ORIGINAL

ENGINEERING DESIGN REPORT

PALISADOES SHORELINE PROTECTION AND REHABILITATION PROJECT – DUNE AND MANGROVE PLANTING AREA CREATION FOR PALISADOES SHORELINE RE-VEGETATION

Submitted to:

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Executive Summary

Introduction

Following the passage of hurricane Ivan in 2004 the Palisadoes shoreline was severely damaged, the sand dunes were overtopped and the roadway blocked. The Government of Jamaica in an effort to protect the Palisadoes shoreline from such damage in the future and to rehabilitate the area implemented the "***Palisadoes Shoreline Protection and Rehabilitation Project"*** and tasked the National Works Agency (NWA) with the responsibility of designing a method of restoring the stability of the Palisadoes. This project involves the dredging of 99,208 m^3 of sand from an offshore sand reserve (borrow area) and placing this dredged material over the buried revetments along the Caribbean Sea side of the Palisadoes. There is also the need to create areas in the Harbour side of the tombolo to facilitate the planting of mangroves within the Harbour.

CEAC has been commissioned by the NWA to provide technical assistance to plan, execute and monitor dredging operations for the creation of suitable substrate for re-vegetation of the dunes to be formed; and plan, execute and monitor the creation of suitable areas for the planting of mangroves on the Harbour Side of the Palisadoes. The consultancy is expected to be executed in the phases outlined below:

1. Phase 1 is the preparation of the Bid document; completion of engineering works for preparation of both replanting areas; and preparation of EIA.
2. Phase 2 is the engineering consultancy and assistance during implementation and construction.
3. Phase 3 is post-construction monitoring of the structure and relocated sensitive species.

The approach taken involved: conducting stakeholders meetings; gathering anecdotal information; field investigations; defining the trends in the climate change drivers; wave, hydrodynamic and sediment analysis studies; laboratory testing; construction methodology study and cost estimating. Parallel to this an environmental impact assessment (EIA) was undertaken of the engineering proposal relative to the environmental resources and socio-economic concerns.

Data Collection Campaign

A bathymetric survey for the project area was developed based on the following surveys:

- CEAC survey conducted along the Caribbean Sea and Harbour side of the Palisadoes, but not including the burrow area
- Cuban survey of the Caribbean Sea side of the Palisadoes conducted in 2008 as a part of their technical report,
- NWA as-built topography carried out after the completion of the revetments along the Palisadoes

Current and wave data was also collected via an Acoustic Doppler Current Profiler (ADCP) and verified using data gathered from two drogue tracking missions. Likewise, water surface elevations were collected during the campaign and it determined a tidal range of 0.43 m. The moored current meter data indicated that the currents moved predominantly in a north-south direction.

A water quality sampling mission was undertaken over 6 stations along the Caribbean Sea side of the project, including a control point in deep water. The water quality parameters measured were TSS, turbidity, pH, salinity and temperature; and all parameters fell within the limits outlined in the 2009 Draft Marine Standards. The water quality readings were also compared to long term water quality data provided by CL Environmental for the 2010 – 2012 period. The TSS, turbidity and salinity parameters were below the long term values while pH and temperature values were greater than the long term values. The TSS typically ranged from 0.5 to 1.5 mg/l.

Sediment grain size collection and analysis confirmed that the sand along the Caribbean Sea side of the Palisadoes ranges between coarse sand and gravel (0.7 – 4.3 mm), the sand was well graded and most samples were positively skewed having more fines in the tail of the distribution.

Samples were also taken from the offshore borrow area and they ranged between fine and coarse sand (0.2 – 0 0.6 mm), the sand samples were mostly poorly sorted. These results were also similar to that obtained by the Cubans for their samples collected from the same borrow area. Two priority areas within the borrow area were identified as providing coarse sand, having a mean grain size between 0.5 – 0.6 mm and a total carbonate composition ranging between 7 – 17%, for use in the sand dune nourishment exercise. This sand is however unsuitable for use in the mangrove nourishment exercise. An alternate source having sand of a suitable nature for mangrove growth was identified for the mangrove replanting areas; this source is in the lower reaches of Hope River where desilting operations are often carried out.

Climate Change, Wave Studies and Storm Surge

A sub-regional climate change study using global and regional scale peer-reviewed information was undertaken by the University of the West Indies Climate Studies Group. The predictions are for global sea levels to rise through to the 21st century at a rate of 3.7 mm/ yr and for annual mean significant wave heights to decrease marginally by 1 – 2%. Additionally, severe storms and hurricanes are predicted to increase in both frequency (5.2%) and magnitude of wave height (4.0%).

Deepwater wave conditions for operational, swell and hurricane waves were derived in order to undertake the near shore wave transformation, sediment transport and structural design studies. These deepwater waves were then used in a special near shore wave transformation model to study pre and post project scenarios with climate change.

The National Oceanic and Atmospheric Administration (NOAA) database of hurricane track data from 1886 to present was utilized in a wave hindcast model to generate historical data on hurricane waves. During the period of data 86 hurricanes passed within 300 km of the project, 6 of which were classified as catastrophic (Category 5). There appears to be a cyclic trend in the number of hurricanes that have passed within 300 km of the project site and that implies that there will be an increase in the number of systems passing the site over the next 40 years, with a general shift in the intensity of the storms from predominantly category 1, 2 and 3 to mostly categories 4 and 5 since the 1940s. South westerly, southern and south easterly waves are the most intense and the 100 year wave height was determined to be 7.6 m. Similarly, the 100 year wave setup inclusive of wave run up was determined to be 1.31 m.

Operational and swell deepwater waves were determined from NOAA long term buoy data to have a wave height of 1.2 m and 2.2 m respectively for the Caribbean Sea side of the project. Nearshore transformations of these waves suggest 0.7 – 1.2 m operational deepwater conditions and 0.8 – 2 m during swell wave conditions. Hurricane conditions result in wave heights of 2 - 3m. The post-construction wave climate (following offshore dredging which will alter the bathymetry) was predicted to have no change in the operational, swell and hurricane wave conditions reaching the shoreline. The two locations to be dredged are approximately 0.6 km and 1.6 km offshore, and they will be dredged to a depth of 1.5m.

Along the harbour side of the project a two-dimensional JONSWAP wind-wave model was used to establish the storm surge over a seven year period (2000 – 2006) for a point just off the harbour. The model determines wave height and period from fetch, storm duration and depth of water in the generating area. The operational and swell deepwater waves have a wave height of 0.2 and 0.6 m respectively. Nearshore transformations of these waves suggest 0.1 – 0.2 m operational deepwater conditions in the pre-project scenario and 0.2 – 0.6 m during swell wave conditions. Hurricane conditions results in wave heights of 1 – 2.5 m. Wave transformation modeling indicates there will be no change in the operational, swell and hurricane wave conditions in the post construction scenarios.

The wave transformation model clearly indicates the vulnerability of the Caribbean Sea side of the project to waves from the south and south west while the Harbour side is vulnerable to waves from the north and North West.

Shoreline Vulnerability

Long term shoreline trends were assessed to identify areas along the Palisadoes that might require stabilization and to also verify wave transformation modeling. Special note was taken of the areas behind the buried revetments. The analysis determined that currently the western section of the Palisadoes (near Gun boat beach) is experiencing erosion while the central and eastern sections (towards Harbour head) are experiencing accretion; as such erosion is occurring along buried revetment 1 whilst along buried revetment 2 accretion is occurring. The shoreline (80%) is accreting at an overall accretion rate of between 0.1 m/year and 0.6m/year, the remaining 20% was observed to be eroding at rates between 0.04 m/year and 0.4 m/year rate.

The alongshore and cross-shore sediment transport modeling determined that the eastern and central sections of the Palisadoes are most vulnerable to erosion due to storm events, this concurs with the long term shoreline data obtained for the same period. It should be noted that the passage of hurricane Ivan in 2004 contributed greatly to the erosion predicted in the alongshore and cross-shore sediment models.

Hydrodynamic Modeling

Currents in the project are driven predominantly by tides with the general movements being from east to west. Current speeds vary from 0.4cm/s to a high of 12cm/s in the near shore areas whereas the offshore areas (in the vicinity of the dredge sites) tend to have a speeds of less than 4cm/s. Sediment dispersion modeling indicate turbidity plumes that can be generated from the operations will be above the NEPA standards. The turbidity plumes are expected to extend up to 2km from the points of operation if precautions are not taken to limit sediments getting to the water column. The offshore plumes are expected to remain offshore and meet the NEPA guidelines for distances further than 1km away from the operations. Similarly the near shore plumes will remain in the near shore and are expected to meet the NEPA guidelines for distances further than 1km away from the operations.

Planning and Design

Calibrated cross-shore models were used to determine the stability and resistivity of the sand dune during both 50 year and 100 year storm events, both for the post project and climate change scenarios. The design process determined that the proposed sand dunes should have a 1:3 slope on both the seaward and landward sides with a 12 m wide crest at an elevation 6.24 m.

Planning of how the dredging operations may affect navigation and utility interests is currently still being investigated with the Port Authority and utility providers. Feedback is still pending.

Draft Dredge Management Plan (DMP)

The most suitable dredging setup is a trailing suction hopper (TSH) which uses a trailing suction drag head to pump fluidized seabed materials to an on board hopper. Sediments are retained in the hopper while water used to pump the material is allowed to discharge from the vessel at the dredging site.

The TSH will operate in 20 m depth of water and be required to pump 99,208 m^3 of sand with a mean grain size ranging between 0.5 – 0.7 mm. This volume and type of material will be dredged from one of the 2 proposed dredge areas identified in the borrow area and placed onshore in a sediment pond to allow the sand to settle. The contractor will then remove this material from the pond and use it to form the sand dunes over the 2 buried revetments. The material will also be used to construct the sand dune between the high revetment and the NWC WWTP once the NWA has agreed to include this option in the project.

Dredging activities result in a number of impacts on the marine environment including the following:

- Changes to water quality,
- Changes to coastal processes (waves and currents)
- Effects on marine ecology (flora and fauna)
- Mobilisation of sediment and pore water contamination

Material Verification and Constructability

Construction of the sand dunes will involve a dredge pumping sand material to a stockpile area along the buried revetment from the borrow area. Excavators will then place the material over the buried revetment for labourers to shape into the design outlined in the engineering drawings submitted; 99,208 m^3 of material is required for the sand dunes.

The mangrove nourishment phase of this project involves using a backhoe, or similar equipment, to place the sand obtained from the Hope River desilting operation along the harbour side of the Palisadoes. The UWI team requires 5,400 m^3 of this material to plant the 6,000 mangroves stipulated.

Engineering Cost Estimate

Procurement is envisaged in two parts, namely: dredging and placement of sand along the Palisadoes (dune nourishment) and the supply and placement of mangrove nourishment along the Harbour side. The dredging contract is expected to involve a dredging contractor with the requisite skills and equipment, while the mangrove nourishment contractor is expected to engage local sources of material working under a main contractor. The engineers estimate for the project is US$4,223,154.10, made up as follows:

- Dredging and Placement of Sand: US$3,971,220
- Supply and Placement of mangrove nourishment: US$251,934.10

It is expected that in the internal project team meetings that various components of the costs will be discussed and prioritized in order to arrive at an agreed approach in the tender document.

Table of Contents

FIGURES

Tables

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1 Introduction

1.1 Background

The Palisadoes shoreline has experienced several severe storm events that have overtopped and blocked the roadway. In 2004, following the passage of Hurricane Ivan, 310 meters of the shoreline was deemed to be in a critical state. The storm caused total destruction of the sand dunes and inundation of the road way which led to the complete shutdown of the Norman Manley International Airport (NMIA), and left residents in Port Royal unable to drive to the mainland. The passage of Hurricane Dean in 2007 exasperated the situation placing approximately 2.65km of the shoreline in a critical state. In 2008 the National Works Agency (NWA) was tasked with the responsibility of designing a method of restoring the stability of the Palisadoes Tombolo. This project is called the "***Palisadoes Shoreline Protection and Rehabilitation Project",*** see Figure 1.1 for the location map. The NWA, in partnership with the Ministry of Local Government and the Environment, and with the technical input from the Ministry of Science, Technology and Environment of Cuba, prepared a report that proposed methods for the re-stabilization of the tombolo. This involved dredging a burrow area close to the shore of the Caribbean Sea side of the Palisadoes and using this material to form dunes along the shoreline. The funding source for the proposed project however required that a reassessment of the design be done. The original proposal was modified as follows:

1. The dune was replaced with rock revetment along the entire shore and elevated road with some 3.7 km of high revetment and 1.3 km of the dune revetments to be buried under the dredged sand.
2. Revetments along the Harbour Side of the Palisadoes.

The updated project therefore indicated the need for the removal of coastal vegetation from both sides of the Palisadoes. As a result the NWA is mandated to replant and restore as much as possible the native vegetation based on the conditions of the Beach licenses and Environmental Permits issued for the project so that there is no net loss of mangroves from the project.

The work to complete the Palisadoes Shoreline Protection and Rehabilitation Project involves the dredging of approximately 99,208 m^3 of sand from the borrow area outlined by the Cubans in their preliminary study and placing this dredged material over the low crest revetment along the Caribbean Sea side of the Palisadoes tombolo. There is also the need to create areas in the Harbour side of the tombolo to facilitate the planting of mangroves within the Harbour.

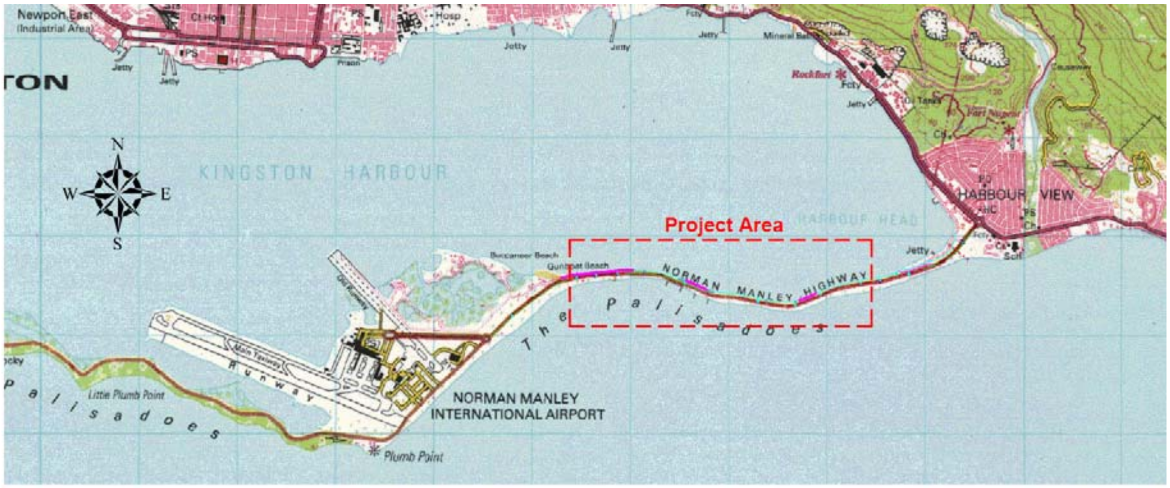


Figure 1.1 Location Map of the Palisadoes in Kingston, Jamaica

1.2 Review of Existing Information on the Palisadoes

1.2.1 Historical Perspective

Many articles have been written about the Palisadoes speculating about how it was formed and what will happen to it in the future. In 2005 the Marine Geology Unit at the University of the West Indies (UWI) (Robinson) contributed to this discussion in light of the severe damage caused by the passage of hurricane Ivan in 2004. The main points of the article are presented herein (subsections 1.2.1.1 to 1.2.1.3) to provide a deeper appreciation and understanding for the vulnerability of the Palisadoes and the importance of this shoreline protection and rehabilitation project.

1.2.1.1 Formation

(Robinson) believes that the Palisadoes was formed by the joining of the Port Royal island and a series of spits extending from the mouth of the Hope River, to the mainland. Dominant waves from the southeast caused the currents to bring the sediments (sand and gravel) from the Hope River and Cane River westward along these shores. See Figure 1.1.

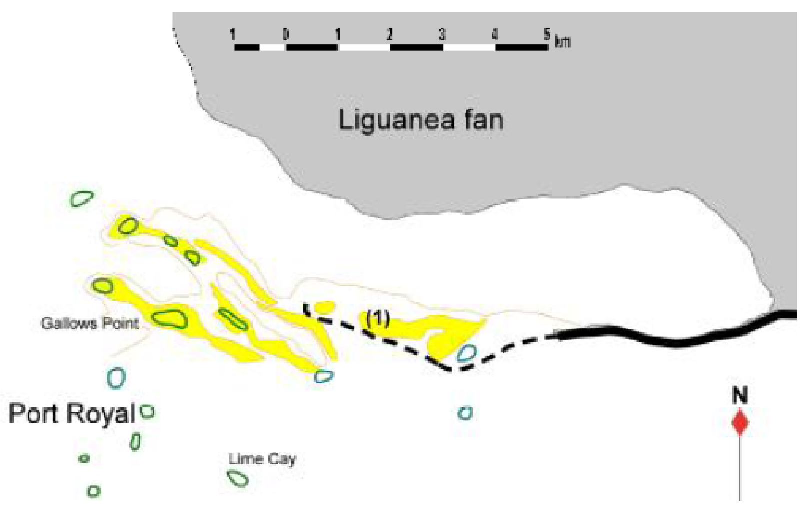


Figure 1.2 Extension of spit (black line) over shallows northwest of the present airport. Green, cays; yellow, shoals; peach line, extent of shallow water

1.2.1.2 Response to Natural Hazards

The article evaluated the effect 3 natural hazards/ phenomenon would have on the Palisadoes: earthquakes, tsunamis and rising sea level (SLR).

Earthquakes occur quite infrequently in Jamaica, and since 1690 three (3) hurricanes have damaged or severely weakened the Palisadoes, in particular the 1692 event which destroyed much of Port Royal. During these severe events liquefaction occurred causing fissuring and the collapse of some sections of the Palisadoes into the sea. Currently the most vulnerable section of the Palisadoes to earthquakes is being protected by a groyne that was severely damaged by hurricane Ivan and now offers some limited protection.

It is believed that the likelihood of a Tsunami occurring in the Caribbean, and more specifically, along the Palisadoes, is small, and if such an event were to occur the storm surge would bring water sand and debris from the Caribbean sea side across the Palisadoes and into the harbour, similar to what happened during Ivan. In an extreme case it is possible for channels to be created along the narrowest part of the Palisadoes, which is in fact in our project area.

Robinson and Rowe also project that the sea level will rise in the future. Naturally the beach elevation will also rise in response because the sediments from the rivers are continuously being moved by the sea. However, the roadway and structures along the Palisadoes will not respond in a similar manner as they are man-made structures. It is thus up to us, as man, to monitor the rising sea level and its effect on the roadway, infrastructure and structures, and take action where necessary. For example, in 2010 the Government of Jamaica raised the road level from between 0.5 and 1 m to 3 – 4 m in response to the damage caused by hurricane Ivan.

1.2.1.3 What of the Future?

Based on the way in which the Palisadoes is believed to have been formed it is possible that in the future the Palisadoes will incorporate the cays now outside the Kingston Harbour. However in terms of its vulnerability to earthquakes and tsunamis, the Palisadoes is susceptible to these phenomena although they occur infrequently. Severe events can create channels along the narrowest section of the Palisadoes which is within our project area, and currently a groyne offers some limited protection from these events.

The Government of Jamaica has also stepped in since hurricane Ivan to protect the Palisadoes from damage caused by a similar event and to minimize the Palisadoes' exposure to natural hazards, including SLR. This project seeks to continue the government's efforts to protect and rehabilitate the Palisadoes by strengthening its most vulnerable section.

1.2.2 Height of Dunes that have survived Hurricane Ivan

Hurricane Ivan destroyed the Palisadoes and attained a category 5 status in 2004. Not all the sand dunes along the Palisadoes were damaged following the passage of hurricane Ivan, some survived, including the dunes in front of the NMIA end of runway and the dunes in the vicinity of Gunboat beach.

Topographic information concerning the shape and size of these dunes was collected and used to inform the design process outlined herein. This information was collected from a 2011 topographic survey conducted by NMIA and from a 2013 topographic survey conducted by CEAC technicians. The surveys provided a range of crest elevations and seaward and landward side slopes for sand dunes that had survived this hurricane event. The dune heights ranged from 4.4 – 7.5 m and the side slopes varied between 1: 3 and 1: 30, see Table 1-1 below. This information was used in the design exercise presented later in this report.

Table 1-1 Characteristics of sand dunes that survived and were not over topped with the passage of hurricane Ivan

Dune Height (m)	Representative locations	Dune Height (m)	Side Slope	
			Seward Slope	Landward Slope
Survived with some damage and limited over topping	South of Gunboat beach and east of CMI/RJYC entrance Between plumb point and end or NMIA runway	4.4 to 6.4	1 : 3	1: 9 to 1: 30
Not significantly overtopped or damaged	Entrance to CMI/RJYC and 300 meters east of NMIA runway and opposite meteorological station	7.68 to 9.06		

1.3 Approved Scope of Work

CEAC was commissioned by the NWA to provide technical assistance to plan, execute and monitor dredging operations for creation of suitable substrate for re-vegetation of the dunes to be formed; and plan, execute and monitor the creation of suitable areas for the planting of mangroves on the Harbour Side of the Palisadoes. The consultancy is expected to be executed in the phases outlined below.

4. Phase 1 is the preparation of the Bid document; completion of engineering works for preparation of both replanting areas; and preparation of EIA.
5. Phase 2 is the engineering consultancy and assistance during implementation and construction.
6. Phase 3 is post-construction monitoring of the structure and relocated sensitive species.

This report is the final report for Phase 1 that covers the engineering design aspects. An EIA document will be delivered as a separate deliverable.

1.4 Design Requirements

1.4.1 Basis

The project has two (2) components – dune and mangrove nourishment – and both areas will be designed to meet the following conditions:

1. 1 in 100 year return period deep water wave conditions,
2. Project life up to 2050 (37 years),
3. Climate change factors for the SRES A1B or A1 scenario up to the design life,
4. Employ the use of locally available materials and the burrow area proposed by the Cuban.

The reasons for these design parameters are discussed below:

Table 1-2 Design parameters for the Palisadoes Shoreline Protection and Rehabilitation Project

Objectives	Design Basis	Reasons
Wave protection and Structural Resilience	1 in 100 year return period deep water wave conditions	Equivalent to a remote chance of occurrence on an annual basis with a 31% probability over the life time of 37 years, (CIRIA)
Climate Resilience	Climate change factors for the SRES A1B or A1 scenario up to the design life	Most adverse set of scenarios and most consistent with current global trends for emissions and observations. (Roeckner, Giorgetta and Crueger) (Knutson, Sirutis and Vecchi) (Murakami)
	Project life up to 2050 (37 years)	Extrapolation beyond 2050 to 2100 will be subject to more uncertainty. As model predictions become increasingly more consistent with predictions (especially with waves) then these can be considered.
Minimize life cycle costs and local economic relevance	Maximize the use of local sand materials	To minimize foreign exchange requirements and maximize local input/economic impact

1.5 Methodology

1.5.1 Anecdotal Evidence

Anecdotal information on the major hurricanes and storm events that have affected the Palisadoes Tombolo was gathered from interviews held with residents and employees in the Harbour View and Port Royal area. The results of these interviews were collated and used to calibrate and verify numerical the models.

1.5.2 Wave Study

1.5.2.1 Deepwater Hurricane Wave Climate

It was necessary to define the deepwater hurricane wave climate in order to define the Palisadoes environ. A thorough statistical analysis of wind-wave hindcasting of hurricane data within the Caribbean was conducted in order to determine the hurricane wind and wave conditions at a deep water location offshore the project area.

1.5.2.2 Deepwater Operational Wave Climate

The NOAA database provided information used to establish the operational wave climate over an eight (8) year period (2000 – 2006) for a point just off the continental shelf.

1.5.2.3 Nearshore Operational Wave Climate

The deepwater wave climate obtained from the NOAA database was used to run a Refraction-Diffraction wave model in order to carry the deepwater waves from the continental shelf to the Palisadoes shoreline.

1.5.3 Hydrodynamic Modeling

Bathymetric data and data on current speed and direction were collected and used to develop a detailed three-dimensional hydrodynamic model (RMA-10) of the area. Both pre and post-project bathymetric configurations of Palisadoes were considered and the effects on flushing and circulation assessed.

1.5.4 Climate Change

A climate change assessment for water level, wave heights and hurricane intensities was conducted with help from the University of the West Indies. This information was used to model the 50 and 100 year return period storm events used in the design.

1.5.5 Dune Design and Mangrove re-planting Areas

Calibrated Sediment transport models were used to design the dune cross sections for that will remain in place after the passage of the 50 and 100 year storm event. Similarly, the mangrove areas should have sufficient area to maintain the mangroves after the annual swell event.

2 Oceanographic and Meteorological Data Collection

2.1 Bathymetric Survey

Bathymetric data is required in order to facilitate the estimation of fill volume for dune placement and mangrove nourishment – project components which are directly related to costs. In addition, bathymetric data forms the basis for wave transformation and hydrodynamic modeling which then allows for modeling of the size, shape and location of the required structures.

The Bathymetric survey was developed based on the following surveys and the contour lines resulting from these surveys are shown in Figure 2.2:

- CEAC survey conducted along the Caribbean Sea and harbour side of the Palisadoes on November 15 and 20, 2013, not including the burrow area
- Cuban survey of the Caribbean Sea side of the Palisadoes conducted in 2008 as a part of their technical report,
- NWA as-built topography carried out after the completion of the revetments along the Palisadoes.

2.1.1 Method

The three (3) surveys outlined above where carried out in the following way:

- The CEAC survey was done to using a Garmin echo sounder along gridlines running parallel and perpendicular to the Caribbean Sea side and harbor side shoreline were followed to collect the bathymetric data. Along the Caribbean Sea side the survey was taken between the NWC treatment plant and the end of the most western low revetment, while the Harbour side survey was taken between Gypsum Quarry and Gun Boat Beach.

- The Cuban survey was carried out using a Biosonics echosounder from Cane River to Little Plumb Point along 37 survey lines perpendicular to the shore between 5 and 30 m deep. The survey was able to identify a sandy basin that would be useful as a borrow area.

- The NWA completed the as-built topography survey after the revetments were constructed, and this information defined the shoreline along both sides of the Palisadoes. All three (3) surveys were then used to develop a comprehensive bathymetry for the project area.

2.1.2 Description of the Palisadoes

The Palisadoes constitutes the extension of land of about 14 km in length, with an East-West projection, that protects Kingston Harbour from the open waters of the Caribbean Sea[1]. The narrow strip of land ends at Port Royal, leaving a deep channel through which even the largest ships can sail. The area lies within 13,000 hectares of cays, reefs and mangroves and is also a National Heritage site.

[1] Juanes, Perez, Izquierdo, Caballero, Rivero (2007), Palisadoes Protection and Rehabilitation Project,

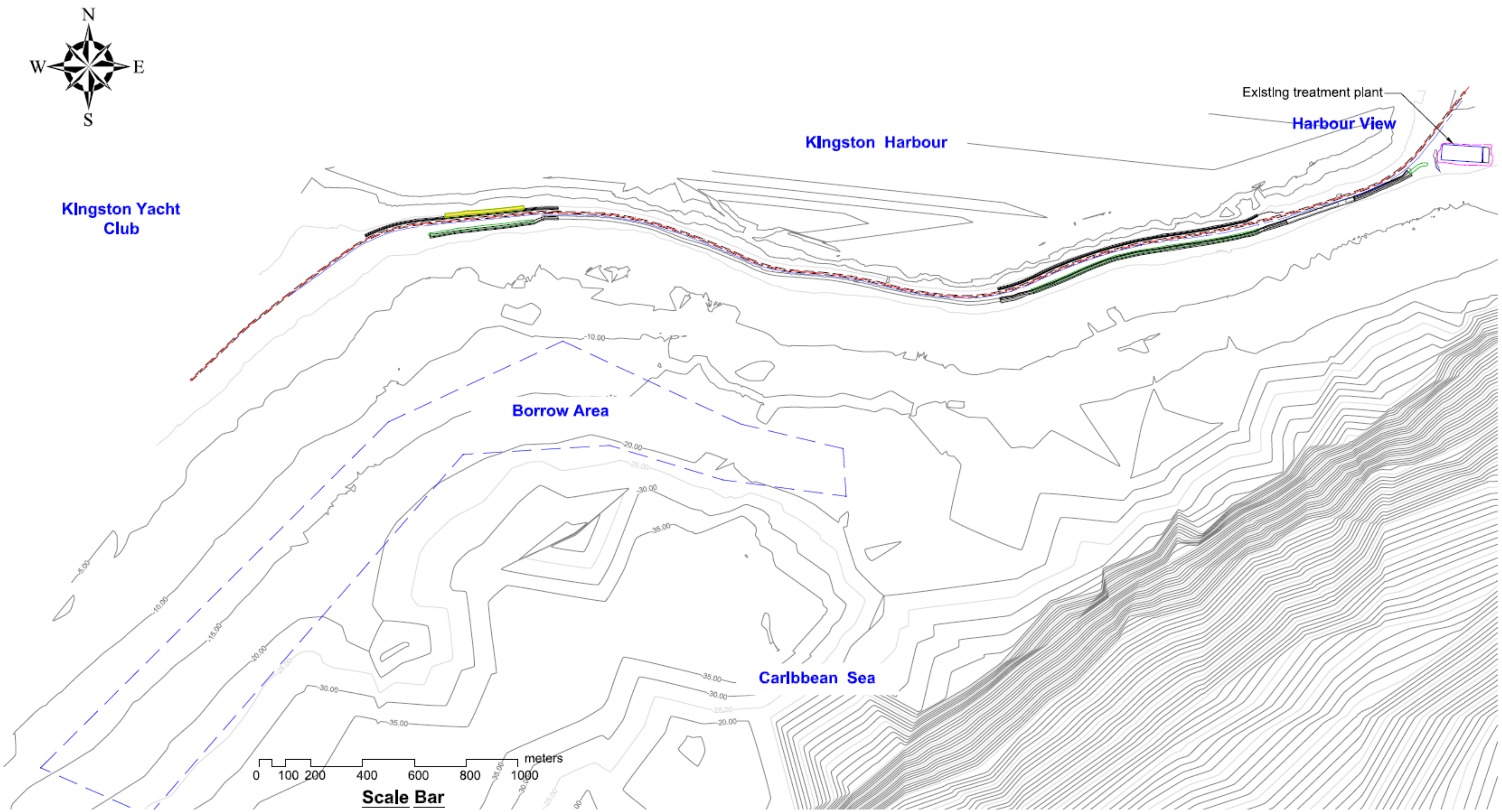


Figure 2.1 Bathymetry for the Palisadoes Shoreline Protection and Rehabilitation Project

2.2 Currents

In order to facilitate the development of the hydrodynamic model for the area it was necessary to collect information on tides, winds and currents. This information was acquired by carrying out two drogue tracking missions and deploying an Acoustic Doppler Current Profiler (ADCP) on the sea floor for approximately one month.

2.2.1 Moored ADCP

An ADCP was deployed in two locations over a 4 week period, and two drogue tracking missions were carried out in the vicinity of the moored ADCP to verify its measurements/readings. An ADCP operates using acoustic signals, and determines the current speed and direction by detecting the Doppler shift of reflected acoustic signals, which bounce off particles moving with the water. With this method of measurement it is therefore able to measure separate section/bins in the water column.

The ADCP was deployed in 20 meters of water at plumb point in the west and 18m in the central section of the project area within the Caribbean Sea. See Figure 2.2. It was set to record averaged current and wave readings at 1 hour intervals.

Figure 2.2 Google imagery showing the two locations where the ADCP was deployed in the Caribbean Sea for the Palisadoes Shoreline Protection and Rehabilitation Project

The time series graphs below (Table 2-2 and Table 2-2) indicate that the current velocities decrease as you move deeper into the water column, that is, the surface currents are faster than the currents at mid depth, and the currents at mid depth are faster than those at the sea floor. This observation is generally the case as surface currents are more likely to be impacted by winds whenever the wind velocities are sufficiently high as well as waves and so they will have larger current velocities.

The scatter plots in

Table 2-3 and

Table 2-4 indicate the currents were generally moving in a north-south direction in the area where the ADCP was first deployed. During the second deployment however the only trend observed was for the surface currents, they were moving in a general north-south direction. There mid depth and sea floor currents displayed no general trend, the currents were erratic moving in all directions.

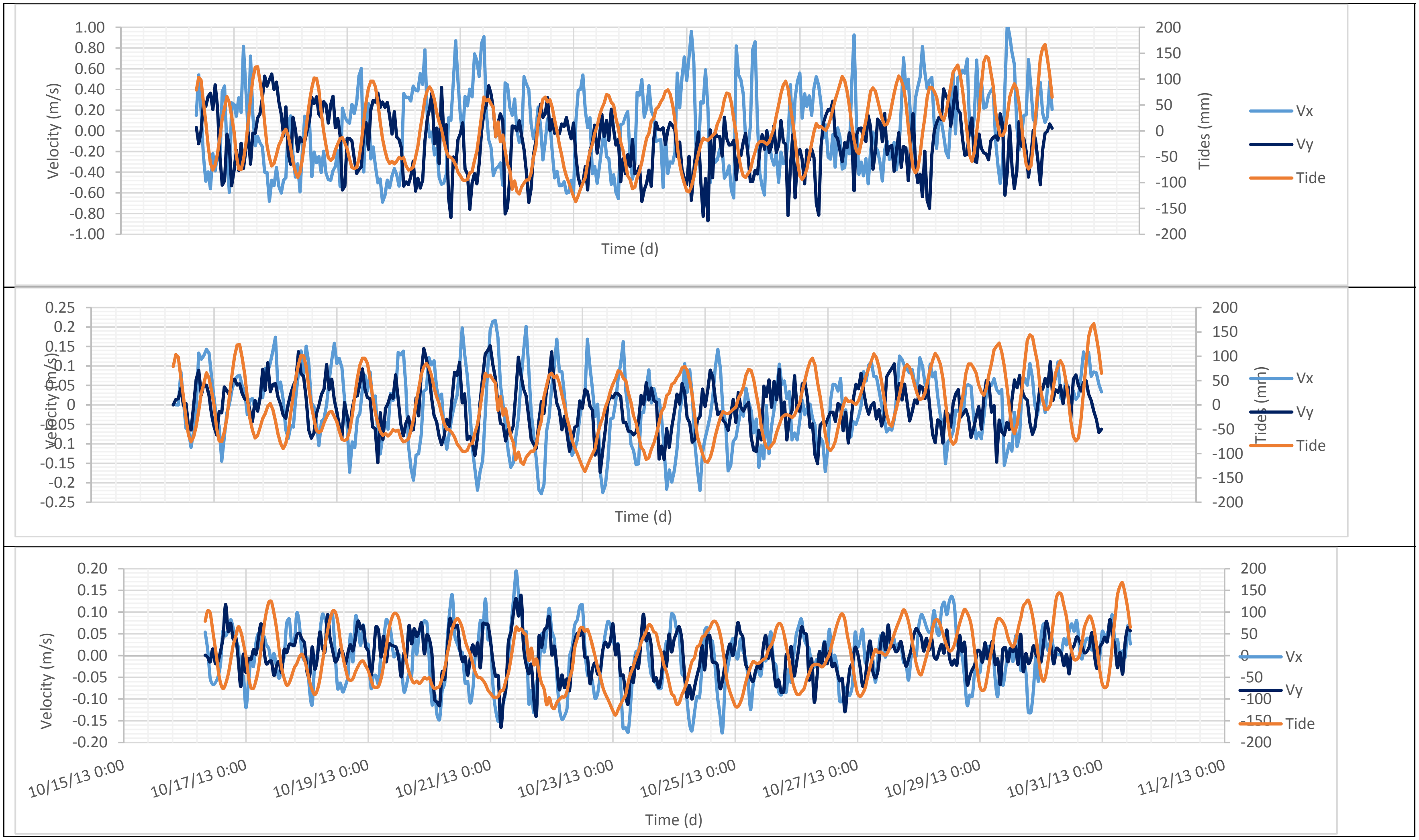


Table 2-1 Current velocities and tide recordings during the first ADCP deployment of the centre of Palisadoes inline with the burrow area for the surface (top panel), mid depth and sea floor (bottom panel) respectively

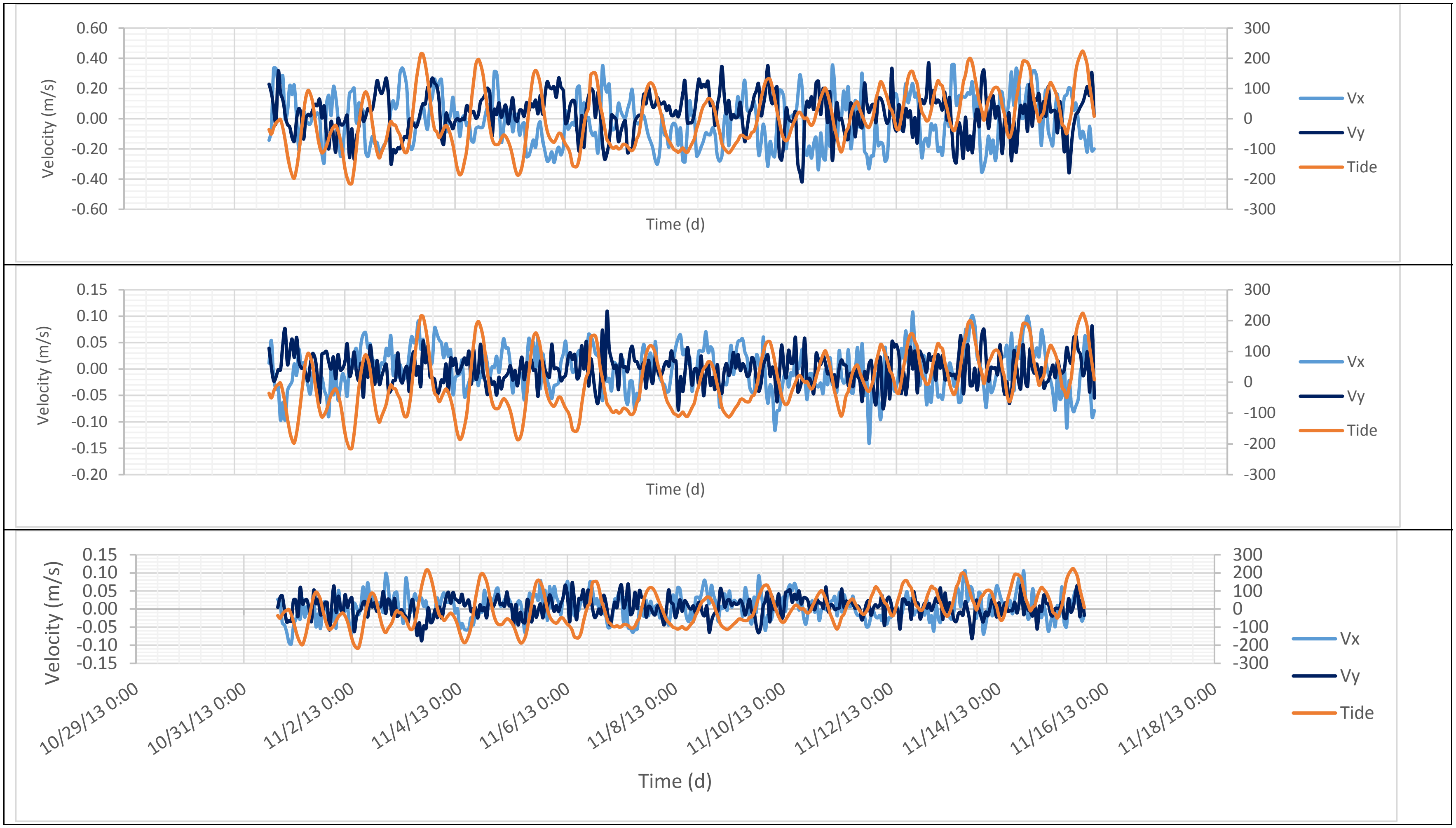


Table 2-2 Current velocities and tide recordings during the second ADCP deployment of Plumb Point for the surface, mid depth and sea floor respectively

Table 2-3 Current velocities recorded along the Palisadoes for the first ADCP deployment (south of Plumb Point) at the surface, mid depth and sea floor respectively

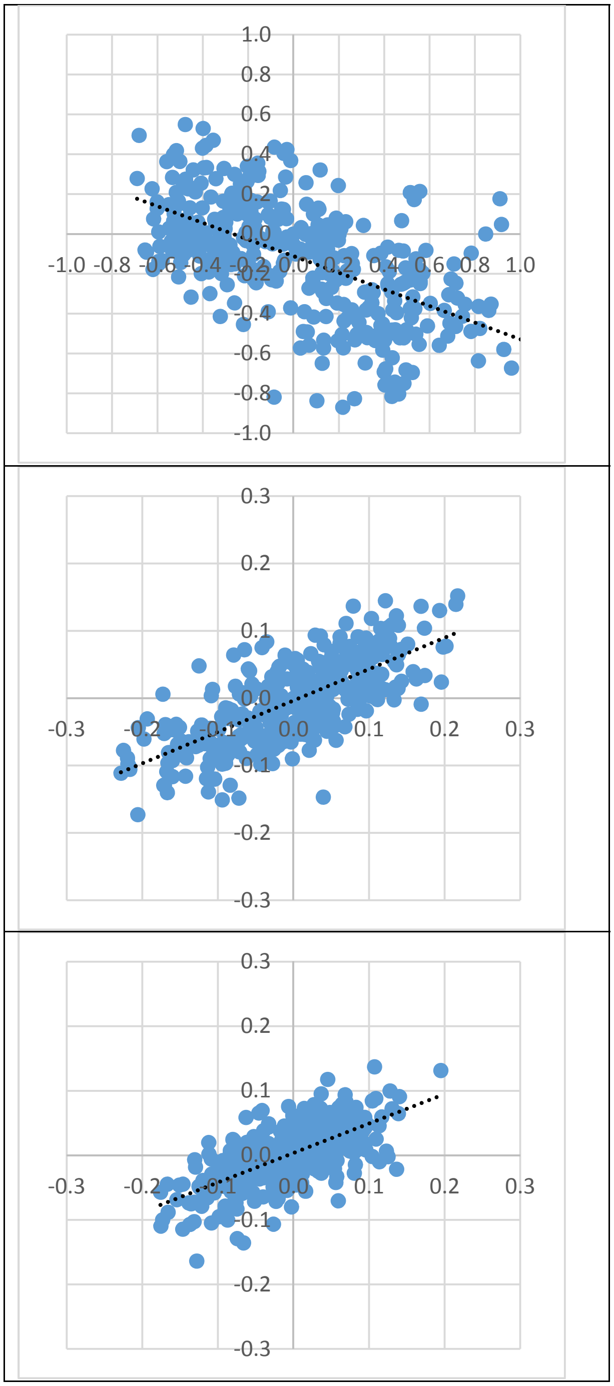
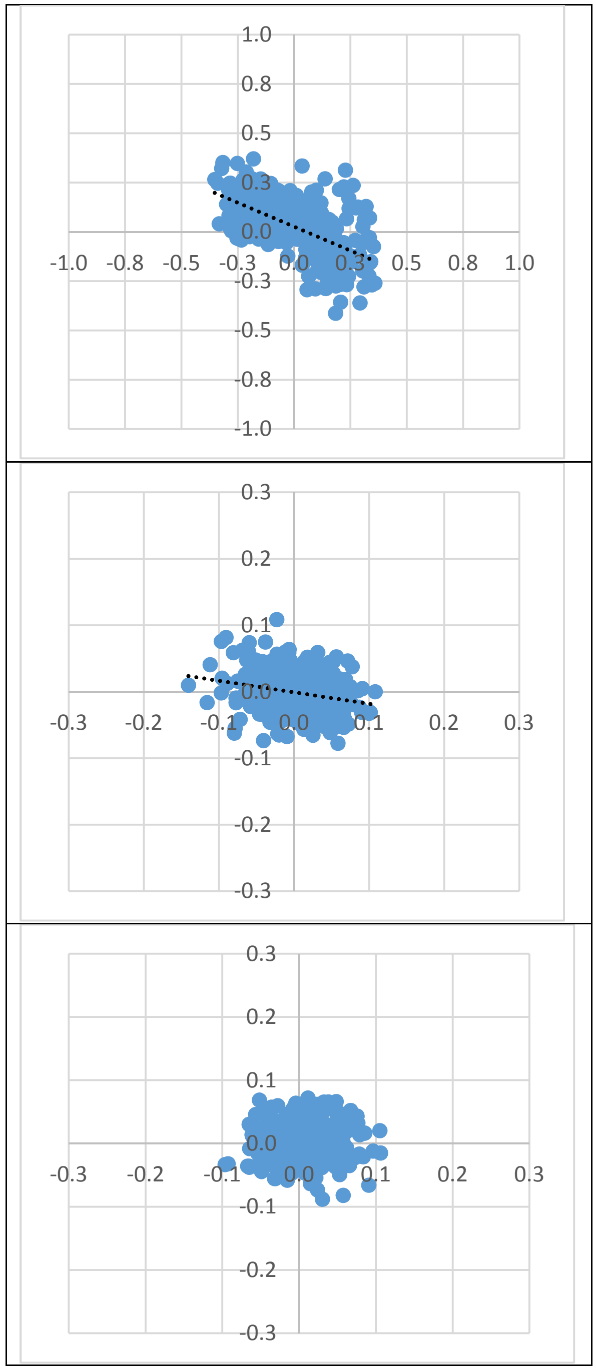


Table 2-4 Current velocities recorded along the Palisadoes for the second ADCP deployment (centre of project site) at the surface, mid depth and sea floor respectively



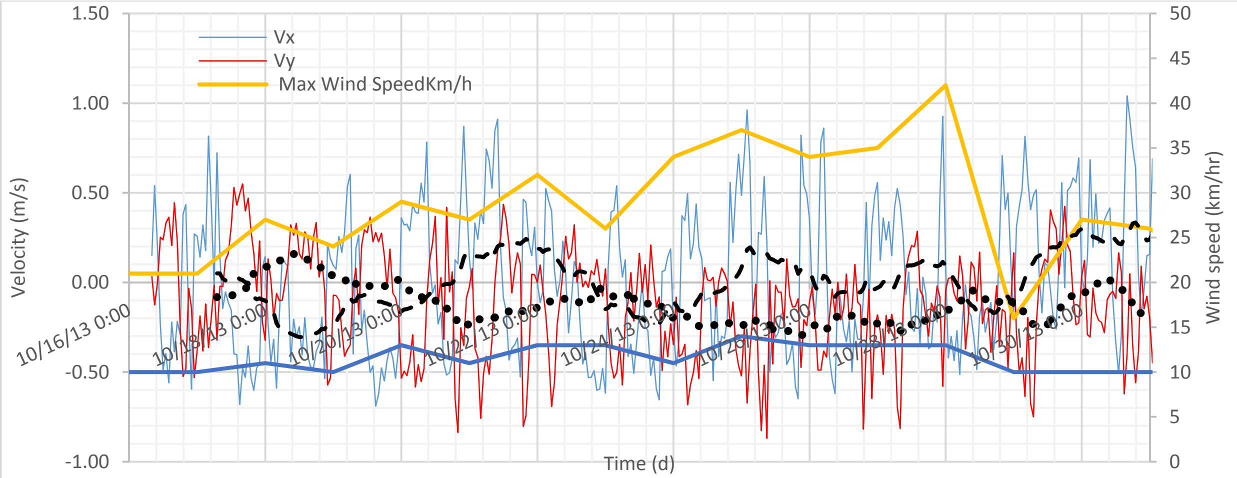


Figure 2.3 – ADCP location 1 (central to Palisadoes) and maximum and mean wind speeds for the period October to November, 2013.

The occurrence of high wind speeds were noted to not correlate with higher surface currents, Figure 2.3. For example during the period 24th to 28th of October when wind speeds were elevated, the mean (24 hour moving average vx and vy remained approximately the same. It is therefore apparent that currents from surface to sub-surface are predominantly tidally driven and influenced by oceanic currents.

2.2.2 Waves and Tides

2.2.2.1 Raw Data

Tidal information was important in order to drive the Finite Element Hydrodynamic Model (FEM) and to also set up the water level in the wave model. More importantly, it was necessary to determine the tide range in order to determine the minimum crest height for the sand placed over the buried revetments and along the Harbour side of the project so as to minimize overtopping and erosion during swell events. The tide range measured during the deployment period was 0.43 m.

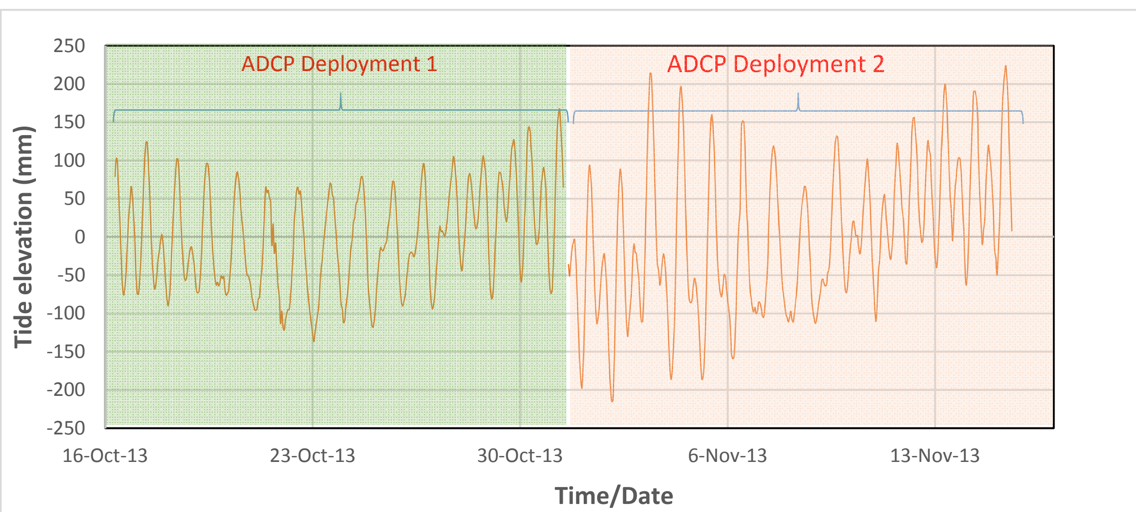


Figure 2.4 Tidal signal recorded during the ADCP deployment for the period 17th of October to 15th of November, 2013 for Palisadoes

2.2.2.2 Tidal Harmonics

Tidal harmonics is essentially the blending of the different sinusoidal curves for each harmonic constituent of the tide until it closely matches that obtained from the recorded tidal signature. This is useful for predicting the tides for future times when there is no data available.

The amplitudes of the seven most significant harmonic constituents were determined from the raw tide data by utilizing the *least squares method*. In this method, a set of cosine terms is used as a model. The blended curve is made to fit the data recorded by the ADCP by choosing the combination of R and N that causes the sum of the squared differences between observed and model-predicted tides is as small as possible. The resulting amplitudes and phase lag are outlined below in Figure 2.5, and it allowed us to make reasonable tide predictions for future times when running FEM and wave models. It is evident that the K1 consistent that is a diurnal tide is dominant. Both semi-diurnal and diurnal tidal constituents were detected.

Table 2-5 Tidal constituents obtained from the harmonic analysis of the raw ADCP data collected along the Palisadoes

Tide constituent	M2	S2	O1	K1	N2	P1	L2
Speed	12.42	12	25.82	23.93	12.66	24.07	12.19
Phase lag	-5.11	0.84	1.97	5.11	0.75	-3.92	-2.63
Amplitude	0.028	0.023	0.050	0.124	0.032	0.067	0.015

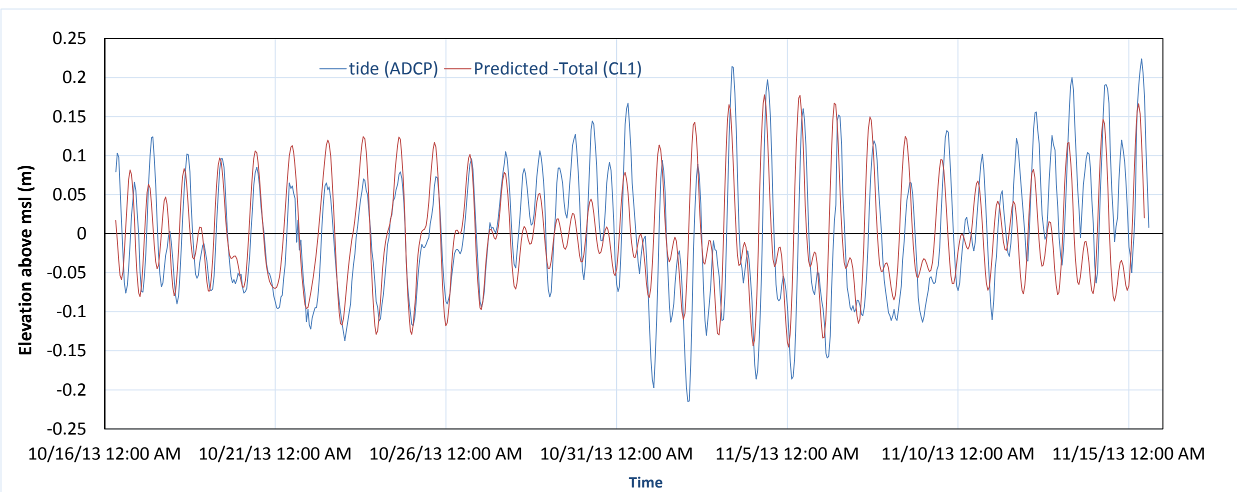


Figure 2.5 Measured and predicted tidal signature for the Palisadoes for the period October 16, 2013 to November 15, 2013

2.2.3 Drogue tracking

Drogue tracking information was necessary in order to verify the ADCP readings, and to provide information on the water circulation pattern throughout the project area. Drogues provide area wide short duration information, whereas ADCP provide a site specific long duration continuous record.

Two sets of drogue tracking missions were executed on October 31 and November 15, 2013. The missions were done to coincide with the first and second ADCP deployment. Six (6) drogues were deployed; three (3) surface and three (3) sub-surface drogues (with depths ranging from 1 to 10 meters). The drogues were deployed at three (3) offshore locations during each mission, and at each location the drogues were tracked during two separate sessions each day to capture the rising and falling tides. Drogues were also deployed at each ADCP location so as to provide information that could be used to verify the ADCP data.

The GPS and drogue log sheet results from the drogue tracking missions were reduced and incorporated into a database. The data was then analyzed in order to determine current speed and directions, and current speed vectors were produced for the rising and falling tides below. This information is presented in the Appendices, Section 10.1 Drogue.

2.2.3.1 Tracking Results

2.2.3.1.1 October 31, 2013 – Rising Tide

During this session the drogues were deployed at three locations – Near shore, Plum Point and offshore over the ADCP. The tide was observed to be moving in a generally south westerly direction similar to the average wind direction, as the near shore drogues were moving south westerly and the plum point drogues were moving westerly. The drogues in the vicinity of the ADCP (deep water) however were moving south easterly. The surface drogues were observed to be moving at an average speed of 7.11 cm/s while the subsurface drogues were slower moving at 6.56 cm/s.

2.2.3.1.2 October 31, 2013 –Falling Tide

The drogues were deployed at the same three locations used in the previous session and during this session the tide was observed to be moving in a generally north easterly direction similar to the average wind direction. The near shore drogues were moving northerly and the plum point drogues were moving north easterly, while the deep water drogues were moving southerly. The surface drogues were observed moving at an average speed of 3.32 cm/s while the subsurface drogues were slower at 1.75 cm/s.

2.2.3.1.3 November 15, 2013 – Falling Tide

During this session the three sets of drogues were deployed at three locations: Near shore, offshore over the ADCP and further offshore. The ADCP Location was westward of that used in the previous session. During this session the tide was observed to be moving in a generally north westerly direction, similar to the average wind direction. The surface drogues were observed at an average speed of 3.47 cm/s while the subsurface drogues were slower at 2.14 cm/s.

2.2.3.1.4 November 15, 2013 – Rising Tide

The drogues were deployed at the same three locations used in the previous session and during this session the tide was observed to be moving in a generally north westerly direction, similar to the average wind direction and similar to the previous session. The surface drogues were observed at an average speed of 8.75 cm/s while the subsurface drogues were slower at 3.83 cm/s.

2.2.3.2 Summary

The drogue tracking missions comprised of 4 sessions – two falling tide and two rising tide sessions – that covered 6 offshore locations on October 31 and November 15, 2013. The current speeds varied between 2.29 cm/s to 10.25 cm/s and 1.48 cm/s to 10.30 cm/s for the surface and sub-surface drogues respectively.

Knowledge of the prevailing wind conditions allowed for the determination of the effect of wind speed and direction. The current speeds are generally higher for the rising tides than for the falling tide session. It is evident that when the wind speed is slow, the tides dominate the currents; however when the wind speeds increase to above 10 cm/s (2.78m/s) then the effect of the tides is negligible. Plots of the drogue tracking sessions are placed in the Appendix.

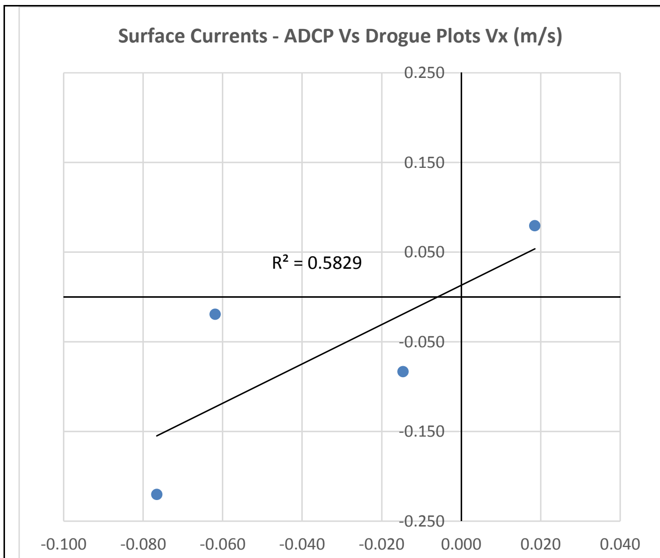
2.2.4 Currents Verification (ADCP/Drogues)

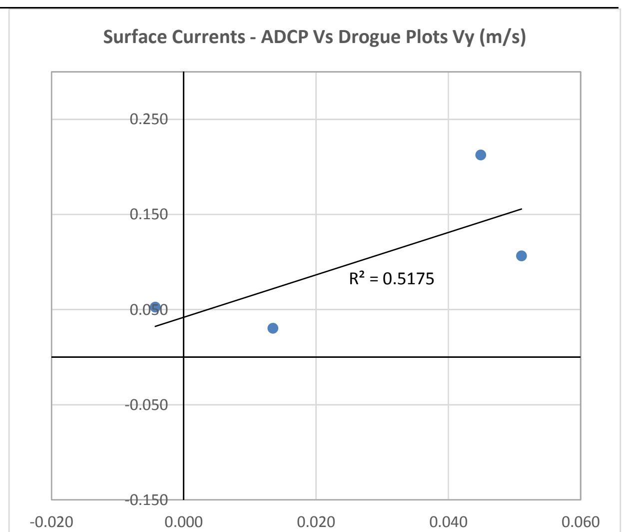
The currents recorded by the ADCP were checked against the drogues to confirm that the ADCP was recording the correct currents (speeds and direction).

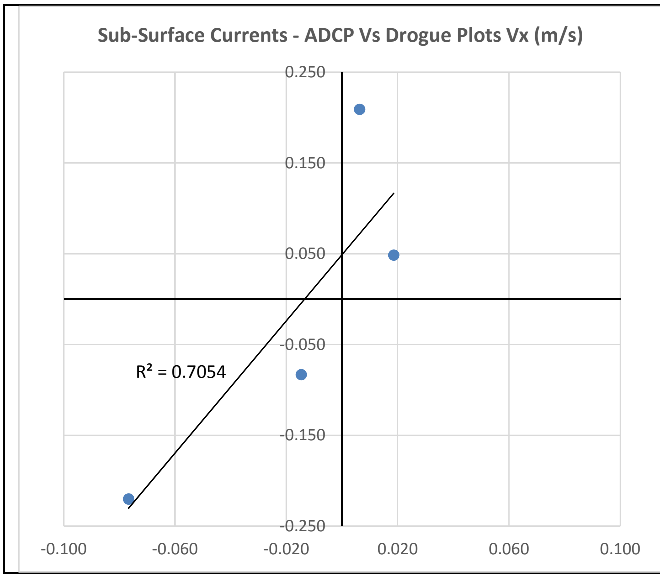
The X and Y components of the currents were compared; for the surface drogues a 76 and 72% correlation was obtained for the X and Y components respectively, while for the sub-surface drogues the correlation was 84 and 88% for the X and Y components respectively. The coefficient of correlation is used as a comparative measure of association of two or more datasets (in this case the X-Y components for the currents). Even though the relationship between the ADCP and drogues were generally good in terms of magnitude, the directions in some cases were slightly different. Similarly the variance can be used to estimate the dispersion about the average measured values. The variances for the surface and subsurface drogues were all below two percent. Overall, it can be concluded that the ADCP was functioning properly.

The graphs in Table 2-6 below highlight the correlation using scatter plots while Table 2-7 below summarizes the correlation in the data.

Table 2-6 Comparison plots for the X and Y components of velocity for the drogues (surface and sub-surface currents) and the ADCP deployed in the Caribbean Sea for the Palisadoes project. The ADCP was deployed twice in the project area.







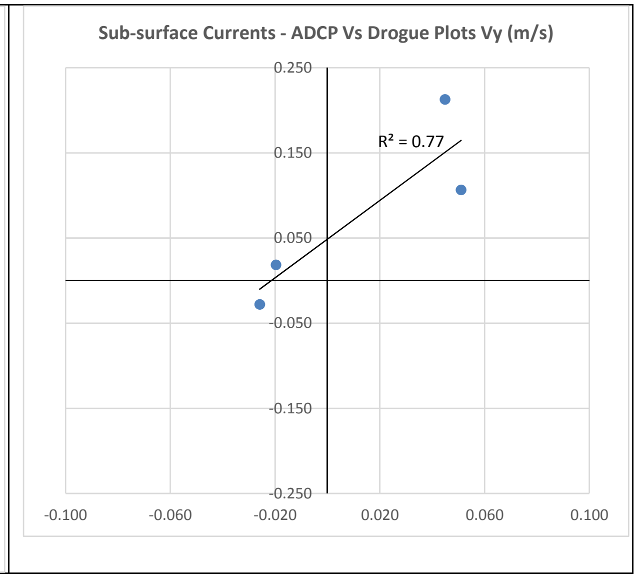


Table 2-7 Statistical comparison of the currents measured by the drogues and DCP deployed in the Caribbean Sea for the Palisadoes project

Correlation			Variance	
Depth	**Vx**	**Vy**	**Vx**	**Vy**
Surface	0.76	0.72	0.5%	0.2%
Subsurface	0.84	0.88	1.11%	0.20%

2.3 Water Quality Surveys

2.3.1 Methodology

Whole water quality samples were collected at different locations. Samples were collected and stored on ice before being taken to the Laboratory for analysis of TSS and turbidity. Temperature, pH and salinity were measured in situ using a Hydrolab DS5 water quality multi-probe. A total of 6 water quality stations were strategically placed across the bay, one of which was a deep water station placed approximately 2.4 km offshore and was designated as the control point (station CC), see Figure 2.6. The control point was an offshore/deep water point that was used to compare the near shore parameters to determine if the bay is polluted, and at stations WQ4, 5 and 6 both deep and surface samples were taken, while only surface samples were collected at the other stations. Water quality data collected between 2010 and 2012 by CL Environmental during the first phase of the Palisadoes Shoreline Protection and Rehabilitation Project was also used as a reference (station CL-P1) providing long term measurements for the water quality parameters.

Figure 2.6 Water quality monitoring points in the Caribbean Sea on November 19, 2013

2.3.2 Comparative Assessment to NEPA Guidelines

The results were averaged and compared to the 2009 Draft Jamaica National Ambient Water Quality Standard for Marine Water, as well as long term values measured by CL Environmental between 2010 and 2012. The values are presented in Table 2-8 below, whereas the summary discussions for the individual parameters are found in the subsections below.

Table 2-8 Recorded values for the water quality parameters assessed in the Palisadoes along with the long term values recorded by CL Environmental

ID	TSS	TUR	TEM	PH	SAL
WQ2	1	3.7	29.29	8.33	34.86
WQ4 (S)	1	3.2	29.55	8.33	34.74
WQ4 (D)	0	3.7	29.34	8.32	34.75
WQ3	1	4.8	29.58	8.23	34.76
WQ5 (S)	0	3.5	29.51	8.24	34.74
WQ5 (D)	0	4.6	29.46	8.24	34.76
WQ6 (S)	0	2.9	29.71	8.15	34.76
WQ6 (D)	0	1.7	29.5	8.15	34.76
CC (S)	***0***	***3.3***	***29.31***	***8.26***	***34.71***
CC (D)	***0***	***3.8***	***29.12***	***8.25***	***34.68***
CL - P1	***1.50***	***7.66***	***28.46***	***8.15***	***36.76***

2.3.2.1 Total Suspended Solids (TSS)

Whilst there are no standards for the TSS levels, in the Caribbean Sea the control point (station CC) gives some indication as to what the ambient levels should be (<1 mg/L). No station had TSS levels greater than 1 mg/L, most were in fact recording TSS levels of 0 mg/L including the control point. The reference data provided by CL Environmental however, determined that the average TSS during the 2010 – 2012 period was 1.5 mg/L, see Figure 2.7.

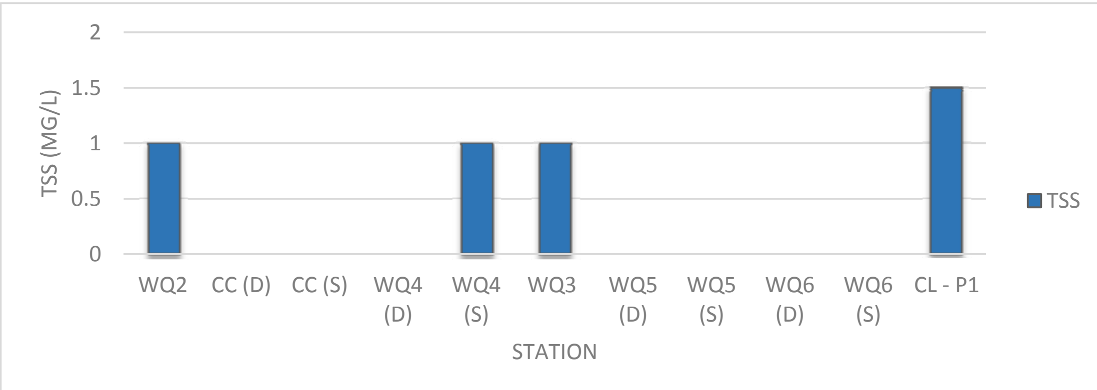


Figure 2.7 Concentration of Total Suspended Solids (TSS) at the selected stations in the Caribbean Sea on November 19, 2013

2.3.2.2 Turbidity

All the water quality samples were below the NEPA standard of 39 (NTU). A comparison plot is shown in Figure 2.8 below.

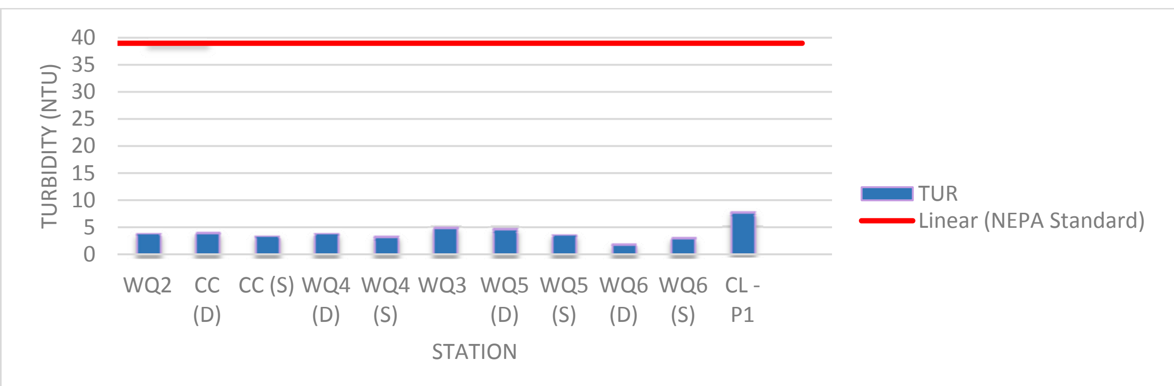


Figure 2.8 Concentration of Turbidity at the selected stations in the Caribbean Sea on November 19, 2013

2.3.2.3 Temperature

The water temperatures measured were all higher than the offshore control station. This can be attributed to the fact that the waters near shore are shallower and therefore require less solar radiation to warm. The long term temperature reading provided by CL Environmental is however smaller than the values recorded on November 2013, and this may be because this station is further offshore than the values we recorded, in deeper water. See Figure 2.9.

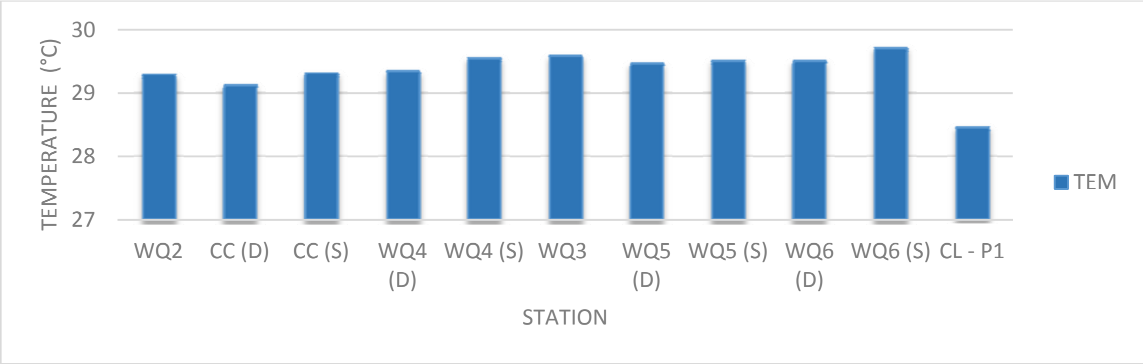


Figure 2.9 Temperature readings at the selected stations in the Caribbean Sea, on November 19, 2013

2.3.2.4 pH

All the stations met the NEPA standard of 8.0 – 8.4 but only station 6 had values similar to the long term pH value determined by CL Environmental, see Figure 2.10.

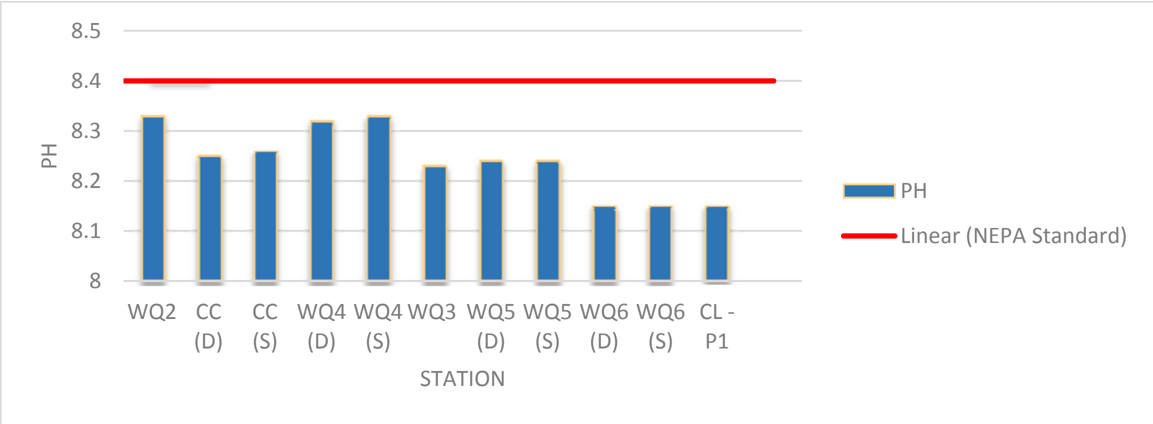


Figure 2.10 pH readings at the selected stations in the Caribbean Sea on November 19, 2013

2.3.2.5 Salinity

Salinity is generally used to gauge whether the water sample is saline/marine or non-saline/fresh water. All stations met the normal seawater salinity standard of 35 ppt except the CL Environmental station and this may be because that particular station is further offshore, and in deeper water. See Figure 2.11.

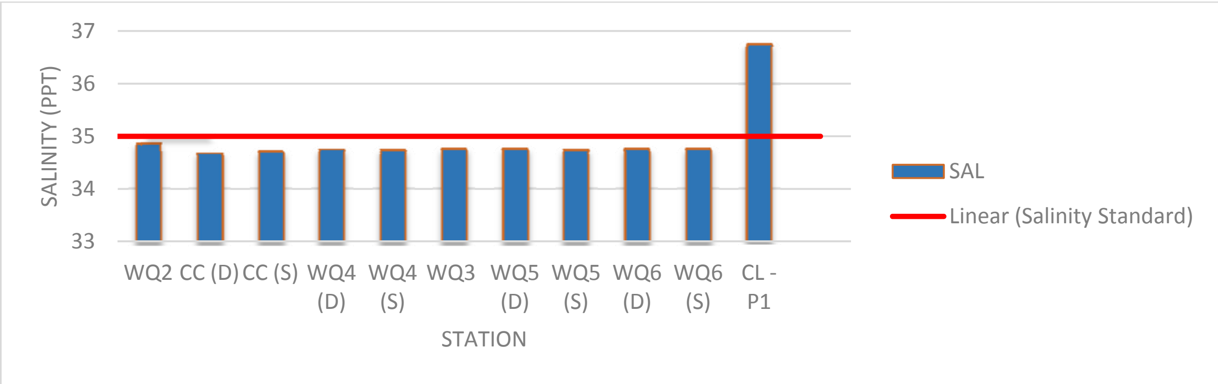


Figure 2.11Salinity readings at the selected stations in the Caribbean Sea on November 19, 2013

2.3.3 Summary

A water quality testing exercise was conducted on November 19, 2013 at 5 offshore stations in the Caribbean Sea side of the project. The measurements recorded were compared to long term readings obtained by CL Environmental and the limits presented in the 2009 Draft Marine Standards. All parameters fell below the limits outlined but there were differences between the measurements for the control station and the long term values. The TSS, turbidity and Salinity parameters were below the long term values while pH and temperature values obtained were greater than the long term values.

2.4 Wind

Historical and current wind data for the project area was obtained from three main sources:

- Norman Manley met station,
- NOAA Climate Service and
- Weather Underground's online database.

2.4.1 NMIA (1999 TO 2004)

The NMIA provided wind data for the airport spanning the period 1999 to 2004 and this information is presented in Figure 2.13. The data revealed that most of the winds are from the N to SE direction and moving at speeds ranging from 8 to 20 m/s.

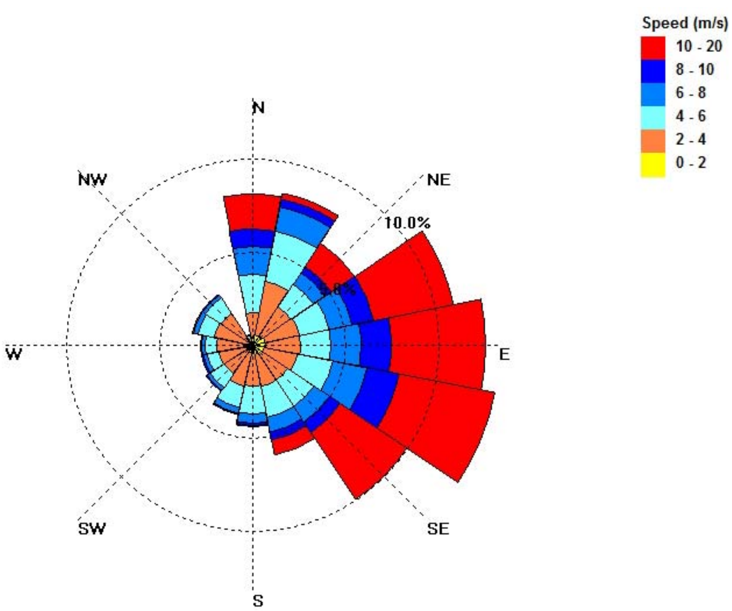


Figure 2.12 Historical wind data for NMIA for data spanning 1999 to 2004

2.4.2 NOAA Climate Service

The NOAA long term wind wave data model was searched for long term wind data for the Palisadoes. A node was chosen along the Caribbean Sea side of the Palisadoes and the wind data corresponding to that node obtained. The node used was:

The data spanned the years 1999 to 2000 and recorded daily values at 3 hour intervals; and it is presented in Figure 2.13. The data was analyzed in terms of the percentage occurrence of various wind speed and direction combinations in order to characterize the wind climate for the site. The analysis revealed that the winds are primarily from the ENE to ESE direction with moving at between 2 – 4 m/s.

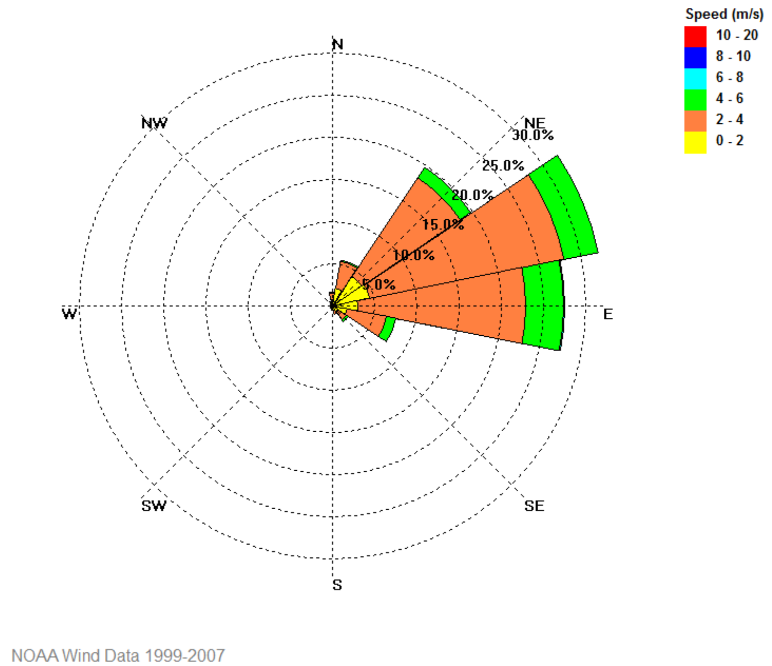


Figure 2.13 NOAA long term wind data for a node offshore the Palisadoes for data spanning 1999 to 2007

2.4.3 Weather Underground

Current wind data was collected for the days on which the drogue tracking missions were carried out from the Weather Underground online database for the Palisadoes area for October 31 and November 15, 2013. Most of the winds on October 31, 2013 were from the SW and SE moving at an approximate speed of 3 to 4 m/s. On November 15, 2013 the winds were again primarily from the SW and SE moving at approximately 5 m/s. See Figure 2.14 and Figure 2.15.

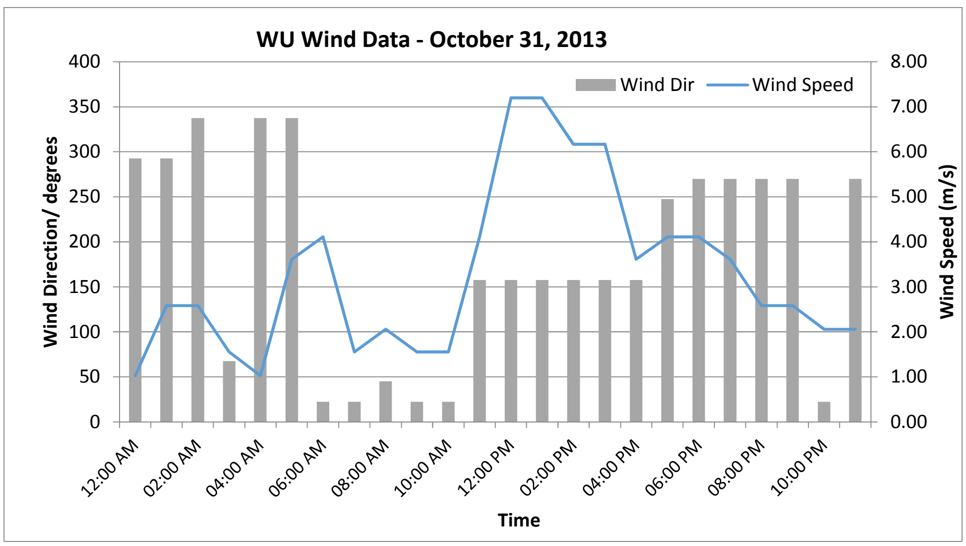


Figure 2.14 Weather Underground wind directions and speeds for the Palisadoes on October 31, 2013

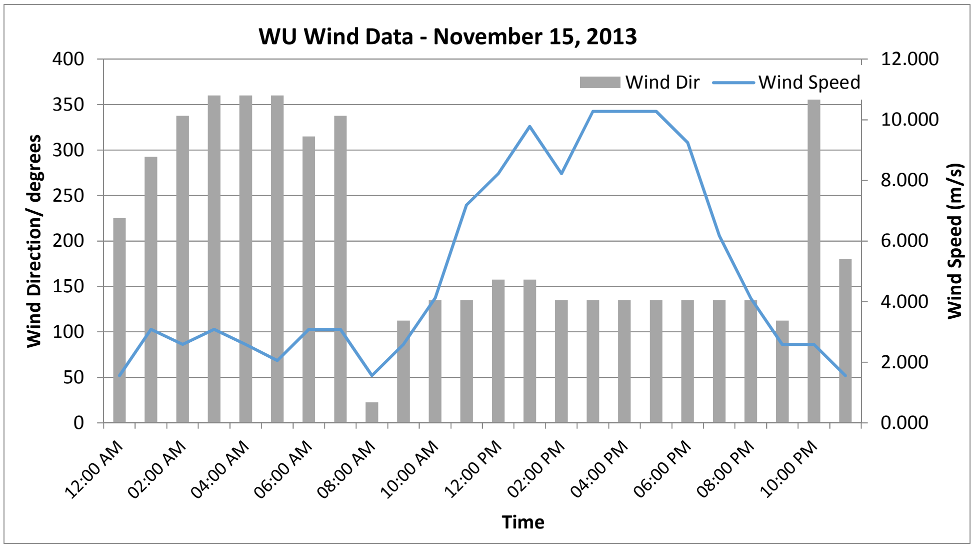


Figure 2.15 Weather Underground wind directions and speeds for the Palisadoes on November 15, 2013

2.4.4 Summary

Wind data for our analysis was obtained from three sources – NMIA, NOAA Climate Service and Weather underground database. NMIA and NOAA provided long term wind data for periods spanning 1999 to 2007, while Weather Underground provided data for the days on which the drogue tracking missions were carried out. Both long term sources indicated that majority of the winds are from the NE to SE, NMIA determined that the average wind speed was between 8 – 20 m/s and NOAA determined the average wind speed to be between 8 – 20 m/s. Current data provided by Weather Underground indicated that most of the wind came from the SW and SE direction at an average wind speed of 4 – 5 m/s.

2.5 Grain Size Analysis

The grain size analysis was done using the Unified Soil Classification System (USCS) which is widely used for the classification of granular material. Sand samples were dried and sieved using ASTM standard sieves and analysed to determine the coefficient of uniformity, standard deviation, skewness and kurtosis. The results are further assessed in the following sections.

2.5.1 Method

Sand samples were collected for analysis along the Palisades shoreline at 8 locations on October 10, 2013, and at each location 3 samples were taken: at the beach face, back of beach and dune. The sampling locations are shown below in Figure 2.16 and the results of this analysis will be incorporated into the dune design outlined in a later section of this report.

Core samples were also taken at 8 points within the offshore sand reserve (burrow area) identified within the Cuban study (Juanes)on October 16 and 17, 2013 to confirm that this material is indeed suitable for use in the project. NEPA has granted approval for dredging this burrow area for the execution of the dune nourishment section of the project based on the original proposal completed by the NWA and the Cuban Government. See Figure 2.17 for the sample locations.

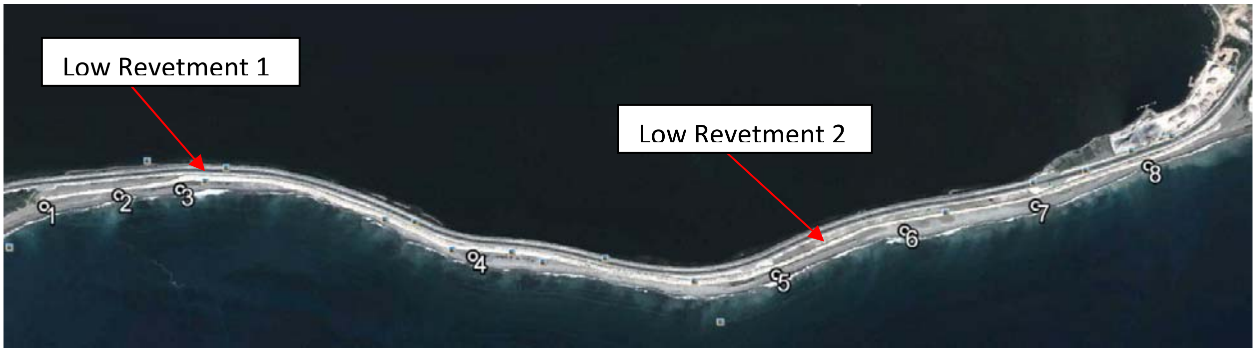


Figure 2.16 Google imagery showing location of sand samples collected along the Palisadoes

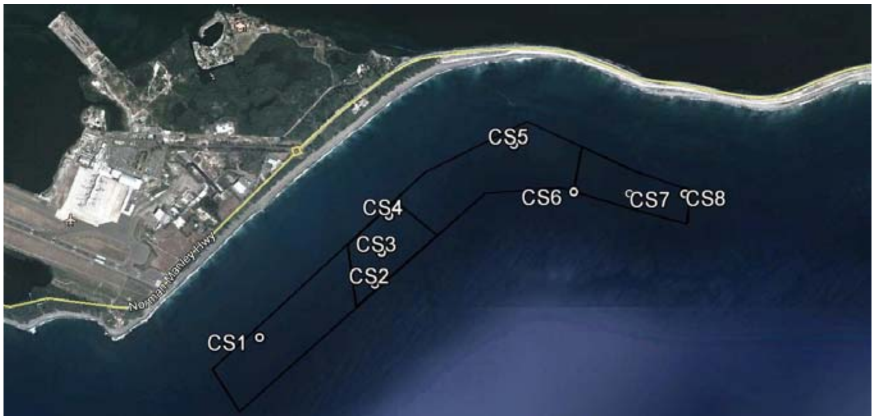


Figure 2.17 Google imagery showing offshore sample locations inside outlined sand reserve

2.5.2 Results

The grain size analysis for the shoreline samples provided the following results (see Table 2-9, Table 2-11 and Table 2-11):

- For low revetment 1 (locations 1 to 3) all the samples were of coarse to very coarse sand with a mean grain size ranging from 0.69 mm to 1.36 mm.
- For low revetment 2 the sand ranged from coarse sand to gravel (locations 5 to 7) having a mean grain size ranging between 0.93 mm to 4.32 mm.
- Along the high revetments (locations 4 and 8) the sand was on average very coarse sand, with the mean grain size ranging between 0.70 mm and 1.94 mm for the beach face, dune and back of beach, except along the beach face (location 8) .

The percentage finer grain plot for the samples is shown in Figure 2.18, Figure 2.19 and Figure 2.20 and they indicate that the sand along the beach face and sand dune is coarser than that at the back of beach.

Table 2-9 Grain size analysis results for sand samples collected along the Palisadoes' beach face at the 8 sample locations

GRAIN SIZE ANALYSIS RESULTS								
Location	1	2	3	4	5	6	7	8
Mean Grain size (mm)	1.07	0.77	0.80	0.95	1.68	1.15	3.77	1.42
Mean (phi)	-0.097	0.384	0.330	0.074	-0.744	-0.197	-1.913	-0.508
Description	V. coarse sand	coarse sand	coarse sand	coarse sand	V. coarse sand	V. coarse sand	gravel	V. coarse sand
Percentage silt	0.003	0.001	0.000	0.000	0.000	0.001	0.000	0.000
Percentage >0.06mm and <6.0 mm	0.997	0.995	0.803	0.998	0.992	0.960	0.729	0.893
Uniformity Coefficient	3.631	1.600	2.336	1.688	1.772	2.836	1.915	2.076
Standard Deviation	1.213	0.544	1.013	0.642	0.521	1.041	0.588	0.685
	poorly sorted	moderately well sorted	poorly sorted	moderately well sorted	moderately well sorted	poorly sorted	moderately well sorted	moderately well sorted
Skewness	0.376	0.654	0.118	-0.217	-1.523	-0.318	-2.148	-0.576
	strongly positively skewed	strongly positively skewed	positively skewed	negatively skewed	strongly positively skewed	strongly positively skewed	strongly positively skewed	strongly positively skewed
Kurtosis	1.115	1.463	0.294	0.811	1.147	1.017	-0.661	0.723
	leptokurti c	leptokurtic	extremely leptokurtic	platykurtic	leptokurtic	mesokurtic	extremely leptokurtic	platykurtic

Table 2-10 Grain size analysis results for sand samples collected along the Palisadoes' back of beach at the 8 sample locations

GRAIN SIZE ANALYSIS RESULTS								
Location	1	2	3	4	5	6	7	8
Mean Grain size (mm)	1.16	0.69	1.36	1.94	0.93	2.21	1.89	0.70
Mean (phi)	-0.211	0.525	-0.445	-0.957	0.105	-1.143	-0.919	0.513
Description	very coarse sand	coarse sand	very coarse sand	very coarse sand	coarse sand	gravel	very coarse sand	coarse sand
Percentage silt	0.001	0.000	0.000	0.000	0.001	0.000	0.000	0.000
Percentage >0.06mm and <6.0 mm	0.999	0.994	0.986	0.857	0.924	0.855	0.873	0.945
Uniformity Coefficient	1.509	1.858	3.098	1.810	1.889	2.148	3.013	1.766

Standard Deviation	0.445	0.807	0.910	1.012	0.869	0.453	0.790	0.955
	well sorted	moderately sorted	moderately sorted	poorly sorted	moderately sorted	well sorted	moderately sorted	moderately sorted
Skewness	-0.614	0.331	-0.293	0.482	-0.195	-0.631	-0.301	0.267
	strongly positively skewed	strongly positively skewed	positively skewed	strongly positively skewed	negatively skewed	positively skewed	strongly positively skewed	positively skewed
Kurtosis	0.820	0.821	1.053	0.276	0.957	0.347	0.445	2.640
	platykurtic	platykurtic	mesokurtic	extremely leptokurtic	mesokurtic	extremely leptokurtic	very platykurtic	very leptokurtic

Table 2-11 Grain size analysis results for sand samples collected along the Palisadoes' dunes at sample locations 1 thru 7

GRAIN SIZE ANALYSIS RESULTS							
Location	1	2	3	4	5	6	7
Mean Grain size (mm)	0.71	1.03	0.71	1.51	0.86	4.32	1.00
Mean (phi)	0.491	-0.039	0.487	-0.596	0.223	-2.110	0.001
Description	coarse sand	V. coarse sand	coarse sand	V. coarse sand	coarse sand	gravel	coarse sand
Percentage silt	0.001	0.000	0.000	0.001	0.001	0.000	0.002
Percentage >0.06mm and <6.0 mm	0.998	0.999	0.971	0.944	0.979	0.561	0.996
Uniformity Coefficient	2.674	2.103	2.333	2.431	1.887	3.243	1.944
Standard Deviation	0.874	0.681	1.002	0.851	0.713	-0.230	0.670
	moderately sorted	moderately well sorted	poorly sorted	moderately sorted	moderately sorted	well sorted	moderately well sorted
Skewness	0.745	0.009	0.348	-0.720	0.114	8.498	0.009
	strongly positively skewed	nearly symmetrical	strongly positively skewed	strongly positively skewed	positively skewed	V. strongly positively skewed	nearly symmetrical
Kurtosis	1.177	0.889	1.317	1.135	1.254	-144.536	0.922
	leptokurtic	platykurtic	leptokurtic	leptokurtic	leptokurtic	extremely leptokurtic	mesokurtic

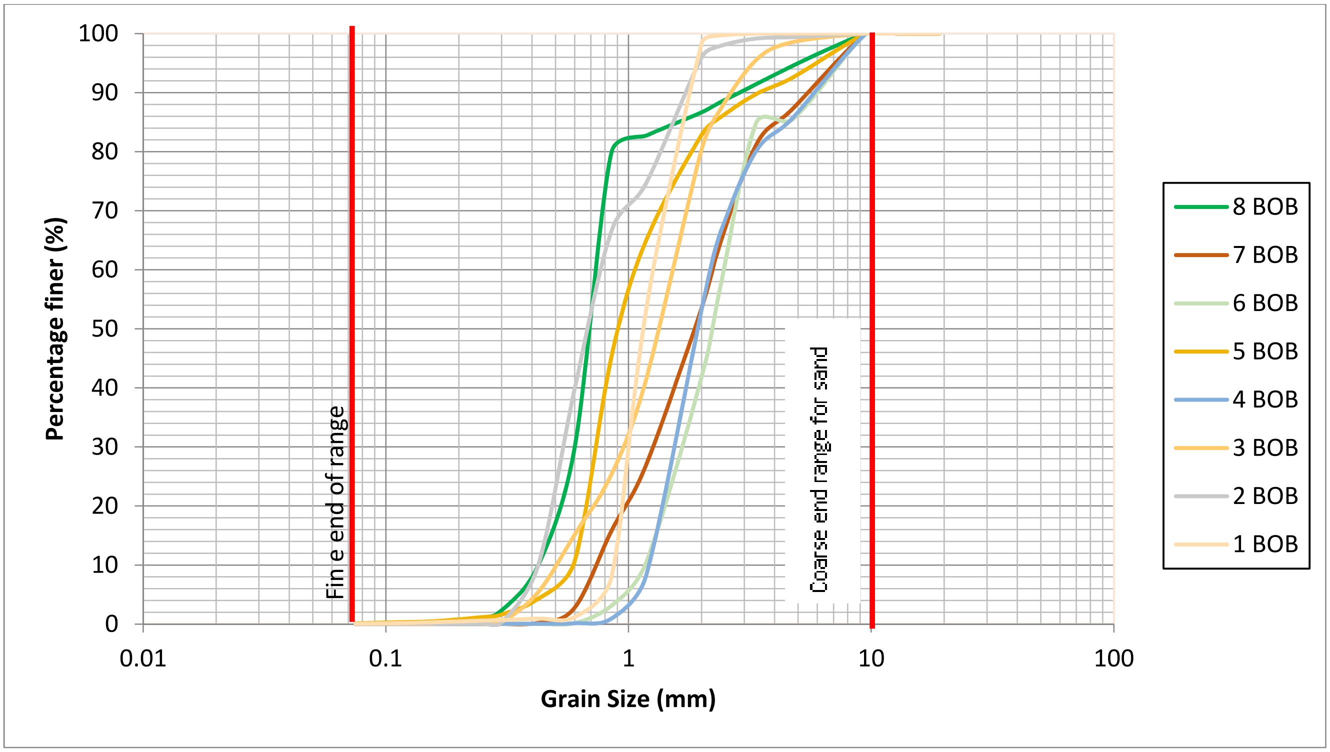


Figure 2.18 Graph showing the grain size results for the sand taken from the Palisadoes along its back of beach (BOB)

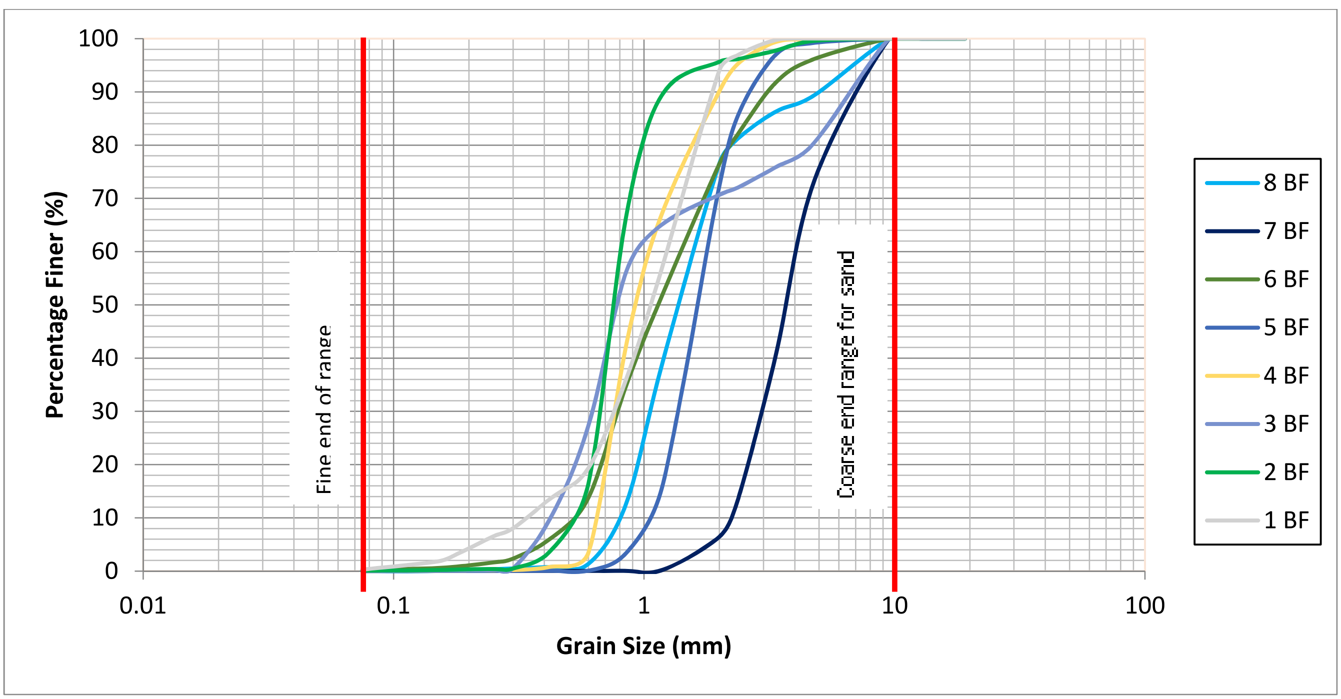


Figure 2.19 Graph showing the grain size plots for the sand taken from the Palisadoes along its beach face (BF)

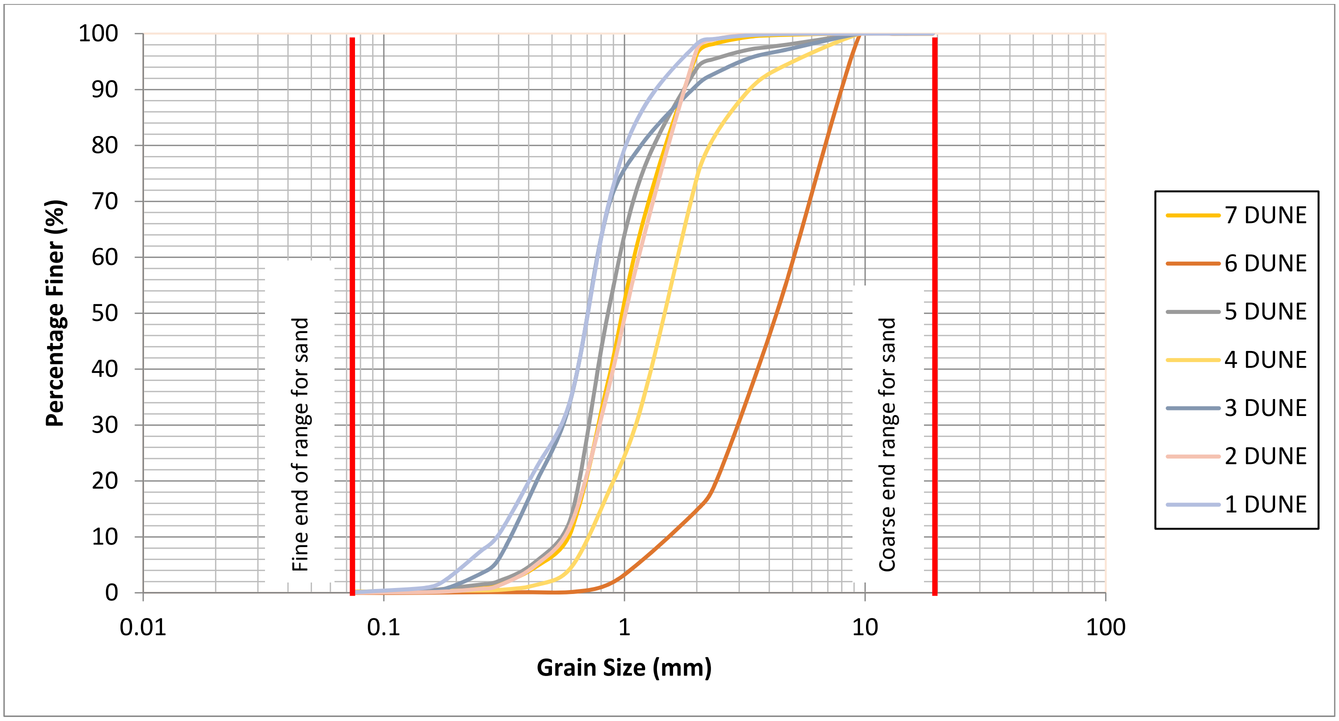


Figure 2.20 Graph showing the grain size results for the sand taken from the burrow area proposed for the Palisadoes sand dune

The grain size analysis for the offshore samples provided the following results:

- The grain size within the area range from fine sand (0.16 mm) to coarse sand (0.60 mm).
- The coarsest sand is found in sample locations CS2, CS3, CS7 and CS8 ($d_{50} \geq 0.50$mm), and this sand is the most suitable for the dune nourishment exercise as outlined in the previously submitted Material Assessment Report (CEAC Solutions Co. Ltd.). The sample results for these 4 locations are highlighted in Table 2-12.

The percentage finer grain plot for the samples is shown in Figure 2.21:

Table 2-12 Grain size analysis for sand samples collected from the sand reserve (borrow area). The most suitable, and coarsest sand is found in the area of CS2, CS3, CS7 and CS8 (highlighted in red).

GRAIN SIZE ANALYSIS RESULTS FOR SAMPLES COLLECTED FROM THE BORROW AREA								
Sample ID	**CS1**	**CS2**	**CS3**	**CS4**	**CS5**	**CS6**	**CS7**	**CS8**
Mean Grain size (mm)	0.164	0.604	0.588	0.306	0.379	0.460	0.515	0.550
Mean (phi)	2.611	0.727	0.766	1.709	1.401	1.121	0.957	0.862
Description	Fine sand	coarse sand	coarse sand	medium sand	medium sand	medium sand	coarse sand	coarse sand
Percentage silt	10.86%	0.23%	1.0%	9.2%	0.7%	2.7%	0.2%	0.3%

Percentage >0.06mm and <6.0 mm	89%	98%	99%	90%	99%	97%	99%	94%
Uniformity Coefficient	0.000	2.232	4.199	4.545	1.707	2.998	1.822	1.923
Standard Deviation	1.060	0.851	1.197	1.453	1.123	1.029	0.848	1.123
	poorly sorted	moderately sorted	poorly sorted	poorly sorted	poorly sorted	poorly sorted	moderately sorted	poorly sorted
Skewness	3.865	0.904	0.912	1.413	1.005	1.269	0.939	0.332
	V. strongly positive skewed	strongly positive skewed	strongly positive skewed	V. strongly positive skewed	V. strongly positive skewed	V. strongly positive skewed	strongly positive skewed	strongly positive skewed
Kurtosis	3.097	1.352	1.405	1.389	1.670	1.365	1.289	1.568
	extremely leptokurtic	leptokurtic	leptokurtic	leptokurtic	very leptokurtic	leptokurtic	leptokurtic	very leptokurtic

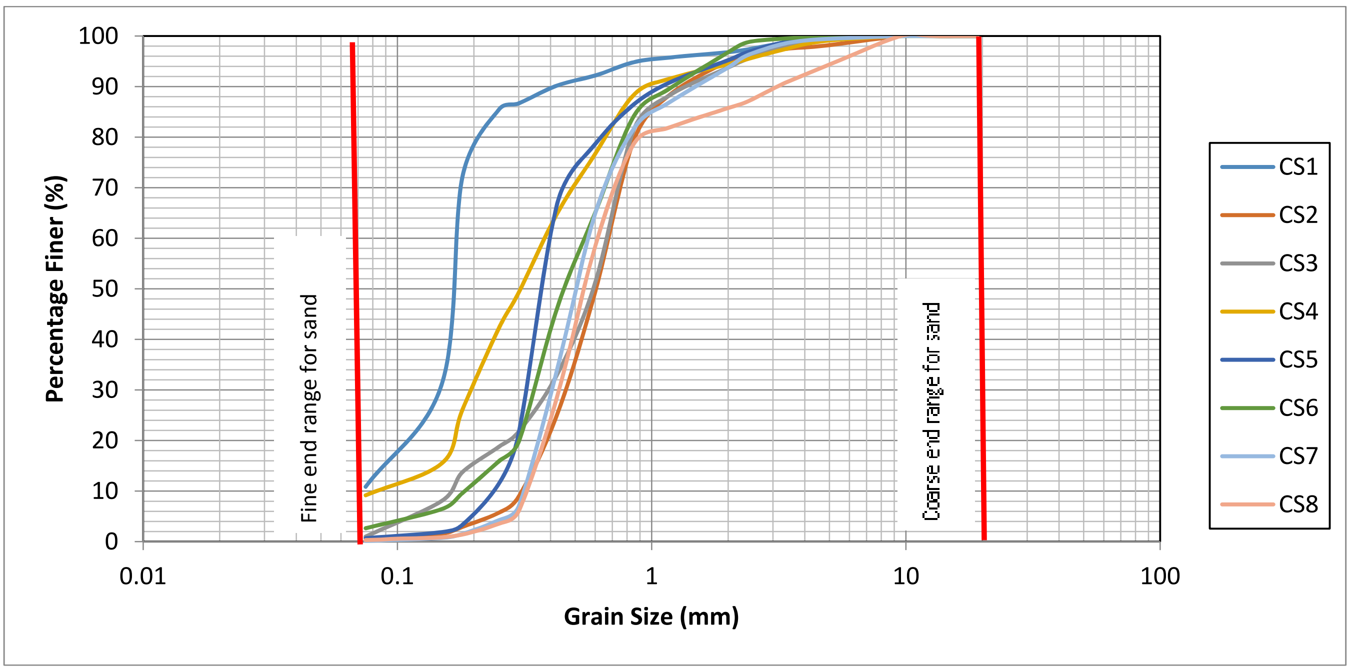


Figure 2.21 Graph showing the grain size plots for the sand samples from the sand reserve (borrow area)

2.5.2.1 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 $U_C = D_{60}/D_{10}$, 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. A plot of the uniformity coefficients are shown in Figure 2.22 and Figure 2.23.

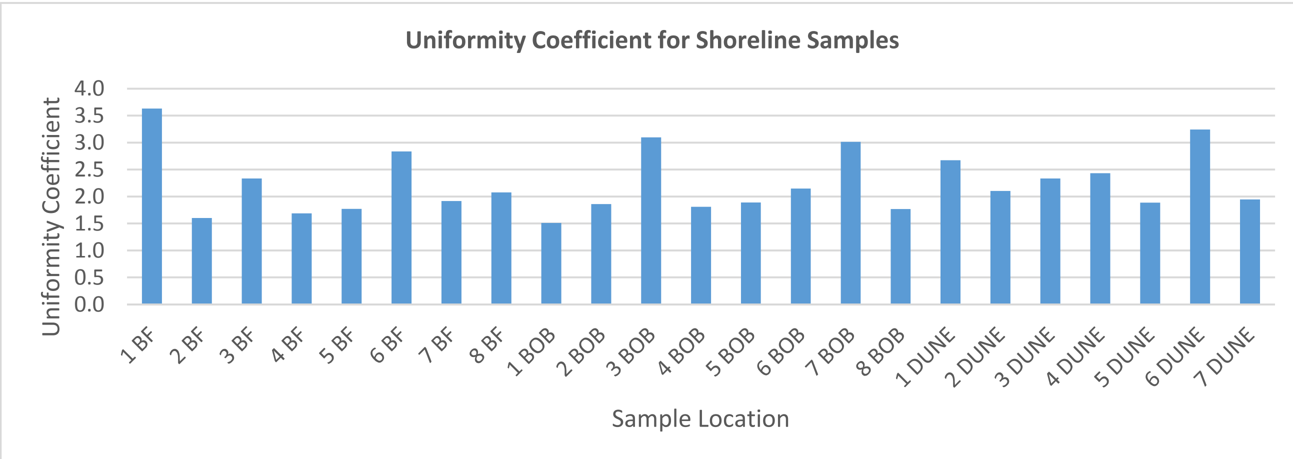


Figure 2.22 Uniformity coefficient for the sand samples taken from the shoreline

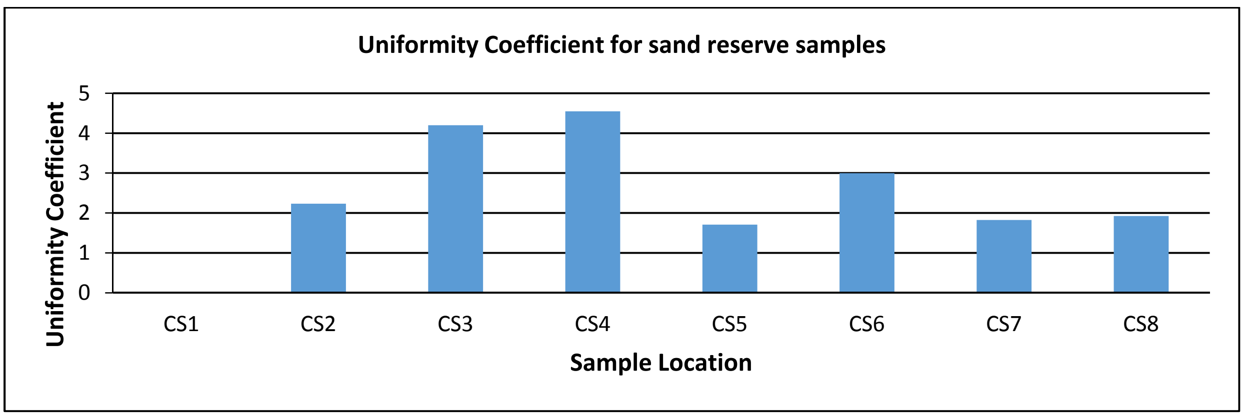


Figure 2.23 Uniformity coefficient for the samples from the sand reserve

The uniformity co-efficient of the shoreline samples ranges from 1.5 to 3.6, indicating that the samples are in the poorly sorted to well sorted range. While the uniformity co-efficient of the offshore reserve samples range from poorly to moderately sorted (0 to 4.5).

2.5.2.2 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. The majority of the shoreline samples were well sorted which is indicative of relatively high wave energy at the shoreline which sorts the particles into their discrete sizes. The sample taken from the dune in location 6 is an exception as it had a negative standard deviation because it comprised of very coarse sand (d_{50} = 4.3 mm). The opposite was true for the sand reserves samples, as majority of the samples had a standard deviation of 1 or greater indicating that they were poorly sorted. The standard deviation plots of the samples are shown in Figure 2.24 and Figure 2.25.

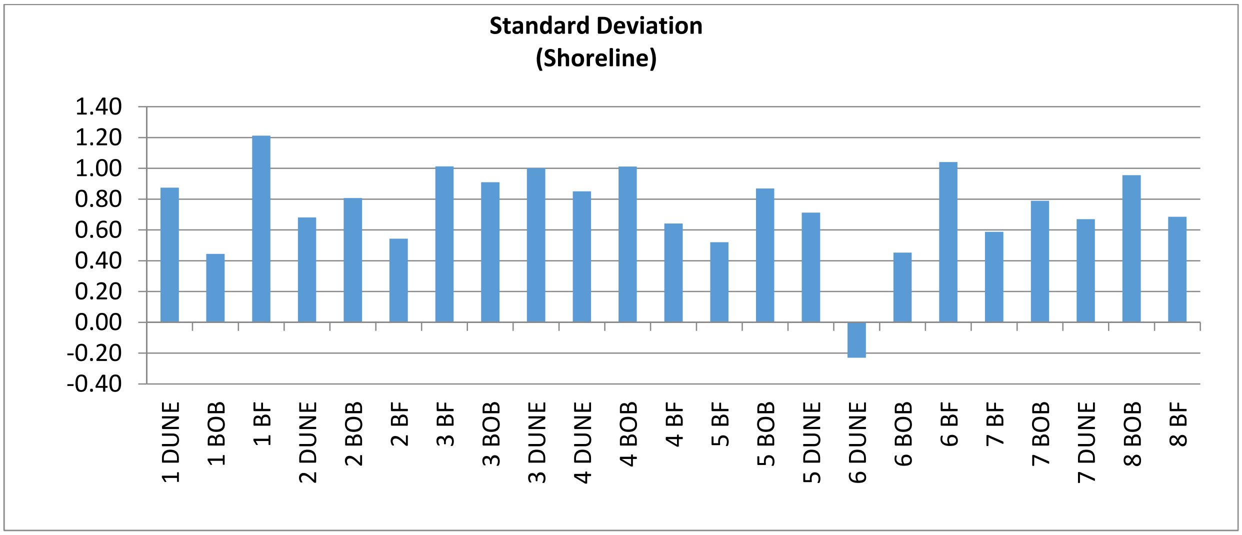


Figure 2.24 Graph showing standard deviation for the shoreline sand samples

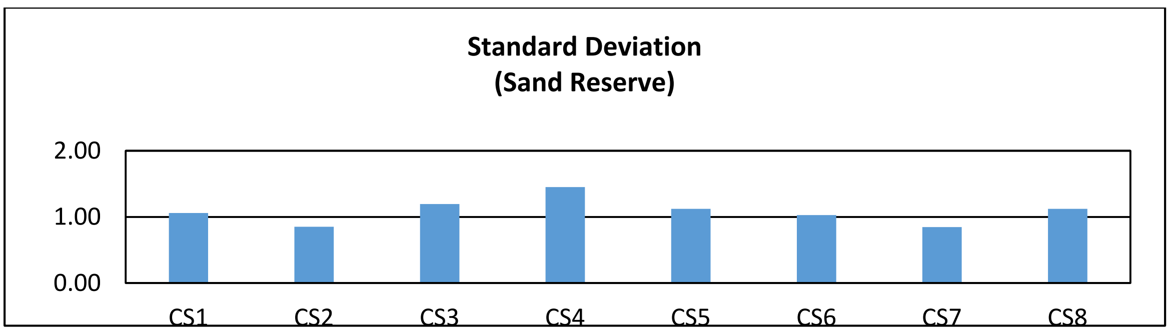


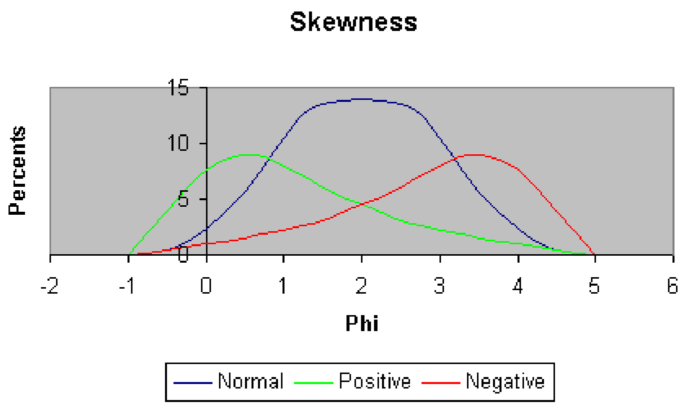
Figure 2.25 Graph showing standard deviation for the sand reserve samples

2.5.2.3 Skewness

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

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

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:

Table 2-13 Verbal limits for 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 shoreline samples ranged from negatively (-2.1) skewed to strong positively skewed (8.5), most were within the positively skewed to strongly positively skewed range indicating the presence of excess fines. While the sand reserve samples ranged from positively (0.3) to strongly positive (3.9) indicating that they have excess fine material, see Figure 2.26 and Figure 2.27.

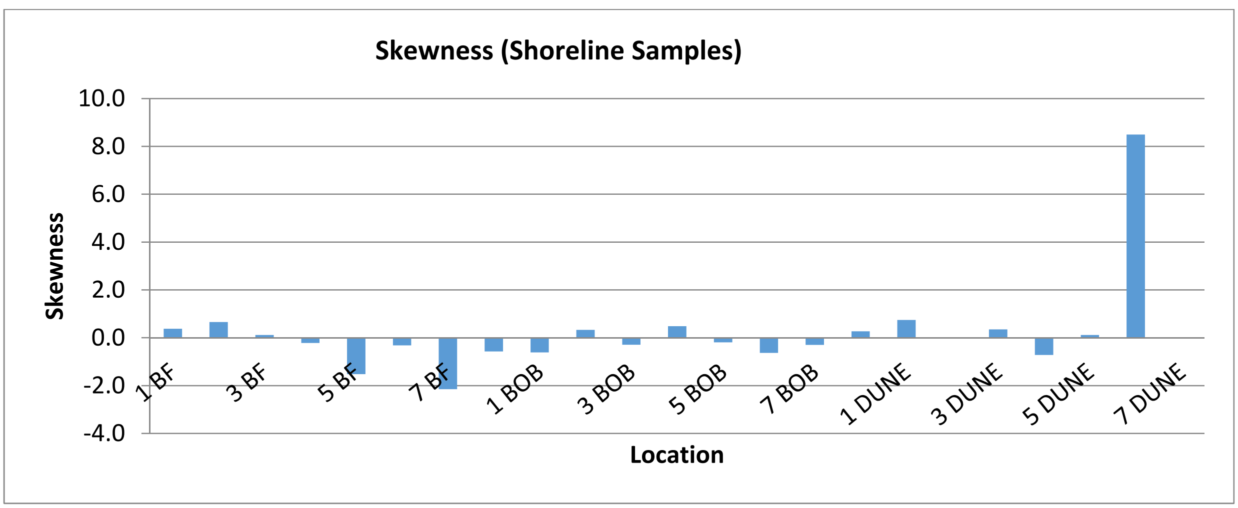


Figure 2.26 Graph showing skewness for the shoreline samples

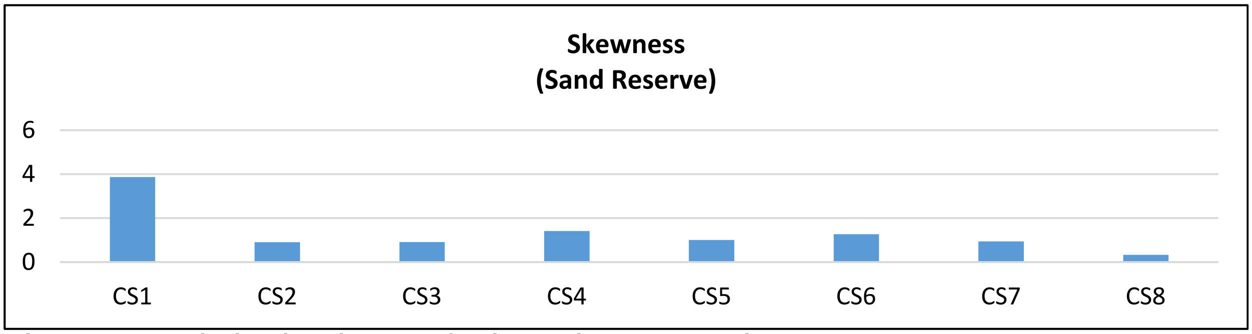
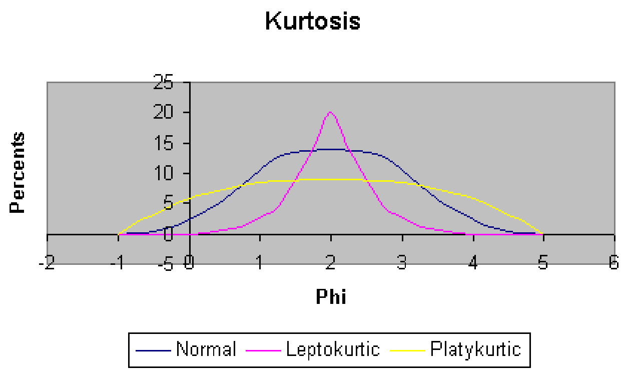


Figure 2.27 Graph showing skewness for the sand reserve samples

2.5.2.3.1 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. 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. The following verbal limits are suggested for values of kurtosis:

Table 2-14 Verbal limits for 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

Kurtosis was determined to be generally within the range 0.3 to 2.6 for the shoreline samples while the sand reserve samples ranged from 1.3 to 3.0. Plots of the kurtosis values are shown in Figure 2.28 and Figure 2.29.

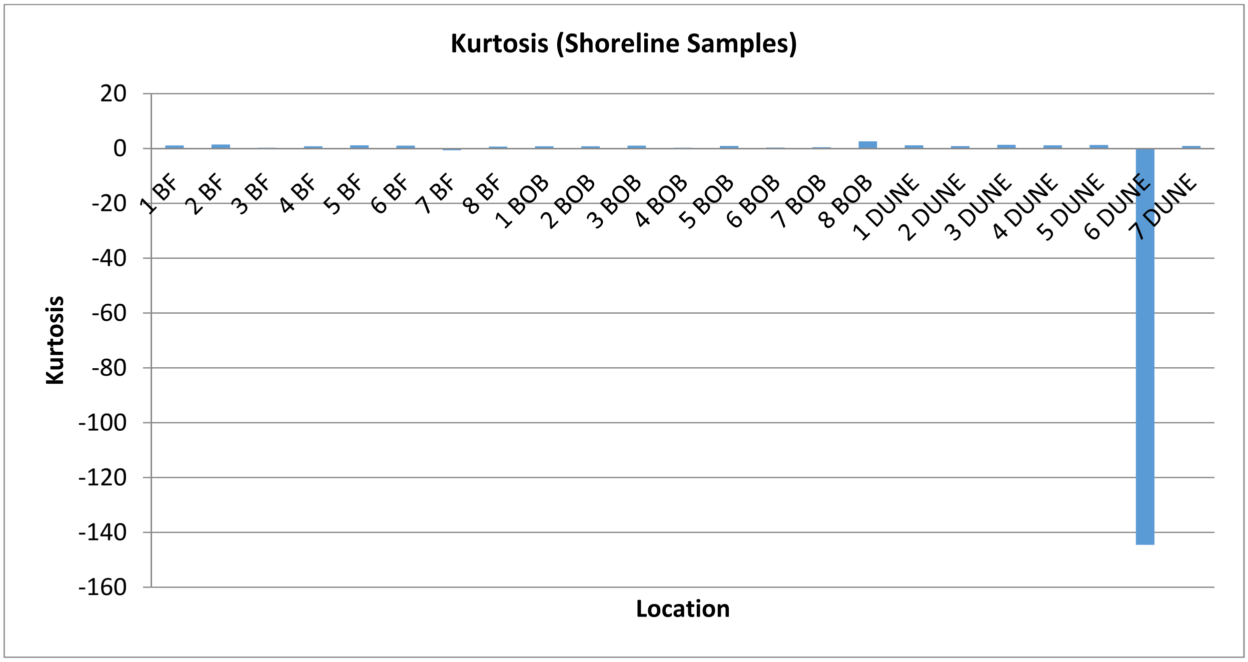


Figure 2.28 Graph showing the kurtosis results for the shoreline sand samples

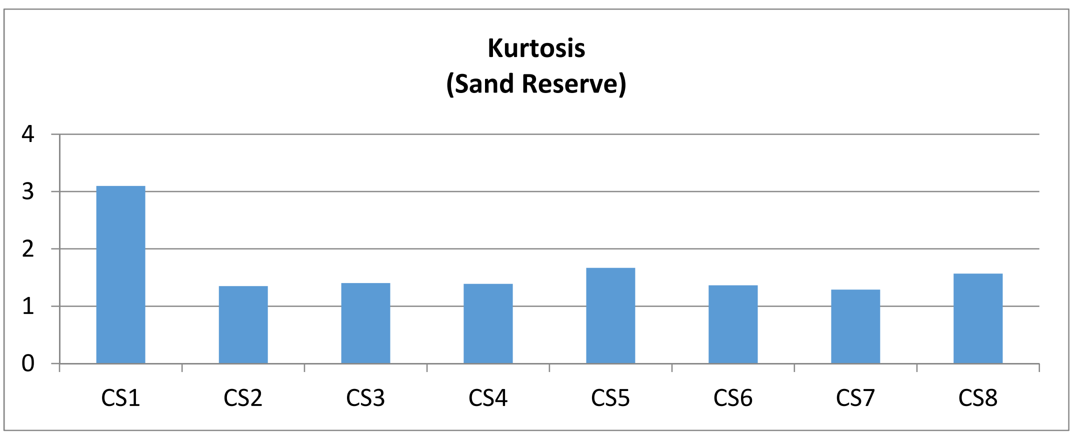


Figure 2.29 Graph showing the kurtosis results for the sand samples from the sand reserve

2.5.3 Suitability Comparison

The shoreline samples and the reserve samples were compared to determine the suitability of the reserve sand for use on the shoreline in creating the dunes. First, a visual inspection was done. Plate 2-1 below shows that there is not much difference in the color of the samples however the back of beach samples are distinctly coarser than burrow area samples.

Second, the grain size analysis results for the shoreline and sand reserve samples were compared and it revealed that the mean grain sizes of the sand reserve samples are smaller than that of the shoreline samples. This is presented in greater detail in the Material Assessment Report. The percentage finer than grain size were also plotted for both sets of samples. The dunes samples were first plotted and then upper and lower bounds were fitted that would encompass majority of the samples. The same upper and lower bounds were placed on a plot of the sand reserve samples to see how they match up. The results are shown below in Figure 2.30. From the plots we can see that the sand reserve samples in the vicinity of CS2, CS3, CS7 and CS8 fall within the bounds (black lines) set by the shoreline samples and have a mean grain size between 0.5 – 0.7 mm. This mean grain size will be used in the dune design modeling exercise.

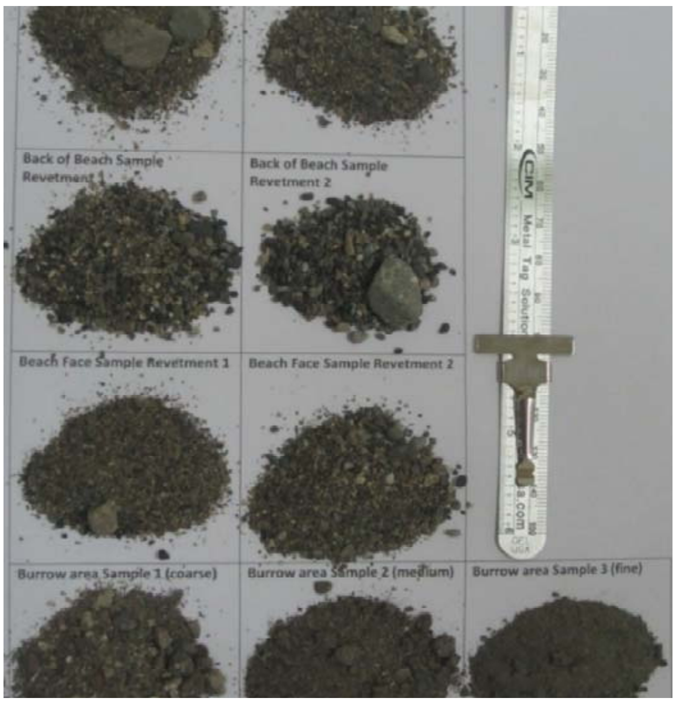


Plate 2-1 Photograph of sand samples collected from the sand reserve (borrow area) and the shoreline

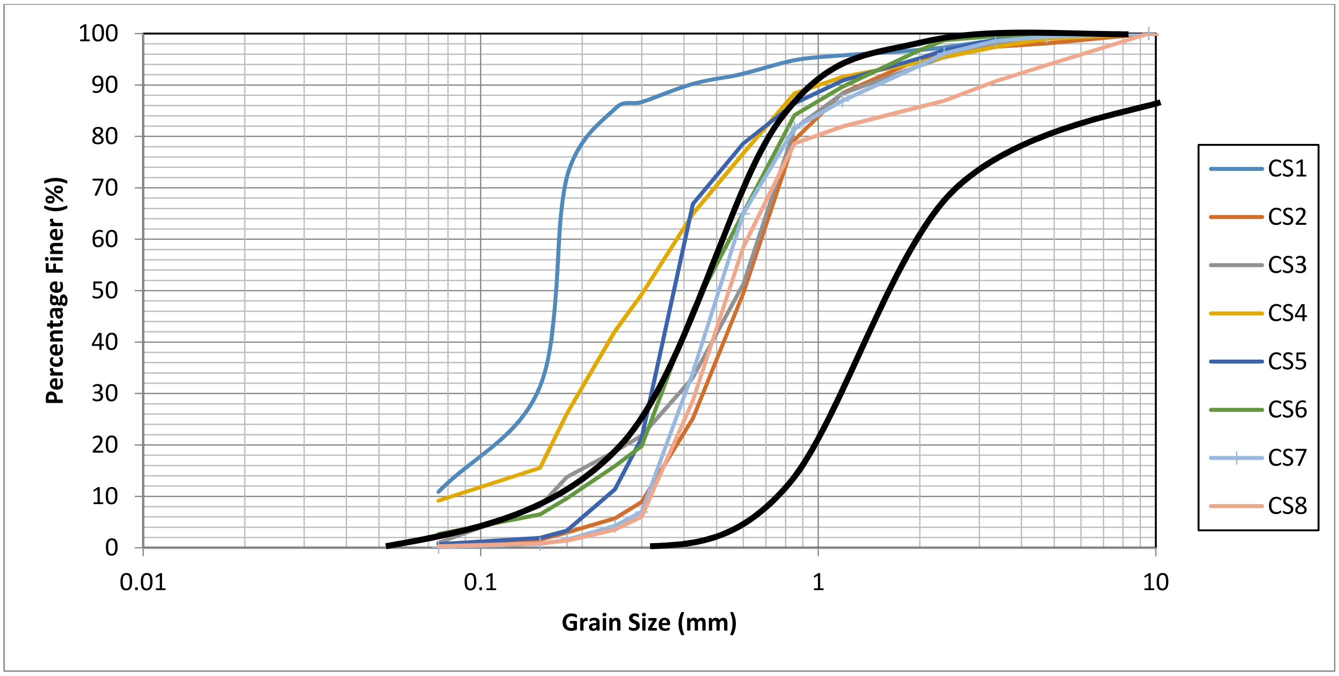


Figure 2.30 Graph showing the grain size for the sand reserve samples with the upper and lower bounds for the shoreline samples (in black) indicating where suitable sand is located

Table 2-15 present the volume of sand suitable for covering the low revetments available within the borrow area and Figure 2.31 shows the location of the suitable sand within the borrow area.

Table 2-15 Estimated fill volume available in the borrow area when dredged to a depth of 1.5 m

Proposed Dredge Area	Area (m^2)	Volume (m^3)
1	87,432	59,132
2	39,421	131,148
Total	**126,853**	**190,280**

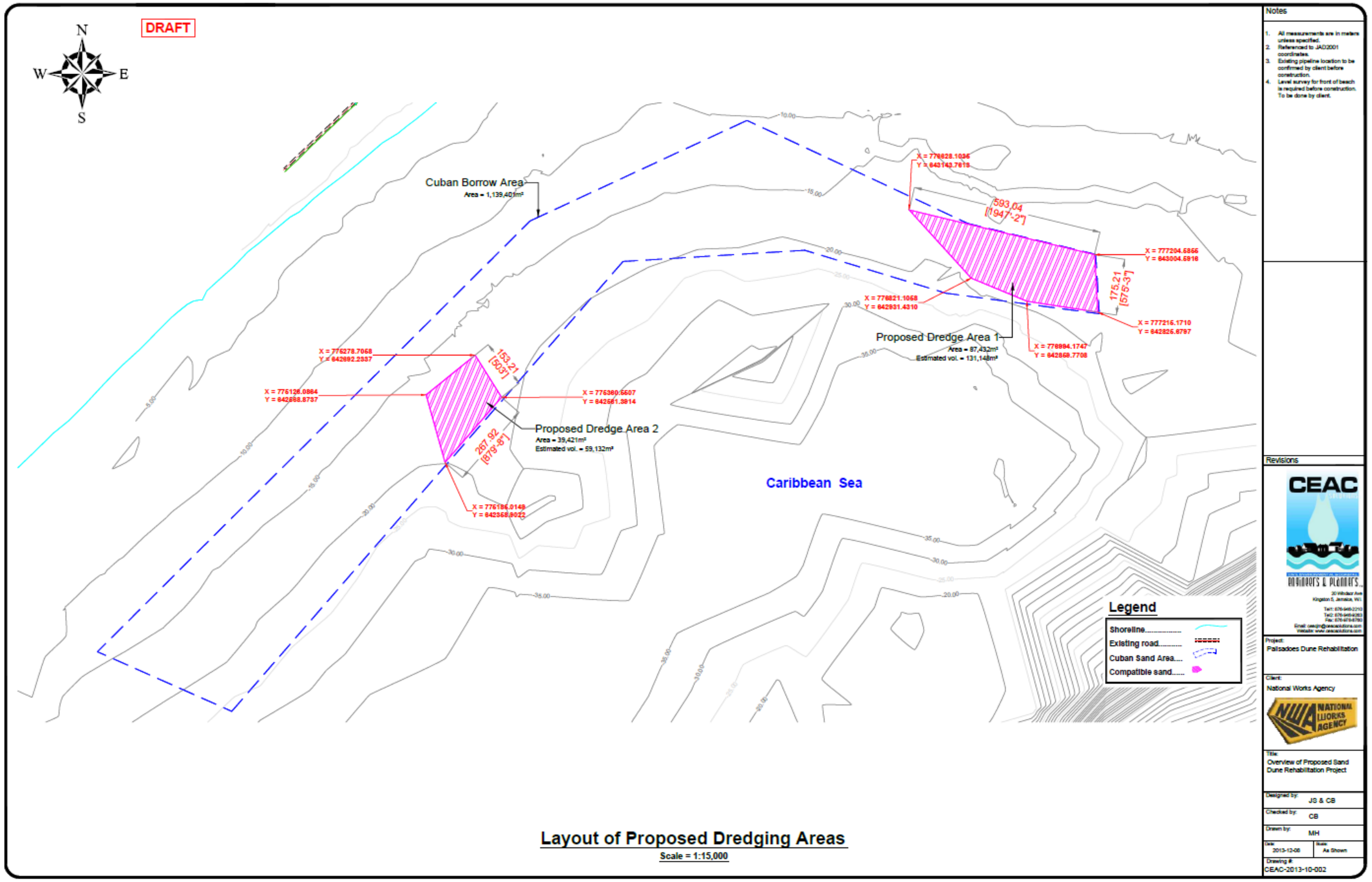
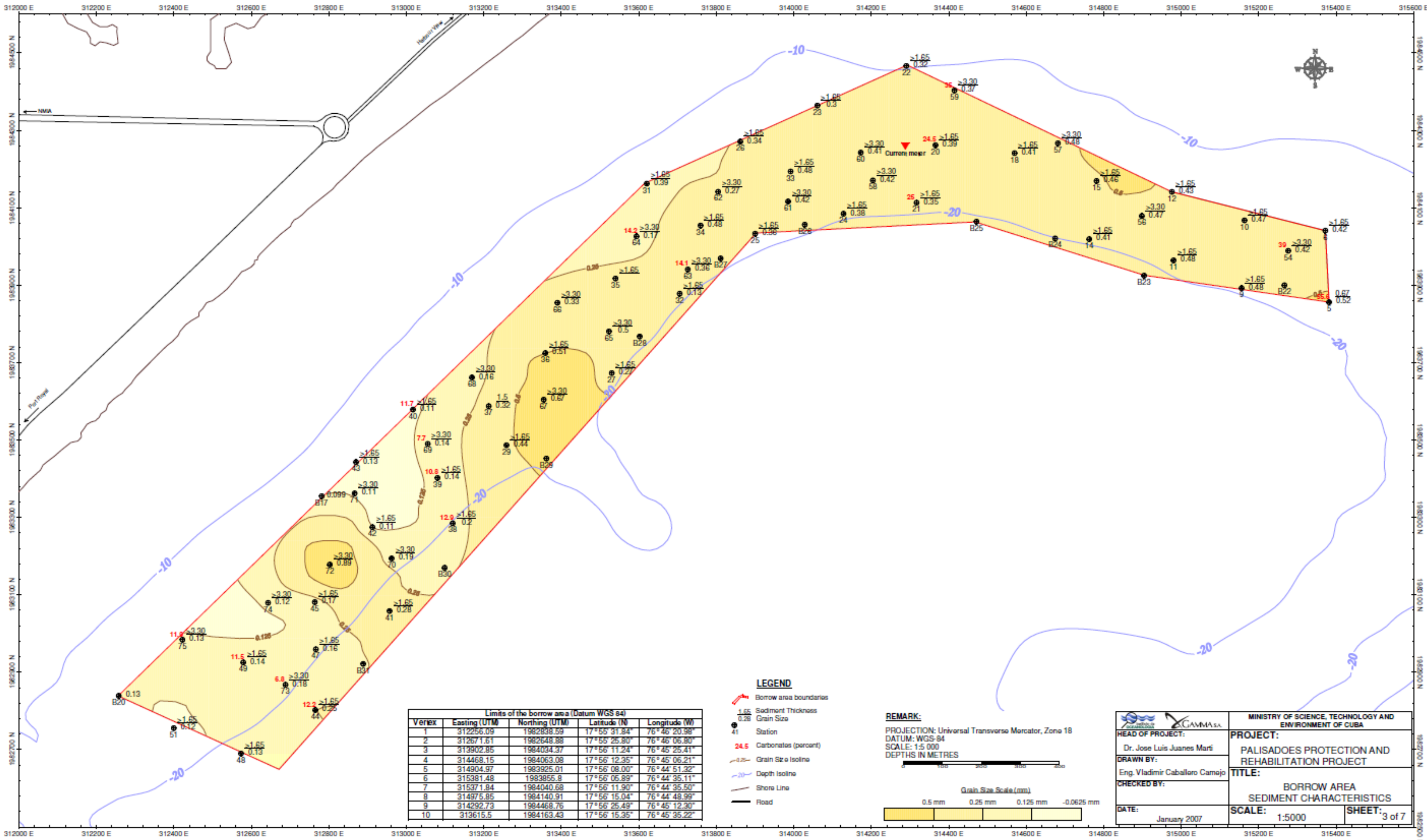


Figure 2.31 Location of sand with a mean grain size of between 0.5 – 0.7 mm within the borrow area. This area should be dredged to a depth of 1.5 m to obtain the required volume



Limits of the borrow area (Datum WGS 84)

Vertex	Easting (UTM)	Northing (UTM)	Latitude (N)	Longitude (W)
1	312256.09	1982838.59	17°55' 31.84"	76°46' 20.98"
2	312671.61	1982648.88	17°55' 25.80"	76°46' 06.80"
3	313902.85	1984034.37	17°56' 11.24"	76°45' 25.41"
4	314468.15	1984063.08	17°56' 12.35"	76°45' 06.21"
5	314904.97	1983925.01	17°56' 08.00"	76°44' 51.32"
6	315381.48	1983855.8	17°56' 05.89"	76°44' 35.11"
7	315371.84	1984040.68	17°56' 11.90"	76°44' 35.50"
8	314975.85	1984140.91	17°56' 15.04"	76°44' 48.99"
9	314292.73	1984468.76	17°56' 25.49"	76°45' 12.30"
10	313615.5	1984163.43	17°56' 15.35"	76°45' 35.22"

Figure 2.32 Location of sand samples taken from the borrow area by the Cuban technical team

2.5.4 Composition of Sand from the Borrow Area

In the Cuban study 97 sand samples were collected from the borrow area and tested to determine their characteristics and composition. The tests determined that the mean grain size of sand in this area varies between 0.11 – 0.89 mm and the standard deviation varies between 0.5 – 1.0. The tests also determined that the material was predominantly terrigenous, non-carbonated material which compares well with the sediment formations found in the region and presented in other studies. This study also determined that the carbonate composition of sand within the priority borrow areas varies between 7 – 17%, see Table 2-16. Table 2-17 and Table 2-18 present the test results for the sand samples taken from Proposed Dredge Area 1 and 2 as identified in Figure 2.31 and Figure 2.32. It also presents the maximum, minimum and average values of these results and compares them with the sediment analysis results obtained for the sand samples we took from same.

The analysis indicated that the sand samples from both studies are similar, producing comparable results for the mean grain size, mean phi, kurtosis values and standard deviation. The mean grain size varied between 0.5 and 0.6 mm, and the mean phi varied between 0.9 and 1.1. The kurtosis values for both samples were greater than 1.1, indicating that the samples were excessively peaked, having the sand in the centre of the distribution better sorted than at the ends. The standard deviation for both samples was also moderately to poorly sorted.

Table 2-16 Chemical composition of sand from the proposed dredge areas within the borrow areas determined by the Cuban study

Constituent	Proposed Dredge Area 1 (%)	Proposed Dredge Area 2 (%)
Total Carbonate	17.07	7.01
Crystalline	17.48	0.00
Opaque	0.81	0.00
Rock Fragments	10.57	0.00
Amphibole	0.81	1.40
Feldspar	2.85	4.67
Quartz	50.41	86.92

Table 2-17 Composition of sand in Proposed Dredge Area 1 as determined by both the Cuban and CEAC study

Sample	Depth (m)	D50 (mm)	Bottom Description and laboratory information	Mean (ϕ)	Std Dev	Skewness	Kurtosis
M5	20.6	0.52	Sandy bottom. Course sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a low density. Sediment thickness: 0.67 m	0.93	0.46	-0.43	6.49
M6	20.2	0.42	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a low density. Sediment thickness: > 1.5 m	1.26	0.3	-0.81	11.9

M9	19	0.48	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a low density. Sediment thickness: > 1.5 m	1.06	0.72	-1.02	5.35
M10	18	0.47	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a low density. Sediment thickness: > 1.5 m	1.23	0.64	-2.01	8.8
M11	19	0.48	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a very low density. Sediment thickness: > 1.5 m	1.16	1.2	-0.92	3.2
M12	19	0.43	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a low density. Sediment thickness: > 1.5 m	1.28	0.56	-1.7	8.75
M15	18.4	0.46	Sandy bottom. Medium sand. Beige-white color. Absence of benthic organisms. Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 1.5 m	1.48	0.53	-0.62	8.62
M18	15	0.41	Sandy bottom. Medium sand. Beige-white color. Absence of benthic organisms. Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 1.5 m	1.36	0.51	-1.46	8.49
M54	20	0.42	Sandy bottom. Medium sand. Beige-white color. Absence of benthic organisms. Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 3.3 m	1.25	0.61	-1.62	9.97
M56	19	0.47	Sandy bottom. Medium sand. Beige-white color. Absence of benthic organisms. Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 3.3 m	1.28	0.71	-1.55	8.08
Max	**20.60**	**0.52**		**1.48**	**1.20**	**-0.43**	**11.90**
Min	**15.00**	**0.41**		**0.93**	**0.30**	**-2.01**	**3.20**
Average	**18.82**	**0.46**		**1.23**	**0.62**	**-1.21**	**7.97**
CS7	**17.98**	**0.52**	**Coarse sand, dark grey in colour; shells and corals present in the sample**	**0.96**	**0.85**	**0.94**	**1.29**
CS8	**18.77**	**0.55**		**0.86**	**1.12**	**0.33**	**1.57**

Table 2-18 Composition of sand in Proposed Dredge Area 2 as determined by both the Cuban and CEAC study

Sample	Depth (m)	D50 (mm)	Bottom Description and laboratory information	Mean (ϕ)	Std Dev	Skewness	Kurtosis
M27	20	0.27	Sandy bottom. Medium sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and coral were observed in a very low density. Sediment thickness: > 1.5 m	1.36	0.6	-0.39	7.1
M29	18	0.44	Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae and corals were observed in a very low density. Sediment thickness: > 1.5 m	1.06	0.6	-0.34	4.83
M36	16.3	0.51	Sandy bottom. Course sand. Beige-Black color. Absence of benthic organisms. Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 1.5 m	2.81	0.89	-1.12	3.97
M67	18	0.67	Sandy bottom. Course sand. Beige-white color. Very low density of benthic organisms (mollusks, Halimeda algae and gorgonians). Spread skeleton remains of calcareous algae were observed in a very low density. Sediment thickness: > 3.3 m	3.23	0.59	-1.64	8.24
Max	**20.00**	**0.67**		**3.23**	**0.89**	**-0.34**	**8.24**
Min	**16.30**	**0.27**		**1.06**	**0.59**	**-1.64**	**3.97**
Average	**18.08**	**0.47**		**2.12**	**0.67**	**-0.87**	**6.04**
CS2	**19.28**	**0.60**	**Coarse sand, dark grey in colour; shells and corals present in the sample**	**0.73**	**0.85**	**0.90**	**1.35**
CS3	**17.83**	**0.58**		**0.77**	**1.20**	**0.91**	**1.41**

2.5.5 Mangrove Nourishment

Sand samples were collected for analysis from the mangrove forest adjacent to the project area (Port Royal) to determine the optimal sand slope and sediment characteristics to be used in the project. UWI team is responsible for replanting the mangroves and they provided three (3) sand samples from an adjacent mangrove forest to be used in our analysis. These samples were compared with samples collected from 3 quarries in St. Thomas, 2 desilting operations in Kingston, and from the 8 points within the sand reserve to determine which source would provide the most suitable sand for mangrove nourishment. A detailed analysis was completed and submitted in the Material Assessment Report previously submitted and it determined that sand with a mean grain size between 1 – 2mm should be used and that un-sieved sand from the Hope River desilting operation would be the most suitable. Table 2-19 and Figure 2.33 provide a summary of the results and indicate that the Hope River will provide on average very coarse sand with a mean grain size of 1.9 mm which falls within the required range.

Table 2-19 Grain size analysis results for the sand samples from the mangrove forest and from the Hope River

Mangrove Samples				
Sample ID	**Fine**	**Course**	**All purpose**	**Hope River**
GRAIN SIZE ANALYSIS RESULTS				
Mean Grainsize (mm)	0.823	4.046	1.755	1.912
Mean (phi)	0.281	-2.016	-0.812	-0.935
Description	coarse sand	gravel	very coarse sand	very coarse sand
Percentage silt	0.14%	0.01%	0.1%	0.2%
Percentage >0.06mm and <6.0 mm	100%	63%	84%	66%
Uniformity Coefficient	2.593	2.376	4.937	6.109
Standard Deviation	0.829	-0.221	1.108	0.611
	moderately sorted	well sorted	poorly sorted	moderately well sorted
Skewness	0.378	16.329	0.149	1.242745
	strongly positive skewed	V. strongly positive skewed	positive skewed	V. strongly positive skewed
Kurtosis	1.008	-2.234	0.367	0.194
	mesokurtic	extremely leptokurtic	extremely leptokurtic	extremely leptokurtic

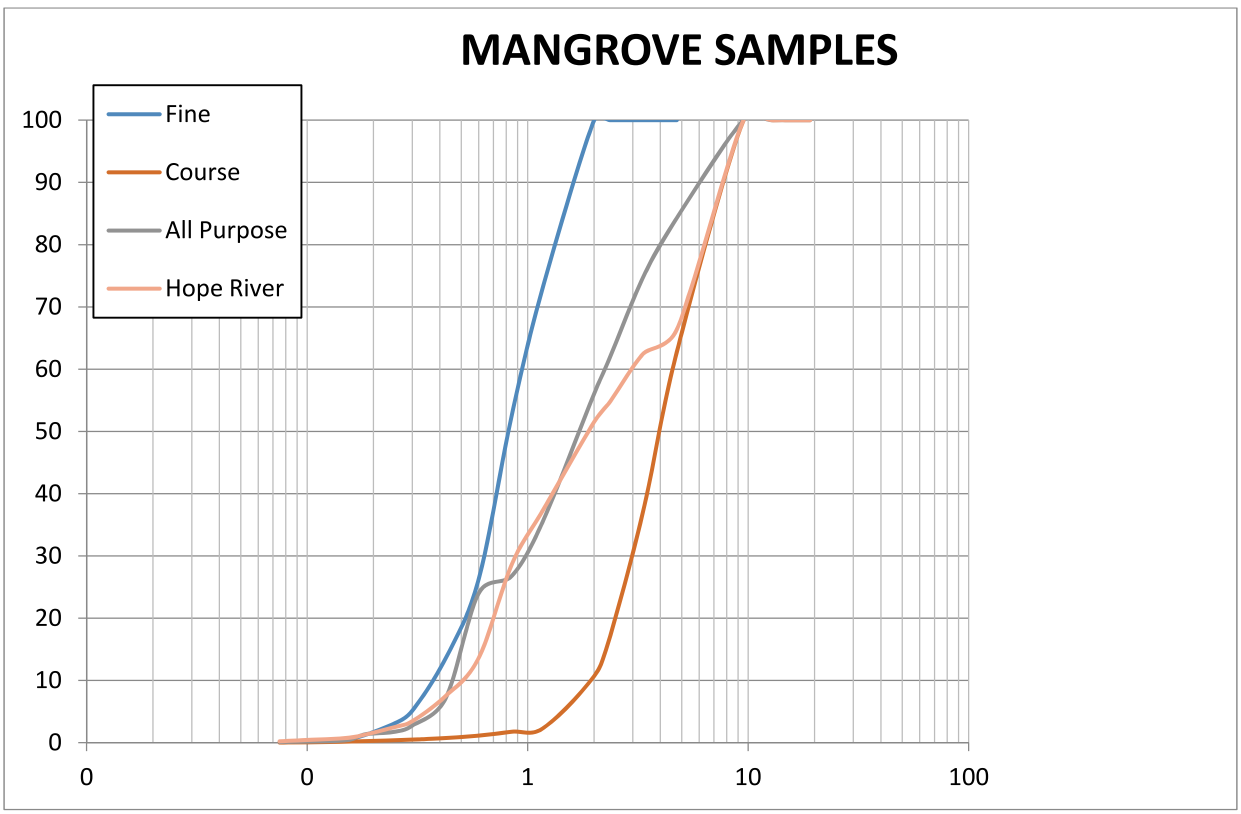


Figure 2.33 Graph showing the grain size for the sand from the mangrove forest and from the Hope River

2.5.6 Summary

Sand samples were collected along the Palisadoes shoreline and from the offshore borrow area identified by the Cuban study. The shoreline samples ranged between coarse sand and gravel (0.7 – 4.3 mm), the sand was well graded, well sorted, and most samples were positively skewed having more fines in the tail of the distribution.

The samples from the borrow area ranged between fines and coarse sand (0.2 – 0 0.6 mm), the sand was well graded, poorly sorted, and positively skewed. These results were also similar to that obtained by the Cubans for their samples collected from the same borrow area. Two priority areas within the borrow area were identified as providing coarse sand to be used for the sand dune nourishment exercise, this sand has a mean grain size ranging between 0.5 – 0.6 mm. This sand is however unsuitable for use in the mangrove nourishment exercise. Mangrove nourishment is best carried out with unsieved sand from the Hope River desilting operation.

2.6 Anecdotal Data Collection

Anecdotal evidence of past storms was collected to aid in the verification of the CSHORE and SBEACH models defined for the project. Interviews were held with persons currently residing and/ or employed in Harbour View, Port Royal and its environs. They reported that Hurricane Ivan (2004) caused the most damage to the Palisadoes and that by the end of its passing the Palisadoes was completely impassable with sand and stones brought up on the road. On average sand mounds were 4 ft high but in some areas they were as high as 6ft. (Juanes) also agreed with interviewees and reported that during Ivan 'an intense process of sand migration from Palisadoes external side toward the Kingston Harbour side took place, which had never before been observed since the event in 1722'. The sand dunes along the Palisadoes were totally destroyed, and there was inundation of the road which led to the complete shutdown of the Norman Manley International Airport (NMIA).

Interviewees also reported that since the construction of the revetments along the Palisadoes in 2010 damage to the extent caused by Ivan has not occurred, even with the passing of Hurricane Sandy in 2012 which was a Category 1 hurricane that pummeled the St. Thomas coastline, St. Mary and Portland.

3 Wave Studies and Storm Surge

3.1 Climate Change Considerations

In completing the design, considerations were made for the effect climate change would have on the design life of the dunes and mangrove nourishment areas. A study[2] was conducted by the Climate Studies Group at the University of the West Indies (UWI) Mona and this was used to inform our design approach; it assessed literature on current and projected trends in sea level rise, wave heights and storm intensities with a particular emphasis on future values for the Palisadoes, in Jamaica.

3.1.1 Current and Projected Trends for Mean and Extreme Sea Levels

At Port Royal sea level measurements indicated a 0.9 mm/ yr rising trend between 1955 and 1971. This however is much lower than global and regional trends and these trends are expected to accelerate through to the 21st century and beyond because of global warming, but their magnitude remains uncertain. Two main factors contribute to this increase: thermal expansion of sea water due to ocean warming and water mass input from land ice melt and land water reservoirs.

In Jamaica, and the region near it, the sea level rise is approximately the global average[3] of 3.2 mm/yr (± 0.4). Projected increases in global and Caribbean mean sea level by 2100 relative to the 1980-1999 is 0.37m[4] (± 0.5 m relative to global mean) and this is equivalent to 3.7 mm/yr.

3.1.2 Current and Projected Trends in Mean and Significant Wave Heights

In 2000 Wang and Swail detected statistical significant changes in the seasonal extremes of significant wave heights in the North Atlantic only for the winter (January – March) season; these changes were found to be linked with the North Atlantic Oscillation. Specifically, significant increases in significant wave heights in the Northeast North Atlantic matched by significant decreases in the subtropical North Atlantic are found to be associated with an intensified Azores High and a deepened Icelandic low.

The IPCC AR5 projects that the annual mean significant wave heights will decrease by approximately 1 – 2%. This marginal figure was however not included in the design so as to enable the dunes and mangrove nourishment areas to best withstand any possible changes to the climate change projections.

3.1.3 Current and Projected Trends in Storm Intensities

The AR5 notes that evidence suggests a virtually certain increase in the frequency and intensity of the strongest cyclones in the Atlantic since the 1970s. It is further noted that the average lifetime of North Atlantic tropical cyclones show an increasing trend 0f 0.07 day/yr for the same period which is statistically significant[5].

The AR4 concluded that a range of modeling studies project a likely increase in peak wind intensity and near storm precipitation in future tropical cyclones. Simulations consistently find that greenhouse

[2] Climate Studies Group, UWI Mona (2013), Evaluation of trends in sea levels, ocean wave characteristics and tropical storm intensities, *Report prepared for CEAC Solutions Co. Ltd.*
[3] IPCC 2013
[4] IPCC 2007
[5] Climate Studies Group, UWI Mona (2013), Evaluation of trends in sea levels, ocean wave characteristics and tropical storm intensities, *Report prepared for CEAC Solutions Co. Ltd.*

warming causes tropical cyclone intensity to shift towards stronger storms by the end of the 21st century (2 to 11% increase in mean maximum wind globally).

3.1.4 Summary

Based on the assessments and literature reviewed the following climate change factors will be incorporated into the design (Table 3-1), specifically the deep water and near shore wave climate analysis carried out in the following sections, thus ensuring the dunes can adequately withstand the future climate change environment.

Table 3-1 Summary of climate change considerations

	Present Climate		***Climate Factor (Cf)***	***Future Climate***	
	50 YR	*100 YR*		*50 YR*	*100 YR*
Water Level	0	0	3.75 mm/yr	0.139	0.139
Operational Wave Height	0.8 (o)	1.6 (s)	1 - 2 % decrease	0.8	1.6
Hurricane Wave Height	5.94	6.23	1.040	6.17	6.48
Wave Frequency (Increase)			2.2 = 100*log(A1B/CTRL)	5.2%	5.2%

3.2 Deep Water Wave Climate Analysis

3.2.1 Methodology

Wave information on the site is crucial in order to understand the likely conditions that the shoreline will be subjected to and hence adequately design the sand dunes to provide maximum protection to the shoreline.

3.2.2 Hurricane Waves

3.2.2.1 Methodology

The following procedure was carried out:

- A database of hurricanes, dating back to 1886, was searched for storms that passed within a 300 km radius of an offshore node located at Latitude 17.76 degrees North and Longitude 76.67 degrees West.
- 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.

After the database was searched the following procedure was carried out:

1. ***Extraction of Storms and Storm Parameters from the historical database***
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.

All the returned values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

3.2.2.2 Results

3.2.2.2.1 Occurrences and Directions

The results of the search from the database for hurricanes that came within the search radius of the site are shown in the Appendices. Extremal analysis results are summarized in the bi-variant Figure 3.2. The results of the search clearly indicate the sites overall vulnerability to such systems. In summary:

- 86 hurricane systems came within 300 kilometers of the project area
- 6 of which were classified as catastrophic (Category 5)
- 15 were classified as extreme (Category 4)

The bi-variant table analysis indicates that the waves generated offshore the site have approached from all seaward possible. However, the most frequent hurricane waves have been noted to come from a **south-westerly** direction, see Table 3-2. In summary, there are:

- 23 (x6 hours) occurrences from the west
- 61 (x6 hours) occurrences from the east
- 66 (x6 hours) occurrence from the south,
- 66 (x6 hours) occurrence from the south-east
- **68 (x6 hours) occurrence from the south-west**

The southern directions are more prevalent for the node considered because of the seaward projection of the northern part of the island that somewhat buffer the site from remote northern waves. The site however becomes more exposed as soon as the passing hurricane systems are more south and west of the island.

Wind direction- NW

Tp(s)	Wave height(m)										Total
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6											
8											
10											
12											
14											
16											
18											
20											
Total											

Wind direction- N

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6											
8											
10											
12											
14											
16											
18											
20											
Total											

Wind direction- NE

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6											
8											
10											
12											
14											
16											
18											
20											
Total											

Wind direction- W

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		4									4
8		13	2								15
10			2	1							3
12				1							1
14											
16											
18											
20											
Total		17	4	2							23

All directions

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4		1									1
6		21									21
8		111	42	1							154
10			49	12							61
12				3							3
14											
16											
18											
20											
Total		133	91	16							240

Wind direction- E

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		2									
8		17	13								
10			17	2							
12											
14											
16											
18											
20											
Total		19	30	2							

Wind direction- SW

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		7									7
8		39	7								46
10			3								3
12											
14											
16											
18											
20											
Total		46	10								56

Wind direction- S

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4		1									1
6		5									5
8		26	5								31
10			15	3							18
12											
14											
16											
18											
20											
Total		32	20	3							55

Wind direction- SE

Tp(s)	Wave height(m)										
<value	2	4	6	8	10	12	14	16	18	20	
2											
4											
6		3									
8		16	15	1							
10			12	6							
12				2							
14											
16											
18											
20											
Total		19	27	9							

Figure 3.1 Bi-variate table for extremal wave action from hurricanes occurring along the Caribbean Sea side of the Palisadoes

3.2.2.3 Waves heights and directions

The bi-variant table generated indicates that hurricane waves originating from the south east (SE) and south (S) are the most severe of all directions (see Table 3-2). The northern waves are not expected to significantly impact the site due to the angle (orientation) of the shoreline and the shape of the land.

Table 3-2 Summary of wave heights and periods from various directions for different return periods

	Wave height (m)											
	All		SW		W		E		SE		S	
Return Periods	**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	1.5	6.2
2	3.8	9.8	3.4	9.3	3.5	9.4	4.5	10.6	4.4	10.5	3.9	9.9
5	5.1	11.3	3.9	9.9	4.5	10.6	5.5	11.7	5.6	11.7	5.1	11.3
10	6.0	12.2	4.2	10.2	5.1	11.3	6.0	12.2	6.2	12.3	5.8	12.0
20	6.8	13.0	4.4	10.5	5.6	11.8	6.5	12.6	6.7	12.8	6.4	12.5
25	7.1	13.2	4.4	10.5	5.7	11.9	6.6	12.8	6.8	13.0	6.5	12.7
50	7.9	13.9	4.6	10.7	6.2	12.4	6.9	13.1	**7.2**	**13.3**	7.0	13.2
75	8.4	14.3	4.7	10.8	6.4	12.6	7.1	13.3	7.5	13.5	7.3	13.4
100	8.7	14.6	4.7	10.9	6.6	12.7	7.3	13.4	**7.6**	**13.7**	7.5	13.6
150	9.1	14.9	4.8	10.9	6.8	12.9	7.4	13.5	7.8	13.8	7.7	13.8
200	9.4	15.2	4.8	11.0	7.0	13.1	7.6	13.6	7.9	14.0	7.9	13.9

The extremal analysis results indicate that the 100-year return period event has a wave height of 7.6 m for south eastern waves. Overall, these are relatively large waves with potential for causing severe damage along the shoreline. They are however deepwater waves that will be impacted by the bathymetry as they approach the shoreline. Their potential for resulting near shore climates were investigated using a wave refraction and diffraction model as outlined in the following section.

3.2.2.4 Storm Surge and Winds

The maximum storm surge that is estimated for this location for the 100 year event is approximately 1.31 m, see Table 3-3 This is essential information when it pertains to construction within the project area in regards to the placement of the sand dunes.

One factor that was unaccounted for in the model prediction, however, is the effect of wave run-up which will inevitably increase the water levels. This parameter would not have been easily differentiable to the observers and would have thus been a part of what was observed. It is against this background that wave run-up was determined and added to the storm surge elevations.

Table 3-3 Extremal storm surge (metres) predictions for the Palisadoes along the profile from shoreline to deepwater for all directional waves possible for the project area

Return Period	Total setup (m)								
	All	SW	W	NW	N	NE	E	SE	S
1	NaN	0.05	NaN	0.00	0.00	0.00	NaN	0.05	0.05
2	0.42	0.40	0.22	0.00	0.00	0.00	0.27	0.57	0.54
5	0.63	0.57	0.35	0.00	0.00	0.00	0.48	0.82	0.78
10	0.76	0.67	0.44	0.00	0.00	0.00	0.61	0.96	0.92
20	0.86	0.75	0.51	0.00	0.00	0.00	0.74	1.08	1.03
25	0.89	0.77	0.53	0.00	0.00	0.00	0.78	1.12	1.07
50	0.98	0.84	0.60	0.00	0.00	0.00	0.89	**1.22**	1.17
75	1.03	0.88	0.64	0.00	0.00	0.00	0.96	1.28	1.22
100	1.07	0.91	0.66	0.00	0.00	0.00	1.00	**1.31**	1.25
150	1.11	0.94	0.70	0.00	0.00	0.00	1.06	1.37	1.30
200	1.14	0.97	0.72	0.00	0.00	0.00	1.11	1.40	1.34

The Software programme CRESS (Coastal and River Engineering Support System) was utilized to estimate the run-up. This software uses the model for wave run-up on smooth and rock slopes of coastal structures according to (Meer and W.)The estimated wave run-up levels range from 1.27m to 2.57m for the 2 to 100 year hurricanes and were added to the model predicted storm surge results (see Table 3-4).

Table 3-4 Summary of CEAC model predicted storm surge with and without wave run-up for different return periods

Return Period	Predicted storm surge from model without run-up (m)	Predicted storm surge from model with run-up (m)
2	0.57	1.27
5	0.82	1.69
10	0.96	1.94
25	1.12	2.22
50	1.22	2.41
100	1.31	2.57

The CEAC model predictions with run-up are more intense than the reported trends within the immediate area. The CEAC model with run-up was therefore chosen as the benchmark model for use in determining the 10, 25, 50 and 100yr return period storm surge levels for the Palisadoes.

3.2.3 Operational and Swells

Historical wave climate data was obtained from the NOAA weather service database for the period 1999 to 2007 at 3 hour intervals for an offshore node (Easting: 760900.04, Northing: 632921.46). This data was used to generate bi-variant tables for the mean wave heights versus periods as well as the wave height versus direction. The operational wave was then determined as the 50 percent wave occurring at the site whereas the swell waves were estimated by taking the highest 5 percent waves from the bi-variant table.

The analysis determined that operational waves have heights of up to 1.2 m, and periods of 6.5s and direction of 112.5°. The swell waves had a wave height of 2.2 m, a wave period f 8 s and a direction of 202.5°. Please see Figure 3.2 and Figure 3.3 which shows the bi-variant tables generated from the historical data and Table 3-5 and Table 3-6 which shows the incident operational and swell wave data deduced and used in the wave model.

Row Labels	0.2	0.4	0.6	0.8	1	1.2	1.4	1.6	1.8	2	2.2	2.4	2.6	2.8	3	3.2	3.4	3.6	3.8	4	4.2	4.4	4.6	4.8	5.2	5.4	5.8	6	6.2	6.4	6.6	6.8	7		Grand Total
2.5		4	4	1																															9
3	2	28	50	16	5																														101
3.5		1	39	57	9																														106
4	6	30	28	41	57	6	2																												170
4.5	92	122	79	17	47	45	14	3																											419
5	21	219	120	77	35	59	25	8	1																										565
5.5	11	230	527	298	166	81	56	25	4																										1398
6	6	126	562	907	606	277	83	46	15	3			1																						2632
6.5	6	53	278	1020	1516	1129	477	167	48	14	10	3	4																						4725
7	12	29	79	321	919	1629	1167	876	309	96	24	10	6	1	1																				5479
7.5	29	32	23	77	181	527	830	988	722	432	208	42	13	5		1		1	1																4112
8	38	33	10	23	21	81	166	376	434	418	274	95	25	5	3	2					1														2005
8.5	25	24	1	5	3	6	13	58	124	198	249	148	67	33	12	1	2			1				1											971
9	20	8	10	1		2	3	13	20	26	43	17	17	27	11	9	2	2	1	1		1					1								235
9.5	6			1				1	2	2	2	22	16	10	12	3	1				1	1										1			81
10	6	11			1		1	2	6	4	1	1	2	1	1						1				1						1			8	48
10.5	5	7					2					1	1													1									17
11	2																												1						3
11.5																																	1		1
12																														1					1
12.5									1		1			1				1					1					1			2				8
13								1										1		1															3
14						1	1																												2
15			1	1																															2
	5	2	4																																11
Grand Total	**292**	**959**	**1815**	**2863**	**3566**	**3843**	**2840**	**2564**	**1686**	**1193**	**812**	**339**	**152**	**83**	**40**	**16**	**5**	**5**	**2**	**3**	**3**	**2**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**3**	**1**	**1**	**8**	**23104**

Figure 3.2 Bi-variant table generated from historical data provided by NOAA for an offshore node. The table presents the wave heights and the corresponding wave periods, and allowed us to deduce the characteristics for the operational and swell occurring at the Palisadoes.

Table 3-5 The wave heights and periods for the operational and swell waves determined from the bivariant table presented in Figure 3-3

	Operational	Swell
Wave Height (m)	1.2	2.2
Wave Period (seconds)	6.5	8

Count of Wave hieght Round	Column Labels																													
Row Labels	**0.2**	**0.4**	**0.6**	**0.8**	**1**	**1.2**	**1.4**	**1.6**	**1.8**	**2**	**2.2**	**2.4**	**2.6**	**2.8**	**3**	**3.2**	**3.4**	**3.6**	**3.8**	**4**	**4.2**	**4.4**	**4.8**	**5.2**	**5.4**	**5.8**	**6.2**	**6.6**	**6.8**	**7**	**Grand Total**
101.25	18	45	117	213	266	356	321	313	252	211	128	52	20	6	8	2				2	1	2								1	2334
112.5	18	55	99	125	155	171	153	142	87	90	73	19	8	1	2	4					1						1				1204
123.75	6	30	61	85	73	77	41	50	33	22	32	14	11	2	1		1	2						1							542
135	8	25	36	48	38	50	32	20	15	9	14	12	6	1		1	1								1						317
146.25	6	16	16	33	30	32	22	13	15	8	6	2	3		1			1										1	1		206
157.5	7	22	23	10	22	27	18	8	4		3	3		2		1			1				1			1					153
168.75	3	13	14	10	5	13	11	2	3	2	1	4	2	2																	85
180	6	10	7	9	4	7	1	2	2			1	1								1										51
191.25	5	9	13	10	5	8	1	3	1				1																		56
202.5	1	12	11	10	4	2	3	2	1			1																			47
213.75	1	11	6	7	3	4	1																								33
225	3	7	7	9	3	4	1				1																				35
236.25	7	12	6	1	1	5		1			1																				34
247.5	2	10	6	5	1	1	1																								26
258.75	2	12	8	6	2	1		2																							33
	8	6	18	15	13	10	3		2	3																					78
Grand Total	**101**	**295**	**448**	**596**	**625**	**768**	**609**	**558**	**415**	**345**	**259**	**108**	**52**	**14**	**12**	**8**	**2**	**3**	**1**	**2**	**3**	**2**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**5234**

Figure 3.3 Bi-variant table generated from historical data provided by NOAA for an offshore node. The table presents the wave heights and the corresponding wave directions, and allowed us to deduce the characteristics for the operational and swell occurring at the Palisadoes.

Table 3-6 The wave height and corresponding wave direction for the operational and swell waves determined from the bivariant table presented in Figure 3-4

	Operational	Swell
Wave Height (m)	1.2	2.2
Direction (degrees)	112.5	202.5

3.2.4 Storm Surge

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
- Global Sea Level Rise (over a 37 year project life - 2050)
- Inverse Barometric Pressure Rise

For the Caribbean Sea side of the Palisadoes the south-eastern and western profiles were focused on in this analysis as they were the most extreme. The results indicate that the expected 100 Year storm surge is 1.31 meters, see Table 3-7.

Table 3-7 Extremal storm surge predictions for the wave height an wave period along the profile

Return Periods	Wave height (m)								
	All	SW	W	NW	N	NE	E	SE	S
1	2.5	1.5	1.5	0.0	0.0	0.0	1.5	1.5	1.5
2	3.8	3.4	3.5	0.0	0.0	0.0	4.5	4.4	3.9
5	5.1	3.9	4.5	0.0	0.0	0.0	5.5	5.6	5.1
10	6.0	4.2	5.1	0.0	0.0	0.0	6.0	6.2	5.8
20	6.8	4.4	5.6	0.0	0.0	0.0	6.5	6.7	6.4
25	7.1	4.4	5.7	0.0	0.0	0.0	6.6	6.8	6.5
50	7.9	4.6	6.2	0.0	0.0	0.0	6.9	**7.2**	7.0
75	8.4	4.7	6.4	0.0	0.0	0.0	7.1	7.5	7.3
100	8.7	4.7	6.6	0.0	0.0	0.0	7.3	**7.6**	7.5
150	9.1	4.8	6.8	0.0	0.0	0.0	7.4	7.8	7.7
200	9.4	4.8	7.0	0.0	0.0	0.0	7.6	7.9	7.9
Return Periods	Wave Period (s)								
	All	SW	W	NW	N	NE	E	SE	S
1	8.0	6.2	6.2	0.0	0.0	0.0	6.2	6.2	6.2
2	9.8	9.3	9.4	0.0	0.0	0.0	10.6	10.5	9.9
5	11.3	9.9	10.6	0.0	0.0	0.0	11.7	11.7	11.3
10	12.2	10.2	11.3	0.0	0.0	0.0	12.2	12.3	12.0
20	13.0	10.5	11.8	0.0	0.0	0.0	12.6	12.8	12.5
25	13.2	10.5	11.9	0.0	0.0	0.0	12.8	13.0	12.7
50	13.9	10.7	12.4	0.0	0.0	0.0	13.1	**13.3**	13.2
75	14.3	10.8	12.6	0.0	0.0	0.0	13.3	13.5	13.4
100	14.6	10.9	12.7	0.0	0.0	0.0	13.4	**13.7**	13.6
150	14.9	10.9	12.9	0.0	0.0	0.0	13.5	13.8	13.8
200	15.2	11.0	13.1	0.0	0.0	0.0	13.6	14.0	13.9

Table 3-8 Extremal storm surge predictions for the wind speed and total setup along the profile

Return Period	Wind speeds (m/s)								
	All	SW	W	NW	N	NE	E	SE	S
1	15.0	NaN	NaN	0.0	0.0	0.0	NaN	NaN	NaN
2	34.5	NaN	NaN	0.0	0.0	0.0	20.1	19.6	17.3
5	46.5	18.6	24.5	0.0	0.0	0.0	24.8	25.4	23.3
10	53.7	21.4	32.5	0.0	0.0	0.0	27.4	29.2	27.4
20	59.9	24.1	40.4	0.0	0.0	0.0	29.6	32.7	31.3
25	61.7	25.0	42.9	0.0	0.0	0.0	30.3	33.7	32.5
50	67.2	27.8	**50.7**	0.0	0.0	0.0	32.2	37.0	36.2
75	70.2	29.4	55.2	0.0	0.0	0.0	33.3	38.8	38.4
100	72.3	30.6	**58.3**	0.0	0.0	0.0	34.0	40.0	39.9
150	75.1	32.2	62.8	0.0	0.0	0.0	35.0	41.8	41.9
200	77.0	33.4	66.0	0.0	0.0	0.0	35.7	43.0	43.4

Return Period	Total setup (m)								
	All	SW	W	NW	N	NE	E	SE	S
1	NaN	0.05	NaN	0.00	0.00	0.00	NaN	0.05	0.05
2	0.42	0.40	0.22	0.00	0.00	0.00	0.27	0.57	0.54
5	0.63	0.57	0.35	0.00	0.00	0.00	0.48	0.82	0.78
10	0.76	0.67	0.44	0.00	0.00	0.00	0.61	0.96	0.92
20	0.86	0.75	0.51	0.00	0.00	0.00	0.74	1.08	1.03
25	0.89	0.77	0.53	0.00	0.00	0.00	0.78	1.12	1.07
50	0.98	0.84	0.60	0.00	0.00	0.00	0.89	**1.22**	1.17
75	1.03	0.88	0.64	0.00	0.00	0.00	0.96	1.28	1.22
100	1.07	0.91	0.66	0.00	0.00	0.00	1.00	**1.31**	1.25
150	1.11	0.94	0.70	0.00	0.00	0.00	1.06	1.37	1.30
200	1.14	0.97	0.72	0.00	0.00	0.00	1.11	1.40	1.34

Along the harbor side of the project a two-dimensional JONSWAP wind-wave model was used to establish the storm surge over a seven year period (2000 – 2006) for a point just off the Harbour. The model determines wave height and period from fetch, storm duration and depth of water in the generating area. Where fetch is the distance into the wind direction from a point of interest to the nearest shoreline[6]. The points chosen in this model provided the greatest fetch for each wind direction, see Table 3-9.

For our project the waves generated in deep water are fetch limited where:

$H_{mo} = 0.0016\ (F^*)^{1/2}$

$T^*_p = 0.286\ (F^*)^{1/3}$

And H_{mo} = wave height

T^*_p = wave period

[6] Kamphuis, J (2002), Introduction to Coastal Engineering and Management, *Advanced Series on Ocean Engineering – Volume 16*

F* = fetch

Table 3-9 Fetch corresponding to wind angle for the Harbour

Angle	Fetch (m)
0	2600
30	2700
60	3500
90	3800
120	3100
150	2300
180	200
210	900
240	1400
260	5600
270	14800
280	14500
300	5000
330	3000

The largest fetch corresponds to a wind angle of 270° and the wave height and period calculations were determined based on this value and presented in Table 3-10.

Table 3-10 Results from the JONSWAP method of determining wave height and period based on fetch limited conditions

Wind direction (Degrees)	Wind Speed (m/s)	Fetch (km)	Duration (hr)	Depth (m)	F*	t*	Feff*	Hmo*	Tp*	Hmo (m)	Tp (s)	Setup (m)	RP/yr
270	50.7	14.80	1	10	57	697	32	0.01	0.91	2.38	4.70	0.79	**50**
270	58.3	14.80	1	10	43	606	26	0.01	0.85	2.83	5.04	1.04	**100**

3.3 Near shore Wave Climate Analysis (Hurricane, Operational and Swells)

3.3.1 Objectives and Approach

Deepwater water wave data by itself offers limited information on how waves reach the shoreline. It was therefore necessary to determine the nearshore bathymetry and wave climate in order to identify areas of the study area that might be vulnerable to shoreline erosion or direct wave attack and to estimate the impact on the proposed structures.

The approach adopted in order to achieve these objectives was as follows:

- Use the deepwater wave data as input for the analysis.
- Determine the operational, swell and hurricane environments along the Harbour side and Caribbean Sea Side shoreline for pre and post project.
- Determine the impact of climate change along the Harbour side and Caribbean Sea Side Shoreline during operational, swell and hurricane event.
- Prepare a bathymetric database of the project domain for extremal analysis.

- Conduct spatial wave transformation analysis within the study area.

3.3.2 Wave Climate Model: STWAVES

The model considers time-independent advection, refraction shoaling, and wave growth as a function of winds. It is a half-plane model in the sense that it only includes spectral energy directed into the computational grid at the seaward boundary. This version does not include diffraction due to surface-piercing structures or islands. Computationally, the model uses a thin-film approach for land and very shallow regions and solves the model equations at all grid points within the domain. As input, the model requires some basic configuration data, a uniform rectilinear grid, and directional spectra given at the seaward boundary. Due to the nature of the integral solutions for some of the terms, this version of the model requires square (dx = dy) grid spacing. STWAVE is a solution of the steady-state spectral balance equation for wave transformation, and it was written by Dr. Donald T. Resio. It is a finite difference model which considers the propagation, growth and dissipation of spectral energy on a 2-dimensional uniform rectilinear grid.

3.3.3 Modeling Approach and Summary Incident Wave Conditions

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. These incident wave heights and periods were then used in the STWAVES 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. See Table 3-11 and Table 3-12 for a summary of the incident wave conditions used for the analysis. Based on deepwater wave climate and storm surge analysis along with the shape of the shoreline and geographical location of the study area.

Table 3-11 Summary of operational and swell wave heights and periods used to model STWAVES

HARBOUR SIDE				CARIBBEAN SEA SIDE			
OPERATIONAL		SWELL		OPERATIONAL		SWELL	
Hs (m)	Ts (s)	Hs (m)	Ts (s)	Hs (m)	Ts (s)	Hs (m)	Ts (s)
0.2	1.5	0.6	2.5	1.2	6.5	2.2	8

Table 3-12 Summary of hurricane wave heights and periods used to model STWAVES

HARBOUR SIDE				CARIBBEAN SEA SIDE			
50 YEAR		100 YEAR		50 YEAR		100 YEAR	
Hs (m)	Ts (s)	Hs (m)	Ts (s)	Hs (m)	Ts (s)	Hs (m)	Ts (s)
2.38	4.70	2.83	5.04	7.2	13.3	7.6	13.7

3.3.4 Caribbean Sea Side

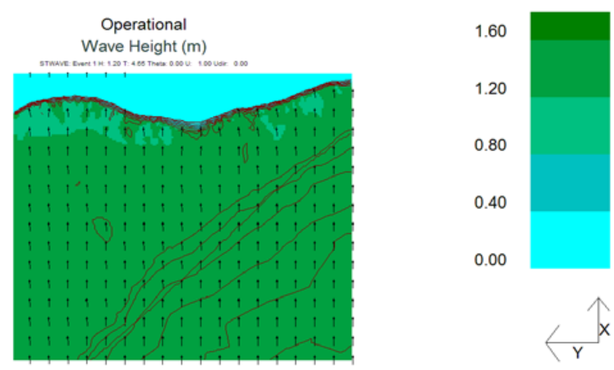
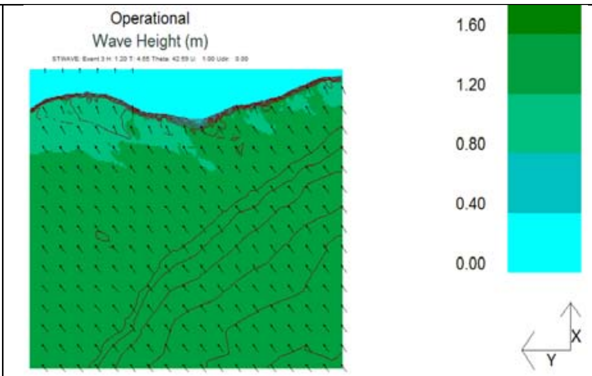
3.3.4.1 Pre-Project Scenario – Caribbean Sea Side

The model was calibrated to run operational, swell and hurricane waves for the E, SE, S, SW and W directions. The existing shoreline was modeled first to better understand the areas which are most vulnerable as well as to estimate the magnitude of wave heights reaching the shoreline based on the wave predictions. The model showed that the S and SE directions had the greatest impact on the shoreline during operational, swell and hurricane conditions.

3.3.4.1.1 Operational Waves – Caribbean Sea Side

The model showed that the shoreline under operational conditions may experience wave heights ranging from 0.7 to 1.2 m from the S and SE directions. Table 3-13 which shows the waves generated during operational conditions.

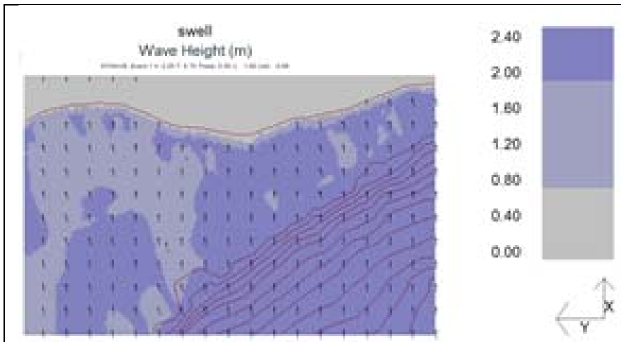
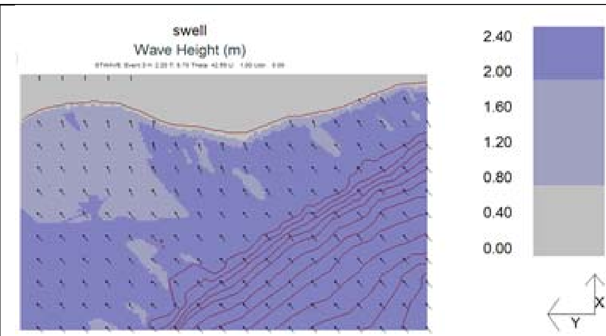
Table 3-13 STWAVES Caribbean Sea side resultant plots of operational waves for the S and SE directions

 South (S)	 South East (SE)

3.3.4.1.2 Swell Waves – Caribbean Sea Side

It was also important to look at the swell wave climate to understand the impact on the existing shoreline and to design shoreline protective structures which can withstand these scenarios. The model showed that the shoreline under swell wave conditions may experience wave heights ranging from 0.8 to 2.0 m from the south and southeast direction. Table 3-14 shows the waves generated due to swells. It is evident that the eastern and central portions of Palisadoes experience more significant wave heights (0.8 to 1.6 meters) than the western sections (0.4 and 0.8 meters). This speaks to the increased vulnerability of the dune at Harbour View side versus NMIA end.

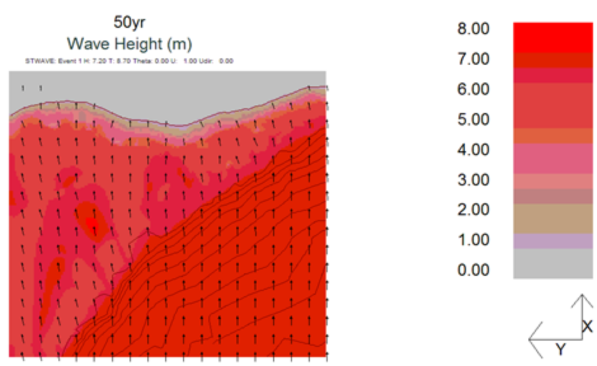
Table 3-14 STWAVES Caribbean Sea side resultant plots of swell waves for the S and SE directions

 South (S)	 South East (SE)

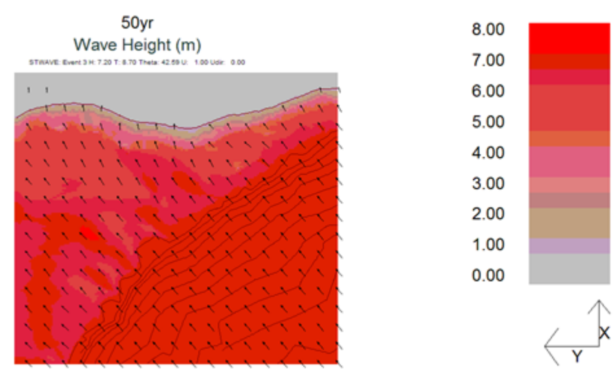
3.3.4.1.3 Hurricane Waves – Caribbean Sea Side

It is also important that hurricane winds generated waves are modeled and investigated. During a storm event there will be wave setup, and so a water set up elevation of 1.22 and 1.31 m were added to the simulation for the 50 and 100 year return period respectively. These elevations were obtained from the storm surge model discussed in an earlier section of the report. The wave plots generated from the model showed that during hurricane conditions wave heights of 2.0m and 3.0 m reach the shoreline for the 50 and 100 year return period respectively. Table 3-15 shows the waves generated due to hurricane waves.

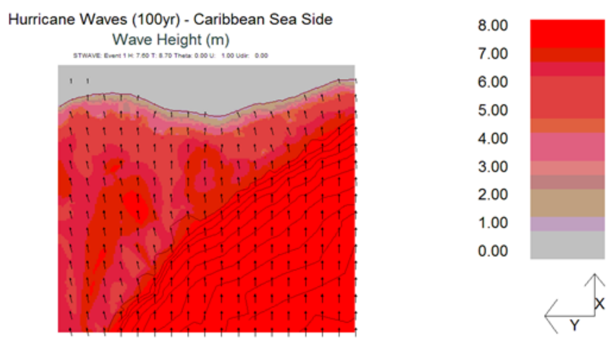
Table 3-15 STWAVES Caribbean Sea side resultant plots for hurricane waves from the S and SE direction



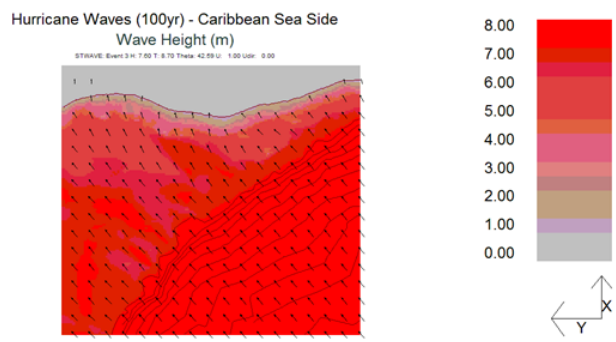
50 year- South (S)



50 year - South East (SE)



100 year- South (S)



100 year - South East (SE)

3.3.4.2 Post Project

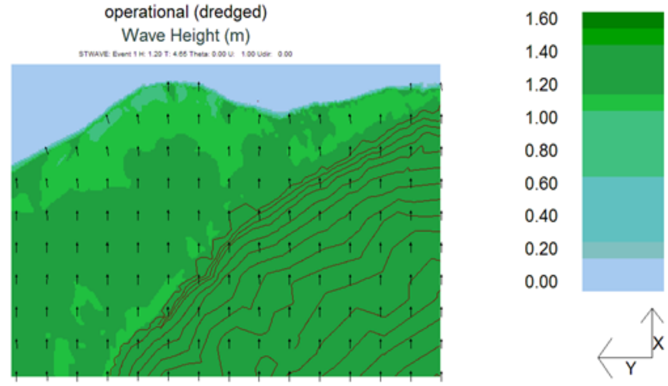
Implementation of the project involves offshore dredging which will alter the bathymetry. The two locations to be dredged are approximately 0.6 km and 1.6 km offshore, and they will be dredged to a depth of 1.5m.

3.3.4.2.1 Operational Waves – Caribbean Sea Side

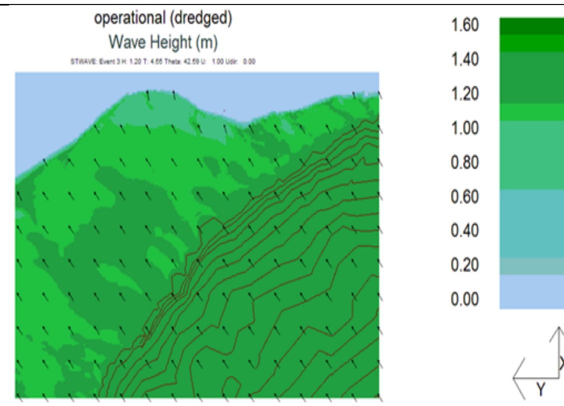
Although the dredging exercise altered the bathymetry it did not affect the magnitude of waves reaching the shoreline under operational conditions. Wave heights of 0.7 to 1.2m were observed to reach the shoreline from the S and SE directions. See Table 3-16. These are similar to the wave heights in the pre-project scenario and consistent with the physical understanding of wave breaking and refraction where the refraction coefficient for the small change in the sea floor from 15 meters to 16.5 meters is

0.95 that is relatively small or equivalent to no change, (refraction coefficient = √(16.5 /15) = 0.95 ~ 1). The burrow areas are in 12 to 18 meters of wave and the incident operational waves have wave lengths of

Table 3-16 STWAVES resultant plots of operational waves for various directions (post project)



South (S)

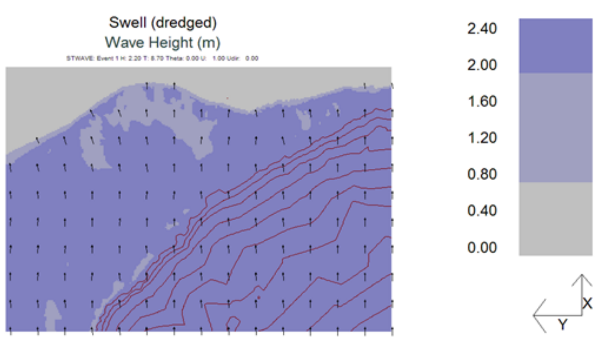


South East (SE)

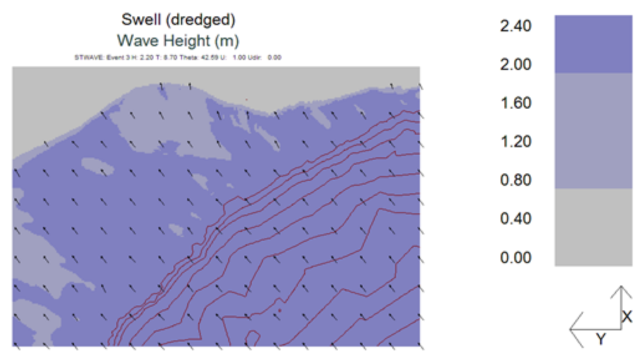
3.3.4.2.2 Swell Waves – Caribbean Sea Side

The pre-project scenario had wave heights of 0.8m to 2.0m reaching the shoreline from the south and south easterly direction during the swell event. The post project scenario saw no change in the resulting wave heights reaching the shoreline see Table 3-17.

Table 3-17 STWAVES Caribbean Sea side resultant plots of swell waves for the S and SE directions



South (S)

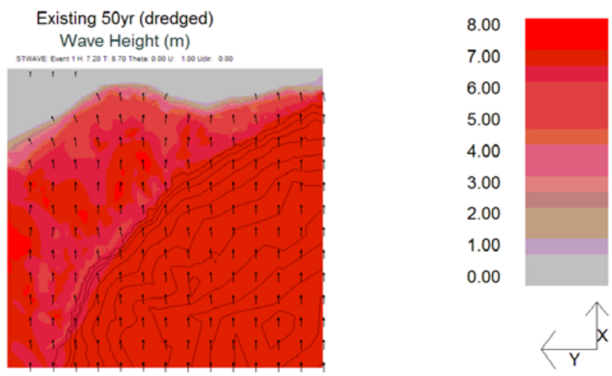


South East (SE)

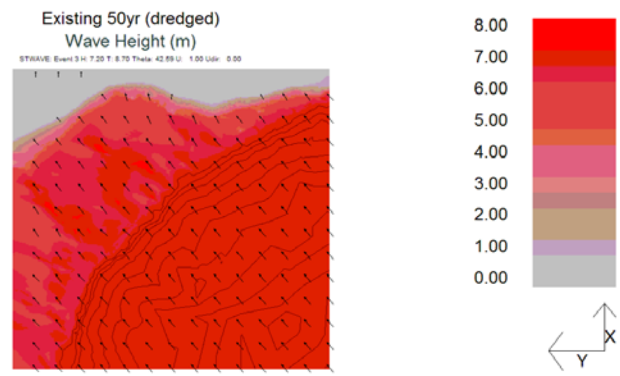
3.3.4.2.3 Hurricane Waves – Caribbean Sea Side

There was also no noticeable change in the wave heights reaching the shoreline under hurricane conditions. The SE and S directions had wave heights of 2 m and 3m for the 50 and 100 year return periods respectively See Table 3-18.

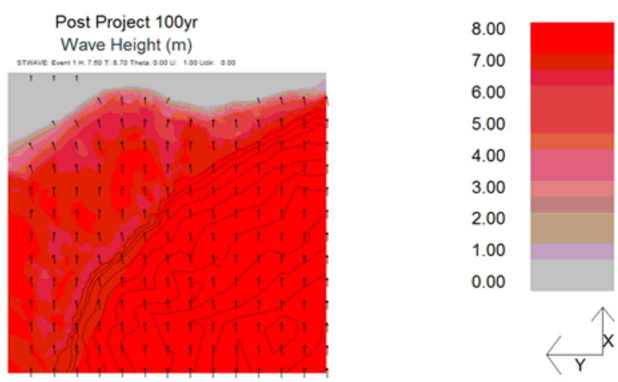
Table 3-18 STWAVES Caribbean Sea Side resultant plots of hurricane waves for various directions (post project)



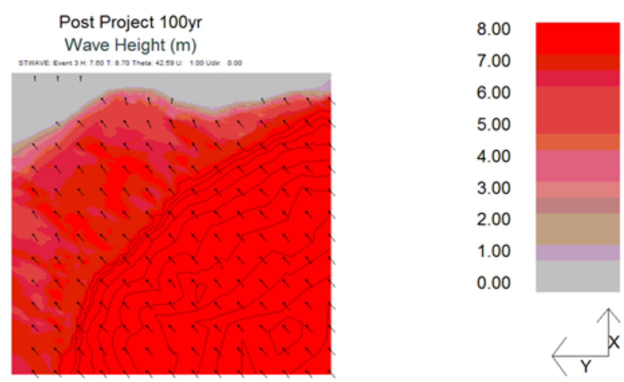
50 year- South (S)



50 year - South East (SE)



100 year- South (S)



100 year - South East (SE)

3.3.4.3 Future Climate

It was important to consider the effect of climate change inclusive of global sea level rise on the study area and the design life of the sand dunes, (Climate Studies Group) determined that sea level rise would result in a water level rising to 0.14m by the year 2050.

When the model was run with considerations made for future climate change there was no change in the magnitude of waves reaching the shoreline for the operation and swell wave condition. However the wave setup increased from 0.74m and 0.98m to 0.88m and 1.12 m for the 50 and 100 year return period respectively under the hurricane wave conditions. Table 3-19 summarizes the incident wave climate used to model the scenario under climate change conditions.

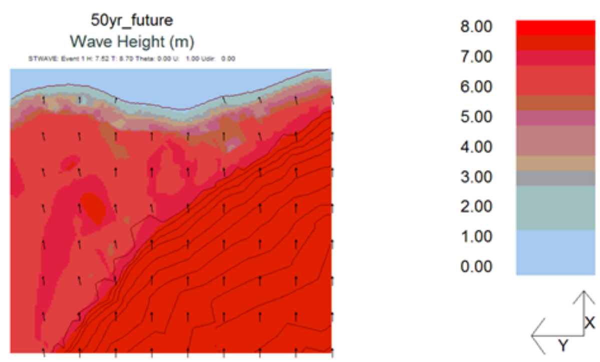
Table 3-19 Summary of hurricane wave heights and periods used to model STWAVES with the consideration of future climate change

RETURN PERIOD (50 YR)		RETURN PERIOD (100 YR)	
Ts (s)	Hs(m)	Ts(s)	Hs(m)
13.9	7.52	14.2	7.91

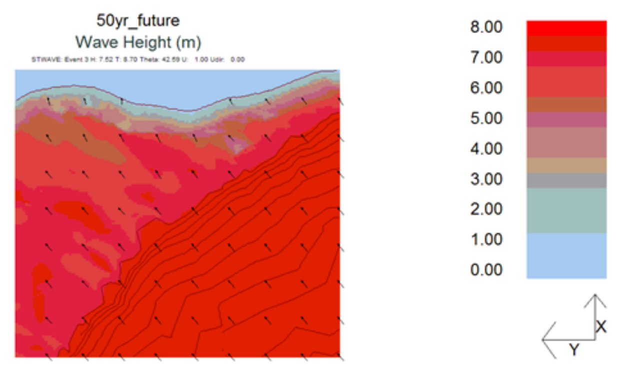
3.3.4.3.1 Future Climate – Pre Project Caribbean Sea Side

The pre project scenario for the S and SE directions were modeled with climate change considerations made. The model showed that the shoreline may experience wave heights ranging from 2 to 3.5 m and 2.5 – 4 m for the 50 and 100 year event respectively. Table 3-20 which shows the waves generated due to hurricane events.

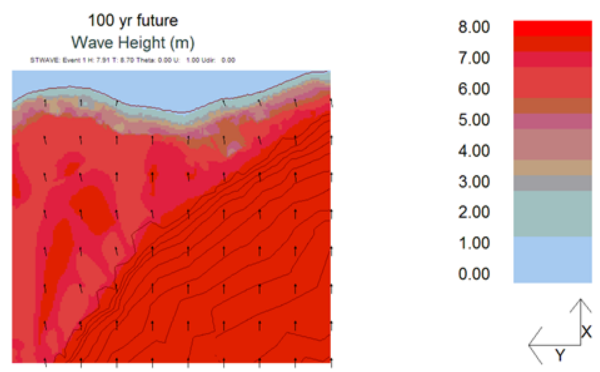
Table 3-20 STWAVES Caribbean Sea side resultant plots for future climate hurricane waves for the S and SE directions



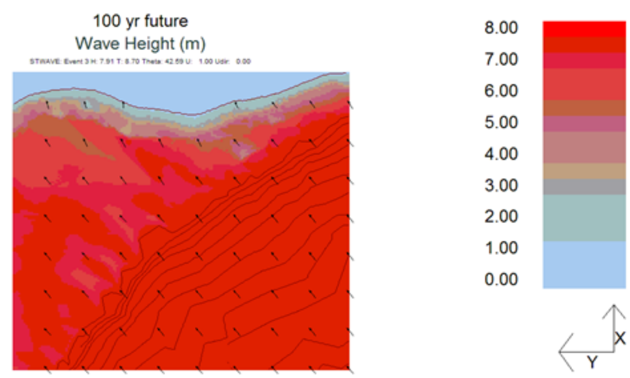
50 year- South (S)



50 year - South East (SE)



100 year- South (S)

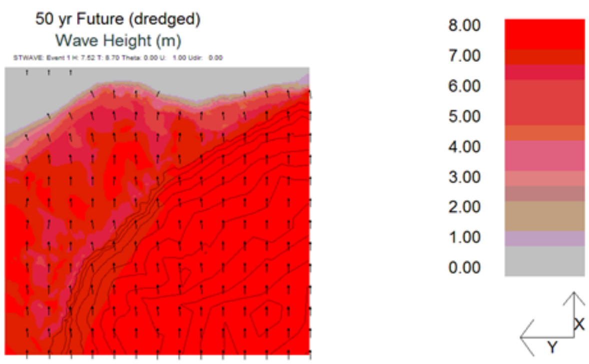
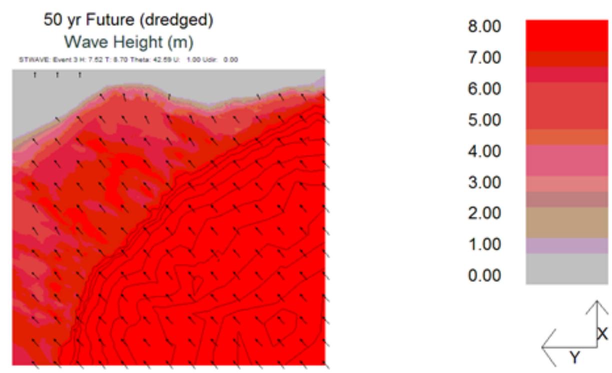
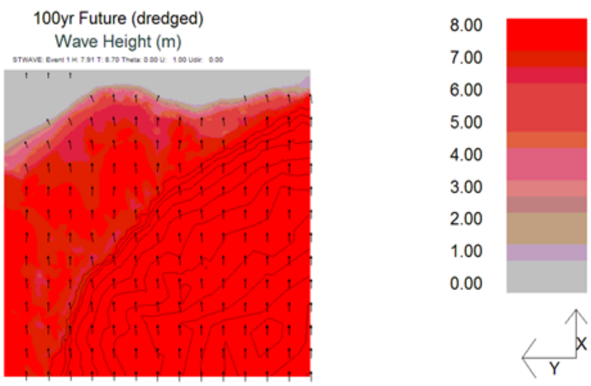
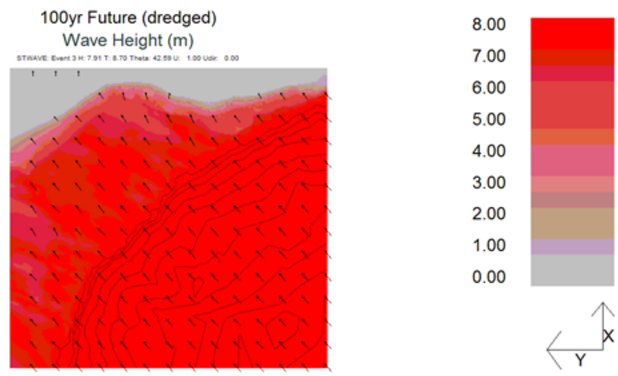


100 year - South east (SE)

3.3.4.3.2 Future Climate – Post Project Caribbean Sea Side

Similar to the post project condition without climate change, the offshore dredging with climate change considerations under the future climate did not result in any significant increase in the magnitude of waves reaching the shoreline. The 50 and 100 year event which had a range of 2 to 3.5 m and 2.5 to 4.0 m, see Table 2-19.

Table 3-21 STWAVES Caribbean Sea side resultant plots for future climate hurricane waves for various directions (post project)

 50 year- South (S)	 50 year - South East (SE)
 100 year- South (S)	 100 year - South East (SE)

3.3.4.4 Discussion

The wave refraction analysis clearly indicates the vulnerability of the shoreline from waves approaching from the south and south east directions, particularly along the eastern section of the Palisadoes. In all scenarios, 7 to 8 m waves are expected some 2.5 km offshore and 2 to 4 m waves are expected at the shoreline during storm events. The model predicts that the post project scenario of the burrow area is not expected to have an impact on the waves reaching the shoreline both with and without the climate change considerations made.

3.3.5 Harbour Side

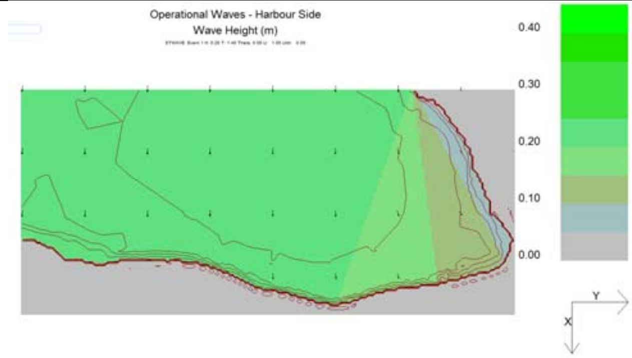
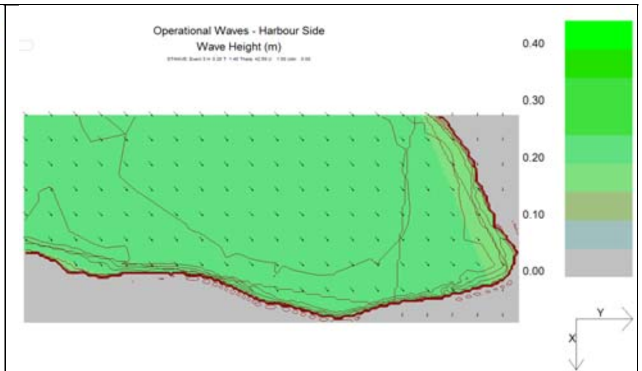
3.3.5.1 Existing Scenario – Harbour Side

The model was calibrated to run operational, swell and hurricane waves from W, NW, N, NE and E directions. The existing shoreline was modeled first to better understand the areas which are most vulnerable as well as to estimate the magnitude of wave heights reaching the shoreline based on the wave predictions. The model showed that the N and NW directions had the greatest impact on the shoreline during operational, Swell and hurricane conditions. See Table 3-22.

3.3.5.2 Operational Waves – Harbour Side

The model showed that the shoreline under operational conditions may experience wave heights ranging from 0.1 to 0.2 m from the N and NW directions. The model predicts the largest waves (0.2m) to impact the shoreline occurring from the NW direction. Table 3-22 which shows the waves generated during operational conditions.

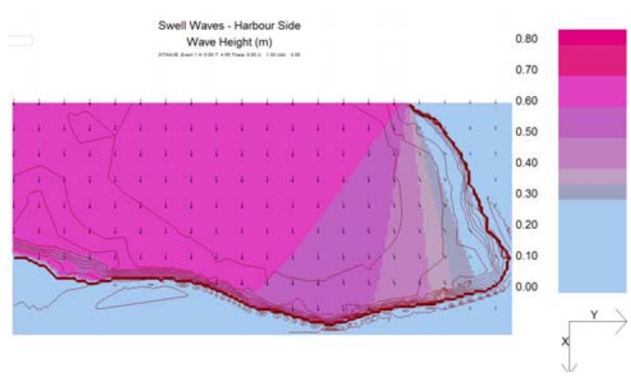
Table 3-22 STWAVES Harbour side resultant plots of operational waves for various directions

 N	 NW

3.3.5.3 Swell Waves – Harbour Side

It was also important to look at the swell wave climate so as to understand the impact on the existing shoreline and to design shoreline protective structures which can withstand these scenarios. The model showed that the shoreline under operational conditions may experience wave heights ranging from 0.2 to 0.6 m from the N and NW directions. The model predicts the largest waves (0.6m) to impact the shoreline occurring from the NW direction. Table 3-23 which shows the waves generated during operational conditions.

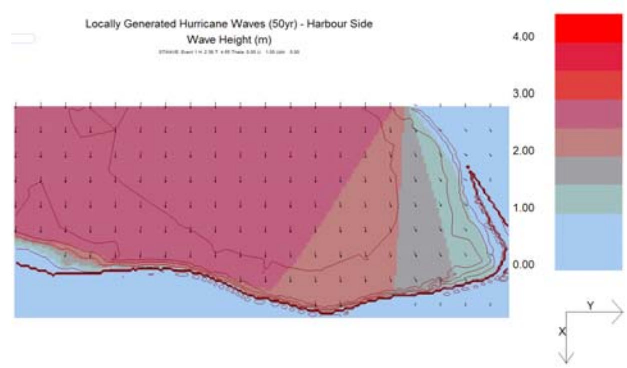
Table 3-23 STWAVES Harbour side resultant plots of swell waves for various directions

 N	 NW

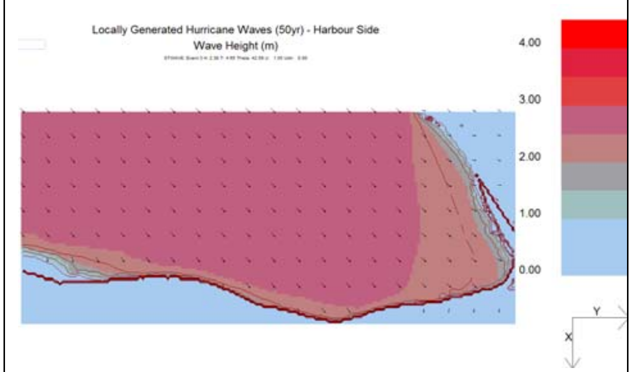
3.3.5.4 Hurricane Waves – Harbour Side

It is also important that hurricane winds generated waves are modeled as well, these can cause the most damage to the beach. During a storm event there will be wave setup, hence a water set up elevation of 0.74 and 0.98 m were added to the simulation for the 50 and 100 year return period respectively based on the storm surge model results. The model showed that the shoreline under hurricane conditions may experience wave heights ranging from 1 to 2 m from the N and NW directions for the 1 in 50 year event. The 1 in 100 year event showed wave heights ranging from 2 to 2.5 meters reaching the shoreline from the N and NW directions. The model predicts the largest waves impacting the shoreline occurring from the NW direction for both return periods. See Table 3-24 which shows the waves generated during operational conditions.

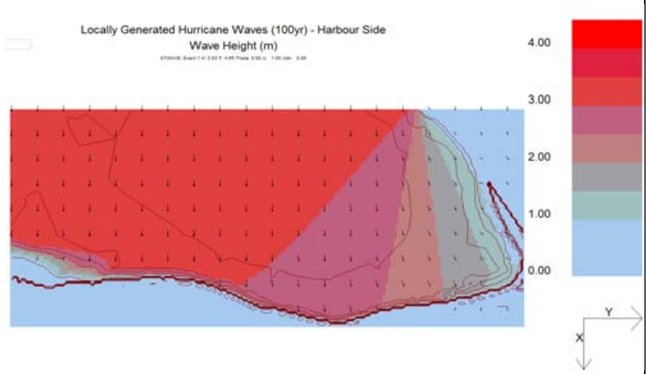
Table 3-24 STWAVES Harbour side resultant plots of hurricane waves for various directions



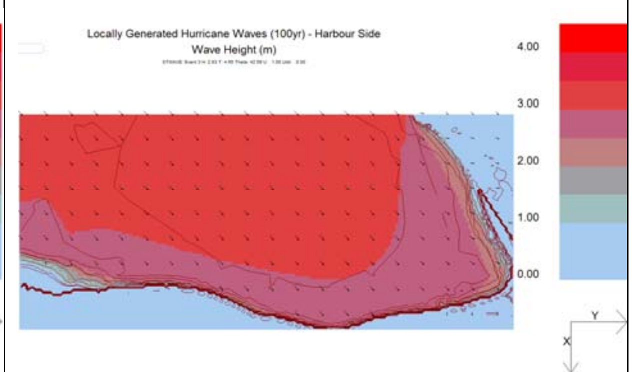
N-50 yr.



NW – 50 yr.



N - 100 yr.



NW – 100 yr.

3.3.5.5 Future Climate – Harbour Side

It was important to consider the effect of climate change inclusive of global sea level rise on the study area and the design life of the sand placed for mangrove nourishment. When the model was run with considerations made for future climate change there was no change in the magnitude of waves reaching the shoreline for the operation and swell wave conditions. However the wave setup increased from 0.74m and 0.98m to 0.88m and 1.12 m for the 50 and 100 year return period respectively under the hurricane wave conditions.

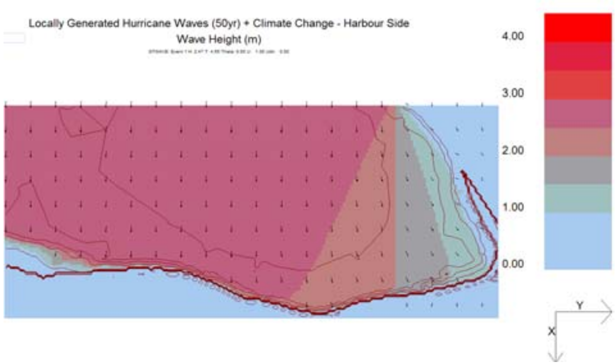
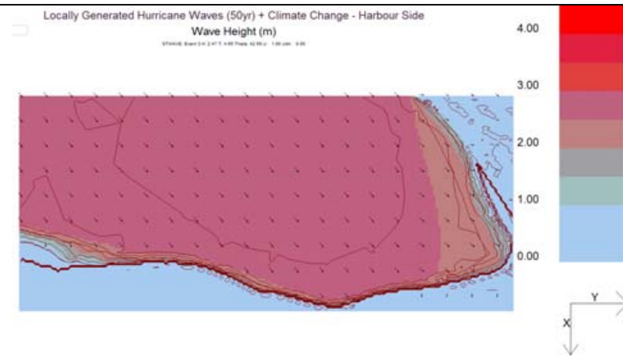
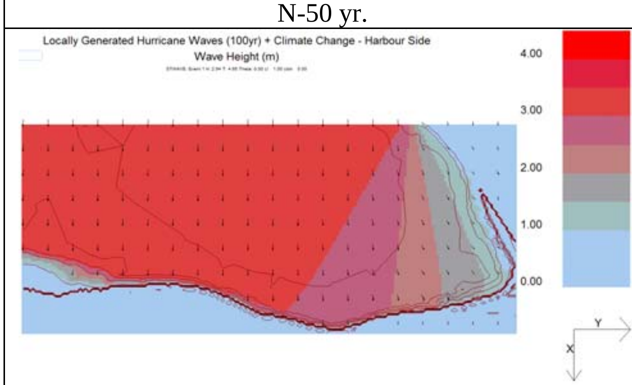
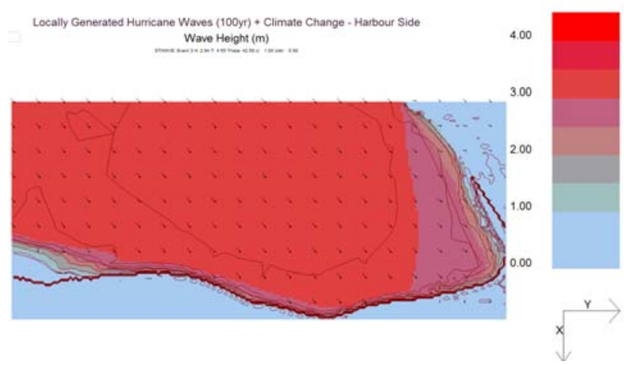
Table 3-25 summarizes the incident wave climate used to model the scenario under climate change conditions.

Table 3-25 Summary of hurricane wave heights and periods used to model STWAVES with climate change considerations made

RETURN PERIOD (50 YR)		RETURN PERIOD (100 YR)	
Ts (s)	Hs(m)	Ts(s)	Hs(m)
4.89	2.47	5.24	2.94

The model results indicated that under hurricane conditions the shoreline may experience wave heights ranging from 1 to 2.5 m and 1.5 to 3 m, for the 50 and 100 year event respectively. See Table 3-26.

Table 3-26 STWAVES Harbour side resultant plots of hurricane waves and climate change for the N and NW directions

 N-50 yr.	 NW – 50 yr.
 N - 100 yr.	 NW – 100 yr.

3.3.5.6 Discussion

The wave refraction analysis clearly indicates the vulnerability of the shoreline from waves approaching from the N and NW directions. Under existing and future climate change scenarios 2.5 to 3 m waves are expected approximately 1 km offshore and under storm conditions 1.5 to 3 m waves are expected at the shoreline. The central to Western end of the shoreline is more vulnerable to wave attacks as the model predicts larger wave heights reaching these sections of the shoreline from the directions modeled.

4 Shoreline Vulnerability

4.1 Long term shoreline change

The shoreline positions along the Palisadoes shore were plotted from 1977 to 2012 and compared in order to determine the long-term spatial and temporal erosion trends across the shore. This was important in order to identify the actual erosion hotspots that might require stabilization and in order to verify wave transformation modeling.

4.1.1 Methodology

The overall long-term erosion trend was estimated by observing:

1) Actual long-term shoreline positions from dated aerial photography and Google Earth imagery – Historical Shoreline Analysis;
2) The global sea level rise component to determine the erosion that was due to chronic global trends versus event based erosion events (i.e. hurricanes and swell events) – Bruun Model.

4.1.2 Rate of Change assessment

4.1.2.1 Historical Shoreline Analysis

Figure 4.2 shows the available satellite imagery (December 2012) over which the observed shorelines from Google Earth and aerial imagery for the years 1977,1991,2002, 2006, 2009 and 2012.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

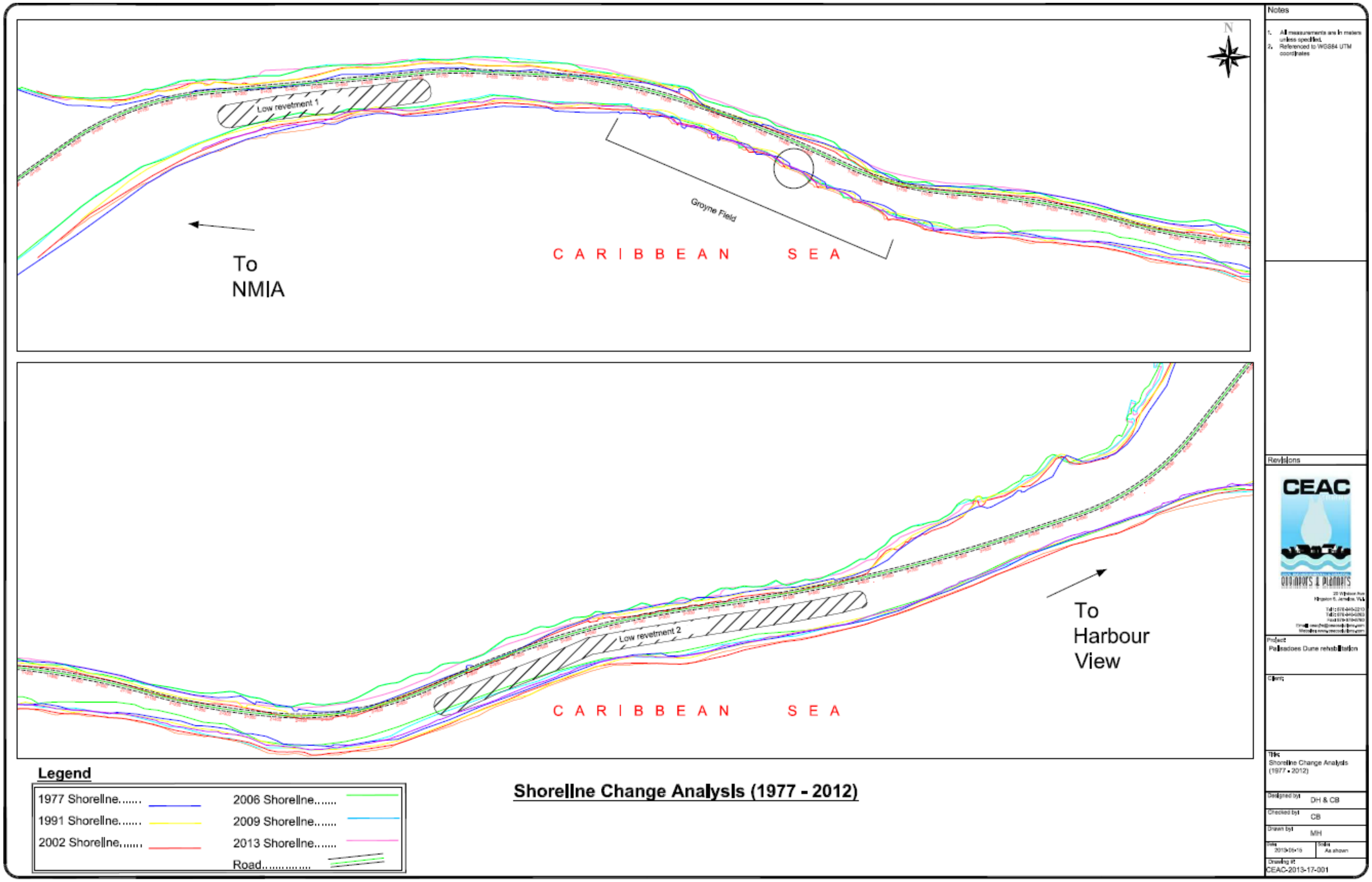


Figure 4.1 Shoreline plots between 1977 and 2013 about the 1968 section for the western section of the Palisadoes closest to the NMIA roundabout (top) and eastern section of the Palisadoes closest to the Harbour View roundabout (bottom)

4.1.2.2 Results

The shoreline analysis was done for the 5km coastline taking note of certain areas of interested namely behind the buried revetments and in the groyne field. A summary of the analysis data is shown in Table 4-1. Figure 4.2 shows a plot of the shoreline movement over the period, it indicates that there has been a general trend of accretion along the Palisadoes shoreline.

Table 4-1 Summary of the displacements of the shoreline for 1991, 2002, 2006, 2009 and 2012 about the 1977 shoreline at 200m intervals

		1991		2002		2006		2009		2012		Overall	
	Location	Process	Accretion/ Erosion Rate (m/year)	Process	Accretion/ Erosion Rate (m/year)	Process	Accretion/ Erosion Rate (m/year)	Process	Accretion/ Erosion Rate (m/year)	Process	Accretion/ Erosion Rate (m/year)	Process	Rate (m/year)
	0+000	erosion	-1.689	accretion	0.278	erosion	-6.488	erosion	-0.153	accretion	10.870	erosion	-0.411
Buried Revetment 1	0+200	erosion	-1.746	accretion	1.320	erosion	-3.848	erosion	-1.113	accretion	9.037	erosion	-0.044
	0+400	erosion	-1.169	accretion	0.535	erosion	-3.940	accretion	0.097	accretion	10.730	erosion	-0.196
	0+600	erosion	-0.795	accretion	0.664	erosion	-1.523	accretion	0.177	accretion	2.690	erosion	-0.251
High Revetment 1	0+800	erosion	-0.918	accretion	0.586	erosion	-3.762	erosion	-0.983	accretion	5.430	erosion	-0.384
	1+000	erosion	-1.170	accretion	0.651	erosion	-2.650	accretion	0.297	accretion	3.257	erosion	-0.497
	1+200	erosion	-0.665	accretion	0.613	erosion	-1.150	erosion	-1.367	accretion	4.180	erosion	-0.018
	1+400	accretion	0.069	erosion	-0.095	erosion	-0.038	erosion	-0.140	accretion	1.630	erosion	-0.063
	1+600	erosion	-0.558	accretion	1.155	erosion	-2.190	accretion	0.380	accretion	2.473	accretion	0.020
	1+800	erosion	-0.281	accretion	1.106	erosion	-5.285	accretion	3.753	accretion	3.213	accretion	0.000
	2+000	accretion	0.246	accretion	0.813	erosion	-7.852	accretion	7.903	accretion	2.810	accretion	0.088
	2+200	accretion	0.604	accretion	0.965	erosion	-5.838	accretion	7.700	accretion	0.883	accretion	0.233
	2+400	accretion	0.648	accretion	0.653	erosion	-4.993	accretion	3.480	accretion	4.553	accretion	0.168
Buried Revetment 2	2+600	accretion	0.511	accretion	0.206	erosion	-8.183	accretion	1.657	accretion	10.240	erosion	-0.195
	2+800	accretion	0.764	erosion	-0.630	erosion	-5.253	accretion	2.687	accretion	6.660	erosion	-0.136
	3+000	accretion	0.284	accretion	1.153	erosion	-3.518	accretion	0.580	accretion	4.990	accretion	0.083

	3+200	accretion	0.041	accretion	1.291	erosion	-6.580	accretion	2.010	accretion	4.613	erosion	-0.041
	3+400	erosion	-0.352	accretion	1.428	erosion	-3.015	accretion	0.207	accretion	3.297	erosion	-0.036
High Revetment 2	3+600	accretion	0.176	accretion	0.599	erosion	-2.798	accretion	0.823	accretion	2.133	erosion	-0.066
	3+800	erosion	-0.444	accretion	1.875	erosion	-4.575	accretion	1.660	accretion	2.653	erosion	-0.131
	4+000	accretion	0.074	accretion	1.597	erosion	-4.245	erosion	-0.487	accretion	5.413	erosion	-0.915
	4+200	accretion	0.188	accretion	0.665	erosion	-1.208	accretion	2.070	accretion	3.907	accretion	0.001

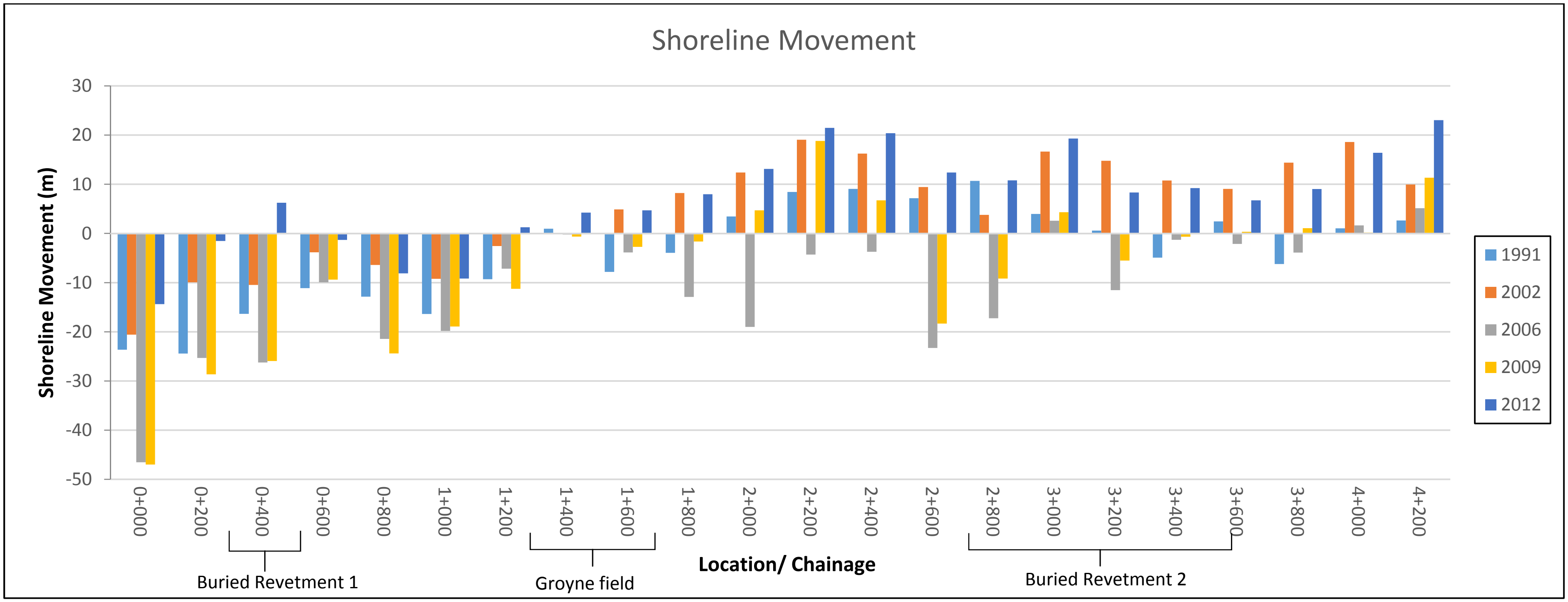


Figure 4.2 Graph showing the displacements of the shoreline for different years about the 1977 shoreline for Palisadoes (1991 to 2012)

The trends observed for the locations of interest are as follows:

Low revetment 1 (0 +200 to 0+600)

- The shoreline show trends of both erosion and accretion between the years 1977 and 2012.
- The highest rate of erosion of 3.9m/year was observed between 2002 and 2006, and this is expected because hurricane Ivan occurred in 2004 and caused severe damage to the shoreline.
- The highest accretion rate of 10.7 m/year was observed between 2009 and 2012.

Low revetment 2 (2 +600 to 3+400)

- The shoreline shows trends of accretion between the years 1977 and 2012 except for between 2002 and 2006 where erosion was observed following the passage of hurricane Ivan.
- The highest rate of erosion of 8.1 m/year was observed between 2002 and 2006, while the highest accretion rate of 10.1 m/year was observed between 2009 and 2012.

Groyne Field (1 +200 to 1+600)

- The shoreline show trends of both erosion and accretion between the years 1977 and 2012.
- The highest rate of erosion of 2.1m/year was observed between 2002 and 2006, while the highest accretion rate of 4.1 m/year was observed between 2009 and 2012.

Hurricane Trends (2002 to 2006)

- Hurricane Ivan was the most significant event in the over thirty years of shoreline observations. Whilst the overall trend was an accreting trend the mode during this period was obviously erosion.
- During the period 2002 to 2006 the entire shoreline eroded by an annual rate of -3.7 to -4.3 meters per annum.
- The estimated impact of the hurricane on the shoreline was a 16 meters erosion of the shoreline with a range of 4 to 26 meters.

General Trends

- The shoreline shows general trends of accretion occurring between 1991 and 2002. The rate of accretion varied between 0.3 m/year and 1.8 m/year.
- High levels of erosion were observed ranging from 1.5 m/year and 8.1 m/year and occurring between 2002 and 2006 following the passage of hurricane Ivan in 2004.
- The shoreline shows trends of accretion between 2009 and 2012 at rates between 0.8 m/year and 10.7 m/year.
- An overall trend of accretion was observed for 80% of the shoreline at rates between 0.1 m/year and 0.6m/year. The remaining 20% was observed to be eroding at rates between 0.04 m/year and 0.4 m/year

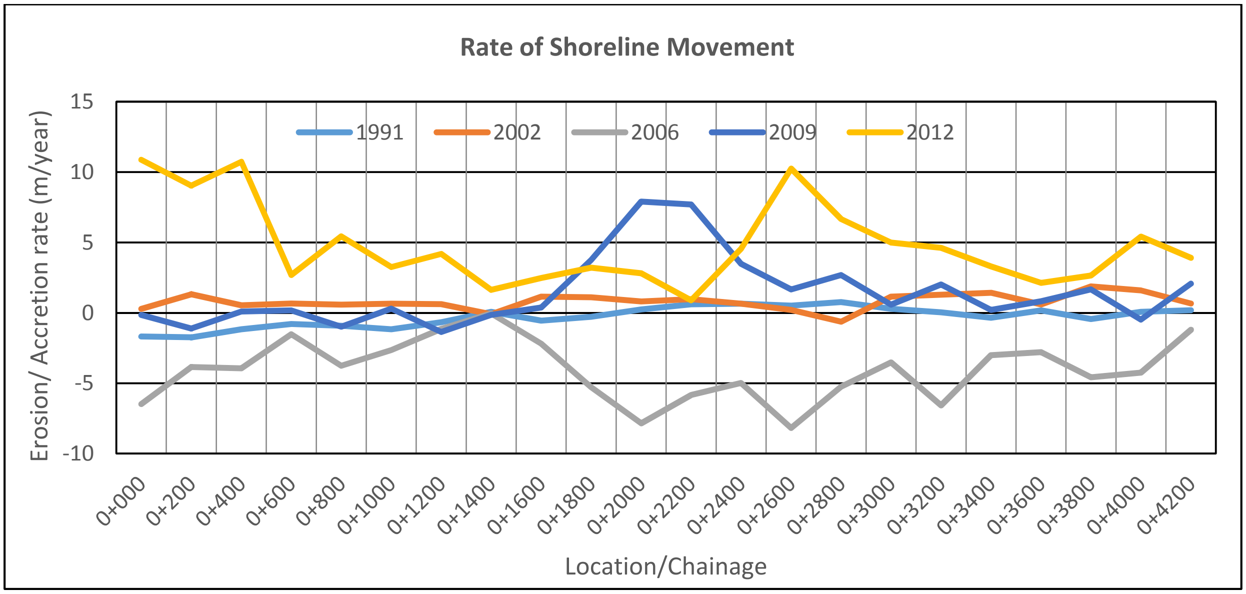


Figure 4.3 Graph showing the rates of erosion/ accretion for the shoreline about the 1968 shoreline for different time intervals between 1991 and 2013. Erosion occurred between 2002 and 2006 because of the passage of hurricane Ivan.

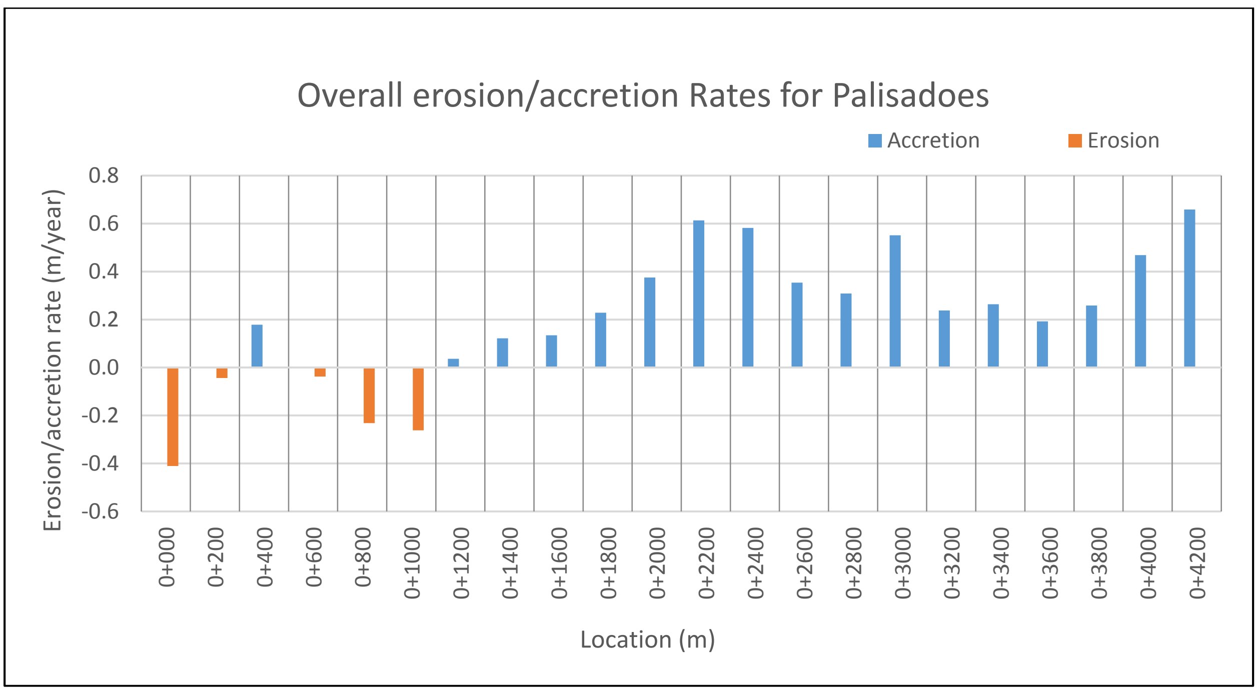


Figure 4.4 Graph showing the overall displacements of the shoreline about the 1968 shoreline for Palisadoes between 1991 and 2013. This graph indicates that the Palisadoes is in accretion mode.

4.1.3 Future Shoreline Projections without Project

4.1.4 Relative Impact of Sea Level Rise (SLR) versus Extreme Events

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 and this mathematical relationship was the basis for estimating shoreline retreat within the study area.

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

Where:

Δ y – Dune line erosion (meters/ year)

Δ s – Rate of sea level rise (meters/ year)

$l*$ – Length of the offshore profile out to a supposed depth, $h*$, of the limit of material exchange from the beach and the offshore (meters)

$h*$ – Depth at offshore limit, $l*$, to which near shore sediments exist (as opposed to finer- grained continental shelf sediments) (metres)

4.1.4.1 Rate of SLR, Δ s

Inspection of research in this area revealed that global sea level has risen as a result of greenhouse gas-induced global warming. Indeed, there will be regional variation in the sea level rise signal, and for this reason regions may undertake sea-level rise scenario modeling, which takes into account various factors such as land movement and region-specific oceanographic data.

For the purposes of this project, a simple scenario, based on one estimate of sea level rise will be utilized (not taking into account any vertical tectonic movements of the shoreline or any discernible change in the ocean geodynamic surface). Typically, a mid-range or upper estimate is chosen for such types of scenarios (A1B scenario from IPCC). The Intergovernmental Panel on Climate Change's (IPCC) Special Report on Emissions Scenarios (SRES) project global and Caribbean mean sea level to rise by on average 0.37 m by 2100 relative to 1980 – 1999 and so it was considered for the calculations, and specially the upper limit of this range.

Sea-level rise was projected to the year 2099, as the project life was chosen to be 2050 years. Using the upper limit value of 6 cm by 2050 allowed this analysis to test whether the Palisadoes is vulnerable to a plausible upper limit of climate change and simultaneous storm-induced short-term erosion for the 100-year return period.

4.1.4.2 Length of Offshore Profile, l^*

The calculated critical depth (or h^*) was used to estimate the length of the offshore profile. This was done by inspecting bathymetric data for both the Caribbean sea and harbor side of the Palisadoes and obtaining profile lengths for the corresponding critical depth. These profile lengths obtained were incorporated into the Bruun Model.

4.1.4.3 Depth to which nearshore sediments exist, h^*

A beach profile has a practical seaward limiting depth, where the wave conditions can no longer change the profile. Sand may move back and forth along this equilibrium profile, but there is no perceptible change in depth. This seaward limiting depth is equivalent to the depth at which nearshore sediments exist (h^*). Hallermeier (Hallermeier, 1981 in Kamphuis, 2000) refers to this depth as the critical or closure depth (d_c), and approximates it using the equation below:

$$d_c = 1.6H_{s,12}$$

Where:

$H_{s,12}$ – Significant wave height which occurs 12 hrs/yr on average

It was therefore necessary to determine the operational wave climate within the study area. Long term wave data available for the Palisadoes was analysed to determine the 12 hour wave ($H_{s,12}$) and it was determined that $H_{s,12}$ is a 2.2 m swell wave for the Caribbean sea side.

4.1.4.4 Calculation and Results

Table 4-2 shows the calculation of the long term trends expected in 25 and 50 years along the coast. As seen in this table, the following input values were incorporated into the Bruun Model to arrive at an estimate for the long-term erosion trend at each of the 3 profile shoreline positions:

- Rate of sea-level rise = 0.0037 m/yr (IPCC 2007)
- Depth to which nearshore sediment exists (h^*, d_c) = 3.5 m
- The offshore profile lengths were found to be approximately 200m

It should be emphasized here that the results of these calculations are an estimate of the projected shoreline retreat using a simplistic approach with an upper limit of global sea level rise. Indeed, the changes in beach profile over the years may have been impacted by the annual sea level rise as well as operational and storm-induced erosion estimated. This estimation of the sea level rise will assist in the determination of the true impacts that are due to operational a storm induces erosion.

The shoreline along the study area was estimated to retreat at a rate of 0.21 metres per year as a result of global sea level rise.

Table 4-2 Estimation of long-term erosion trends for Palisadoes using Bruun Model

Parameter	Profile		
	Low Revetment 1	Groyne field	Low Revetment 2
Chainage	0+500	1+400	3+000
Rate of sea level rise, Δs (m/yr)	0.0037	0.0037	0.0037
Offshore profile, l* (m)	200	200	200
Depth of offshore limit, h* (m)	3.52	3.52	3.52
Dune line Erosion, Δy (m)	0.21	0.21	0.21
Projected change/erosion in 25 years (m)	5.26	5.26	5.26
Projected change/erosion in 50 years (m)	10.51	10.51	10.51

4.1.5 Discussion and Comparison of Results

The historical model shows a general trend of accretion of about 0.35 meters per year, for the period between 1991 and 2013, except for between 2002 and 2006.The significant erosion observed can be attributed to the passage of Hurricane Charley (August 2004) and Hurricane Ivan (September 2004). Both hurricanes passed to the south of the island with Charley being a category 1 and Ivan category 4 at the time of passing.

The Bruun model, even though it deals specifically with erosion due to sea level rise, can still be applied to our case of general accretion. This means that even though the coastline is accreting, the rate at which it is growing is reduced by the effect of sea level rise. According to the Bruun model the rate of shoreline change for the Palisadoes is 0.21 m/year while the historical analysis determined an overall accretion rate of between 0.1 m/year and 0.6m/year for 80% of the shoreline, and the remaining 20% was observed to be eroding at rates between 0.04 m/year and 0.4 m/year rate.

4.1.6 Limitations

Both methods of estimating long term erosion trends have their own imitations. For the Brunn method, estimating long-term erosion trends as result of global sea level rise was not the main focus of this section. Given the anecdotal information in the area, it was important to know how the area is affected by long term and short term weather/climate events.

While for the historical model, the maps obtained were only snapshots at a moment in time that cannot be manipulated to show years or times of interest (such as immediately before and after the hurricanes). Therefore some of the maps may be displaying short term shoreline configurations while others long term. The accuracy of the rates is therefore subjected to the use of more Arial photos at strategic times which cannot be sourced.

4.1.7 Comparison to Other Beaches Across Jamaica

It was possible to compare the observations for Palisadoes to that of nine other beaches across Jamaica. A report provided by CEAC Solutions[7] determined if there was an underlying erosion pattern across Jamaica and estimated the risk associated. Specifically, nine beaches were analysed to determine their historical erosion rate and the influence of sea level rise versus storm induced erosion: Plumb Point, Long Bay (Portland), San San, Fort Clarence, Old Harbour Bay, Little Ochi, Priory, Annotto Bay and Long Bay Beach (Negril).

Short-term analysis revealed that eight of the nine beaches experienced short-term erosion varying between 0.1 to 0.52 metres per year. Only Little Ochi beach in St. Elizabeth exhibited accretion of the shoreline, see Table 4-4. The average short-term erosion rate observed was 0.26 metres per annum. Long-term shoreline retreat rates were observed to vary between 0.17 to 0.76 metres per annum, with an average of 0.26 metres per annum. The fastest eroding beaches were observed to be the Long Bay Beach (Negril) at 0.76 metres per annum followed by the Old Harbour Bay (St. Catherine) at 0.74 metres per annum. While the slowest eroding beaches were Annotto Bay (St. Mary) at 0.08 m/ yr, and Priory (St. Ann) at 0.10 m/yr followed by Plumb Point (Kingston) at 0.19 m/yr. Plumb Point is 2 km from the Palisadoes project and the erosion rate determined in this study (0.19 m/yr) compares favourably with the erosion rate determined by the Bruun Method (0.21 m/yr), the historical shoreline analysis however, determine a general accretion trend for 80% of the shoreline between 0.04 – 0.4 m/yr.

It is evident that the Palisadoes shoreline is accreting whilst just downstream at Plumb Point there is underlining erosion. Likewise, what is happening at Palisadoes is relatively unique (but similar to Little Ochi/Alligator Pond) where accretion is underway. It is therefore likely that localized processes with spatial variations of accretion and erosion are underway for the project, against a backdrop of island wide erosion. In light of these uncertainties it is recommended that monitoring be emphasized.

Table 4-3 Summary of analysis for the 9 beaches selected for the period 1968 to 2010

Beaches	Short-term rate of shoreline loss (m/ yr)	Long-term rate of shoreline loss (m/ yr)	Length of beach	Interval between profile (m)	Number of profiles used	Location/ Parish
Long Bay	-0.52	-0.36	1400	200	8	Portland
Priory	-0.10	-0.08	1000	200	11	St Ann
Fort Clarence	-0.48	-0.42	1250	250	4	St Catherine
Old Harbour Bay Fishing Beach	-0.59	-0.74	1000	200	6	St Catherine
Little Ochi	0.57	0.61	3000	500	4	St Elizabeth
Negril	-0.56	-0.76	5000	500	6	Westmoreland
Annotto Bay	-0.08	-0.25	3633	200	7	St Mary
San San	-0.38	-0.17	1600	500	8	Portland

[7] C. Burgess, C. Johnson, Shoreline Change in Jamaica: Observations for the period 1968 to 2010 and Risks for up to 2060

Plumb Point, Palisadoes	-0.19	-0.21	1200	200	8	Kingston
Overall average	**-0.26**	**-0.26**				

4.2 Sand Dune Design – Cross-shore Sediment Transport (CSHORE)

The CSHORE modeling exercise was carried out to confirm the dune cross section design and response to the 100 year return period storm event.

4.2.1 Model Description

Cross-SHORE (CSHORE) is a one-dimensional time-averaged nearshore profile model for predictions of wave height, water level, wave-induced steady currents, and profile evolution. The CSHORE model was originally developed by the University of Delaware to predict nearshore hydrodynamics and beach profile evolution for cases with upper beach profiles. The CSHORE model is a transect model that permits the specification of the actual beach profiles and sediment characteristics, thereby avoiding the ambiguity associated with the application of parametric models.

CSHORE assumes alongshore uniformity but computes the wave and current fields simultaneously. The combined wave and current model operates under the assumption of longshore uniformity and includes the effects of a wave roller and quadratic bottom shear stress. Computation times including nearshore morphology are typically 10^{-5} of the modeled time duration. Some of the features within CSHORE include, but are not limited to:

- Longshore Uniform Formulation;
- Steady Formulation;
- Shallow Water Hydrodynamics;
- Probabilistic Representation of Sediment Transport;
- Entrainment driven by Energy Dissipation;
- Includes Wave and Current Transport;
- Bed load and Suspended load.

4.2.2 Wave climate input, Calibration and Verification

4.2.2.1 Wave Characteristics Input

The wave data corresponding to Hurricane Ivan and anecdotal information collected from residents and employees in the area was used to calibrate and verify the model results, while the wave data corresponding to the 50 and 100 year storm events was used to model the existing and climate change scenarios. See **Error! Reference source not found.** for the input parameters for the calibration and modeling exercise.

Table 4-4 CSHORE input parameters for calibration and modeling for the Caribbean Sea side of the Palisadoes

Storm	H_s (m)	T_p (s)
IVAN	7.6	12.3
50 YR	5.9	12.1
100 YR	6.2	12.4

4.2.2.2 Calibration

Based on anecdotal information collected, it was determined that the calibration process could be undertaken and verified using this data for hurricane Ivan (2004). It was observed that approximately 1.2m of sand was transported and deposited on the main road, which became the benchmark for calibration. However, the challenge of obtaining shoreline topographic data before the hurricane event proved futile. Hence, the data was obtained from a survey conducted by Cuban team. Although this survey served as the most representative shoreline available, it still did not accurately represent the shoreline at the time of hurricane Ivan. Profiles from varying directions were cut from deep water to land, with a maximum depth below mean seal level (MSL) of 391 m, along the Caribbean Seas side in the vicinity of Plumb Point (near the airport) for the purpose of calibration.

Due to the challenge of obtaining a 2004 shoreline survey, another location near Plumb Point was used for fine tuning the existing calibration. A current eastern dune was observed to not have been affected by the passage of hurricane Ivan and this scenario was duplicated within the calibration. A profile from the western direction was cut and used for the second calibration process. The eastern dune was surveyed to have a crest elevation of 7.4 m and slopes of 1:7. The dimensions were input into the model, where parameters were modified to reflect the resistivity of the dune towards hurricane Ivan. During the modeling exercise, however, profiles were cut at western and eastern directions along the low revetment (Caribbean Sea side).

4.2.2.3 Verification

The first calibration method involved simulating the hurricane event Ivan depositing 1.2m of sand on the Palisadoes road in the vicinity of Plumb Point and it illustrated erosion of the seaward dune face of approximately 12 m inland and a reduction in crest height by 0.4m. Aside from these noticeable changes, the main feature used within the calibration was the deposition of sand on the roadway. The anecdotal data collected from both employees and residents within the surrounding areas recalled a height of 1.2 m of sand deposited on the roadway after the passage of Ivan. The model was calibrated to a tolerance of no more than 10%. This resulted in a model predicted accretion of approximately 1.3 m (8.3%) on the roadway.

See Table 4-5 Average height of sand on the Palisadoes roadway (Plumb Point) following the passage of Hurricane Ivan below for the average height of sand deposited on the roadway following the passage of Hurricane Ivan based on anecdotal data in comparison to model predictions obtained after the calibration exercise. The table shows that the model compares favorably with the anecdotal information gathered from residents. This calibration method was also used in the SBEACH modeling exercise.

Table 4-5 Average height of sand on the Palisadoes roadway (Plumb Point) following the passage of Hurricane Ivan

Location	Observed (m)	Model (m)
Plumb Point (roadway)	1.2	1.3

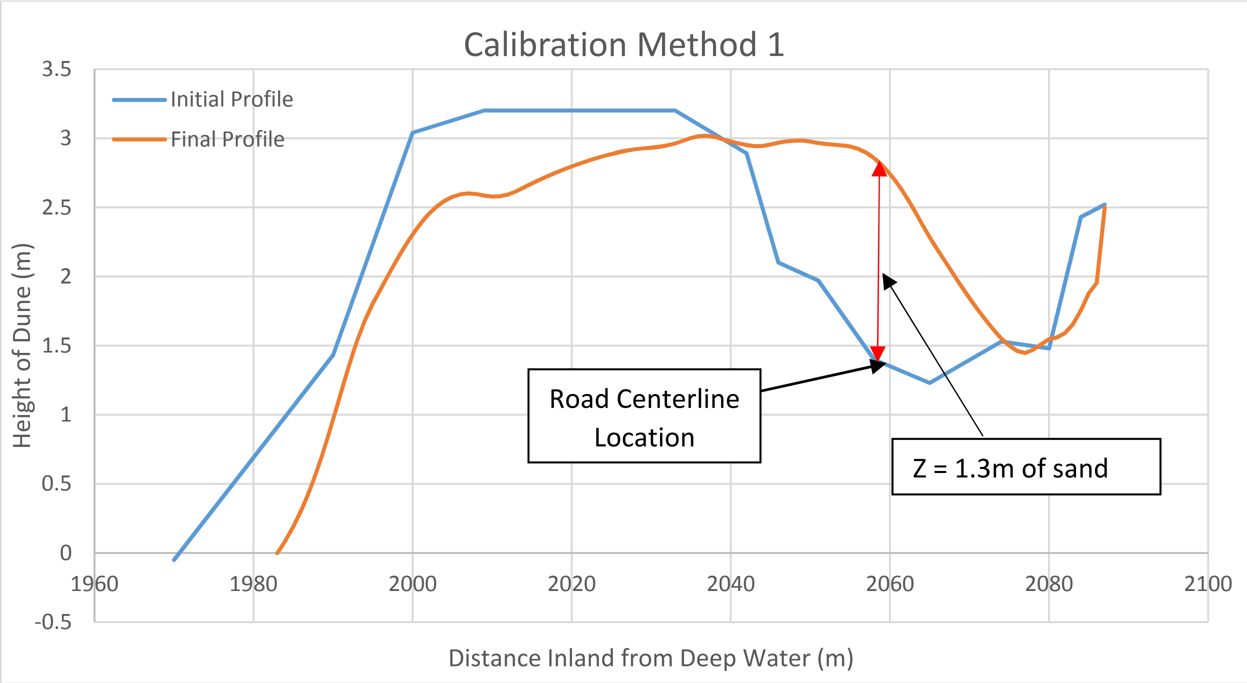


Figure 4.5 Calibration results comparing observed sand deposition versus model predictions

The second calibration method involved simulating the same hurricane event traversing an eastern dune along the shoreline in the vicinity of Plumb Point and it was used to further fine tune the first calibration method. As observed, following the passage of the hurricane, the dune did not sustain any physical changes which was simulated as best as possible within the model. This calibration run illustrated no erosion of the seaward dune face nor a reduction in crest height (see **Error! Reference source not found.**). **Error! Reference source not found.** below compares the observed dune height versus that predicted by the model post hurricane Ivan. The table shows that the model compares favorably with the anecdotal information gathered from residents.

Table 4-6 Average height of eastern sand dune along the Palisadoes shoreline

Location	Observed (m)	Model (m)
Plum Point (Eastern Dune)	7.4	7.4

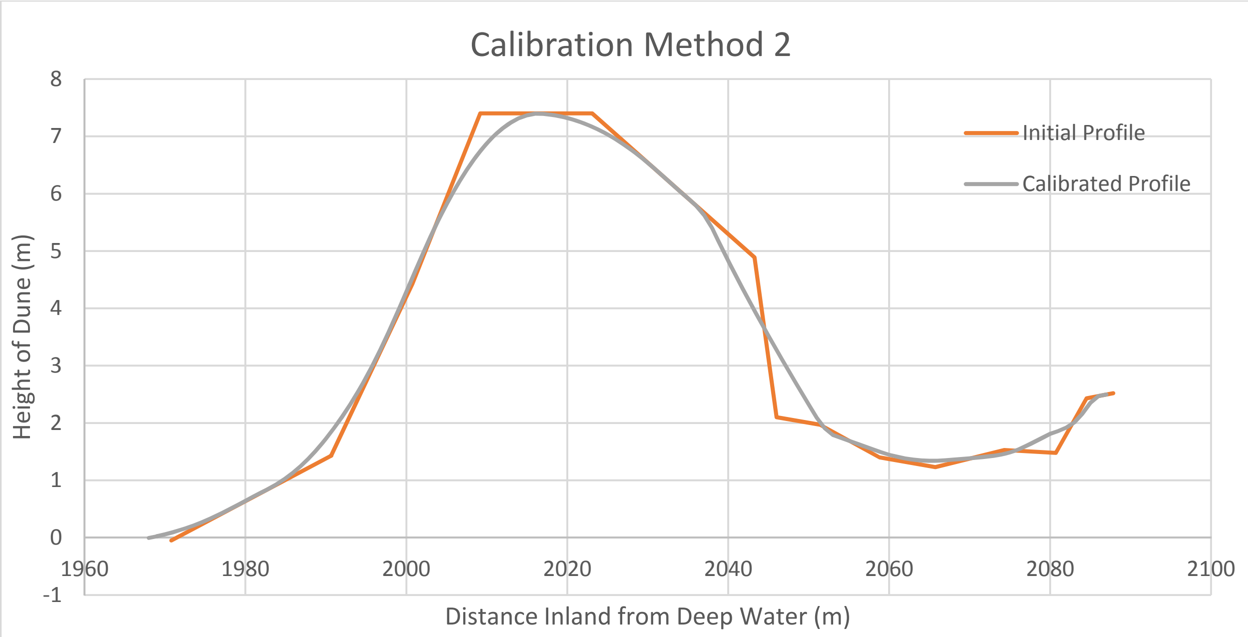


Figure 4.6 Calibration results comparing observed sand dune movement versus model predictions

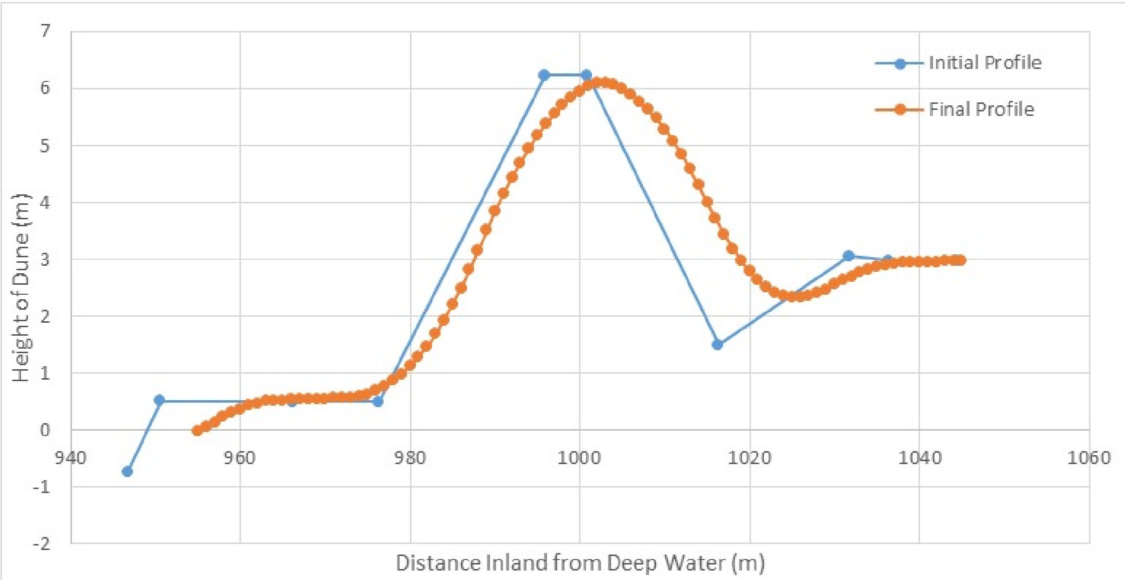
4.2.3 Post-Project Scenario with the Climate Change Wave Climate – Caribbean Sea

4.2.3.1 Eastern Direction

The model was run for the post project and climate change scenario so as to determine the stability and resistivity sand dune during both 50 year and 100 year rainfall events. The design process determined that the proposed sand dunes should have a 1: 3 slope on both the seaward and landward sides with a 12 m wide crest of an elevation 6.24 m.

The erosion vulnerability of the shoreline along the eastern profile was modeled and plotted for both 50 year and 100 year scenarios. The results revealed that the possibility of erosion of the seaward dune face exists for a distance of up to 2 m inland for the 50yr storm. Sand is also predicted to be deposited on the landward side of the dune 75 m from mean sea level (MSL). Essentially, the height of the dune will be reduced by 2.1% to a height of 6.11 m for this particular storm event. **Error! Reference source not found.** below illustrates the possible erosion of the proposed sand dune graphically. It can be determined that smaller return periods will subsequently produce less erosion.

In regards to the 100 year storm event, the results revealed that the possibility of erosion and accretion of the seaward dune face up to a distance of 10m and 15m inland respectively. Sand is also predicted to be deposited on the landward side of the dune 77 m from mean sea level (MSL). Essentially, the height of the dune will be reduced by 7.2% down to a height of 5.82 m for this particular storm event. Figure 4.7 below illustrates the possible erosion of the proposed sand dune graphically.



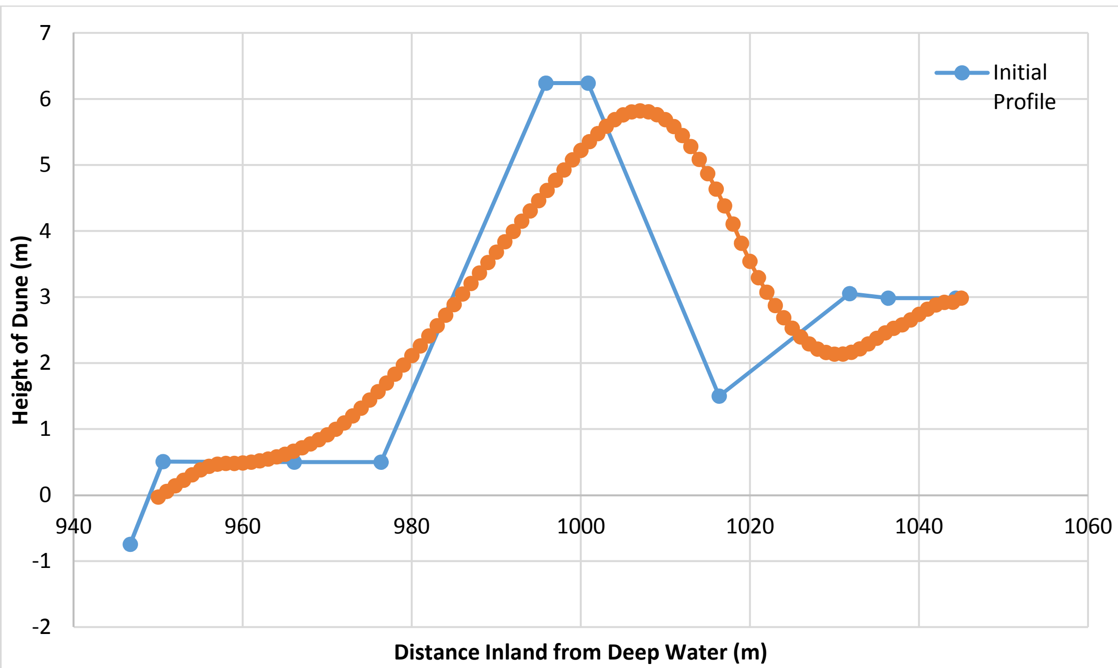


Figure 4.7 Simulation results comparing pre and post sand dune erosion predictions during 50 (top) and 100 (below) year event

4.2.3.2 Western Direction

The model was run for the post project and climate change scenario so as to determine the stability and resistivity of the sand dune during both 50 year and 100 year rainfall events. The design process determined that the proposed sand dunes should have a 1: 3 slope on both the seaward and landward sides with a 12 m wide crest of an elevation 6.24 m.

The erosion vulnerability of the shoreline along the eastern profile was modeled and plotted for both 50 year and 100 year scenarios. The results revealed that the possibility of erosion of the seaward dune face exist up to a distance of 0.5m inland for the 50yr storm. Essentially, the height of the dune will be reduced by 0.81% to a height of 6.19 m for this particular storm event. **Error! Reference source not found.** below illustrates the possible erosion of the proposed sand dune graphically. It can be determined that smaller return periods will subsequently produce less erosion.

In regards to the 100 year storm event, the results revealed that the possibility of erosion up to a distance of 0.5m inland. Sand is also predicted to be deposited on the landward side of the dune a height of 0.25m above existing ground. Essentially, the height of the dune will be reduced by 3.3% down to a height of 6.04 m for this particular storm event. Figure 4.8 below illustrates the possible erosion of the proposed sand dune graphically.

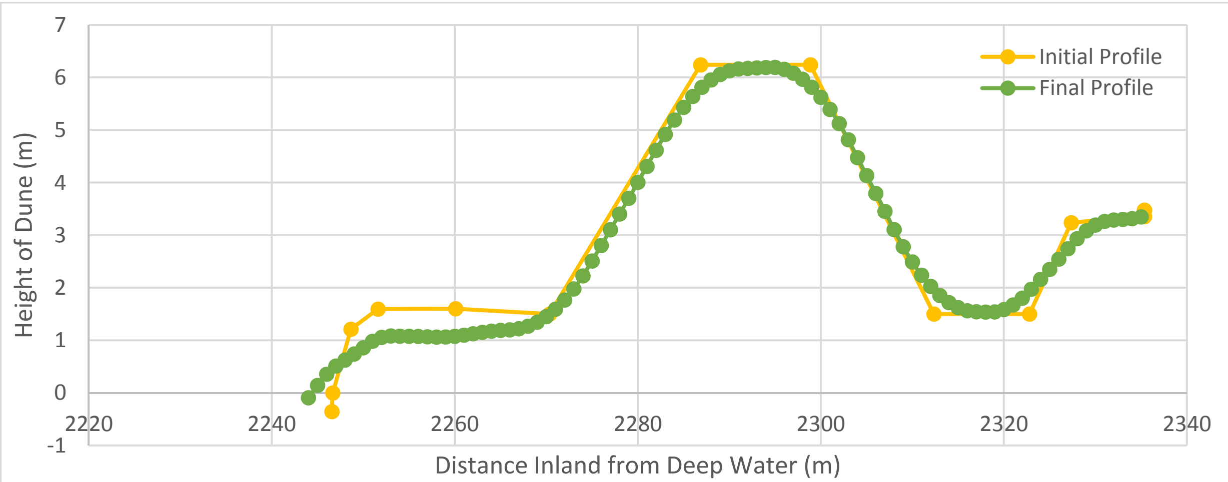


Figure 4.8 Simulation results comparing pre and post sand dune erosion predictions during 50 year event

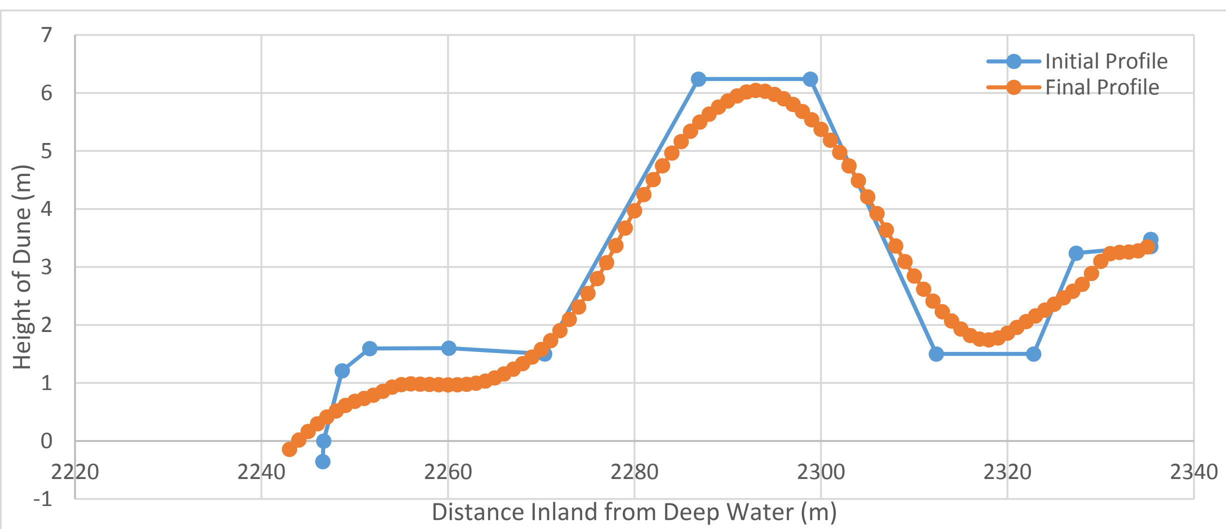


Figure 4.9 Simulation results comparing pre and post sand dune erosion predictions during 100 year event

4.2.3.3 Discussion

The Cross-SHORE (CSHORE) numerical model used to simulate cross-shore sediment transport allowed for an accurate calibration in conjunction with observations at two (2) different locations. The initial calibration involved anecdotal data obtained from workers and resident in proximity to the Palisadoes roadway after the passage of hurricane Ivan in 2004. This process yielded a percent error of 8.3% which falls below a tolerance of 10%, deeming this calibration run acceptable. In addition, further calibration runs were executed within the model involving a pre and post survey conducted near Plumb Point along the Palisadoes roadway during Ivan.

Based on the simulated model runs, it can be established that the seaward face of the proposed sand dunes are more vulnerable to erosion than the landward side. This was determined for both east and west directions during the 50 year and 100 year rainfall events. More specifically, the waves originating from the eastern direction proved to be more destructive as the model predicted. The eastern profile simulated within the model displayed greater erosion and accretion than that of the western profile. As a result the dunes will need to be inspected and restored as required after construction.

The design recommends western and eastern sand dunes with a 12 m long crest at an elevation of 6.24 m and a seaward and landward slope of 1: 3, this will prevent the waves from the 50 and 100 year storm event, with climate change considerations made, from damaging the roadway. Figure 4.10 and Figure 4.11 present the design cross sections for the sand dunes to be placed over the buried revetments. The volume of sand needed for construction is placed in Figure 4.7**Error! Reference source not found.**.

Table 4-7 Volume of sand required for sand dune construction

Sand Dune	Volume (m^3)
Buried Revetment 1	21,750
Buried Revetment 2	77,565
Sand Dune Option at Harbour Head	10,928
Total	**110,243**

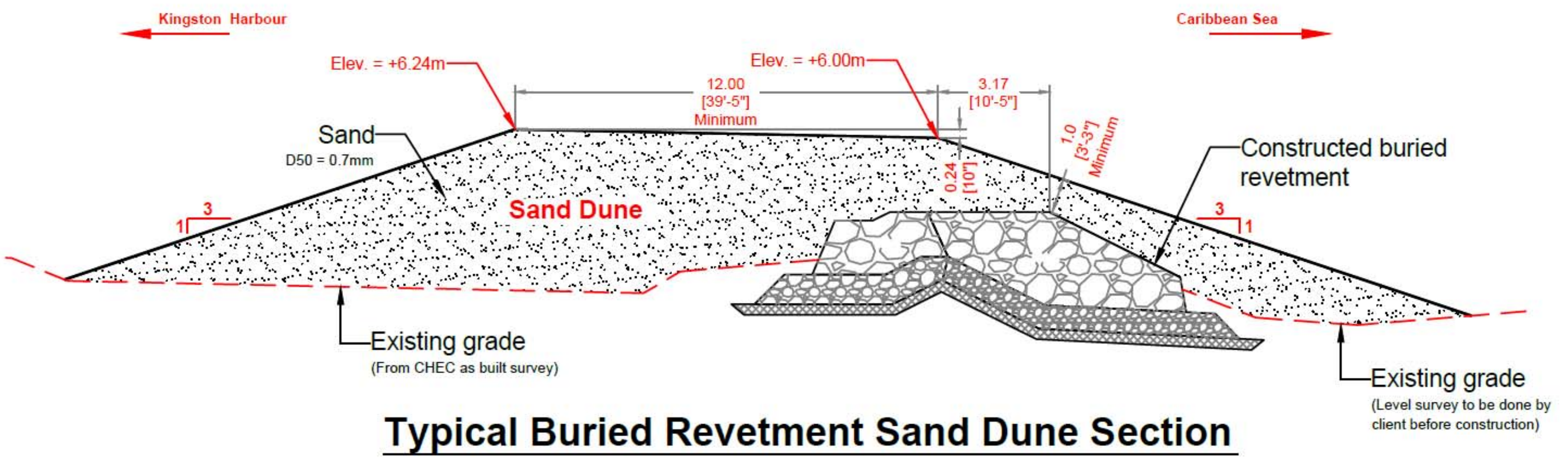


Figure 4.10 Typical section of the sand dune to be placed over the buried revetment along the Caribbean Sea side of the Palisadoes

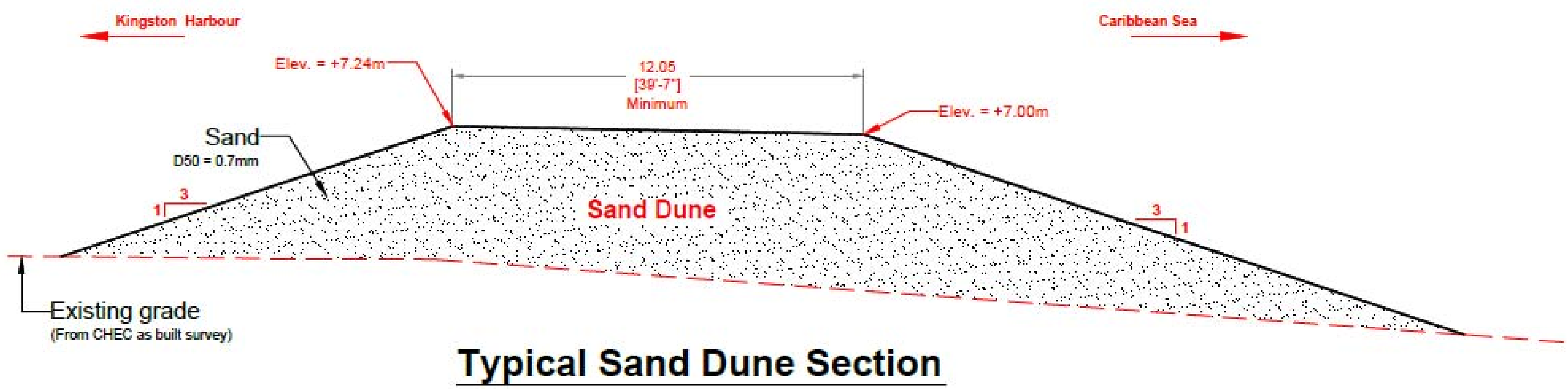


Figure 4.11 Typical section of the sand dune to close the 104 m gap between the high revetment and the NWC WWTP. This is an optional structure to be decided upon by NWA

4.3 Mangrove Nourishment Design – Cross-shore Sediment Transport (SBEACH)

The SBEACH modeling exercise was carried out to confirm a stable sand slope for the mangrove nourishment exercise along the harbor. The sand is to withstand the annual swell event.

4.3.1 Model Description

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).

4.3.2 Wave climate input, Calibration and Verification

Profiles were cut from deep water to land up to a maximum depth of 200 m at locations spanning the entire project area along its Caribbean Sea side. It should be noted that although we had accurate storm data for hurricane Ivan, we did not have accurate profile data for the Palisadoes at the time of the hurricane. Survey information collected in 2011 for Plumb Point was used, the dunes in that area were not overtopped during the passage of hurricane Ivan, and survey information for the project area provided by the Cuban technical team in 2007 were used to approximate the beach profile at the time of the hurricane's passing.

During the calibration exercise profiles were cut at a western, central and eastern point along the Caribbean Sea side of the project area and plumb point. However during the modeling exercise profiles were cut at a western, central and eastern point along the harbor side. The wave data corresponding to Hurricane Ivan, and anecdotal information collected from residents and employees in the area, was used to calibrate and verify the model results while the wave data corresponding to the 50 and 100 year storm events was used to model the existing and climate change scenarios. See Table 4-8 for the input parameters for the calibration and modeling exercise.

Table 4-8 SBEACH input parameters for calibration and modeling for the Caribbean Sea side of the Palisadoes

Input Parameters			
Storm	**Hs (m)**	**Tp (s)**	**Wind Speed (m/s)**
IVAN	7.6	12.3	64.3
50 YR	5.9	12.1	34.7
100 YR	6.2	12.4	38.4

See Table 4-9 for average height of sand brought up on the roadway following the passage of Hurricane Ivan as determined from interviewees and compared to model results obtained after the calibration exercise. The table shows that the model compares favorably with the anecdotal information gathered from residents.

Table 4-9 Average height of sand along the Palisadoes roadway following the passage of Hurricane Ivan, observed vs model results

Location	Observed (m)	Model (m)
East	1.2	1.7
Central	1.2	1
West	1.2	1.2
Plum Point	1.2	1.1

4.3.3 Beach Planform Modeling for the Post-Project Scenario with the Climate Change Wave Climate

The SBEACH model was also run for the post project and climate change scenario to design (determine) the stable sand slope for sand placement and mangrove nourishment. This was an iterative design process that incorporated feedback from the UWI team responsible for planting the mangroves. An effective mean grain sand size of 1 mm was used, based on the average mean grain size for the 3 sand samples taken from the mangrove adjacent to the project area.

The design process determined that the sand should have a back of beach elevation of 1.0 m, a seaward slope of 1: 10 to MSL, and a 1: 2 slope from MSL to the existing grade to provide the 6,000 m^2 of sand required to re-plant the mangroves that were previously lost during hurricane Ivan storm event. The sand in the western section of the harbour, when subjected to wave action of the annual swell, will move to a more stable slope of 1: 7. While for the sand placed in the central section of the harbor the 1: 10 slope is stable and will not move when subjected to the annual swell event. No sand should be placed in the eastern section of the harbor (between road chainage 3+000 and 4+200 m from the NMIA round-a-bout) as the slope of the sea floor is so steep that any sand placed there will be eroded when subjected to the annual swell event. Figure 4.12, Figure 4.13 and Figure 4.14 show the SBEACH results following the annual storm event at the western, central and eastern section of the harbor.

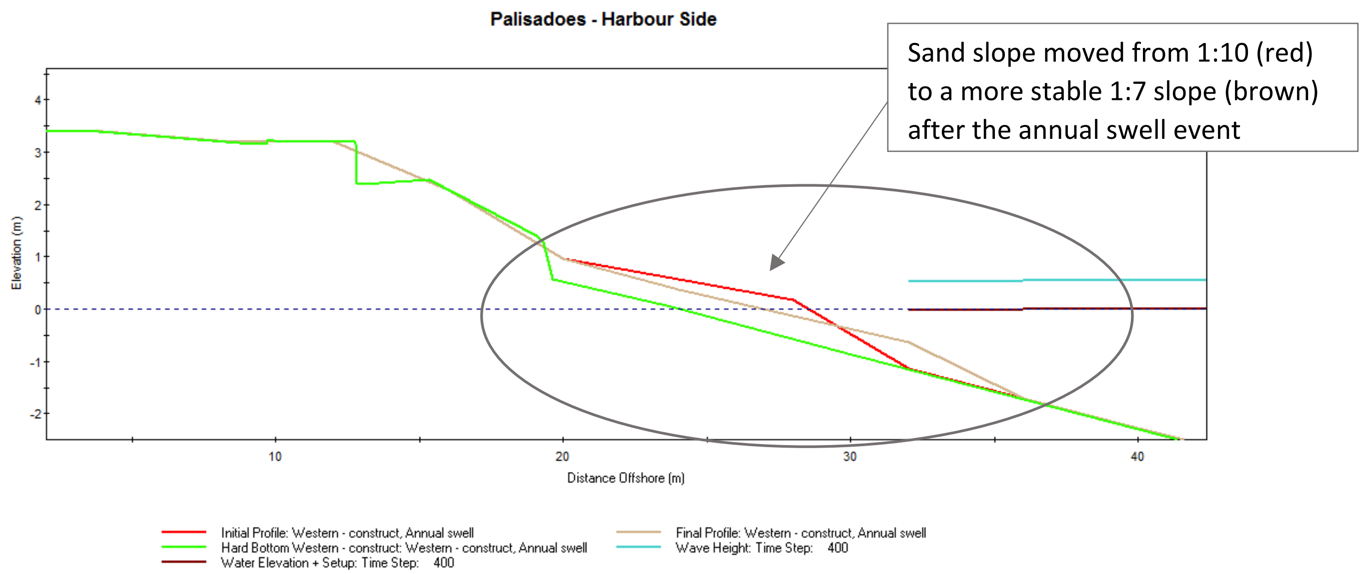


Figure 4.12 SBEACH results showing the sand placement for mangrove nourishment at the western section of the harbour. The sand is placed at a 1: 10 slope and moves to a 1:7 slope after the annual swell event. The 1: 7 slope is the stable slope for sand in this area.

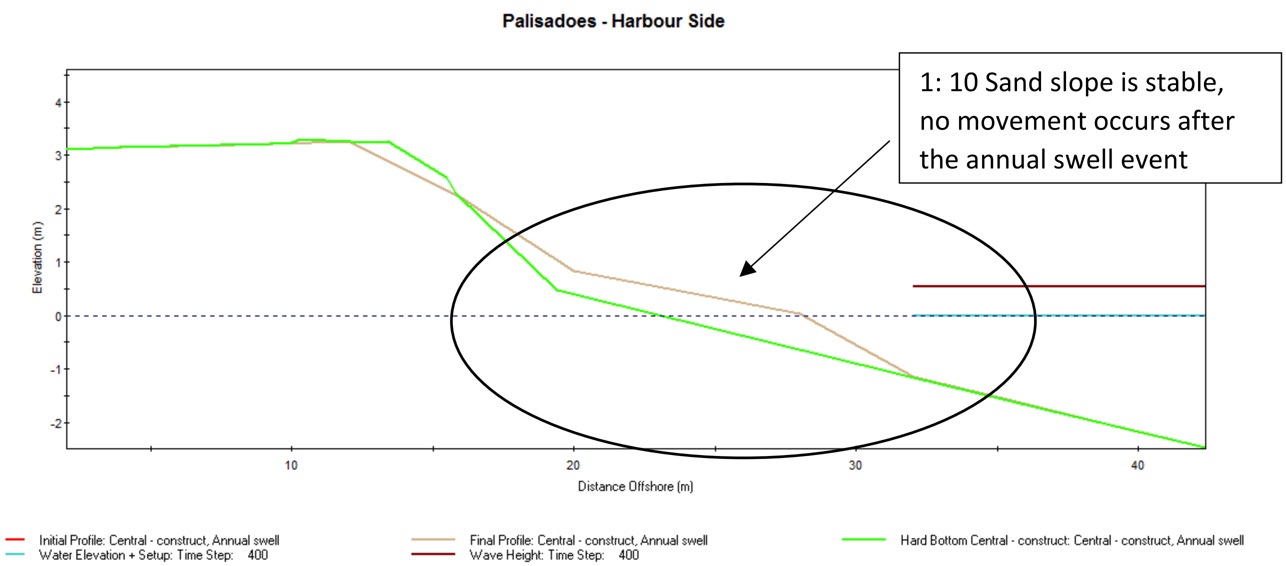


Figure 4.13 SBEACH results showing the sand placement for mangrove nourishment at the central section of the harbour. The sand is placed at a 1: 10 slope and it doesn't move after the annual swell event, the 1:10 slope is stable

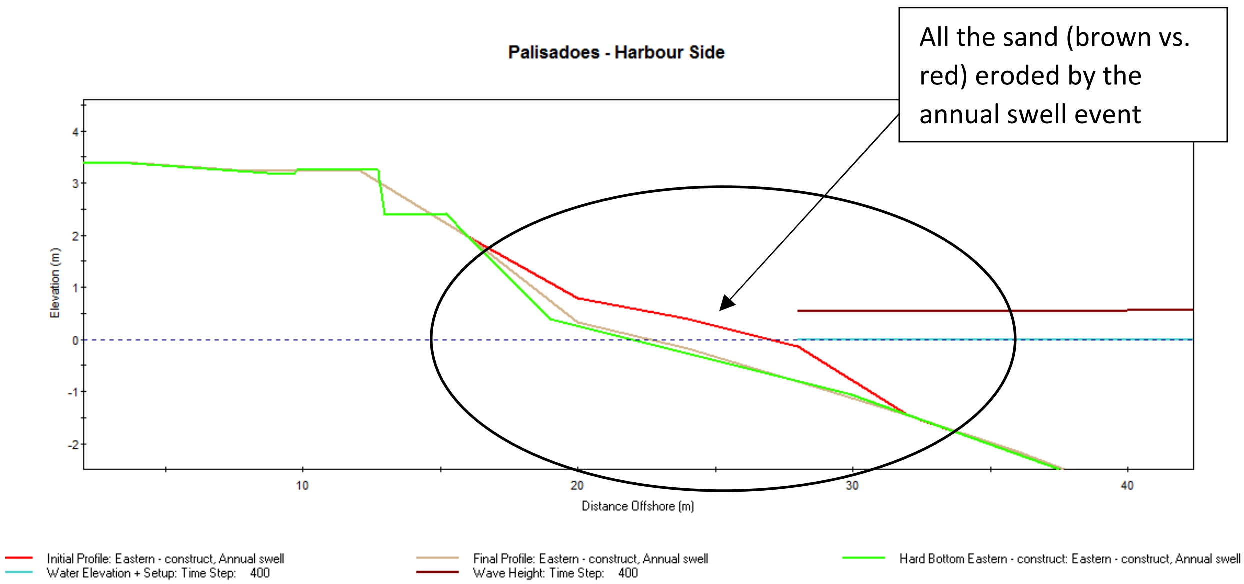
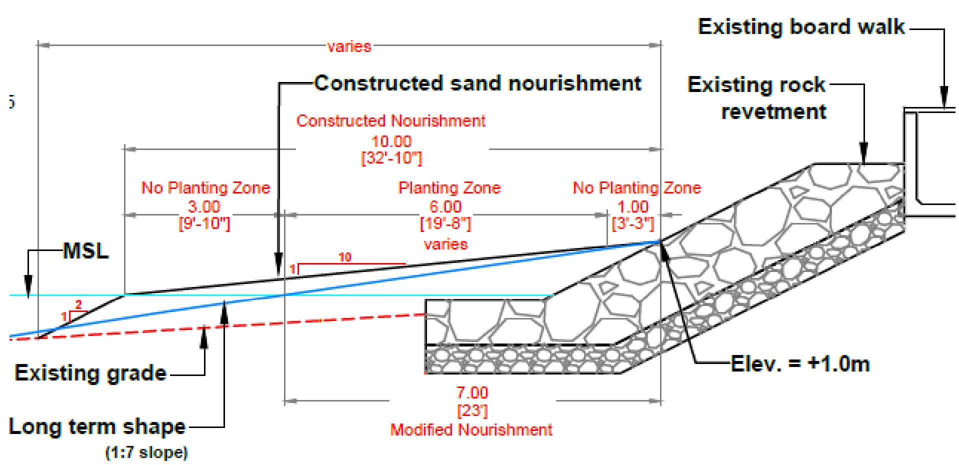


Figure 4.14 SBEACH results showing the sand placement for mangrove nourishment at the eastern section of the harbour. The sand is placed at a 1: 10 slope and it is almost completely eroded after the passing of the annual swell event

4.3.4 Summary

The SBEACH model was used to design a stable cross section for sand to be placed for mangrove nourishment. This sand must withstand the annual swell event. The final sand placement design will have a back of beach elevation of 1.0 m and have a seaward slope of 1: 10 to MSL, and a 1: 2 slope from MSL to the existing grade, see Figure 4.15. Reshaping is expected after the initial placement and due consideration should be given to monitoring the slopes before the vegetation is full established.



Typical Sand Nourishment section

Figure 4.15 Typical cross section for sand nourishment to be placed along the harbour side of the Palisadoes

4.4 Alongshore Sediment Transport Regime (GENESIS)

Sediments in the near shore are susceptible to movements in the direction of the shoreline or alongshore due to waves arriving at the shoreline at an angle less than ninety degrees. It was therefore necessary to investigate the long-term shoreline trends due to the operational, swell and hurricane wave climate in the near shore to determine the ideal areas placing the sand dunes and for replanting the mangroves.

4.4.1 Model Description and Development

The tool used for investigating the long term shoreline change was the Genesis model developed by the US army Corps. This Generalized Model for Simulating Shoreline Change simulates the long-term platform evolution of the beach in response to imposed wave conditions, coastal structures, and other engineering activity (e.g., beach nourishment). The region modeled was the Harbour Side and Caribbean Sea side Shoreline along the Palisadoes see Figure 4.16.

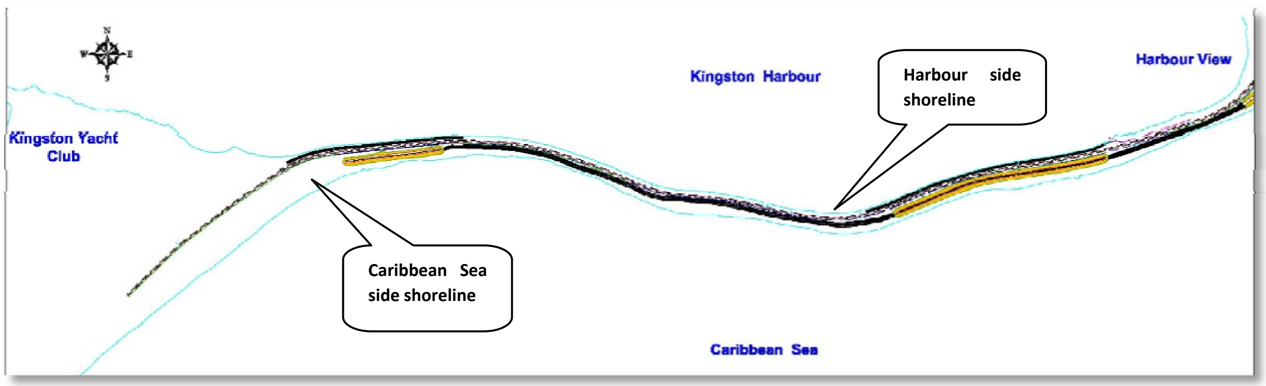


Figure 4.16 Shoreline locations used in the Genesis model

4.4.2 Wave climate input, Calibration and Verification

4.4.2.1 Wave Data

The most recent and complete annual wave data available for the Caribbean Sea and the Harbour side was for 2006. Wave data documented at three hour intervals were used to run the model, for the period 2000 through 2006 see Figure 4.17 and Figure 4.18.

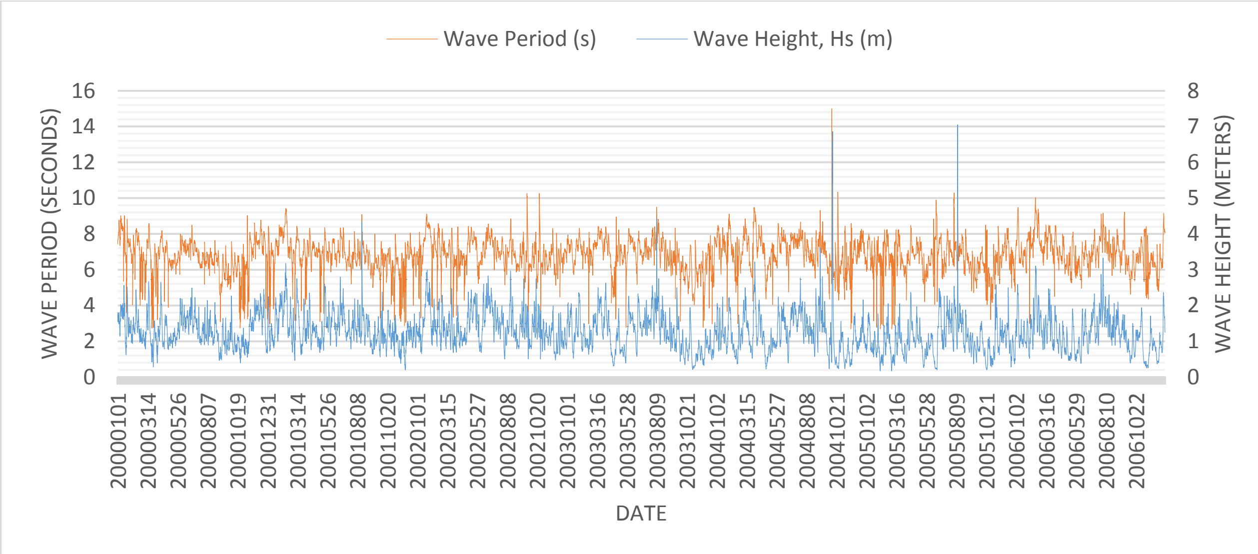


Figure 4.17 NOAA grib wave data for 2000 - 2006 that was used in sediment transport modeling for the Caribbean Sea side

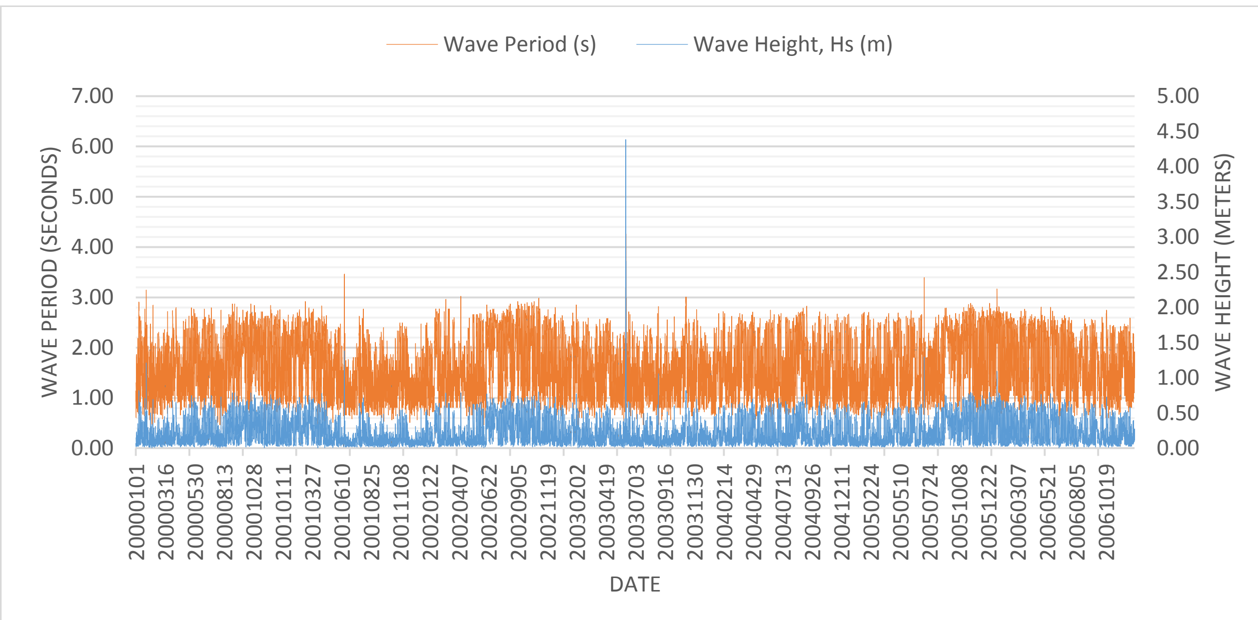


Figure 4.18 NOAA grib wave data for 2000 - 2006 that was used in sediment transport modeling for the Harbour Side

4.4.3 Initial Setup

The shoreline and bathymetry were defined as XYZ points and imported to setup the files required to run GENESIS. The operational, swell and hurricane wave data for a point offshore the Caribbean Sea Side and Harbour side Shoreline of the Palisadoes was obtained from the NOAA grib database for 2000 to 2006 (see Figure 4.17 and Figure 4.18 above) and implemented within the model to simulate the beach platform. An effective grain size of 0.3 and 1.1 mm determined from the sand sieving exercise conducted and used in the model for the Caribbean Sea Side and Harbour side respectively.

4.4.4 Calibration

4.4.4.1 Caribbean Sea Side

The model was calibrated based on movement of the Caribbean Sea side's shoreline observed from Google Earth and aerial imagery for the years 2000 and 2006 as outlined in previous sections of this report, and during this period there was a major storm event (hurricane Ivan, 2004). The calibration run (with long shore sand transport calibration coefficients parameters K1 = 0.15 and K2 = 0.075) was able to predict similar shoreline movement along the Caribbean Sea side Shoreline. The model's prediction is in line with observations even though the model, albeit slightly more conservative, and it was decided that this was sufficient to give accurate pre dictions. See Figure 4.19 below.

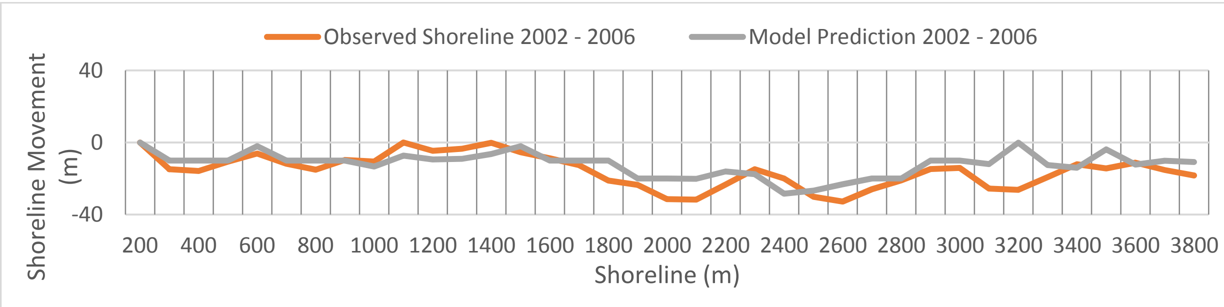


Figure 4.19 Calibration plots for the observed accretion patterns along the Harbour Side Shoreline in comparison to the models (Genesis) predication.

4.4.4.2 Harbour Side

The model was calibrated based on current accretion pattern along the Harbour side shoreline as observed and measured by CEAC Solutions. The calibration run (with long shore sand transport calibration coefficients parameters K1 = 0.75 and K2 = 0.375) was able to predict similar accretion patters along the Harbour side shoreline. The model's predation is in line with observations of accretion (post the shoreline project in 2012) and historical where mangrove grew in significant patches. It was decided that this was sufficient to give accurate predictions. See Figure 4.20 below.

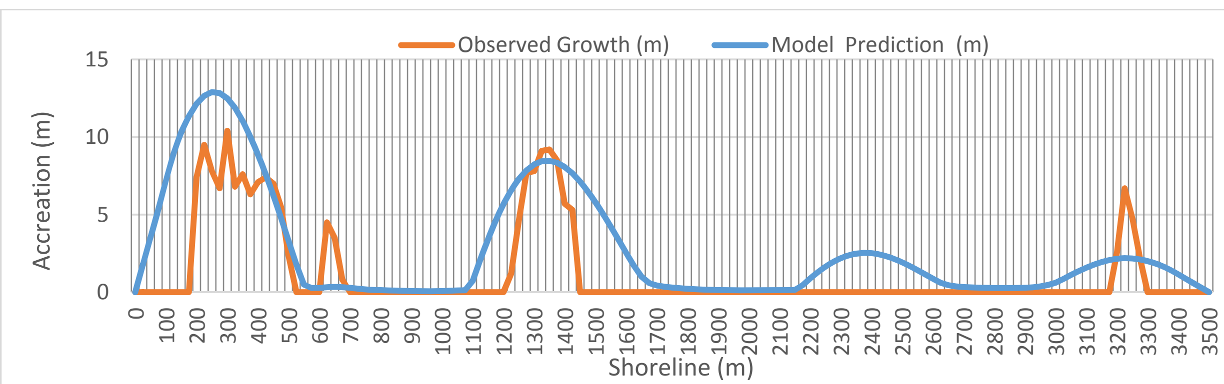


Figure 4.20 Calibration plots for the observed accretion patterns along the Harbour Side Shoreline in comparison to the models (Genesis) predication.

4.4.5 Results

4.4.5.1 Caribbean Side Pre-Project Scenario

The pre- project/existing Caribbean Sea side scenario revealed that the shoreline modeled is in erosion mode, resulting in a total volumetric loss of 647,000 m^3 in numerical simulations for the period 2000 to 2006. The model predicts that the central and eastern section of the Caribbean Sea side is more vulnerable to erosion with erosion widths of 20 to 30 m respectively. The average erosion along the shoreline is predicted to be 12 m in width. This correlates with the observations of 3.7 to 4.3 meters per annum over the four year period of 2002 to 2006. The total average erosion was 16 meters that is slightly larger than the model predictions for the same period with the intense hurricane Ivan.

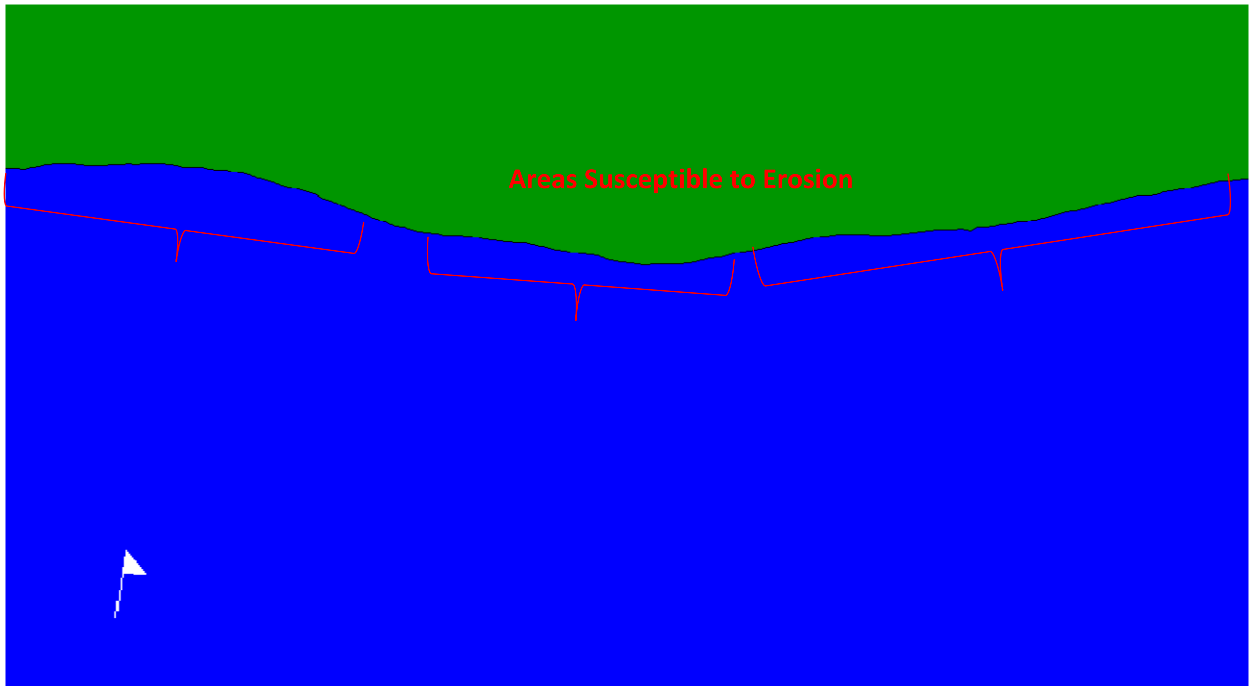


Figure 4.21- Beach planform after 6 years of simulation for the pre-project Palisadoes Caribbean Sea side shoreline

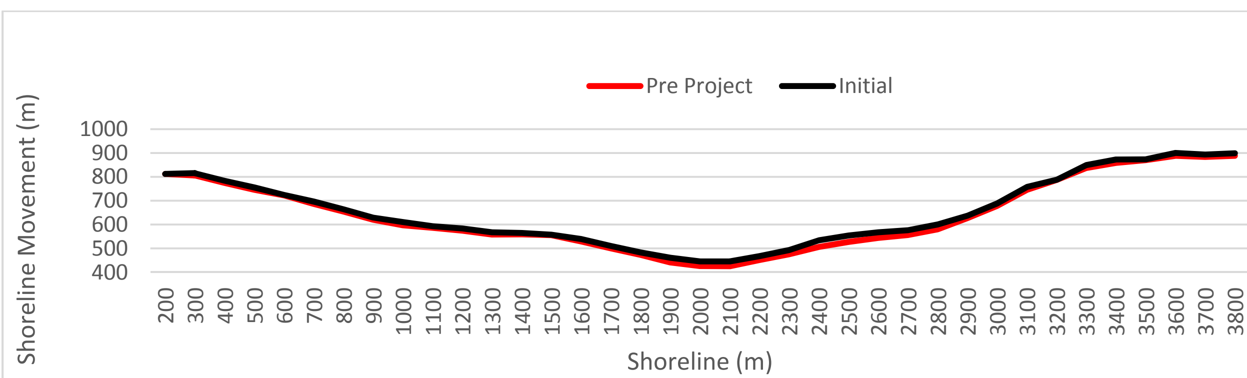


Figure 4.22 Comparative analysis of initial and pre-project shorelines for the Caribbean Sea side of the Palisadoes project

4.4.5.2 Caribbean Side Post-Project Scenario

The post project scenario for the Caribbean Sea side involves a change in bathymetry due to dredging of the barrow areas to a depth of 1.5 meters below its original depth as outlined in previous sections of this report. However this process did not affect the sediment transport along the shoreline when compared to the pre project scenario, see Figure 4.23.

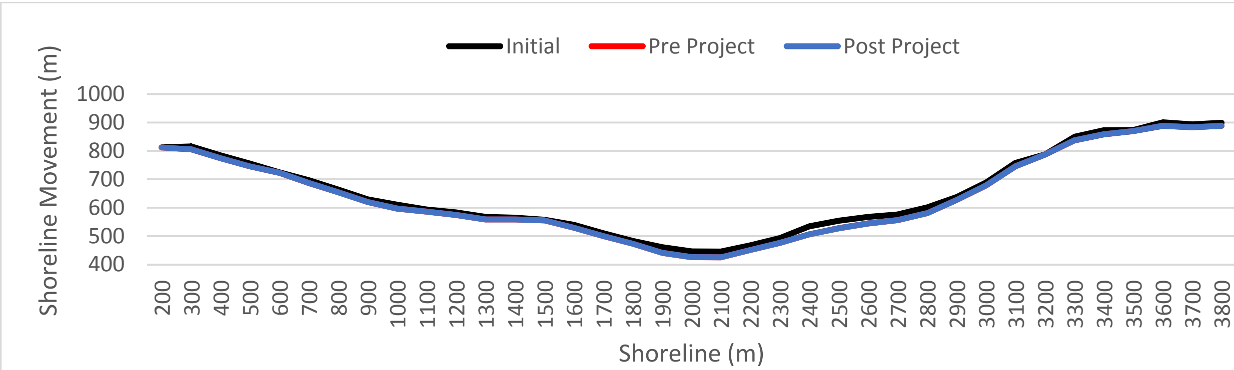


Figure 4.23 Comparative analysis of initial and post-project shoreline for the Caribbean Sea side of the Palisadoes project

4.4.5.3 Harbour Side Existing Scenario

The model predicts that the Western Section of the Harbour side shoreline is most stable and consistent with growth. Additionally a small area along the central and eastern section of the shoreline shows growth and stability, indicating that the shoreline model is in an accretion mode resulting in a total volumetric growth of 6,000 m^3, see Figure 4.24 and Figure 4.25.

Figure 4.24 Beach planform after 6 years of simulation for the existing bathymetry and conditions showing pre-project (north arrow shown) along the Palisadoes Harbour Side shoreline

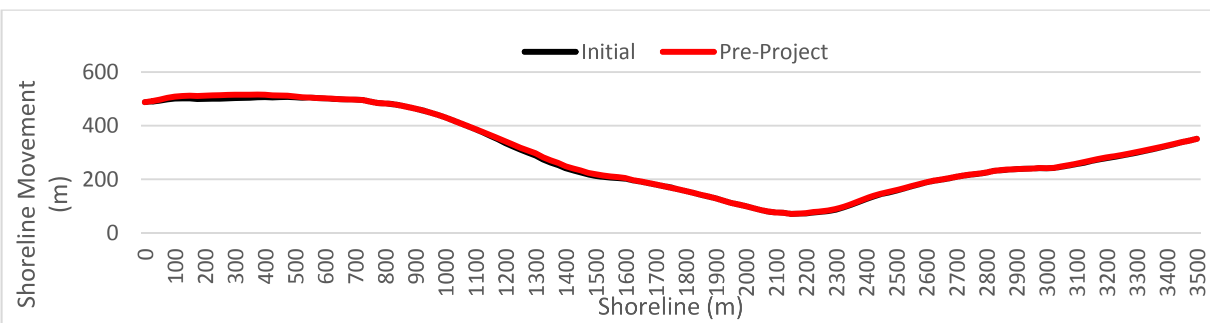


Figure 4.25 Comparative analysis of initial and pre-project shorelines for the Palisadoes Harbour Side shoreline

4.4.6 Existing and Climate change scenarios

When climate change was considered for the Harbour side shoreline, no change was observed when compared to the existing scenario without climate change. Similarly, when climate change was considered for the Caribbean Sea side pre and post project scenarios, no significant change was observed in the alongshore processes when compared to the pre and post project scenarios without climate change.

5 Hydrodynamic Modeling and Sediment Dispersion Modeling

5.1 Introduction

The current regime (i.e. patterns and speeds) in the coastal setting determines the ability of an area to flush and maintain sufficiently good water quality. Currents are generated mostly by winds, tides and waves. For tides and winds the simplified mechanisms are as follows:

- **Tides -** Rising tides will cause water to enter the bay and a portion will leave on falling tide that follows. This will result in some exchange of water between the outside and inside of the project area. This result is dependent on the ratio of the water entering to the water leaving; this ratio is dependent on the tide range, hydraulic efficiency of the entrance, and the water internal depths.
- **Wind -** Wind action over the water surface will generate a surface current that will essentially be in the direction of the wind. The wind generated current will be a few degrees to the right of the wind, (in the northern hemisphere), owing to the Coriolis effect (Bowden, 1983)[8]. If the fetch and duration are sufficient, the surface current speeds may approach 2-3% of the wind speeds.

Circulation patterns can be predicted by numerical, physical models or by field studies. Numerical models are most often used as they simply require collection of field data to calibrate and verify the model for use in a predictive mode. The models are also robust enough to include prediction of sediments and nutrients dispersion in the project area.

5.2 Description of Models

Investigation of currents was undertaken using RMA10. It utilizes bathymetric information on the project area and driving forces from tides and winds to solve the 3-dimensional flow equations. This model is calibrated on the observations of currents through the project area from drogues and the moored ADCP. The sediment plume models were generated using RMA11. RMA11 is a finite element water quality model for simulation of three-dimensional estuaries, bays, lakes and rivers. It is also capable of simulating one and two dimensional approximations to systems either separately or in combined form. It is designed to accept input of velocities and depths, either from an ASCII data file or from binary results files produced by the two-dimensional hydrodynamic model, RMA2, or the three-dimensional stratified flow model, RMA10. Results in the form of velocities and depth from the hydrodynamic models are used in the solution of the advection diffusion constituent transport equations.

5.2.1 RMA 10

RMA-10 is a three-dimensional finite element model for stratified flow by King (1993). The primary features of RMA-10 are:

- The solution of the Navier-Stokes equations in three-dimensions;
- The use of the shallow-water and hydrostatic assumptions;
- Coupling of advection and diffusion of temperature, salinity and sediment to the hydrodynamics;
- The inclusion of turbulence in Reynolds stress form;
- Horizontal components of the non-linear terms are included;

[8] Bowden, KF . 1983. ***Physical Oceanography of Coastal Waters***, John Wiley, NY

- A capacity to include one-dimensional, depth-averaged, laterally-averaged and three-dimensional elements within a single mesh as appropriate;
- No-, partial- and full-slip conditions can be applied at both lateral boundaries;
- Partial or no-slip conditions can be applied at the bed;
- Depth-averaged elements can be made wet and dry during a simulation; and
- Vertical turbulence quantities are estimated by either a quadratic parameterization of turbulent exchange or a Mellor-Yamada Level 2 turbulence sub-model.

5.2.2 RMA 11

The RMA 11 sediment transport model by (King and Rachiele, Multi-dimensional modeling of hydrodynamics and salinity in San Francisco Bay) (King and DeGeorge, Multi Dimensional Modeling of Water Quality Using the Finite Element Method) is a three dimensional finite element model that can also function as a two dimensional depth averaged model. The primary features of RMA11 are as follows.

- RMA11 shares many of the same capabilities of the RMA2/RMA10 hydrodynamics models including irregular boundary configurations, variable element size, one-dimensional elements, and the wetting and drying of shallow portions of the modeled region.
- RMA11 may be executed in steady-state or dynamic mode. The velocities supplied may be constant or interpolated from an input file (This may be RMA2 or RMA10 output).
- Source pollutants loads may be input to the system either at discrete points, over elements, or as fixed boundary values.
- In formulating the element equations, the element coordinate system is realigned with the local flow direction. This permits the longitudinal and transverse diffusion terms to be separated, with the net effect being to limit excessive constituent dispersion in the direction transverse to flow.
- For increased computational efficiency, up to fifteen constituents may be modeled at one time, each with separately defined loading, decay and initial conditions.
- The model may be used to simulate temperature with a full heat exchange with the atmosphere, nitrogen and phosphorous nutrient cycles, BOD-DO, algae, cohesive or non-cohesive suspended sediments and other non-conservative constituents.

A multi-layer bed model for the cohesive sediment transport constituent keeps track of thickness and consolidation of each layer.

The process of mesh developments entails the following steps:

- Input of bathymetric data for the wider area and in detail for the project area
- Specifying of nodes in the mesh
- Element construction in the mesh
- Interpolation for depth at nodes
- Specifying of open boundaries

The mesh constructed for the calibration and existing configuration extended some 7.7 kilometer s in a westerly direction. The outer deep water areas were gridded with large mesh which gradually decreases on approach to the project area. See Figure 5.1 below. The eastern and western boundaries were used as the open boundaries on which tides were applied.

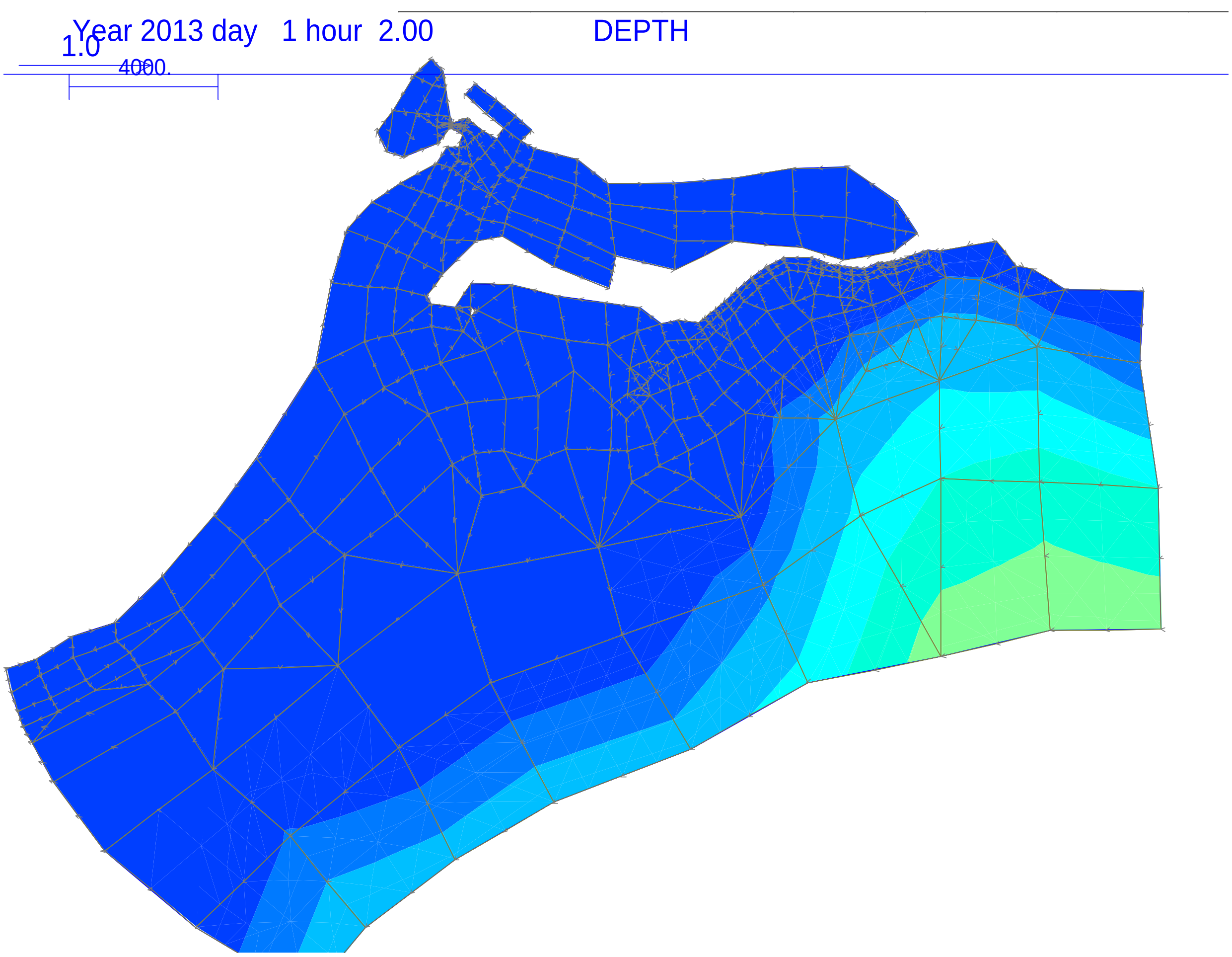


Figure 5.1 Overview of entire Finite Element Mesh used for this project showing depth in meters

5.3 Calibration

The model was calibrated by adjusting the tide elevation signal on the model boundaries, turbulence and viscosity parameters, until there was reasonable agreement between the observed currents and model predictions.

Correlations were 0.7 and 0.8 for the Vx and Vy components respectively, when obvious outliers were not considered. The predicted current speeds and directions, versus the data from the drogue tracking sessions are summarized in

Table 5-1 for the correlation coefficient and variance between the predicted and observed currents. The model predictions agreed with the observations in most instances and indicate that the model can be used with confidence.

Table 5-1 Correlation coefficient and bias between the observed (ADCP for October 15 2013 and November 15 2013) and predicted (hydrodynamic model) currents.

Direction (vector)	Vx (m/s)	Vy (m/s)
Correlation (model predictions VS ADCP readings)	0.7	0.8
Variance	1.0%	0.3%
Std. Deviation	0.10	0.06

5.4 Current Predictions

5.4.1 Approach

The current speeds were investigated for different wind speeds and directions given their impacts on currents in the bay. The wind directions and speeds investigated were the Easterly direction as the occurrences were predominantly from the ENE to ESE directions. See Table 5.2 below for the wind speeds and directions used. The results are summarized in the sections below as well as in Table 5.2 and Table 5.5.

Table 5.2 Wind Speeds and Directions investigated in the Hydrodynamic model

Wind Speed (M/S)	Wind Direction
	ENE
Slow	1.5
Average	5.5
Fast	15.5

5.4.2 Slow Wind Speed Days

During rising tides, the currents were predominantly east to west in the vicinity of the offshore dredge sites. The western dredge site also had currents moving to the southwest to align with the coast. The currents are generally between 6 and 12cm/s western limits of the project both for offshore and near shore currents. The eastern section of the site however has currents of up to 12cm near shore whereas offshore currents are in the order of 4-6cm/s.

During the falling tides, the currents are generally faster in the near shore and tend to move to west along the shoreline. The speeds are predicted be as high as 0.6 to 0.9cm/s. the offshore currents are however less defined in terms of a direction. Most if the currents appeared to be moving offshore to the south at speeds of less than 3cm/s.

The winds speeds were general slow during both sessions and did not appear to have any noticeable impact on the currents.

5.4.3 Average Wind Speed Days

During rising tides, the currents were predominantly east to west in the vicinity of the offshore dredge sites. The western dredge site also had currents moving to the southwest to align with the coast. The currents are generally between 6 and 12cm/s western limits of the project both for offshore and near shore currents. The eastern section of the site however has currents of up to 12cm/s near shore whereas offshore currents are in the order of 4 - 6 cm/s.

During the falling tides, the currents are generally faster in the near shore and tend to move to west along the shoreline. The speeds are predicted be as high as 0.6 to 0.9cm/s. the offshore currents are however less defined in terms of a direction. Most if the currents appeared to be moving offshore to the south at speeds of less than 3cm/s.

The average wind speeds used during both rising and falling tides. The winds did not appear to have any more impact on the currents than the slow winds.

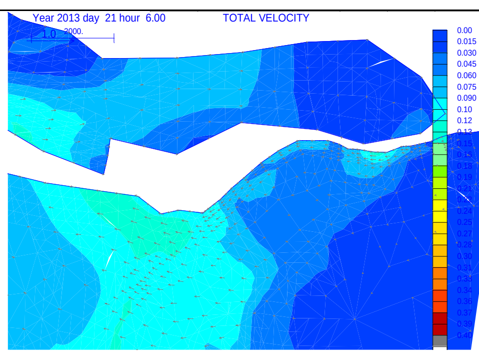
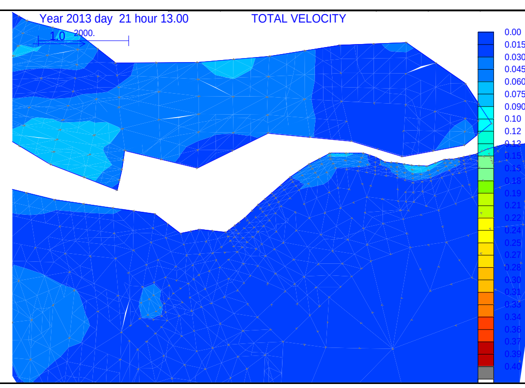
5.4.4 Fast Day

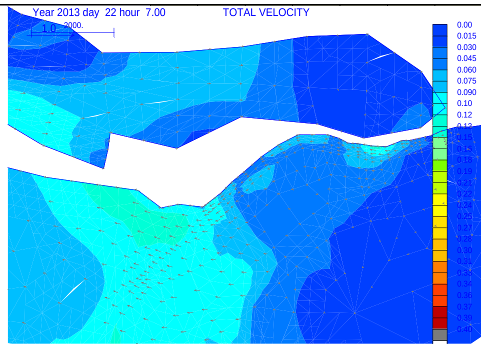
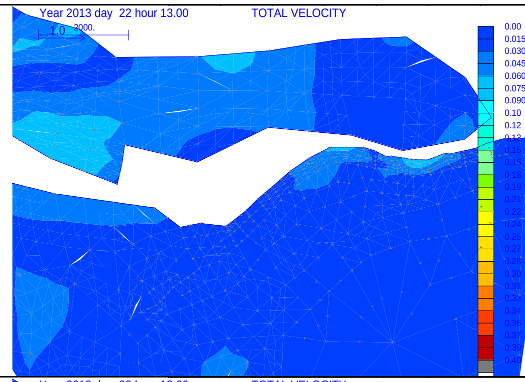
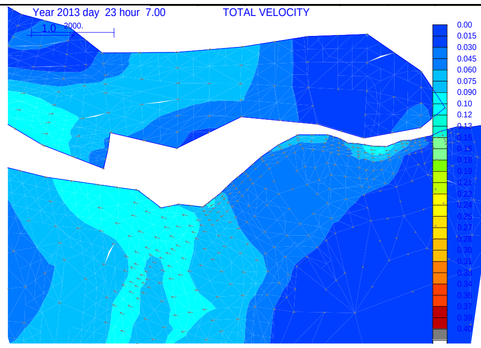
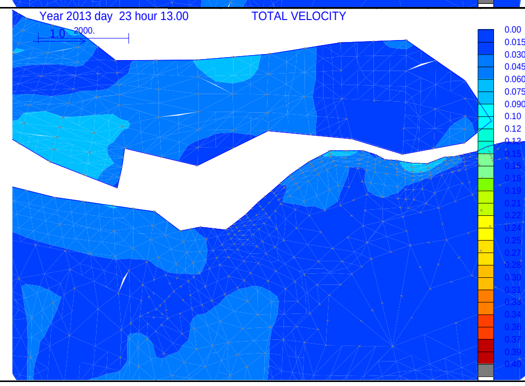
During rising tides, the currents were predominantly east to west in the vicinity of the offshore dredge sites. The western dredge site also had currents moving to the southwest to align with the coast. The currents are generally between 6 and 12cm/s western limits of the project both for offshore and near shore currents. The eastern section of the site however has currents of up to 12cm/s near shore whereas offshore currents are in the order of 4-6cm/s.

During the falling tides, the currents are generally faster in the near shore and tend to move to west along the shoreline. The speeds are predicted be as high as 0.6 to 0.9cm/s. The near shore fast current speeds had a wider offshore spread than the slow and average wind days. The offshore currents are less defined in terms of a direction. Most if the currents appeared to be moving offshore to the south at speeds of less than 3cm/s.

The fast wind speeds used during both rising and falling tides. The winds did not appear to have any more impact on the currents than the slow winds.

Table 5.3 Current speed predictions for the preconstruction and post-construction scenarios at Long Bay Negril for predominantly ENE winds

Wind speed	Rising	Falling
Slow (1.5 m/s)		

Average (5.5 m/s		
Fast (15.5 m/s)		

5.5 Sediment Plume Modeling

5.5.1 Acceptable limits of Suspended Solids

It was important to establish the acceptable sediment plume concentration for use in this study. The National Environment and Planning Agency (NEPA) have guidelines on this matter and recommend a maximum of 10 mg/l (Natural Resources Conservation Authority). This is in comparison to an existing background level ranging from 3 to 5 mg/l.

Observations of requirements and other international guidelines suggest a higher range may be suitable for marine vegetation and corals. For example (Dennison, Orth and Moore) and (Gallegos and Kenworthy) suggest a value of 15 mg/l for both tropical and freshwater lake settings, and (Devlin and Schaffelke) suggested levels of up to 23 mg/l on the Great Barrier Reef after flood events.

Whilst a guideline of 10 mg/l exists locally, the results of the analysis will be interpreted in the context of the range of international guidelines as well of up to 15 mg/l.

5.5.2 Source of Sediments

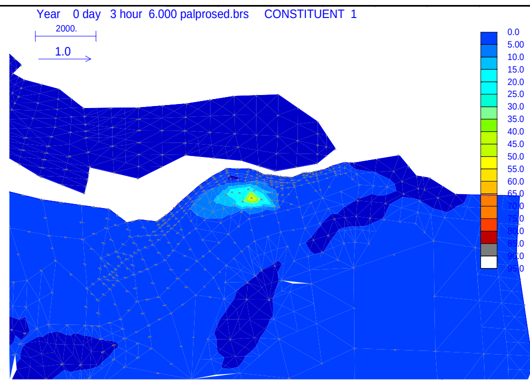
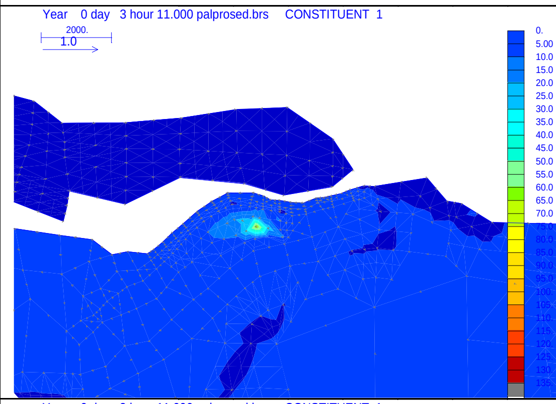
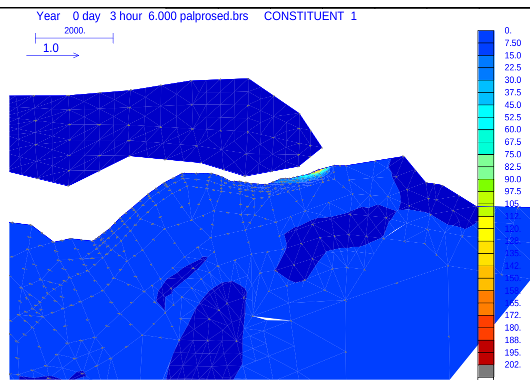
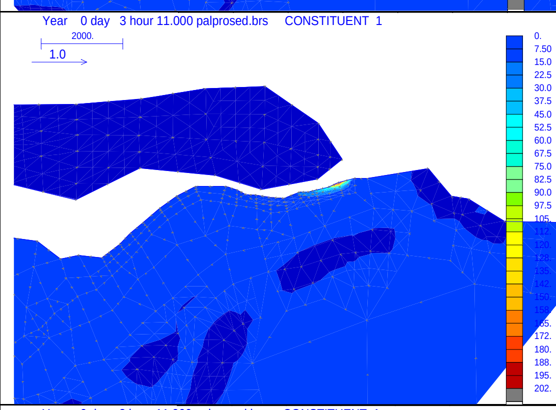
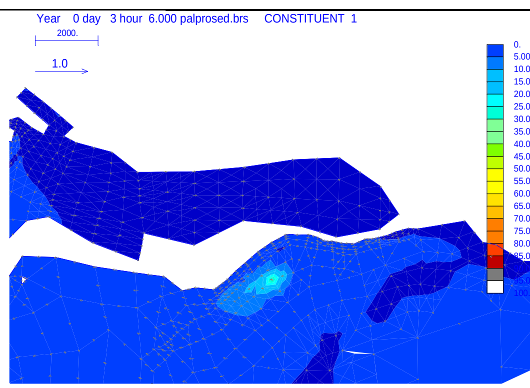
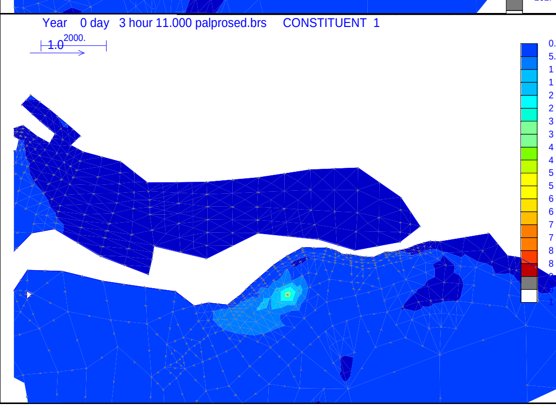
Samples of the sediments (that will be dredged) were observed to have less than 1 percent silt. An attempt was made to rationalize the likely silt load at the dredge sites as well as at the shoreline where the settling ponds will overflow back into the sea. It was estimated that the sediment loading at will be 9.5 and 1.9 grams per litre at the dredge sites and at the shoreline respectively. This rate was applied uniformly over the 24 hours of each day to account for the possibility of the contractors working during the nights.

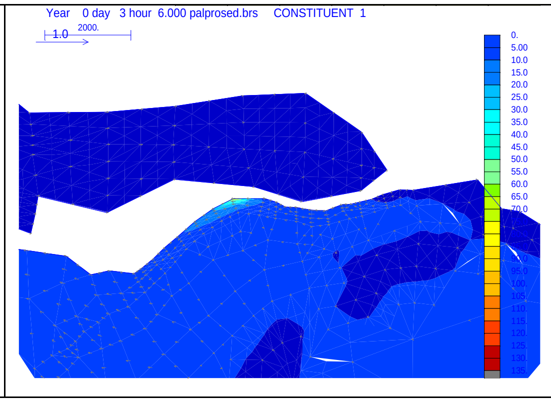
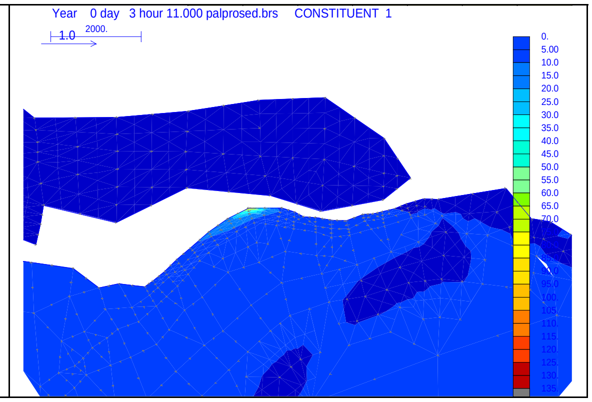
5.5.3 Results

Plume modeling suggests that the extent of the offshore plumes are in general larger than those in the near shore. In all cases the plumes travel to the west up to distances of 2km from the discharge points. In all cases the concentrations were below 20mg/L after 1.5km. This is in comparison to the background concentrations of less than 10mg/L.

Turbidity barriers should be installed around the works especially on the western side where the currents are most likely to travel. The bio-physical features on this side are therefore vulnerable to the associated risks of turbidity in the water column. There is also an increased risk of the plume contacting the shoreline along plumb point shoreline if the turbidity is not adequately controlled.

Table 5.4 – Sediment plume modeling results (mg/l of TSS) for rising and falling tides

Wind Speed	Rising tide	Falling tide
East offshore		
East Nearshore		
West offshore		

West Nearshore		

5.6 Summary

The currents in the bay move predominantly in a westerly and south westerly direction during the rising and falling tides. The currents are generally similar for all three scenarios (slow average and fast wind days.

On slow wind days (1.0m/s) the current speeds are generally below 6cm/s. On average and fast days the current speeds will go up to as much as 9 and 12 cm/s respectively in the bay. The greatest speeds are generally in the central and northern section of Long Bay.

During rising tides, the currents are generally between 4 and 12cm/s to the western limits of the project site for offshore and near shore currents. During the falling tides, the currents are generally faster in the near shore than the offshore and tend to move westerly along the shoreline. The speeds are predicted be as high as 6 to 9cm/s near shore whereas the offshore currents are less than 3cm/s. The wind speeds do not appear to have any significant impact on the currents at the project site.

Sediment dispersion modeling underlines the importance utilizing turbidity barriers at the dredge site as well as the locations onshore where the sedimentation basins will overflow into the sea. The turbidity plumes are expected to extend up to 2km from the points of interest if precautions are not taken to limit sediments getting to the water column. The offshore plumes are expected to remain offshore and meet the NEPA guidelines for distances further than 1km away from the operations. Similarly the near shore plumes will remain in the near shore and are expected to meet the NEPA guidelines for distances further than 1km away from the operations.

6 Planning and Structural Design

6.1.1 Draft Dredge Management Plan (DMP)

6.1.1.1 Introduction

The draft DMP has been prepared to support environmental approvals for the proposed *Palisadoes Shoreline and Protection Rehabilitation Project*. The draft DMP details the proposed dredging work and the measures recommended in managing its potential environmental impacts. The draft DMP specifically addresses:

- The probable dredging methods (capital and maintenance work)
- The quantity and characteristics of material to be dredged, and the use of this material in forming the sand dunes
- The environmental management framework for the proposed dredging work, comprising the environmental management objectives, performance criteria, mitigation measures and reporting and monitoring requirements.

The purpose of this draft DMP is to provide a general framework for planning and implementation of dredging and soil management activities along the Palisadoes. It is prepared at a high level and refers to broad principles and objectives, nominating potential actions and equipment/ plant for adoption.

6.1.1.2 Geotechnical Information

Eight (8) core samples were taken from the borrow area to determine the characteristics of the sand in the area and to determine their suitability for use in this project. The analysis determined that silt and medium to coarse sand is present in the area, and that the most suitable sand is in the vicinity of the CS2, CS3, CS7 and CS8 samples, the results are presented in greater detail in section **Error! Reference source not found. Error! Reference source not found.**.

6.1.1.3 Dredging Method

Dredging will be undertaken using a Trailing Suction Hopper (TSH). This dredger uses trailing suction drag heads to pump fluidized seabed materials to an on-board hopper. Sediments are retained in the hopper, while water used to pump the material is allowed to discharge from the vessel at the dredging site. Dredged material is transported in the hopper to the placement location, in this case, the project area along the Palisadoes. A schematic of a TSH is shown below in Figure 6.1.

A TSH dredge is best suited to:

- Deep water such as in the area of the borrow area.
- The dredging of loose, unconsolidated materials like sand, and this is present in the borrow area based on the coring results.
- The dredging of large volumes of material located a long distance from the placement site. The borrow area is some 600 m – 1,200 m from the nearest buried revetment.
- Dredging under offshore conditions where the dredge must move off-line to allow for the passage of commercial vessels. Commercial vessels do move in the area of the borrow area and the TSH dredge will be able to respond to these changes while operating.

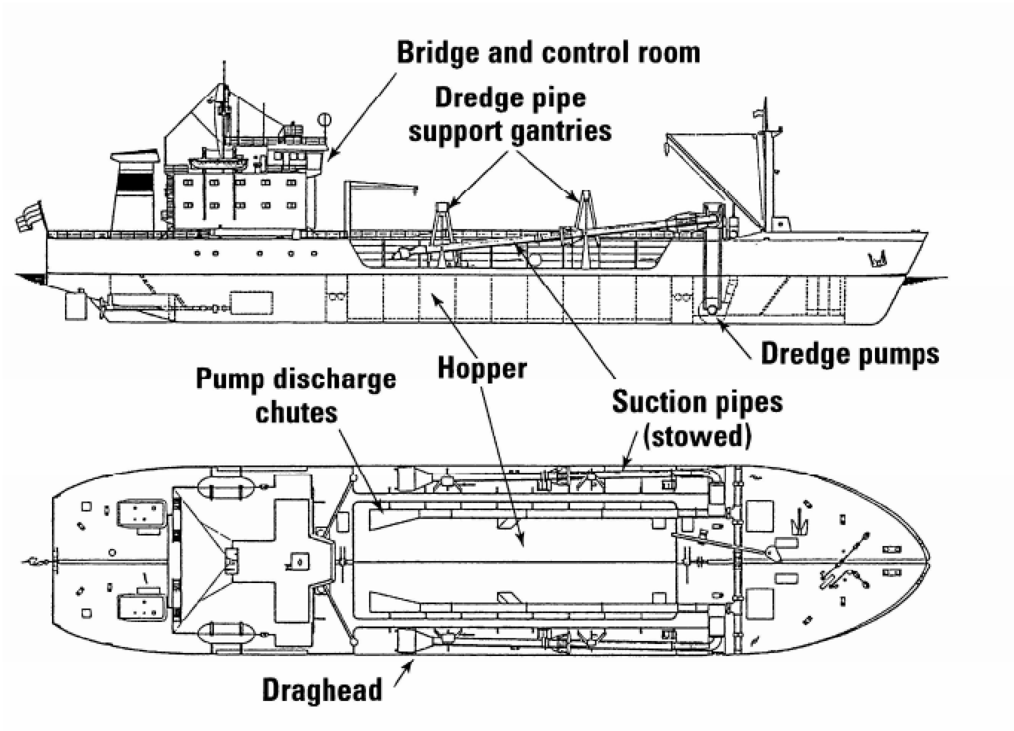


Figure 6.1 Schematic of Trailing Suction Hopper Dredger

The dredging operations likely to be required are summarized in Table 6-1; it should be noted that the optional sand dune adjacent to the NWC WWTP has not been included.

Table 6-1 Summary of dredging operations required for the Palisdaoes Shoreline Protection and Rehabilitation project

Area of Operation	Palisadoes (Caribbean Sea)
Vessel Type	TSH
Dredge elevation	-20 m MSL
Estimated Dredge Volume (m^3)	99,208

6.1.1.4 Dredge Material Placement

The sand dune design presented in section **Error! Reference source not found.Error! Reference source not found. Error! Reference source not found.** requires approximately 99,208 m^3 of sand with a mean grain size ranging between 0.5 – 0.7 mm. This volume and type of material will be dredged from one of the 2 proposed dredge areas identified in the borrow area and placed onshore in a sediment pond to allow the sand to settle. See Figure 6.2. The contractor will then remove this material from the pond and use it to form the sand dunes over the 2 buried revetments. The material will also be used to construct the sand dune between the high revetment and the NWC WWTP once the NWA has agreed to include this option in the project.

6.1.1.5 Environmental Effects

Dredging activities result in a number of impacts on the marine environment. Environmental issues that are relevant for this project include the following:

- Changes to water quality,
- Changes to coastal processes (waves and currents)
- Effects on marine ecology (flora and fauna)
- Mobilisation of sediment and pore water contamination

6.1.2 Mangrove planting area formation Area

Mangrove nourishment locations were chosen based on the best information available, and this comprised of the following:

- Aerial imagery identifying the historic location of mangroves between 1961 and 2004. This information was provided by the National Land Agency (NLA),
- Current survey information along the harbor identifying areas where sand is accreting,
- Alongshore sediment transport model results which determined that sand accretes along the western and central areas of the harbor.

Six (6) planting areas have been identified along the Palisadoes shoreline for mangrove nourishment and they will provide a total planting area of 6,534 m^2, see Figure 6.3.

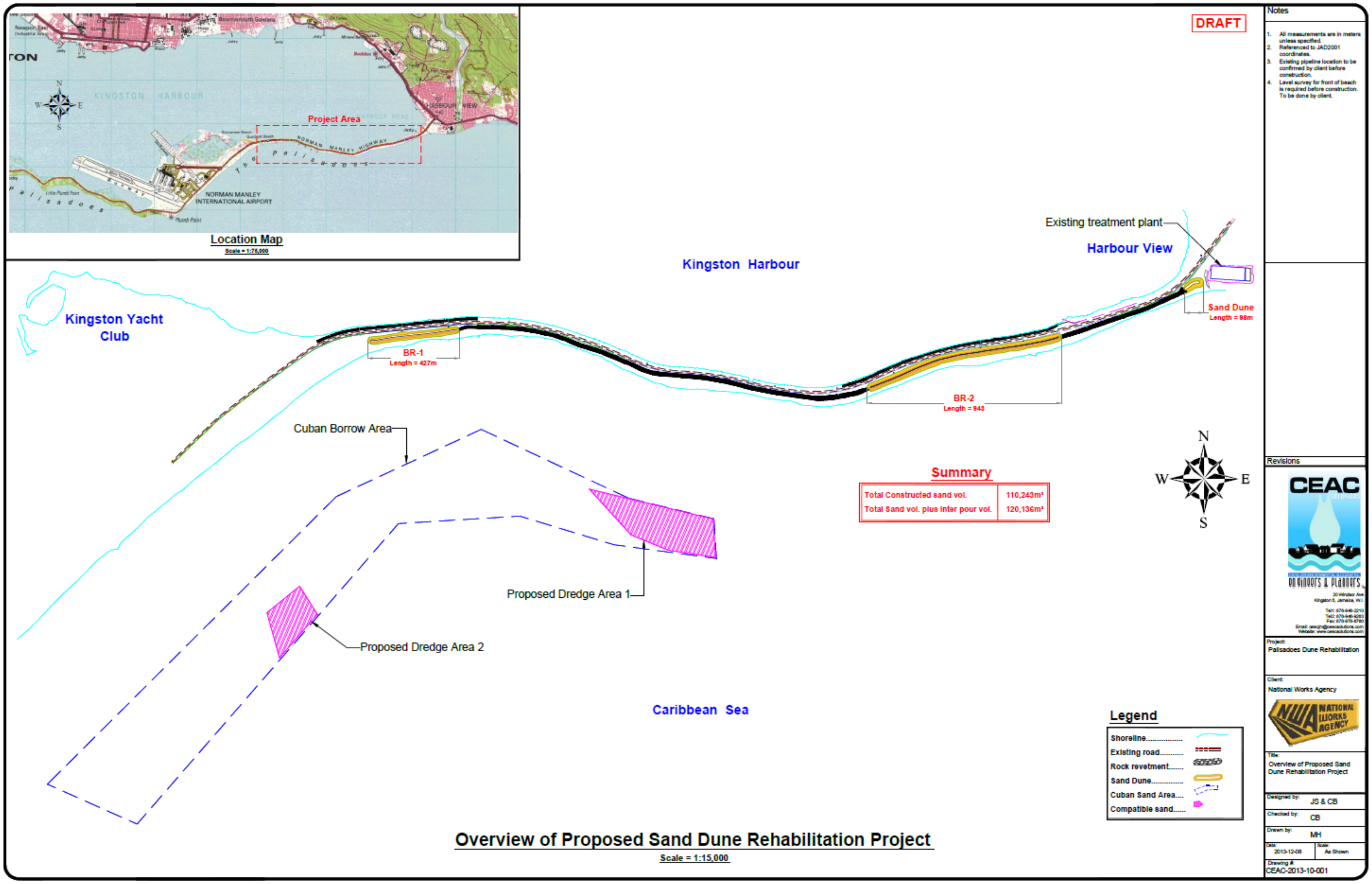


Figure 6.2 Overview of proposed dredging areas for the Palisadoes Shoreline Protection and Rehabilitation Project

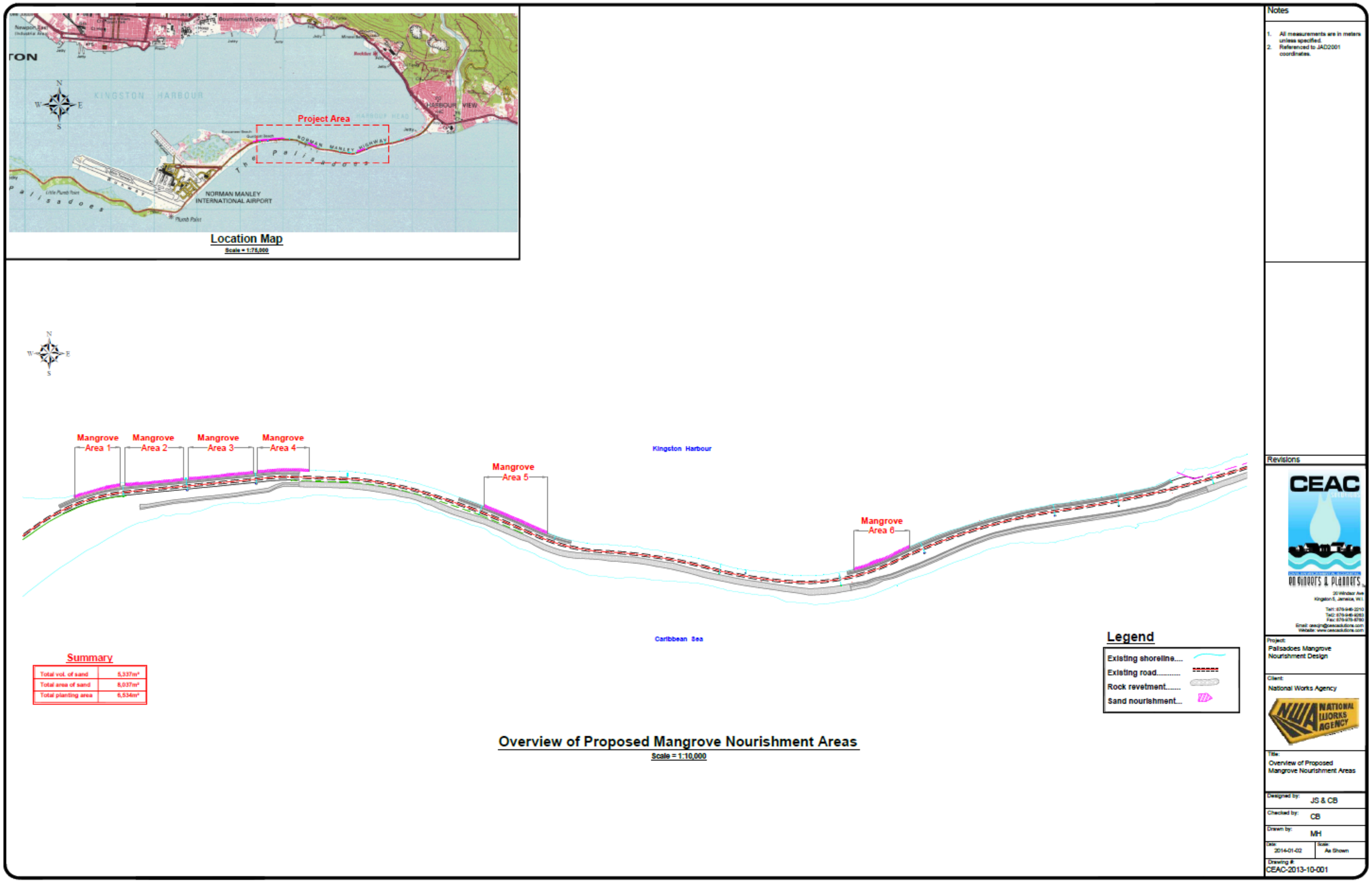


Figure 6.3 Overview of proposed mangrove nourishment areas for the Palisadoes Shoreline Protection and Rehabilitation Project

6.2 Material Verification and Constructability

6.2.1 Sand Dune Nourishment

6.2.1.1 Construction Methodology Study

The Trailing suction hopper dredger will first dredge the sand from the borrow area into the hopper, travel to the sand discharge pipeline that will extend from the dewatering area, across the shoreline and into suitable depths to accommodate the dredge. The sand will be pumped to dewatering basing from the dredge, and it is from this area that sand dune construction will be initiated. The methodology can be broken down in the following stages:

6.2.1.1.1 Dewatering basin Preparation and Dredging

6.2.1.1.1.1 Construction of the Material Storage Area

The storage site will be formed by placing a berm with a 3 m wide crest along the seaward side of the buried revetment so that the sand that is pumped from the dredger can be placed between the buried revetment and the berm. This will be done by the placement contractor using either a bulldozer or excavator using the sand from the beach to form the berm.

6.2.1.1.1.2 Placement of Turbidity Barriers around the Storage Area

Turbidity barriers/curtains 6’ to 8’ deep will be placed offshore the dewatering areas and anchored properly. These will move with the work and damaged sections will have to be replaced in order to maintain water quality requirements.

6.2.1.1.1.3 Dredging and Filling the dewatering Area

The dredger will pump the sand from the borrow area offshore to the storage areas via a flexible hose anchored to the seafloor. This sand will be a part of a slurry mix and so it will be given time to settle in the storage area before the contractor begins to place the sand over the buried revetments. The storage area will also have discharge pipes to remove the water that is a part of the slurry mix.

6.2.1.1.2 Sand Dune Construction

The placement contractor will use a bulldozer tractor or excavator to place the sand over the buried revetments so that the crest width is at an elevation of 6.0 m above MSL, and the landward and seaward slopes are 1: 3.

6.2.1.1.3 Relocation of Storage Areas

Once the sand dunes have been placed over a buried revetment, the placement contractor will level the berms to the surrounding grade. Another dewatering/storage area shall then be placed along the second buried revetment using sand from that area, and the above steps repeated.

6.2.1.1.4 Quality Control Measures

Quality control activities will be required of both the contractor and dredger including sand sampling tests of the dredged material. Environmental specifications will be enforced and will require strict observance of the NWA and NEPA guidelines and conditions.

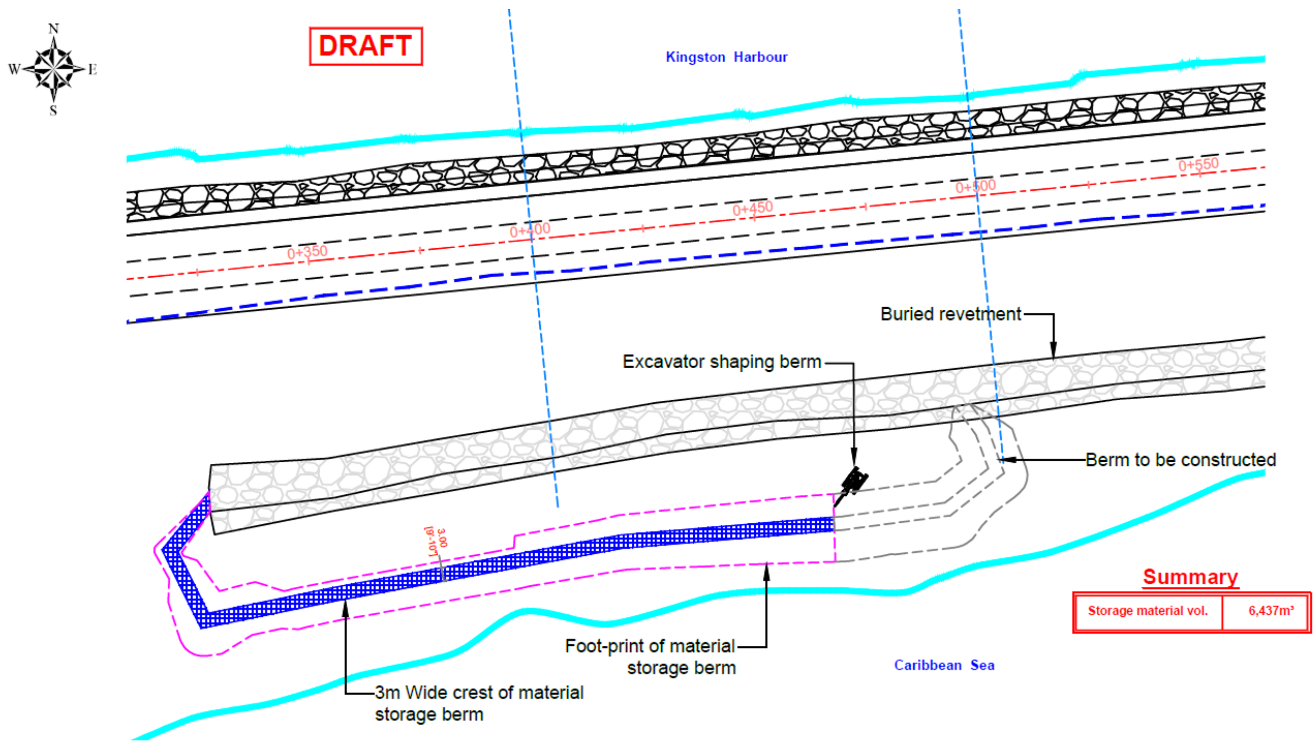
6.2.1.1.5 Equipment Requirements

It is envisaged that the works will be carried out by a team consisting of bulldozers and/ or excavator.

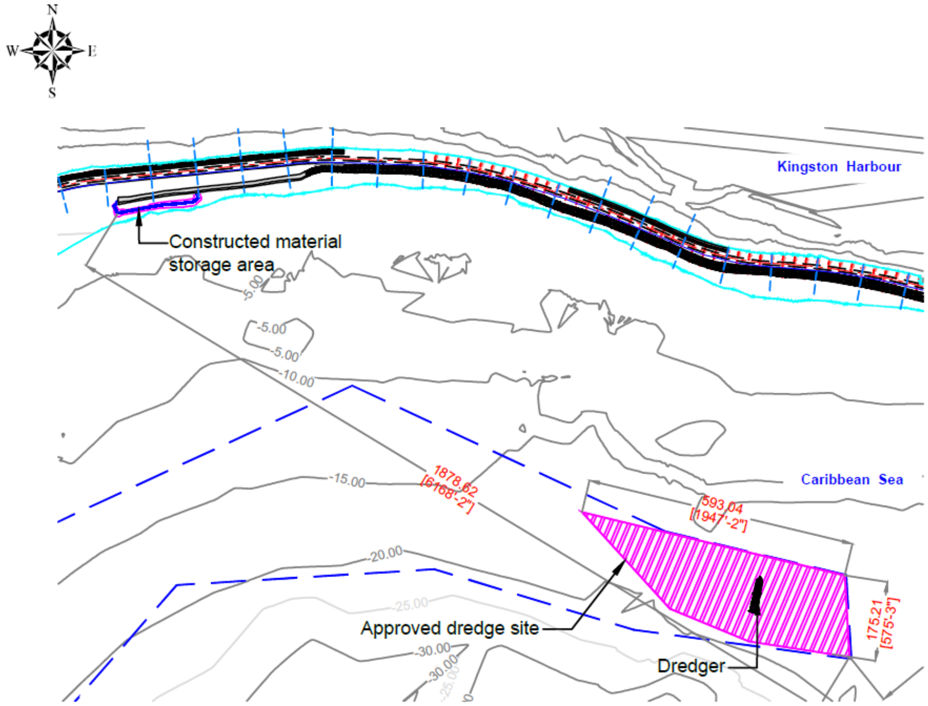
6.2.1.1.6 Options for construction Method

The contractor is at liberty to modify the method and seek approval from the client for variations to the method outline. Another possible method that is envisage include a central dewatering basin at harbour head/Harbour View Roundabout opposite gypsum quarry pier. This area has the required land space. However, trucks will have to transport the material to the dune construction sites.

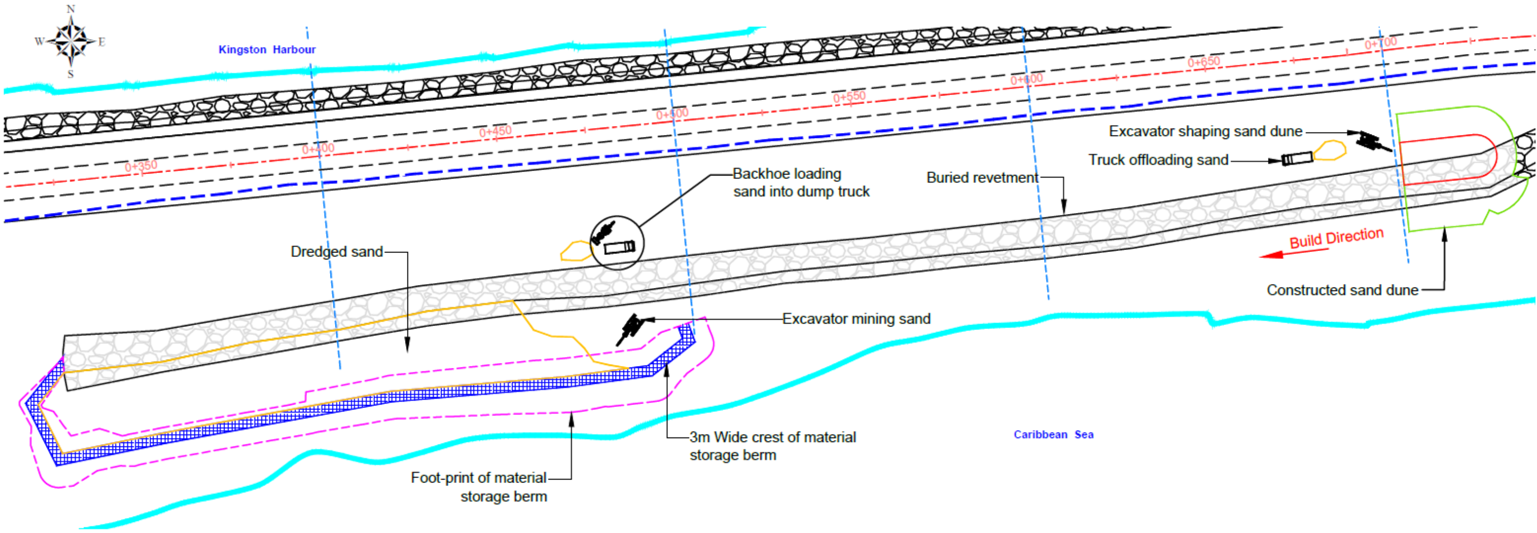
Sheet piling temporary basins is another possible option for quickly creating the basin for dewatering. The contractor will have to pull the sheetpiles after each stretched and re drive in the adjacent location.



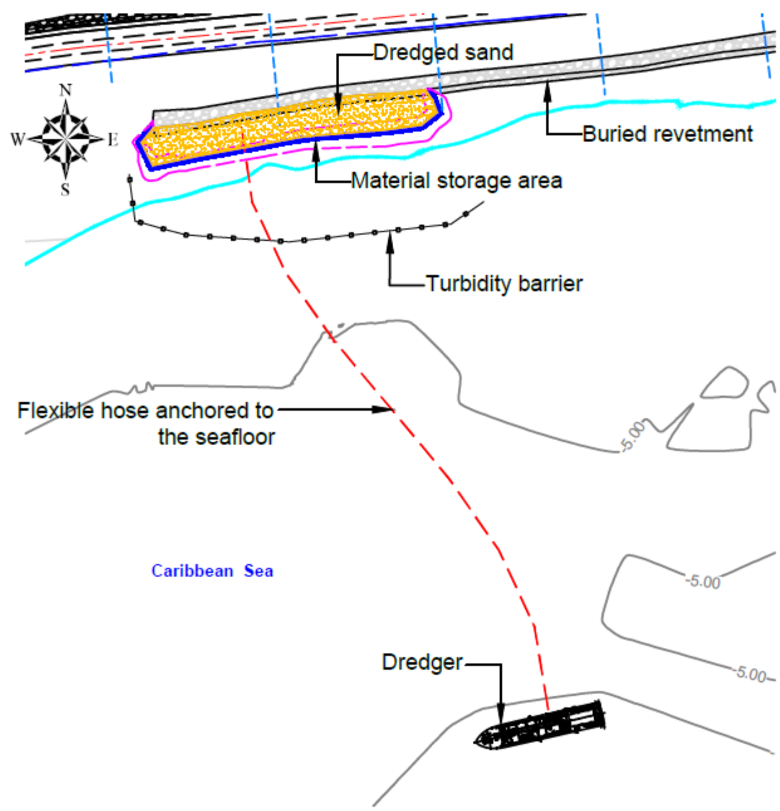
Step 1 - Construction of BR1 Material Storage Site



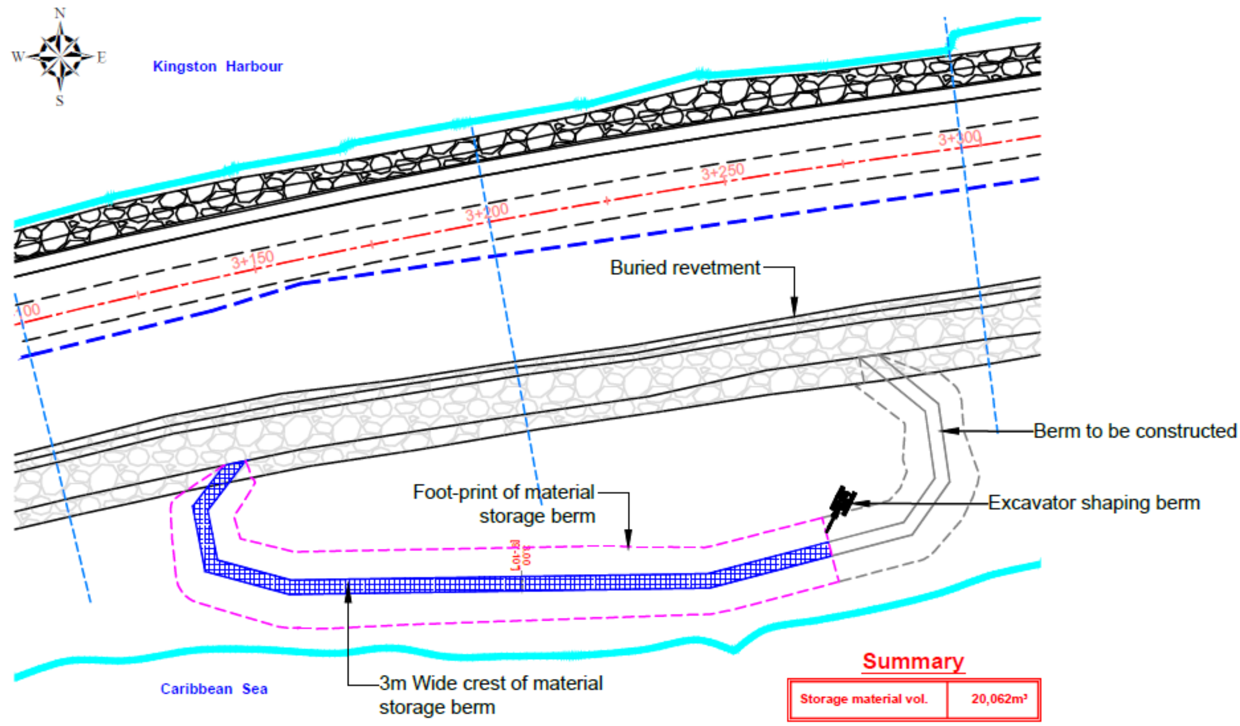
Step 2 - Dredging of Sand



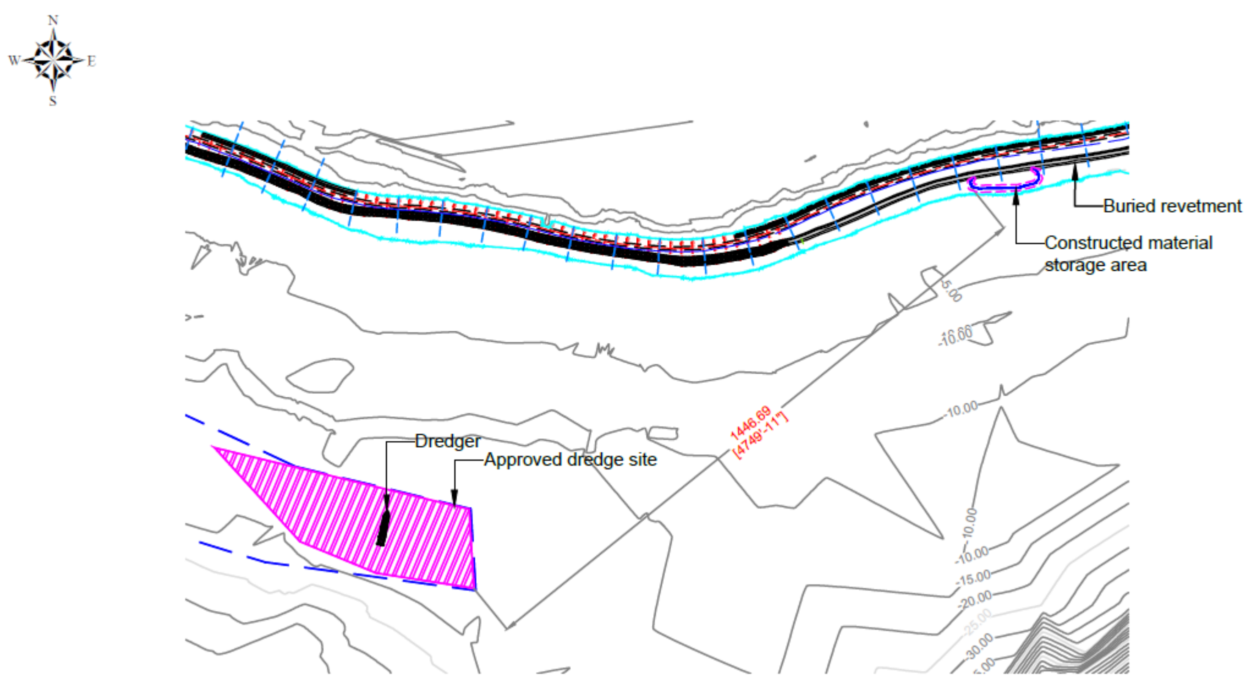
Step 4 - Shaping of Sand Dune



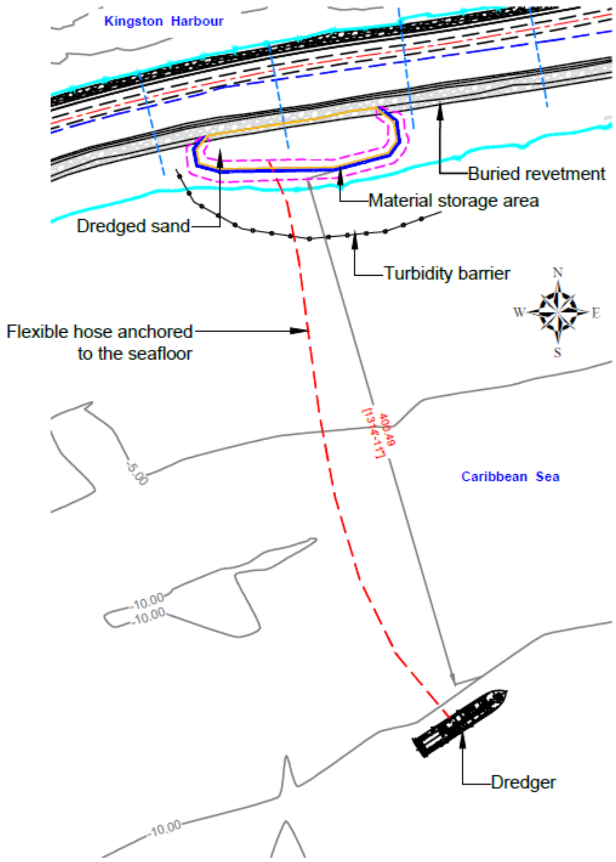
Step 3 - Filling Material Storage Site with Sand



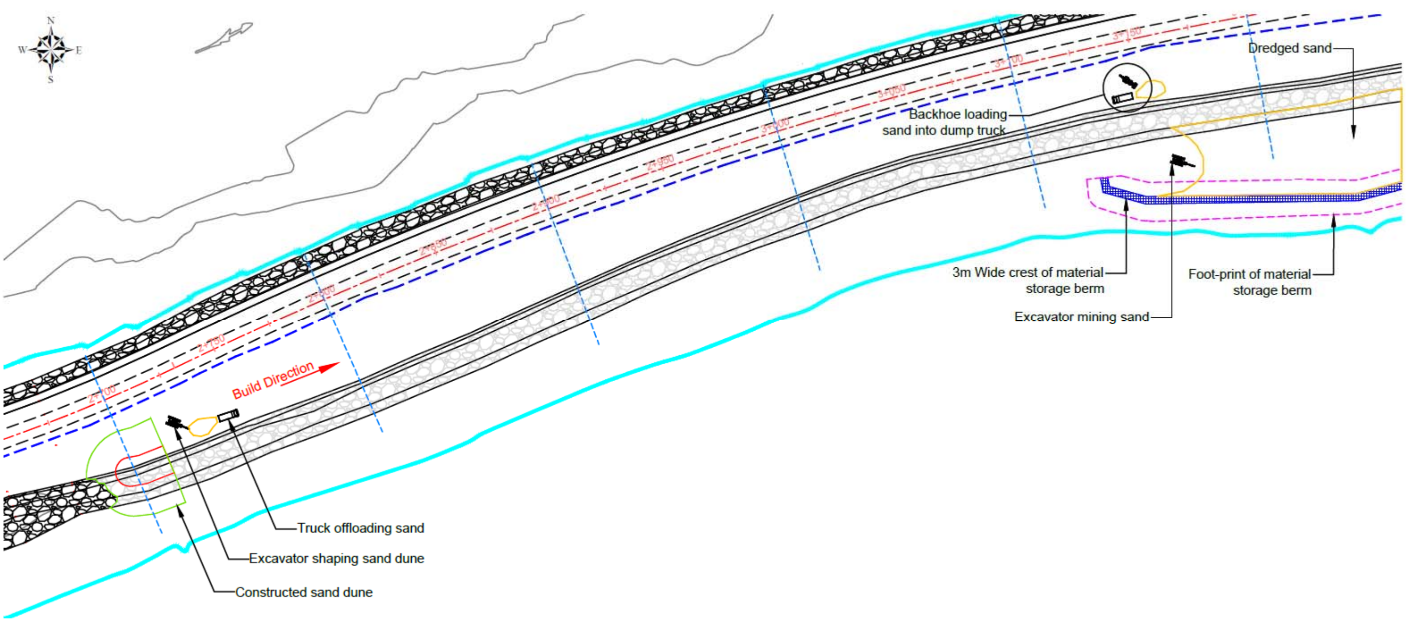
Step 5 - Construction of BR2 Material Storage Site



Step 6 - Dredging of Sand



Step 7 - Filling Material Storage Site with Sand



Step 8 - Shaping of Sand Dune

6.2.2 Mangrove Nourishment

6.2.2.1 Mangrove Planting Areas Quarry Survey and Assessment

Quarry surveys were undertaken at 3 sand mining operations in St. Thomas and 2 desilting operations in Kingston to determine which source would provide a suitable source of sand for use in the mangrove nourishment exercise. Sand samples were taken from each operation and analysed. The Hope River desilting operation's un-sieved sand proved to be the most suitable sand for the project, see (CEAC Solutions Co. Ltd.).

6.2.2.2 Construction Methodology

Sand from the Hope River desilting operation will be placed along the harbour side of the Palisadoes for the mangrove nourishment activity. The methodology can be broken down in the following stages:

6.2.2.2.1 Site Preparation

6.2.2.2.1.1 Installation of Protective Metal Sheeting and Delivery of Material

To protect the boardwalk along the harbour side of the Palisadoes during the sand placement exercise, a 3 x 10 m metal sheet will be installed over the boardwalk before the works begin. Sand will then be trucked from the Hope River desilting operation and placed alongside the protective metal sheeting.

6.2.2.2.2 Placement and Shaping of Sand

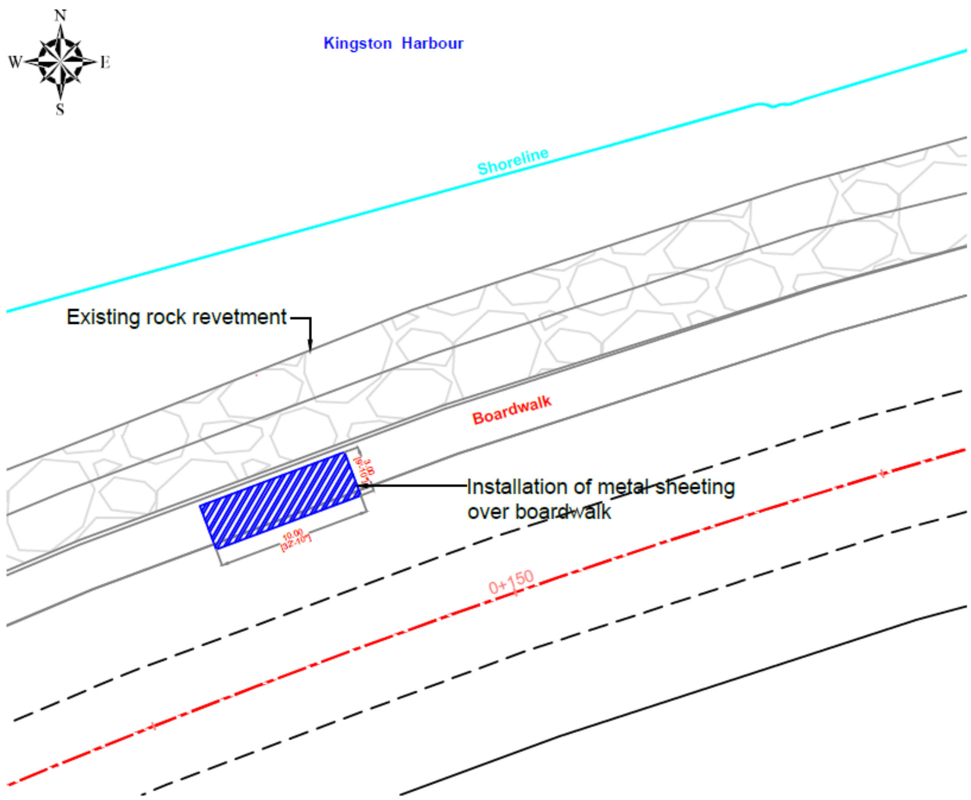
A backhoe, or a suitable alternative, with a cleaning bucket, will work from the metal sheeting where it will place the sand over the revetment along the harbour side. Due care will be taken to avoid th electrical wires. Construction workers will then shape the sand over the revetment so that it has a back of beach elevation of 1.0 m and a seaward slope to MSL of 1: 10.

6.2.2.2.3 Quality Control Measures

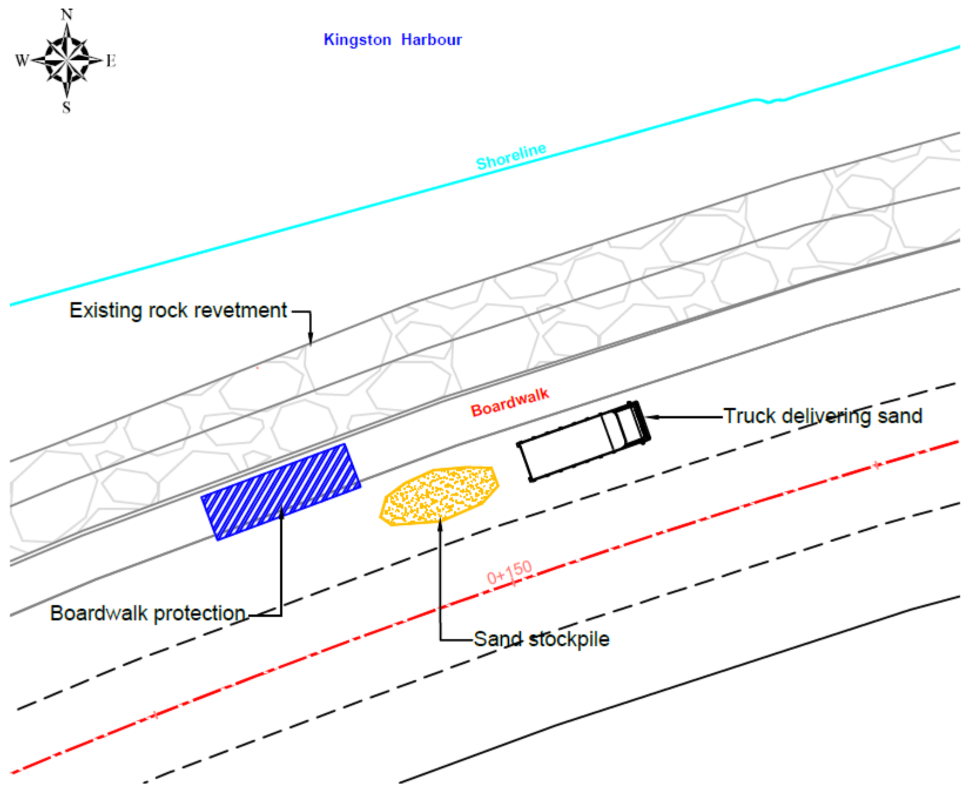
Quality control activities will be required of both the contractor and dredger including sand sampling tests of the dredged material. Environmental specifications will be enforced and will require strict observance of the NWA and NEPA guidelines and conditions.

6.2.2.2.4 Equipment Requirements

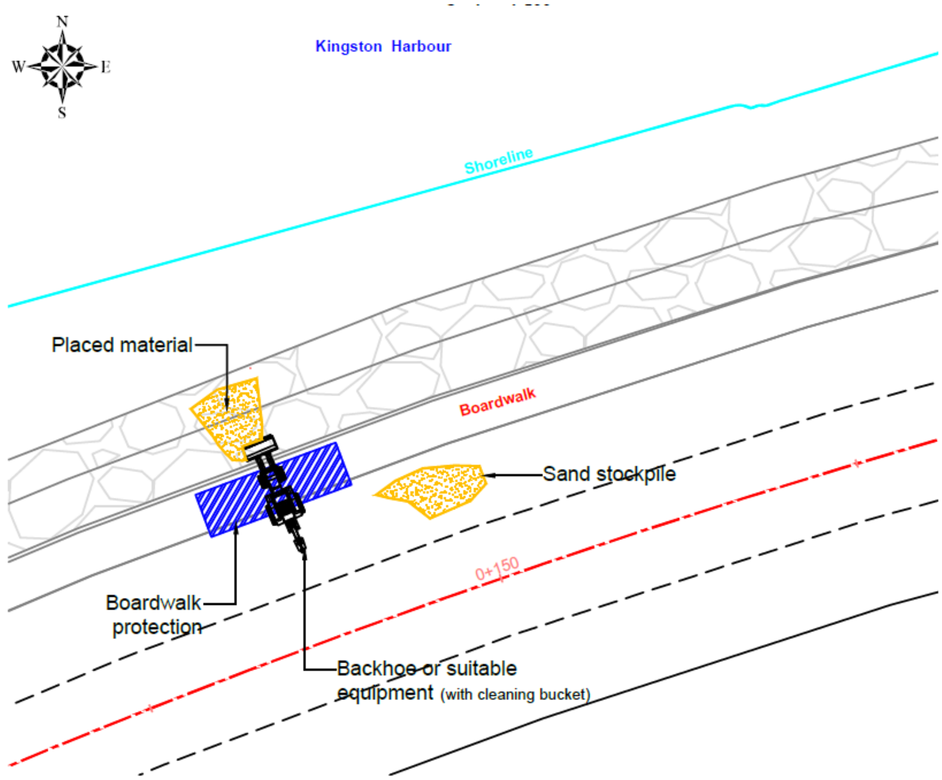
It is envisaged that the works will be carried out by a team consisting of a backhoe and/ or excavator. Once enough sand is placed in a section the metal sheeting and trucked sand is moved to another area along the harbour for sand placement.



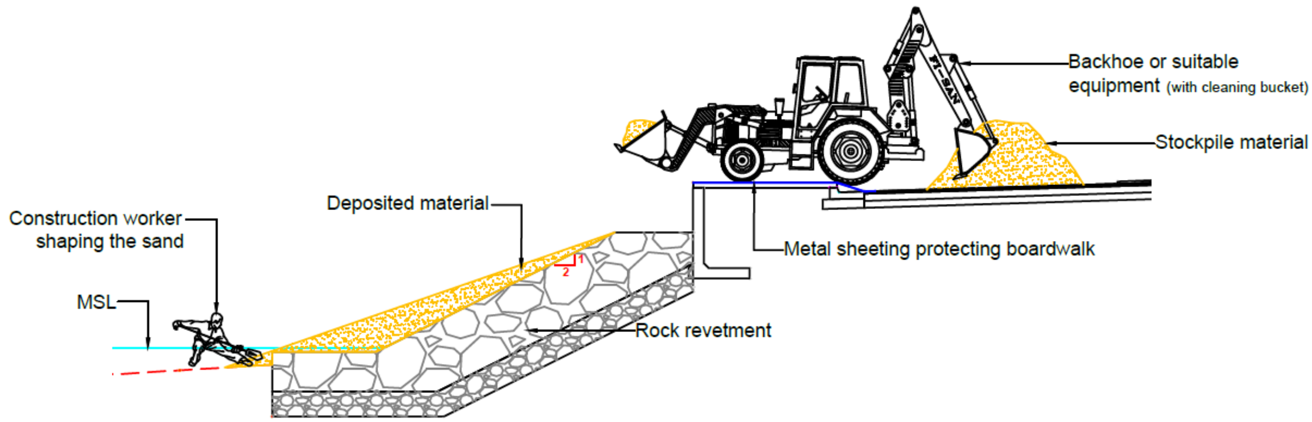
Step 1 Layout - Installation of Protective Metal Sheeting



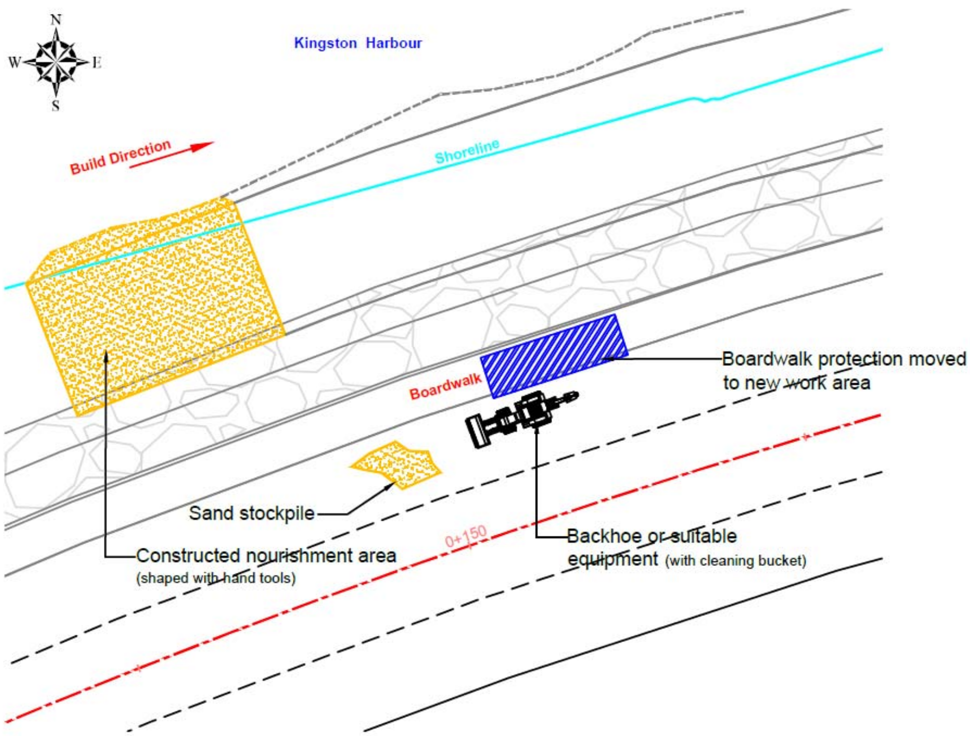
Step 2 Layout - Delivery Of Material



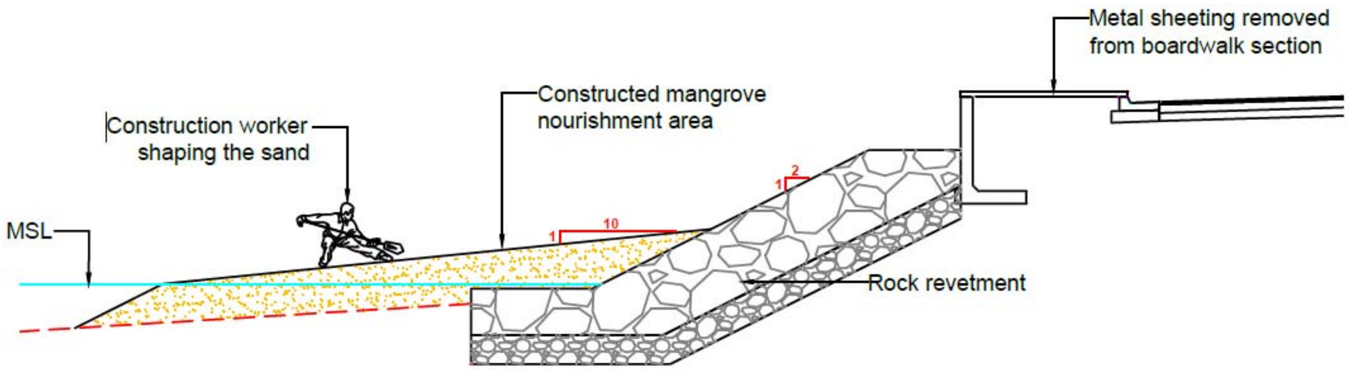
Step 3 Layout - Placing Material On Revetment



Step 3 - Section



Step 4 Layout - Shaping of Sand Material



Step 4 - Section

7 Engineering Cost Estimate

7.1 Summary of Costs

Procurement is envisaged in two parts, namely: dredging and placement of sand along the Palisadoes (dune nourishment) and the supply and placement of mangrove nourishment along the harbour side. The dredging contract is expected to involve a dredging contractor with the requisite skills and equipment, while the mangrove nourishment contractor is expected to engage local sources of material working under a main contractor. The engineers estimate for the project is US$4,223,154.10, made up as follows:

- Dredging and Placement of Sand for Dune Nourishment: US$3,971,220
- Supply and Placement of Mangrove Nourishment: US$251,934.10

It is expected that in the internal project team meetings that various components of the costs will be discussed and prioritized in order to arrive at an agreed approach in the tender document.

7.1.1 Dredging and Placement of Sand for Dune Nourishment

An international dredging contractor is expected to supply the material needed for the dune nourishment activity; this consists of dredging the designated borrow areas 1 and 2 to achieve the required volume for land reclamation of dunes 1 and 2. This material will be deposited in thin layers over the buried revetments with sufficient intervals between successive increases in the depth of fill to ensure that the underlying soil does not fail. Each layer shall be compacted and maintained at all times with a sufficiently even surface in order to drain away the surface water. Quality control measures include:

- Protection of all vegetation and/ or property within limits of disturbance.
- Periodic inspections and/ or verification by the Engineer during and after the dredging work.
- Testing of the dredged material at regular intervals to determined its uniformity/ conformity with the source samples. Any discrepancies discovered with sediment characterization shall be immediately brought to the attention of the engineer.

7.1.2 Supply and Placement of Sand for Mangrove Nourishment

This section of the project is aimed at providing sufficient soft coastal protection for the Palisadoes and to rehabilitate the coastal ecosystem through mangrove re-vegetation. Four (4) mangrove replanting areas along the Kingston Harbour will be formed to create over 6,000 m^2 of replanting area. The filling operation shall be done by mechanical placement in the nourishment areas and shall follow the recommended engineering and EIA guidelines.

Quality control measures include the regular sampling and testing of the fill material to determine its uniformity/ conformity with the source samples.

7.1.3 Summary

The draft tender document has requirements the contractor is expected to meet, they are also to progressively present several submittal requirements within the specifications. These include work plans, safety, environmental, surveying and material quality plans and reporting. These have been estimated within the traditional 10% margin of the main project costs.

The total anticipated cost estimate is **US$4,223,154.10** for the dredging and placement of sand for dune nourishment and the supply and placement of sand for mangrove nourishment. This cost also includes contingencies and preliminaries.

8 Conclusions and Recommendations

8.1 Conclusions

Based on the data collected and numerous analyses conducted to date, the following conclusions can be drawn:

1. Dune along the Palisadoes that have survived hurricane Ivan range in height from 4.4 to 9.08 meters, with those between 7.7 to 9.08 meters in elevation having very little evidence of overtopping. The presence of vegetation on the crest of the dunes appears to have stabilized the higher dunes as well as other factors.
2. A water quality monitoring programme was undertaken on November 19, 2013 at a total of six (6) nearshore and offshore stations and the results were compared to 2009 Draft Marine Standards. All results fell below the limits outlined. Measures will therefore have to be taken to secure the pristine water quality profile of the outer shelf of the Kingston Harbour.
3. Climate change studies have shown sea levels are rising at a rate of 3.7mm/year. Additionally, operational waves are expected to decrease by 1 to 2 percent in the next 50 to 100 years whereas hurricane wave heights are expected to increase by 1.04 percent in the next 50 to 100 years. Analysis of hurricanes passing the site since 1852 indicates the project site is becoming more vulnerable to hurricanes due to the increasing numbers and frequency of more intense hurricanes (specifically categories 4 and 5) which have tracked within 300km of site. Due consider therefore have to be given to these trends in the design and maintenance of the dunes.
4. The wave refraction analysis clearly indicates that the project shoreline along the Caribbean Sea is most vulnerable to hurricane waves from the south and southeast. In both scenarios, 7 to 8 m waves are expected some 2.5km offshore and 2 to 4 m waves are expected at the shoreline. For the post project scenario the changes in the bathymetry caused by the dredging operations did not affect the wave heights reaching the shoreline. Along the harbour side of the project the shoreline is most vulnerable to storm surge waves from the north and northwest, particularly the central and western portion of the shoreline, and 2.5 to 3 meter waves are expected some 1 km offshore and 1.5 to 3 m waves are expected at the shoreline. When climate change is considered, there will be a marginal increase in wave heights for the pre and post construction scenario.
5. Currents in the project are driven predominantly by tides with the general movements being from east to west. Current speeds vary from 0.4cm/s to a high of 12cm/s in the near shore areas whereas the offshore areas (in the vicinity of the dredge sites) tend to have a speeds of less than 4cm/s.
6. Sediment dispersion modeling indicate turbidity plumes that can be generated from the operations will be above the NEPA standards. The turbidity plumes are expected to extend up to 2km from the points of operation if precautions are not taken to limit sediments getting to the water column. The offshore plumes are expected to remain offshore and meet the NEPA guidelines for distances further than 1km away from the operations. Similarly the near shore

plumes will remain in the near shore and are expected to meet the NEPA guidelines for distances further than 1km away from the operations.

7. Sand dunes are required over the 2 buried revetments along the Palisadoes to protect the Palisadoes road from possible damage caused by a 50 and 100 year return period hurricane wave events. These dunes should require a crest elevation of 6.24 m, a crest width of 12.0 m and a seaward and landward slope of 1:3. Vegetation is anticipated in order to stabilize the dunes further from wind driven and overtopping events. Whilst some overtopping and movement is expected, the material is not expected to be deposit on the roadway in significant quantities.
8. Cross shore modeling of the proposed mangrove nourishment sites indicate that the fill can be expected to reshape due to swell events. This reshaping should become more stable with time as the mangroves also continue to reinforce the substrate.

8.2 Recommendations

The following recommendations should be considered based on the analysis conducted to date:

1. A larger borrow area was initially defined by the Cuban study to provide the sand needed for the dunes. The areas that would provide sand that is suitable for the project's needs is however much smaller. It is recommended that the material be dredged from the designated borrow area sites for constructing the dune after further verification. A total of 99,208 m^3 of this material is required for construction with a mean grain size varying between 0.5 and 0.7mm, and sediment characteristics similar to that identified earlier in this report.
2. For the mangrove nourishment activity it is recommended that sand be placed along the harbour side of the Palisadoes in the western and central sections of the project as these areas are currently experiencing accretion. The sand placed should have a back of beach elevation of 1.0 m, a seaward slope of 1: 10 to MSL, and a 1: 2 slope from MSL to the existing grade to provide the 6,000 m^2 of sand required to re-plant the mangroves that were previously lost during hurricane Ivan storm event. It is recommended that un-sieved sand from the Hope River desilting operation be used for the mangrove nourishment exercise as it is the most suitable sand based on our analysis and has a mean grain size varying between 0.8 and 4.0 mm

3. A Construction and Environmental Monitoring Programme is recommended for the duration of the construction period. These programmes should consist of:
   - Use of appropriate dredging equipment.
   - Frequent measurements of water turbidity at the active dredging areas, and at two sampling locations to be decided by the NEPA.
   - Material and workmanship monitoring to ensure compliance with engineering specifications and drawings.
   - Environmental monitoring of water quality, dust and noise to ensure that reasonable local and relevant international guidelines are being followed and met.
4. The following mitigation measures should be implemented during construction:

- Schedule dredging operations with consultation with the Port Authority of Jamaica (PAJ) so as to avoid or minimize the disruption of marine traffic.
- Advise local residents and users of the Palisadoes roadway prior to commencement of the intended dredging and construction operations along both sides of the Palisadoes.
- Turbidity barriers/screens should be utilized to minimize the impact of plumes from construction materials in the marine environment.

5. The works should be constructed as per the attached specifications and drawings

9 Works Cited

CEAC Solutions Co. Ltd. *Dune Investigation*. Kingston: National Works Agency, 2013.

—. *Inception Report*. Kingston: National Works Agency, 2013.

—. *Material Source Assessment Report*. Kingston: National Works Agency, 2013.

CIRIA. *The rock manual. The use of rock in hydraulic engineering (2nd Edition), Report C683*. London: CIRIA, CUR and CETMEF., 2007.

Climate Studies Group, Mona, The University of the West Indies. *Evaluation of trends in sea levels, ocean wave characteristics and tropical storm intensities*. Kingston, 2013.

Dennison, William C, et al. "Assessing water quality with submersed aquatic vegetation." *BioScience* (1993): 43, no. 2: 86-94.

Devlin, Michelle and Britta Schaffelke. "Spatial extent of riverine flood plumes and exposure of marine ecosystems in the Tully coastal region, Great Barrier Reef." *Marine and Freshwater Research* (2009): 60, no. 11: 1109-1122.

Emanuel, Kerry, Ragoth Sundararajan and John Williams. "Hurricanes and global warming: Results from downscaling IPCC AR4 simulations." *Bulletin of the American Meteorological Society* (2008): 89, no. 3: 347-367.

Gallegos, C. L. and W. J. Kenworthy. "Seagrass depth limits in the Indian River Lagoon (Florida, USA): Application of an optical water quality model." *Estuarine, coastal and shelf science* (1996): 42, no. 3: 267-288.

Juanes, Pérez,Izquierdo,Caballero,Rivero. "Palisadoes Protection and Rehabilitation Project." 2007.

King, Ian and John DeGeorge. "Multi Dimensional Modeling of Water Quality Using the Finite Element Method." *ASCE* (1995): pp. 340-354.

King, Ian and Richard Rachiele. "Multi-dimensional modeling of hydrodynamics and salinity in San Francisco Bay." *In Estuarine and Coastal Modeling*. 1989. pp. 511-521. ASCE.

Knutson, Thomas R., et al. ""Dynamical downscaling projections of 21st century Atlantic hurricane activity: CMIP3 and CMIP5 model-based scenarios." *J Clim* (2013).

Meer, Van der and Jentsje W. *Conceptual design of rubble mound breakwaters*. Delft, Netherlands: Delft Hydraulics, 1993.

Murakami, Tomokazu, Jun Yoshino, and Takashi Yasuda. "PREDICTION OF MAXIMUM POSSIBLE STORM SURGES IN ISE BAY UNDER A FUTURE CLIMATE." *Coastal Engineering Proceedings,* . 2012. no. 33: currents-47.

Natural Resources Conservation Authority. "Coral Reef Protection and Preservation Policy and Regulations." 1997.

Robinson, E and Rowe, D. "The Island of the Palisadoes?" *MGU-WP/0001/ 040105* (2005).

Roeckner, Erich, et al. ". "Historical and future anthropogenic emission pathways derived from coupled climate–carbon cycle simulations." *Climatic Change* (2011): 105, no. 1-2: 91-108.

Ruggiero, Peter, Paul D. Komar, and Jonathan C. Allan. "Increasing wave heights and extreme value projections: The wave climate of the US Pacific Northwest." *Coastal Engineering* (2010): 57, no. 5: 539-552.

Wang, Xiaolan L., and Val R. Swail. "Climate change signal and uncertainty in projections of ocean wave heights." *Climate Dynamics* (2006): 26, no. 2-3: 109-126.

Wigley, Tom ML. "The effect of changing climate on the frequency of absolute extreme events." *Climatic Change* (2009): 97, no. 1-2: 67-76.

10 Appendices

10.1 Drogues Results

Rising Tide Drogue Session - Conducted October 31th, 2013															
Drogue #	GPS WP #	Time (am)	Date	Depth of Sail	Notes	Easting	Northing	Location	Distance Travelled (m)	Time (s)	Speed (cm/s)	Direction of Motion	Average Speed (cm/s)	Average Direction of Motion	
2A	34	6:51	31-Oct	2	deploy	312131	1983213	NEARSHORE	123.810	1593	7.772	216.129	10.301	226.296	South Westerly
2A	40	7:17	31-Oct		measurement	312058	1983113		140.071	1279	10.952	223.264			
2A	47	7:38	31-Oct		measurement	311962	1983011		146.349	1316	11.121	230.545			
2A	52	8:00	31-Oct		measurement	311849	1982918		110.422	972	11.360	231.988			
2A	57	8:17	31-Oct		remove	311762	1982850								
8	35	6:52	31-Oct	surface	deploy	312134	1983215		123.004	1547	7.951	216.404	10.245	223.449	South Westerly
8	41	7:17	31-Oct		measurement	312061	1983116		134.015	1255	10.678	217.725			
8	46	7:38	31-Oct		measurement	311979	1983010		145.097	1358	10.685	227.514			
8	53	8:01	31-Oct		measurement	311872	1982912		102.176	876	11.664	229.764			
8	56	8:15	31-Oct		remove	311794	1982846								
1	32	6:48	31-Oct	3	deploy	311802	1982943	PLUM POINT	145.413	1632	8.910	241.673	6.201	250.792	Westerly
1	39	7:15	31-Oct		deploy	311674	1982874		98.955	1591	6.220	255.964			
1	48	7:41	31-Oct		measurement	311578	1982850		56.080	1003	5.591	238.861			
1	51	7:58	31-Oct		measurement	311530	1982821		85.053	1252	6.793	267.979			
1	58	8:19	31-Oct		remove	311445	1982818								
5	33	6:48	31-Oct	surface	deploy	311806	1982943		156.541	1584	9.883	242.616	6.242	266.243	Westerly
5	38	7:14	31-Oct		deploy	311667	1982871		131.187	1651	7.946	266.941			
5	49	7:42	31-Oct		measurement	311536	1982864		55.027	903	6.094	289.093			
5	50	7:57	31-Oct		measurement	311484	1982882		66.219	1413	4.686	284.876			

5	59	8:20	31-Oct		remove	311420	1982899								
11	36	6:56	31-Oct	surface	deploy	312317	1982467	ADCP 1	74.626	1610	4.635	147.588	4.833	154.612	South Easterly
11	43	7:23	31-Oct		measurement	312357	1982404		25.495	584	4.366	154.440			
11	44	7:33	31-Oct		measurement	312368	1982381		120.814	2197	5.499	160.665			
11	55	8:09	31-Oct		remove	312408	1982267								
6B	37	6:58	31-Oct	8	deploy	312320	1982471		46.615	1437	3.244	125.395	3.180	144.272	South Easterly
6B	42	7:22	31-Oct		measurement	312358	1982444		19.235	688	2.796	152.103			
6B	45	7:34	31-Oct		measurement	312367	1982427		69.354	1981	3.501	155.283			
6B	54	8:07	31-Oct		remove	312396	1982364								

Falling Tide Drogue Session - Conducted October 31, 2013

Drogue #	GPS WP #	Time (pm)	Date	Depth of Sail	Notes	Easting	Northing	Location	Distance Travelled (m)	Time (s)	Speed (cm/s)	Direction of Motion	Average Speed (cm/s)	Average Direction of Motion	
2A	68	10:19	31-Oct	2	deploy	311997	1983141	NEARSHORE	20.591	1556	1.323	299.055	1.743	339.330	North Westerly
2A	74	10:45	31-Oct		measurement	311979	1983151		68.072	3147	2.163	2.627			
2A	80	11:37	31-Oct		remove	311982.12	1983219								
8	75	10:45	31-Oct	surface	deploy	312003	1983188		157.407	3223	4.884	7.667	4.884	7.667	Northerly
8	81	11:39	31-Oct		remove	312024	1983344								
1	66	10:15	31-Oct	3	deploy	311305	1982835	PLUM POINT	13.454	1565	0.860	41.987	1.481	39.785	North Easterly
1	72	10:41	31-Oct		measurement	311314	1982845		79.649	3788	2.103	38.884			
1	83	11:44	31-Oct		remove	311364	1982907								

5	67	10:15	31-Oct		deploy	311309	1982842		10.770	1528	0.705	68.199	2.291	54.490	North Easterly
5	73	10:41	31-Oct		measurement	311319	1982846		144.627	3731	3.876	52.021			
5	82	11:43	31-Oct	surface	remove	311433	1982935								
11	65	10:10	31-Oct		deploy	312394	1982602		29.017	778	3.730	181.975	2.775	175.114	Southerly
11	70	10:22	31-Oct		measurement	312393	1982573		48.052	1597	3.009	167.989			
11	77	10:49	31-Oct		measurement	312403	1982526		38.328	2414	1.588	172.504			
11	78	11:29	31-Oct	surface	remove	312408	1982488	ADCP 1							
6B	64	10:09	31-Oct		deploy	312400	1982596		24.331	811	3.000	189.462	2.060	162.064	Southerly
6B	71	10:23	31-Oct		measurement	312396	1982572		24.166	1568	1.541	155.556			
6B	76	10:49	31-Oct		measurement	312406	1982550		40.719	2485	1.639	114.677			
6B	79	11:30	31-Oct	8	remove	312443	1982533								

Rising Tide Drogue Session - Conducted November 15th, 2013															
Drogue #	**GPS WP #**	**Time** (pm)	**Date**	**Depth of Sail**	**Notes**	**Easting**	**Northing**	**Location**	**Distance Travelled** (m)	**Time** (s)	**Speed** (cm/s)	**Direction of Motion**	**Average Speed** (cm/s)	**Average Direction of Motion**	
2A	101	8:22	15-Nov	2	deploy	314804	1984482		78.230	4372	1.789	327.529	1.789	327.529	North Westerly
2A	112	9:35	15-Nov		remove	314762	1984548	NEARSHORE							
9	100	8:22	15-Nov	SURFACE	deploy	314793	1984483		117.992	4365	2.703	328.870	2.703	328.870	North Westerly

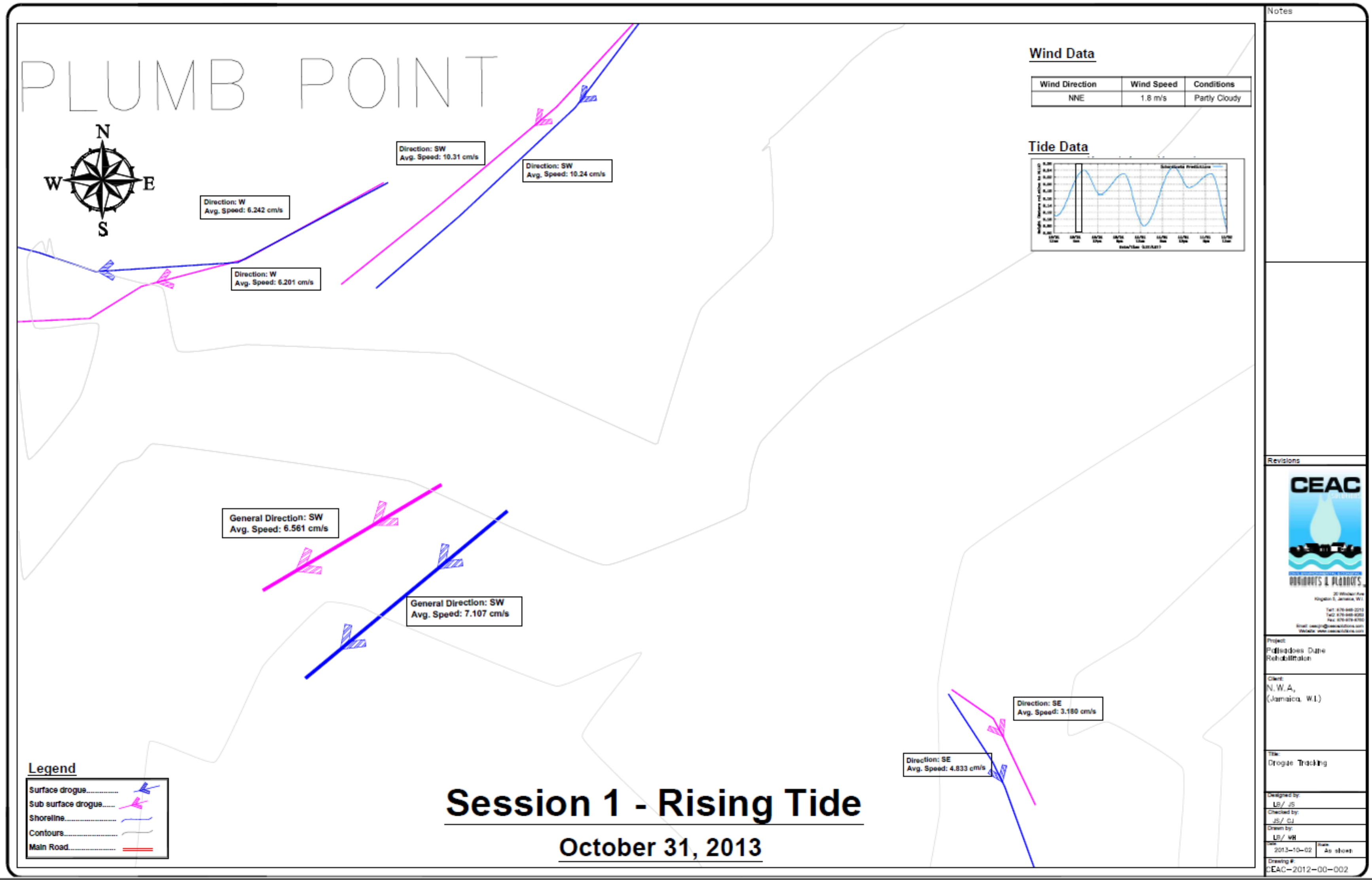
9	111	9:34	15-Nov		remove	314732	1984584								
6B	102	8:30	15-Nov	8	deploy	314938	1983889	ADCP 2	93.963	3609	2.604	253.301	2.604	253.301	Westerly
6B	109	9:30	15-Nov		remove	314848	1983862								
8	103	8:31	15-Nov	SURFACE	deploy	314938	1983890		192.855	3632	5.310	257.117	5.310	257.117	Westerly
8	110	9:31	15-Nov		remove	314750	1983847								
5	104	8:40	15-Nov	10	deploy	315115	1983268	OFFSHORE	56.045	2772	2.022	344.476	2.022	344.476	North Westerly
5	107	9:26	15-Nov		remove	315100	1983322								
1	105	8:41	15-Nov	2	deploy	315112	1983269		66.573	2778	2.396	327.265	2.396	327.265	North Westerly
1	108	9:27	15-Nov		remove	315076	1983325								

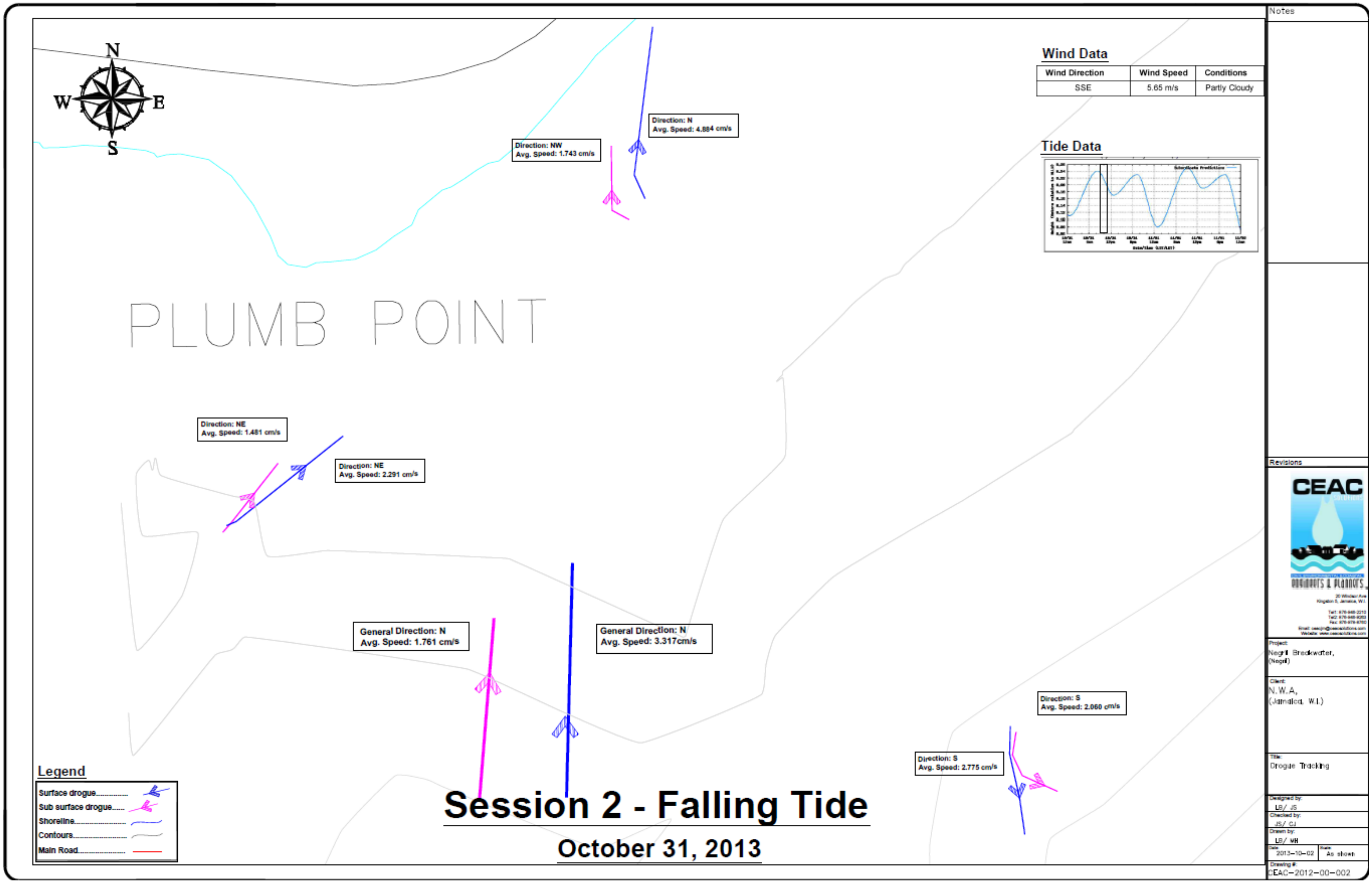
Falling Tide Drogue Session - Conducted November 15th, 2013

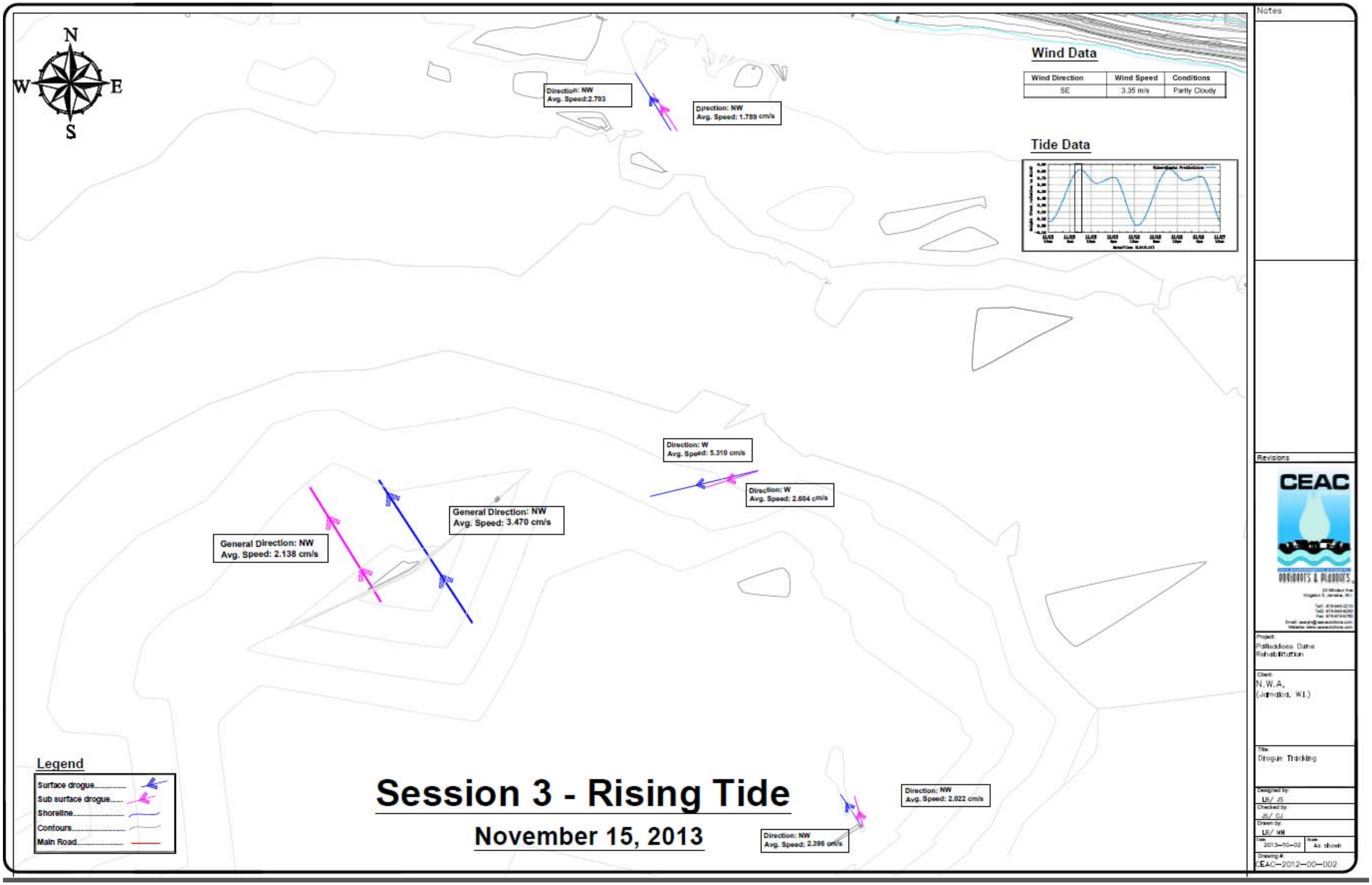
Drogue #	GPS WP #	Time (pm)	Date	Depth of Sail	Notes	Easting	Northing	Location	Distance Travelled (m)	Time (s)	Speed (cm/s)	Direction of Motion	Average Speed (cm/s)	Average Direction of Motion	
2A	134	10:51	15-Nov	2	deploy	314535	1984486	NEARSHORE	122.674	2261	5.426	299.281	5.426	299.281	North Westerly
2A	145	11:29	15-Nov		remove	314428	1984546								

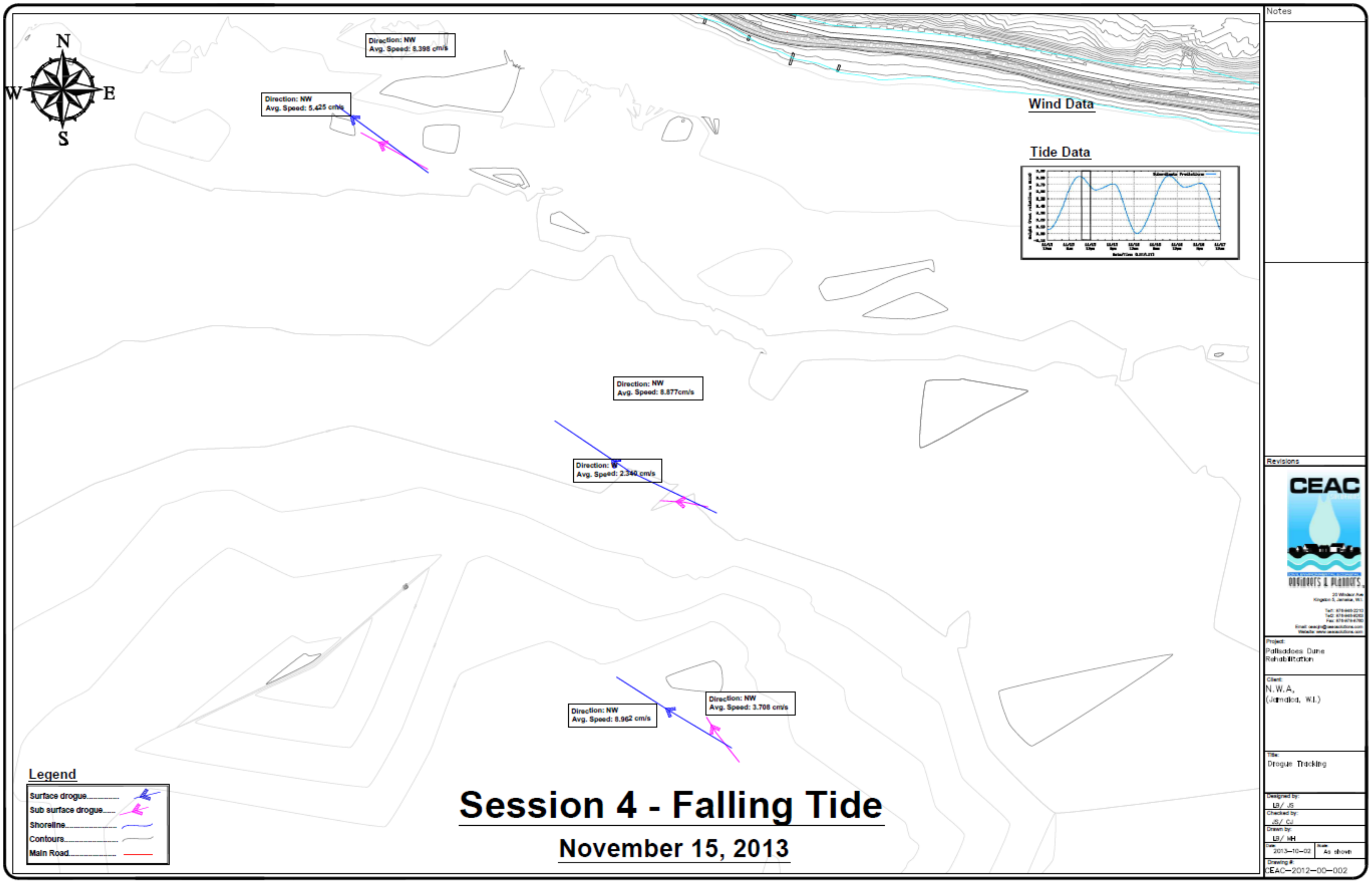
9	135	10:51	15-Nov	SURFACE	deploy	314536	1984480		185.000	2203	8.398	306.870	8.398	306.870	North Westerly
9	144	11:28	15-Nov		remove	314388	1984591								
6B	126	10:27	15-Nov	8	deploy	314980	1983938	ADCP 2	38.833	1919	2.024	281.889	2.340	278.144	Westerly
6B	137	10:59	15-Nov		measurement	314942	1983946		39.115	1473	2.655	274.399			
6B	142	11:23	15-Nov		remove	314903	1983949								
8	127	10:29	15-Nov	SURFACE	deploy	314994	1983928		154.742	1726	8.965	296.068	8.877	300.369	North Westerly
8	136	10:57	15-Nov		measurement	314855	1983996		145.908	1660	8.790	304.670			
8	143	11:25	15-Nov		remove	314735	1984079								
5	130	10:38	15-Nov	10	deploy	315026	1983527	OFFSHORE	60.605	1603	3.781	322.374	3.708	325.705	North Westerly
5	139	11:05	15-Nov		measurement	314989	1983575		29.155	802	3.635	329.036			
5	140	11:18	15-Nov		remove	314974	1983600								
1	131	10:39	15-Nov	2	deploy	315014	1983550		124.342	1453	8.558	301.517	8.962	302.348	North Westerly
1	138	11:03	15-Nov		measurement	314908	1983615		93.193	995	9.366	303.179			
1	141	11:20	15-Nov		remove	314830	1983666								

10.2 Drogue Plots

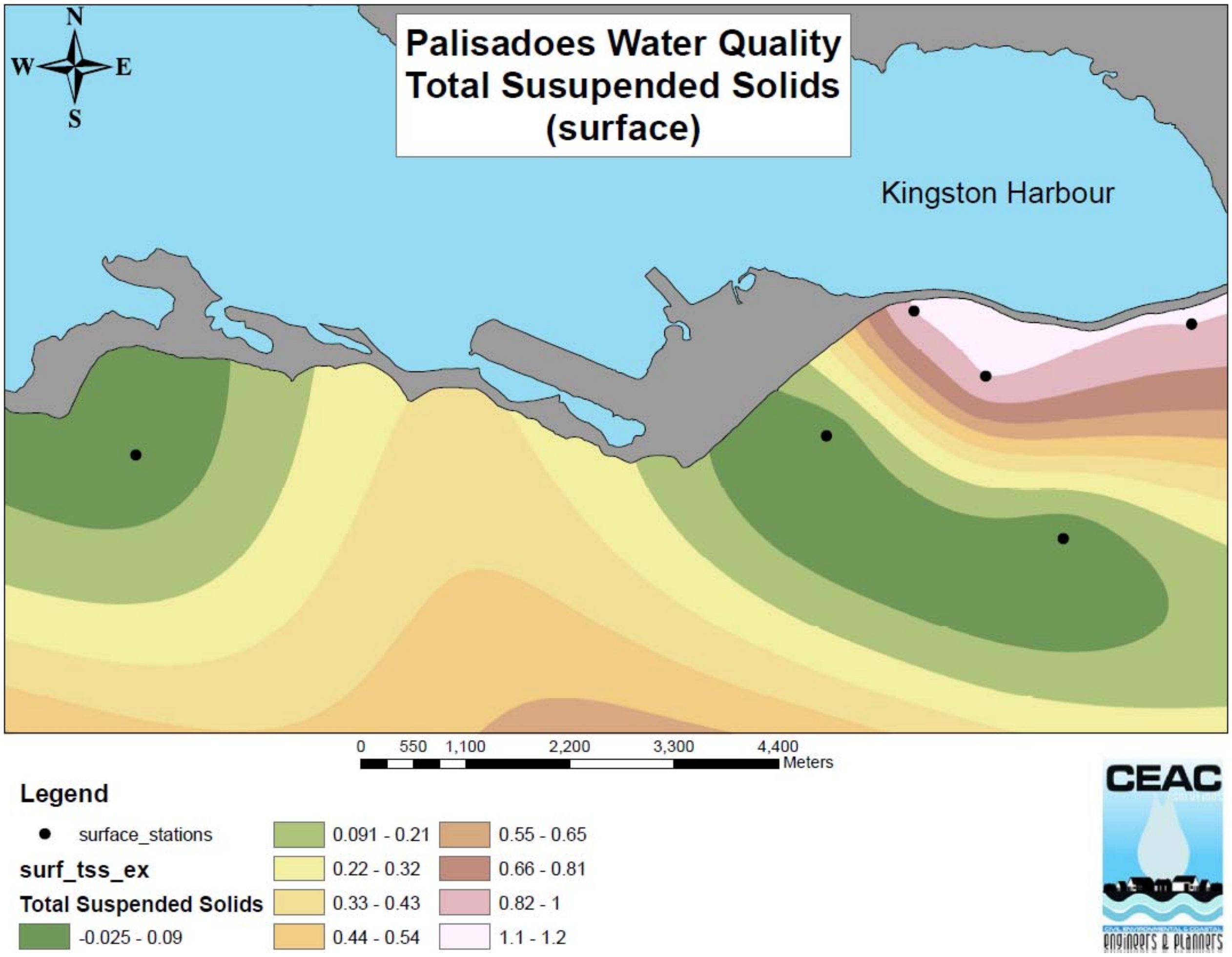


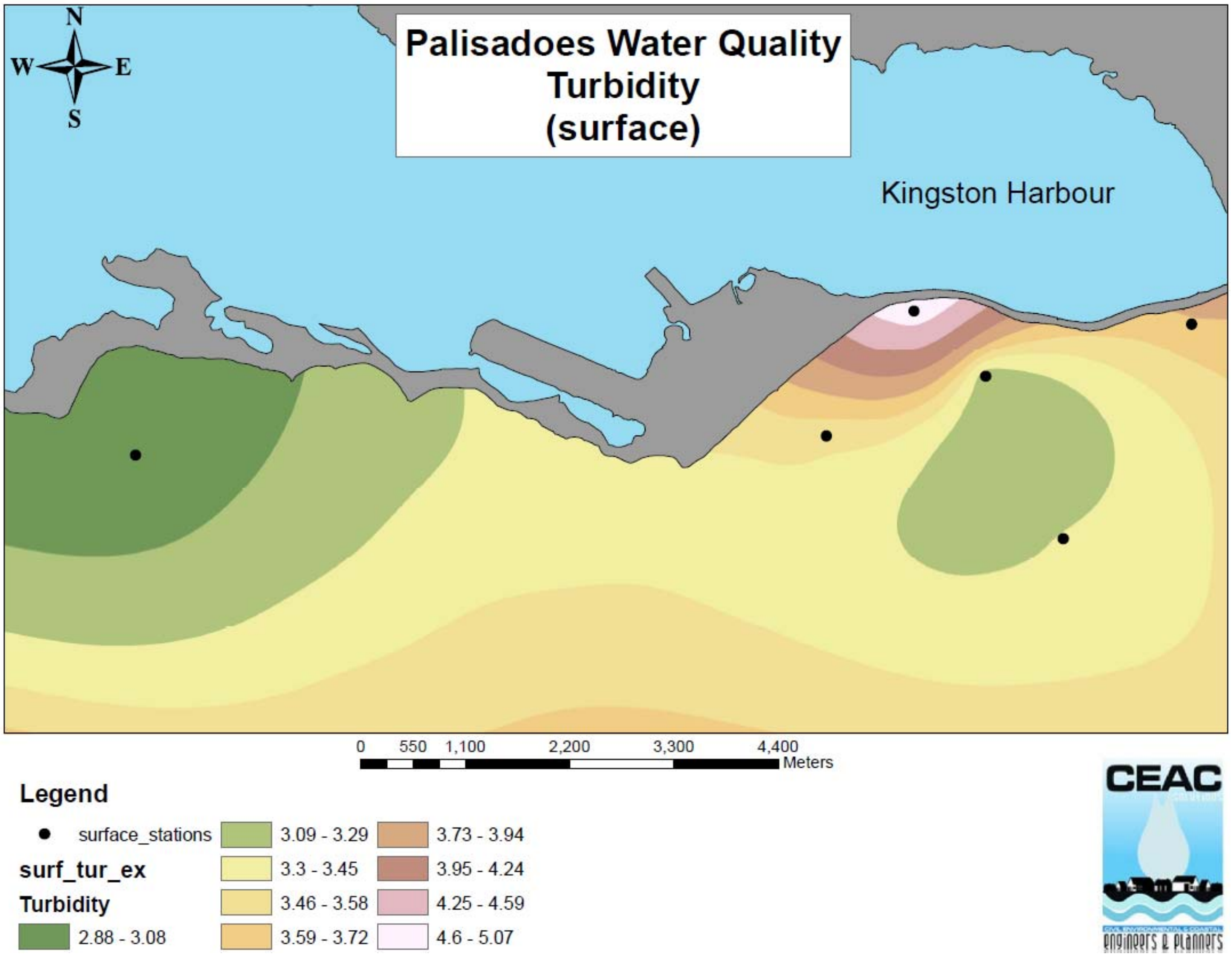

Notes
N
W
E
S
PLUMB POINT
Wind Data
Wind Direction | Wind Speed | Conditions
SSE | 5.65 m/s | Partly Cloudy
Tide Data
Direction: N
Avg. Speed: 4.884 cm/s
Direction: NW
Avg. Speed: 1.743 cm/s
Direction: NE
Avg. Speed: 1.481 cm/s
Direction: NE
Avg. Speed: 2.291 cm/s
General Direction: N
Avg. Speed: 1.761 cm/s
General Direction: N
Avg. Speed: 3.317cm/s
Direction: S
Avg. Speed: 2.060 cm/s
Direction: S
Avg. Speed: 2.775 cm/s
Legend
Surface drogue
Sub surface drogue
Shoreline
Contours
Main Road
Session 2 - Falling Tide
October 31, 2013
Revisions
CEAC
Project:
Negril Breakwater,
(Negril)
Client:
N.W.A,
(Jamaica, W.I.)
Title:
Drogue Tracking
Designed by:
LB/ JS
Checked by:
JS/ CJ
Drawn by:
LB/ MH
Date
2013-10-02
Scale
As shown
Drawing #:
CEAC-2012-00-002

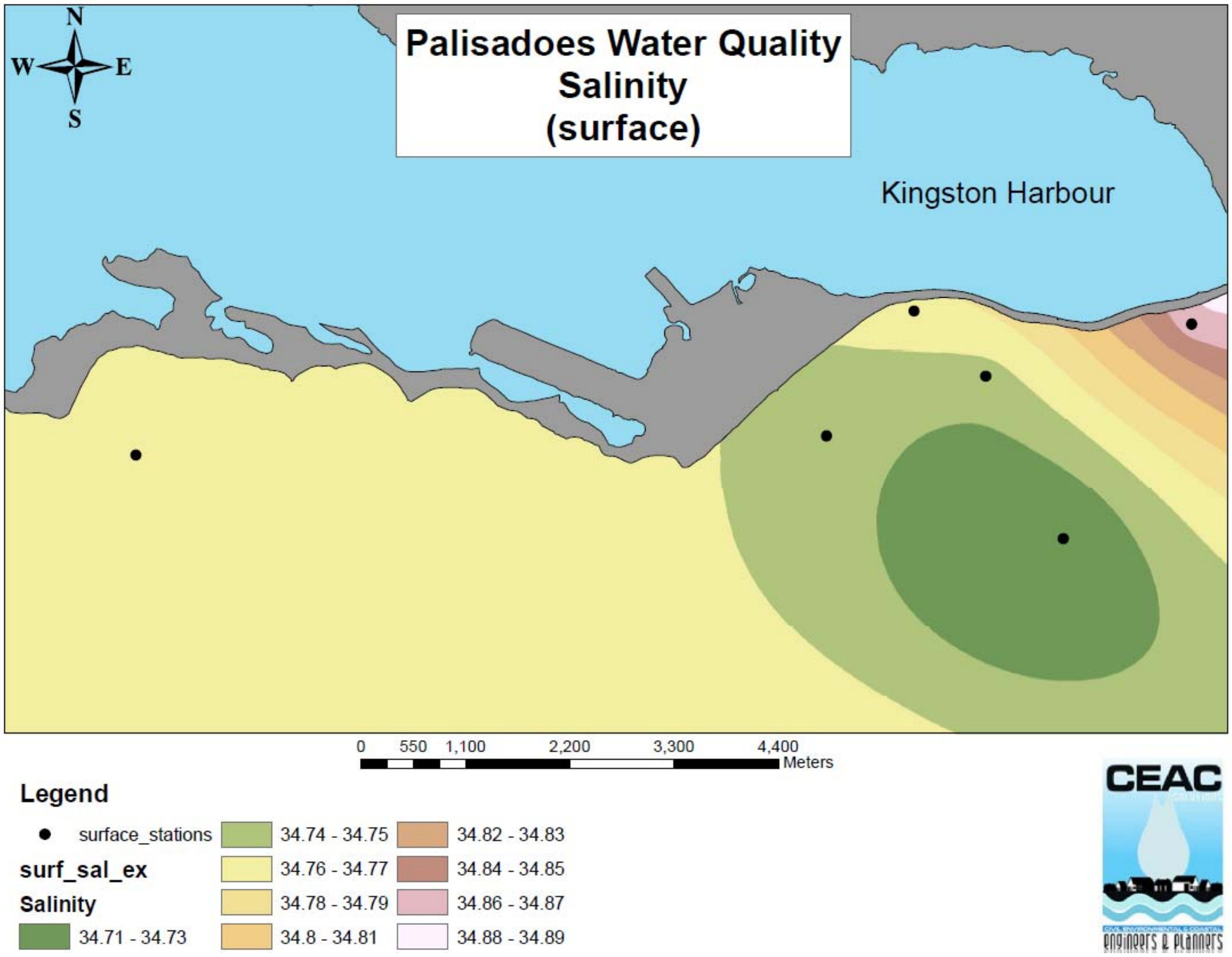

N
W
E
S
Direction: NW
Avg. Speed:2.703
Direction: NW
Avg. Speed: 1.789 cm/s
Wind Data
Wind Direction | Wind Speed | Conditions
SE | 3.35 m/s | Partly Cloudy
Tide Data
Direction: W
Avg. Speed: 5.310 cm/s
Direction: W
Avg. Speed: 2.604 cm/s
General Direction: NW
Avg. Speed: 3.470 cm/s
General Direction: NW
Avg. Speed: 2.138 cm/s
Legend
Surface drogue
Sub surface drogue
Shoreline
Contours
Main Road
Session 3 - Rising Tide
November 15, 2013
Direction: NW
Avg. Speed: 2.022 cm/s
Direction: NW
Avg. Speed: 2.396 cm/s
Notes
Revisions
CEAC
Project:
Palisadoes Dune Rehabilitation
Client:
N.W.A.
(Jamaica, W.I.)
Title:
Drogue Tracking
Designed by:
LB/ JS
Checked by:
JS/ CJ
Drawn by:
LB/ MH
Date:
2013–10–02
Scale:
As shown
Drawing #:
CEAC–2012–00–002

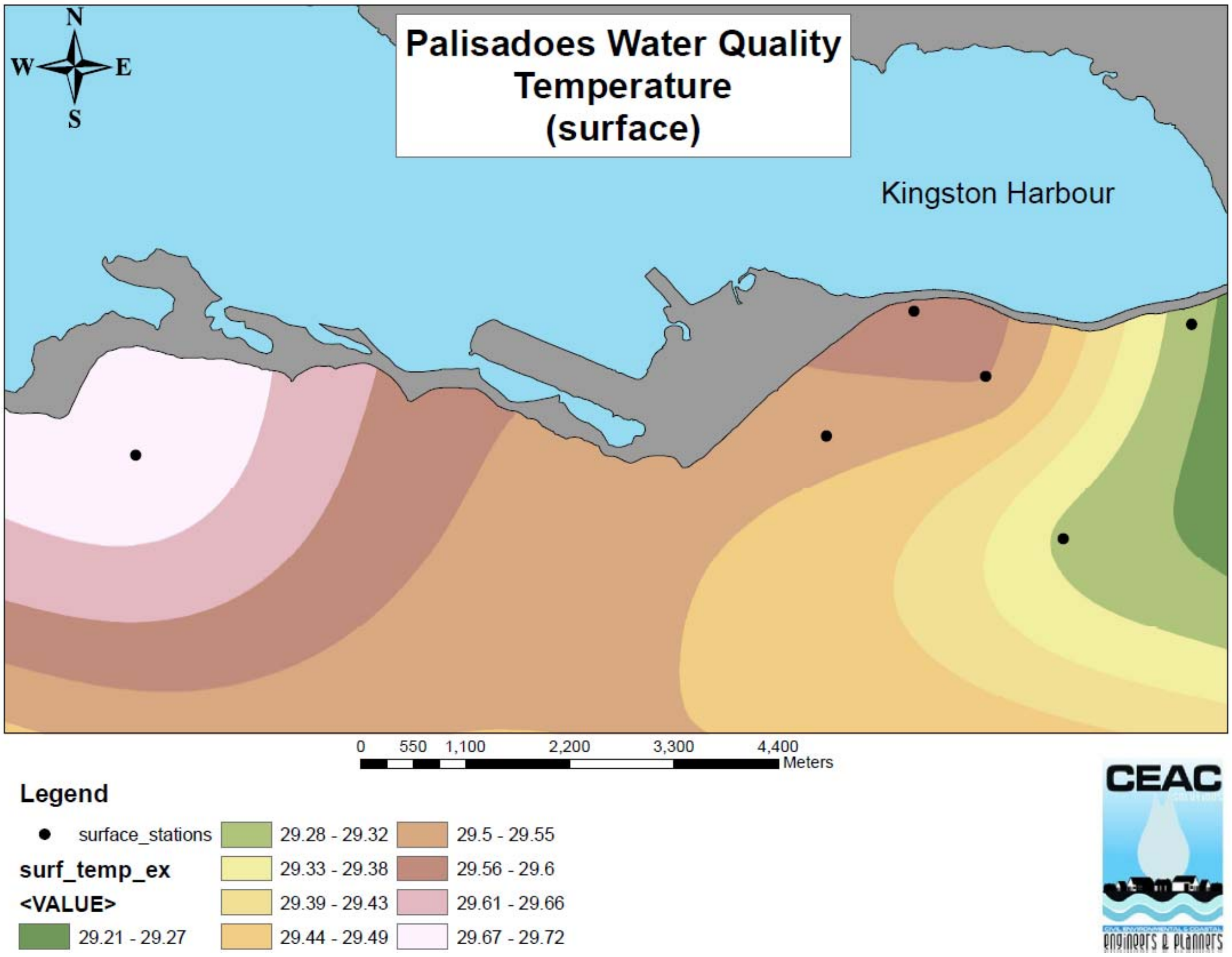

N
E
S
W
Direction: NW
Avg. Speed: 8.398 cm/s
Direction: NW
Avg. Speed: 5.425 cm/s
Wind Data
Tide Data
Direction: NW
Avg. Speed: 8.877cm/s
Direction: W
Avg. Speed: 2.340 cm/s
Direction: NW
Avg. Speed: 8.962 cm/s
Direction: NW
Avg. Speed: 3.708 cm/s
Legend
Surface drogue
Sub surface drogue
Shoreline
Contours
Main Road
Session 4 - Falling Tide
November 15, 2013
Notes
Revisions
CEAC
Project:
Palisadoes Dune Rehabilitation
Client:
N.W.A.
(Jamaica, W.I.)
Title:
Drogue Tracking
Designed by:
LB/ JS
Checked by:
JS/ CJ
Drawn by:
LB/ MH
2013-10-02
As shown
Drawing #:
CEAC-2012-00-002

10.3 Water Quality Plots




Palisadoes Water Quality
Turbidity
(surface)
N
W
E
S
Kingston Harbour
0 550 1,100 2,200 3,300 4,400
Meters
Legend
surface_stations
surf_tur_ex
Turbidity
2.88 - 3.08
3.09 - 3.29
3.3 - 3.45
3.46 - 3.58
3.59 - 3.72
3.73 - 3.94
3.95 - 4.24
4.25 - 4.59
4.6 - 5.07
CEAC


Palisadoes Water Quality
Salinity
(surface)
N
W
E
S
Kingston Harbour
0 550 1,100 2,200 3,300 4,400
Meters
Legend
surface_stations
surf_sal_ex
Salinity
34.71 - 34.73
34.74 - 34.75
34.76 - 34.77
34.78 - 34.79
34.8 - 34.81
34.82 - 34.83
34.84 - 34.85
34.86 - 34.87
34.88 - 34.89
CEAC
engineers & planners


Palisadoes Water Quality
Temperature
(surface)
N
W
E
S
Kingston Harbour
0 550 1,100 2,200 3,300 4,400
Meters
Legend
surface_stations
surf_temp_ex
<VALUE>
29.21 - 29.27
29.28 - 29.32
29.33 - 29.38
29.39 - 29.43
29.44 - 29.49
29.5 - 29.55
29.56 - 29.6
29.61 - 29.66
29.67 - 29.72
CEAC
Engineers & Planners

10.4 Engineering Drawings