6
100	11.4	15.3	39.5

Storm surge resulting form a hurricane is an increase in ocean level due to a combination of:

- Direct wind-driven water (wind setup),
- Potential energy due to wave breaking (wave setup), and
- Uplift induced by atmospheric pressure drop (inverse barometric pressure rise).

Figure 3.7 shows the fit of the data set of water level increases (resulting from the IBR component) to the exponential distribution.

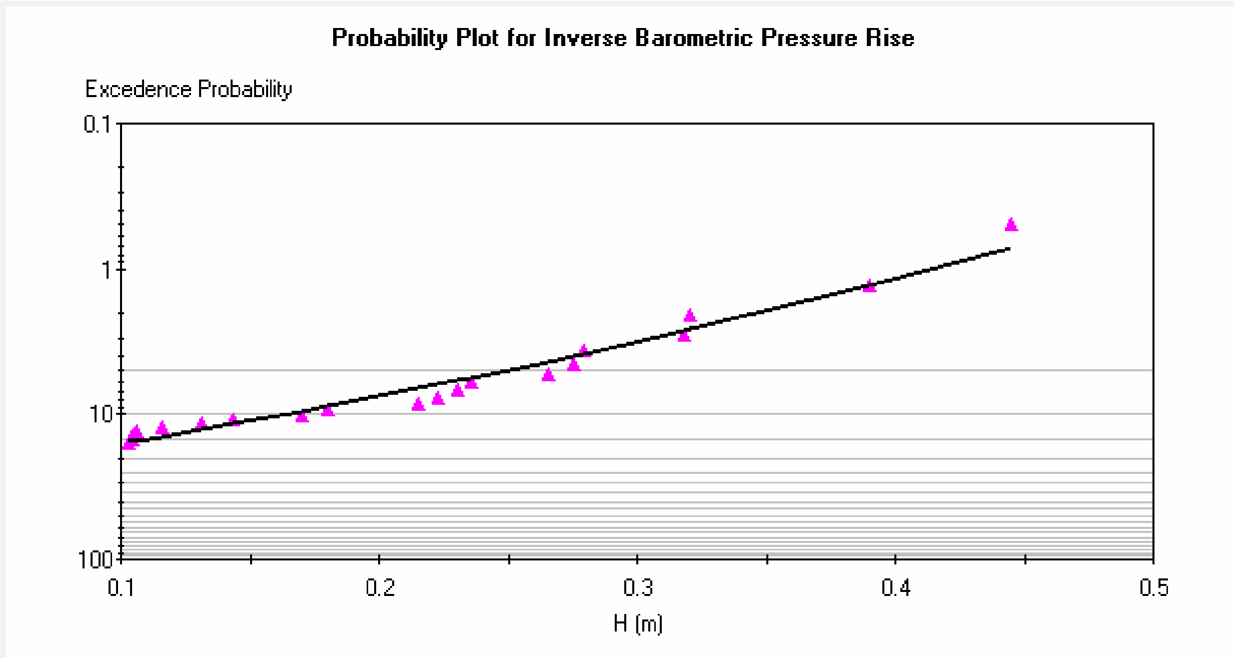


Figure 3.7 IBR versus Exceedance Probability

Table 3-6 shows the expected water level rise due to IBR for a range of return periods, along with the Exceedance Probability (the chance that the event will occur at least once within the next 50 years). Thus, using the 50-year exceedance probability for the IBR, the total deep water sea level rise consists of 0.36m for IBR, maximum tide level of 0.3m and a sea level rise due to global warming of 0.25 m in 50 years, resulting in a value of 0.91m.

Table 3-6 Chance of Exceedance of Barometric Pressure Rise

Weibull Distribution, k =	2.0	
Return Period (years)	**Water Level Rise due to IBR (m)**	**Exceedance Probability (%) for 50 yrs**
2	0	100.0
5	0.09	100.0
10	0.19	99.5
20	0.27	92.3
25	0.29	87.0
50	0.36	63.6
100	0.43	39.5

Table 3-7 shows the results for the deep water design conditions for extreme (hurricane) waves. These conditions will be modelled to estimate water levels on site resulting from storm events.

Table 3-7 Extreme Deep Water Wave Climate

Waves	Return Value (years)	Hs (m)	Period (s)	Wind Velocity (m/s)	Sea Level Rise (m)	Wave Direction from North (°)
Sector 1 W-NW waves	50	5.66	9.84	22.6	0.91	292.5
Sector 2 NW-N waves	50	6.16	10.37	24.6	0.91	337.5
Sector 3 N-NE waves	50	5.45	9.60	21.8	0.91	22.5
Sector 4 NE-E waves	50	9.70	13.81	38.8	0.91	67.5

3.3 Transformation of Waves to Nearshore

Operational deep water waves were transformed to the nearshore regions using SWAN (Simulating Waves Nearshore), a third-generation wave computer model that computes random, short-crested wind-generated waves in coastal regions and inland waters. Nearshore conditions obtained from the simulation were then used as input in determining alongshore sediment transport for the study area. Wave conditions used as input to SWAN represent the average wave height and period for each of the four sectors of waves reaching the site.

A rectangular grid of the seabed topography was created of an area approximately 2850m x 1800m, extending from about 2m above mean sea level to the 100m water depth on the northern boundary. For the northern boundary condition, a JONSWAP wave spectrum was created in SWAN by using the input deep water wave conditions. The spectrum was defined by the significant wave height, the characteristic period of the energy spectrum, the peak wave direction and the coefficient of directional spreading (m). The minimum and maximum periods were set to 4.5 and 12.5 seconds, respectively. The wind speed and direction used in the simulation is the same as the values for each direction presented in Table 3-1.

The results from the model runs are presented in Figure 3.8 to Figure 3.11, which illustrate the variation of wave heights over the project area. The results confirm the fact that the reef system reduces wave heights through the process of wave breaking, therefore providing some amount of protection to the shoreline from wave attack. Wave heights shoreward of the reefs are generally between 0.05m and 0.3m however, in areas where there are gaps, for instance in the most western bay, wave heights range from 0.25m to 0.7m.

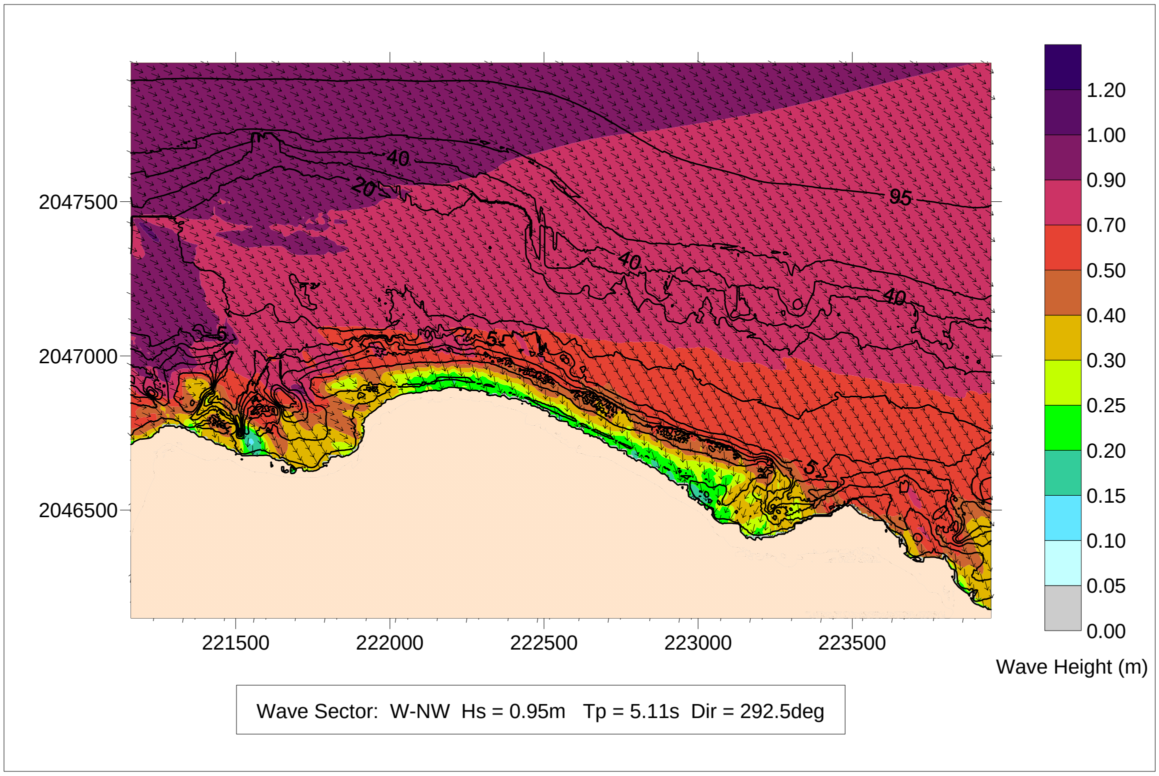


Figure 3.8 Waves in study area dues to W-NW deep water wave conditions

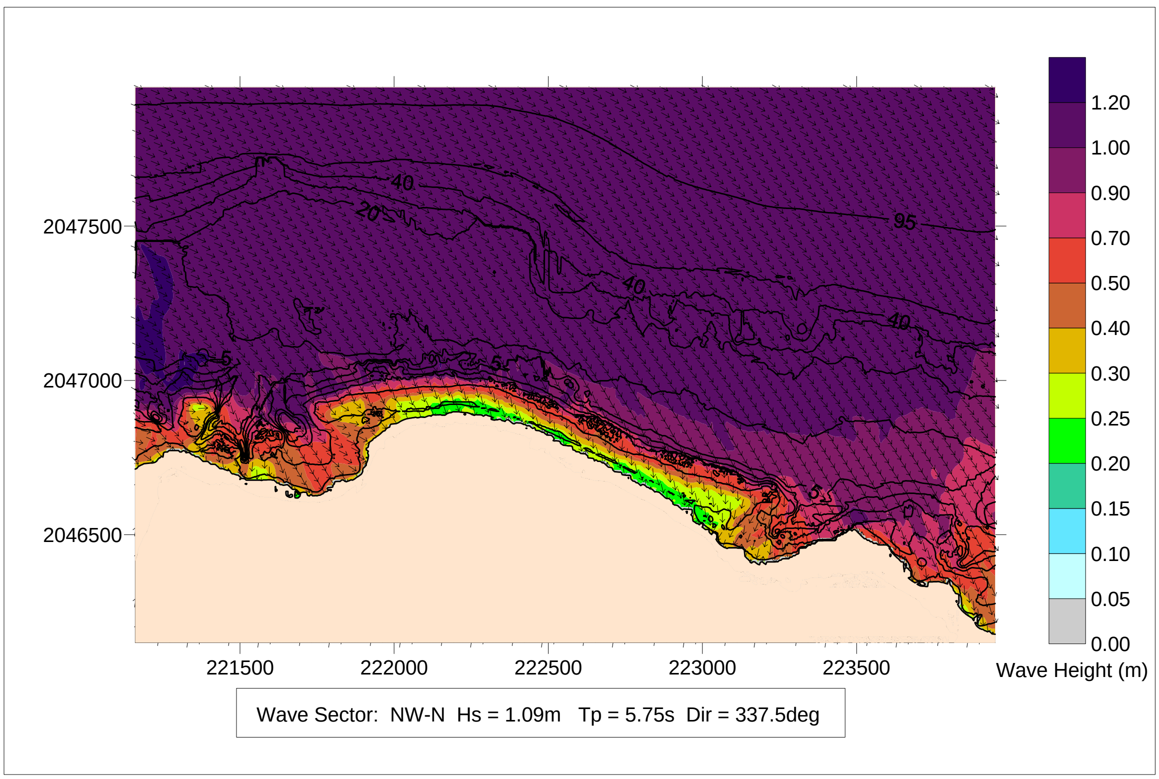


Figure 3.9 Waves in study area dues to NW-N deep water wave conditions

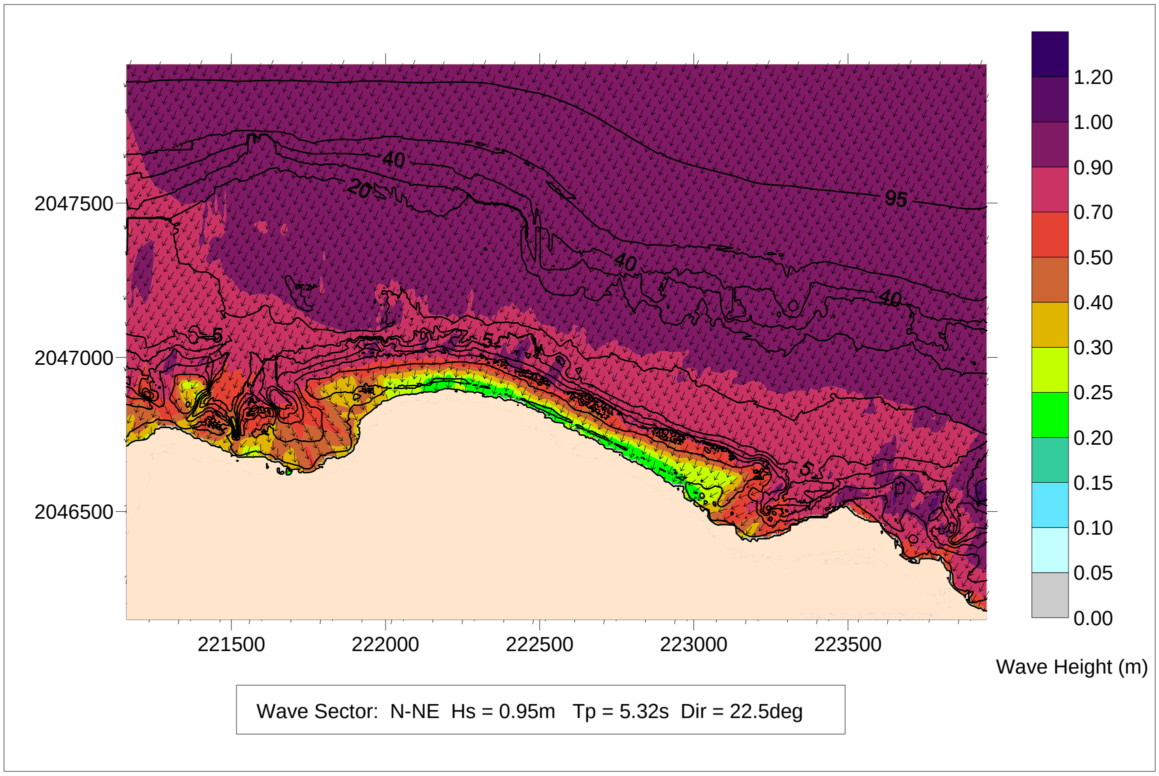


Figure 3.10 Waves in study area dues to N-NE deep water wave conditions

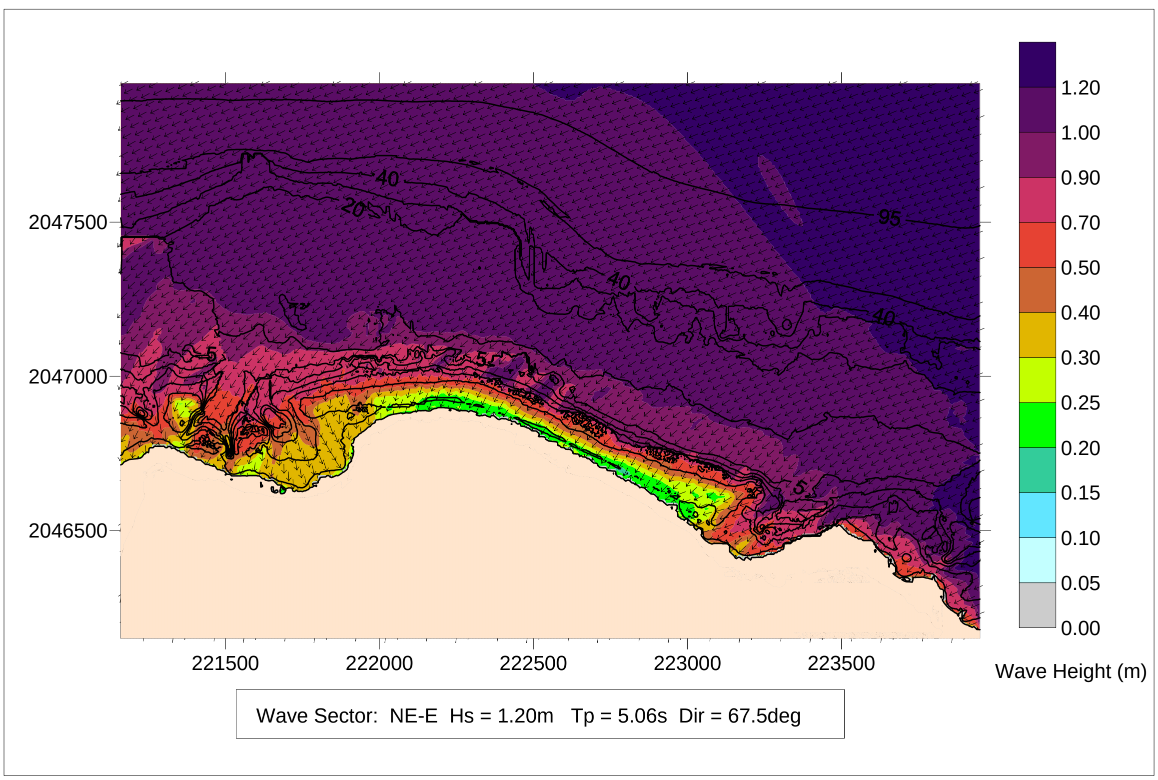


Figure 3.11 Waves in study area dues to W-NW deep water wave conditions

4. Sediment Transport and Storm Surge

4.1 Sediment Transport

An assessment of aerial photographs taken over the last century indicates that there is a high rate of sediment transport along the project site. The headland on the western end of the site (Bush Cay) was once detached, and was conjoined to the major land mass over the years as a result of sediment transport processes. This growth of the headland is as a result of sediments supplied from the reefs and lagoon being deposited in this location. The sediments on the seaward shore are primarily as a result of the abrasive action of waves on the coral reef, which is transported to the shore. While, sediments on the lagoon side of the shoreline are predominantly, those transported into the lagoon from the Martha Brae River. In order to properly design any development for this area, it is important to understand the prevailing processes that are responsible for mobilization of sediments both onto and off the beach. Generally, the characteristics of the existing sediments, waves and current regimes are responsible for the rate of sediment movement along the shoreline.

An investigation was carried out to assess the capacity of the prevailing waves to transport sand along the shoreline. This was done with the aid of a Danish Hydraulics Institute (DHI) one-dimensional sediment transport program called LITPACK. In order to capture the shoaling and refraction of waves, the deep water waves were first transformed into the nearshore zone using SWAN. The wave heights and directions at the inshore nodes were then input to LITPACK, which models the inshore waves up to the shoreline. During this transformation, the model is able to estimate the sediment transport rates from the incoming wave energy as the waves undergo the complex non-linear changes in the shallow nearshore zone.

The model was run along six cross-shore profiles strategically located on the shoreline, in order to capture the regions of accretion and erosion of sediment along the shore (see Figure 4.1). Profiles were oriented so that they were perpendicular to the shoreline with angles between 330^0 and 41^0 clockwise from north and extended seaward to a depth of 25m. The operational wave height and direction from the SWAN runs were used as input at the seaward end of the profiles to obtain estimates for annual sediment transport rates. The waves were split into four directional bins of 45^0, W-NW, NW-N, N-NE, and NE-E. For each directional bin, potential sediment transport was calculated based on the average wave height in the bin. The volume of yearly transport along each profile was calculated by summing the volumes transported from each direction, based on the percentage occurrence of waves.

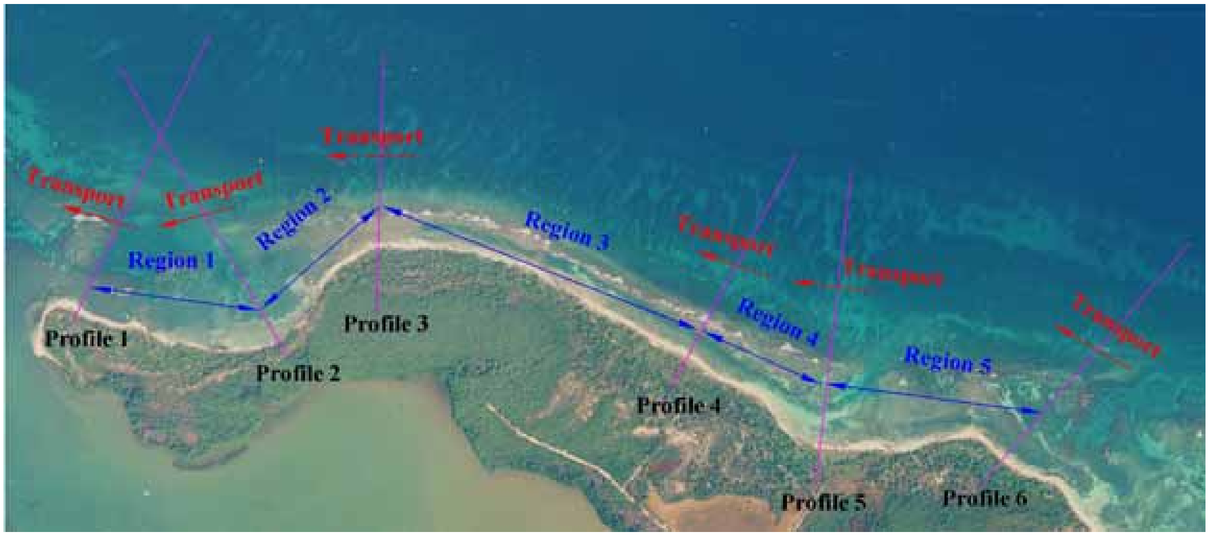


Figure 4.1 LITPACK Model Profile Lines and Transport Results (arrow lengths indicate relative transport rates)

The following tables (Table 4-1 to Table 4-6) show the results of the simulations for all six profiles, giving the potential sediment transport for each wave direction in cubic metres per year. The sediment transport values represent the alongshore sediment transport rates. Positive transport is towards the east, while negative transport is towards the west. Based on the results shown in the tables, the net transport for all profiles is in the westerly direction. Waves from the NE-E cause transport in the western direction for all profiles, and since this wave direction is overwhelmingly dominant, the resulting net transport is to the west. All profiles exhibit only a small quantity of sediment movement from west to east, and these events coincide with waves coming from the W-NW and sometimes from the NW-W. Although these wave directions cause transport in the easterly direction, they are infrequent and therefore easterly transport is minimal, relative to transport in the western direction.

The findings suggest that regions 2, 3 and 5 are areas of erosion while regions 1 and 4 are areas of accretion. The qualitative results of the LITPACK runs concur with the likely areas of accretion and erosion observed on the aerial photographs. It should be noted that, quantitatively, the sediment transport modelling resulted in the prediction of considerably large transport rates. It was found that the model is fairly sensitive to the angle at which waves approach the profile. A sensitivity analysis conducted on profile 6 reveal that an increase or decrease of the approach angle by 10^0 can result in a difference in transport rate of approximately 50,000 m^3/yr. Therefore, it is anticipated that accuracy of the predictions can be improved by decreasing the size of the directional bins, hence reducing the error of the angle of approach for the wave heights used in the simulation. Taking this into consideration, emphasis was placed on the qualitative finding of the analysis and, as a result, the potential and direction of sediment transport along the shoreline has been highlighted (See Figure 4.2).

Figure 4.2 Regions of erosion and accretion along the shoreline

Sediment transport rates were found to be lowest behind the reef system, which is believed to be as a result of reduced wave activity in this area. This area is also one of erosion, which can be attributed to an alongshore current. This current is caused by the collection of water behind the reef when waves that break move towards the gaps in the reef. The water movement aids the erosion process, as sediment is transported from behind the reef, with some deposition in the vicinity of the gaps.

Table 4-1 Sediment Transport Modelling Results for Profile 1

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.91	289.0	3968	3968	0
NW-N	2.5%	5.75	1.19	337.5	1.18	337.5	10905	10905	0
N-NE	12.3%	5.32	0.95	22.5	0.93	22.7	1939	1939	0
NE-E	82.8%	5.06	1.2	67.5	1.09	58.1	-120114	0	-120114
								16813	-120114
Net Transport									**-103301**

Table 4-2 Sediment Transport Modelling Results for Profile 2

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.95	292.5	13584	13584	0
NW-N	2.5%	5.75	1.19	337.5	1.18	337.1	-2960	0	-2960
N-NE	12.3%	5.32	0.95	22.5	0.95	22.6	-77191	0	-77191
NE-E	82.8%	5.06	1.2	67.5	1.12	61.1	-53007	0	-53007
								13584	-133158
Net Transport									**-119573**

Table 4-3 Sediment Transport Modelling Results for Profile 3

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.89	286.8	916	916	0
NW-N	2.5%	5.75	1.19	337.5	1.16	337.1	623	623	0
N-NE	12.3%	5.32	0.95	22.5	0.95	22.3	-2327	0	-2327
NE-E	82.8%	5.06	1.2	67.5	1.12	61.4	-31334	0	-31334
								1539	-33662
Net Transport									**-32123**

Table 4-4 Sediment Transport Modelling Results for Profile 4

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.76	274.2	458	458	0
NW-N	2.5%	5.75	1.19	337.5	1.14	334.9	1168	1168	0
N-NE	12.3%	5.32	0.95	22.5	0.95	22.3	776	776	0
NE-E	82.8%	5.06	1.2	67.5	1.15	63.3	-28723	0	-28723
								2402	-28723
Net Transport									**-26321**

Table 4-5 Sediment Transport Modelling Results for Profile 5

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.75	273.2	1526	1526	0
NW-N	2.5%	5.75	1.19	337.5	1.13	335.3	3739	3739	0
N-NE	12.3%	5.32	0.95	22.5	0.94	21.8	-8534	0	-8534
NE-E	82.8%	5.06	1.2	67.5	1.15	64.1	-203672	0	-203672
								5265	-212206
Net Transport									**-206940**

Table 4-6 Sediment Transport Modelling Results for Profile 6

			Offshore Wave Climate		Inshore Wave Climate		Sediment Transport		
	Percent Occurence	T (s)	H (m)	Direction (deg from N)	H (m)	Direction (deg from N)	Transport (m3/year)	Total East Transpost (m3/year)	Total West Transpost (m3/year)
W-NW	2.4%	5.11	0.95	292.5	0.76	275.6	4045	4045	0
NW-N	2.5%	5.75	1.19	337.5	0.98	349.5	8490	8490	0
N-NE	12.3%	5.32	0.95	22.5	0.95	22.6	15128	15128	0
NE-E	82.8%	5.06	1.2	67.5	1.20	67.6	-169727	0	-169727
								27663	-169727
Net Transport									**-142064**

4.2 Shoreline Stability

The sediment transport analysis conducted for the study area indicates that the shoreline is a dynamic one. The alongshore sediment transport essentially terminates at the western headland, and therefore downdrift impacts will be contained within the study area. On a whole, there appears to be a net deposition of sediment within the study area. Although there are regions of erosion along the shoreline within the project, it is a case where sediment eroded from one region is being deposited in another within the study area. In addition, the active sediment transport pathway is quite narrow due to the presence of the reefs and the absence of a dune. This means that the shoreline response to an episode of high water levels or waves may be greater than for a wide stable beach.

Due to the dynamic nature of the shoreline within the study area, infrastructure including walls, pools, etc. less than 15 metres from the shoreline should be constructed with adequate scour protection measures. It may be possible to reduce the dynamic nature of the shoreline within the study area by introducing groynes and other shoreline protection structures. These structures often have downdrift impacts, but in this case these impacts would not extend beyond the western end of the peninsula.

4.3 Storm Surge

Storm surge analyses were conducted along the six profiles used in the sediment transport modelling. In this case, extreme wave conditions were applied at the seaward end of the profile, at a depth of 200m. These conditions were transformed to the shoreline using a one-dimensional non-linear model, sBEACH, which computes wave transformation and water level increase in the nearshore zone. The results are shown in Figure 4.3 to Figure 4.8 for the 50-year storm wave. The 50-year wave conditions, the water level increase due to IBR, and the maximum tide of 0.3m, along with a projected Global Sea level rise of 0.25m in the next 50 years were used. The plot shows the seabed bathymetry and the variation in wave height and water level up to the shoreline. For any structure being designed, the most extreme wave height found from these figures, at the location of the proposed structure, should be used as the design criteria.

The figures indicate that structures along the shoreline will have to be designed for water levels as high as 3.7 metres and for a wave heights as high as 3.1m at a depth of 4m. The site is on average approximately 2m above MSL and as such the site is expected to flood in extreme storms. All habitable structures should be elevated to an acceptable height to avoid major flooding from the anticipated storm water levels and also, securely anchored to an adequate pile foundation. As a result, pile foundations for habitable structures should be designed to withstand all reasonable anticipated erosion, scour and loads resulting from a 50-year storm including wind and wave forces acting simultaneously with typical structural (live and dead) loads.

Storm surge computations have been conducted using return periods of 20 and 10 years, in addition to the 50 year return period previously presented. The results indicate that 10 year storm surge ranges up to 2.1 metres, and the 20 year, 2.8 metres. It should be noted that storm surge does not include wave run-up which could carry moving water at least 1.0

metres higher. As the ground elevation is rarely greater than 2.0 metres, this analysis suggests that the entire site would be inundated at even the 10 year return period. Normal set-back computations are based on the intersection of the storm surge level with the corresponding topographic contour. In this case, this is not possible. Instead, the setback distance should account for possible shoreline movements, and all permanent infrastructure should include suitable scour protection measures.

Table 7: Water levels due to 20 and 10 year storm events

Profiles	Water Level (m)	
	20 yr	10 yr
Profile 1	2.3	1.8
Profile 2	2.6	1.9
Profile 3	2.8	2.0
Profile 4	2.8	2.1
Profile 5	2.7	2.1
Profile 6	2.6	1.8

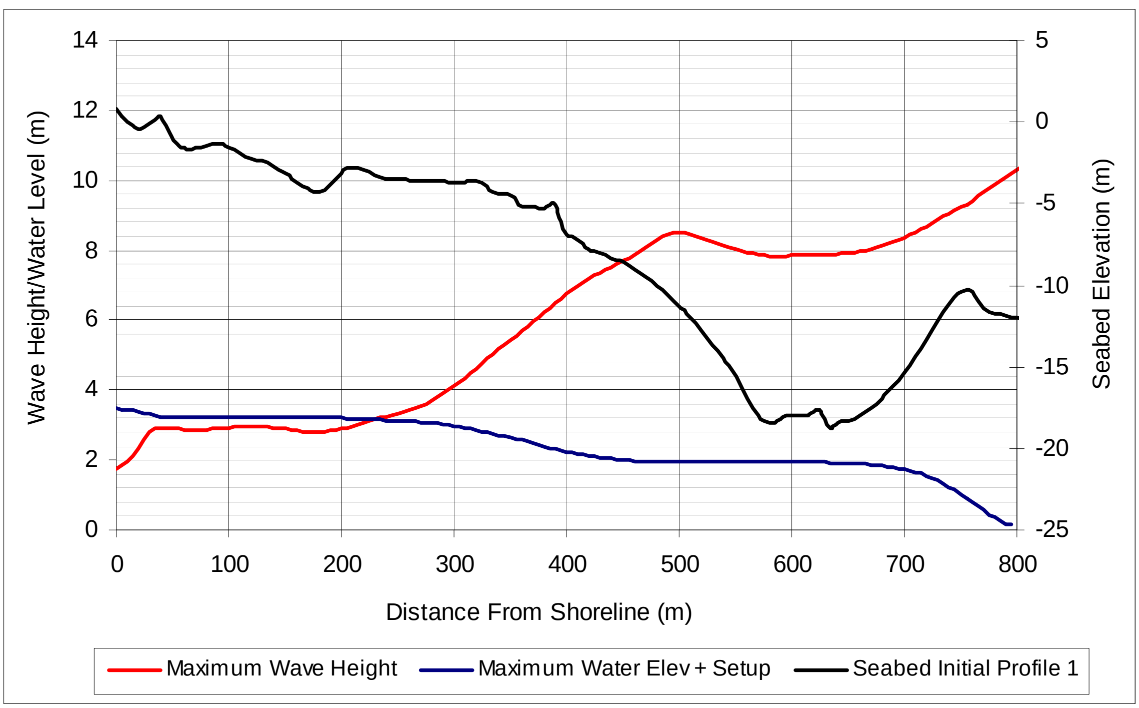


Figure 4.3 Wave heights and water levels along Profile 1

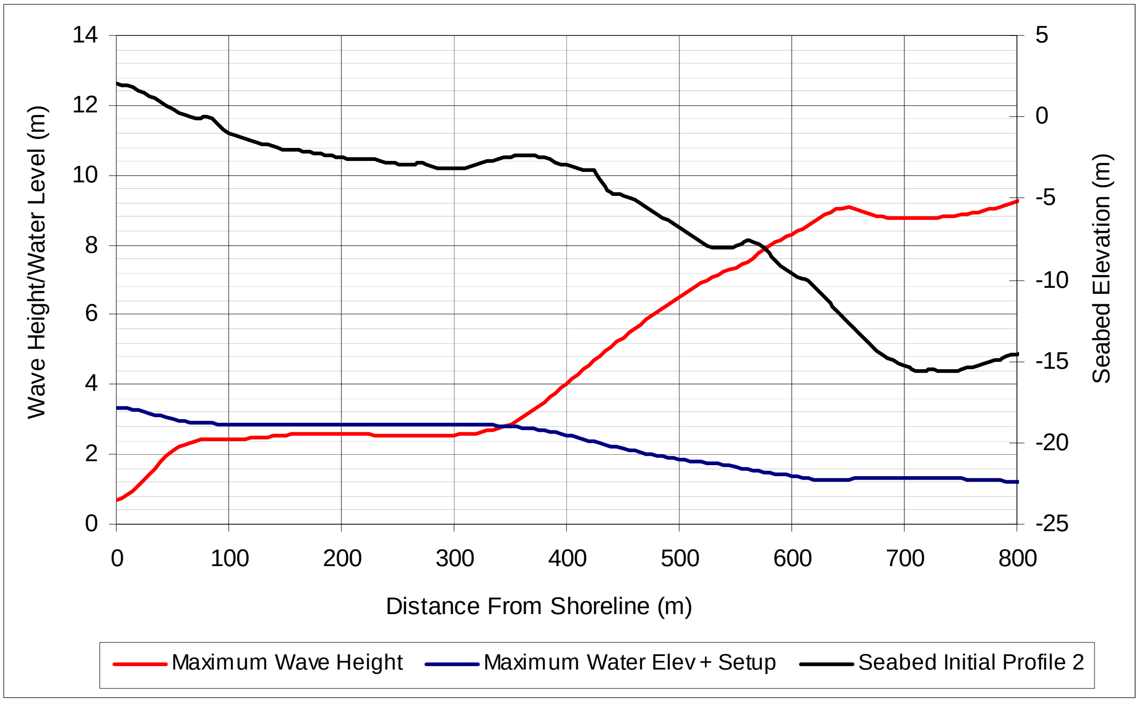


Figure 4.4 Wave heights and water levels along Profile 2

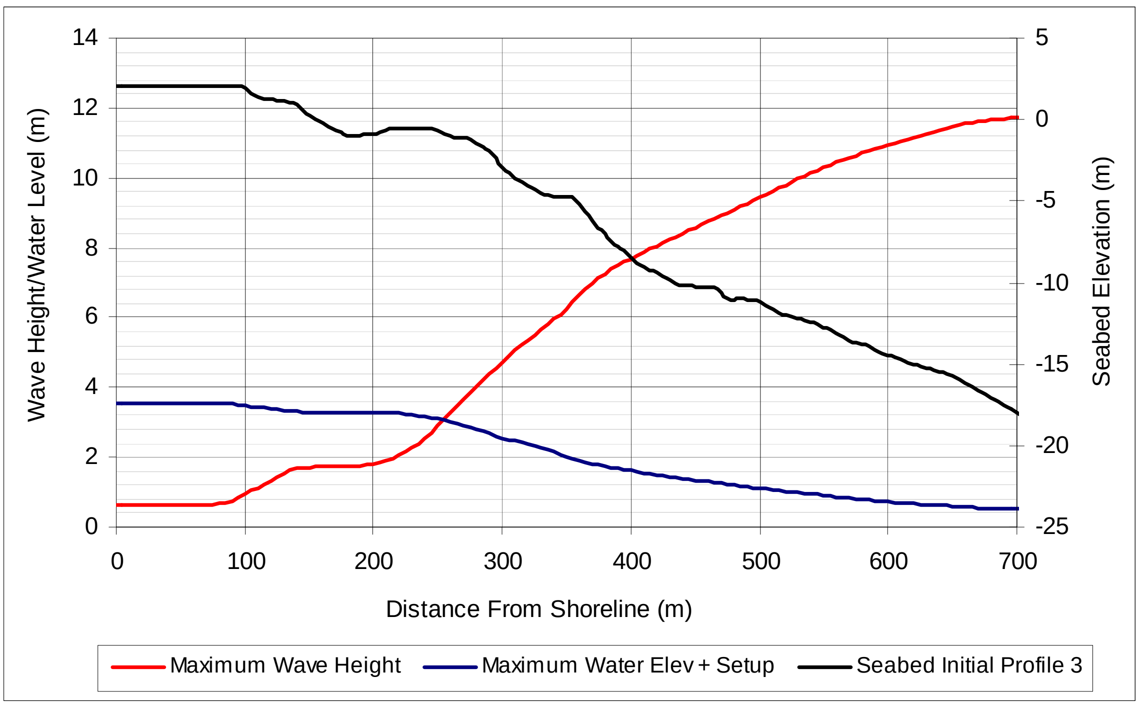


Figure 4.5 Wave heights and water levels along Profile 3

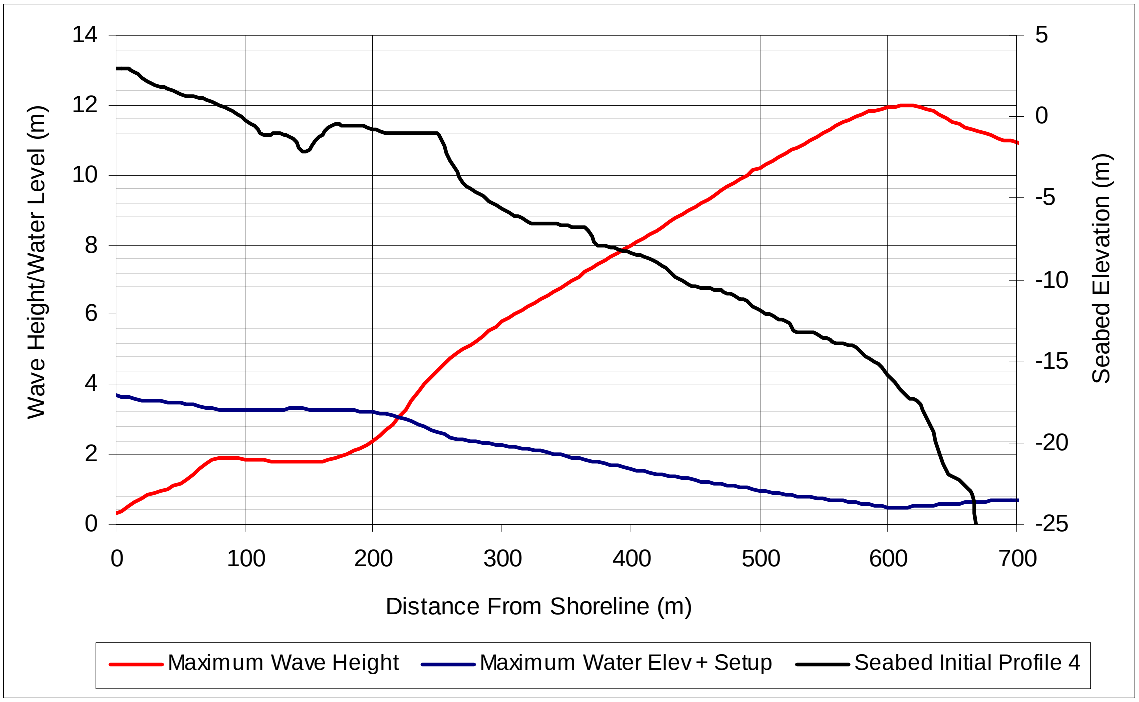


Figure 4.6 Wave heights and water levels along Profile 4

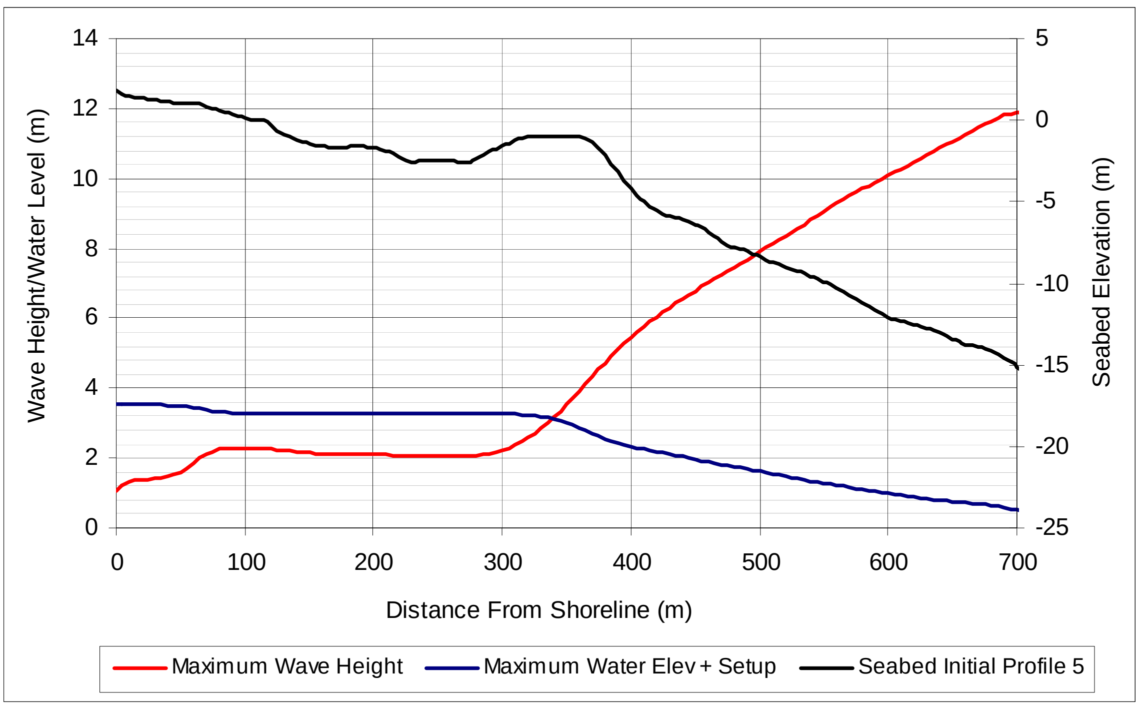


Figure 4.7 Wave heights and water levels along Profile 5

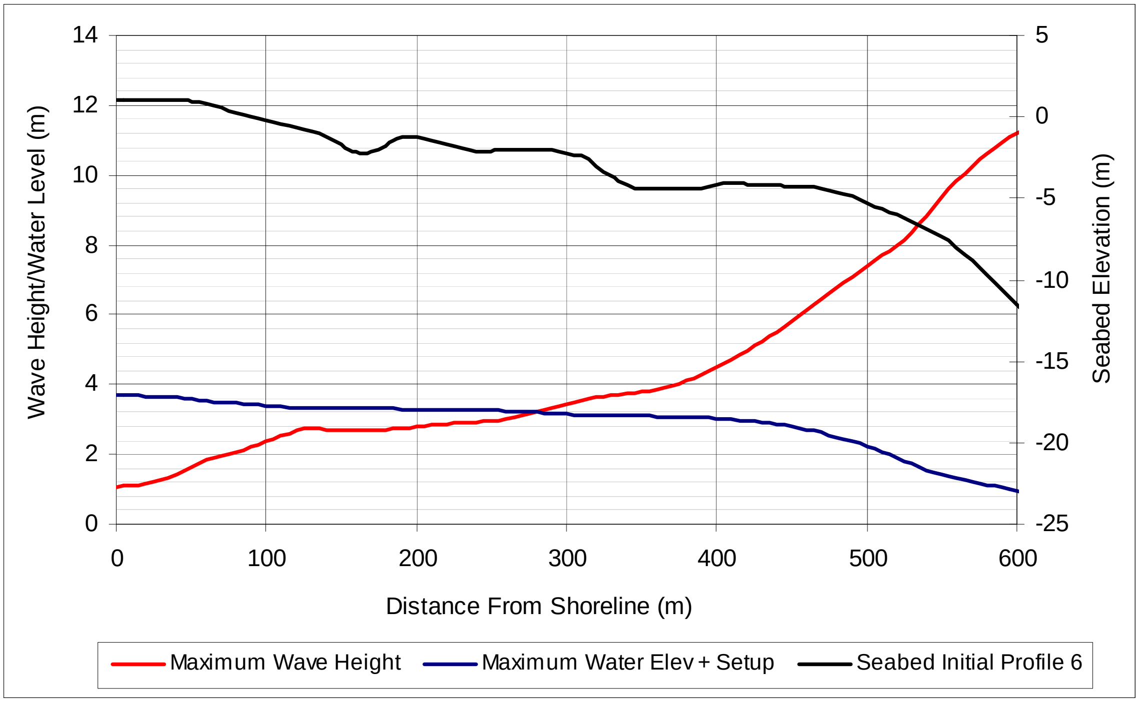


Figure 4.8 Wave heights and water levels along Profile 6

5. Installation and Effects of Pile system

5.1 Effects on currents

Coastal structures such as jetties and piled dwellings over the water can affect current patterns and can also reduce sediment yield. Beach changes result from sediment transport which is controlled by nearshore currents, originated by external forces such as wind and waves. Coastal currents in the nearshore zone are generated by both breaking waves inside the surf zone and by wind over an extended area of the coastal zone. The reef system running parallel to the shoreline of the project area contributes to a strong longshore current, as the water behind the reef resulting from waves breaking moves towards the gaps in the reef. This suggests that wave-induced longshore current near the surf zone is the dominant factor of sediment transport under day to day conditions. Longshore currents flow in one direction along the shore between the breakers and the shore. These currents may form circulatory patterns, leaving the shore in narrow rip currents and returning to shore as dispersed flow with the breaking waves. Generally, waves approach the shoreline, further contributing to the longshore current. The highest current velocities are typically near the water surface and nearer the breakers than the shore.

Construction of the over-water dwellings behind the reef will modify the flow pattern in the vicinity of each structure. They will change the existing current patterns and disrupt the alongshore currents, as they create a barrier to free water movement and sediment transport. It is recommended the spacing between piles and structures be maximized in order to minimize their negative effects on the currents and the surrounding environment on a whole. The spacing of piles affects the probability of scouring and deposition of sediments at the piling foundation. When modified flow pattern caused by near-pile vortices, along with, the contraction of streamlines leads to a greater capacity for transport away from the base of the pile than the rate at which material is supplied, scour will occur. Scour may reduce structural stability, and thus lead to their failure. Conversely, where the transport capacity is less than the rate of supply because of the frictional effects of the piles, deposition will occur. The resultant changes in the stream bed will likely modify the flow pattern until a new equilibrium between transport capacity and supply is again achieved at every point on the seabed.

Further study and evaluation is required to ascertain long term effects of piles on water circulation, however, initial indications are that there may be an accumulation of sediment on the lee side of the pile structures. The spacing of piles is a major consideration for water circulation, as it affects both currents and waves.

It is recommended that the reduction in currents and waves be not greater than 10 percent. If the assumption is made that the portion of the power transmitted through the pile structure is proportional to the portion of gaps between the piles, then

$$\frac{H_T}{H_i} = \sqrt{\frac{P_T}{P_i}} = \sqrt{\frac{b}{D+b}}$$

Where, H_T – Transmitted wave height
H_i – Incident wave height
P_T – Transmitted wave power
P_i – Incident wave power
D – Pile diameter
b – Distance between piles

Setting this equation equal to 0.9 (i.e. a 10% reduction) produces a minimum pile spacing of 5 pile diameters. Larger pile spacings will result in less obstruction to water flows, however, larger pile spacings typically require larger diameter piles, longer deck spans, and deeper deck sections.

5.2 Options for Pile installation.

The installation process and method of installations are equally important factors as of the design process of pile foundations. In this section the two main types of pile installation methods; installation by pile hammer and boring by mechanical auger will be discussed. In order to avoid damages to the piles, installation methods and installation equipment should be carefully selected. The type of piles selected should meet the specific needs of the structure, site conditions and budget. There are variety of materials and shapes of pile that can be utilised.

- Steel
    - H-Pile
    - Pipe (open-end or closed-end)
    - Tapered
    - Shell (mandrel driven)
    - Sheet Pile
- Concrete
    - Square
    - Octagonal
    - Cylinder
    - Sheet Pile
- Timber
- Composite piles that combine pile types (e.g., a concrete pile with a steel tip extension

5.2.1 Pile-hammer (driven piles)

There are a number of methods of pile driving installation which can be considered, these include dropping weight, explosion, vibration, jacking and jetting. When employing this method of installation the following factors should be taken in to consideration:

- the size and the weight of the pile

- the driving resistance which has to be overcome to achieve the design penetration
- the available space and head room on the site
- the availability of cranes and
- the noise restrictions which may be in force in the locality.

Driven piles easily adapt to variable site conditions to achieve uniform minimum capacity with high reliability, thus eliminating uncertainty due to site variability. Due to the fact that, they are normally driven to a blow count to assure the desired minimum capacity, pile lengths may vary when subsurface conditions are not uniform. As a result, piles may either be cut-off to shorten their length or spliced to extend their length. Splice designs usually meet or exceed the strength of the pile itself.

5.2.2 Boring

The equipment comprises of a mobile base carrier fitted with a hollow-stemmed flight auger, which is rotated into the ground to the required depth of pilling. To form the pile, concrete is placed through the flight auger as it is withdrawn from the ground. The method is especially effective on soft ground and enables the installation of piles that are able to penetrate a multitude of soil conditions. However, for successful operation of rotary auger the soil must be self-supporting and reasonably free of tree roots, cobbles, and boulders. During the process, little soil is brought upwards by the auger in order to maintain lateral stresses in the soil and that voiding or excessive loosening of the soil is minimised.

5.2.3 Comparison of Installation methods

Driven piles are more suited for marine and other near shore applications than bored piles. There are no special casings required for driven piles and there are no delays related to the curing of concrete. Piles driven through water can be used immediately, allowing construction to proceed in a timely manner. For piers or piled dwellings, driven piles can be quickly incorporated into a usable structure allowing the structure itself to be used as the work platform for the installation of succeeding. To minimize disturbance in wetlands or allow work over water, driven piles can be used to construct temporary trestles and allow for easy extraction at the end.

Bored piles process can remove soil and result in considerable subsidence, which can undermine the support of adjacent structures and cause excessive deformations, both of which can result in structural problems. Considering the existing site and the proposed development, drilling for cast-in-place piles is not a very practical option and the process could also relief soil pressures and reduces unit shaft resistances. The construction of the “Bora Bora” style clusters will require pile groups. The use of drilled piles could result in:

- the removal of soil, consequently, affecting water quality and habitat adversely,
- the loosening and weaken the soil structure,
- the reduction of the capacity of previously installed piles.

Employing the method of driven piles generally improves the soil density and the capacity of previously driven piles. In groups, driven production piles usually have a higher capacity than the test pile, while, drilled production piles often have a lower capacity than the test pile.

Thus, driven piles generally have higher capacities than other pile types of the same diameter and length. For structural and environmental purposes, it is recommended that the choice of pile installation be the Driven Pile Method.

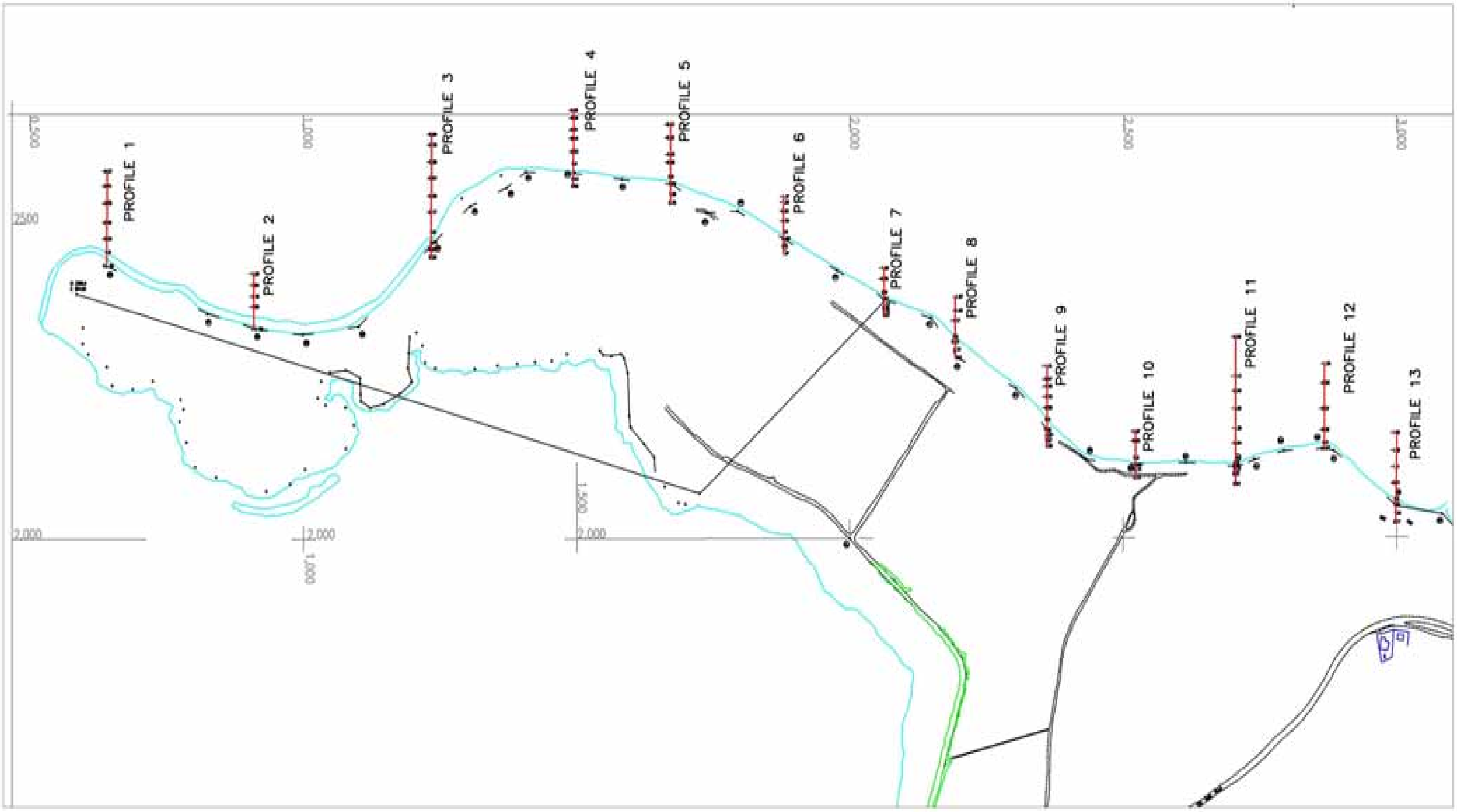
6. Conclusion

The sediment transport analysis shows that the project area has a dynamic shoreline, with both regions of accretion and erosion. The alongshore sediment transport essentially terminates at the western headland (a region of accretion), effectively containing any 'downdrift' impacts to within the study area. Given the dynamic nature of the shoreline within the study area, infrastructure placed less than 15 metres from the present shoreline should be constructed with adequate scour protection measures. The stability of the shoreline may be improved through the introduction of shoreline protection structures, such as groynes. These structures often have 'downdrift' impacts, but in this case these impacts would not extend beyond the western end of the peninsula.

Storm surge analyses were conducted for storm events with return periods of 10, 20 and 50 years. The maximum water levels resulting from these storm events ranged from 2.1 metres for the 10 year storm to 3.7 metres for the 50year storm. These figures indicate that the site will be inundated even for a storm with a return period of only 10 years. Therefore, buildings should be elevated to an acceptable height and securely anchored to an adequate pile foundation, in order to avoid substantial flooding from the anticipated storm water levels.

Piles driven along the shoreline in order to create over water dwellings can significantly reduce currents and waves heights to the extent that they have a negative impact on the environment. To avoid this, it is recommended that pile spacing be at a minimum of 5 times the pile diameter, which translates into approximately 10 percent reduction in waves and currents.

Rip currents pose a safety issue for users of the beach, as they can be swept offshore by the force of the current. It is recommended that designated swimming areas be identified along the shoreline where the swimming hazard posed by rip-currents is non-existent. As an alternative, artificial reefs could be created in the reef gaps to reduce the velocity of the rip currents. Re-profiling of the shoreline with the intention of reducing longshore currents and in turn reducing the rip currents is also another possible option. A final option is to construct deepened swimming areas along with cross-shore barriers to slow the alongshore current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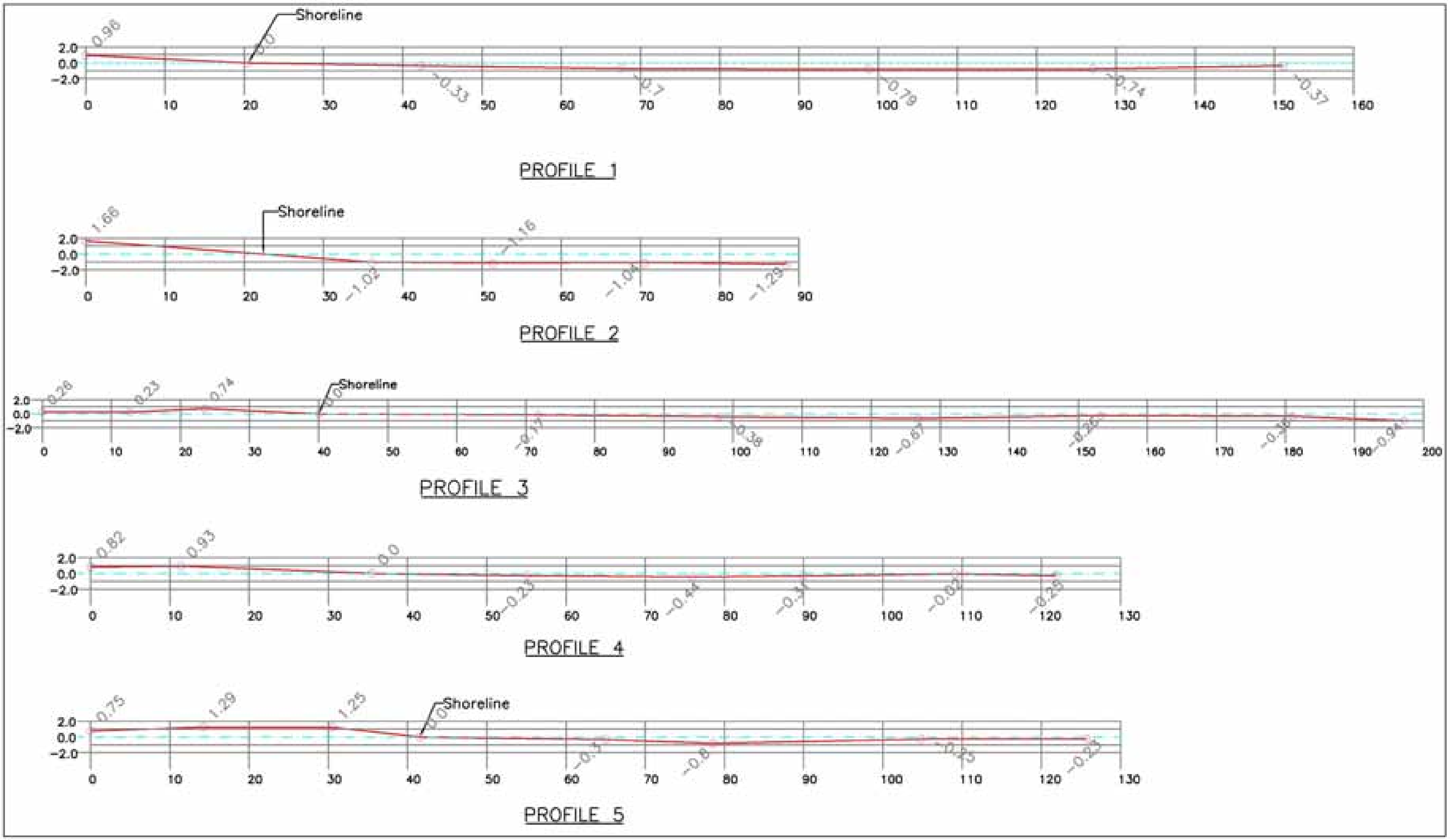
PROFILE 13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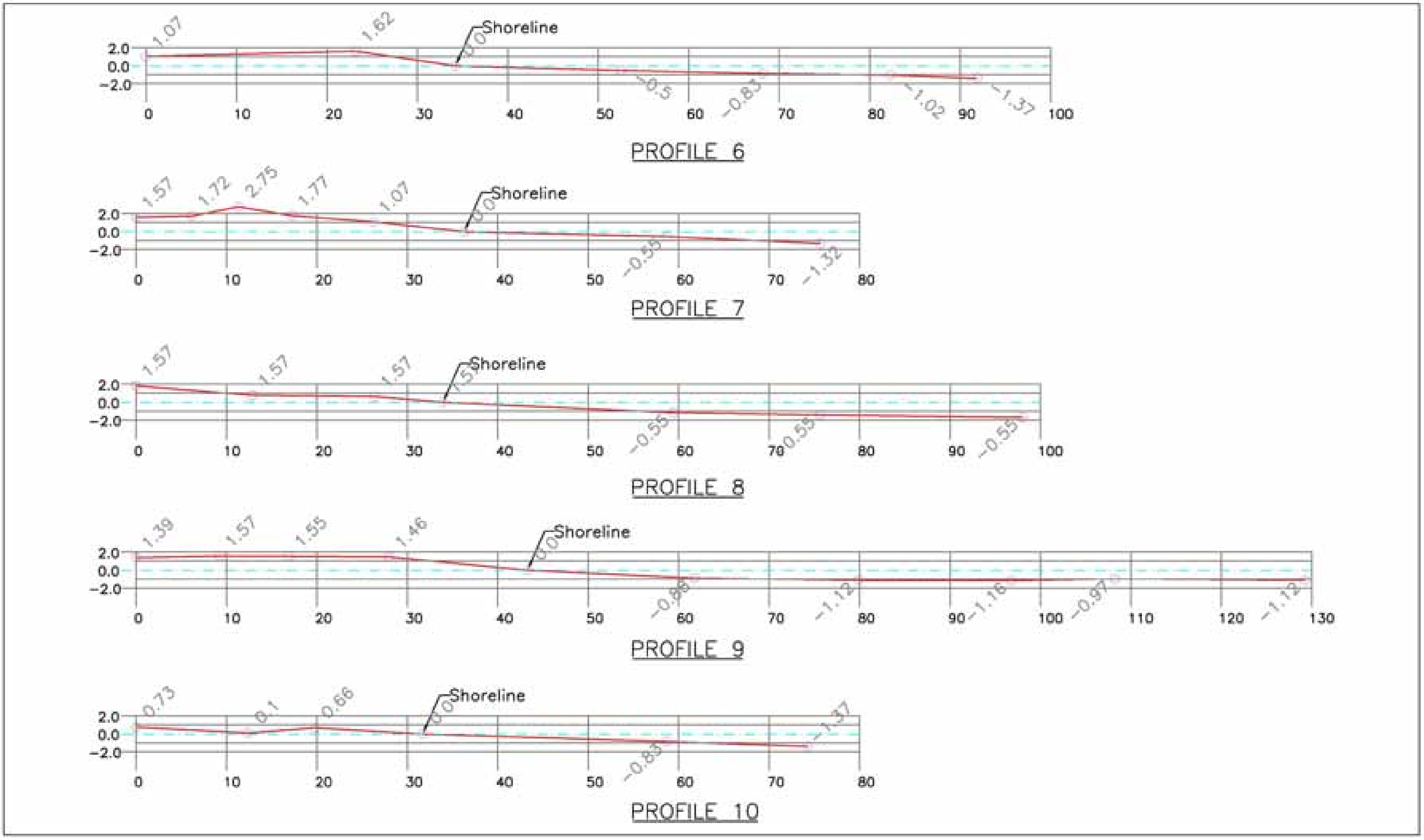
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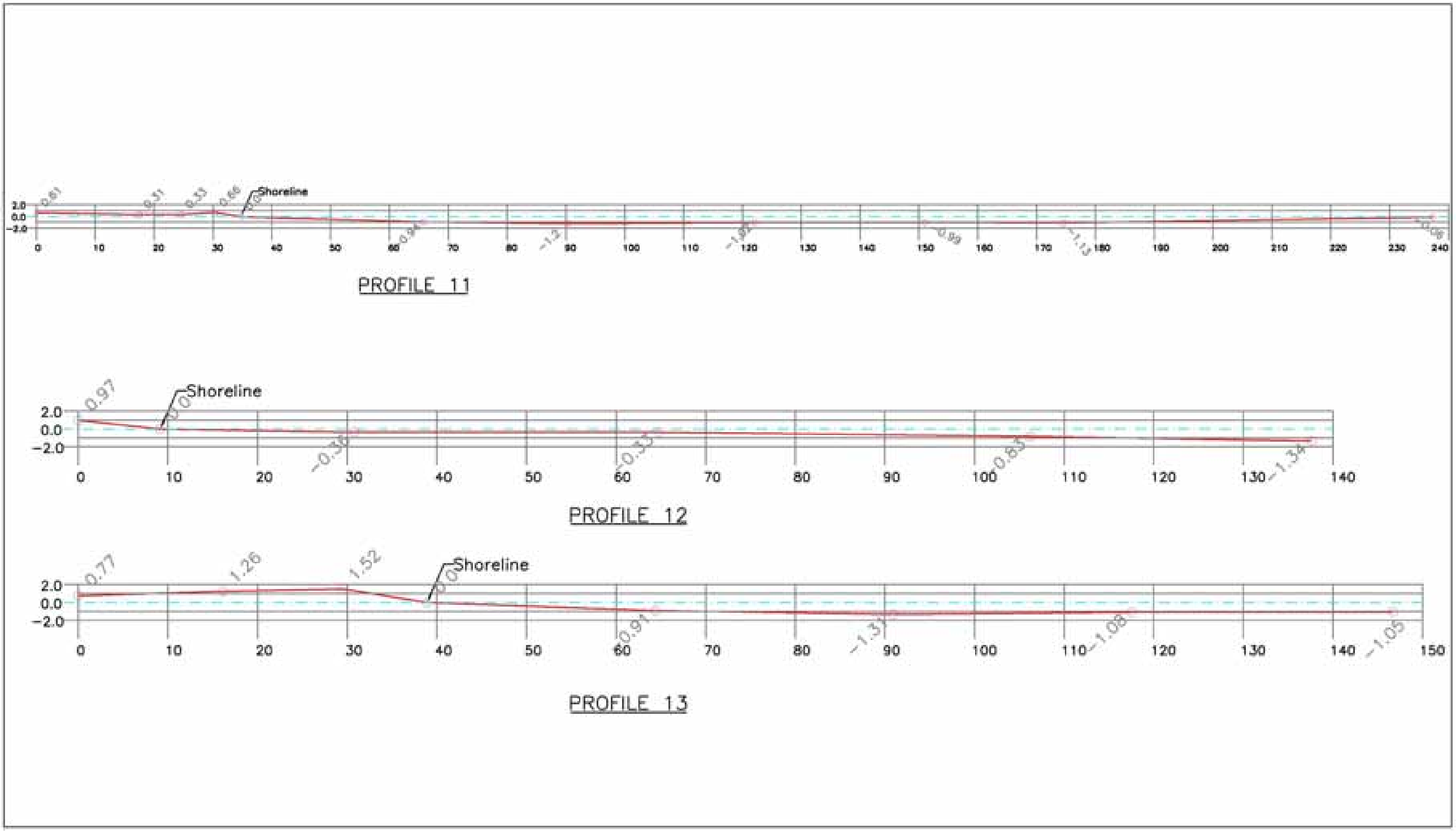
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