Preliminary Engineering Design Report

for

NMIA END OF RUNWAY SHORELINE PROTECTION AND REHABILITATION WORKS

PREPARED FOR

AND

PREPARED BY

MARCH 2011

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Revision Notes

Date	March 29, 2011			
Prepared by:	CJ/MH/KF			
Reviewed by:	CB			
Approved by:	CB			
Notes:	Prepared for review and submission to NWA			
Comments:	Further discussions on extent of works from vulnerability assessment required.			

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Prepared By:
CEAC Solutions Co. Ltd.
20 Windsor Avenue, Kgn 5
www.ceacsolutions.com

Executive Summary

Introduction

Background

The historical development and vulnerability of Palisadoes is well known. The gradual formation of the Tombolo over thousands of years and the actual splitting of the Palisadoes by a severe historical hurricane in 1722 is documented. The recent vulnerability of the Palisadoes has also been very evident with a spate of hurricanes in the last decade. Namely: Dean (2007); Dennis (2007); Ivan (2004) (See Plate 3.1) and Iris (2001). Historically, both a category 1 and 2 hurricane went through the Palisadoes in 1880 and 1886. With the redevelopment of The Norman International Airport as per its 20 Year Master Plan with recent expansions works the importance of reducing the associated vulnerability is critical. The timeless is also a worthwhile factor for consideration given the implementation Of the Palisadoes Shoreline Protection Works.

Overall, the NMIA is both nationally important and vulnerable to hurricane wave attacks. The protection of the associated shoreline is therefore critical to safeguard its contribution to the nation building.

Design Criteria and Parameters

The design criteria adopted for this design had to address a number of coastal and meteorological phenomena both individually and in certain worst case scenarios of combinations. The design project life adopted for this project is 50 years. A design frequency of remote chance of failure and a return period of 100 Years was adopted for this design, as per the recommendations of CIRIA (The Rock Manual, 2006). Functional Requirements were also considered. Overtopping of the revetment therefore has to minimized to acceptable levels to allow safe passage in minor storms (i.e. <10 Year Return Period) and limited or no damage to the road in the design storm event (i.e. <100 Year Return Period).

Airport Runway Approach Angle Criterion

The runway approach angle is the angle between the airplane's flight path and the runway. It was important for the purposes of this report to define that angle and to ensure that the coastal protection structure was not exceeding an elevation that would pose a threat to aircrafts approaching the runway for landing or taking off from the runway. The angle of approach for the aircraft to the runway is 2 degrees as communicated by NMIA representative. The design of the revetment and any associated structure will have to conform to elevations below elevations that may pose a danger to normal landing or taking off of the planes from the runway.

Data Collection

A topographic survey of the project area was conducted from the shoreline to the road for 1000m stretch of shoreline. The survey was conducted by Gordon and Company Ltd (Commissioned Land Surveyors). The extents of the survey area were offset at 500m to the left and to the right of the end of the runway.

Bathymetric surveys could not be conducted with the required time period due to rough sea conditions. The bathymetric data for the site was substituted and taken from the British Admiralty charts 456 and 454 for the area. The data is sufficient for this study in light of the information gleaned from the charts.

Surface sediment samples were recovered from the project area at eight locations along the beach/shoreline. Grain size analysis was done using the unified classification which is widely used for classification of granular material. The results showed the sediment sizes varied from coarse sands to gravel and indicates the operational wave climate is more aggressive on the eastern end of the project site.

Vulnerability Study

Wave climate study indicate that the 100 Year Return Period waves are more severe from Easterly and East south East waves. 4 to 6 metres wave are expected 500 metres offshore and the project area is an area of focusing of waves from offshore.

Long-term Vulnerability

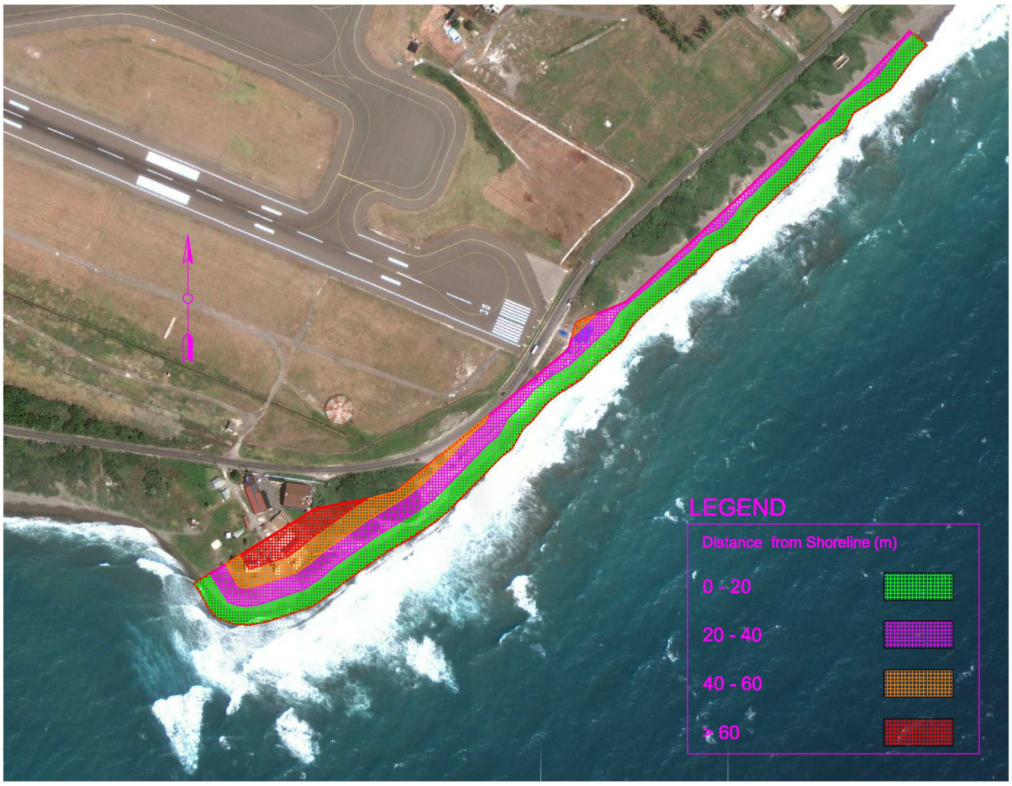
Long term and short term erosion trends and or impacts were investigated from dated aerial photography and secondly, the global sea level rise component was estimated to determine the erosion that was due to chronic global trends versus event based erosion events (i.e. hurricanes and swell events). Long-term aerial photograph and satellite analysis from 1968 to 2010 indicate an overall erosion trend in the project area of 1.4 to 23.5 metres over the 42 year period. Global Sea Level Rise was also determined to be responsible for approximately 57% to 100% of observed erosion. It therefore suggests that the shoreline will continue to erode is left unprotected, in light of continued increases in sea levels.

Estimated Short-term Vulnerability

It was considered necessary to determine the erosion hazard of the 1.0km of shoreline considered to the 100 year return storm event. This was due to the increased number of extreme storms events over the past 70 years as well as anecdotal information pointing to erosion taking place on the beach during storm events.

The results of this analysis revealed:

1. The entire stretch of shoreline is vulnerable to erosion varying from 10 to 84 metres for the 100 Year Event
2. The area to the southwest of the runway is most vulnerable to erosion due to a 100 yr storm
3. The section of the main road immediately to the south if the runway is susceptible to undermining due to erosion of the shoreline



Given that the road and the NMIA are of national importance, the associated shoreline needs to be protected from the attacks of waves being generated by more intense and more frequent hurricanes.

Proposed Shoreline Protection Works

The Van der Meer Stability Equation as per the Rock Manual (2006) and Kamphius (2000) was utilized to size the armour stones. The equation is valid for the estimation of the stability of armour stone for the trunk, head and toe of coastal structures, for a wide range of wave conditions. The design calculations revealed that a range of stone sizes from 4 Tonnes to 13 Tonnes are required to resist the 100 Year Return Period Design wave conditions. These stone sizes are comparable to the existing permanent works taking place on the Palisadoes.

Rapid Environmental Assessment

Three communities of plants were found within the project area (on the coastline and on the dunes). One endemic was found on the dunes closer to the lighthouse and away from the construction area, this endemic is the cactus O. tuna (tuna). The endemic identified presently. The construction of the revetment will impact some recoverable flora, specifically runners on the seaward face of the dunes. The disturbance is expected to be minimal and will not impact the one endemic identified in the area.

Conclusions and Recommendations

Conclusions

The following conclusions can be drawn from the study conducted to date:

1. The wave refraction analysis clearly indicates that the project shoreline is most vulnerable to hurricane waves from the east and ESE. In both scenarios, 5 to 6 metre waves are expected some 500 metres offshore and 2.5 metres waves are expected at the shoreline. The waves appear to be focused from offshore bathymetric features such as submerged cays or mounds. The 100 year conditions from S and SSE waves are less severe.
2. Satellite imagery and aerial photograph analysis of images from 1968 to 2010 indicate a long-term trend of erosion of 1.7 to 23.0 metres in the last 42 years. Global Sea Level rise analysis (using Bruun Rule) indicates that 57% to 100% of this can be explained by sea level rise.
3. Short-term vulnerability for the shoreline in the 100 Year event, indicates that erosion losses of can range from 10 to 83 metres from a single event. This estimate is supported by observations after Hurricane Ivan which was a 1 in 90 year hurricane.
4. The proposed works are estimated to cost US $16,393,373.97
5. The occurrence of endemics within the project is very low. The implementation of the proposed protection structures should cause minimal disturbance to the dominant flora and fauna on the shoreline.

Recommendations

The following are our recommendations:

1. A revetment 1,105 metres long, with 4 to 13 Tonne armour stone and a crest elevation of 6.4 metres above Mean Sea Level is necessary to resist the 100 Year Return Period wave conditions at the eastern end of runway for the Norman Manley International Airport. See proposed works in drawings:
    a. CEAC-2011-02-001
    b. CEAC-2011-02-002
    c. CEAC-2011-02-003
    d. CEAC-2011-02-004

   The section of revetment from the end of the runway to the Plumb Point Lighthouse is most in need of this level of protection. Whilst the eastern end (0+740 to 1+105) is just as vulnerable the consequences of damage and the likely lost of shoreline is expected to be less. Consideration can be given in the final design stage to reducing the crest elevation to 5.49m for cost savings in this eastern area.
2. Consideration should be given in the final design stage to raising the road level to a minimum of 2.3m above Mean Sea Level. Storm water drainage improvements should also be made to redirect storm water flows from the existing low point into the mangroves.
3. Pending successful project financing the following necessary activities are envisaged:
    a. Further detailed site specific wave modeling for 50 and 100 Year Return Period wave climate. Additional structural design and overtopping analysis with a view to reduce crest elevation should also be undertaken
    b. Drainage assessment of possible options Namely, Option 1: Keeping as-is and improving or Option 2: Lifting road and re-directing to mangroves.
    c. Road lifting feasibility and discussions with NMIA to finalize acceptability criterion.
    d. Vegetation mapping and replanting
    e. NEPA permitting

1 Introduction

1.1 Background

The historical development and vulnerability of Palisadoes is well known. It has been investigated by several authors, namely Robinson, E. , et al (2004 and 2005) and OAS/CDMP Project (1999). Robinson in his paper highlights the gradual formation of the Tombolo and the actual splitting of the Palisadoes by a severe historical hurricane in 1722. See Figure 1.1 and Figure 1.2 below. The recent vulnerability of the Palisadoes has also been very evident with a spate of hurricanes in the last decade. Namely: Dean (2007); Dennis (2007); Ivan (2004) (See Plate 3.1) and Iris (2001). Historically, both a category 1 and 2 hurricane went through the Palisadoes in 1880 and 1886. See Figure 1.3

The Norman International Airport is a 2,710 metres runway at headings of 300 and 120 degrees. In 2008 it handled 1.7 Million Passenger movements. This is in comparison to 3.4 Million for Sangster International Airport in Montego Bay. NMIA, therefore contributed approximately 33% of the airlift capacity. NMIA is considered a major economic catalyst, contributing approximately 5.6% of the GDP. The recent expansions works of the airport are predicated on a 2004 Master Plan that has an estimated construction cost of USD 112 Million for Phase 1 and 2.

Overall, the NMIA is both nationally important and vulnerable to hurricane wave attacks. The protection of the associated shoreline is therefore critical to safeguard its contribution to the nation building.

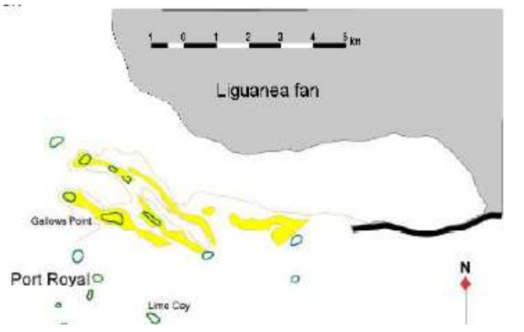


Figure 1.1 - Speculative cays an dformation of the Palisadoes, some 4,000 years ago

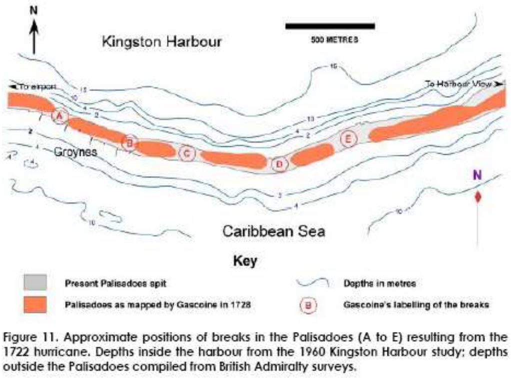


Figure 1.2 - 1722 mapped cays of Palisadoes by Gasgoine

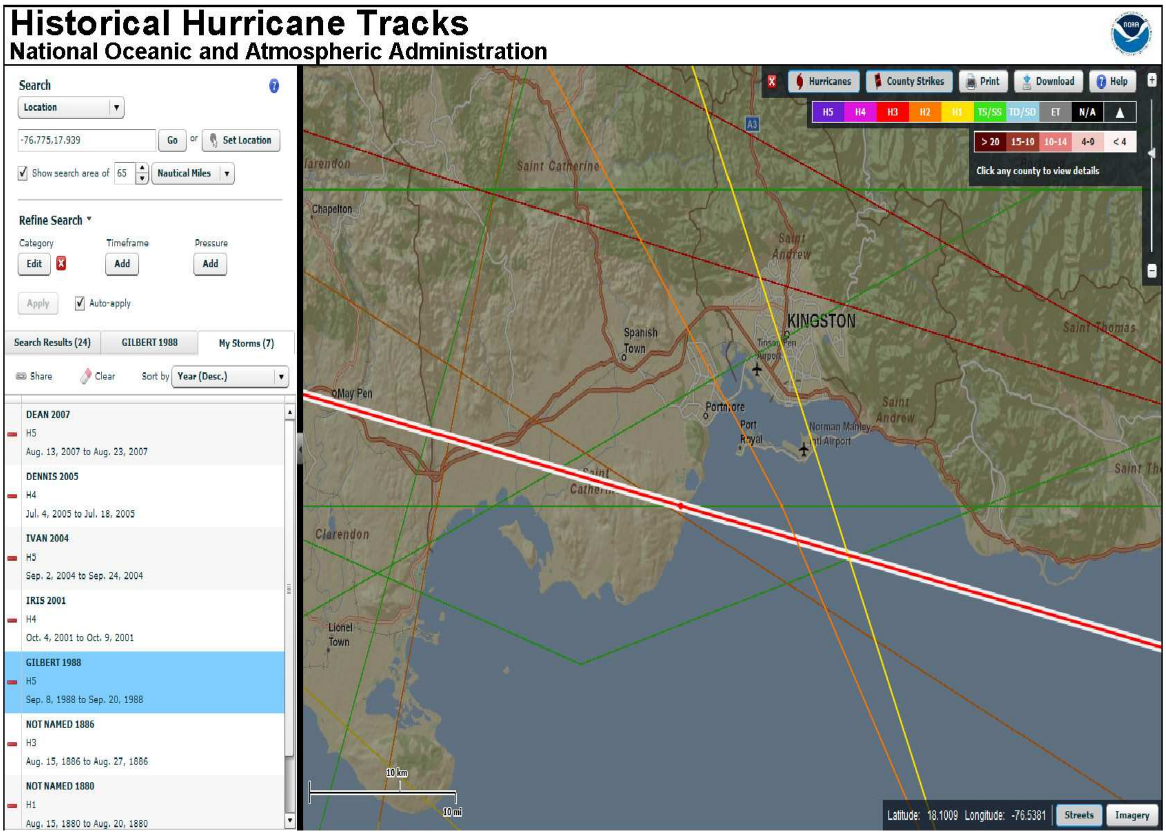


Figure 1.3 – NOAA hurricane tracks within 65 kilometres of NMIA

1.2 Approved Scope of works

1. Conduct autonomous bathymetric survey from 0.9 metres depth to 20 metres water depth to facilitate wave modeling. Additionally, topographic surveys of the dunes for approximately 1,000 metres will be undertaken.
2. Conduct preliminary 100 Year return period wave modelling to define the vulnerable stretch of shoreline and preliminary structural design to facilitate conservative budget estimates.
3. Conduct a Rapid Environmental Assessment and define the requirements of detail studies that will be necessary
4. Prepare preliminary engineering drawing and budget estimates, and
5. Outline the necessary final design work to be undertaken.

1.3 Design Criteria and Parameters

The design criteria adopted for this design had to address a number of coastal and meteorological phenomena both individually and in certain worst case scenarios of combinations.

1.3.1 Design life

The design project life adopted for this project is 50 years. The assumptions are assumed to hold true during this interval and the revetment is expected to maintain a useful service condition, providing that the design conditions are not exceeded.

1.3.2 Return Period Interval: Wind and Wave Intensities

The design of engineering infrastructure requires that the owner and client appreciate the robustness of the design. The design of coastal works are usually undertaken to withstand the 50 to 100 Year storm conditions. A design frequency of remote chance of failure and **a return period of 100 Years** was adopted for this design, as per the recommendations of CIRIA (The Rock Manual, 2006).

Table 1.1 – Design Criteria recommendation for a design life of 30 to 100 Years

Subject of evaluation	Event frequency and return period (years)				
	Frequent	Probable	Occasional	Remote	Improbable
Permanent structure design (lifetime 30–100 years)	0.1	1	10	100	1000
	In addition, if the structure is designed to be optimal, or if its performance is to be changed significantly, at other annual frequencies of event occurrence, information should be given and evaluation prepared for those events as well				
Design for temporary state during construction (duration: a few months or years)	0.01	0.1	1	10	100

This approach is consistent with the approach taken with both local and regional coastal defense projects. This criterion will be applied both individually to waves and winds for the project area in combination with storm surge

1.3.3 Functional Requirements

The Revetment proposed herein is immediately adjacent to a road. Overtopping of the revetment therefore has to minimized to acceptable levels to allow safe passage in minor storms (i.e. <10 Year Return Period) and limited or no damage to the road in the design storm event (i.e. <100 Year Return Period).

The Technical Advisory Committee on Flood Defence (Netherlands) has published a manual that addresses this issue, Wave Run-up and Wave Overtopping of Dikes (2002). Likewise H. R. Wallingford (1999), Wave Overtopping of Seawalls-Design and Assessment Manual presents clear guidance to professionals in this area.

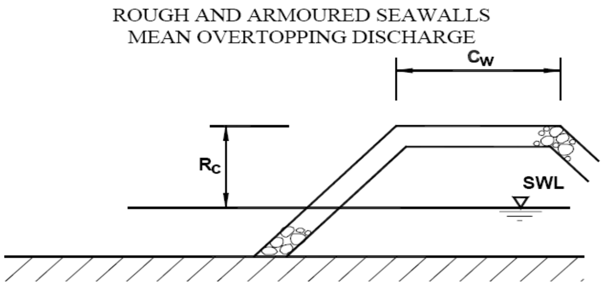


Figure 1.4 – Conceptual cross section of revetment used in estimating overtopping (H. R. Wallingford, 1999)

The revetment crest elevation will be designed to the following criteria:

1. Mean discharge in 10 Year Return Period = 0.05 cubic metres per metre
2. Mean discharge in 100 Year Return Period = 0.20 cubic metres per metre

See Table 1.2 for reference.

Table 1.2 - Overtopping design criteria (H. R. Wallingford, 1999)

TOLERABLE MEAN DISCHARGES ($m^3/s/m$)

Buildings :-					
		Q	<	$1x10^{-6}$	No damage
$1x10^{-6}$	<	Q	<	$3x10^{-5}$	Minor damage to fittings etc
		Q	>	$3x10^{-5}$	Structural damage
Embankment Seawalls :-					
		Q	<	0.002	No damage
0.002	<	Q	<	0.02	Damage if crest not protected
0.02	<	Q	<	0.05	Damage if back slope not protected
		Q	>	0.05	Damage even if fully protected
Revetment Seawalls :-					
		Q	<	0.05	No damage
0.05	<	Q	<	0.2	Damage if promenade not paved
		Q	>	0.2	Damage even if promenade paved

1.3.4 Locally Availably Material Properties

The design of coastal structures should be initiated with an understanding of the materials that are locally available and thus cost effective for the construction of the structures. The specific density of armour stone local quarries is typically around 2.45 to 2.5. In addition to the specific density of the armour material, there are a number of other engineering specifications that armour should meet. These are summarized below:

a) A minimum specific density of 2.47
b) Angular in shape.
c) Absorption of less than 1.2%
d) Abrasions of less than 25% losses after 1,000 revolutions.
e) MgSO4 soundness of less than 2% losses safer 5 cycles.
f) Field drop test: no breakage or cracking.

1.3.5 Damage Level

There will always be some movement of the stones in the armour structures, even in wave conditions less intense than the design wave conditions. The amount of movement can be qualitatively estimated or considered in the design phase of a project and it is a critical design input of the armour sizing models. The amount of tolerable movement anticipated in the design is reflected in the Damage Level (S). The less damage required or desired, is the more expensive the structure as the cross-section becomes thicker and the stones become larger. See Table 1.3.

A damage level of 2 was utilized in the design. It is therefore anticipated that two stones will move per unit width of the structures when the design conditions occur or are exceeded. This is categorized as within initial damage stage, i.e. less than intermediate or failure damage levels.

Table 1.3 Damage levels for armour structures

Damage level by S for two-layer armor (van der Meer 1988).

Unit	Slope	Initial damage	Intermediate damage	Failure
Rock	1 : 1.5	2	3–5	8
Rock	1 : 2	2	4–6	8
Rock	1 : 3	2	6–9	12
Rock	1 : 4 – 1 : 6	3	8–12	17

1.3.6 Airport Runway Approach Angle Criterion

The runway approach angle is the angle between the airplane's flight path and the runway. It was important for the purposes of this report to define that angle and to ensure that the coastal protection structure was not exceeding an elevation that would pose a threat to aircrafts approaching the runway for landing or taking off from the runway. The angle of approach for the aircraft to the runway is 2 degrees as communicated by NMIA representative. The design of the revetment and any associated structure will have to conform to elevations below elevations that may pose a danger to normal landing or taking off of the planes from the runway.

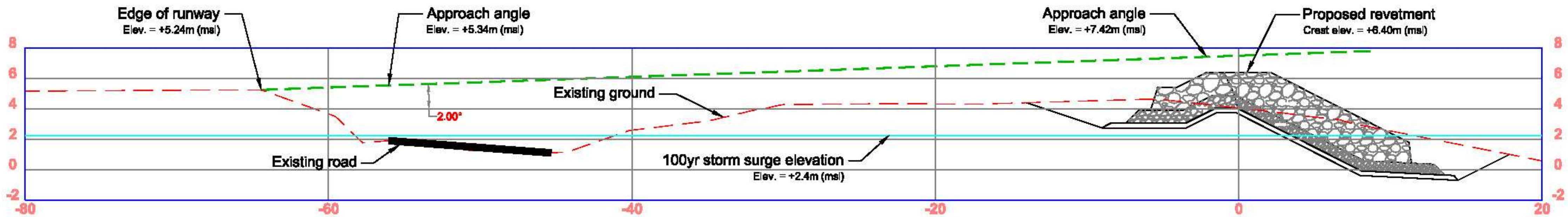


Figure 1.5 Profile from runway through to the shoreline

2 Data Collection

2.1 Topographic Surveys

A topographic survey of the project area was conducted from the shoreline to the road for 1000m stretch of shoreline. The survey was conducted by Gordon and Company Ltd (Commissioned Land Surveyors). The extents of the survey area were offset at 500m to the left and to the right of the end of the runway. See Figure 2.1 below. The survey datum was mean sea level and the projection was JD2001.

The terrain between the shoreline and the road varies from approximately 4.5m to 2m on average when moving from the Queens Warehouse intersection in the north-east to the lighthouse in the south-west. The elevation of the end of the runway is approximately 4.9metres which is almost a metre above the elevation of the dunes. The road elevation within the project area varies from 4.3m in the northeast to 1.1m in the south-west with a sag point at the end of the runway having an average elevation of 0.9m.

Figure 2.1 Topographic survey points along the 1km project area

2.2 Bathymetric Surveys

Bathymetric surveys could not be conducted with the required time period due to rough sea conditions. The bathymetric data for the site was taken from the British Admiralty charts 456 and 454 for the area. The data is sufficient for this study in light of the information gleaned from the charts. Additionally, the observed and predicted wave climate model indicated significant offshore features that generated the dominant nearshore wave conditions. These features would more than likely also be observed in any single beam survey.

2.3 Grain Size Analysis

2.3.1 Sediment size

Surface sediment samples were recovered from the project area at eight locations along the beach/shoreline. A Global Positioning Point (gps) waypoint was taken with a *Garmin 530HCx* hand held device at each point to mark the location. See Figure 2.2 below for the sediment sample location points.

Figure 2.2 –Sediment sample locations.

Grain size analysis of these samples was conducted and the results of this analysis are summarized in Figure 2.3 and Table 2.1.

The grain size analysis was done using the unified classification which is widely used for classification of granular material. The sand sizes varied from coarse sands to gravel based on their mean grain size.

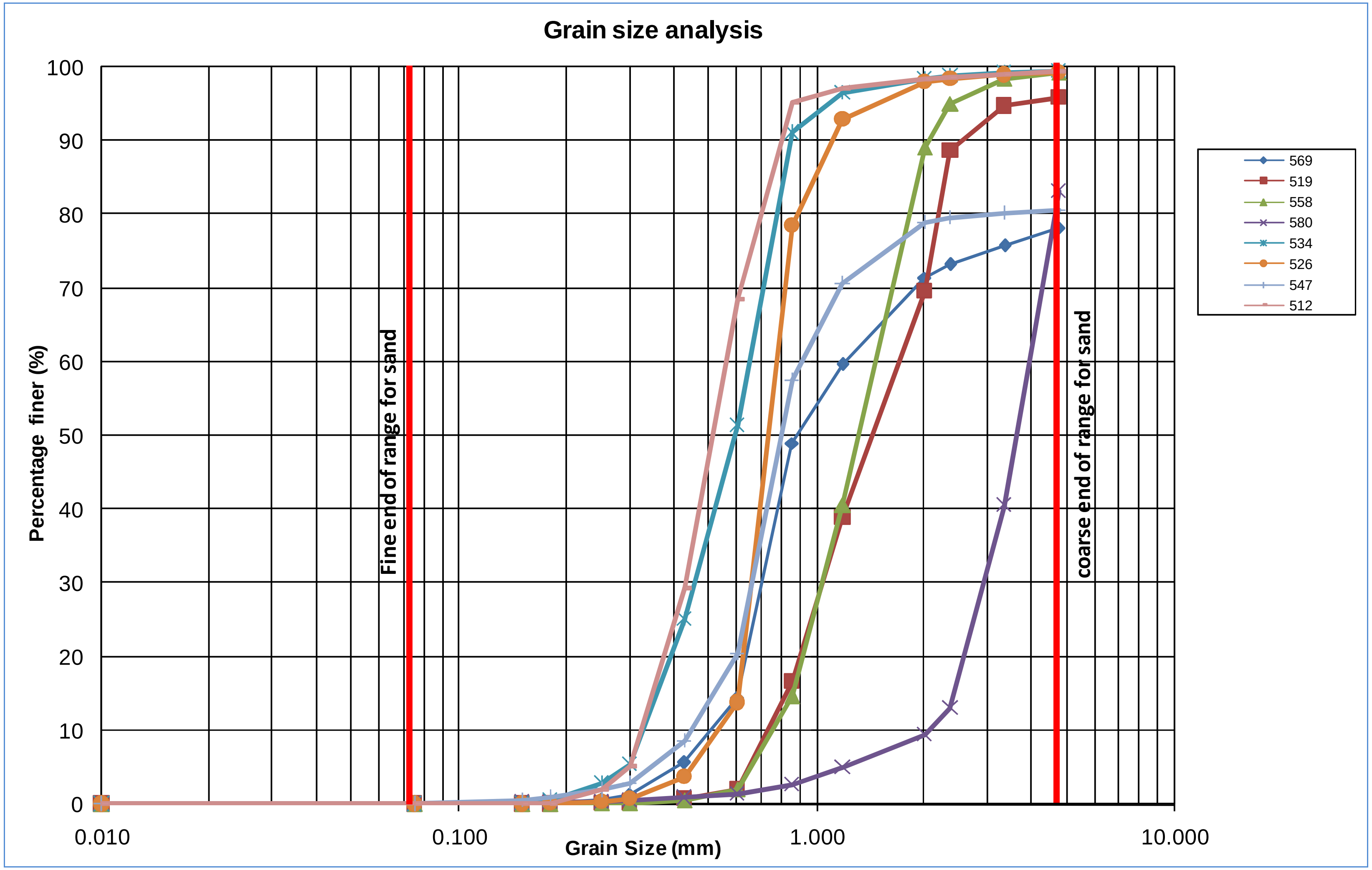


Figure 2.3 Sieve analysis results

Sample ID	**569**	**558**	**580**	**534**	**526**	**547**	**512**	**519**
Location (Relative to runway)	**East**	**East**	**East**	**West**	**West**	**West**	**West**	**West**
GRAIN SIZE ANALYSIS RESULTS								
Mean (mm)	0.880	1.339	3.660	0.591	0.740	0.799	0.518	1.477
Mean (phi)	0.184	-0.421	-1.872	0.759	0.435	0.324	0.950	-0.562
Description	coarse sand	very coarse sand	gravel	coarse sand	coarse sand	coarse sand	coarse sand	very coarse sand
Percentage silt	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Percentage >0.06mm and <6.0 mm	78%	99%	83%	100%	99%	81%	99%	96%
Uniformity Coefficient	2.350	1.992	1.940	1.989	1.458	2.047	1.728	2.371
Standard Deviation	-	0.565	-	0.571	0.436	-	0.494	0.739
	extremelely poorly sorted	moderately well sorted	extremelely poorly sorted	moderately well sorted	well sorted	extremelely poorly sorted	well sorted	moderately sorted
Skewness	-	-0.631	-	1.483	1.106	-	1.820	-0.740
	V. strongly positive skewed	strongly negative skewed	V. strongly positive skewed	strongly positive skewed	strongly positive skewed	V. strongly positive skewed	strongly positive skewed	strongly negative skewed
Kurtosis	-	0.894	-	0.950	1.898	-	0.864	0.924
	extremely leptokurtic	platykurtic	extremely leptokurtic	mesokurtic	very leptokurtic	extremely leptokurtic	platykurtic	mesokurtic

Table 2.1 Grain size analysis on beach sand samples east and west of the end of the runway

2.3.2 Uniformity coefficient

The uniformity coefficient is a measure of the variation in particle sizes. It is defined as the ratio of the size of particle that has 60 percent of the material finer than itself, to the size of the particle that has 10 percent finer than itself.

The uniformity coefficient is calculated as Uc = D_{60}/D_{10}

Where U_c – uniformity coefficient

D_{60} - The grain size, in mm, for which 60% by weight of a soil sample is finer
D_{10} - The grain size, in mm, for which 10% by weight of a soil sample is finer

Within the unified classification system, the sand is well graded if U_c is greater than or equal to 6. All the samples analyzed had uniformity coefficient much less than 6 and are therefore not well graded. The soils can be classified as sorted. This is indicative of wave energy suspending finer particles and removing them offshore and depositing coarser particles on shore.

2.3.3 Standard Deviation

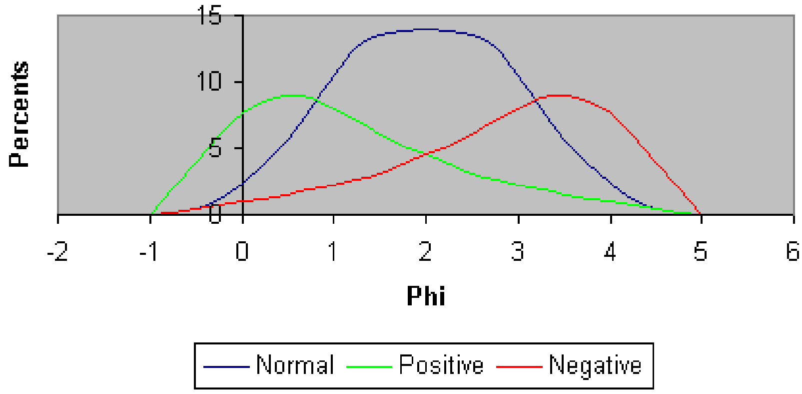
The Standard deviation is a measure of the degree of sorting of the particles in the sample. A standard deviation of one or less defines a sample that is well sorted while values above one are poorly sorted. Three of the eight or 37.5 percent of the samples were extremely poorly sorted while the remainder varied from moderately to well sorted. Two of the three extremely poorly sorted samples were to the far west of the end of the runway while the third was the closest to the west of the end of the runway. This is indicative of increased energy at different point of the shoreline, highlighting the fact wave energy is being focused on different areas of the shoreline.

2.3.4 Skewness

Skewness describes the shift in the distribution about the normal. The skewness is described by the equation:

$$\mathbf{S} = \frac{\phi 84 + \phi 16 - 2(\phi 50)}{2(\phi 84 - \phi 16)} + \frac{\phi 95 + \phi 5 - 2(\phi 50)}{2(\phi 95 - \phi 5)}$$

This formula simply averages the skewness obtained using the 16 phi and 84 phi points with the skewness obtained by using the 5 phi and 95 phi points, both determined by exactly the same principle. This is the best skewness measure to use because it determines the skewness of the “tails” of the curve, not just the central portion, and the “tails” are just where the most critical differences between samples lie. Furthermore, it is geometrically independent of the sorting of the sample.



Symmetrical curves have skewness=0.00; those with excess fine material (a tail to the right) have positive skewness and those with excess coarse material (a tail to the left) have negative skewness. The more the skewness value departs from 0.00, the greater the degree of asymmetry. The following verbal limits on skewness are suggested: for values of skewness:

Values from	To	Mathematically:	Graphically Skewed to the:
+1.00	**+0.30**	Strongly positive skewed	Very Negative phi values, coarse
+0.30	**+0.10**	Positive skewed	Negative phi values
+0.10	**- 0.10**	Near symmetrical	Symmetrical
- 0.10	**- 0.30**	Negative skewed	Positive phi values
- 0.30	**- 1.00**	Strongly negative skewed	Very Positive phi values, fine

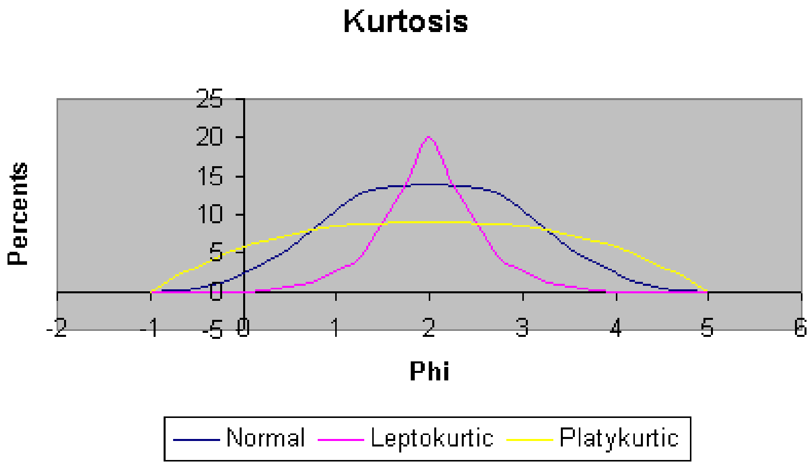
The results for skewness for the stretch of shoreline can be summarized as follows:

- Two of the five samples ranged from strong to very strong negative skewness. This is indicative of a coarse tail and an aggressive wave climate at the shoreline that washes out the fines at these locations.
- Three of the five samples had very strongly positive skewness indicative of locations with moderate wave climates allowing fines to remain on the beach.

2.3.5 Kurtosis

Kurtosis describes the degree of peakedness or departure from the "normal" frequency or cumulative curve

In the normal probability curve, defined by the gaussian formula; the phi diameter interval between the 5 phi and 95 phi points should be exactly 2.44 times the phi diameter interval between the 25 phi and 75 phi points. If the sample curve plots as a straight line on probability paper (i.e., if it follows the normal curve), this ratio will be obeyed and we say it has normal kurtosis (1.00). Departure from a straight line will alter this ratio, and kurtosis is the quantitative measure used to describe this departure from normality. It measures the ratio between the sorting in the "tails" of the curve and the sorting in the central portion. If the central portion is better sorted than the tails, the curve is said to be excessively peaked or leptokurtic; if the tails are better sorted than the central portion, the curve is deficiently or flat-peaked and platykurtic.



Strongly platykurtic curves are often bimodal with subequal amounts of the two modes; these plot out as a two-peaked frequency curve, with the sag in the middle of the two peaks accounting for its platykurtic character. For normal curves, kurtosis equals 1.00. Leptokurtic curves have a kurtosis over 1.00 (for example a curve with kurtosis=2.00 has exactly twice as large a spread in the tails as it should have, hence it is less well sorted in the tails than in the central portion); and platykurtic have kurtosis under 1.00. Kurtosis involves a ratio of spreads; hence it is a pure number and should not be written with a phi attached. The following verbal limits are suggested for values of kurtosis:

Values from	To	Equal
0.41	**0.67**	very platykurtic
0.67	**0.90**	platykurtic
0.90	**1.11**	mesokurtic
1.10	**1.50**	leptokurtic
1.50	**3.00**	very leptokurtic
3.00	**∞**	extremely leptokurtic

The results for kurtosis for the stretch of shoreline can be summarized as follows:

- Two of the five samples are platykurtic. This is indicative of a flat top or sediments that are well graded.
- Three of the five samples are leptokurtic. This is indicative of a flat top or sediments that are well sorted.

3 Vulnerability Study

3.1 Wave Climate

Wave information on the project site is crucial in order to understand the likely conditions that the shoreline will be subjected to during a hurricane and hence design adequate mitigating structures. The site is exposed to the Caribbean Sea from a southerly direction, and the potential is there for significantly high waves to impact the shoreline and so must be examined. An in-house hindcast model was used to estimate the waves that may be generated by a 100yr return storm. Examination of the wave climate as the waves approach the shoreline was done using the refraction and diffraction software known as REFDIF distributed by the US Army Corp.

3.1.1 Deepwater

3.1.1.1 Methodology

It was necessary to define the deepwater hurricane wave climate at the site as a part of defining the wave climate that the shoreline is subject to. Hurricane wave track data in the Caribbean Sea was available which enabled us to carry out a thorough statistical analysis to determine the hurricane wind and wave conditions at a deep-water location offshore the site.

A database of hurricanes, dating back to 1886, was searched for storms that passed within a 300km radius from the site. The following procedure was carried out.

1. **Extraction of storms and storm parameters from the historical database:** A historical database of storms was searched for all storms passing within a 300km radius of the site.
2. **Application of the JONSWAP wind-wave model.** A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.
3. **Application of extremal statistics.** Here the predicted maximum wave height from each hurricane was arranged in descending order and each assigned an exceedance probability by Weibull's distribution.
4. A bathymetric profile from deepwater to the site was then defined and each hurricane wave transformed along the profile. The wave height at the nearshore end of the profile was then extracted from the model and stored in a database. All the returned nearshore values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

3.1.1.2 Results

The results of this analysis indicate that between 1852 and 2008, 77 hurricanes have passed to the south and within 300km of the project site. The number and frequency of more intense hurricanes occurring within the vicinity of the site have increased over the past 70 years. See Table 3.1 below for a summary of these results.

Table 3.1 Summary of the number of hurricanes passing the south coast and the project site between 1885 and 2008

	Storm No.	Name	Date	Max. SS Category	
1	10	NOTNAMED	1852	2-	MODERATE
2	38	NOTNAMED	1857	2-	MODERATE
3	50	NOTNAMED	1859	3-	EXTENSIVE
4	83	NOTNAMED	1864	1-	WEAK
5	94	NOTNAMED	1866	3-	EXTENSIVE
6	127	NOTNAMED	1870	2-	MODERATE
7	157	NOTNAMED	1874	2-	MODERATE
8	188	NOTNAMED	1878	1-	WEAK
9	194	NOTNAMED	1879	1-	WEAK
10	198	NOTNAMED	1880	4-	EXTREME
11	199	NOTNAMED	1880	1-	WEAK
12	227	NOTNAMED	1884	2-	MODERATE
13	240	NOTNAMED	1886	2-	MODERATE
14	241	NOTNAMED	1886	2-	MODERATE
15	242	NOTNAMED	1886	3-	EXTENSIVE
16	252	NOTNAMED	1887	2-	MODERATE
17	277	NOTNAMED	1889	2-	MODERATE
18	321	NOTNAMED	1895	2-	MODERATE
19	324	NOTNAMED	1895	3-	EXTENSIVE
20	329	NOTNAMED	1896	3-	EXTENSIVE
21	344	NOTNAMED	1898	1-	WEAK
22	345	NOTNAMED	1898	1-	WEAK
23	375	NOTNAMED	1903	3-	EXTENSIVE
24	391	NOTNAMED	1905	2-	MODERATE
25	400	NOTNAMED	1906	4-	EXTREME
26	418	NOTNAMED	1909	4-	EXTREME
27	419	NOTNAMED	1909	1-	WEAK
28	422	NOTNAMED	1909	4-	EXTREME
29	424	NOTNAMED	1909	3-	EXTENSIVE
30	425	NOTNAMED	1909	1-	WEAK
31	426	NOTNAMED	1910	1-	WEAK
32	446	NOTNAMED	1915	4-	EXTREME
33	448	NOTNAMED	1915	2-	MODERATE
34	449	NOTNAMED	1915	4-	EXTREME
35	453	NOTNAMED	1916	3-	EXTENSIVE
36	455	NOTNAMED	1916	3-	EXTENSIVE
37	462	NOTNAMED	1916	3-	EXTENSIVE
38	466	NOTNAMED	1917	3-	EXTENSIVE
39	467	NOTNAMED	1918	2-	MODERATE
40	503	NOTNAMED	1924	2-	MODERATE
41	526	NOTNAMED	1928	1-	WEAK
42	537	NOTNAMED	1931	1-	WEAK
43	539	NOTNAMED	1931	3-	EXTENSIVE
44	550	NOTNAMED	1932	3-	EXTENSIVE
45	560	NOTNAMED	1933	1-	WEAK
46	569	NOTNAMED	1933	2-	MODERATE
47	573	NOTNAMED	1933	2-	MODERATE
48	591	NOTNAMED	1935	1-	WEAK
49	619	NOTNAMED	1938	2-	MODERATE
50	620	NOTNAMED	1938	2-	MODERATE
51	646	NOTNAMED	1942	3-	EXTENSIVE
52	648	NOTNAMED	1942	1-	WEAK
53	666	NOTNAMED	1944	1-	WEAK
54	668	NOTNAMED	1944	3-	EXTENSIVE
55	739	CHARLIE	1951	4-	EXTREME
56	740	DOG	1951	3-	EXTENSIVE
57	761	FLORENCE	1953	3-	EXTENSIVE
58	776	HAZEL	1954	4-	EXTREME
59	788	JANET	1955	5-	CATASTROPHIC
60	890	BEULAH	1967	5-	CATASTROPHIC

	Storm No.	Name	Date	Max. SS Category	
61	910	FRANCELIA	1969	3-	EXTENSIVE
62	966	CARMEN	1974	4-	EXTREME
63	969	FIFI	1974	2-	MODERATE
64	1095	GILBERT	1988	5-	CATASTROPHIC
65	1111	ARTHUR	1990	1-	WEAK
66	1186	MARCO	1996	1-	WEAK
67	1220	LENNY	1999	4-	EXTREME
68	1228	HELENE	2000	1-	WEAK
69	1244	IRIS	2001	4-	EXTREME
70	1259	ISIDORE	2002	3-	EXTENSIVE
71	1262	LILI	2002	4-	EXTREME
72	1326	IVAN	2004	5-	CATASTROPHIC
73	1336	DENNIS	2005	4-	EXTREME
74	1337	EMILY	2005	5-	CATASTROPHIC
75	1366	ERNESTO	2006	1-	WEAK
76	1374	DEAN	2007	5-	CATASTROPHIC
77	1401	GUSTAV	2008	4-	EXTREME

The wave heights for the 100 year return period hurricane varied from 4.9m to 8.1m, with the lowest being from the south-west and the highest from the East. For practical purposes of running the near shore model however, the eastern direction cannot be used and will therefore be substitutes with the ESE direction.

Table 3.2 Summary of the waveheights and wave periods for different return periods for Palisadoes

Return Periods	Wave height (m)																	
	All		SW		W		NW		N		NE		E		SE		S	
	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp
1	2.5	8.0	1.5	6.2									1.5	6.2	1.5	6.2	1.5	6.2
2	3.7	9.6	3.4	9.3									4.6	10.7	4.4	10.5	3.5	9.5
5	4.9	11.1	4.0	10.0									5.8	12.0	5.5	11.7	4.5	10.6
10	5.8	12.0	4.2	10.3									6.5	12.7	6.2	12.4	5.0	11.1
20	6.6	12.7	4.5	10.6									7.0	13.2	6.7	12.8	5.4	11.6
25	6.8	13.0	4.5	10.6									7.2	13.3	6.8	13.0	5.5	11.7
50	7.6	13.7	4.7	10.8									7.7	13.7	7.3	13.4	5.9	12.0
75	8.0	14.0	4.8	10.9									7.9	13.9	7.5	13.6	6.0	12.2
100	8.4	14.3	4.9	11.0									8.1	14.1	7.7	13.7	6.2	12.4
150	8.8	14.7	4.9	11.1									8.3	14.3	7.9	13.9	6.3	12.5
200	9.1	14.9	5.0	11.2									8.5	14.4	8.0	14.0	6.5	12.6

3.1.1.3 Storm Surge

It was important to define the design water levels in the project area in order to define the appropriate crest elevations for the revetment structure. Static storm surge was investigated in the analysis for all major components of storm surge. The phenomena considered were:

- Wave breaking and shoaling
- Wind set-up
- Refraction
- Tides and
- Global Sea Level Rise (over a 50 year project life)
- Inverse Barometric Pressure Rise

The estimated storm surge is greatest from the SE direction at 2.2m for the 100yr return storm. See summary in Table 3.4 below.

Table 3.3 summary of estimated storm surge for 100yr return storm

Return Period	Total setup (m)								
	All	SW	W	NW	N	NE	E	SE	S
1							0.05	0.05	
2	0.19	0.09					0.34	0.49	0.19
5	0.49	0.16					0.64	0.94	0.37
10	0.75	0.22					0.85	1.26	0.51
20	1.03	0.28					1.05	1.55	0.65
25	1.12	0.29					1.11	1.65	0.69
50	1.43	0.36					1.31	1.93	0.83
75	1.61	0.39					1.42	2.09	0.91
100	1.74	0.42					1.49	$\boxed{2.20}$	0.96
150	1.94	0.45					1.60	2.36	1.04
200	2.08	0.48					1.68	2.47	1.10

3.1.2 Comparative Deepwater Wave Climate from Similar studies

Three other studies have been carried out in this region from 1999. The studies include the Sandwell Study of 2008, Cuban study of 2007 and OAS study of 1999. The present design parameters which describe the deep water wave climate were compared to these studies and are summarized in Table 3.4 below. In summary the estimated deepwater wave heights vary for 8.1 metres (for this study) to a peculiarly high value of 15.5 metres (Sandwell). The median of the other studies is approximately 10.75 metres. It must be noted that whilst deepwater wave heights are an input to the design process, the ultimate design input is the nearshore wave heights. It therefore follows that the resulting nearshore wave heights are more important to observe and estimate for design purposes. These wave heights are greatly controlled by nearshore bathymetry.

Table 3.4 Summary of design wave and wind parameters

Design Parameter	Present 2011	Sandwell (2008/2009) Design	Cuban Design (2007)	Cuban Design (2007) - 100 Year parameters	OAS (1999)	Comments
Return Period	100 Years (equivalent to Category 4 to 5)	100 Years (equivalent to Category 4 to 5)	Category 3 (~23 Year Return Period)	100 Years	100 Years	Return period consistent with the design of permanent works
Wave Height (metres)	Hs = 8.1 metres Tp = 14.1 seconds	Hs = 15.5 metres Tp = 18.65 seconds	Hs = 6.0 metres	Hs ~ 10 metres	11.5 metres	Sandwell design likely to be on the cautious side.
Hurricane Wind Speed (metres per second)	46.7	1.7 metres waves from 65 knots winds from West over 13 kilometres			51	
Storm Surge (metres)	2.20	Minimum road elevation of 3.2 metres for road next to revetment and 2.4 metres for road removed from revetment			5.5 metres	

3.1.3 Nearshore 100 Year Return Period

3.1.3.1 *Objectives and Approach*

It was necessary to estimate what the wave climate is like near to the shoreline in order to determine the nearshore wave regime on the existing shoreline and likely forces on the proposed protective structures to be implemented.

Deepwater water wave data forms the input for such analysis and by itself offers limited information on how waves reach the shoreline. The objective of this exercise was to derive a nearshore wave climate in order to better understand the environment and processes involved. The approach adopted in order to achieve these objectives was as follows:

1. Prepare a bathymetric database of the project domain for extremal analysis.
2. Determine the nearshore wave climate for the project area.
3. Conduct spatial wave transformation analysis around the reefs, etc in the model.

3.1.3.2 *Wave Climate Model: REFDIF*

The weakly nonlinear combined refraction and diffraction model described here denoted REFDIF simulates the behaviour of a random sea over irregular bottom bathymetry incorporating the effects of shoaling, refraction, energy dissipation and diffraction. Although the model is developed to simulate a random sea state it can also be used to model the behaviour of monochromatic waves. REFDIF was developed by Kirby and Dalrymple[1]. The model REFDIF is constructed in parabolic form and thus there is a restriction of the model to cases where the propagation direction is within the assumed mean wave direction

3.1.3.3 *Modelling Approach and Summary of Incident Wave Conditions Modelled*

The output from the storm surge model used for hurricane impact analysis provided us with the incident wave height and period as well as the water setup for the deepwater extremal analysis. Locally generated waves i.e. waves generated by wind action within the bay were predicted using the JONSWAP equations. These incident wave heights and periods were then used in the REFDIF model to generate the nearshore wave climate. The spatial patterns of wave breaking and shoaling were noted in relation to the proposed site. Should intense wave focusing be noted, then it would probably be advisable that this be considered in the design of adequate structural engineering provisions.

[1] Kirby and Dalrymple 1984

Kirby, J. T., and Dalrymple, R. A. 1984. "Verification of a Parabolic Equation for Propagation of Weakly-Nonlinear Waves," *Coastal Engineering*, Vol 8, pp 219-232

Table 3.5 summary of the waves and setups used to run the refdif model

Direction	Wave height (m)	Period (s)	Total Setup
ESE	8.1	14.1	1.49
SE	7.7	13.7	2.20
SSE	7.7	13.7	2.20
S	6.2	12.4	0.96

3.1.3.4 Results

The results from the hurricane wave and storm surge analysis where used as input for REFDIF. The spatial patterns of wave breaking and shoaling were noted in relation to the existing shoreline.

The results for the hurricane scenarios investigated are shown in Figure 3.1 to Figure 3.4. These results indicate that the site is definitely more vulnerable to waves from the East-Southeast for the 100yr event.

Wave heights ranging from of 1.8m to 6.0 m were predicted to arrive at the shoreline from all four directions analyzed. This is comparable to the rest of the Palisadoes shoreline which will be affected by waves of the same order of magnitude.

3.1.3.5 Results

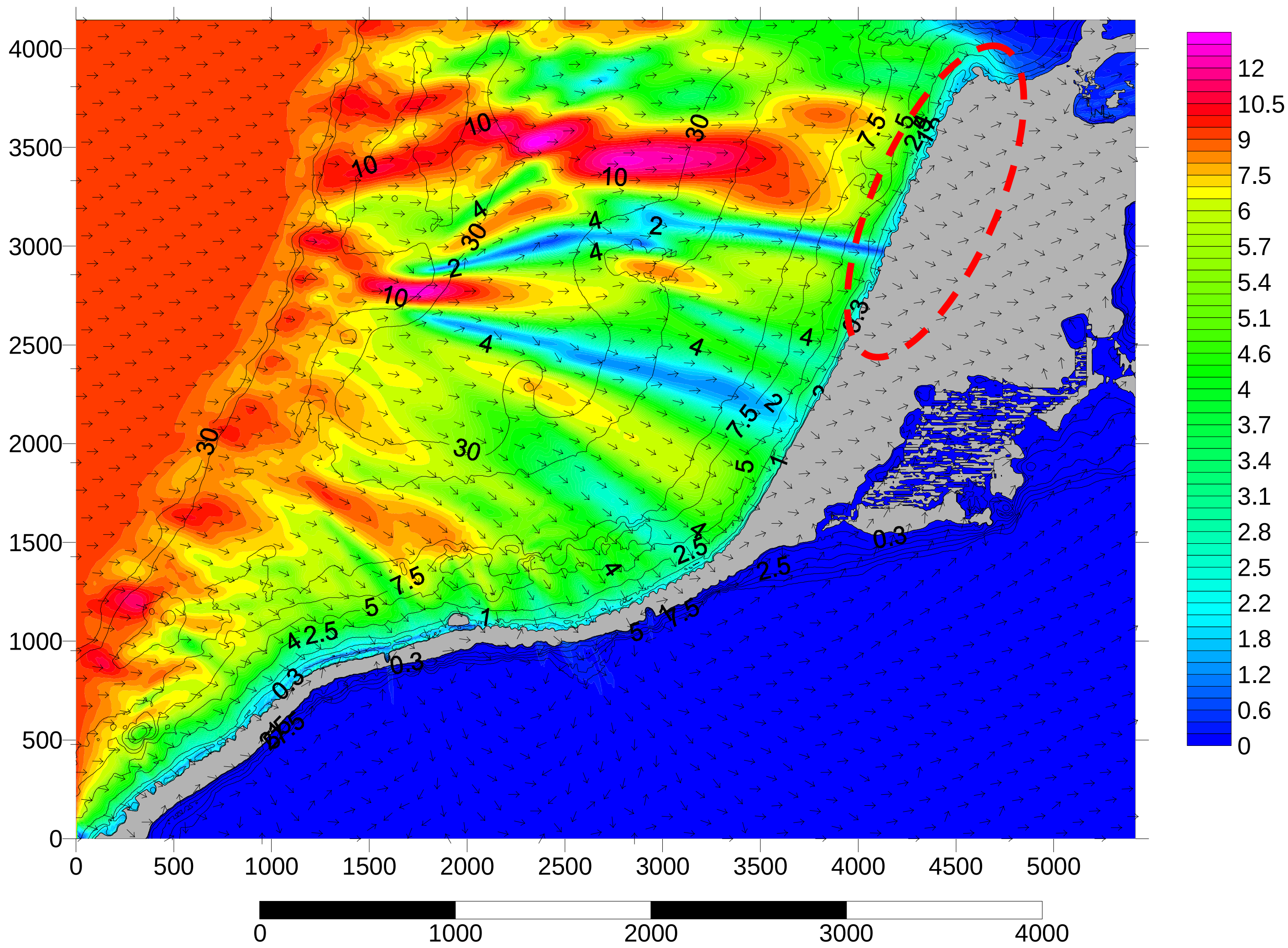


Figure 3.1 - 100 Year Return Period wave heights (m) for Palisadoes (ESE direction)

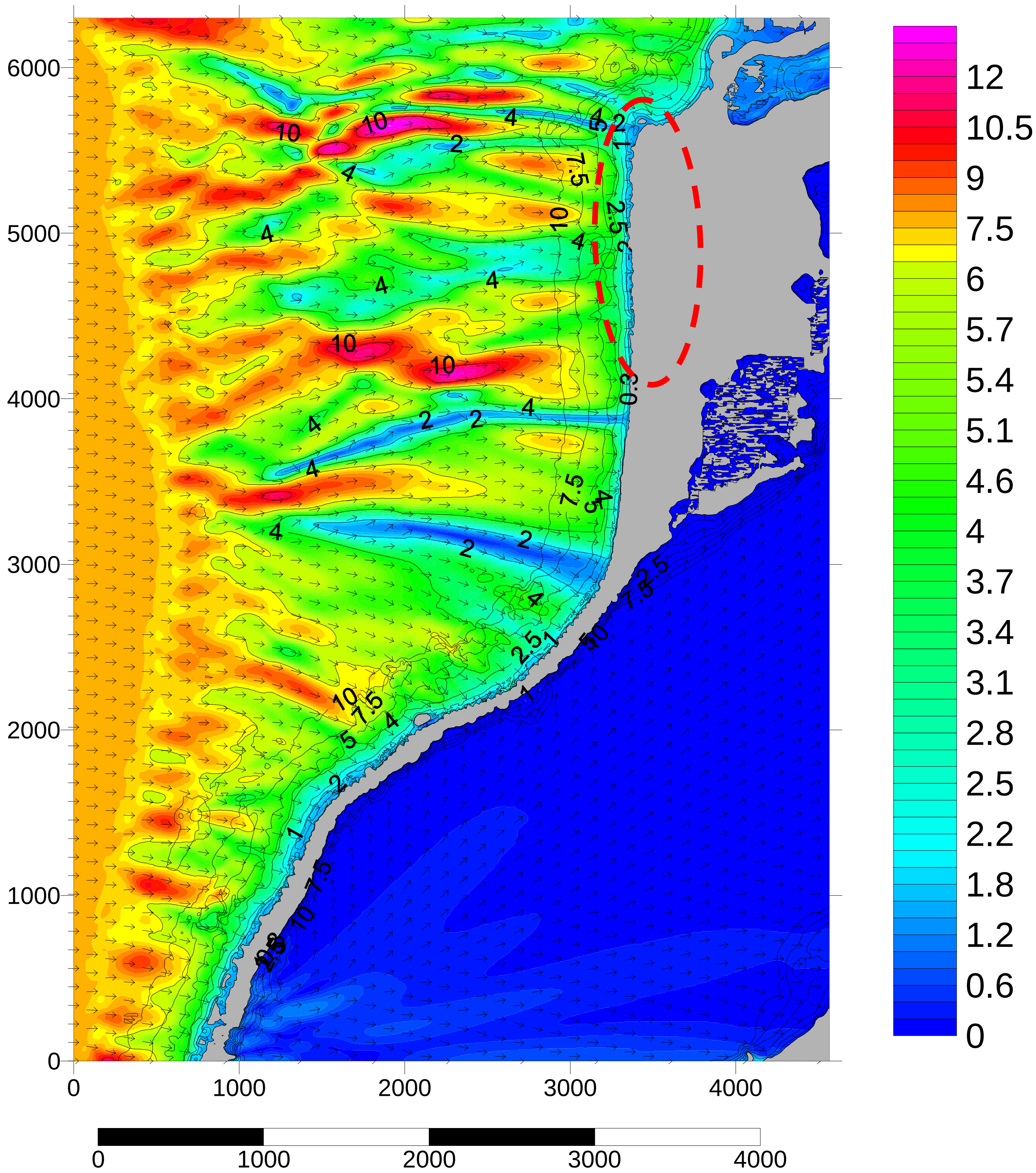


Figure 3.2 - 100 Year Return Period wave heights (m) for Palisadoes (SE direction)

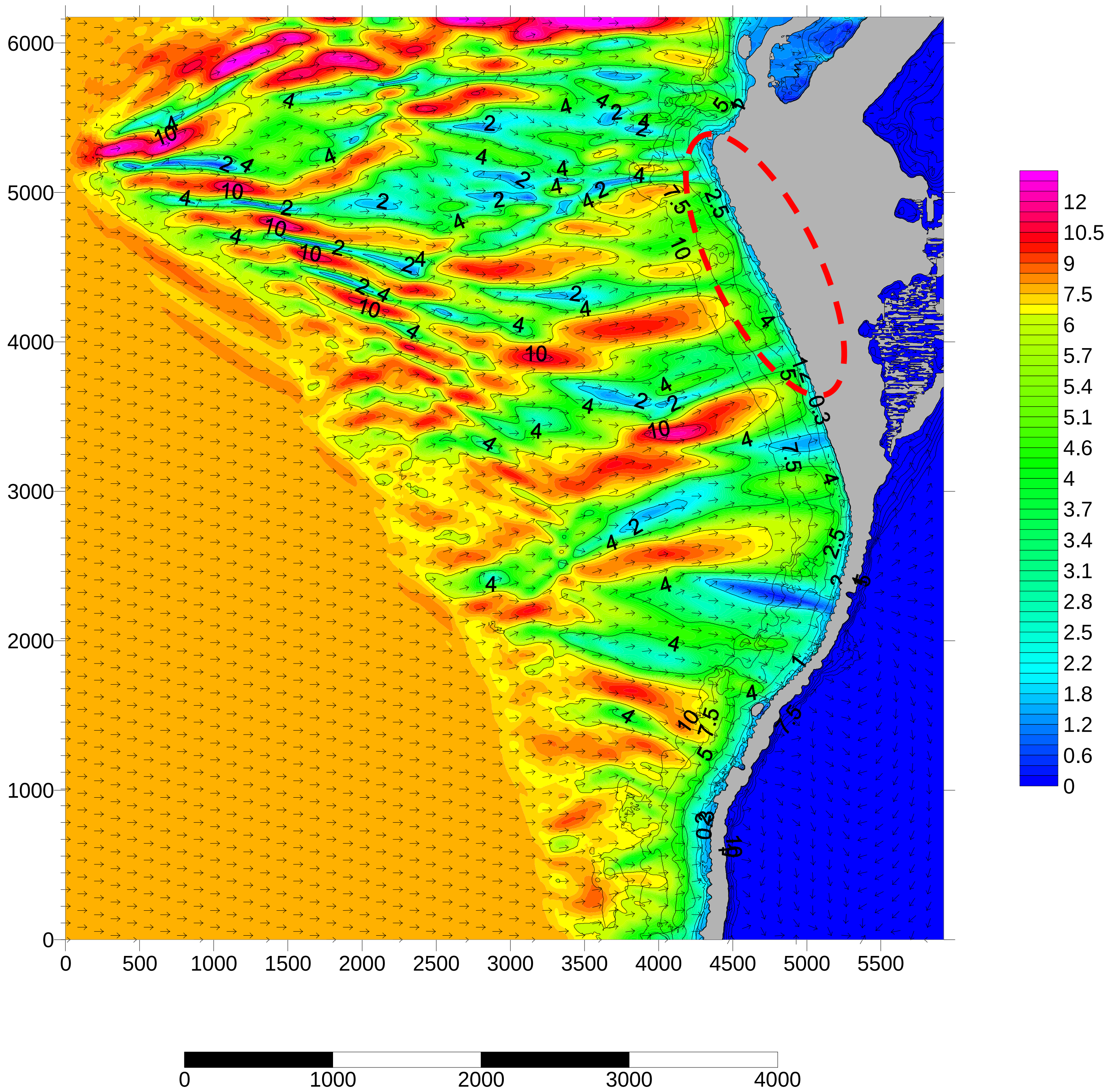


Figure 3.3 - 100 Year Return Period wave heights (m) for Palisadoes (SSE direction)

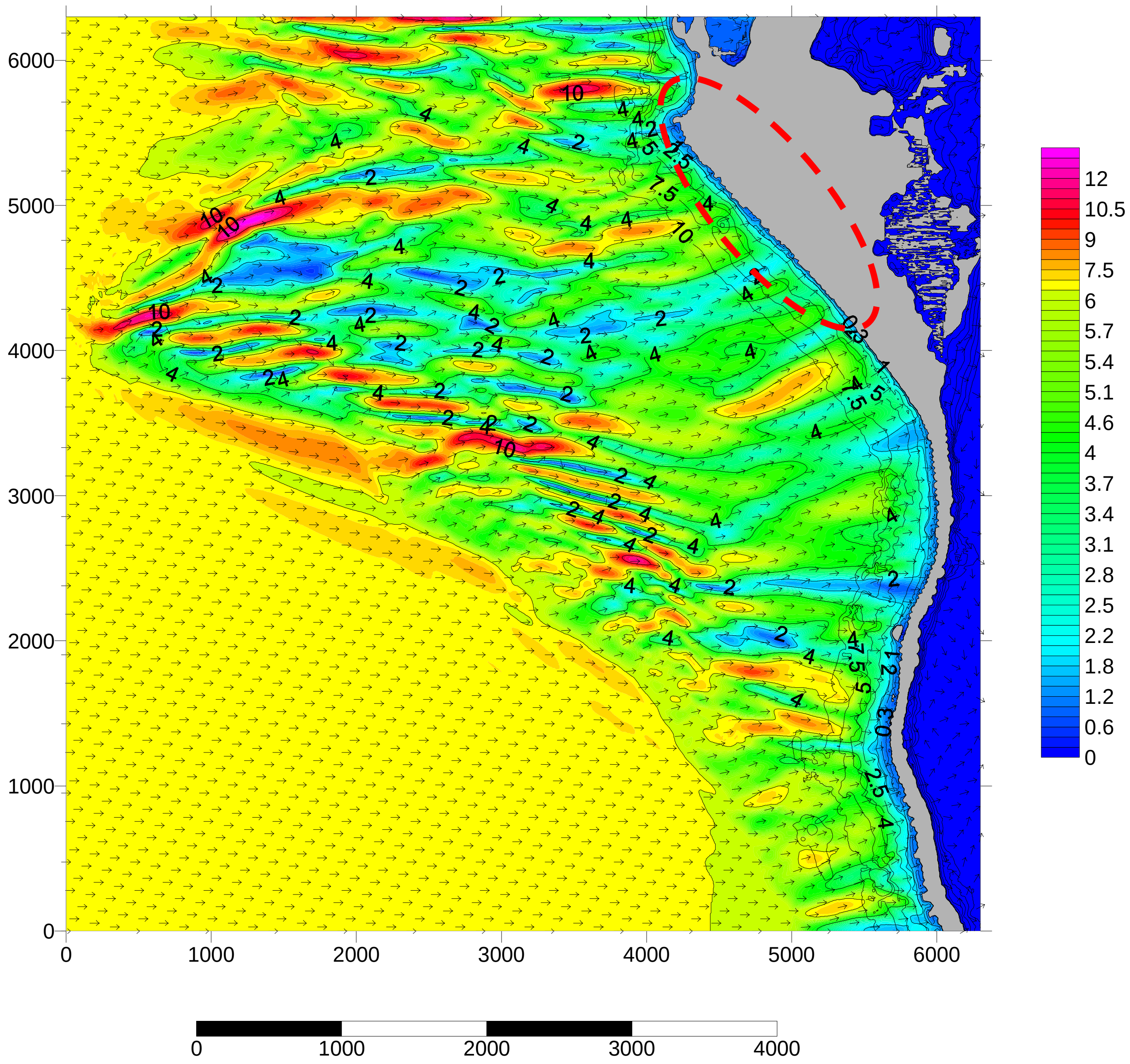


Figure 3.4 - 100 Year Return Period wave heights (m) for Palisadoes (South direction)

3.1.3.6 Discussion

The wave refraction analysis clearly indicates the vulnerability of the shoreline from waves approaching from the east and ESE. In both scenarios, 5 to 6 metre waves are expected some 500 metres offshore and 2.5 metres waves are expected at the shoreline. The waves appear to be focused from offshore bathymetric features such as submerged cays or mounds. The 100 year conditions from S and SSE waves are less severe with only 3 to 4 metre waves being predicted.

3.2 Shoreline Vulnerability

Shorelines are typically vulnerable to erosion due to passing storms as well as to daily wave action and or climate change impacts. It is therefore necessary to investigate the vulnerability of the site to these parameters in order to determine the necessity of providing protective structures. Both long term and short term erosion trends and or impacts were investigated.

3.2.1 Long term Erosion Trends

The overall long-term erosion trend was estimated by:

1) Firstly, observation of actual long-term shoreline positions from dated aerial photography.
2) Secondly, the global sea level rise component was estimated to determine the erosion that was due to chronic global trends versus event based erosion events (i.e. hurricanes and swell events)

3.2.1.1 Historical Shoreline Positions

The shoreline positions over a number of years were plotted and compared in order to determine the long-term spatial and temporal erosion trends across the project area. This was important in order to identify the actual erosion hotspots that might require stabilization.

Figure 3.5 show the most recently available satellite imagery (March 2010) over which the observed shorelines from Aerial photos of the area obtained from the Survey department for the years 1968 and 1991 were superimposed.

Close examination of the image in Figure 3.5 reveals a general trend of erosion occurring at all but one location from 1968 to 2010. The most western location had accretion.

Table 3.6 summarizes the results of measuring and noting the displacements of the shoreline at intervals of 100m for all the beaches. The rates of accretion and or erosion between the time intervals and the overall time interval were determined using the following relationship:

$E_y^1 = \frac{D}{N}$, where

E = the rate of erosion or accretion between two successive intervals (metres per year)

D = the displacement between two intervals (metres)

N = the number of years between two successive intervals (years) and

$E_y^0 = \frac{D_T}{N_T}$, where

E_y^0 = the rate of erosion or accretion from the datum year to the final interval

D_T = the displacement from the datum to the final interval

N_T = the number of years from datum year to final interval

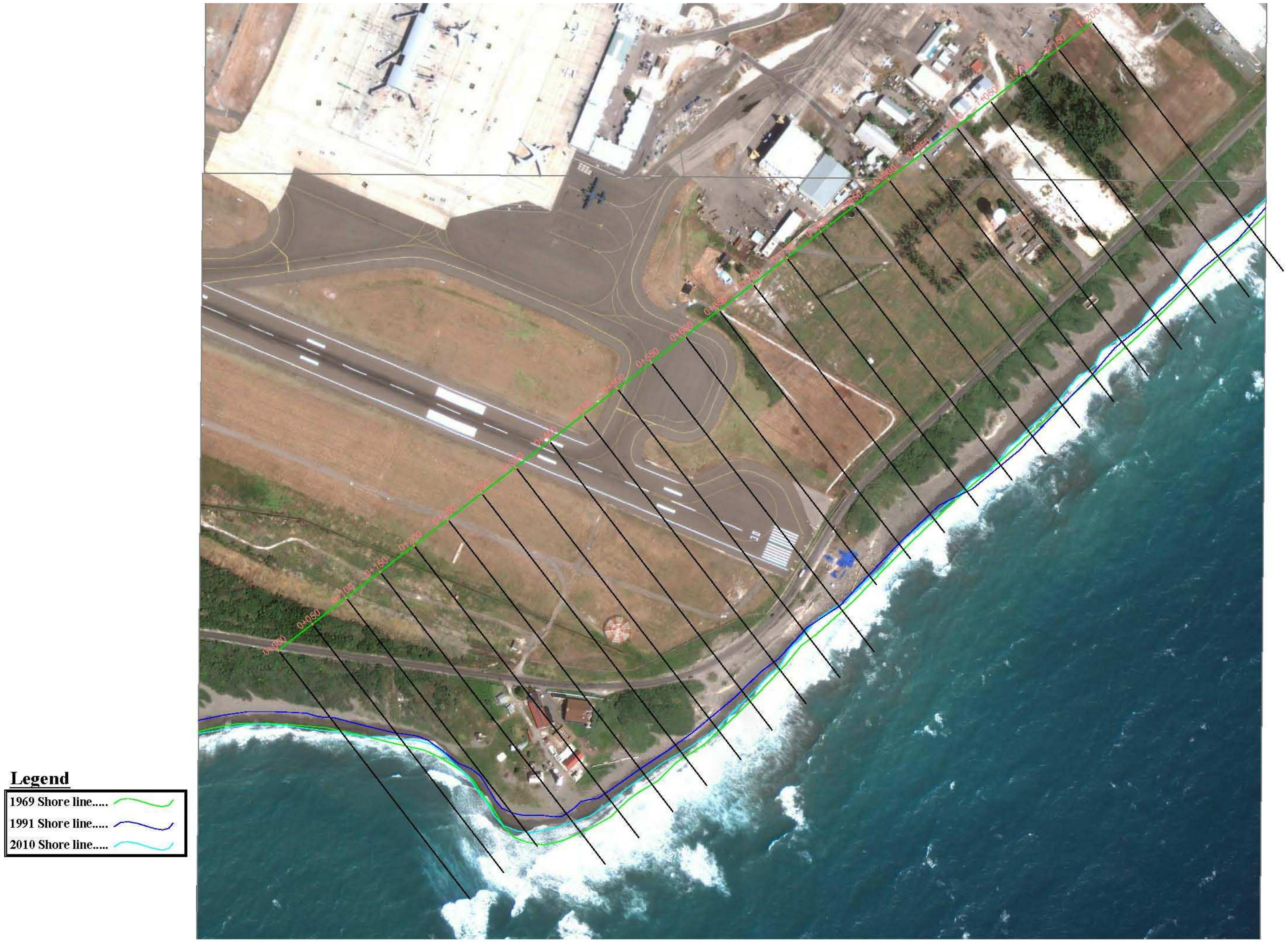


Figure 3.5 2010 Satellite imagery of project area with historical shorelines superimposed

A summary of the analysis of the shoreline data is shown in Table 3.6. Consult both Figure 3.6 and Figure 3.7 for graphical representation of the results obtained Table 3.6.

Table 3.6 Summary of the displacements of the shoreline for 1991 and 2010 about the 1968 shoreline at 50m intervals

Location		Shoreline Intervals										
		1968			1991			2010			Overall (1968 - 2010)	
Beach	Chainage	Distance from Datum	Process	Rate of movement (mm/year)	distance from datum (m)	Process	Rate of movement (mm/year)	distance from datum (m)	Process	Accretion/Erosion Rate (m/year)	Process	Rate
South-west of Runway	0+000	118.50	-	-	99.15	erosion	-0.841	121.68	accretion	1.186	accretion	0.076
	0+050	179.81	-	-	160.45	erosion	-0.842	169.79	accretion	0.492	erosion	-0.239
	0+100	362.00	-	-	316.43	erosion	-1.981	344.89	accretion	1.498	erosion	-0.407
	0+150	384.61	-	-	356.37	erosion	-1.228	369.27	accretion	0.679	erosion	-0.365
	0+200	389.77	-	-	365.55	erosion	-1.053	376.94	accretion	0.599	erosion	-0.305
	0+250	390.50	-	-	367.82	erosion	-0.986	377.45	accretion	0.507	erosion	-0.311
	0+300	388.48	-	-	370.80	erosion	-0.769	381.83	accretion	0.581	erosion	-0.158
	0+350	388.83	-	-	370.59	erosion	-0.793	384.49	accretion	0.732	erosion	-0.103
	0+400	383.52	-	-	367.73	erosion	-0.687	378.38	accretion	0.561	erosion	-0.122
End of Runway	0+450	376.46	-	-	364.53	erosion	-0.519	372.66	accretion	0.428	erosion	-0.090
	0+500	368.91	-	-	356.86	erosion	-0.524	359.64	accretion	0.146	erosion	-0.221
	0+550	366.01	-	-	359.26	erosion	-0.293	357.91	erosion	-0.071	erosion	-0.193
	0+600	359.89	-	-	351.69	erosion	-0.357	354.96	accretion	0.172	erosion	-0.117
North East of Runway	0+650	353.07	-	-	344.19	erosion	-0.386	344.89	accretion	0.037	erosion	-0.195
	0+700	347.57	-	-	338.81	erosion	-0.381	342.88	accretion	0.214	erosion	-0.112
	0+750	342.94	-	-	344.54	accretion	0.070	340.56	erosion	-0.209	erosion	-0.057
	0+800	340.13	-	-	344.69	accretion	0.198	338.36	erosion	-0.333	erosion	-0.042
	0+850	336.03	-	-	340.45	accretion	0.192	330.69	erosion	-0.514	erosion	-0.127
	0+900	331.90	-	-	332.95	accretion	0.046	325.17	erosion	-0.409	erosion	-0.160
	0+950	328.07	-	-	325.69	erosion	-0.103	319.28	erosion	-0.337	erosion	-0.209
	1+000	324.34	-	-	316.99	erosion	-0.320	312.20	erosion	-0.252	erosion	-0.289
	1+050	319.24	-	-	315.27	erosion	-0.173	306.71	erosion	-0.451	erosion	-0.298
	1+100	313.13	-	-	309.82	erosion	-0.144	296.91	erosion	-0.679	erosion	-0.386
	1+150	308.97	-	-	305.63	erosion	-0.145	285.47	erosion	-1.061	erosion	-0.560
	1+200	304.48	-	-	300.81	erosion	-0.160	288.91	erosion	-0.626	erosion	-0.371

Table 3.6 shows that for the different time intervals, the following occurred:

- 1968 to 1991
  - Net erosion took place on most of the shoreline, accretion took place over approximately 200m (chainage 0+750 to 0+900) of the 1200m of shoreline analyzed
  - The shoreline eroded a maximum of 45 metres and accreted 4.37m
- 1991 to 2010
  - More than 50% of the shoreline experienced a net accretion from the southwestern end of the project area to just below the end of the runway(chainage 0+000 to 0+700), the level of accretion varied from 0.57 to 62metres
  - Erosion occurred predominantly to the northeast of the runway, the erosions recorded varied from 3.8 to 44metres
- Overall (1968 – 2010)
  - A net erosion occurred for all location except one on the southwest of the runway where the rate of accretion was 0.076m/yr or 1.44metres.
  - The location to the south of the end of runway at a rate of 0.09 to 0.22 metres per year (3.7 to 9.2metres)
  - The location east of the end of runway is eroding at a rate of 0.057 to 0.56 metres per year (2.3 to 23.52metres)

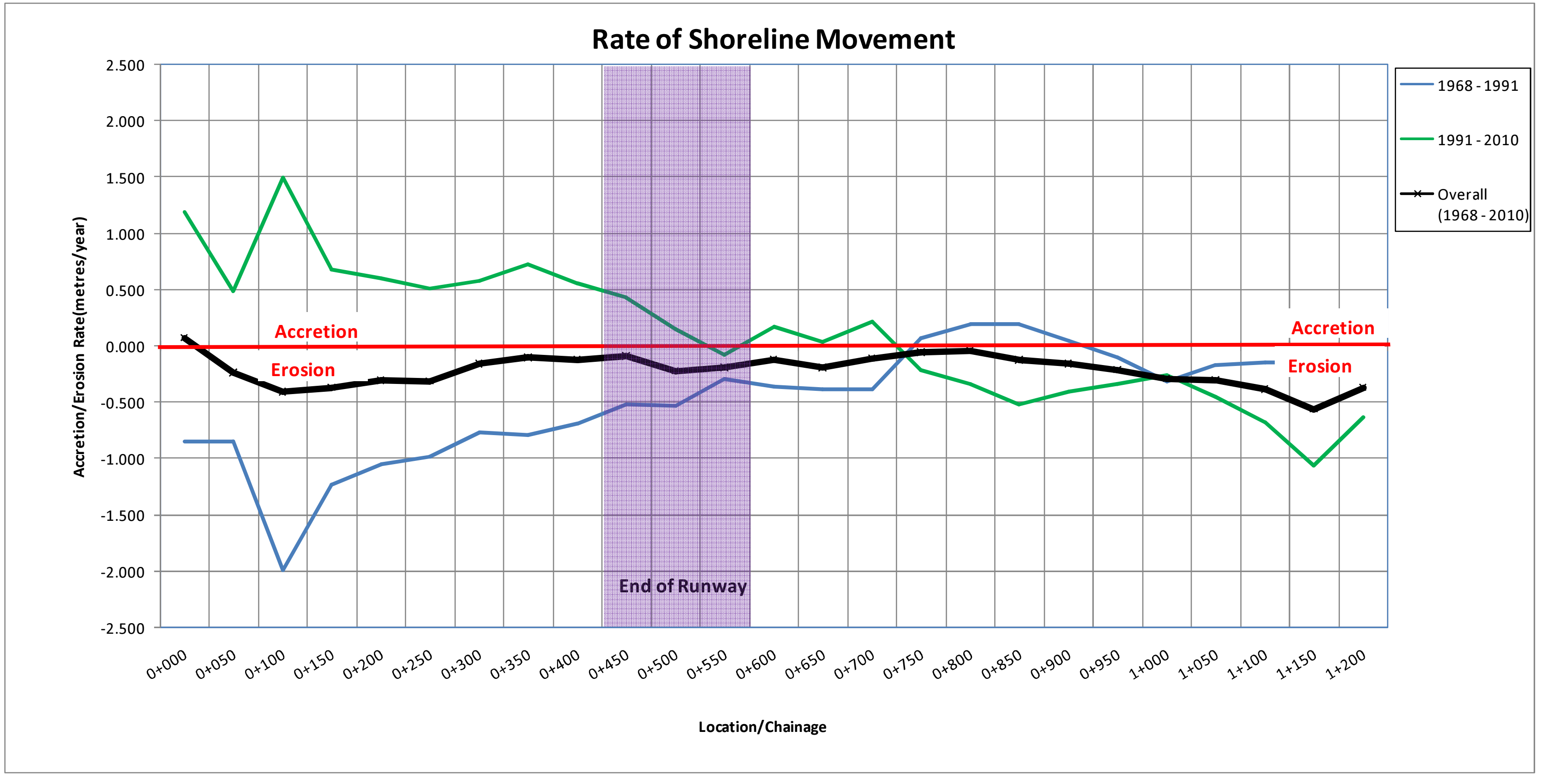


Figure 3.6 Graph showing the rates of erosion/accretion for the shoreline about the 1968 shoreline for different time intervals

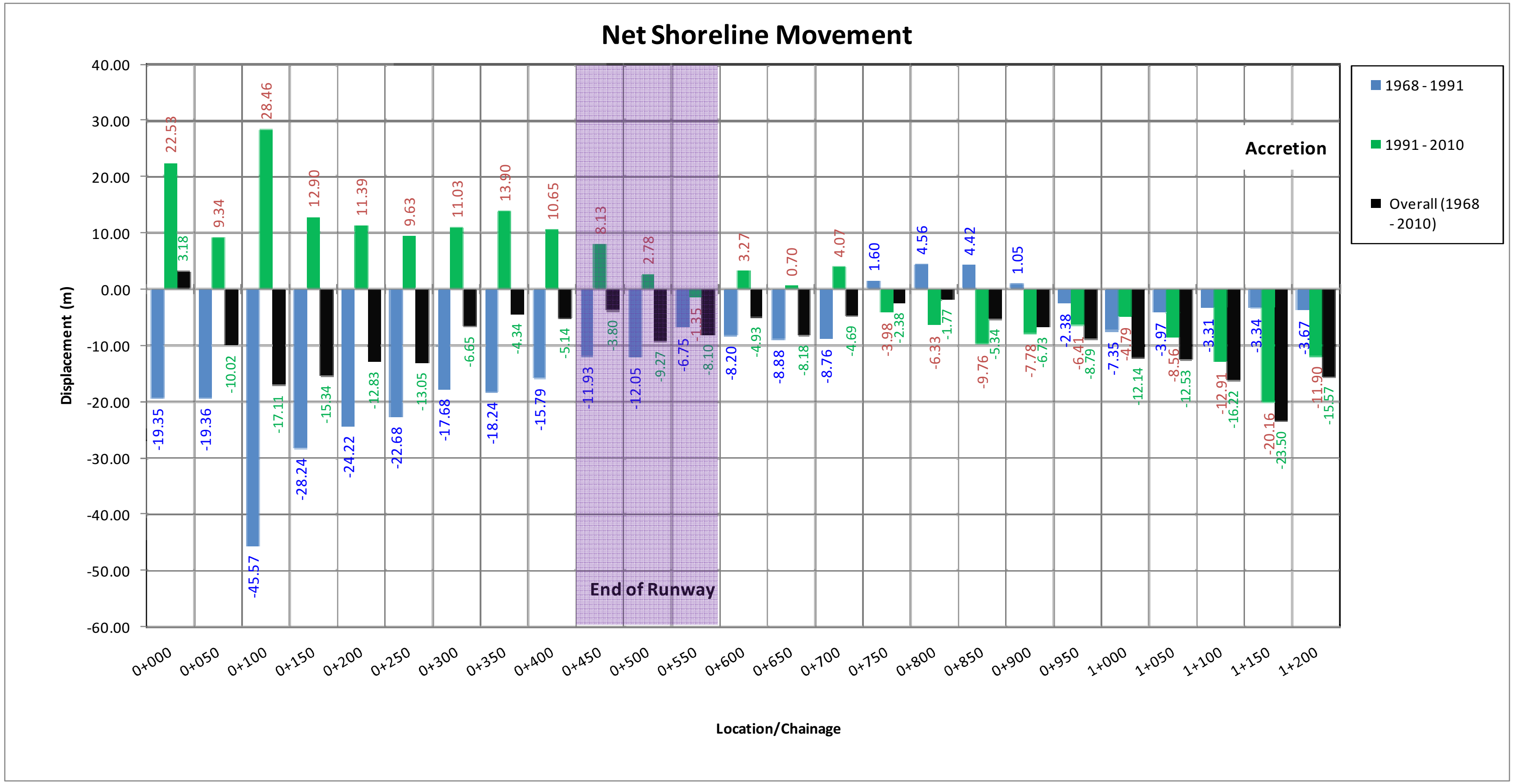


Figure 3.7 Graph showing the displacements of the shoreline for different years about the 1968 shoreline

3.2.1.2 Estimated Long-Term Recession rate (Global sea level rise component)

The Bruun model is perhaps the best-known and most commonly used of the models that relate shoreline retreat to sea level rise. This two-dimensional model assumes an equilibrium profile. Thus, it inherently assumes that the volume of sediment deposited is equal to that eroded from the dunes and that the rise in the nearshore bottom as a result of the deposited sediment is equal to the rise in sea level.

The original Bruun model is expressed below in Equation 5 1, and this mathematical relationship was the basis for estimating shoreline retreat within the study area.

Equation 3-1 – Bruun model

$$\Delta y = \frac{\Delta s \cdot l*}{h*}$$

Where:

Parameter	Description	Units
Δy	Dune line erosion	m
Δs	Rate of sea level rise. The Intergovernmental Panel on Climate Change's (IPCC) 2007 Report[2] estimate global sea-level to rise 2.3-4.7 mm/yr. The upper limit of 4.7mm/yr was utilized.	m
$l*$	Length of the offshore profile out to a supposed depth, h*, of the limit of material exchange from the beach and the offshore	m
$h*$	Depth at offshore limit of l*, to which nearshore sediments exist (as opposed to finer- grained continental shelf sediments) based on operational wave climate in the area determined from a previous study.	m

The results of this analysis showed the rate of shoreline retreat to vary between 0.14m and 0.21m as a result of global sea level rise. The estimated long-term erosion trends were compared with the historical trends to determine/estimate to what degree it has affected actual shoreline over the past 42 years and the project the estimated change over the 50 year design life of the structures

[2] IPCC Fourth Assessment Report: Climate Change 2007 Working Group II Report, "Impacts, Adaptation and Vulnerability" M.L. Parry, O.F. Canziani, J.P. Palutikof, P.J. van der Linden and C.E. Hanson (eds)

The Bruun model estimates that the long term erosion trends are as follows:

1. The shorelines are eroding at a rate of 0.14 to 0.21 metres per year
   a. The location west of the end of runway is eroding at a rate of 0.1 to 0.4metres per year
   b. The location to the south of the end of runway at a rate of 0.09 to 0.22 metres per year
   c. The location east of the end of runway is eroding at a rate of 0.057 to 0.56 metres per year
2. GSLR is estimated to be responsible for approximately 57% to 100% of observed erosion. In other words, the majority of the erosion trend observed is believed to due to long term processes.
3. The long term trend due to global sea level rise is expected to be in the order of 7.2 to 10.7 metres over the next fifty years.

Table 3.7 Summary of estimated long-term for the project area using Bruun Model

Parameters	**Profile**		
Estimation of shoreline recedance	**1**	**2**	**3**
Location	**0+200**	**0+600**	**1+025**
	Plumb Point	**Infront of runway**	**Close to round-about**
Annual Rate of sea level rise, Δs(m/year)	0.0047	0.0047	0.0047
Offshore profile, l* (m)	91	61	70
depth of offshore limit, h* (m)	2	2	2
Annual Dune line Erosion, Δy (m/year)	0.21	0.14	0.16
Cumulative Erosion 1968 - 2010 (m)	**8.98**	**6.02**	**6.91**
Projected change in 50 years 2010-2060(m)	**10.69**	**7.17**	**8.23**
Comparison to historical shoreline 1968 - 2010 (42 yrs)			
Bruun estimate (m)	8.98	6.02	6.91
Historical/observed (m)	12.83	4.93	12.14
Difference (m)	3.85	-1.09	5.23
Difference (%)	70%	122%	57%

3.2.2 Estimated Short-term Vulnerability

It was considered necessary to determine the erosion hazard of the 1.0km of shoreline considered to the 100 year return storm event. This was due to:

- The increased number of extreme storms events over the past 28 years as is evident in Table 3.1
- Anecdotal information pointing to erosions taking place on the beach during storm events.

Photos taken of the area just south of the runway after a storm show the extent to which erosion is possible at the shoreline. See Plate 3.1. It is evident from the plates that the storm surge and wave overtopping was sufficient to reach an elevation of 4.5 metres and to totally expose the buried conduits.

Plate 3.1 Photo taken of eroded area in front south of the end of the Runway after Hurricane Ivan looking towards the Lighthouse

Plate 3.2 Photo taken of eroded area in front south of the end of the Runway after hurricane Ivan looking towards the airport roundabout

3.2.2.1 Model Description and input

SBEACH is an empirically based numerical model for estimating beach and dune erosion due to storm waves and water levels. The magnitude of cross-shore sand transport is related to wave energy dissipation per unit water volume in the main portion of the surf zone. The direction of transport is dependent on deep water wave steepness and sediment fall speed. SBEACH is a short-term storm processes model and is intended for the estimation of beach profile response to storm events. Typical simulation durations are limited to hours to days (1 week maximum).

Profiles were cut from deepwater to land up to a maximum elevation of approximately 10 metres at three locations spanning the 1km area of interest along the shoreline. The wave data from the deep water hurricane model were utilized for this analysis. Table 3.8 shows the 100 year return period wave characteristics utilized in the model.

Table 3.8 Input parameters of wave characteristics for 100yr storm for SBEACH model

Locations	Direction	Input parameters		
		Hs	Tp	Storm duration (hrs)
1 - 3	ESE	8.1	14.1	12
	SE	7.7	13.7	12
	SSE	7.7	13.7	12
	S	6.2	12.4	12

3.2.2.2 Results

The estimated erosion prone areas along each profile line varied from 0 to 138 metres. Close observation of the results as presented in Table 3.9 shows the ESE profile to consistently be the worst direction at all three locations.

Table 3.9 Estimated erosion along each profile for NMIA/Palisadoes for 100 year return period event

Location	Profile	Erosion (m)	WS elevation (m)
	ESE	138	1.89
	SE	123	1.57
	SSE	114	1.63
1	S	86	1.56
	ESE	49	1.99
	SE	49	1.57
	SSE	23	1.57
2	S	72	1.4
	ESE	45	1.88
	SE	0	1.6
	SSE	0	1.52
3	S	0	1.47

The estimated erosion prone areas were plotted over a satellite image of the area. The plot reveals:

1. The entire stretch of shoreline is vulnerable to erosion varying from 10 to 83 metres
2. The area to the southwest of the runway is most vulnerable to erosion due to a 100 yr storm
3. The section of the main road immediately to the south if the runway is susceptible to failure due to erosion of the shoreline

One of the beacons used by approaching aircrafts is located in the area that is susceptible to erosion. The airport infrastructure as well as the road leading to Port royal are of national importance and should as far as is possible be protected from hurricanes.

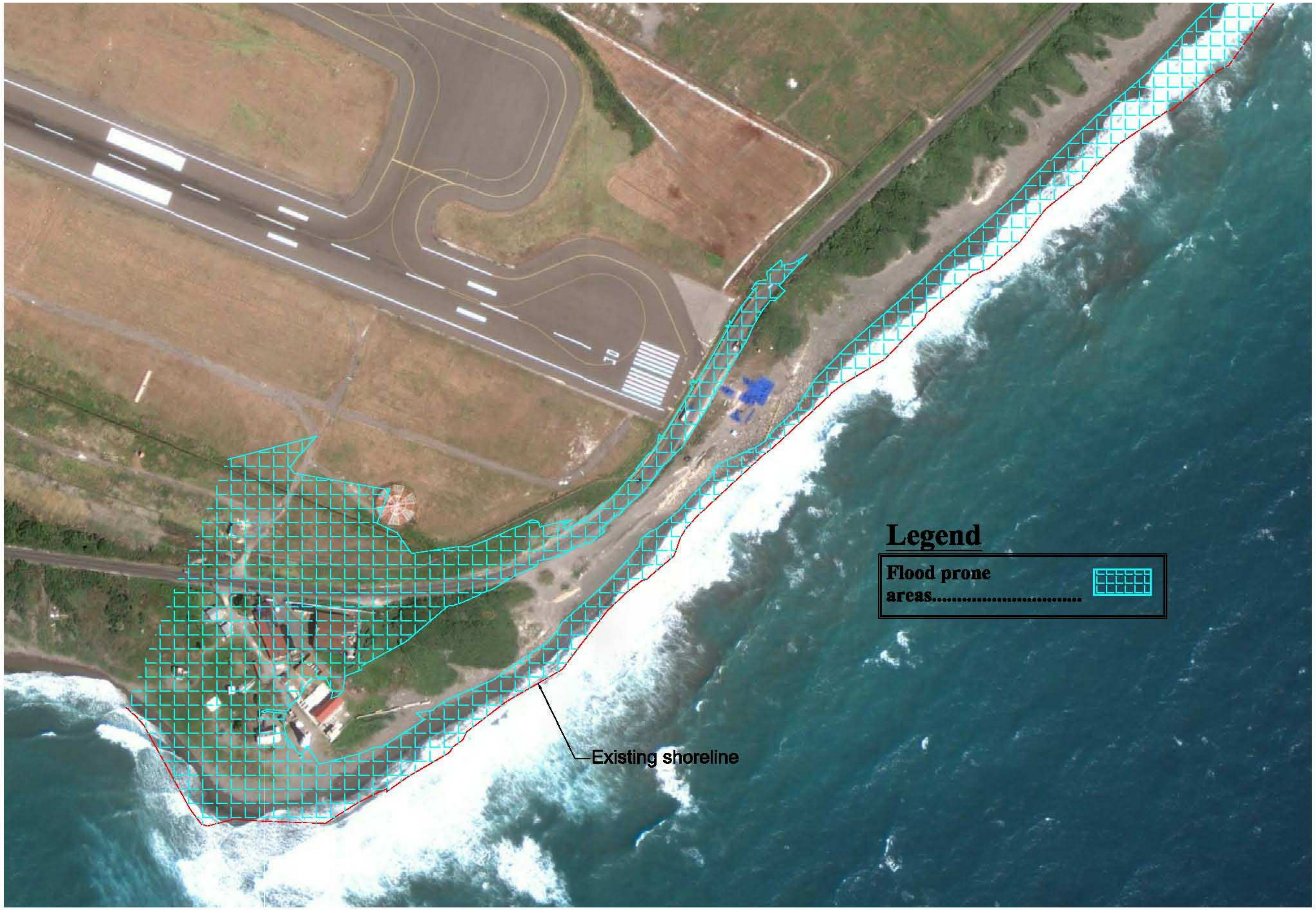


Figure 3.8 Areas next to the NMIA shoreline prone to flooding due to 100yr return hurricane storm surge

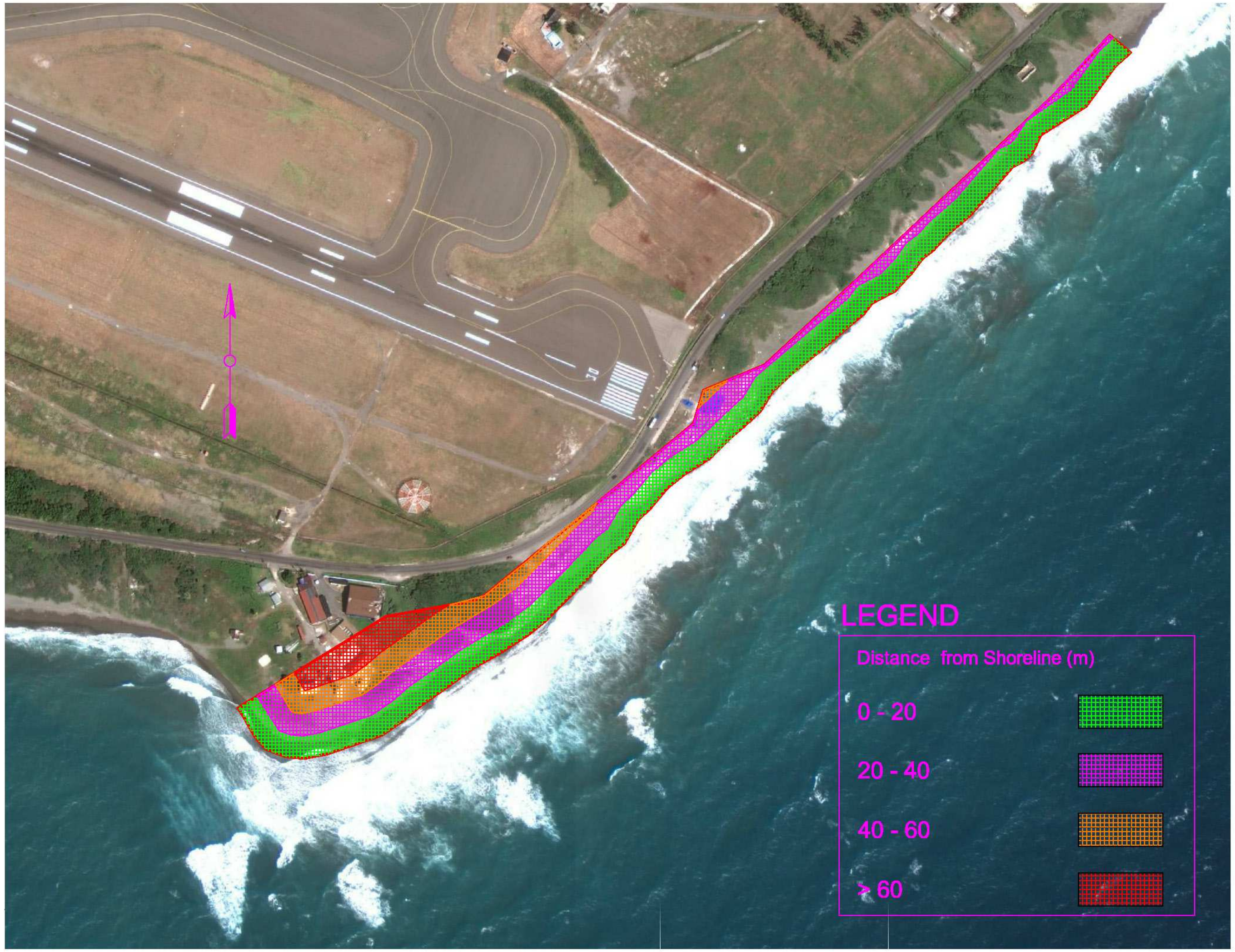


Figure 3.9 Erosion prone area based on 100yr storm results from SBEACH plotted on satellite image

4 Proposed Shoreline Protection Works

4.1 Design

4.1.1 Armour stone sizing

The Van der Meer Stability Equation as per the Rock Manual (2006) and Kamphius (2000)[3] was utilized to size the armour stones. The equation is valid for the estimation of the stability of armour stone for the trunk, head and toe of coastal structures, for both breaking and non-breaking wave conditions. See Equation 4 1 for the Van der Meer stability equation.

Equation 4-1

For plunging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7P^{0.18}\left(S/\sqrt{N}\right)^{0.2} 1/\sqrt{\xi_m}$$

For surging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4P^{-0.13}\left(S/\sqrt{N}\right)^{0.2} \sqrt{\cot\alpha}.\xi_m^P$$

where:

$H_{2\%}$ - two percent highest waves
Δ - relative density
P – Porosity of the structure
S – damage level
N – number of waves
ξ_m – surf similarity parameter
α – angle of seaward slope of structure

The design procedure for the sizing of the armour stone involved:

a) Estimating the at-toe wave height for the design deepwater wave conditions
b) Estimating the surf-similarity parameter, hence determine if the waves were breaking or non-breaking
c) Applying the design parameters to yield a recommended armour weight.
    a. Conducting a sensitivity analysis to determine the wave period at which the nominal size of the stone required is a maximum for the design wave.

[3] Kamphius (2000), Introduction to Coastal Engineering and Management, World Scientific

b. Applying this period along with the other parameters to determine the required size armour

The resulting design calculation for the armour structures is shown in Table 4.2. The design calculations revealed that a range of stone sizes from 4 Tonnes to 13 Tonnes are required to resist the 100 Year Return Period Design wave conditions.

Table 4.2 and Table 4.3 below shows the proposed gradation for the armourstones and filter stones required.

Table 4.1 – Structural design results for Revetment

ROCK ARMOUR STABILITY (NEARSHORE VAN DE MEER)			
	Project:	Palisadoes/NMIA Revetment	
	Location/Direction:	Palisadoes	
	Structure:	Revetment	
1.00	**DESIGN PARAMETERS**		**Units**
1.01	Porosity, P	0.4	
1.02	Structure slope	2	
1.03	Structure slope (TAN)	0.50	
1.04	Duration (hours)	6	
1.05	Number of waves	1543	
1.06	Nearshore slope	1.97%	
1.07	Offshore depth	20	
1.08	Nearshore depth	2	
1.09	Distance	915	
1.10	Angle of Seaward Slope (of structure)	31.3	
1.11	Density		
1.12	Armour stone	2500	Kg/m3
1.13	Seawater	1024	Kg/m3
1.14	Damage level	2.0	
2.00	**WAVES**	**Value**	**Units**
2.01	**Deepwater Wave**		
2.02	Length	306.0	metres
2.03	Height	6.0	metres
2.04	Period	14.0	seconds
2.05	**Shallow water wave characteristics**		
2.06	Assume a value for L1	85	metres
2.07	Wave length, L2 =	85	metres
2.08	Depth (MSL)	0.0	metres
2.09	Incremental Depth (MSL)-at one wavelength	1.7	
2.10	Nearshore Wave Height, Hs	3.9	metres
2.11	Hmax, H2%	5.4	metres
2.12	Steepness	0.045	
3.00	**DESIGN CONFIGURATION**		
3.01	Ibarren Number	2.346	
3.02	Ibarren comment	Plunging	
3.03	Submergence (MSL) pre storm	0.0	metres
3.04	Height of structure (before storm surge)	0.00	metres
3.05	Storm surge elevation (above MSL)	2.20	metres
3.06	Depth of water (during storm surge)	3.87	metres
4.00	**ARMOUR SIZING**	**Value**	**Units**
4.01	**Calculations**		
4.02	Delta	1.44	
4.03	Ns*	2.65	
4.04	**Dn50**	1.42	**metres**
4.05	D50-Toe	1.14	
4.06	Rc	2.20	metres
4.07	Reduction factor for submergence (Rd)	0.98	
4.08	**D50 (Rd)**	**1.39**	**metres**

Table 4.2 – Gradation of armour stones

	7.1 TONNES ARMOUR STONES (QUARRY STONES)	
NOMINAL SIZE (TONNES)	7.1	
PERCENTAGE FINER THAN	MASS (KG)	NOMINAL DIMENSION (M)
2%	3.9	1.16
15%	4.6	1.23
50%	7.1	1.42
85%	11.0	1.64
97%	12.8	1.73

Table 4.3 Gradation of Filter Stone

	300 KG FILTER STONES		600 KG FILTER STONES	
NOMINAL SIZE (KG)	300		600	
PERCENTAGE FINER THAN	MASS (KG)	NOMINAL DIMENSION (M)	MASS (KG)	NOMINAL DIMENSION (M)
2%	152	0.39	304	0.50
15%	183	0.42	365	0.53
50%	300	0.49	600	0.62
85%	493	0.58	986	0.73
97%	584	0.62	1169	0.78

Figure 4.1 – Proposed revetment Cross sections

4.2 Design Cross Section

A revetment consists of various parts as can be seen in Figure 4.3. The primary armour is layers provide the function to resist the forces of the waves and keep the structure in place. The secondary armour provides a transition from the primary armour to the corefill or subgrade. It also functions to reduce the porosity of the structure thereby limiting transmission of water through the structure which will erode the subgrade and cause the structure to fail. The core and toe may be based on filter layers which also protect sand coming through the rock which may lead to settlement of the structure. It is therefore necessary to check the filter size.

4.2.1 Filter Criteria

The Design of the structure is such that it allows the transmission of water through the structure. The filter material has larger particle sizes than the subgrade which is being protected from erosion. It is therefore possible for the subgrade to leach through the filter and reduce the stability of the structure. The USACE has after many years of experience with these designs, developed a stability criteria which allows the transmission of water through the structure while preventing the leaching out of the finer subgrade through the filter. The criteria to limit the transmission of the lower layer of finer particle size into an upper layer of larger particle sizes is $D15_{upper\ layer}/D85_{lower\ layer}<5$. An analysis was conducted on the interface between the proposed filter (600kg and 300kg) and the existing subgrade. They both failed the criterion at all locations checked. This has given the need for the insertion of a fill layer between the filter and the subgrade which will meet the criteria at both interfaces. The D85 and D15 were determined for the fill material based on the filter material and the subgrade respectively. The minimum D85 determined is 125mm (5 inches) and the maximum D15 is 9mm.

For construction purposes however the proposed gradation for use will be such that it will meet the stability criteria and will be able to be compacted onsite. The proposed gradation is as follows:

1. Maximum particle size of 150mm
2. 60% to 95% finer than or equal to 100 mm
3. 30% to 60% finer than or equal to 50 mm
4. Maximum of 30% passing the 4.75 mm sieve

Table 4.4 Summary of the analysis to determine the fill material size require below the structure

Layer	Location	558	534	526	512	519
Subgrade	D50 (mm)	1.3	0.6	0.7	0.5	1.5
	Classification	very coarse sand	coarse sand	coarse sand	coarse sand	very coarse sand
	D85 (mm)	1.9	0.8	1.0	0.8	2.3
Filter (300kg)	D15 (mm)	420.0	420.0	420.0	420.0	420.0
Filter (600kg)	D15 (mm)	530.0	530.0	530.0	530.0	530.0
	D15(mm)	9.0	9.0	9.0	9.0	9.0
Proposed fill	D85 (mm)	125.0	125.0	125.0	125.0	125.0
Test (Subgrade)	$D15_{filter(0.3T)}/D85_{subgrade}$	217.6	517.6	420.8	556.9	183.5
		fail	fail	fail	fail	fail
	$D15_{fill(0.6T)}/D85_{subgrade}$	274.54	653.16	530.98	702.72	231.52
		fail	fail	fail	fail	fail
Test (Proposed fill)	$D15_{filter(0.3T)}/D85_{fill}$	4.24	4.24	4.24	4.24	4.24
		pass	pass	pass	pass	pass
	$D15_{filter(0.6T)}/D85_{fill}$	3.36	3.36	3.36	3.36	3.36
		pass	pass	pass	pass	pass
	$D15_{fill}/D85_{subgrade}$	4.7	4.7	4.7	4.7	4.7
		pass	pass	pass	pass	pass

4.2.2 Layer Thicknesses

The number of layers of stones required as first determined based on the porosity of the structure. The rock manual[4] recommends two layers of primary armour and 1.5 layer of filter/secondary armour for a porosity of 0.4. This requirement is for the front face of the structure and does not specify any differentiation for the back face. See Table 4.5 below. The proposed cross-section will use the 2 layers of armour of nominal/median size for the front face and one layer of the top end armourstones for the back face.

[4] Manual on the use of rock in coastal and shoreline engineering Construction Industry Research and Information association (CIRIA) Centre for Civil Engineering Research and Codes, 1991

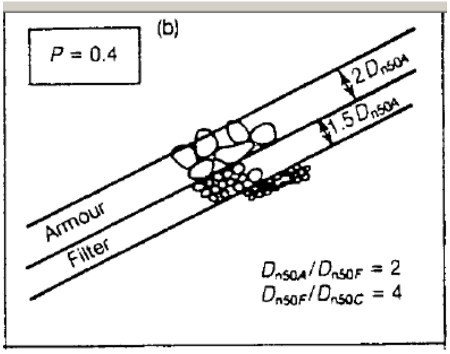


Figure 4.2 Armour and filter requirements stipulated in the Rock manual

The CIRIA rock manual also proposes thickness for the armour layer based on the packing method. The method of packing the layers will affect the porosity of the structure. The thickness of layers is given by:

t_a= t_u= t_f= n k tDn50, where:

t_a, t_u, t_f = thickness of armour, under layer or filter

n = number of layers
kt = layer thickness coefficient

Table 4.5 Table taken from the rock manual which shows the thickness coefficients for stone the packing methods and stone shapes

Table 20 Thickness and porosity in narrow gradation armour layers

Shape spec. or description	Placement	Above/below water	Layer coeff. k_t	Porosity (%) n_v
Irregular	(b)+(d)	Above	1.20	39
	(b)+(e)	Above	1.05	39
	(c)+(f)	Above or below	0.75	40
Semi-round	(b)+(d)	Above	1.25	36
	(b)+(e)	Above	1.10	36
	(c)+(f)	Above or below	0.75	37
Equant	(b)+(d)	Above	1.15	37
	(b)+(e)	Above	1.00	37
	(c)+(f)	Above or below	0.80	38
Very round	(b)+(d)	Above	1.20	35
	(b)+(e)	Above	1.05	35
	(c)+(f)	Above or below	0.80	36

Notes
1. Shape descriptions refer to the block population classifications of Table 11
2. Placement techniques and/or their implied effects
   (b) Section 12.3 of Appendix 1
   (c) Random placement to achieve a high porosity for wave energy dissipation
   (d) Long axes of elongate, tabular and irregular blocks to be placed normal to slope
   (e) Long axes of elongate, tabular and irregular blocks to be placed upslope with short axes along slope
   (f) Long axes of elongate, tabular and irregular blocks may take any orientation

The summary of the estimated layer thickness is shown below in Table 4.6 for irregular stones packed with the long axis perpendicular to the structure face.

Table 4.6 Summary of minimum armour thicknesses

	Filter (300kg)	Filter (600kg)	irregular amour (4-13T)
W50 (Ton)	0.3	0.6	7
weight density	2.56	2.56	2.56
Dn50 (m)	0.49	0.62	1.40
Kt	0.87	0.87	0.87
n	1.5	1.5	2
Dn50	0.49	0.62	1.40
ta	0.64	0.80	2.43

4.2.3 Overtopping Analysis

The physical model report also highlighted the dangers posed to the road if the structure is overtopped by waves plunging on the structure. The elevation of the crest is directly related to the volume of water that will be discharged unto the road when the structure is overtopped.

The worse case storm scenario was investigated for overtopping of the proposed structure and the existing structure. The existing structure considered is single layered with a roughness coefficient of 0.6; whilst the proposed structure has a roughness coefficient of 0.55. The wave height arriving at the shoreline was determined from the REFDEF model plots to be 2.5m.

The analysis indicates:

- The existing structure will be overtopped with a discharge of 0.33m^3/m
- The proposed structure will be overtopped at 6.39m crest elevation with a discharge of 0.09m^3/m which is with within the safe limits for preservation of the road.
- The proposed structure will be overtopped at 5.49m crest elevation with a discharge of 0.19m^3/m which is with within the safe limits for non-critical areas.

The 6.4m elevation is recommended for use as it will limit the amount of water that will be discharged over the structure and as a result will erode the road structure.

Table 4.7 – Summary of Overtopping analysis at the end of the NMIA Runway

Direction	**ESE**	**ESE**	**ESE**	
Location	**0+600**	**0+600**	**0+600**	
	End of runway	**End of runway**	**End of runway**	
Hs	2.50	2.50	2.50	JONSWAP/100 Return Period
Water elevation+setup	2.39	2.39	2.39	
Overtopping calculations				
A	9.39E-03	9.39E-03	9.39E-03	
B	21.6	21.6	21.6	
Roughness Coefficient, r	0.6	0.55	0.55	Two layers of armour stone
Freeboard., R	2.60	4.01	3.10	M
Tp	15.10	15.10	15.10	SEC
Tm	9.82	9.82	9.82	SEC
R*	0.05	0.08	0.06	
Q*	0.00	0.00	0.00	
Q	**0.33**	**0.09**	**0.19**	M3/S.M
Design Crest elevation	**4.99**	**6.40**	**5.49**	M

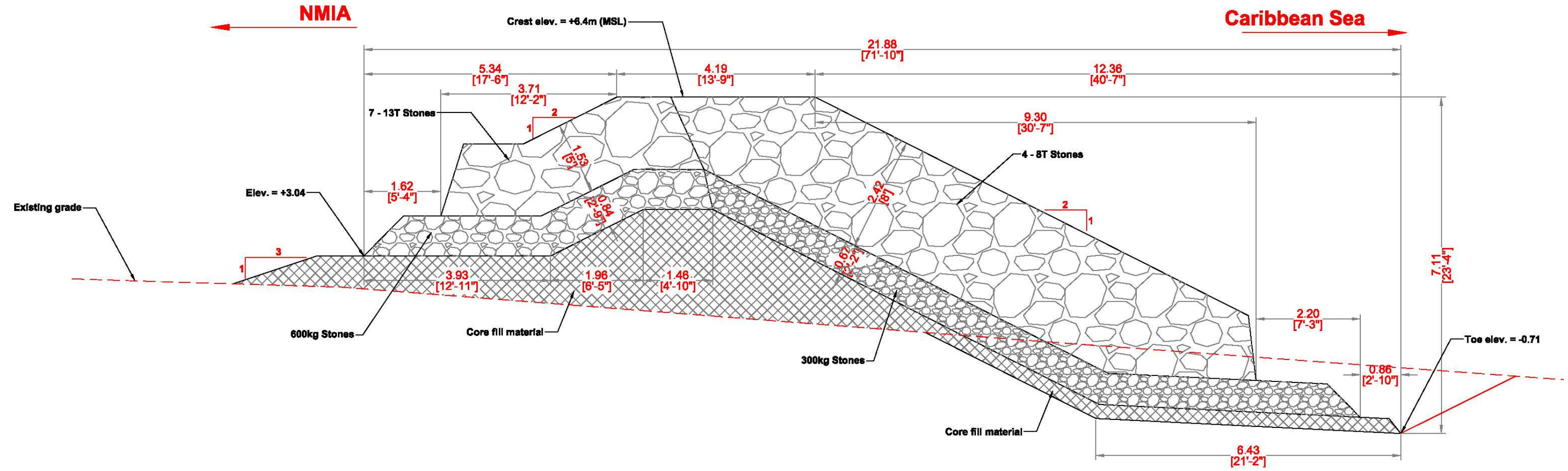


Figure 4.3 Typical cross section of the resulting revetment

4.3 Cost estimate

The estimated cost for the construction of the 1,105 metres of revetment is US $15,591,745.43 . These figures include a contingency of 10% and Institutional Strengthening. The project design and monitoring costs are expected to be US $801,628.54 . This includes considerations for final design, environmental studies, permitting and monitoring. The total project cost is US $16,393,373.97

4.4 Bill of Quantities

NMIA END OF RUNWAY SHORELINE PROTECTION					
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**AMOUNT (USD)**
1.00	**PRELIMINARIES**				
1.01	General Provisions				
1.02	Supply and establish (with fencing and furniture)				
1.03	Contractor's Site Office and Work Shop	SUM	0	$125,000.00	$ -
1.04	Resident Engineer (400 sq. ft)	SUM	0	$ 31,200.00	$ -
1.05	Mess room and sanitary accomodation (2,000 sq. ft. with chain link fence, canteens, eating area, with tables and benches, four washrooms). Include for four potable toilets along work area with wash stands	SUM	0	$ 75,500.00	$ -
1.06	To maintain (cleaning, electricity, air conditioning, water, etc.)				
1.07	Contractor's Site Office	MTHS	6	$ 4,250.00	$ 25,500.00
1.08	Resident Engineer	MTHS	6	$ 3,195.00	$ 19,170.00
1.09	Mess room and sanitary accomodations	MTHS	6	$ 4,000.00	$ 24,000.00
1.10	Safety, Health and Welfare: Personal equipment; First Aid Facilities, for duration of contract	Sum	1	$120,000.00	$ 120,000.00
1.11	Water and lighting for the Works, include for dust control	MTHS	6	$ 7,500.00	$ 45,000.00

1.12	Allow for the covering up and protection of works throughout the contract period	NR	3	$ 7,200.00	$ 21,600.00
1.13	Removal of rubbish and debris during and after the contract	MTHS	6	$ 7,200.00	$ 43,200.00
1.14	Supervision of works				
1.15	Personnel	MTHS	6	$110,000.00	$ 660,000.00
1.16	Welfare, accomodation/housing, Communication, transportation including maintenance	MTHS	6	$ 30,000.00	$ 180,000.00
1.17	Bonds and Insurance				
1.18	Tender Bond (of 10% of works from Bank)	Sum	1	$ 12,900.00	$ 12,900.00
1.19	Performance Bond (of 100% of works from Bank)	Sum	1	$300,000.00	$ 300,000.00
1.20	Insurance: Third Party, Public Liability, Employers Liability and Contractors Equipment for the duration of the contract (US$5.0 Million exposure)				
1.21	Contractor's All Risk, inclusive of contractor's equipment and works	SUM	1	$300,000.00	$ 300,000.00
1.22	Public Liability (USD$1.0 Million)	SUM	1	$ 30,000.00	$ 30,000.00
1.23	Employers' Liability (USD$0.5 Million)	SUM	1	$ 30,000.00	$ 30,000.00
1.24	Quality Control Plan	NR	1	$ 30,000.00	$ 30,000.00
1.25	Turbidity Barriers				
1.26	To supply, fabricate maintain as per Specifications	PS	0	$100,000.00	$ -
1.27	To place/deploy and remove as per Specifications as needed	PS	1	$ 80,000.00	$ 80,000.00
1.28	Progress Report	MTHS	6	$ 1,500.00	$ 9,000.00
1.29	Traffic Management and Control				
1.30	Traffic Management Plan	SUM	1	$ 3,000.00	$ 3,000.00
1.31	Signs and traffic controls				

1.32	During contract period: Signs and traffic controls (including temporary lighting, flagmen, temporary traffic lights)	MTHS	6	$ 2,100.00	$ 12,600.00
1.33	Signs (to NWA specifications)	SUM	1	$ 4,000.00	$ 4,000.00
1.34	Surveying				
1.35	Initial topographic survey	NR	1	$ 19,500.00	$ 19,500.00
1.36	Setting out, checks on elevations and as-built	MTHS	6	$ 7,500.00	$ 45,000.00
1.37	Testing				
1.38	Armour Stone	SUM	1	$ 44,210.00	$ 44,210.00
1.39	Road Works				
1.40	Spread rate test	SUM	1	$ 8,550.00	$ 8,550.00
1.41	Proctor density	SUM	1	$ 8,550.00	$ 8,550.00
1.42	Compaction	SUM	1	$ 8,550.00	$ 8,550.00
1.43	Proctor density	SUM	1	$ 8,550.00	$ 8,550.00
1.44	All other road works tests as specified	SUM	1	$ 4,275.00	$ 4,275.00
	SUB-TOTAL				**$ 2,097,155.00**
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**AMOUNT (USD)**
2.00	**EARTHWORKS**				
2.01	GRUB: Clear footprint of light bushes, small trees of girth n/e 630 mm, grub up roots and dispose of from revetment and road to approved Landfill (within a 15 miles radius from proejct site).				

2.02	0+000 to 1+105.84 (+6.7m Revetment)	m^2	44,743.3	$ 6.00	$ 268,459.70
2.03	To Demolish existing way leave (consisting of reinforced concrete, masonry structures, cart away)				
2.04	Existing cross drain	NR	1	$ 6,000.00	
2.05	EXCAVATION: For base of revetments limits of footprint to reduce levels for subgrade. Place over revetement or in core as directed by engineer. Cart away or dispose of excess excavated material to landfill.				
2.06	0+000 to 1+105.84 (+6.7m Revetment)	M3	33,202	$ 9.00	$ 298,817.75
2.07					
2.08	**To scarify existing asphalt surface and base course, as dispose of or incoproate into the works**				
2.09	0+000 to 1+700 (rounda-about)	m^2	13,600	$ 2.04	$ 27,696.63
	SUB-TOTAL				**$ 594,974.08**
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**AMOUNT (USD)**
3.00	**DRAINAGE WORKS**				
3.01	To supply, place and install Corrugated HDPE pipe by excavating for trench in core (no greater than 2.0m), supply and place minimum 150mm sand for bedding surround and cover to pipe and backfill in core, with geotextile to armour stones and restoration of road fill/core				
3.02	24" diameter	m	20.0	$ 300.00	$ 6,000.00
3.03	Box drain (1.2m wide x 1.5m deep) to NMIA side, inclusive of reinforcement, excavation, fill to base, masonary, fair face finish to internal walls and top edge	m	500	$ 320.00	$ 160,000.00
3.04	V-Channel: Excavate to grade (concrete lining 150mm thick, with JRC126)				
3.05	2000mm W x 330mm D	m^2	270	$ 80.00	$ 21,600.00
3.06	To supply, place and install 1.2m x1.2m Reinforced concrete drop inlet box (1.0m deep) with 600mmx600mm CAST IRON metal grating to drop inlets at end of V-Channels and start of culvert to Harbour side at	Nr.	1	$ 2,840.00	$ 2,840.00

	specified locations				
	SUB-TOTAL				**$ 190,440.00**
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**AMOUNT (USD)**
4.00	**COASTAL WORKS**				
4.010	**SHORELINE PROTECTION**				
4.020	600kg: To supply, truck to site, store, wash, transport to work area, place and shape, as per Specifications and drawings				
4.030	0+000 to 1+105.84 (+6.7m Revetment)	m^3	6,162.5	$ 85.00	$ 523,811.65
4.040	300kg:To supply, truck to site, store, wash, transport to work area, place and shape, as per Specifications and drawings				
4.050	0+000 to 1+105.84 (+6.7m Revetment)	m^3	11,684	$ 85.00	$ 993,133.68
4.060	7-13ton:To supply, truck to site, store, wash, transport to work area, place and shape, as per Specifications and drawings				
4.070	0+000 to 1+105.84 (+6.7m Revetment)	m^3	8,190	$ 180.00	$ 1,474,115.40
4.080	4-8ton: To supply, truck to site, store, wash, transport to work area, place and shape, as per Specifications and drawings				
4.090	0+000 to 1+105.84 (+6.7m Revetment)	m^3	34,240	$ 160.00	$ 5,478,345.47
4.100	Core material: To supply, truck to site, store, transport to work area, place, compact and shape, as per Specifications and drawings				
4.110	0+000 to 1+105.84 (+6.7m Revetment)	m^3	16,211	$ 40.00	$ 648,429.58
	SUB-TOTAL				**$ 9,117,835.78**
NR	**DESCRIPTION**	**UNIT**	**QUANTITY**	**RATE**	**AMOUNT (USD)**

5.00	**ROADWORKS**				
5.01	Core Material/Fill for road. Compacted in lifts not exceed 0.2 m for fill above the water line. Varying in elevations for finished crest between 2.4 to 3.0 metres above MSL and depths of -3.5 to 0.0				
5.02	0+200 to 0+780	m^3	4,820	$ 35.00	$ 168,700.00
5.03	Base (150mm) and sub-base (150mm) to fill for road: Supply, spread and grade sub-base course and base course of approved graded material. Roll surface of base with a vibratory roller to a minimum CBR of 80 and density of 95% Density				
5.04	0+200 to 0+780	m^3	1,446	$ 35.00	$ 50,610.00
5.05	Hard shoulder and remainder of reservation (19.4 m - 7.62 m - 0.73 = 11.05 m) wide double surface dressing (5/8" and 3/8" aggregates for construction period), to finish 25mm				
5.06	0+200 to 0+780	m^2	1,446	$ 12.00	$ 17,352.00
5.07	DRIVING LANES: Supply and apply 100mm Asphaltic concrete (2X3.6M LANES) to final grade (in a minimum of two layers) using appropriate paving machines, rolled to falls and cambers in accordance with the specification. Include for primer (MCO) and tack coat	m^2	3,470	$ 45.00	
5.08	ROAD MARKINGS (inclusive of sides, lay bys, centre-line and reflectors)				
5.09	Continuous line 100mm wide	M	482	$ 3.00	$ 1,446.00
5.10	Broken line 100mm wide (WITH GAPS)	M		$ 1.50	$ -
5.11	Parking areas (CONTINUOUS LINES)	M		$ 3.00	$ -
5.12	Curb and channel: Mass Concrete as per drawings to edge of road, parking areas	M	482	$ 41.10	$ 19,809.47
5.13	Reflector: supply after approval by Engineer, install to pavement and replace during maintenance period				
5.14	Bi-directional	NR	96	$ 4.00	$ 385.60
5.15	Uni-directional	NR	96	$ 4.00	$ 385.60
	SUB-TOTAL				**$ 258,688.67**

NR	DESCRIPTION	UNIT	QUANTITY	RATE	AMOUNT (USD)
6.00	**PROVISIONAL SUMS and DAYWORKS**				
6.01	**Provisional Sums: To be expended as directed by the engineer**				
6.02	**UTILITIES**				
6.03	**Power and Telecommunication**				
6.04	Underground conduits and manholes, no allowance for conductors	M	620	$ 965.22	$ 598,434.78
6.05	**Water Supply**				
6.06	12" DI main to Port Royal	M	620	$ 571.43	$ 354,285.71
6.07					
6.08	**Day Works**				
6.09	The contractor will be reimbursed as defined below for the cost dayworks in accordance with Condition of Contract. Insert below the percentage addition as required to each section of the prime cost for overheads and profit.				
6.10	Provided in respect of the Prime Cost of Labour	Sum	1	$100,000.00	$ 100,000.00
6.11	Percentage addition for overhead and profit for Labour	%	350,000	$ 0.25	$ 87,500.00
6.12	Provided in respect of the Prime Cost of Material	Sum	1	$100,000.00	$ 100,000.00
6.13	Percentage addition for overhead and profit for Material	%	350,000	$ 0.25	$ 87,500.00
6.14	Provided in respect of the Prime Cost of Plant	Sum	1	$100,000.00	$ 100,000.00
6.15	Percentage addition for overhead and profit for Plant	%	350,000	$ 0.25	$ 87,500.00
	SUB-TOTAL (To summary)				**$ 1,515,220.50**

	SUMMARY				
	BILLS				
1.00	PRELIMINARIES				**$ 2,097,155.00**
2.00	EARTHWORKS				**$ 594,974.08**
3.00	DRAINAGE WORKS				**$ 190,440.00**
4.00	COASTAL WORKS				**$ 9,117,835.78**
5.00	ROADWORKS				**$ 258,688.67**
6.00	PROVISIONAL SUMS and DAYWORKS				**$ 1,515,220.50**
7.00	INSTITUTIONAL STRENGHTENING				**$ 400,000.00**
	SUB-TOTAL				**$ 14,174,314.03**
	CONTINGENCIES (10% OF SUB-TOTAL)				**$ 1,417,431.40**
	GRAND TOTAL				**$ 15,591,745.43**

5 Rapid Environmental Assessment

5.1 Description of Existing Floral and Fauna

The following is a report on a vegetation survey that was conducted on Wednesday, March 2, 2011. The purpose was to carry out a preliminary assessment of the dune vegetation along the seaward side of the Palisadoes Road in the vicinity of the Norman Manley International Airport (NMIA) runway and Palisadoes Lighthouse. The information garnered from this exercise was to help characterise the vegetation present as well as to determine the potential impact to any endemic or ecologically important flora resulting from the proposed erection of coastal reinforcement works (revetments) along this segment of coastline.

Coastal vegetation, apart from providing a habitat for local fauna, also serves a mechanical function: that of sand stabilization and dune reinforcement. The coastal strand community on-site could be characterised into three main successional categories. Firstly, the strand-beach component (Asprey & Robbins 1953) is a pioneer associes that typically begins above the tidal limits. It is normally located upon mobile or potentially mobile sand where salinity levels are highest. This zone was dominated by runners such as *Ipomoea pes-caprae* ssp. *brasiliensis* (Beach Morning Glory), *Sessuvium portulacastrum* (Seaside Purselane) and *Sporobolus virginicus* (figures 1 & 2). The legume *Canavalia maritima* (Seaside Bean) was also an occasional constituent.

Figure 5.1Seaward view of strand-beach associes.

Figure 5.2: Typical strand-beach community found on-site with permanent structure at right

The strand-dune component followed. It is usually characterised as being a denser and predominantly herbaceous community, occurring on fixed dunes (seaward side), emerging from the previous (pioneer) zone (Asprey & Robbins 1953). The common species found in this zone, at the study area, were *S. virginicus*, Seaside Purselane, *Melochia crenata*, *Stigmaphyllon emarginatum* and *Alternanthera ficoidea* (Crab Withe) (figure 3). *Calotropis procera* (French Cotton) was a conspicuous member found atop the dunes (figure 4).

Figure 5.3: Strand-beach component transitioning into stand-dune associes

Figure 5.4: Strand-dune associes with *C. procera* (French Cotton) plants (background)

The final component found was the strand-scrub associes. Development of this zone tends to be limited due to insufficient land area and anthropogenic disturbance (Asprey & Robbins 1953). The make-up of these communities along the south coast tends to be similar to a cactus thorn-scrub (Asprey & Robbins 1953). At the study-site, this zone was found on the landward side of dunes: near and alongside the main road. The average canopy height was between 1 – 1.5 m and tended to be dominated by the thorny shrub *Acacia tortuosa* (Wild Poponax) (figures 5 & 6). Other common constituents of this community also included the French Cotton and Crab Withe in addition to *Leucaena leucocephala* (Lead Tree) and *Pithecellobium unguis-cati* (Privet). Cacti such as *Stenocereus hystrix* (Dildo Pear) and the endemic, *Opuntia tuna* (Tuna) (figure 7) were rare constituents.

Figure 5.5: Sand-scrub community with the shrubby trees *A. tortuosa* (Wild poponax) and *L. leucocephala*.

Figure 5.6: Sand-scrub community dominated by *A. tortuosa* (Wild poponax)

Figure 5.7: The endemic cactus *O. tuna* (Tuna)

5.2 Assessment of Possible Impacts and Mitigation Measures

The following points are the identified impacts and the mitigation measured needed:

1. The proportion of land occupied by the **vegetation** of the three successional communities that were found to occur within the study area could be estimated as follows (seaward to landward):
   - Strand-beach - 10-15 %
   - Strand-dune - 20-30 %
   - Strand-scrub - 50-60%
2. Taking into consideration the footprint of the proposed revetments and 10 m buffer work area:

- o The associes to be most impacted would be the strand-beach constituent. As this is a pioneer community located closest to the tidal edge, its diversity was lowest. It was constituted by plants (runners) that tend to rejuvenate readily, with the strand-dune community appearing to be their point of origin or refuge. These plants assist in stabilising shifting/windswept sand. The proposed revetment may help in this process although it may also limit the recovery of the vegetation here which prefers to grow over sandy substrate.
- o A small fraction of the strand-dune associes appears to occur in the buffer area and as such would be the next community to be affected, albeit significantly less than the strand-beach associes. Floral diversity was greater here and they served to stabilise the seaward face and tops of the sand-dunes present. Greater care should be exercised here (in terms of vegetation removal) however f the 10 m buffer is enforced the vegetation should recover.
- o The strand-scrub associes had the highest floral diversity of the three and according to your layout, should be the least affected. However, it is the community through which access to the shoreline may be necessary. The vegetation here served to support the rear-dune and would be the community that would be most difficult to recover. It is also the community that possessed the endemic cactus As such, my suggestion would be that access to the site be limited to pre-existing access-points and/or areas where the vegetation density is least or narrowest. No temporary structures should be placed here.

3. Only 1 endemic was found, it can be concluded that endemism **within the study area <u>appears</u>** very low.

6 Conclusions and Recommendations

6.1 Conclusions

The following conclusions can be drawn from the study conducted to date:

6. The wave refraction analysis clearly indicates that the project shoreline is most vulnerable to hurricane waves from the east and ESE. In both scenarios, 5 to 6 metre waves are expected some 500 metres offshore and 2.5 metres waves are expected at the shoreline. The waves appear to be focused from offshore bathymetric features such as submerged cays or mounds. The 100 year conditions from S and SSE waves are less severe.
7. Satellite imagery and aerial photograph analysis of images from 1968 to 2010 indicate a long-term trend of erosion of 1.7 to 23.0 metres in the last 42 years. Global Sea Level rise analysis (using Bruun Rule) indicates that 57% to 100% of this can be explained by sea level rise.
8. Short-term vulnerability for the shoreline in the 100 Year event indicates that erosion losses of can range from 10 to 83 metres from a single event. This estimate is supported by observations after Hurricane Ivan which was a 1 in 90 year hurricane.
9. The proposed works are estimated to cost US $16,393,373.97
10. The occurrence of endemic plants within the project is very low. The implementation of the proposed protection structures should cause minimal disturbance to the dominant flora and fauna on the shoreline.

6.2 Recommendations

The following are our recommendations:

1. A revetment 1,105 metres long, with 4 to 13 Tonne armour stone and a crest elevation of 6.4 metres above Mean Sea Level is necessary to resist the 100 Year Return Period wave conditions at the eastern end of runway for the Norman Manley International Airport. See proposed works in drawings:
    a. CEAC-2011-02-001
    b. CEAC-2011-02-002
    c. CEAC-2011-02-003
    d. CEAC-2011-02-004

   The section of revetment from the end of the runway to the Plumb Point Lighthouse is most in need of this level of protection. Whilst the eastern end (0+740 to 1+105) is just as vulnerable the consequences of damage and the likely lost of shoreline is expected to be less. Consideration can be given in the final design stage to reducing the crest elevation to 5.49m for cost savings in this eastern area.
2. Consideration should be given in the final design stage to raising the road level to a minimum of 2.3m above Mean Sea Level. Storm water drainage improvements should also be made to redirect storm water flows from the existing low point into the mangroves.
3. Pending successful project financing the following necessary activities are envisaged:
    a. Further detailed site specific wave modeling for 50 and 100 Year Return Period wave climate. Additional structural design and overtopping analysis with a view to reduce crest elevation should also be undertaken
    b. Drainage assessment of possible options Namely, Option 1: Keeping as-is and improving or Option 2: Lifting road and re-directing to mangroves.
    c. Road lifting feasibility and discussions with NMIA to finalize acceptability criterion.
    d. Vegetation mapping and replanting
    e. NEPA permitting

7 Appendices

7.1 Engineering Drawings

7.2 Topographic Survey Drawing