FINAL ENGINEERING DESIGN REPORT

SHORELINE PROTECTION - RUNWAY 30 END AT THE NORMAN MANLEY INTERNATIONAL AIRPORT (NMIA): PALISADOES COASTAL HAZARDS, VULNERABILITY AND RISK ASSESSMENT REVIEW

airports authority of jamaica

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Date:	May 2017		
Comments:	Initial submission to client		

1 Introduction

1.1 Background

The Caribbean Sea shoreline at the southern end of the Norman Manley International Airport (NMIA) Runway is vulnerable to extreme hurricane waves, long term coastal erosion from sea level rise and short term storm events. The shoreline is associated with the Port Royal main road and end of runway 12. The Norman Manley highway is the only thoroughfare which connects the Port Royal community, commercial and institutional activities along in proximity to the airport to the main land. Additionally, the utility services are also in the road, including: water, telecom and electricity. The width of the buffer between the edge of road and shoreline varies between 30 to 80 meters. Observations and predictions suggest that 30 to 60 meters of shoreline can be lost in a single event and clearly suggest that both the road and end of runway are vulnerable. It is therefore desirous to protect the stretch of shoreline to secure both the airport, access for the residents of Port Royal, other stakeholders and critical infrastructures.

The National Works Agency (NWA) designed a solution in 2013 for this stretch of shoreline. This design was deemed unsuitable as it would not have met the requirements of the Jamaica Civil Aviation Authority (JCAA) Obstacle limitation surface (OLS) envelope for approaching aircrafts. The present design focus is aimed at producing designs with crest elevations and configuration that reduce the overtopping to safe limits during the design storm event as well as to meet the JCAA requirement.

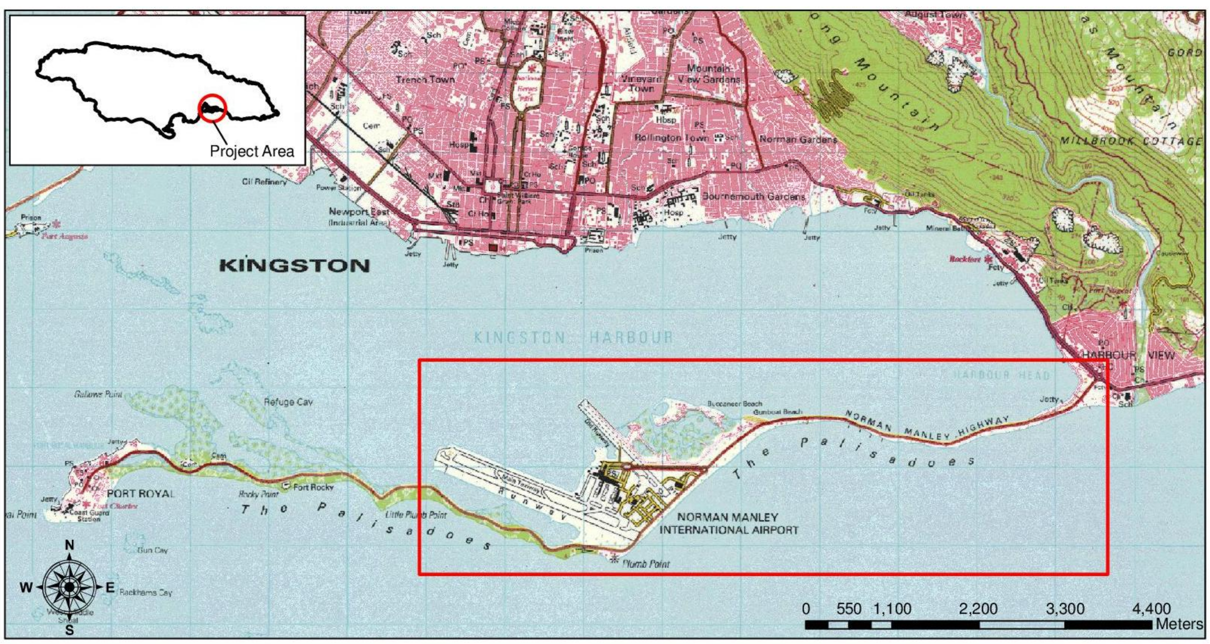

Figure 1.1. Location Plan of the Norman Manley International Airport situated along the Palisadoes.

1.2 Objectives and Scope of work

CEAC has been commissioned to conduct the following works:

i. A detailed engineering report with the findings will be prepared and submitted to the client. To include but not limited to:
    a. Description of the environment;
    b. Storm Surge and Wave Studies;
    c. Shoreline vulnerability (erosion);
    d. Proposed protection works:
        i. Drawings of plans, profiles and cross sections;
        ii. Footprint of both work areas and permanent works;
    e. Construction methodology statements;
    f. Ecological Impact statement and matrix;
    g. Specifications on the drawing sheets and standalone document will be provided

ii. Preliminary design drawings for each option for protection will be presented for review and comments. And engineering cost estimate based on prevailing rates will also be presented for each option considered.

iii. Final engineering designs will be prepared after the presentation and review process. These drawings will be sufficient for tendering and construction processes.

iv. Complete Bills of Quantities using the standardize method of preparing the measurement for building works and civil engineering works will be prepared.

v. The NEPA permit application form and project information forms will be completed and submitted with any supporting information. The client will be required to provide the necessary authorization letters and fees along with the copy of title.

1.3 Climate Resilience: Hurricanes affecting Jamaica

In recent times, global and regional climate change models have been predicting changes in the climate conditions that may increase the impacts of the coastal hazards. Sections of this area have experienced coastal flooding during storm events most notably being hurricane Allen in 1980. As such we believe that any infrastructure being implemented in this area must consider climate change impacts. In fact, Jamaica's Second National Communication (SNC) on Climate Change1 lists the main climate change hazards as follows:

- sea level rise
- increase in extreme events – precipitation and drought
- ***more intense storms and increased storm surge levels***
- increased temperature

[1] The Second National Communication Of Jamaica To The United Nations Framework Convention On Climate Change. Available at http://www.energycommunity.org/documents/snc2jamaica.pdf

The consideration of changes in these hazards and thus their impact will enable a better understanding of the risks to the assets to be implemented, and will therefore guide the elevation requirements to minimize such risks.

Extreme rainfalls and sea levels are typically associated with hurricanes and tropical storms and depressions. Hurricanes can form almost anywhere in the Tropical Atlantic Basin from the West Coast of Africa near the Cape Verde Islands, to the Gulf of Mexico and the Caribbean Sea which are the main development areas. Jamaica lies in the Atlantic hurricane belt west of one of the Main Development Area, Cape Verde Islands. Over the past twenty years, at least five major hurricanes have impacted the Caribbean region. The occurrences of these phenomena appear to be dependent on both natural variability and climate change factors. The natural variability occurs on a 30 to 60 years cyclical basis, corresponding to the Atlantic Multidecadal Oscillation (AMO)[2], and other more frequent cyclical basis from other teleconnections, such as El Nino, which vary from year to year. The natural climate features influence vertical wind shear and sea surface temperatures that steer hurricanes though the Main Development Region (MDR)[34]. Warm phase AMO is known to favor more hurricanes in the Caribbean versus cool phase AMO.

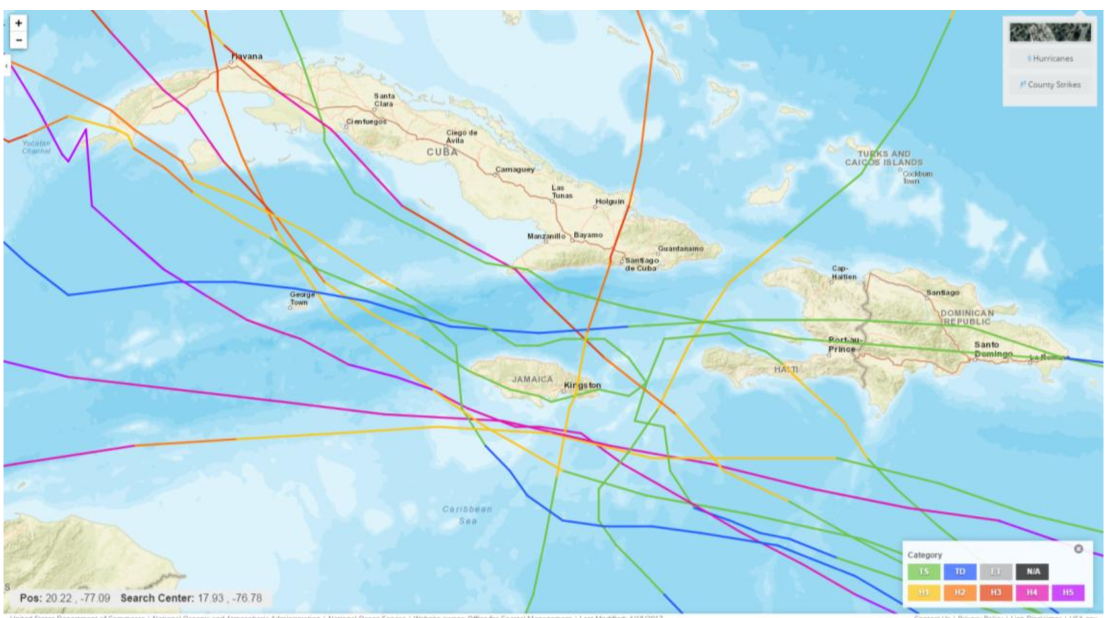

Figure 1.2. Tropical storms/Hurricanes passing through the Caribbean and within 500km of the project site, Jamaica over the past fifteen (15) years.

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[2] Goldenberg, Stanley B., Christopher W. Landsea, Alberto M. Mestas-Nuñez, and William M. Gray. "The recent increase in Atlantic hurricane activity: Causes and implications." Science 293, no. 5529 (2001): 474-479.

[3] Landsea, Christopher W., Gerald D. Bell, William M. Gray, and Stanley B. Goldenberg. "The extremely active 1995 Atlantic hurricane season: Environmental conditions and verification of seasonal forecasts." Monthly Weather Review 126, no. 5 (1998).

[4] Xie, Lian, Tingzhuang Yan, Leonard J. Pietrafesa, John M. Morrison, and Thomas Karl. "Climatology and interannual variability of North Atlantic hurricane tracks." Journal of Climate 18, no. 24 (2005).

The Intergovernmental panel on Climate Change (IPCC) have made projections based on numerical models which indicate tropical storms are far more intense storms than in previous years. The (2007)[5] IPCC report stated the following:

"There is evidence from modelling studies that future tropical cyclones could become more severe, with greater wind speeds and more intense precipitation. Studies suggest that such changes may already be underway; there are indications that the average number of Category 4 and 5 hurricanes per year has increased over the past 30 years."

Others have isolated the influence of increasing temperatures on the frequency of hurricanes and have suggested that a 0.5C increase will result in a 40% increase in hurricane activities.[6] This increase in sea surface temperature will more than likely occur as a result of increase in CO2. The direct relationship between CO2 and hurricane activities have also been explored and shown to be positively correlated, wherein an increase in CO2 will result in an increase in hurricane activities.[7] The predictions of the IPCC are consistent with the number of category 4 and 5 storms that have tracked within 400 kilometres Jamaica in the past 130 years. This determination was made by querying the National Oceanographic and Atmospheric Administration Hurricane Centre database on hurricane tracks for storms that passed within 400 kilometres of Jamaica shorelines. Nodes off the north, south, east and west coast was used See Figure 1.3 which clearly shows that the number of category 4 and 5 storms has increased from 10 to 15 storms per twenty year intervals up to 1950 to 30 to 35 storms per twenty years after 1950. This doubling of storm occurrences coupled with increased sea level rise can result in shoreline retreat as beach profiles adjust to a more intense wave climate.

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[5] Solomon, Susan, ed. Climate change 2007-the physical science basis: Working group I contribution to the fourth assessment report of the IPCC. Vol. 4. Cambridge University Press, 2007.

[6] Saunders, Mark A., and Adam S. Lea. "Large contribution of sea surface warming to recent increase in Atlantic hurricane activity." Nature 451, no. 7178 (2008): 557-560.

[7] Krishnamurti, T. N., R. I. C. A. R. D. O. CORREA-TORRES, Mojib Latif, and Glenn Daughenbaugh. "The impact of current and possibly future sea surface temperature anomalies on the frequency of Atlantic hurricanes." Tellus A 50, no. 2 (1998): 186-210.

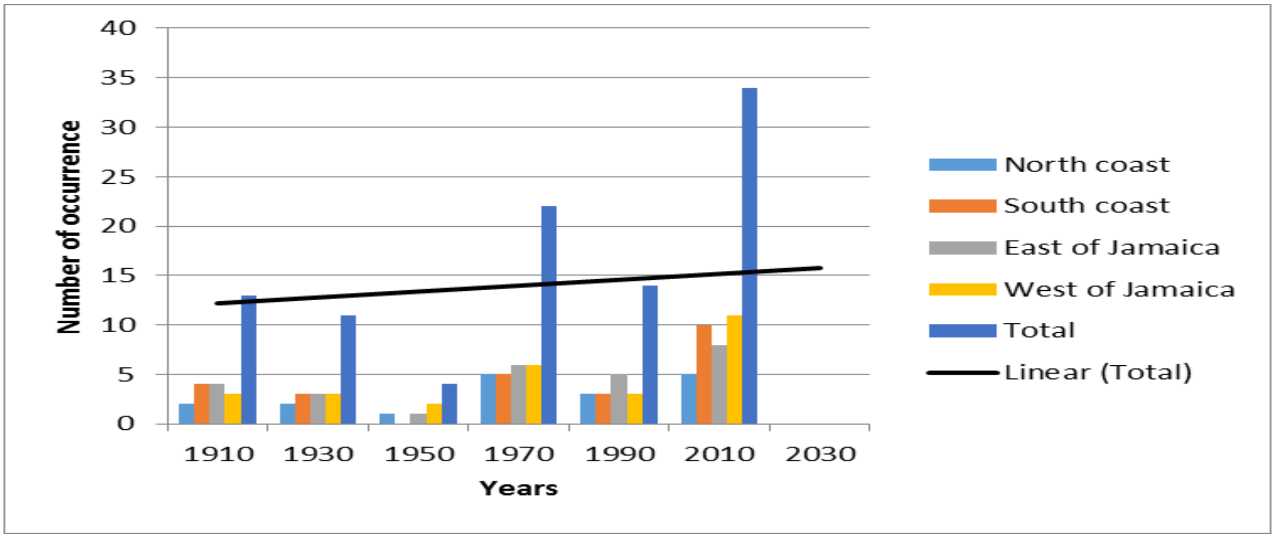


Figure 1.3. Occurrences of Category 4 and 5 hurricanes that have passed within 300 kilometers of Jamaica's shoreline since 1890 to 2014, in twenty years intervals.

The observations of increase frequency for extreme hurricanes (Category 4 and 5) in the recent past (<30 years) would suggest that the predictions of storm surge based upon observations are conservative.

2 Data Collection and Analysis

2.1 Topographic Surveys

Topographical data for the project area was obtained from an aerial survey conducted during March of 2017 (see Figure 2.1). The methodology employed consisted of:

1. Setting out of Twenty (20) ground controls by a commissioned surveyor to reference the aerial survey data to Mean Sea Level (MSL);

2. Conducting the aerial survey of approximately 800 hectares of land at 5 cm accuracy.

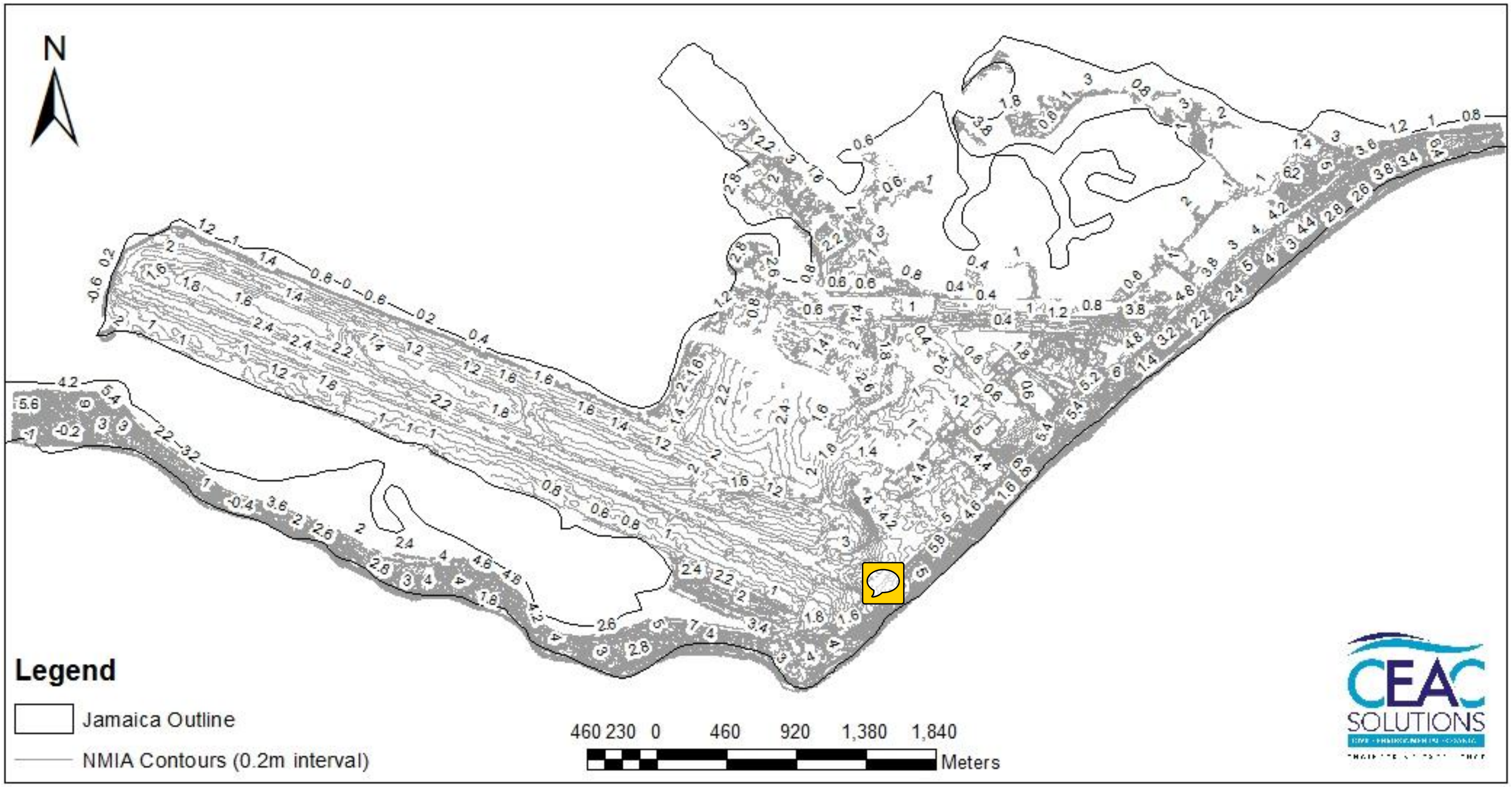


Figure 2.1 Contours generated from topographic survey of the Norman Manley International Airport (NMIA) and its environs

2.2 Anecdotal Information

Anecdotal information[8] on the effects of past storms was collected to aid in the verification of the storm surge model for the area. Such evidence was also used to generate an estimate of the return period for actual storm surge versus estimated for verification purposes. The role of anecdotal information in coastal engineering[9] and other scientific research areas has been discussed elsewhere[10], and it is our opinion that the gathering of this information creates a wealth of information to facilitate the management of the storm surge risk.

Interviews were conducted in March of 2017 with available residents and workers in the immediate area with first hand memory of hurricane events (see Figure 2.2). Overall, fifty-eight (58) interviews were conducted with residents having an average age of 53 years and living an average of 35 years in the immediate area (see Table 11.2 within the Appendix). The respondents recalled eight (8) storms, including: Charlie (1951), Allen (1980), Gilbert (1988), Lili (2002), Ivan (2004), Dean (2007), Gustav (2008) and Sandy (2012). Of the respondents, approximately 41.4 percent indicated Hurricane Ivan had storm surges ranging from 0.99 to 3.38 meters in elevation with an average of 1.96 meters. Another 24.1 percent remembered Dean, being the most recent, having storm surge elevations ranging from 1.38 to 2.77 meters with an average elevation of 2.17 meters. The remaining 34.5 percent is shared among the other hurricanes.

[8] Defined as the informal observations to make causal inference

[9] Woodworth, Philip L., and David L. Blackman. "Changes in extreme high waters at Liverpool since 1768." International Journal of Climatology 22, no. 6 (2002): 697-714.

[10] Enkin, Murray W., and Alejandro R. Jadad. "Using anecdotal information in evidence-based health care: heresy or necessity?." Annals of Oncology 9, no. 9 (1998): 963-966.

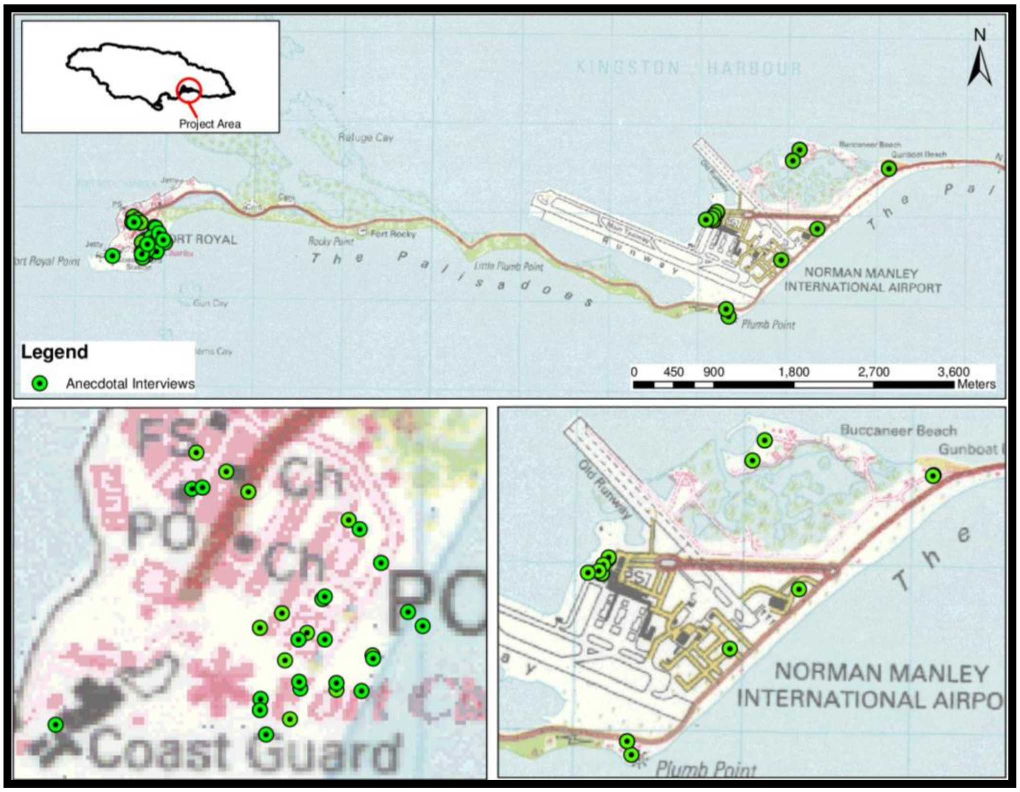


Figure 2.2. Locations in Port Royal and along Palisadoes where anecdotal interviews were conducted.

2.3 Quarries Assessment

An assessment was conducted for five (5) quarries which were used by contractors for the supply of core fill and armor stones under the Palisadoes Road Rehabilitation project completed in 2010. The five (5) quarries are located in St. Catherine and St. Thomas as follows:

- St Catherine
    - Hill Run - Armor Stones
    - Paul Mountain - Armor Stones
    - Sure Products (Ferry) - Armor Stones
- St. Thomas
    - Blacks Quarry – Armor Stones
    - Caribbean Aggregates – Core fill

The findings of the assessments conducted in 2010 are discussed below. Before construction commences, a current quarry assessment will have to be undertaken to confirm both the operation of the quarry and availability of stones from the aforementioned quarries. This exercise may include identifying other feasible quarries, if necessary.

2.3.1 Methodology

The following methodology was adopted for the past assessment.

1. Site visit to observe and measure boulders on quarry floor.
2. Obtain Information on quarry production rates and previous projects worked on.

3. Documentation of these operations with photographic evidence. Photos were taken of the stones at the quarries and the area from which stones would be produced where possible.
4. Visual inspection of onsite quarry operations.
5. Desktop research to confirm quarry licenses were up to date.

2.3.2 Observations

2.3.2.1 Mogul Transport & Construction (Hill Run Aggregates) – QL1679

This quarry is located in Hill Run, approximately 4.3 km south west of Portmore, St. Catherine and is approximately 22.3 km west of the project area. The quarry has been involved in previous projects such as:

- Highway 2000,
- New Era Homes developments
- Gordon Cay expansion project.

The quarry had a reserve of approximately 404,682 square meters and expected to produce 1000 tons of armor stone per day.

The unused sections of is estimated to have had reserves amounting to 971,238 cubic meters using a 15% production rate. In other words, the quarry has the potential resource to satisfy this project. The boulders stockpiled on site were irregular in shape which also ideal for the purposes of this project. It should be noted however that some staining as well as weathered rocks were noticeable in the stockpiles. Please see Plate 2.1 and Plate 2.2 below.

Samples taken from this quarry were tested for both their water absorption and specific density characteristics. The results showed that the stones met and exceeded the specifications described in the contract. The water absorption coefficient of the stones was found to be 2.8% which is within the 5% specified.

Plate 2.1 Water pump in disrepair at the Hill Run quarry

Plate 2.2 Stockpile of unsorted stones seen at Hill Run quarry

2.3.2.2 Sure Products Manufacturing Company Limited (Ferry Pen) - QL1950

Sure Products Manufacturing Company Limited has a licensed quarry at Ferry Pen, St. Catherine which is approximately 17.3 km northwest of the project site.

Samples taken from this quarry were tested for both their water absorption and specific density characteristics. The results showed that the rocks surveyed met and exceeded the specifications set out in the contract. The water absorption coefficient was found to be 1.4% which exceeded the 5% maximum coefficient stated.

In summary, the Ferry Pen quarry has stones which are suitably shaped and sized for the works, the majority of which meet the criteria for the primary armour stones in the revetment. The geology of the stones also met and exceeded the specifications set out in the contract. It is important to note however that visual inspection revealed cracking in the rocks and the rock profile making these stones unsuitable for the revetment works.

Plate 2.3. Stockpile at Ferry Pen Quarry. Rocks are weathered and fractured.

Plate 2.4 Stones at Ferry Pen looking weathered. Honeycomb rocks are also noticeable.

2.3.2.3 Black's Quarry

Black's Quarry is located in Bull Bay St. Thomas approximately 5.2 Km north east of the site and was involved in the initial Palisadoes Emergency works. In 2010, the quarry was capable of producing 1500 tonnes per day.

Samples taken from this quarry were tested for both their water absorption and specific density characteristics. The results were in agreement with the initial visual observation as the water absorption coefficient was 5.4% which exceeds the maximum allowable water absorption coefficient specified in the contract of 5%. The stones did meet the criteria for specific density with a value of 2.49% which fall within the range specified (2.45 to 2.5).

In summary, Black's quarry is estimated to have stones which are suitably shaped and sized for the works, however the geology of the stone material is not suitable for these works.

Plate 2.5. Picture shows stones stockpiled at Black's Quarry. The picture shows the soft brittle material which makes up some of these stones.

Plate 2.6. Soft chalky white limestone at Black's Quarry evident here.

2.3.2.4 Earthcrane Haulage Ltd. (Caribbean Aggregate Yallahs) – QL No.1225

Caribbean Aggregate is located along the Yallahs River at Albion, St. Thomas, approximately 16.8 km southeast of the site with licenses to quarry only 0.1 hectares of the parcel of land with Volume 606 and Folio 14. The Quarry has been involved in previous projects, the most recent being the Palisadoes rehabilitation project.

Visual inspection of the stones under the crusher onsite revealed that the cut stones were non-plastic and free from clay or any other deleterious material. The stones were semi-rounded however this is less important when the being used as fill material.

In summary, the material at the site is geologically suitable however the gradation tests show that the material was meeting the specifications before crushing. Crushing is therefore unnecessary based on the sample taken from the source. The quarry owner should therefore re-access the source and adjust fill material production methods accordingly. Please see plates which show stockpile of fill material at quarry.

Plate 2.7.Crusher actively cutting stones. Crusher cutting stones too small.

Plate 2.8. Stockpile of material at quarry previously removed from river bed.

2.3.3 **Comparative Analysis**

After assessing each quarry in 2010, a comparative analysis was conducted using several field indicators as well as laboratory test indicators. Each quarry was given a rating for each parameter ranging from 1 to 4 with poor and excellent being the two extremes respectively. Each criteria rated was given a weighting rating based on the importance of the criteria to the suitability of the stone.

The analysis revealed that Hill Run Quarries Ltd. had the most suitable armour stones for the project achieving a weighted score of 80%. Lithographically, the stones from this quarry were the best of the four quarries. It is important to recall however that this quarry is prone to flooding and so certain areas of the quarry may be inaccessible during or after heavy rainfall events. This should be taken into account in the present hurricane season.

Hill Run Quarry would however be a suitable supplier for the primary armour for both types of revetments to be constructed.

Ferry Pen Quarry received the second highest weighted score of 70% with regard to supplying primary armour stones. The noted drawbacks being that cracking was noticed as well as some honey comb rocks. The method of retrieving the stones is blasting which is also not preferred. Stone sizes at this quarry show that the majority meet the specifications for the dune revetment.

Black's Quarries Ltd received the lowest weighted score of 63 % with respect to the supply of armour stones. Despite being the closest quarry to the project site, blasting is the likely method of stone production which is a poor method of retrieval for the purposes of revetment building as the shock and stresses from the blast decrease the integrity of the stone. Lithographically, stones from this quarry are the worst of the three quarries as it was evident from the site inspection that the stones were comprised of mostly calcium carbonate. Although meeting the specifications for specific density, it did not meet the specifications for water absorption with a value of 5.4%.

The Yallahs Quarry (Carribbean Aggregate) was the only quarry visited which is expected to supply fill material. However, based on the criteria for the assessment, the two samples taken from this quarry (i.e. the source and stockpile under the crusher) were investigated. The investigation found that the source material was more ideal with a 94% weighted score as opposed to the 83% scored by the sample taken from the stockpile under the crusher. The difference can be attributed to the fill material failing to meet the specifications after crushing.

Please see Table 11.1 within the Appendix which shows a basic comparison between the quarries inspected.

3 Storm Surge Hazard

3.1 Introduction

Hurricane storm surge is an increase in the water levels during the passage of a hurricane. The increases are due to several factors, the major ones include:

1. Inverse barometric pressure
2. Tides
3. Waves
4. Wind
5. Bathymetry

Increases in water levels will cause further flooding of the near shore area as well as it will cause more destructive waves to reach closer to the shoreline or further inland. It is crucial to determine the setups that will be generated at the project site in order set the design parameters for the floor levels and sensitive equipment.

3.2 Climate Change Considerations

It was necessary to consider the effect of climate change on the project area. A review of several peer reviewed research papers was conducted in order to inform the approach to applying the climate change variables to each hazard. The hazards included sea level rise, storm intensities and the associated storm surge and wave heights.

3.2.1 Current and Projected Trends for Mean and Extreme Sea Levels

Global sea levels have risen through the 20th century, and it is 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. A review of the IPCC fifth report AR5 indicate the following:

- GMSLR during 1901–2010 can be accounted for by ocean thermal expansion, ice loss by glaciers and ice sheets, and change in liquid water storage on land. It is very likely that the 21st-century mean rate of GMSLR under all RCPs will exceed that of 1971–2010, due to the same processes.
- A likely range of GMSLR for 2081–2100 compared with 1986–2005, depending on emissions (0.40 [0.26–0.55] m for RCP2.6, 0.63 [0.45–0.82] m for RCP8.5), can be projected with medium confidence, including the contribution from ice-sheet rapid dynamics.
- That for Jamaica and the region, the sea level rise is approximately the global average[11] 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[12] (± 0.5 m relative to global mean) and this is equivalent to 3.7 mm/yr.

[11] IPCC 2013

[12] IPCC 2007

The sea levels will be adjusted by 3.7mm/year to account for the changes in sea level at the anticipated end of the project life.

3.2.2 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 shows an increasing trend 0f 0.07 day/yr. for the same period which is statistically significant[13].

Other recent studies[14] focused on the North Atlantic region highlighted several documentary evidence of trends in the North Atlantic cyclone frequencies over the past century. The trends show higher North Atlantic TC activity (+60%°C_1) since 1995 [Goldenberg et al., 2001] and increased frequency of very intense TCs (~ + 17%°C_1) within the North Atlantic region since the 1990s [Emanuel, 2007; Holland and Webster, 2007; Bender et al., 2010]. The studies further indicate the trends have been observed in association with long term changes in tropical Atlantic oceanic and atmospheric conditions important to North Atlantic TC development including increased mean surface temperatures (0.12 ± 0.04°C per decade for 1951–2010), increased tropospheric water vapour (7%°C_1 since 1970s), and fluctuations in vertical wind shear (within 6ms_1 since 1995) [Goldenberg et al., 2001; Intergovernmental Panel on Climate Change (IPCC AR5), 2013]. Changes in some of these local factors as well as the influence of other remote factors such as the variability of sea surface temperatures (SSTs) in the central and eastern equatorial Pacific associated with El Niño–Southern Oscillation and/or multi-decadal North Atlantic variations have also been shown to influence TC variability on inter-annual and decadal timescales [Gray, 1984a; Goldenberg and Shapiro, 1996; Bell and Chelliah, 2006. Global climate Models (GCM's) utilizing SST and near-surface wind predictors suggest significant increases in mean annual frequency by 2-8 TCs by 2070-2090, compared to a single surface wind predictor model, indicating that positive trends in SSTs under global warming have a larger relative influence on projections than changes in the variability of the surface winds. Even though similar researches[15] show an overall decrease in Global and hemispheric TC genesis numbers (13%–25%) under the IPCC A1B global warming scenario. This must not be confused with the 2-8 increase per year noted by others which is Caribbean region specific. Additionally it was also shown through use of high resolution models that when the instantaneous maximum surface wind velocities for TCs are averaged, all coastal regions show an increasing intensity by 1%-7%.

These increases pointed out by these studies are therefore indicating an increasing potential for future catastrophic damage due to TCs in this region which should be accounted for in designs being

[13] 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.*

[14] Investigating the Use of Statistical Models for Projecting Future North Atlantic Tropical Cyclone Frequency, Jhordane Jones, 2016.

[15] Future Changes in Tropical Cyclone Activity Projected by the New High-Resolution MRIAGCM, Hiroyuki Murakami

contemplated for important infrastructures. Storm surge and extreme wave assessment will include these increases for the prediction of future climate scenarios.

3.2.3 Summary

Based on the assessments and literature reviewed the following climate change factors were incorporated into the design (Table 3.1), specifically the deep water and nearshore wave climate analysis carried out in the following sections.

Table 3.1. Summary of climate change considerations.

Parameter	Climate Factor (Cf)
Water Level (above existing MSL)	3.75 mm/yr.
Storm Intensity	7%
Current Number of Storms Per year	5
Increase in number of storms per year	1 – 3

3.3 Storm Surge Estimation

3.3.1 Methodology

Tropical storms and or hurricanes passing a site will generate significantly large waves that have the potential to cause damage to shorelines. Even though they may be generated up to a hundred km offshore they will travel long distances where the destructive effects are felt. Similarly, they will also generate storm surges which also have the potential to cause localized flooding. It was therefore necessary to define the deep-water hurricane wave climate at a point offshore the project area:

- Latitude: 17.59 degrees North
- Longitude: 76.73 degrees West

This chosen offshore point is one that it is felt will definitely be impacted by offshore waves and will, give a good estimation of the offshore deep water wave condition. A southerly profile was investigated from the offshore point to the shoreline where the runway will be protected.

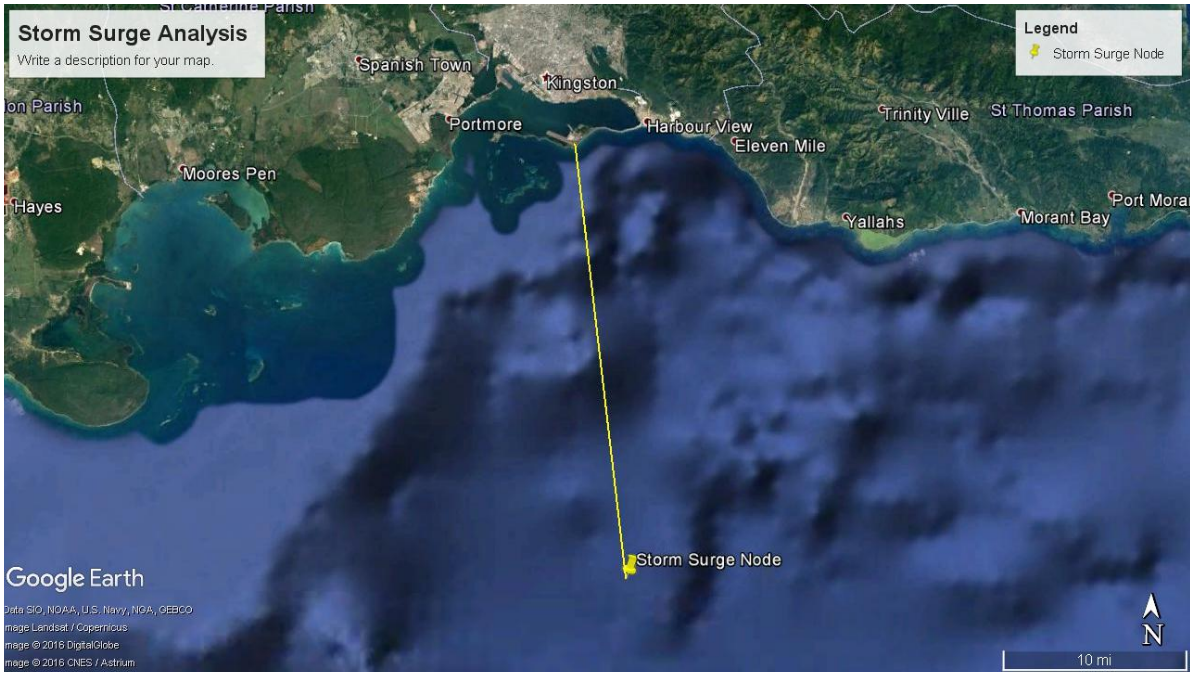


Figure 3.1. Location of offshore point used for Extremal analysis, showing the track used in the analysis.

The following procedure was carried out in order to arrive at the statistical return period surge levels:

1. Extraction of Storms and Storm Parameters from the historical database. A historical database of storms was searched for all storms passing within a search radius of 300km radius of the site.
2. Application of the JONSWAP Wind-Wave Model. A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.
3. A bathymetric profile from deepwater to the site was then defined and each hurricane wave transformed along the profile. The storm surge and wave height at the nearshore end of the profile was then extracted from the model and stored in a database. All the returned nearshore values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

3.3.2 **Results**

3.3.2.1 <u>Hurricane Occurrences and resulting waves</u>

The results of the search from the database for hurricanes that came within the search radius of the site are shown in the appendix. Extremal analysis results are summarized in the bi-variant. The results of the search shows the site's overall vulnerability to such systems. In summary:

- 103 hurricane systems came within 300 kilometres of the project area
- 8 of which were classified as catastrophic (Category 5)
- 16 were classified as extreme (Category 4)

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

- **69 (x6 hours) occurrences from the South-West**
- 66 (x6 hours) occurrences from the South
- 65 (x6 hours) occurrences from the West
- 63 (x6 hours) occurrences from the East
- 61 (x6 hours) occurrences from the South-East

The South, South-West and South-East directions are more prevalent for the node considered because of the unobstructed path (fetch) for waves to propagate and reach shore. The site, becomes more exposed as soon as the passing hurricane systems are more to the south of the island.

3.3.2.2 Analyzed Results

A total of 103 storms passed within 300km of the site for the period 1851 to present (166 years). The calculated results for storm surges inclusive of wave run-up ranged from 1.25m to 5.19m, under the current scenario. The annual maximum values of storm surges were then fitted to a generalized type III extreme value (GEV) distribution to determine the return period storm surge values.

Boot resampling was employed in order to improve the estimates of standard errors and confidence intervals for given the dataset is only 66 of over 166 years.

Approximately 207 samples were generated for the exercise. For all samples generated, the number of elements in each corresponded to the number of elements found in the original data set. The range of sample estimates obtained enabled the model to establish the uncertainty of the mean values estimated.

The climate change considerations were also included in the exercise, they were the number of event per year (5) and a seven percent (7%) expected increase in intensity for the more intense events. The increase in intensity was applied to all the events exceeding the mean values in the sample.

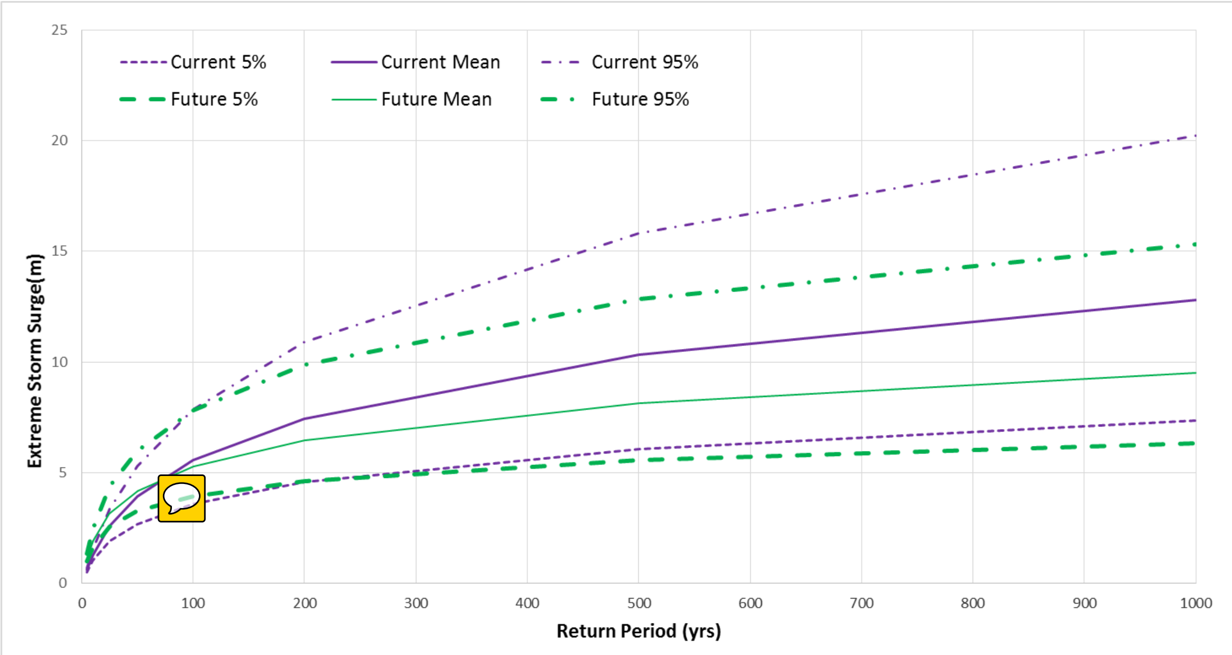


Figure 3.2 Storm Surge elevations for return periods for project area under the present and future climate

The climate change scenario as predicted by the model show an increase in intensity for the more frequent events and lower intensity for less frequent higher intensity events as a net effect of increasing the number of storms per year to 5 as well as applying the increase of 7 percent to the most intense storms. The model is however limited in how it can apply the change to a sample with significantly higher number of very small events. The final results were therefore modified to reflect no change in the intensity of the less frequent events.

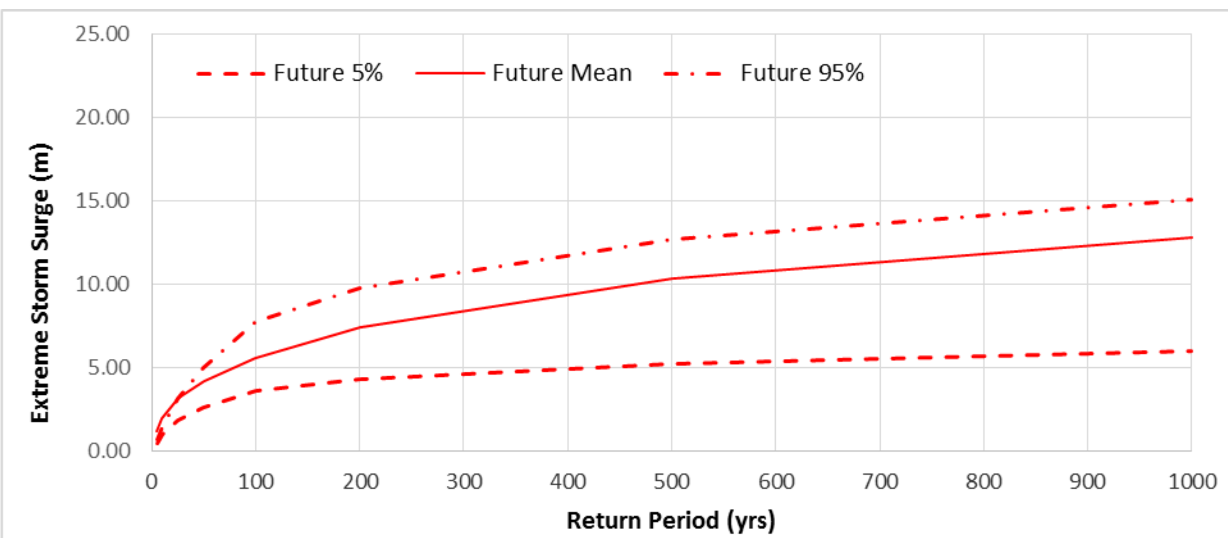


Figure 3.3 Storm Surge elevations for return periods for project area under the combined future climate

Table 3.2 Comparative Storm Surge values with and without sea-level rise (SLR) inclusive of runup

Return Period	Current	Future (2050) without SLR	Future (2050) with SLR
5	0.59	1.03	1.15
10	1.27	1.81	1.93
25	2.58	3.03	3.15
50	3.92	4.06	4.18
100	5.55	5.15	5.27

3.3.3 Worst Case Storm Surge Simulations: Category, Landfall and Approach Analysis

Worst case storm surge simulations for the project shoreline were carried out. The approach was to investigate, based on the general movement of Hurricanes from east to west as the passes Jamaica, possible worst case hurricane paths relative to the project shoreline.

The four paths investigated were as follows:

1. Parallel Direct Hit 1: Hurricane moving from south to north, 10 km west of the site.
2. Parallel Direct Hit 2: Hurricane moving from south to north, 40km west of the site.
3. Parallel Jamaica Southern Shoreline 1: Hurricane moving east to west along the south coast 10km offshore of the site
4. Parallel Jamaica Southern Shoreline 2: Hurricane moving east to west along the south coast of Jamaica 40km offshore of the site.

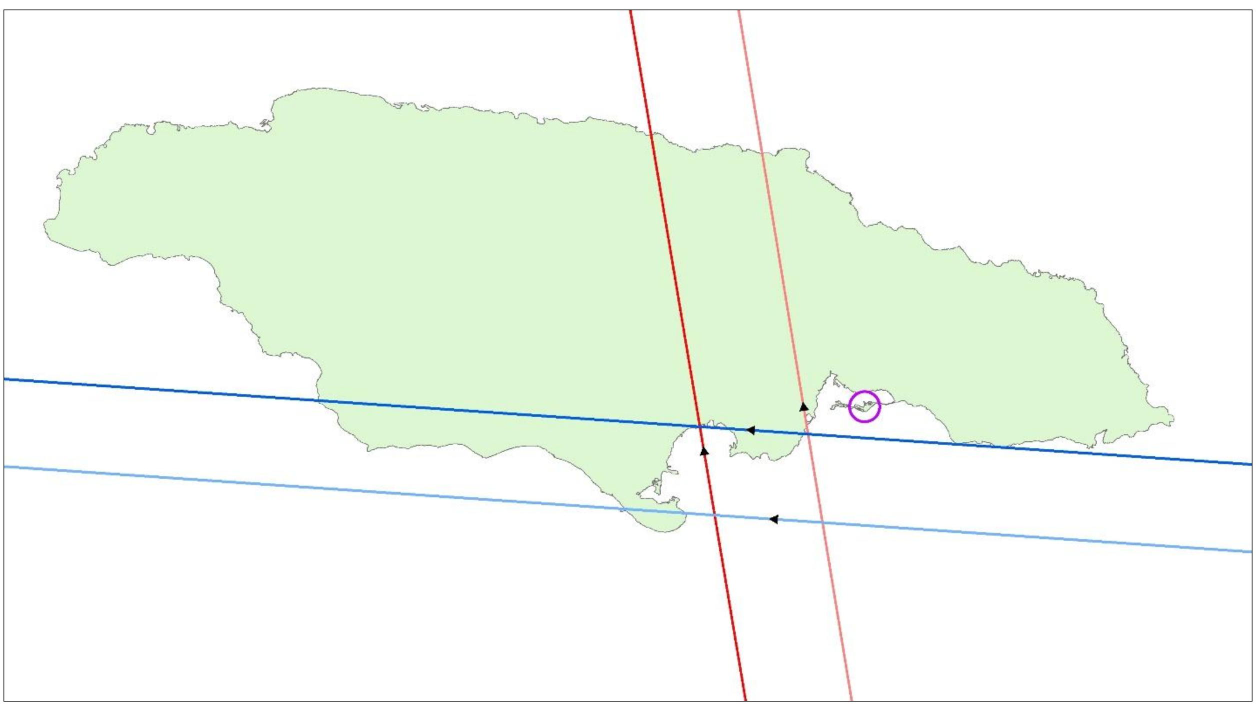


Figure 3.4 Illustrating the trajectories of worse case storms modeled

3.3.3.1 Results

3.3.3.1.1 Direct hit from South to north

The direct hit scenario revealed setup levels ranging 1.74 to 2.97 and 4.81 to 6.23 meters for category 3 to 5 hurricanes without and with run-up respectively travelling south to north 40km west of the site. This scenario proved to be the worst case scenario and would have the most impact on the project site. The scenario where the hurricane tracks 10km to the west of the site is the second most extreme condition where the water levels vary from 1.34m to 2.24m without run-up and 4.39 to 5.49m with run-up.

The return periods for the setups with run up were comparable to the 50 and 100 year return periods. The Direct hit Category 5 scenario has a return period of 112.8 years and is the most extreme scenario and only one that exceeds the 100year RP.

3.3.3.1.2 Coastline tracking (Parallel to shoreline)

For hurricanes tracking east to west (Shore parallel), the setup levels range from 1.07 to 1.94m and 4.09 to 5.18 meters without and with run-up respectively for category 3 – 5 hurricanes tracking 40km south of the site. This was the more dangerous of the two shore parallel scenarios investigated. The more extreme of the shore parallel scenarios has setup plus run up of 5.33meters which corresponds to a return period of 74.2 years.

Hurricane path/Track	**Hurricane Scenario Modelling**					
	Category 3 - Setup (m) without run up [Return Period yrs.]	**Category 3 - Setup (m) with run up [Return Period yrs.]**	**Category 4 - Setup (m) without run up [Return Period yrs.]**	**Category 4 - Setup (m) with run up [Return Period yrs.]**	**Category 5 - Setup (m) without run up [Return Period yrs.]**	**Category 5 - Setup (m) with run up [Return Period yrs.]**
Direct Hit 10 km west of site (from South)	1.34 [11.6]	4.39 [48.0]	1.78 [14.3]	4.95 [62.2]	2.24 [17.7]	5.49 [80]
Direct Hit 40 KM west of site (from South)	**1.74 [14]**	**4.81 [58.0]**	**2.34 [18.5]**	**5.52 [81.0]**	**2.97 [25.0]**	**6.23 [112.8]**
10 KM Parallel offshore of site (from east)	1.07 [10.3]	4.09 [41.8]	1.57 [12.9]	4.75 [56.7]	1.94 [15.4]	5.18 [70.0]
40 KM Parallel offshore of site (from east)	1.18 [10.8]	4.41 [48.4]	1.54 [12.8]	4.87 [60.0]	1.92 [15.2]	5.33 [74.2]

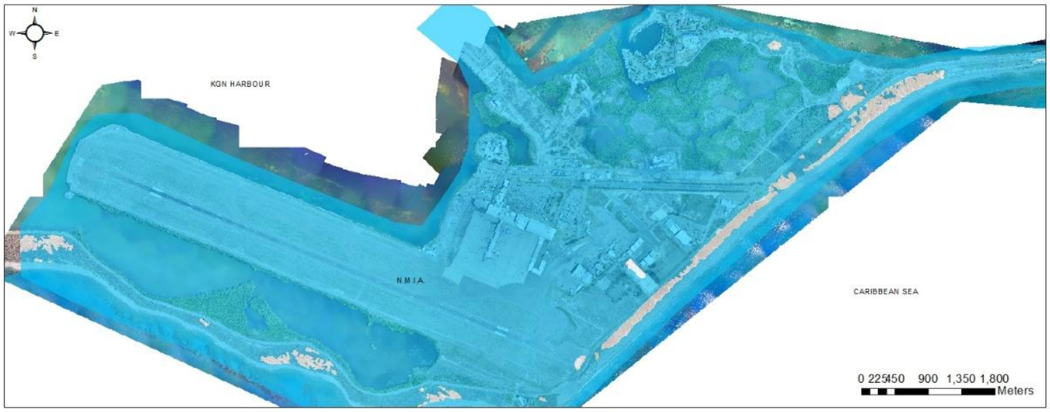
3.3.3.2 Summary

Four worst case scenarios of setup with run-up were investigated and it was found that the worst case scenarios had the following:

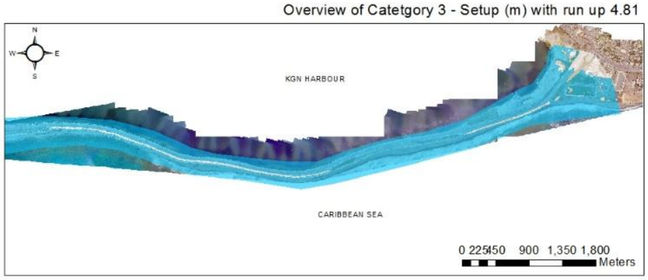
1. Category 3 elevations corresponding to return periods of 41.8 to 58 years;
2. Category 4 elevations corresponding to return periods of 56.7 to 81 years;
3. Category 5 elevations corresponding to return periods of 70 to 112.8 years;

The worst case scenario in all categories is a direct hit 40 km to the west of the project site.

Table 3.3 Inundation levels associated with the worst case storm surge modeling (Direct Hit CAT 3 X 40 KM west of site)

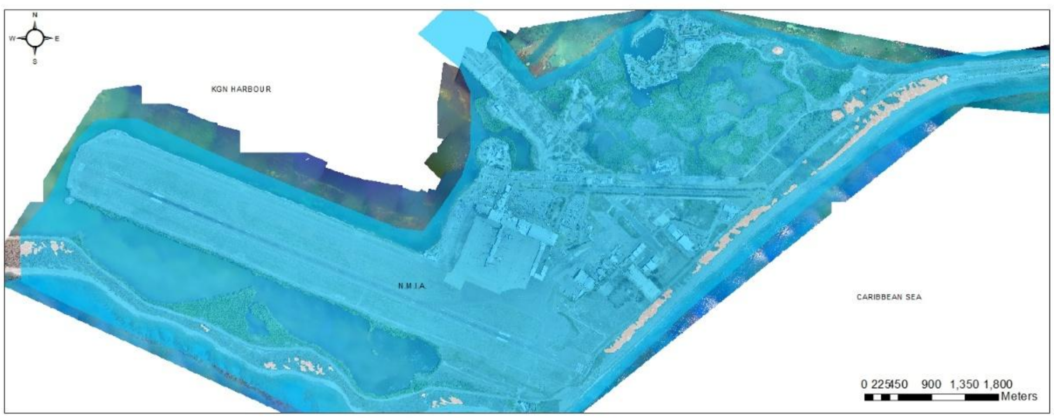


Overview of Catetgory 3 - Setup (m) with run up 4.81

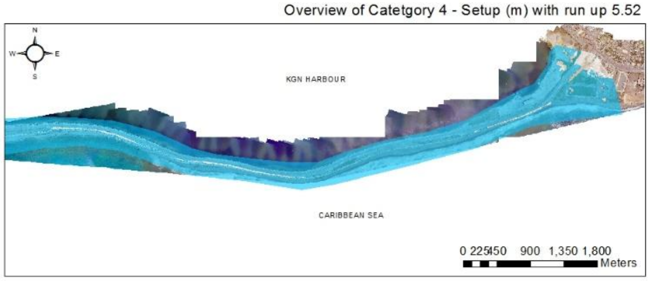


Overview of Catetgory 3 - Setup (m) with run up 4.81

(a) Category 3 Storm Event with runup

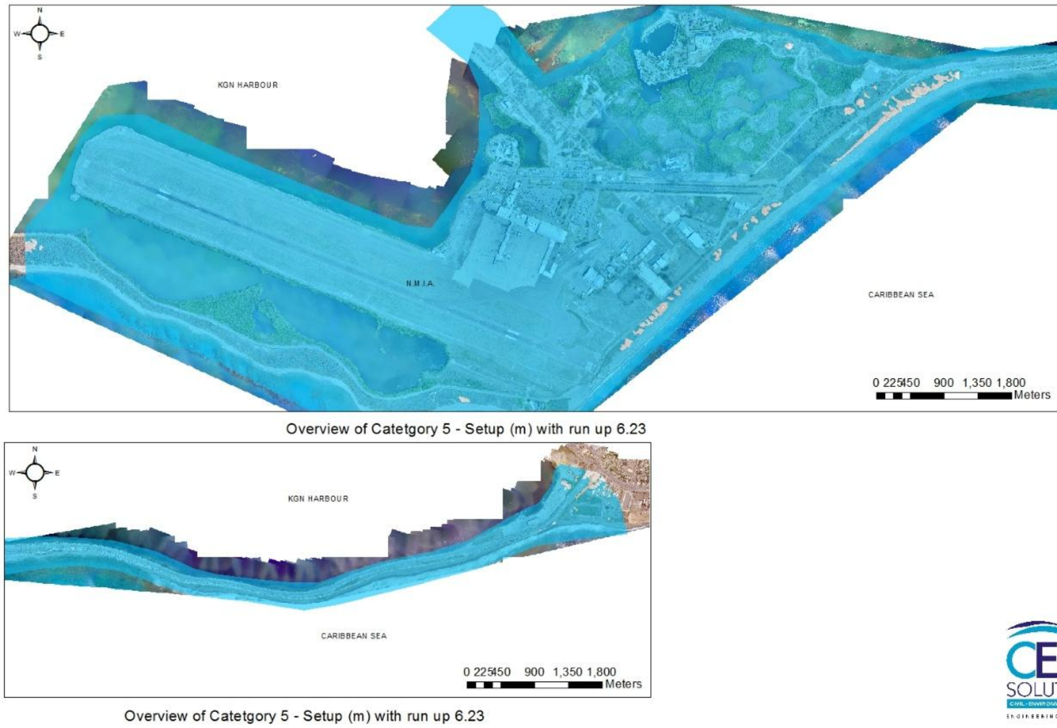


Overview of Catetgory 4 - Setup (m) with run up 5.52



Overview of Catetgory 4 - Setup (m) with run up 5.52

(b) Category 4 Storm Event with runup



(c) Category 5 Storm Event with runup

3.3.4 **CEAC Storm Surge Inundation Duration Analysis (2015)**

The National Hurricane Centre (NOAA) historical database of hurricane tracks throughout the Caribbean Sea was used to carry out this exercise. These hurricane tracks would have affected Jamaica throughout their occurrences. The database contained storm events from the years 1852 through 2012 with six (6) hour intervals. All storm events were then classified according to their occurrences and storm number to give a total of 103 storms.

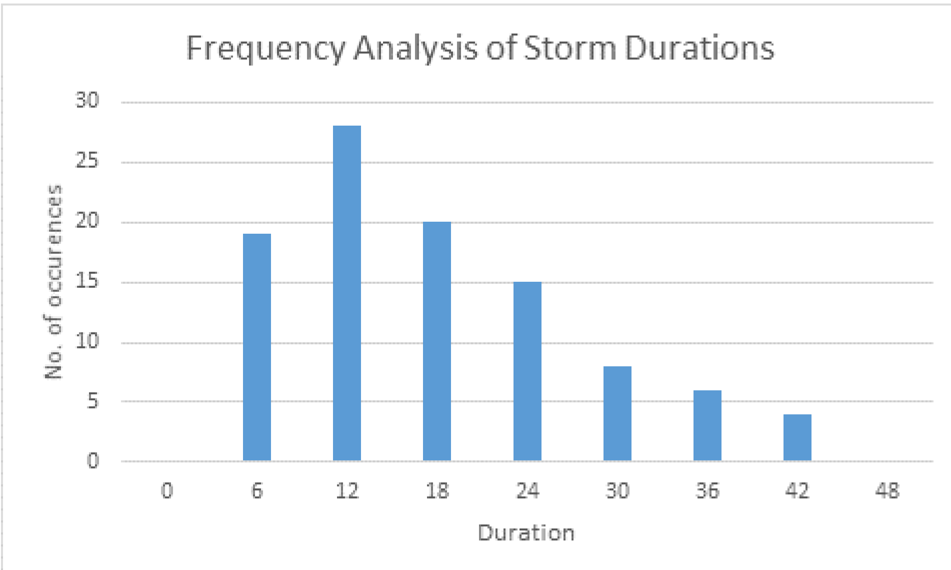
A frequency analysis was executed on each storm event to determine the exact number of periods (intervals) for which that given storm had occurred. Given the data was distributed in six (6) hour intervals, a total duration for any storm event could be established.

A frequency analysis was then executed on the storm durations determined previously in order to quantify the number of occurrences for each duration period. This implies classifying all duration periods at six (6) hour intervals as shown below in Table 3.4.

Table 3.4 Frequency analysis of storm durations from 1852-2012 obtained from NOAA.

Duration (hours)	No. of Occurrences
0	0
6	19
12	28

Duration (hours)	No. of Occurrences
18	20
24	15
30	8
36	6
42	4
48	0



It is observed that, based on the NOAA historical database of hurricane tracks throughout the Caribbean Sea, majority (28%) of the storm events occurred for only twelve (12) hours, 20% occurred for eighteen (18) hours and 19% occurred for six (6) hours. Eight percent (8%) of storm events occurred for thirty (30) hours while another six percent (6%) lasted thirty-six (36) hours.

3.3.5 Frequency Analysis using Bootstrapping Resampling Method

In regards to statistics, resampling refers to the estimation of the precision of sample statistics (Medians, variances, percentiles) by utilizing subsets of available data or drawing randomly with replacement from a set of data points. This methodology was employed due to the necessity for additional data to remove/limit biases in the existing data. This resampling methodology also allows the validation of models by using random subsets known as bootstrapping. The bootstrap procedure is recommended when:

a. When the theoretical distribution of a statistic of interest is complicated or unknown;
b. When the sample size is insufficient for straightforward statistical inference;
c. When power calculations have to be performed, and a small pilot sample is available.

One main advantage of bootstrap resampling is its simplicity in deriving estimates of standard errors and confidence intervals for complex estimators of complex parameters of the distribution, such as percentile points, proportions, odds ratio, and correlation coefficients.

3.3.5.1 Anecdotal Information

3.3.5.1.1 Methodology

Resampling by method of bootstrapping is a statistical method for estimating the sampling distribution of an estimator. This procedure involves choosing random samples with replacement from a data set and analyzing each sample the same way.

Sampling with replacement means that each storm surge observation was selected separately at random from the original dataset. Therefore, a particular data point from the original observation data set could appear multiple times in a given bootstrap sample. In the case of the anecdotal data collected in 2011 and 2017, 111 samples were generated for the exercise. For all samples generated, the number of elements in each corresponded to the number of elements found in the original data set. Each sample contained one hundred and eleven (111) elements representing the last year before available anecdotal data (1916) through to the year 2017. These elements were randomly generated as probabilities between 0 and 0.6. The range of sample estimates obtained enables the model to establish the uncertainty of the quantity which is being estimated.

The bootstrap distribution of the parameter-estimator was used to calculate confidence intervals for its population-parameter. Since the bootstrap distribution of an estimator is symmetric, then percentile confidence-interval can be appropriately used. The confidence intervals were estimated by using the empirical quantiles from the bootstrap distribution of the parameter while implementing L-moment analysis and estimation of distributions.

3.3.5.1.2 Results

We now obtain from our list of bootstrap sample means a confidence interval. Since we want a 90% confidence interval, we use the 95th and 5th percentiles as the endpoints of the intervals. The reason for this is that we split 100% - 90% = 10% in half so that we will have the middle 90% of all of the bootstrap sample means. For this example, a confidence interval of 0.65 to 5.63 was determined.

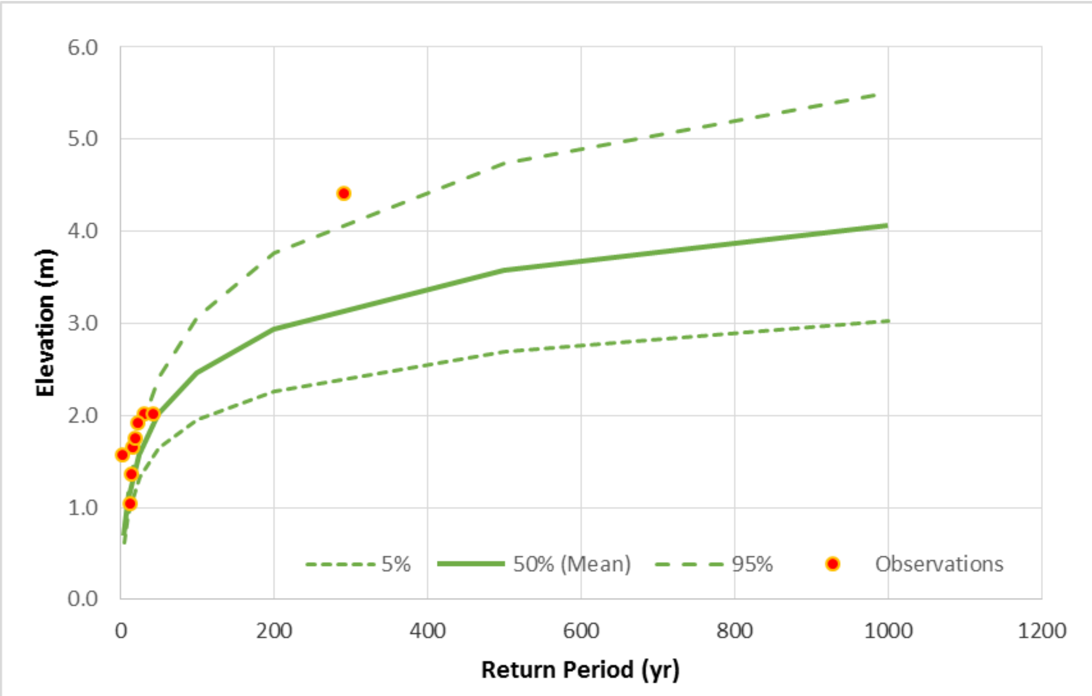


Figure 3.5 Bootstrap resampling of observations of storm surge for Palisadoes, Kingston

3.3.5.2 <u>Comparative Analysis of Model Results to Anecdotal Storm Surge</u>

3.3.5.2.1 Kolmogorov-Smirnov Goodness-of-Fit Test (K-S Test)

A test for goodness of fit usually involves examining a random sample from some unknown distribution in order to test the null hypothesis that the unknown distribution function is in fact a known, specified function. The Kolmogorov–Smirnov test can be modified to serve as a goodness of fit test. In the special case of testing for normality of the distribution, samples are standardized and compared with a standard normal distribution.

The two-sample K–S test is one of the most useful and general nonparametric methods for comparing two samples, as it is sensitive to differences in both location and shape of the empirical cumulative distribution functions of the two samples. The Kolmogorov-Smirnov test is defined by:

$$F_n(x) = \frac{1}{n}\sum_{i=1}^{n} (F(Y_i) - \frac{i-1}{n}, \frac{i}{n} - F(Y_i))$$

where, F is the theoretical cumulative distribution of the distribution being tested which must be a continuous distribution.

All values were converted to ranks which did not change the maximum difference between the cumulative frequency distributions. Thus, although the test analyzed the actual data, it is equivalent to an analysis of ranks and is fairly robust to outliers (similar to the Mann-Whitney test).

3.3.5.2.2 Conclusion

The results of the two-sample K-S test revealed that the distribution of the resampled anecdotal data collected and the results predicted by the model were sampled from populations of different distributions. The P-value determined from the analysis was calculated to be 0.078. The K-S test was completed with a 5% confidence limit (0.05), however, due to the P-value being greater than this implemented 5% limit it can be concluded that both samples are of two (2) different distributions. The anecdotal sample was selected as having the better distribution of the two (2) due to the narrower spread of data (less variance from mean values) (see Figure 3.6). The anecdotal data, in comparison with the analytical (model) results, was estimated to have mean values in the range 0.70m – 3.99m and standard deviations of 0.64m – 5.30m (see Table 3.5).

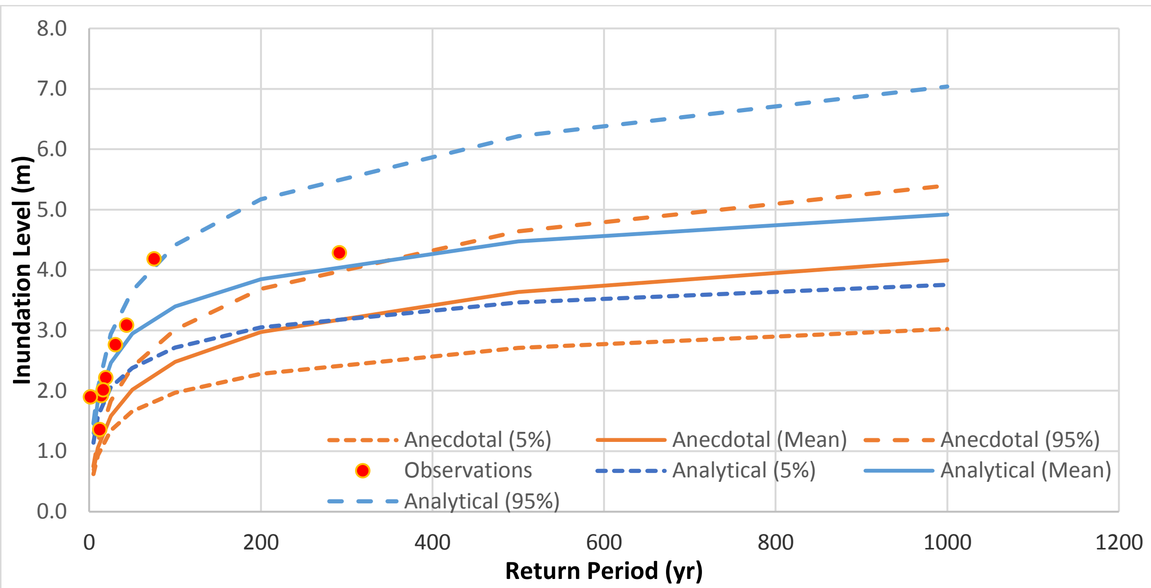


Figure 3.6 Comparison of analytical results and anecdotal observations for Palisadoes, Kingston

Table 3.5 Summary of resampled anecdotal observations

Return Period (yr.)	Mean (m)	Standard Deviation (m)	
		5% Confidence Limit	95% Confidence Limit
5	0.70	0.64	0.75
10	1.06	0.93	1.16
25	1.58	1.31	1.81
50	2.00	1.62	2.36
100	2.43	1.93	2.97
200	2.89	2.23	3.63
500	3.51	2.64	4.56
1000	3.99	2.95	5.30

4 Extreme Waves

4.1 Offshore at Selected Point

Waves were analysed in a similar manner as the storm surge results. The calculated results for extreme waves ranged from 9.74 m and 12.17m for the 50 and 100 year return period respectively. The annual maximum values of extreme waves were then fitted to a generalized type III extreme value (GEV) distribution to determine the return period wave heights. Bootstrap resampling was employed in order to improve the estimates of standard errors and confidence intervals for given the dataset is only 66 of over 166 years.

Just over 200 samples were generated for the exercise. For all samples generated, the number of elements in each corresponded to the number of elements found in the original data set. The range of sample estimates obtained enabled the model to establish the uncertainty of the mean values estimated.

The climate change considerations were also included in the exercise, they were the number of event per year (5) and a seven percent (7%) expected increase in intensity for the more intense events. The increase in intensity was applied to all the events exceeding the mean values in the sample. Similar to the storm surges the climate change scenario as predicted by the model show an increase in wave heights for the more frequent events and a reduction in wave heights for less events as a net effect of increasing the number of storms per year to 5 as well as applying the increase of 7 percent to the most intense storms. Final results for wave heights were also modified to reflect no change for the less frequent events. See Table 4.1 and Figure 4.1 below.

Table 4.1 Future climate extreme wave heights (meters) waves predicted for selected offshore point from Palisadoes.

RP	5%	Mean	95%
5	4.43	4.68	4.91
10	5.84	6.12	6.40
25	7.37	7.78	8.24
50	9.18	9.74	10.08
100	11.40	12.17	12.65
200	13.72	14.69	15.37
500	16.94	18.18	19.32
1000	19.48	20.98	22.45

It must be noted that these are deep water wave heights and further analysis is required to determine the nearshore wave heights.

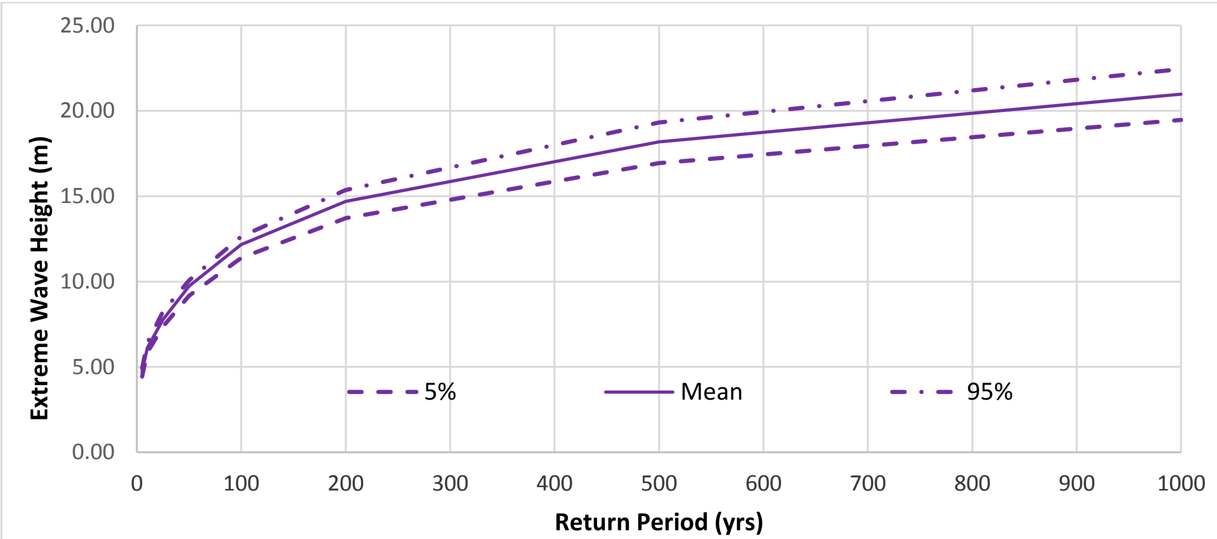


Figure 4.1 Future climate (2050) extreme deep water wave heights waves for offshore Palisadoes, showing 95%, mean and 5% limits.

4.2 Nearshore Wave Climate

4.2.1 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 shoreline
- Sea Side shoreline for pre and post project.
- Determine the impact of climate change along the 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.

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 Mike 21 SW 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 4.7 and Table 4.8 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.

4.2.2 Wave Climate Model: Mike 21 SW

The Mike 21 suite of computer programs was used to simulate the waves and currents in the area. The wave propagation and transformation from offshore up to the shoreline was calculated using the spectral wave component of Mike 21 (Mike 21 SW).

MIKE 21 SW is a third generation spectral wind-wave model based on unstructured meshes. The model simulates the growth, decay and transformation of wind-generated waves and swells in offshore and coastal areas (DHI 2004). MIKE 21 SW includes the following physical phenomena:

- Wave growth by action of wind
- Non-linear wave-wave interaction
- Dissipation due to white-capping
- Dissipation due to bottom friction
- Dissipation due to depth-induced wave breaking
- Refraction and shoaling due to depth variations
- Wave-current interaction
- Effect of time-varying water depth and flooding and drying

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

4.2.3 Model Setup

A finite element network for the study area was constructed using the bathymetric data collected by CEAC solutions. The model properties were as follows

- Unstructured mesh, 2952 elements and 1950 nodes
- Smaller grids sizes in against the shoreline
- Larger grids in deep waters offshore
- Open boundaries on the North, East and West
- Time steps of 360 seconds

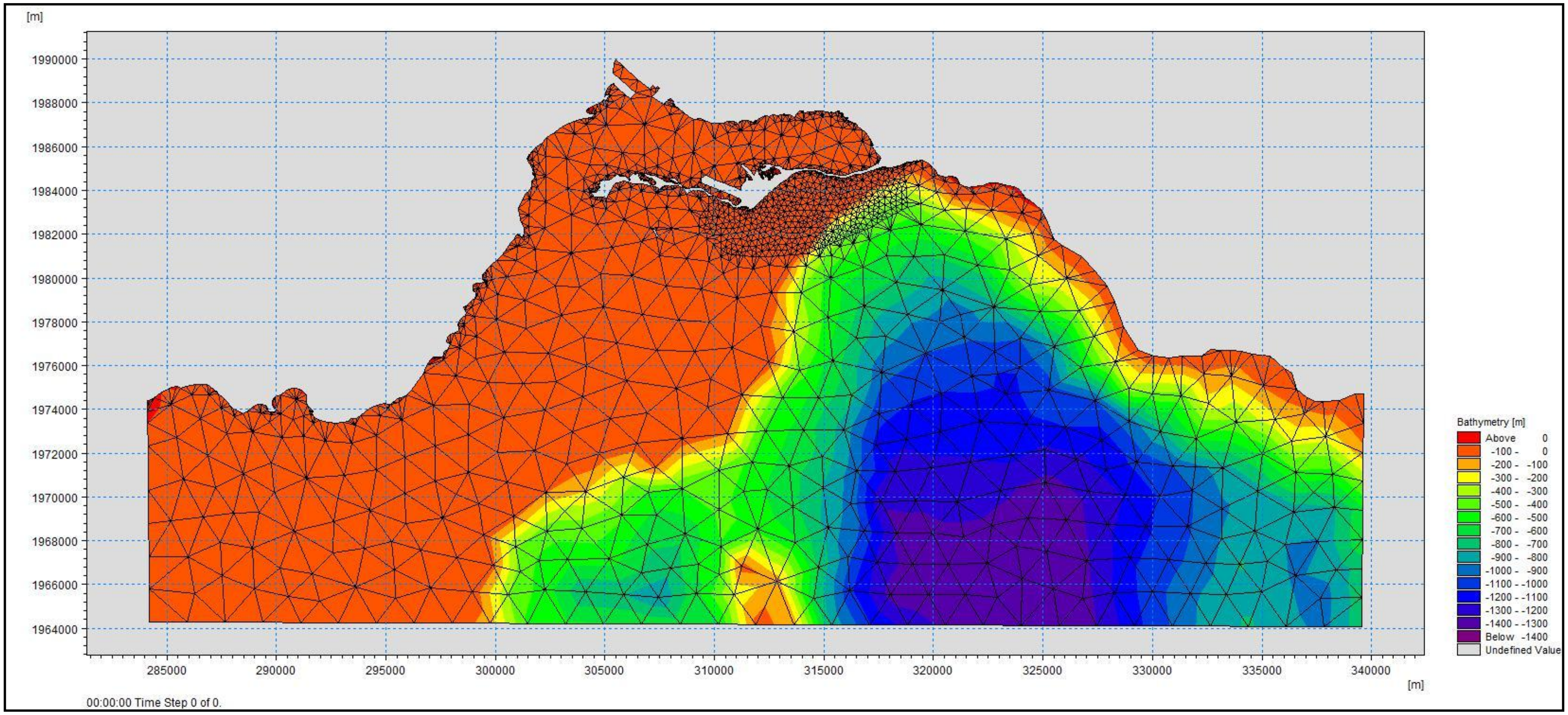


Figure 4.2 Mesh created for the Project area

4.3 Results

The project area is more vulnerable to hurricane waves than the remainder of the Palisadoes. Wave transformation analysis was done to track how the wave changes as it moves from deepwater to the shoreline. The analysis revealed that the wave heights nearshore ranges from 3.5m to 4.0m for the 100yr RP and 2.5m to 3.0m for the 50year RP (see Table 4.2 and Table 4.3). In essence, the end of runway and dune sections of the project area are exposed to wave heights of up to 3.6 to 4.0 meters. This section of shoreline is exposed to more hazardous wave conditions that the remainder of the Palisadoes, where wave height typically range from 2.4 to 3.2 meters. Revetment requirements are therefore more robust than that of Palisadoes shoreline protection works.

Table 4.2 summary of wave heights expected nearshore for the different directions and return periods

Future	100yr	50yr	25yr	10yr
E	3.6-4.0	2.8-3.0	1.8-2.0	1.0-1.2
SE	3.5-4.0	2.4-2.8	1.6-2.0	0.8-1.0
S	3.6-4.0	2.5-3.0	1.6-2.0	0.75-1.0
SW	3.6-4.0	3.2-3.6	2.2-2.4	1.6-1.8

Table 4.3 Contour plots of nearshore wave modelling results for 50year and 100 year RP respectively.

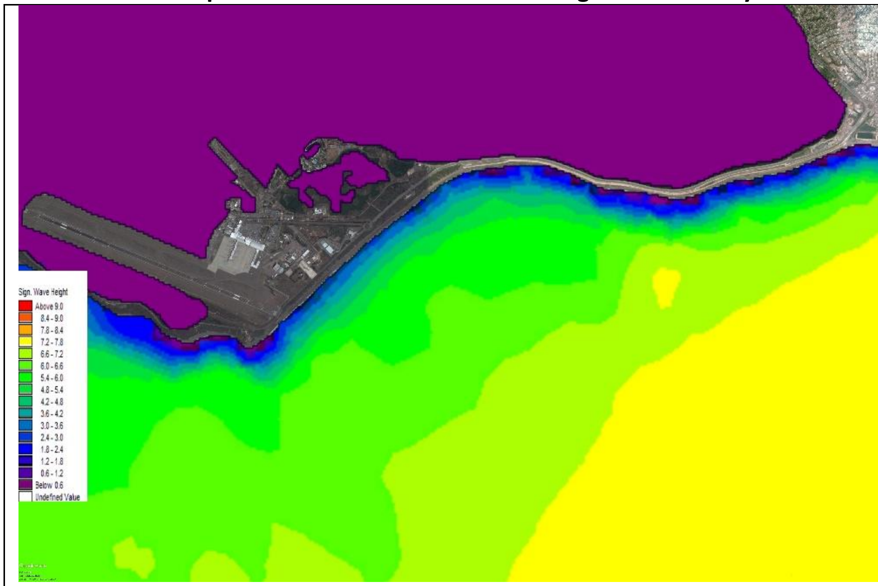


Figure 4.3 50 yr. Storm from the southern Approach

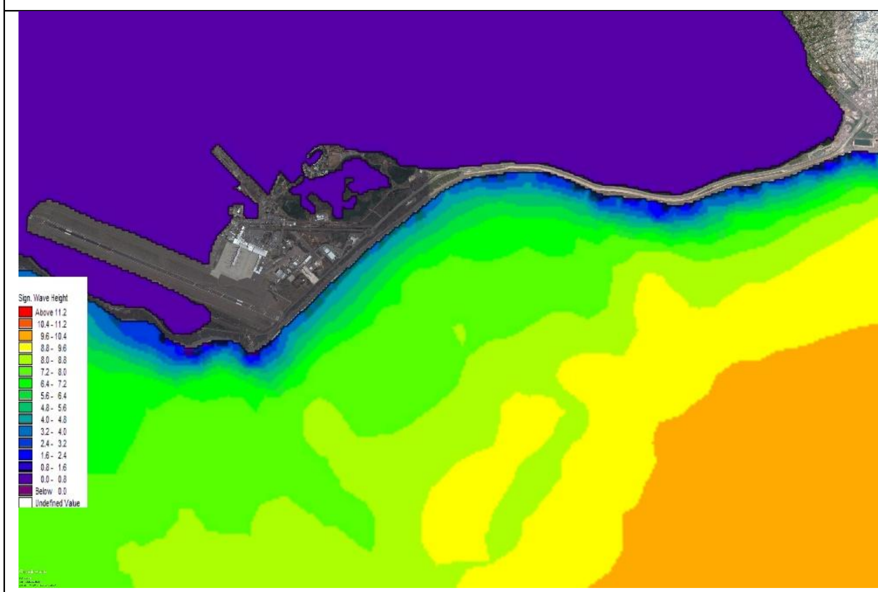


Figure 4.4 100 yr. Storm from the southern Approach

5 Shoreline Erosion

5.1 Short Term Erosion

5.1.1 Introduction

It is necessary to determine how the Palisadoes shoreline will respond to the severe wave climate anticipated. Estimates of how the beach will accrete or erode in response to particular storm events will also serve as a benchmark during the monitoring phase as well. The adopted approach was to utilize the cross-shore sediment transport model (SBEACH).

5.1.2 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 in comparison to the exposure times to historical storms.

5.1.3 Wave climate input

Profiles were cut from land to deep water (3000 km offshore) at 5 locations covering the affected areas of the shoreline. The wave data used in the model correspond to 50 and 100 year storm events. See Table 5.1. For input parameters.

Table 5.1 SBEACH input parameters for each storm events

Input Parameters			
Storm	**Hs (m)**	**Tp (s)**	**Wind Speed (m/s)**
50 YR	9.74	10.92	89.7
100 YR	12.17	12.21	111.4

5.1.4 Scenarios

SBEACH was used to determine the existing shoreline's response to a 50 year and 100 year storm event at 5 locations along the Palisadoes shoreline (see Figure 5.1). The locations are referred to as: Lighthouse, End of Runway, Dune, Low Revetment and High Revetment.

Figure 5.1 Location of the profile lines used in the storm surge model to determine the shoreline response to a 50 and 100 year storm event.

5.1.4.1 Pre-Project Scenario

The pre-project scenario considered storm induced beach change under the existing conditions. The locations considered in this scenario were limited to the project area. These locations, in particular, are the Lighthouse and the End of Runway. The remaining Dune, Low Revetment and High Revetment locations will remain unaffected by construction, hence, the pre-project results will remain valid after the completion of the project.

5.1.4.1.1 Results

It can be concluded that the shoreline in the vicinity of the Lighthouse is the most vulnerable to erosion based on the model with a maximum of 64m and 71m horizontal erosion for the 50 and 100 year return periods respectively. The shoreline in the vicinity of the end of runway can be expected to experience a maximum erosion of 42m and 61m inland, for the 50 and 100 year return periods respectively. The (sand) dune segment of the shoreline will experience the maximum erosion inland of all investigated locations. For the 50 and 100 year storm events, the erosion is estimated to be 139 and 142 respectively. Modeling of the high revetment along the Palisadoes did not yield any signs of erosion, however the low revetment did reveal signs of erosion. The maximum erosion of 33m and 40m inland, for the 50 and 100 year return periods respectively, can be expected.

Table 5.2 Summary of results showing the expected (worst case) shoreline erosion under pre-project conditions.

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (50 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse	64	1.7	71	2.3
End of Runway	34	0.9	61	2.3
Dune	139	0.4	142	0.8
Low Revetment	33	0.8	33	1.3
High Revetment	0	0	0	0

5.1.4.2 Post Project Scenario.

The post project scenario considered storm induced beach change after the revetment has been implemented. The locations considered in this scenario were limited to the project area where construction of the revetment is expected to undergo. These locations, in particular, are the Lighthouse and the End of Runway. The remaining Dune, Low Revetment and High Revetment locations will remain unaffected by construction, hence, the pre-project results will remain valid after the completion of the project.

5.1.4.2.1 Results

It can be concluded that the dune segment of the Palisadoes (east of end of runway) is the most vulnerable to erosion, given the proposed shoreline protection. Based on the model, the shoreline erosion in the vicinity of the lighthouse will be reduced to a maximum of 26m horizontal erosion for both the 50 and 100 year return periods. The shoreline in the vicinity of the end of runway can also be expected to reduce to a maximum erosion of 32m and 31m inland, for the 50 and 100 year return periods respectively.

Table 5.3 Summary of results showing the expected (worst case) shoreline erosion under post-project conditions

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (50 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse	26	0.7	26	0.9
End of Runway	32	0.2	31	0.4
Dune	139	0.4	142	0.8
Low Revetment	33	0.8	33	1.3
High Revetment	0	0	0	0

5.1.5 **Summary**

The SBEACH model was used to determine how the beach will respond to severe wave climates. The model indicated that under existing climate conditions, the shoreline at the **Lighthouse** and **End of Runway** would experience the greatest damage with a maximum vertical erosion of 1.7 and 1.3 m respectively for a 50 year storm event. This erosion corresponds to an extent of 64m and 35m inland, respectively. In the event of a 100 year storm The **Lighthouse** and **End of Runway** are expected to experience 2.7 m and 2.3 m of erosion, with extents of 71m and 61m respectively. The Shoreline at the **Dune** is anticipated to experience minor vertical erosion, however the inland reach of erosion was greatest at this location for both the 50 year (139m) and 100 year storm (142m). The **Low Revetment** is expected to experience a maximum of 1.3m and 1.4m of erosion at the base of the revetment for the 50 and 100 year storm event respectively. This erosion corresponds to an extent of 33m and 40m inland, respectively. The **High Revetment** shoreline is anticipated to experience minimal to no damage for both the 50 and 100 year storm event (see Figure 5.2).

In the event of a 50 year storm The **Lighthouse** and **End of Runway** are expected to experience a reduced 0.7 m and 0.2 m of erosion, with extents of 26m and 32m respectively. In the event of a 100 year storm The **Lighthouse** and **End of Runway** are expected to experience a reduced 0.9 m and 0.4 m of erosion, with extents of 26m and 31m respectively. These scenarios can be expected at the end of project completion (see Figure 5.3).

During the pre-project scenario, during both the 50 and 100 year storm event, the erosion is expected to reach the Palisadoes main road just west of the end of runway. The 100 year storm event can be expected to cause erosion to the entire reservation of roadway whereas the 50 year event will merely erode up to the edge of the road reservation.

Figure 5.2 Erosion plots for both 50 year and 100 year return periods (pre-project scenario)

Figure 5.3 Erosion plots for both 50 year and 100 year return periods (post-project scenario)

5.2 Long Term Erosion

5.2.1 Methodology

Long term shoreline change was determined from 1968 to 2017 from the shoreline positions along the Palisadoes and compared in order to determine the spatial and temporal erosion trends. This was important in order to identify the high risk areas that are erosion hotspots and in order to verify the wave transformation modelling. Long-term erosion trends may be estimated in two ways:

1) Observation of actual long-term shoreline movement involves estimating the past rate of change in the shoreline from aerial photography.
2) Implication from global sea level rise involves using an estimated rate of global sea level rise and an erosion model to obtain the predicted long-term erosion trend.

5.2.2 Historical Shoreline

Figure 5.4 shows the observed shorelines from Satellite and aerial imagery for the years 1968, 1992, 1999 and 2005. The most recent (January 2014) shoreline position was marked manually by walking the beach and taking GPS points. The shoreline movement was analyzed by measuring and noting the displacements of the shoreline at intervals of 100m along the shoreline. The rates of accretion and or erosion between the time intervals and the overall time interval were determined using the following two (2) relationships:

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

where,

E – the rate of erosion or accretion between two successive intervals (meters per year);

D – the displacement between two intervals (meters);

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;

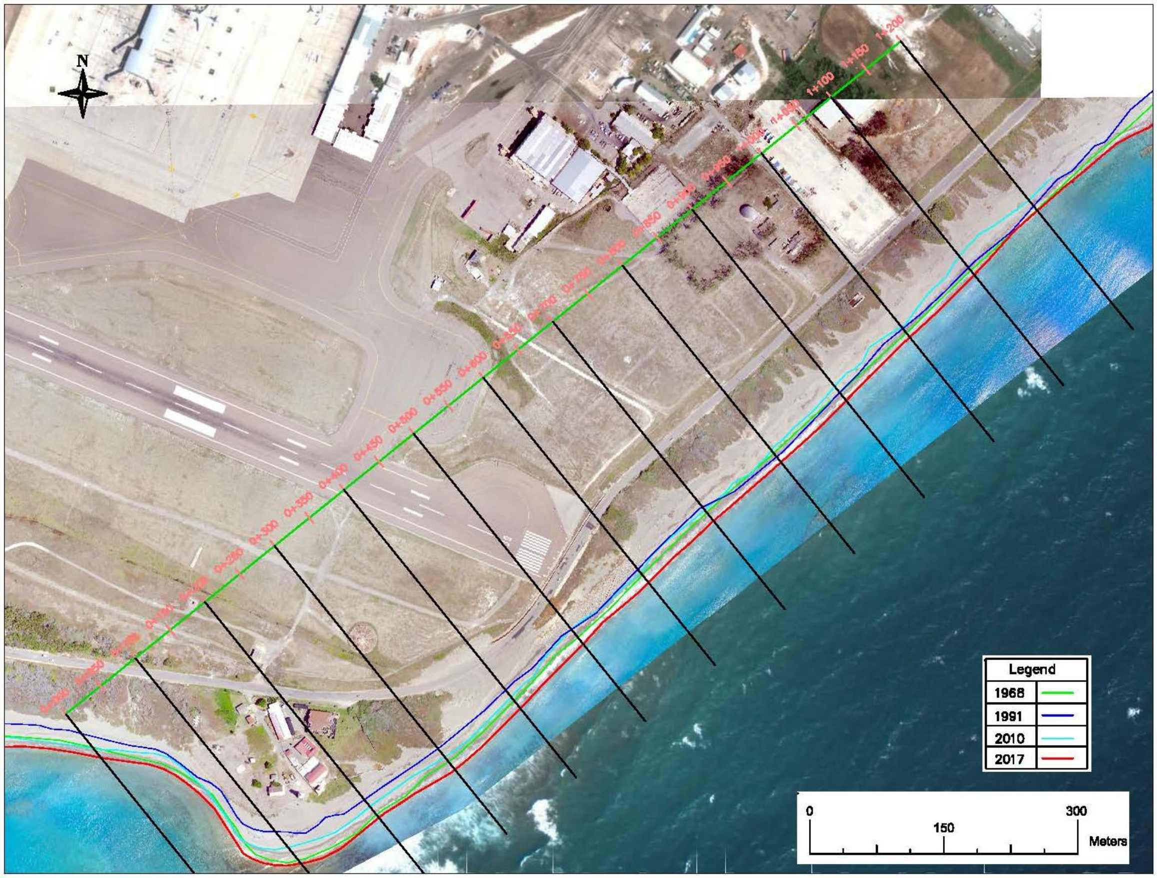


Figure 5.4 Historical Shoreline Changes for Project Area (1968, 1991, 2010, and 2017).

5.2.2.1 Results

A summary of the analysis data is shown below in Table 5.4. From the analysis we see a general trend of accretion occurring from 1968 to 2017. Some erosion was observed between 1968 and 1991 as well as between 1991 and 2010. Plots of the shoreline movement can be seen in Table 5.4 to Table 5.5 below. One can see that over the past 49 years, a maximum around of 55.01 meters of erosion and 13.74 meters of accretion that has occurred along the Palisadoes shoreline based on the observation of historical areal and satellite images of the area.

1968 to 1991

- Number of hurricanes in this interval:
- The shoreline eroded a maximum rate of 1.787 m/year
- The overall change in the shoreline showed an average erosion rate of -0.573 m/year

1991 to 2010

- Number of hurricanes in this interval:
- The shoreline had a maximum accretion rate of 1.137 m/year and maximum erosion rate of 0.505m/year.
- The overall change in the shoreline showed an average accretion rate of 0.285 m/year

- The trend along the shoreline shows accretion occurring at the western end of the shoreline and erosion at the eastern end. This is indicative of sediment being transported from the eastern to the western end of the shoreline.

2010 to 2017

- Number of hurricanes in this interval:
- The shoreline had a maximum accretion rate of 3.35 m/year with no determined erosion.
- The overall change in the shoreline showed an average accretion rate of 2.406 m/year.

Table 5.4 Summary of shoreline change between 1968 and 2017 for Palisadoes, Kingston

	1968	1991			2010			2017			Overall	
Location	Datum	Process	Accretion / Erosion Rate (m/year)	distance from datum (m)	Process	Accretion / Erosion Rate (m/year)	distance from datum (m)	Process	Accretion / Erosion Rate (m/year)	distance from datum (m)	Process	Rate (m/year)
0+000	47.63	erosion	-0.863	27.78	accretion	1.137	49.39	accretion	1.91	62.76	accretion	0.309
0+100	290.56	erosion	-1.787	249.46	accretion	1.18	271.88	accretion	3.35	295.33	accretion	0.097
0+200	305.84	erosion	-0.942	284.18	accretion	0.519	294.05	accretion	2.307	310.2	accretion	0.089
0+300	310.14	erosion	-0.814	291.41	accretion	0.638	303.54	accretion	2.764	322.89	accretion	0.26
0+400	303.29	erosion	-0.673	287.81	accretion	0.536	298	accretion	2.344	314.41	accretion	0.227
0+500	292.75	erosion	-0.498	281.29	erosion	-0.018	280.94	accretion	3.043	302.24	accretion	0.194
0+600	283.74	erosion	-0.36	275.47	accretion	0.238	280	accretion	2.087	294.61	accretion	0.222
0+700	274.54	erosion	-0.34	266.71	accretion	0.203	270.57	accretion	2.26	286.39	accretion	0.242
0+800	269.77	accretion	0.227	274.98	erosion	-0.505	265.39	accretion	2.47	282.68	accretion	0.263
0+900	263.54	erosion	-0.01	263.31	erosion	-0.481	254.18	accretion	2.933	274.71	accretion	0.228
1+000	258.2	erosion	-0.282	251.71	erosion	-0.122	249.4	accretion	2.173	264.61	accretion	0.131
1+100	248.96	erosion	-0.877	228.8	accretion	0.841	244.78	accretion	1.473	255.09	accretion	0.125
1+200	242.27	erosion	-0.231	236.95	erosion	-0.455	228.31	accretion	2.163	243.45	accretion	0.024

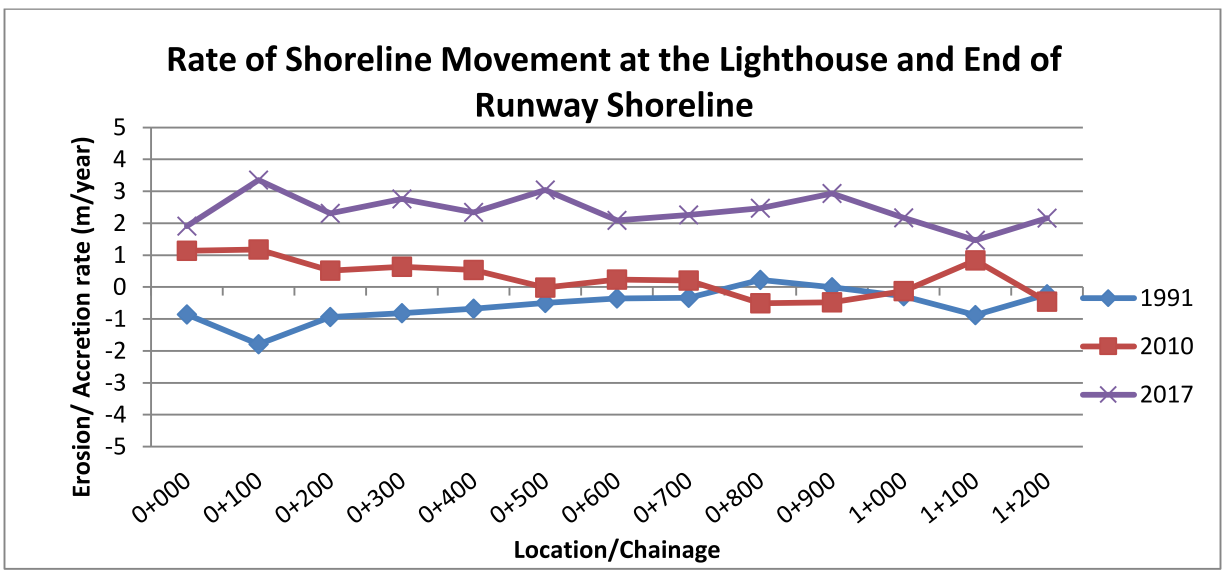


Figure 5.5: Rate of Shoreline Erosion/Accretion at the End of Runway Shoreline.

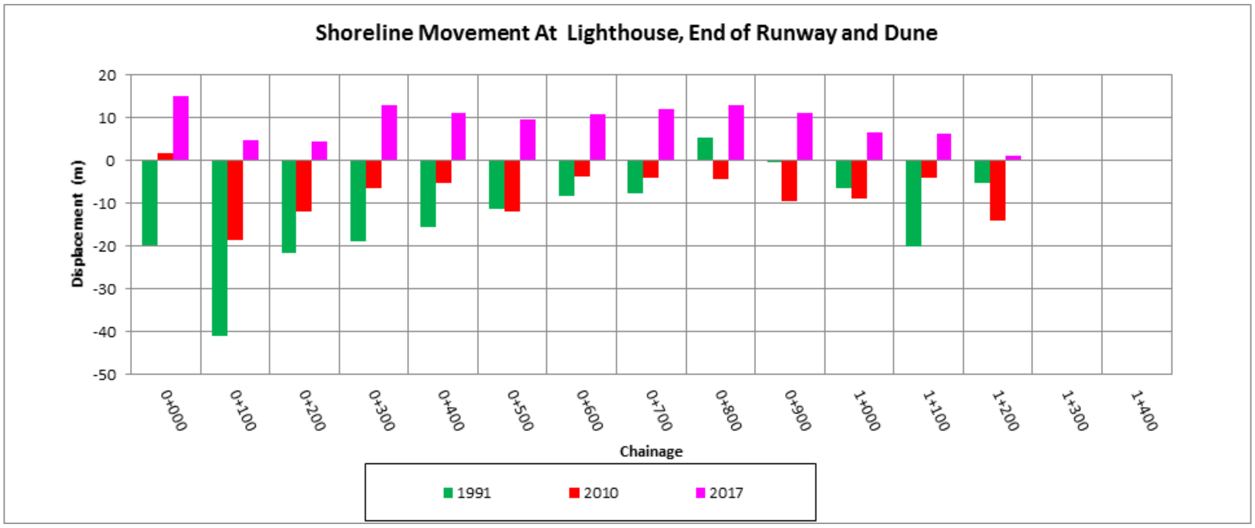


Figure 5.6 Displacement of the Palisadoes shoreline for different years about the 1968 shoreline (1991 to 2017)

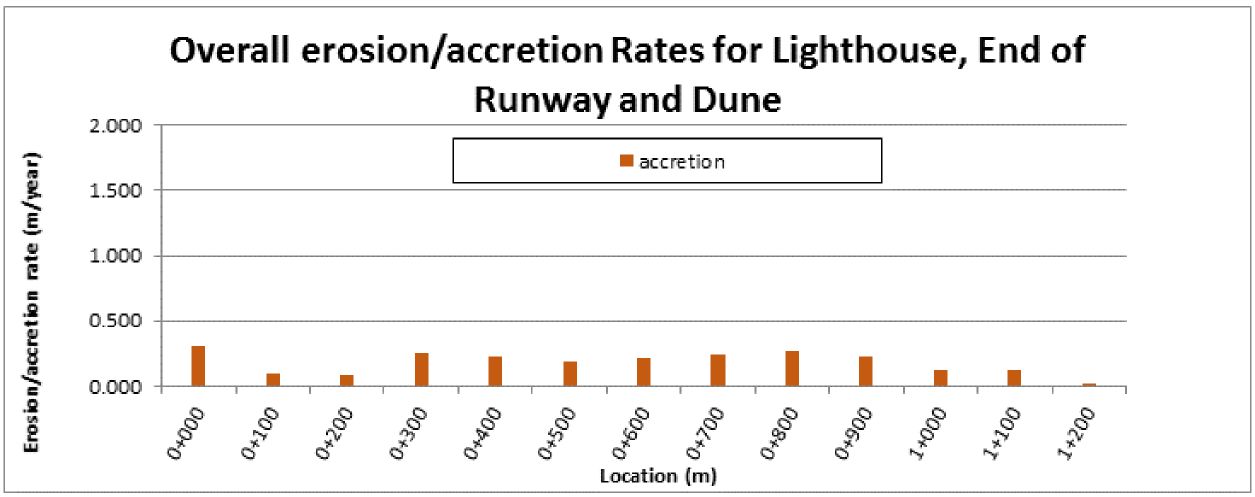


Figure 5.7 Overall erosion/ accretion rates for the Palisadoes shoreline during the 1968-2017 period

5.2.3 Future Shoreline Projections without Project

The 2050 shoreline was projected relative to the datum by applying the overall rate of erosion determined from the historical shoreline analysis (see **Error! Reference source not found.**). This projection highlights the fact that the entire shoreline will undergo accretion. The overall maximum accretion expected to occur could be as much as 10.19 meters over the next 33 years.

	2050 projection		
Location	Process	overall Rate of erosion (m/year)	distance from 2017 datum (m)
0+000	accretion	0.309	10.19

0+100	accretion	0.097	3.21
0+200	accretion	0.089	2.94
0+300	accretion	0.260	8.59
0+400	accretion	0.227	7.49
0+500	accretion	0.194	6.39
0+600	accretion	0.222	7.32
0+700	accretion	0.242	7.98
0+800	accretion	0.263	8.69
0+900	accretion	0.228	7.52
1+000	accretion	0.131	4.32
1+100	accretion	0.125	4.13
1+200	accretion	0.024	0.79

5.2.4 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 is a result of the deposited sediment and is equal to the rise in sea level.

The original Bruun model is expressed below in Adger,W.N., Agrawala, S., Mirza, M.M.Q., Conde, C., O'Brien, K., Pulhin, J., Pulwarty, R., Smit, B., Takahashi, K., 2007. Assessment of adaptation practices, options, constraints and capacity. In: Parry, M.L., Canziani, O.F., Palutikof, J.P., van der Linden, P.J., Hanson, C.E. (Eds.), Climate Change 2007: Impacts, Adaptation and Vulnerability. Contribution of Working Group II to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change. Cambridge University Press, Cambridge, UK, pp. 717-743., and this mathematical relationship was the basis for estimating shoreline retreat within the study area.

Equation 5-1 – Bruun model

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

Where:

Parameter	Description	Units
Δy	Dune line erosion	m
Δs	Rate of sea level rise	m

*l**	Length of the offshore profile out to a supposed depth, h*, of the limit of material exchange from the beach and the offshore	m
*h**	Depth at offshore limit of l*, to which nearshore sediments exist (as opposed to finer- grained continental shelf sediments)	m

5.2.4.1 Rate of sea level rise, Δs

Inspection of research in this area revealed that global sea level may rise as a result of greenhouse gas-induced global warming at a rate of 3.7 mm/year over the next 100 years. 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 nor any discernible change in the ocean geodynamic surface). Typically, a mid-range or upper estimate is chosen for such types of scenarios.

5.2.4.2 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 Adger,W.N., Agrawala, S., Mirza, M.M.Q., Conde, C., O'Brien, K., Pulhin, J., Pulwarty, R., Smit, B., Takahashi, K., 2007. Assessment of adaptation practices, options, constraints and capacity. In: Parry, M.L., Canziani, O.F., Palutikof, J.P., van der Linden, P.J., Hanson, C.E. (Eds.), Climate Change 2007: Impacts, Adaptation and Vulnerability. Contribution of Working Group II to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change. Cambridge University Press, Cambridge, UK, pp. 717-743.below.

Equation 5-2 – Hallermeier estimation of critical or closure depth

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

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 between the shoreline and the reefs in order to estimate the critical depth.

The operational/swell wave climate was obtained from a previous in-house study of a nearby site the 12 hour significant wave height was estimated at 2.5m.

5.2.4.3 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 each of the profiles cut for the SBEACH modelling and obtaining profile lengths for the corresponding critical depth. These profile lengths obtained were incorporated into Equation 5-1.

5.2.4.4 Calculation and Results

Table 5.5 shows the calculation of the long term trends expected in 25 years along the Palisadoes shoreline. 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 four (4) profile shoreline positions:

- Rate of sea-level rise = 3.7 mm/yr. (IPCC, 2007)
- Depth to which near shore sediment exists (h *, d_c) = 2.56 m

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 and storm induced erosion.

The shoreline along the study area was estimated to retreat at varying rates between 0.09 and 0.16 meters per year as a result of global sea level rise. The shoreline in the vicinity of the lighthouse, low revetment and high revetment has the largest erosion rate of 0.16 m/year.

Table 5.5: Projected Shoreline Change along the Palisadoes Main Road.

Parameter	Profile					
	1	2	3	4	5	6
	Lighthouse	End of Runway	Dune	Low Revetment	High Revetment	Donald Quarrie
Rate of sea level rise, Δs (m/yr.)	0.0037	0.0037	0.0037	0.0037	0.0037	0.0037
Offshore profile, l* (m)	167.85	131.52	98.87	174.46	172.09	144.18
depth of offshore limit, h* (m)	4	4	4	4	4	4
Dune line Erosion, Δy (m)	0.16	0.12	0.09	0.16	0.16	0.13
Projected change in 25 years (m)	**3.88**	**3.04**	**2.29**	**4.03**	**3.98**	**3.33**
Projected change in 50 years (m)	**7.76**	**6.08**	**4.57**	**8.07**	**7.96**	**6.67**

It can be concluded that the estimated accretion rates have been slowed down due to ongoing erosion, based on the historical overall shoreline accretion trend as well as the projected erosion due to sea level rise. This simply means that in the absence of sea level rise, the accretion rates would have been greater while the opposite for rates of erosion.

With focus on the project area, it is estimated that sea level rise accounts for an average of 62.3%, 36.7% and 40.4% of sand lost during the accretion process for the Lighthouse, End of Runway and Dune shoreline (west of runway) respectively. See Table 5.6 below.

Table 5.6 Illustrating the effect of sea level rise (SLR) on overall accretion rates of shoreline within project area

Parameter	Lighthouse	End of Runway	Dune	Unit
Historical rate of Accretion with SLR	0.097	0.208	0.169	m/year
Historical rate of Accretion without SLR	0.257	0.328	0.259	m/year
% Accretion lost to SLR	62.3%	36.7%	40.4%	

Applying the projected erosion due to sea level rise, the shoreline within the project area can estimated as follows (year 2050):

Table 5.7 Projections for shoreline in the year 2050

Parameter	Lighthouse	End of Runway	Dune	Unit
Projected accretion (2017 shoreline datum)	10.19	7.32	8.69	m
Dune line Erosion due to SLR	0.155	0.122	0.091	m/year
Projected total erosion in 33 years (m)	5.12	4.01	3.02	m
Projected total accretion (2017 shoreline datum)	5.07	3.31	5.67	m

5.2.4.5 Limitations

Both methods of estimating long term erosion trends have their own limitations. For the Bruun 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 display long term. The accuracy of the rates is therefore subjected to the use of more aerial photos at strategic times which were not available at the time of this study.

5.2.5 Comparison to other beaches across Jamaica

A study of beaches across the island (Burgess & Johnson, 2013) looked at prevailing erosion rates and risks associated with the continuation of these patterns up to 2030. Specifically, nine (9) beaches were analyzed to determine their historical erosion rate and the influence of sea level rise versus storm induced erosion:

1. Plum Point
2. Long Bay (Portland)
3. San-San
4. Fort Clarence
5. Old Harbour Bay
6. Little Ochi
7. Priory
8. Annotto Bay
9. Long Bay Beach (Negril)

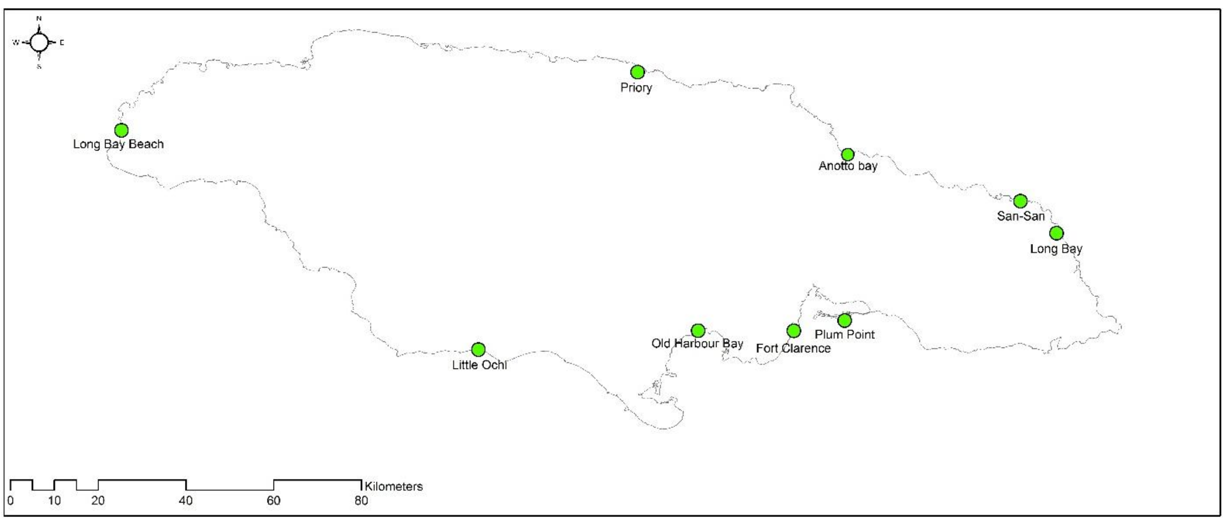


Figure 5.8 Location of beaches used in comparison

Short-term analysis revealed that eight (8) of the nine (9) beaches experienced short-term erosion varying between 0.1 to 0.52 meters per year. Only Little Ochi beach in St. Elizabeth exhibited accretion of the shoreline (see Table 5.8). The average short-term erosion rate observed was 0.26 meters per annum. Long-term shoreline retreat rates that are more relevant to this study were observed to vary between 0.17 to 0.76 meters per annum, with an average of 0.26 meters per annum. The fastest eroding beaches were observed to be the Long Bay Beach (Negril) at 0.76 meters per annum followed by the Old Harbour Bay (St. Catherine) at 0.74 meters per annum.

Table 5.8 Summary of analysis for the nine (9) beaches selected for the period 1968 to 2010, (Burgess & Johnson, 2013)

Beaches	Short-term rate of shoreline loss (m/ yr.)	Long-term rate of shoreline loss (m/ yr.)	Length of beach (m)	Interval between profile (m)	Number of profiles used	Location/ Parish
Long Bay, Portland	-0.52	-0.36	1400	200	8	Portland
Priory, St. Ann	-0.10	-0.08	1000	200	11	St Ann
Fort Clarence, St. Catherine	-0.48	-0.42	1250	250	4	St Catherine
Old Harbour Bay Fishing Beach, St. Catherine	-0.59	-0.74	1000	200	6	St Catherine
Little Ochi, St. Elizabeth	0.57	0.61	3000	500	4	St Elizabeth
Negril, Westmoreland	-0.56	-0.76	5000	500	6	Westmoreland
Annotto Bay, St. Mary	-0.08	-0.25	3633	200	7	St Mary
San San, Port Antonio	-0.38	-0.17	1600	500	8	Portland
Plum Point, Palisadoes, Kingston	**0.18**	**0.10**	**1200**	**200**	**8**	**Kingston**
Overall average	**-0.22**	**-0.23**				

6 Proposed Shoreline Protection

6.1 Design Criteria and Parameters

6.1.1 JCAA Requirements

The NMIA runway has two classifications; the north end/runway 12 is a Precision Code 1 approach whereas the southern/runway 30 is a Non precision - code 4. The requirements for the southern section of requires the following for us to define the vertical and horizontal extents of the OLS:

- Vertical – offset 60m from end of runway then apply 2% increase in elevation outwards
- Horizontal – offset 150m on either side parallel to the runway centerline and 60m from end of runway, after which a 15 degrees divergence is applied.

Table 6.1 Section of table 8-1 taken from JCAA manual of Aerodrome Standards

INNER APPROACH										
Width	-	-	-	-	-	-	-	90m	120m*	120m*
Distance from threshold	-	-	-	-	-	-	-	60m	60m	60m
Length	-	-	-	-	-	-	-	900m	900m	900m
Slope	-	-	-	-	-	-	-	2.5%	2%	2%
APPROACH										
Length of inner edge	60m	80m	150m	150m	150m	300m	300m	150m	300m	300m
Distance from threshold	30m	60m	60m	60m	60m	60m	60m	60m	60m	60m
Divergence (each side	10%	10%	10%	10%	15%	15%	15%	15%	15%	15%
FIRST SECTION										
Length	1600m	2500m	300m	300m	2500m	3000m	3000m	3000m	3000m	3000m
Slope	5%	4%	3.33%	2.5%	3.33%	2%	2%	2.5%	2%	2%

The project area falls within two zones of the ICAO/JCAA guidelines. Namely:

1. the inner edge; and
2. the 1st section of approach.

In addition to these requirements, additional safety measures were considered separately including tolerances of boulder placement. The resulting restrictions are as follows:

Table 6.2. OLS restrictions on crest elevation based on a run way threshold centerline elevation of 5.2 meters above MSL.

	Between threshold and inner edge	From inner edge to 1st section of approach
Width	60 meters	Varies: 60 to 3000 meters
Length	150 x 2 = 300 meters	
Slope	0	2%
Divergence angle	0	15%

Restriction on elevation	5.2 meters	
Distance from inner edge:		
50 meters		6.2 meters
100 meters		7.2 meters

6.1.2 Revetment Works

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

6.1.2.1 Return Period Interval: Wind and Wave Intensities

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

Table 6.3. Design Criteria recommendation for a design life of 30 to 100 Years

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

6.1.2.2 Functional Requirements

The Revetment proposed herein is immediately adjacent to the runway fill and the service road. Safe passage of persons in this location during a storm is not necessarily required, however it is necessary for damage to the pavement be minimized to acceptable levels in minor storms (i.e. <10 Year Return Period) and in the design storm event (i.e. <100 Year Return Period).

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

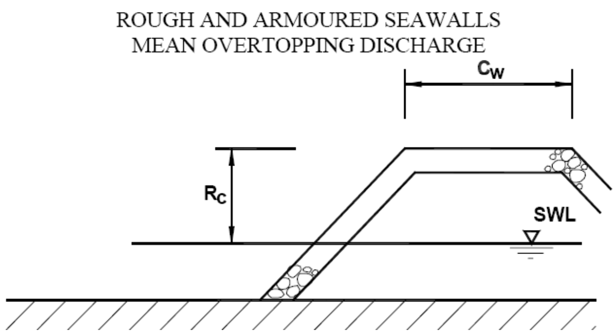


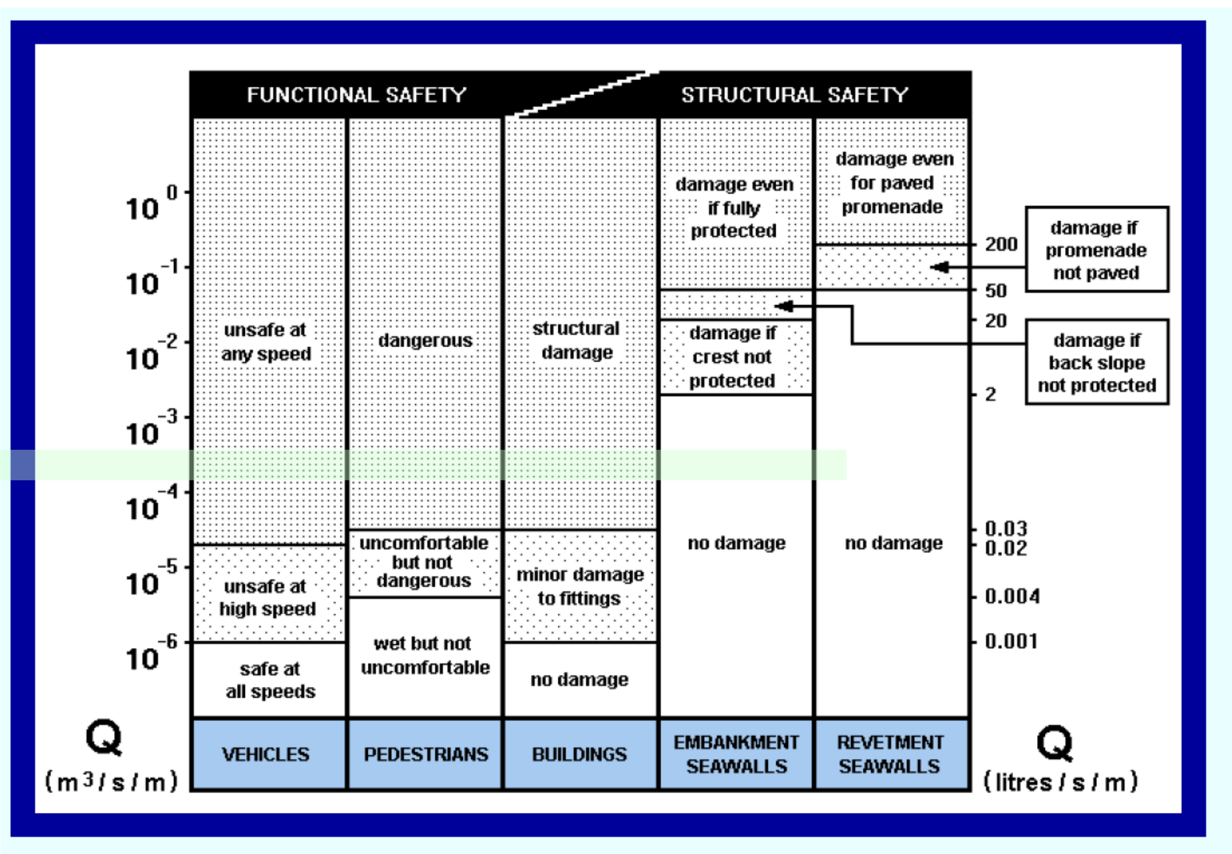
Figure 6.1 – Conceptual cross section of revetment used in estimating overtopping (H. R. Wallingford, 1999)

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

- Mean discharge in 100 Year Return Period = 0.05 cubic metres per metre or less

See Table 6.4 for reference.

Table 6.4 - Overtopping design criteria (H. R. Wallingford, 1999)



6.1.2.3 <u>Locally Availably Material Properties</u>

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

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

6.1.2.4 Damage Level

There will always be some movement of the stones in the armor structures, even in wave conditions less intense than the design wave conditions. The amount of movement can be qualitatively estimated or considered in the design phase of a project and it is a critical design input of the armor sizing models. The amount of tolerable movement anticipated in the design is reflected in the Damage Level (S). The less damage required or desired, is the more expensive the structure as the cross-section becomes thicker and the stones become larger. See Table 6.5.

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

Table 6.5 Damage levels for armour structures

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

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

6.2 Structural Design

6.2.1 Overtopping Analysis

Several areas along the Palisadoes were chosen to determine the overtopping rates, if any, that would be imminent during a storm event. The revetment crest was determined by conducting overtopping analysis. A layered rock structure was considered, with a roughness coefficient ranging from 0.55 to 1. The storm surge for the different scenarios vary from 1.68 m from hurricane Allen to a maximum of 2.37m for the 100yr return period.

The analysis indicates that with the proposed crest elevation (5 metres) of the dune revetment, the overtopping will be limited to a desirable rate of less than 0.03 cubic metres per meter of structure length. For the proposed revetment adjacent to the end of runway, the determined crest elevation of 4.83 metres will reduce overtopping to 0.01 cubic metres per meter of structure length.

Overtopping along the newly constructed revetment along the Palisadoes main road was also investigated. The existing Palisadoes revetment consists of a low and high revetment with crest elevations of 4.16 and 6.4 metres respectively. The overtopping rates were determined to be 3.06 and 0.3 cubic metres per meter of structure length for the low and high revetment respectively.

The initially proposed works for the Palisadoes low revetment included constructed a bermed dune supplementary to the low revetment. This second phase for the low revetment did not materialize due to unknown issues. However, the proposed dune revetment was modeled nonetheless, where the results provided insight on the reduction in overtopping rates. If the second phase of the dune revetment is executed as proposed to the National Works Agency (NWA), the overtopping rate at the dune revetment will be reduced from 3.06 to 1.03 cubic metres per meter of structure length (see Table 11.11 in the Appendix).

6.2.2 **Rock Armour Sizing**

The Van der Meer Stability Equation as per the Rock Manual (2006) and Kamphius (2000)[16] was utilized to size the armour stones. The equation is valid for the estimation of the stability of armour stone for the trunk, head and toe of coastal structures, for both breaking and non-breaking wave conditions. See Equation 6-1 for the Van der Meer stability equation.

Equation 6-1 Van der Meer stability equation

For plunging waves:

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

For surging waves:

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

where:

$H_{2\%}$ - two percent highest waves

Δ - relative density

P – Porosity of the structure

S – Damage level

N – Number of waves

ξ_m – surf similarity parameter

α – angle of seaward slope of structure

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

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

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

The resulting design calculations for the armor structures is shown in the appendices (see Table 11.12). The design calculations revealed that a range of stone sizes. The primary armour stones will range from 1.09 Tons to 3.67 Tons (0.8m – 1.15m) while the secondary armour stones will vary from 0.14 Tons to 0.46 Tons (0.4m – 0.58m). Both layers of armour are required in order to resist the 100 Year Return Period Design wave conditions. Sensitivity analysis revealed the maximum size armor units correspond to a peak wave period of 11 seconds. The resulting cross sections are shown below and on the attached drawings.

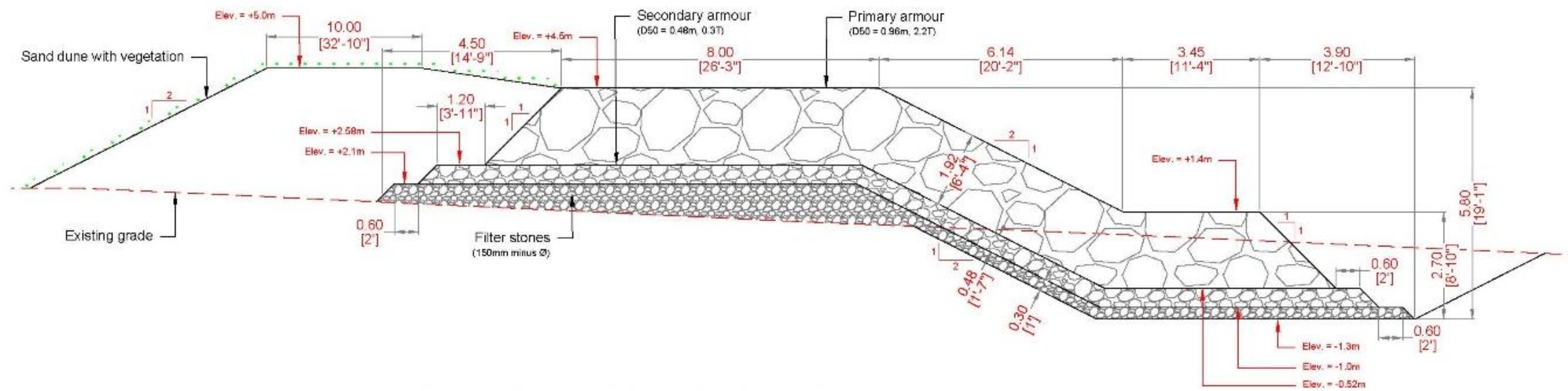


Figure 6.2 Proposed design cross section of the proposed high dune revetment structure (+4.5m crest elevation).

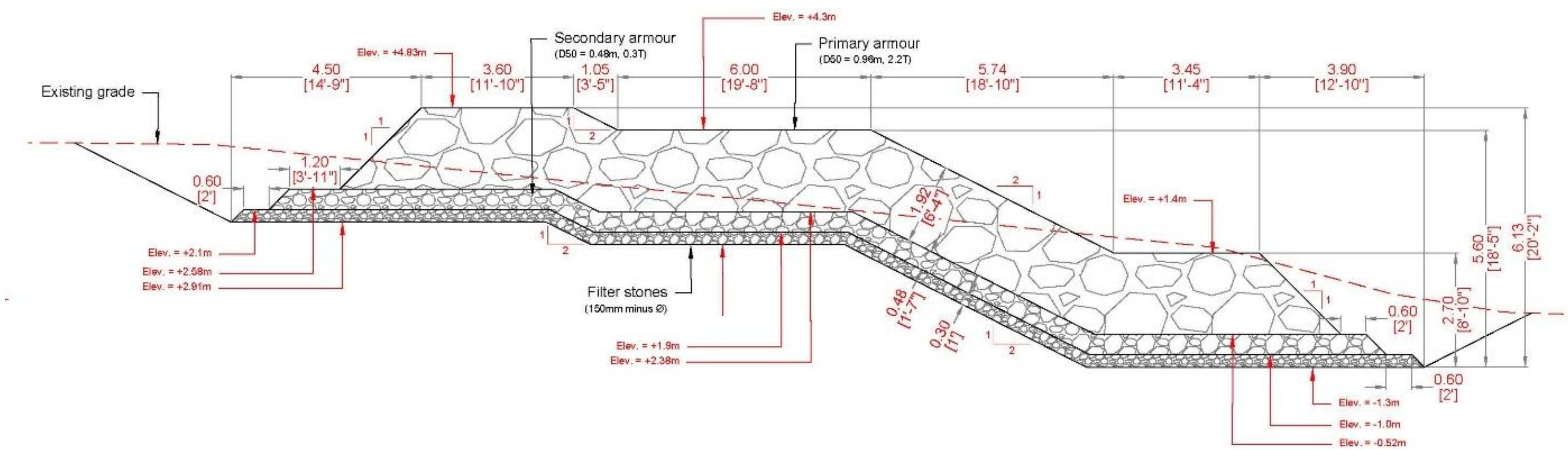


Figure 6.3 Proposed design cross section of the proposed high revetment structure (+4.83 crest elevation).

7 Engineering Estimate

An engineering estimate for the revetment works was done. The rock revetment costing includes the cost to supply , transport and place rocks on grade. This item accounts for 24% of the overall cost. This is as a result of good material not located within reasonable distance to the site. The total cost of this option is USD $12,219,012.50. See Table 7.1 and **Error! Reference source not found.** for the comparison of the options as well as the break down in the appendix.

Table 7.1 Summary of engineering estimate for both xbloc and rock revetment options.

Options	Cost (USD)
Xbloc Revetment + slope protection	$10,053,775.63
Rock Revetment + slope Protection	$11,380,001.88
Difference	($1,326,226.25)

8 Construction Methodology

8.1 Site Preparation

8.1.1 Grubbing

The project area will be cleared of any vegetative matters to make way for site access road and for material storage.

8.1.2 Demolishment of Derelict Wall

The footprint of the proposed dune revetment will encroach on an existing boundary wall of the National Screwworm Eradication Programme Distribution Centre. It is proposed that the wall be demolished and a new wall constructed to facilitate the construction of the revetment. It will therefore be demolished down to foundations and the rubble cart away to an approved landfill.

8.1.3 Removal of Rubble

The area will be cleared of rubble to make way for site access road and for material storage. It is anticipated that 4-7 days storage will be need to ensure the works have adequate supply during construction.

8.1.4 Prepare Haul Road

Access roads will be constructed, within or on the structure, to facilitate the excavators/backhoes gaining access to the shoreline. The roads shall be constructed of free-draining local material if available and suitable for this purpose, or of other free-draining material approved.

8.1.5 Sorting of boulders and Stockpiling

All armour and fill stones required for the construction will be stored at the designated stockpile area at or near the construction site. Separate stockpiles shall be made and identified for different grades of armour stones before transportation to work area.

8.2 Construction

8.2.1 Revetment

8.2.1.1 <u>Placement of Turbidity Barriers around Work Areas</u>

Curtains 6’ to 8’ deep will be displayed around the work areas and anchored properly. These will be adjusted daily or as required to move with the work and replace damaged sections in order to maintain water quality requirements.

8.2.1.2 <u>Excavation of revetment foundation</u>

The foundations will be excavated with a backhoe/excavator from the shoreline. This process will remove the existing rubble and some amount of earth to create the formation level for the revetment. Approved excavation material will be stockpiled for re-use and placement in dune.

8.2.1.3 <u>Retrieve Boulders from Stockpile Area and Placement in Footprint of Revetment</u>

Supply trucks will deliver boulders to the stockpile area where they will be sorted into two size classes by the contractor.

Placement will be initiated with the filter stones on the sea floor. The shapes will be achieved and surveyed by the contractor for accuracy to the designs. This will be undertaken by a grapple with mechanical mechanism versus hydraulic mechanisms to mitigate hydraulic fluid leakage in the marine environment. The actual process envisaged will be:

1. Retrieve boulders from a stockpile area;
2. Place boulders on sea floor;
3. Move placement excavator along shoreline if needed;
4. Continue placement of filter stones for 30 to 50m;
5. Survey Placement;
6. Retrieve secondary and primary armor from stockpile area, and repeat process 2 to 5 for crest armor as well.

For heavy equipment operations which penetrate above the main runway Obstacle Limitation Surface (OLS), construction will be executed outside the airport operational period. This period will be either governed by the last flight or generally between the hours of 23:00hrs to 04:30hrs.

8.2.1.4 Retrieve Excavated Material from Stockpile Area and Placement in Footprint of Sand Dune

Material that is excavated for the stone revetment will be stockpiled for use in the sand dune after being approved by the Engineer.

8.2.2 Construction of Retaining Wall

Retaining wall construction will take place after the placement of the base and trunk of the revetment. The contractor has the option of using precast or cast in place. The crest of the dune will then be placed after construction of the wall. The steps for the wall will be as follows:

1. Excavation of existing soil and shaping foundation;
2. Fill with granular material and compact;
3. Construction of wall (formwork and steel work then concrete and removal of formwork) The construction will take place in 10m sections between construction joints if the precast option is chosen;
4. Backfill wall with core fill and compact.

8.3 Landscaping and Plant Vegetation

The ground cover will be restored as soon as the earthworks permit, while taking the necessary measures to promote (re)generation of native vegetation. Measures may include the use of degradable geotextile, the use of seeds or live stakes of native grasses, re-vegetating with local shrub and tree species (especially those characterized by rapid growth and deep roots) and the reuse of top soil and mulch.

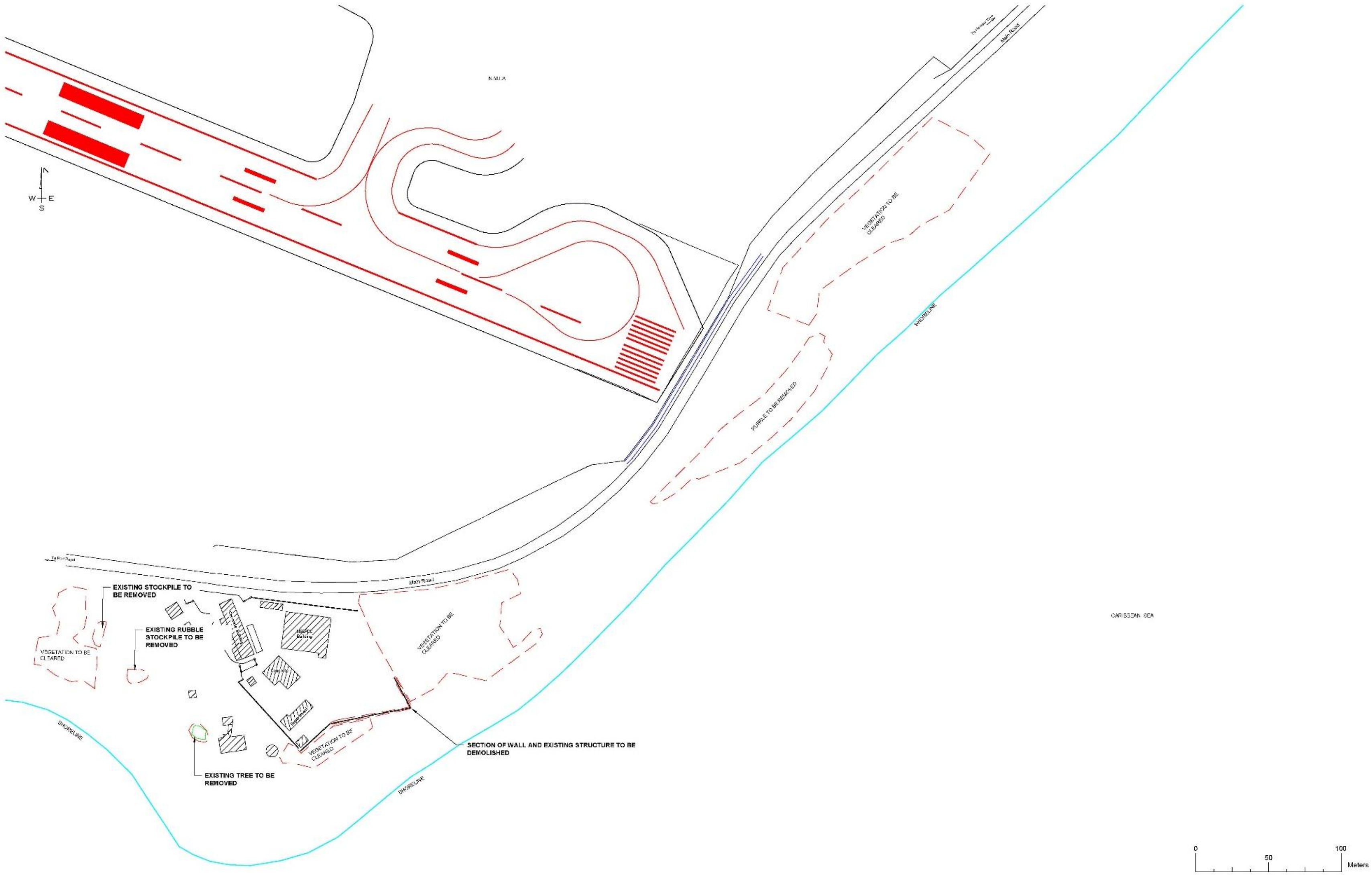


Figure 8.1 Identified areas to be grubbed and cleared

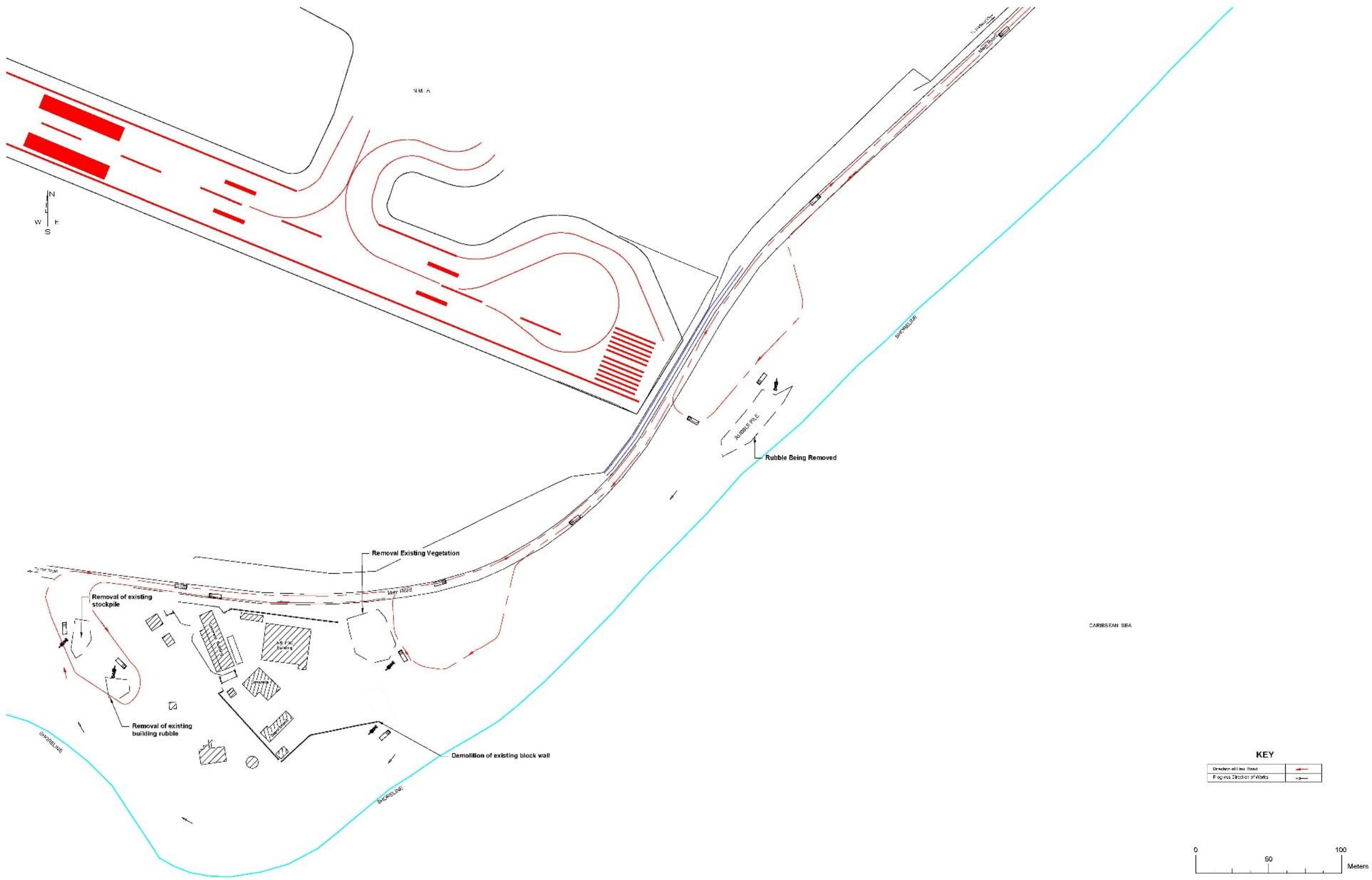


Figure 8.2 Grubbing and site clearance

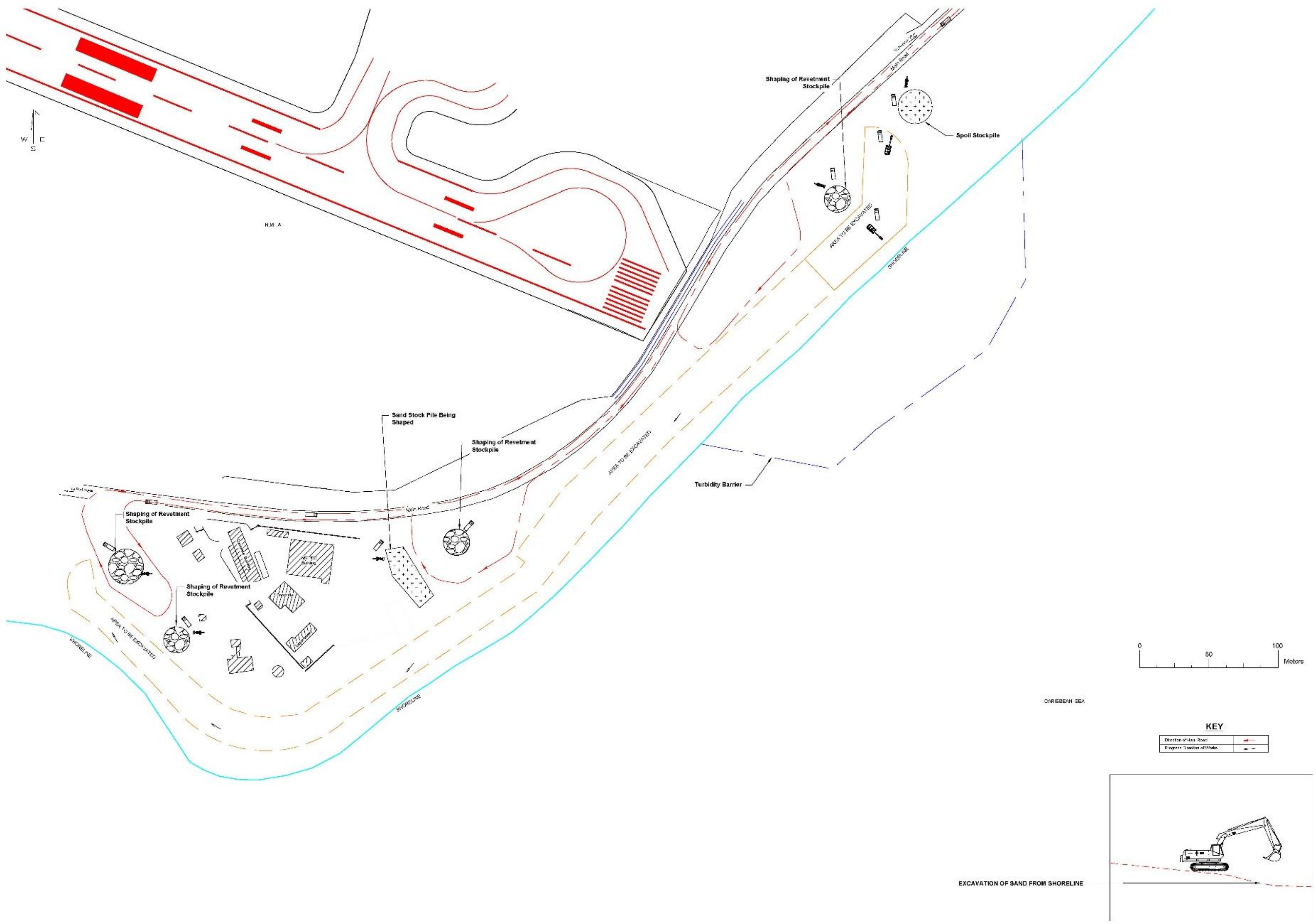


Figure 8.3 Excavation of Sand From Shoreline & Stockpiling of Material

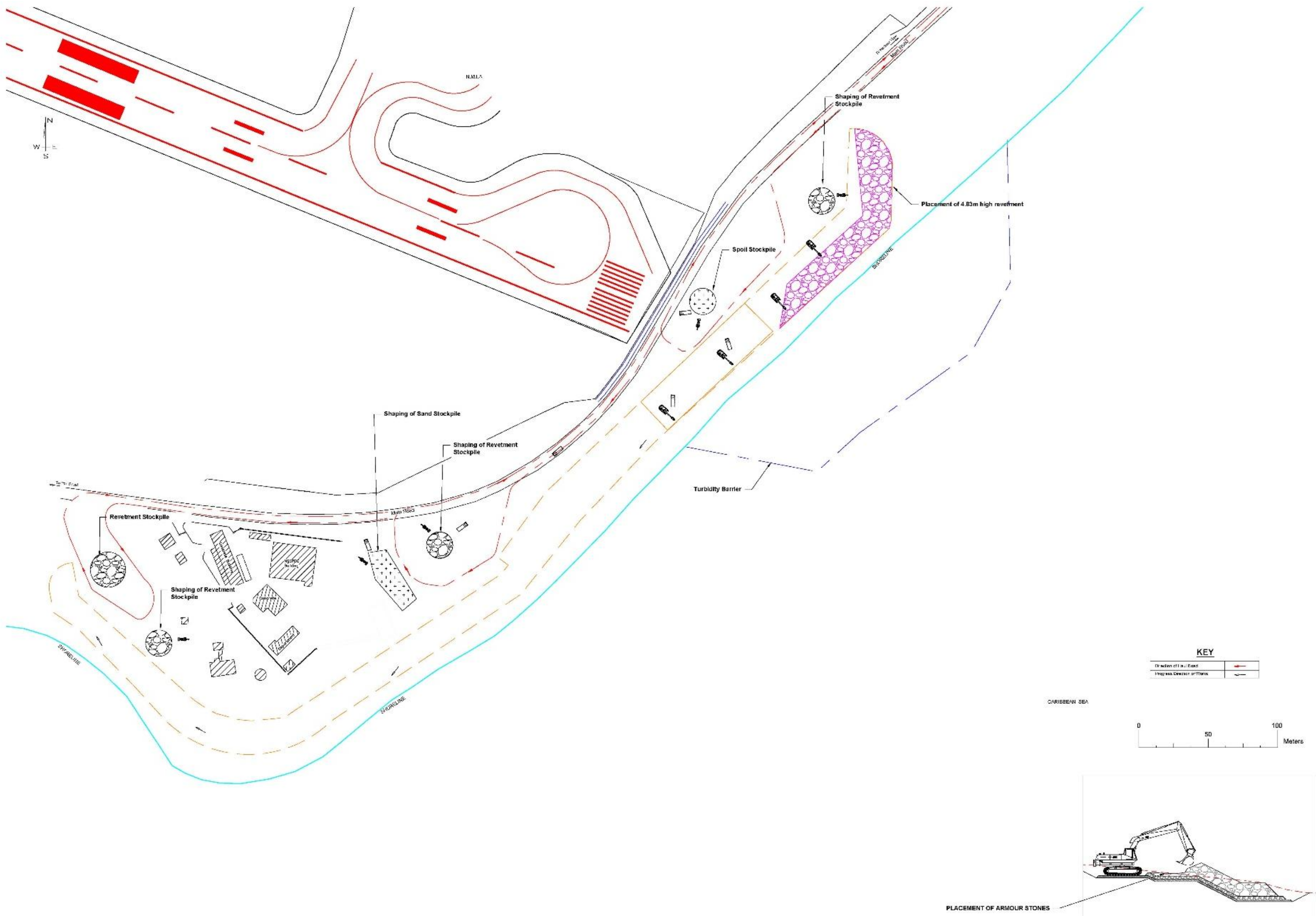


Figure 8.4 Further Excavation and Placement of 4.83m Revetment

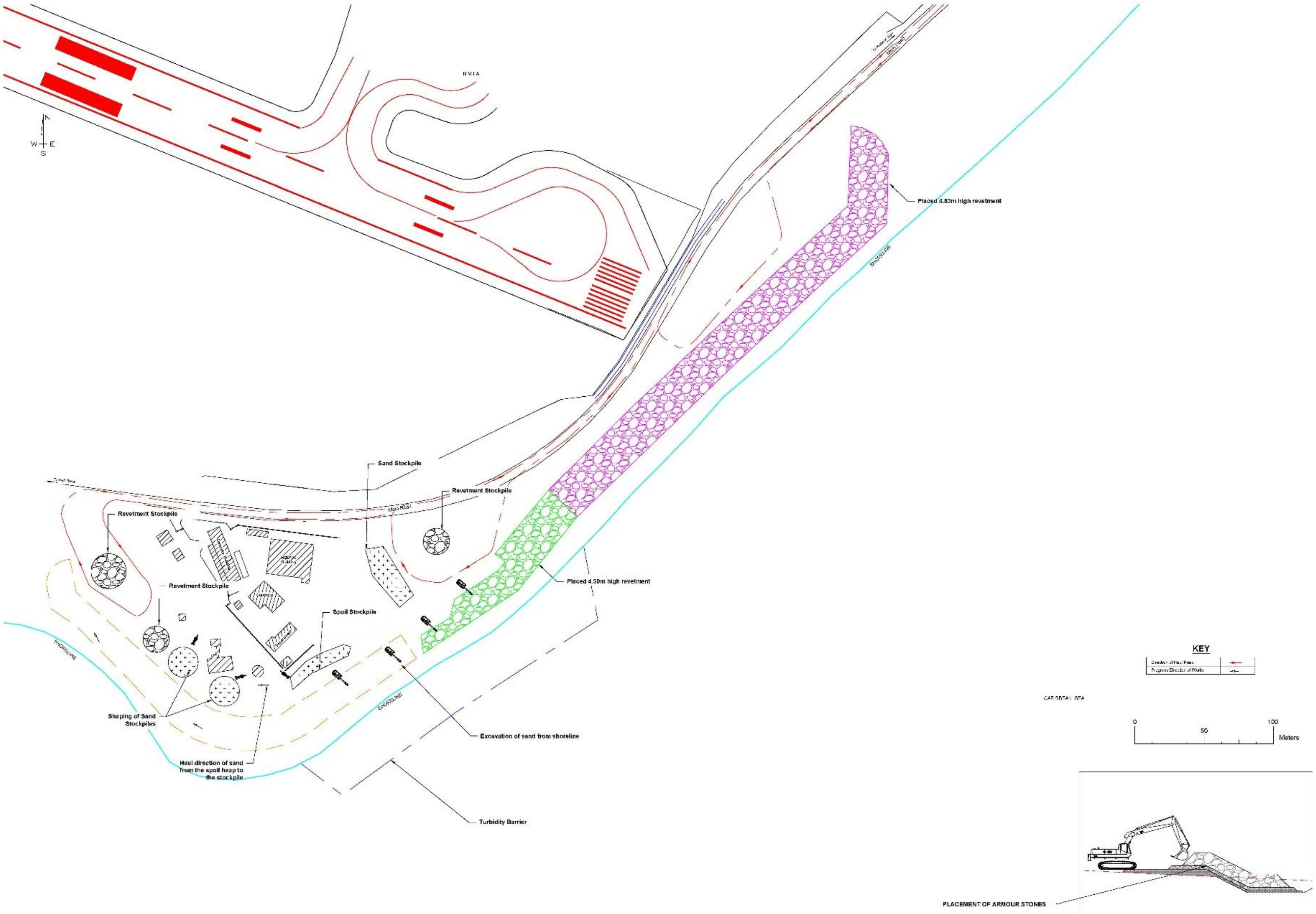


Figure 8.5 Further Excavation and Placement of 4.5m Revetment

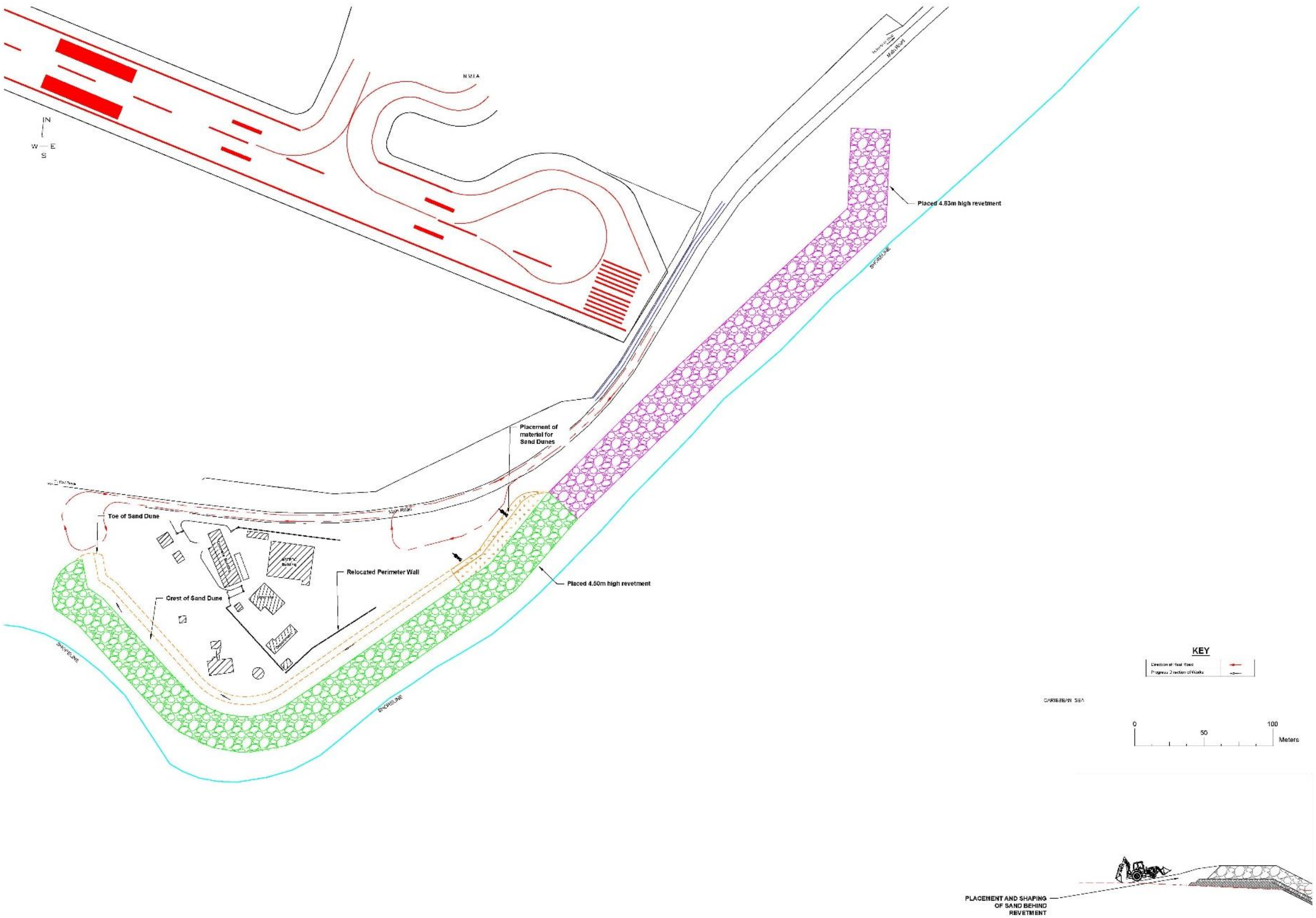


Figure 8.6 Placement and Shaping of Sand Dunes

9 Biophysical Survey

10 Conclusions and Recommendations

10.1 Conclusions

The following could be concluded from the analysis conducted to date:

1) A storm surge analysis of the proposed revetment offshore the revealed that storm surges for the different return periods may be generated as follows:

Return Period	Current	Future (2050) without SLR	Future (2050) with SLR
5	0.59	1.14	1.26
10	1.25	1.95	2.07
25	2.48	3.21	3.33
50	3.71	4.33	4.45
100	5.19	5.55	5.67

*It must be noted that the future flows have incorporated climate change increases.

2) Worst case extreme (storm surge) scenarios involving category three to five hurricanes were investigated and the estimated surges are as follows:

Hurricane path/Track	Hurricane Scenario Modelling					
	Category 3 - Setup (m) without run up [Return Period yrs.]	Category 3 - Setup (m) with run up [Return Period yrs.]	Category 4 - Setup (m) without run up [Return Period yrs.]	Category 4 - Setup (m) with run up [Return Period yrs.]	Category 5 - Setup (m) without run up [Return Period yrs.]	Category 5 - Setup (m) with run up [Return Period yrs.]
Direct Hit 10 km west of site (from South)	1.34 [11.6]	4.39 [48.0]	1.78 [14.3]	4.95 [62.2]	2.24 [17.7]	5.49 [80]
Direct Hit 40 KM west of site (from South)	**1.74 [14]**	**4.81 [58.0]**	**2.34 [18.5]**	**5.52 [81.0]**	**2.97 [25.0]**	**6.23 [112.8]**
10 KM Parallel offshore of site (from east)	1.07 [10.3]	4.09 [41.8]	1.57 [12.9]	4.75 [56.7]	1.94 [15.4]	5.18 [70.0]
40 KM Parallel offshore of site (from east)	1.18 [10.8]	4.41 [48.4]	1.54 [12.8]	4.87 [60.0]	1.92 [15.2]	5.33 [74.2]

3) Wave Climate Analysis for offshore deep water waves propagating into the area from the following areas are summarized as follows:

Future	100yr	50yr	25yr	10yr
E	3.6-4.0	2.8-3.0	1.8-2.0	1.0-1.2
SE	3.5-4.0	2.4-2.8	1.6-2.0	0.8-1.0
S	3.6-4.0	2.5-3.0	1.6-2.0	0.75-1.0
SW	3.6-4.0	3.2-3.6	2.2-2.4	1.6-1.8

4) The shoreline located adjacent to the lighthouse and the end of runway experience the greatest erosion extents and depths within the project area. The estimated erosion under the pre-project scenarios are as follows:

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (50 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse	64	1.7	71	2.3
End of Runway	34	0.9	61	2.3
Dune	139	0.4	142	0.8
Low Revetment	33	0.8	33	1.3
High Revetment	0	0	0	0

The estimated erosion under the post-project scenario are as follows:

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (50 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse	26	0.7	26	0.9
End of Runway	32	0.2	31	0.4
Dune	139	0.4	142	0.8
Low Revetment	33	0.8	33	1.3
High Revetment	0	0	0	0

The proposed shoreline protection will account for 5.9% - 59.4% of short term erosion within the project area for the 50 year return period. For the 100 year return period, the reduction rates are within the range of 49.2% - 63.4%.

5) Applying the projected erosion due to sea level rise and accretion based on historical shoreline movements, the shoreline within the project area can estimated as follows (year 2050):

Parameter	ghthouse	End of Runway	Dune	Unit
Projected accretion (2017 shoreline datum)	10.19	7.32	8.69	m
Dune line Erosion due to SLR	0.155	0.122	0.091	m/year
Projected total erosion in 33 years (m)	5.12	4.01	3.02	m
Projected total accretion (2017 shoreline datum)	5.07	3.31	5.67	m

6) Overtopping analysis revealed that the required crest elevation for the revetment to limit seawater discharges over the structure to safe limits is in the order 4.83 metres (above mean sea level) for the rock revetment. Similarly the dune revetment will require approximately an elevation 5m.

7) The required primary armour sizes for the revetment range from 1.09 Tons to 3.67 Tons (0.8m – 1.15m) while the secondary armour stones will vary from 0.14 Tons to 0.46 Tons (0.4m – 0.58m). Both layers of armour are required in order to resist the 100 Year Return Period Design wave conditions.

10.2 Recommendations

1) During construction, it must be enforced that the Silt/turbidity screen be installed at all times to limit the environmental impacts associated with the works.

2) It will not be possible not to penetrate OLS during entire construction, therefore planning should include times budgeted when this can be done during down times such as during late night to early morning.

3) It is recommended that the proposed revetment be constructed as per the drawings and specifications.

11 Appendix

11.1 Quarry Assessment

Table 11.1 Summary of quarry assessment conducted in 2010

Quality Rating Assessment (adopted from Lienhart, 1998: Rock engineering rating system for assessing the suitability of armourstone sources)

				Black			Yallahs (stockpile)			Yallahs (river bed)			Hill run			Ferry pen		
		Criteria	Weighting	Quality	Quality rating (Excellent = 4; Good = 3; Marginal = 2; Poor = 1)	Weighting Rating	Quality	Quality rating (Excellent = 4; Good = 3; Marginal = 2; Poor = 1)	Weighting Rating	Quality	Quality rating (Excellent = 4; Good = 3; Marginal = 2; Poor = 1)	Weighting Rating	Quality	Quality rating (Excellent = 4; Good = 3; Marginal = 2; Poor = 1)	Weighting Rating	Quality	Quality rating (Excellent = 4; Good = 3; Marginal = 2; Poor = 1)	Weighting Rating
1.1	Field based indicators	**Litographic classification**	60%	Lots of calcium carbonate, little dolomites	2	1.2	Volcanics	4	2.4	Volcanics	4	2.4	Some dolomites	2.5	1.5	Some dolomites	2.5	1.5
1.2		**Weathering grade**	75%	some staining, faint weathering	3.5	2.625	No staining, unweathered	3.5	2.625	No staining, unweathered	3.5	2.625	Not much staining, faint weathering	3	2.25	Some decomposed , stained and weathered	1	0.75
1.3		**Production Method**	95%	Aggregate blasting	1	0.95	crushing of river stones	3	2.85		4	3.8	Non-blasting method	4	3.8	blasting used	4	3.8
1.4		**Set-aside**	70%	immediate transport	1	0.7	stockpiled	4	2.8	From river	4	2.8	stockpiled	4	2.8	stockpiles noticed	4	2.8
1.5		**Rock block quality**	80%	No fractures noted	4	3.2	No fractures noted	4	3.2	No fractures noted	4	3.2	no fractures	4	3.2	Less than 85% fracture free	1	0.8
1.6		**Stone shape**	80%		4	3.2		4	3.2		4	3.2		4	3.2		4	3.2
1.7		**Stone size/gradation**	90%	Majority meeting required stone size specifications	3	2.7	Stones being cut too small hence failing to meet specifications	1	0.9	Stones meet gradation criteria	4	3.6	Majority of stones meeting required armour stone specifications	4	3.6	Less than 50% of the stones meeting the specifications	4	3.6
1.8		**Proximity to site/transport costs and risks**	80%	5.2 km	4	3.2	16.8 km	2	1.6	16.8 km	2	1.6	22.3 km	1	0.8	17.3 km	2	1.6
2.1	Laborartory Results indicators	**Specfic density**	70%	2.49	4	2.8	NA	4	2.8	NA	4	2.8	2.56	4	2.8	2.61	4	2.8
2.2		**Water adsorption**	70%	5.40%	1	0.7	NA	4	2.8	NA	4	2.8	2.80%	4	2.8	1.40%	4	2.8
		Score				21.275			27.975			31.625			26.75			23.65
		Maxium possible score				33.6			33.6			33.6			33.6			33.6
		Weighted score				**63%**			**83%**			**94%**			**80%**			**70%**

KEY
Excellent
Good
Marginal
Poor

11.2 Hurricane Names

	Storm No.	Name	Date	Max. SS Category	
1	3	NOTNAMED	1851	3-	EXTENSIVE
2	50	NOTNAMED	1859	3-	EXTENSIVE
3	89	NOTNAMED	1865	3-	EXTENSIVE
4	127	NOTNAMED	1870	2-	MODERATE
5	150	NOTNAMED	1873	3-	EXTENSIVE
6	157	NOTNAMED	1874	2-	MODERATE
7	160	NOTNAMED	1875	3-	EXTENSIVE
8	181	NOTNAMED	1878	2-	MODERATE
9	199	NOTNAMED	1880	1-	WEAK
10	227	NOTNAMED	1884	2-	MODERATE
11	240	NOTNAMED	1886	2-	MODERATE
12	241	NOTNAMED	1886	2-	MODERATE
13	242	NOTNAMED	1886	3-	EXTENSIVE
14	246	NOTNAMED	1887	1-	WEAK
15	248	NOTNAMED	1887	1-	WEAK
16	256	NOTNAMED	1887	2-	MODERATE
17	288	NOTNAMED	1891	1-	WEAK
18	316	NOTNAMED	1894	3-	EXTENSIVE
19	324	NOTNAMED	1895	3-	EXTENSIVE
20	352	NOTNAMED	1900	4-	EXTREME
21	364	NOTNAMED	1901	1-	WEAK
22	375	NOTNAMED	1903	3-	EXTENSIVE
23	383	NOTNAMED	1904	1-	WEAK
24	391	NOTNAMED	1905	2-	MODERATE
25	403	NOTNAMED	1906	1-	WEAK
26	420	NOTNAMED	1909	3-	EXTENSIVE
27	427	NOTNAMED	1910	3-	EXTENSIVE
28	433	NOTNAMED	1911	1-	WEAK
29	439	NOTNAMED	1912	4-	EXTREME
30	446	NOTNAMED	1915	4-	EXTREME

	Storm No.	Name	Date	Max. SS Category	
31	453	NOTNAMED	1916	3-	EXTENSIVE
32	466	NOTNAMED	1917	3-	EXTENSIVE
33	503	NOTNAMED	1924	2-	MODERATE
34	522	NOTNAMED	1927	1-	WEAK
35	525	NOTNAMED	1928	1-	WEAK
36	540	NOTNAMED	1931	2-	MODERATE
37	550	NOTNAMED	1932	3-	EXTENSIVE
38	557	NOTNAMED	1933	1-	WEAK
39	573	NOTNAMED	1933	2-	MODERATE
40	585	NOTNAMED	1934	1-	WEAK
41	590	NOTNAMED	1935	3-	EXTENSIVE
42	591	NOTNAMED	1935	1-	WEAK
43	630	NOTNAMED	1939	2-	MODERATE
44	653	NOTNAMED	1942	1-	WEAK
45	668	NOTNAMED	1944	3-	EXTENSIVE
46	698	NOTNAMED	1947	1-	WEAK
47	721	NOTNAMED	1949	2-	MODERATE
48	734	KING	1950	3-	EXTENSIVE
49	776	HAZEL	1954	4-	EXTREME
50	786	HILDA	1955	3-	EXTENSIVE
51	811	ELLA	1958	3-	EXTENSIVE
52	813	GERDA	1958	1-	WEAK
53	842	GERDA	1961	1-	WEAK
54	857	FLORA	1963	4-	EXTREME
55	864	CLEO	1964	4-	EXTREME
56	886	INEZ	1966	4-	EXTREME
57	974	CAROLINE	1975	3-	EXTENSIVE
58	976	ELOISE	1975	3-	EXTENSIVE
59	1011	CLAUDETTE	1979	1-	WEAK
60	1018	ALLEN	1980	5-	CATASTROPHIC

	Storm No.	Name	Date	Max. SS	Category
61	1029	ARLENE	1981	1-	WEAK
62	1154	GORDON	1994	1-	WEAK
63	1224	DEBBY	2000	1-	WEAK
64	1228	HELENE	2000	1-	WEAK
65	1259	ISIDORE	2002	3-	EXTENSIVE
66	1262	LILI	2002	4-	EXTREME
67	1336	DENNIS	2005	4-	EXTREME
68	1366	ERNESTO	2006	1-	WEAK
69	1385	OLGA	2007	1-	WEAK
70	1391	FAY	2008	1-	WEAK
71	1392	GUSTAV	2008	4-	EXTREME
72	1401	PALOMA	2008	4-	EXTREME
73	1435	TOMAS	2010	2-	MODERATE
74	1440	EMILY	2011	1-	WEAK
75	1402	SANDY	2012	3-	EXTENSIVE

11.3 Anecdotal information collected throughout Palisadoes and Port Royal area

Table 11.2 Summary of anecdotal interviews documented

Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
Lascelles Campbell	80	5	Dean	2007	-	-	Foreshore Ave	-	2.013	-	-
Commander Albred	63	19	Lili	2002	-	-	JDF Coast Guard Base	1.524	0.400	1.924	-
Commander Albred	63	19	Dean	2007	-	-	Florizel Avenue	0.152	1.576	1.729	-
Alice Murphy	58	58	Dean	2007	Sand on Florizel Avenue	-	Florizel Avenue	0.305	1.075	1.38	-
Richard Brown	60	15	Dean	2007	Sand on Foreshore Ave	-	Foreshore Ave	0.305	2.085	2.39	-
Henroy Hunt	69	69	Ivan	2004	-	-	Lue's Grocery Store	0.229	0.760	0.989	-
Oran Hall	40	40	Gilbert	1988	-	-	Cannon Street	0.152	0.866	1.019	-
Oran Hall	40	40	Ivan	2004	-	-	Cannon Street	0.152	0.866	1.019	-
Michelle Watler	42	42	Dean	2007	Sand on Henry Morgan Boulevard	-	Henry Morgan Boulevard	0.305	1.789	2.094	-
Charn Watler	39	39	Dean	2007	-	-	Henry Morgan Boulevard	0.305	1.786	2.091	-
Charn Watler	39	39	Ivan	2004	-	-	Henry Morgan Boulevard	0.305	1.786	2.091	-
Judith Clay	45	45	Ivan	2004	-	-	Michelin Ave	0.152	0.975	1.127	-
Edwin Dobson	56	56	Gilbert	1988	Drift Wood on Henry	-	Henry Morgan Boulevard	0.152	2.009	2.161	-

Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
					Morgan Boulevard						
Edwin Dobson	56	56	Ivan	2004	-	-	Henry Morgan Boulevard	0.178	2.009	2.187	-
Beris Brown	58	34	Ivan	2004	-	-	Foreshore Ave	0.305	1.496	1.801	-
Beris Brown	58	34	Ivan	2004	-	-	Florizel Ave	0.61	1.585	2.195	-
Heather Limtom	49	49	Ivan	2004	Sand on Henry Morgan Boulevard	-	Henry Morgan Boulevard	1.829	1.550	3.379	-
Heather Limtom	49	49	Dean	2007	Sand on Henry Morgan Boulevard	-	Henry Morgan Boulevard	1.219	1.550	2.769	-
Debby Goldson	48	48	Ivan	2004	Sand and sea weed on Henry Morgan Boulevard	-	Henry Morgan Boulevard	1.219	1.513	2.732	-
Debby Goldson	48	48	Dean	2007	Sand and sea weed on Henry Morgan Boulevard	-	Henry Morgan Boulevard	1.219	1.513	2.732	-
Marjorie Taylor	61	6	Ivan	2004	-	-	Foreshore Ave	0.203	1.869	2.072	-
Maurice Taylor	55	55	Ivan	2004	-	-	Foreshore Ave	0.152	1.439	1.591	-
Maurice Taylor	55	55	Ivan	2004	-	-	Henry Morgan Boulevard	0.61	1.734	2.344	-

Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
Orane Chin	38	10	Sandy	2012	Yes	-	-	0.300	-	-	Sand on road, Damage to airport North Side. Airport shut down for 2 days. Storm surge came up to fence from harbor side and sewage. Less than 2 ft.
Orane Chin	38	10	Dean	2007	Yes	-	-	0.000	-	-	-
Orane Chin	38	10	Matthew	2016	Yes	-	-	0.600	-	-	Waves and swells. Road blocked. No significant damage to airport.
Junior Taylor	40	17	Ivan	2004	Yes	-	-	0.000	-	-	3 ft. of sand on road from gun boat 300m towards harbor view. Waves on road moved truck. Road to airport blocked. Waves came over and brought up debris up to 4-5 feet on the road. Took two full days to

Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
											clear fully. Emergency Acces
Junior Taylor	41	18	Dean	2007	Yes	-	-	0.000	-	-	More debris than Ivan, i.e. exacerbated what Ivan did. Sand on runway at runway 30. More intense than Ivan.
Mr. Norris (CMI)	40	0	2004 Ivan	2004	Yes	-	-	0.000	-	-	Damage to jetty and boats. utility poles and 2-3 ft stones washed into harbour.
Mr. Norris (CMI)	40	0	Sandy	2012	Yes	-	-	0.000	-	-	Damage to jetty and boats. utility poles and 2-3 ft stones washed into harbour.
Mr. Norris (CMI)	40	0	Dean	2007	Yes	-	-	0.000	-	-	Damage to jetty and boats. utility poles and 2-3 ft stones washed into harbour.
Paul Enser	65	65	Gilbert	1988	Yes	-	-	0.900	-	-	waist high water during flood.

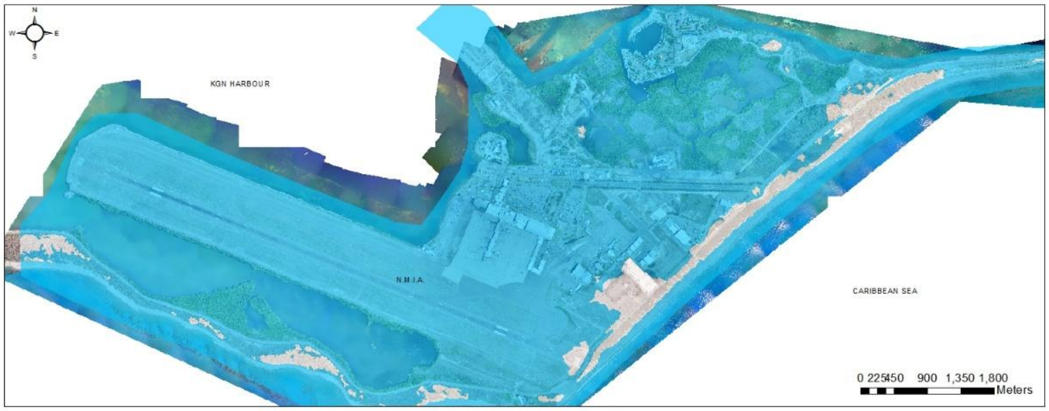
Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
											Boats washed inland.
Roy Brown	58	40	Ivan	2004	Yes	-	-	0.600	-	-	Sand on beach front, harbor side. 1-1.5 ft at Gloria's, Harbour side. Boats washed inland.
Darien Lin (Chinno)	52	52	Ivan	2004	Yes	-	-	0.300	-	-	Sand deposits by cemetery and airport. Sand on Palisados. On sea side water over dune and unto Henry Morgan Blvd.
Darien Lin (Chinno)	52	52	Dean	2007	Yes	-	-	0.300	-	-	Sand deposits by cemetery and airport. Sand on Palisados. On sea side water over dune and unto Henry Morgan Blvd.
Com. Evon Clark	78	78	Charlie	1951	No	-	-	0.000	-	-	3 people died after building collapsed. water from sea flooded community

Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
Com. Evon Clark	78	78	Gilbert	1988	-	-	-	0.000	-	-	Generally no flooding in area.
Tim Bailey	58	58	Gilbert	1988	-	-	-	0.000	-	-	heavy winds damaged house. reef breaks wave before they come in land
Ms. Henry	60	40	Ivan	2004	-	-	-	0.900	-	-	Trees brought up. Mangrove was cut down before.
Marquese Mais	37	37	Ivan	2004	Yes	-	-	0.600	-	-	Tress and sand brought up by water
Nadia McKen	64	64	Gilbert	1988	-	-	-	0.000	-	-	no storm surge, Just wind. Area protected by reef.
Clifton McKen	59	59	Ivan	2004	Yes	-	-	0.355	-	-	Sand deposits up to Wave Way.
Fay Morrell	79	79	Ivan	2004	Yes	-	-	0.900	-	-	storm surge on road
Yvonne McFarlane	63	63	Dean	2007	Yes	-	-	1.500	-	-	Sand, trees and garbage deposits.
Yvonne McFarlane	64	64	Ivan	2004	Yes	-	-	0.457	-	-	Effects less severe than that of Dean
Busta (RJYC)	79	66	Charlie	1951	-	-	-	0.000	-	-	harbour side water level was 2 feet above msl plus 4 ft waves

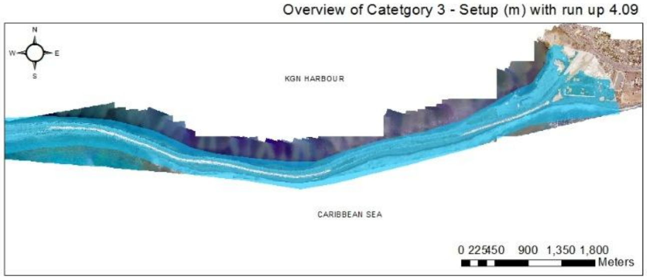
Name	Age	Time in Area	Storm	Year of Storm	Debris on Road	Watermarks	Storm Surge	Depth of Water (m)	Ground Elevation (m)	Water Elevation (m)	Comments
Busta (RJYC)	79	66	Gilbert	1988	-	-	-	0.000	-	-	road was overtopped from sea side, water levels on harbour side were approximately 3 feet above mean sea level
Anonymous	47	20	Ivan	2004	Yes	No	Port Royal Street	0	0.21	-	Storm surge port royal street
Moreen Clayton	49	25	Gilbert	1988	No	Yes		1.6	1.6	-	
Everton Thomas	39	3	Allen	1981	-	-		0	0	-	
Damian	35	15	Allen	1981	-	-		0	0	-	Never experienced a hazard
C. Rankine	42	4	Allen	1981	Yes	No	Port Royal Street	0.24	0.96	-	Storm Surge port royal street
Paul Mae	45	45	Dean	2007	Yes	Yes	Port Royal Street	0	2.3499999	-	Storm surge covered port royal street
Dorset Brown	58	0	Ivan	2004	Yes	No		0.15	0	-	
Dorset Brown	58	0	Gustav	2008	Yes	No		0.25	0	-	
Oneil Lewis	47	0			Yes	-		0.05	0	-	Debris from harbour deposited
John Rawlin	46	0	Ivan	2004	-	-		0.05	0	-	Runoff backed up
Mark Philips	42	0	Ivan	2004	-			0	0	-	No damage, Sand dumped on runway

11.4 Worst Case Storm Surge Simulations

Table 11.3 Inundation levels associated with the worst case storm surge modeling (10 KM Parallel offshore of site)

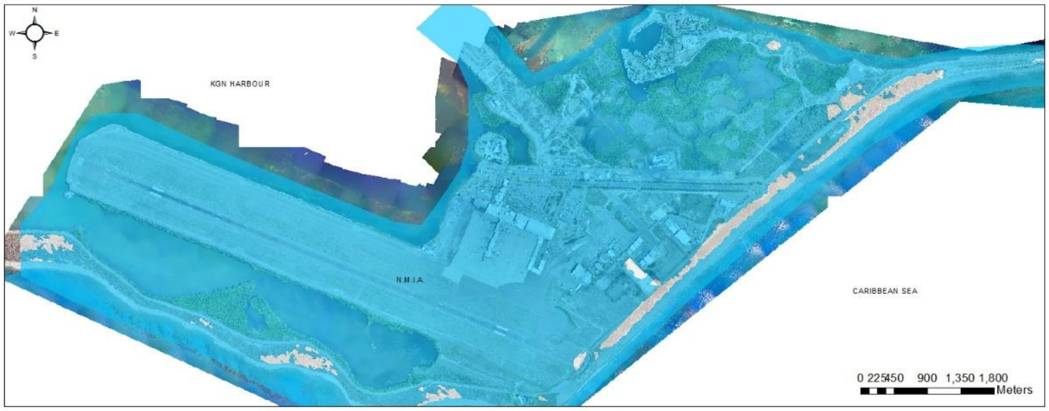


Overview of Catetgory 3 - Setup (m) with run up 4.09

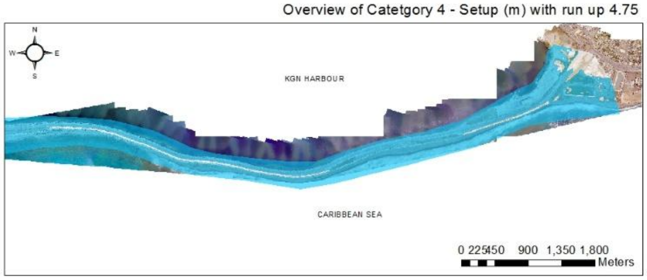


Overview of Catetgory 3 - Setup (m) with run up 4.09

(a) Category 3 Storm Event with runup

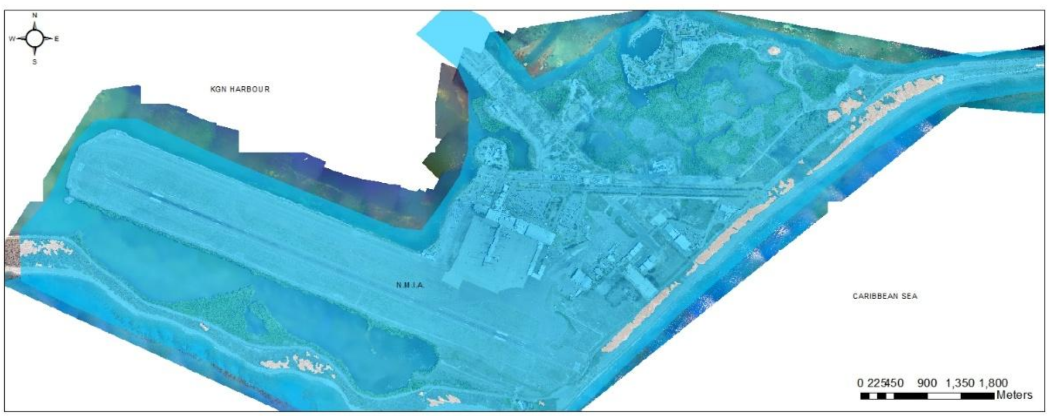


Overview of Catetgory 4 - Setup (m) with run up 4.75

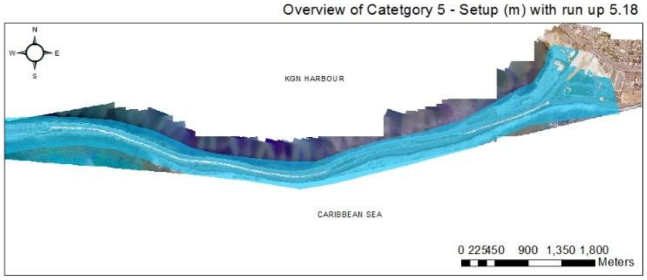


Overview of Catetgory 4 - Setup (m) with run up 4.75

(b) Category 4 Storm Event with runup



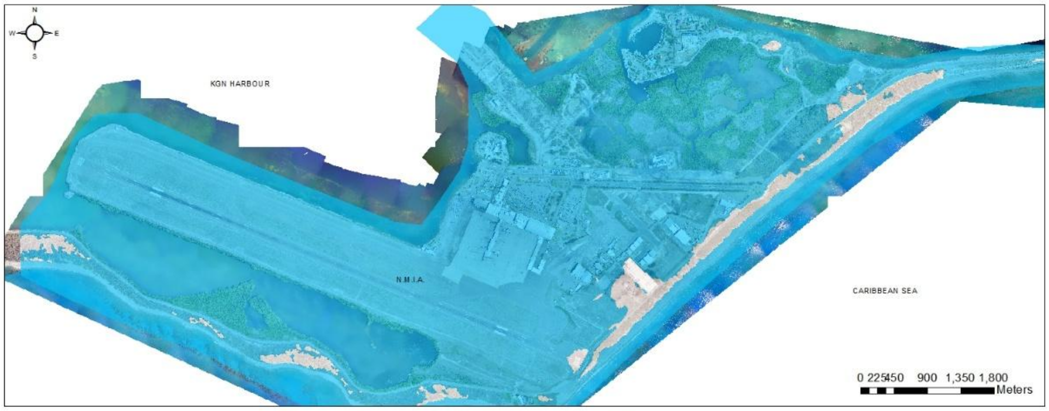
Overview of Catetgory 5 - Setup (m) with run up 5.18



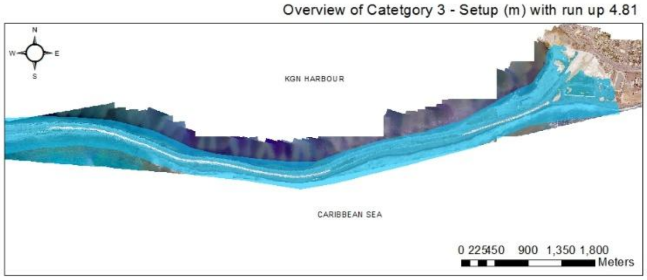
Overview of Catetgory 5 - Setup (m) with run up 5.18

(c) Category 5 Storm Event with runup

Table 11.4 Inundation levels associated with the worst case storm surge modeling (Direct Hit 10 km west of site)

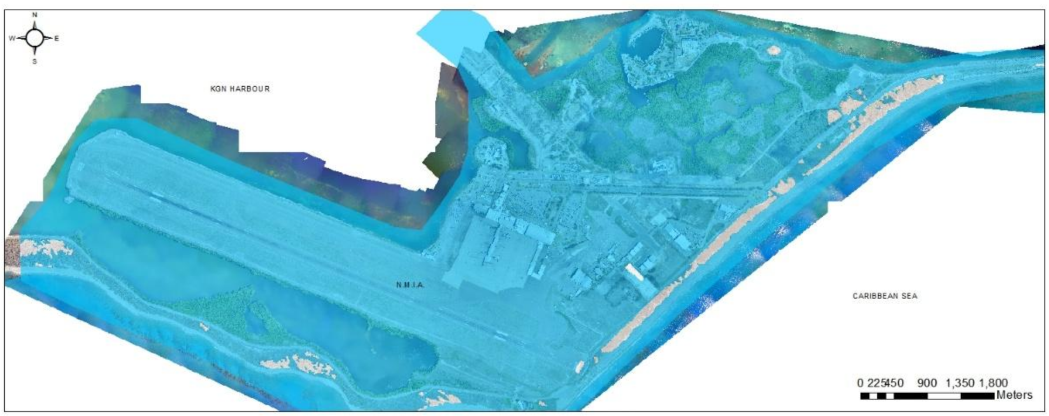


Overview of Catetgory 3 - Setup (m) with run up 4.81

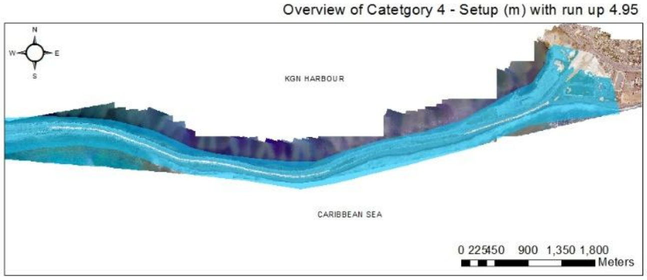


Overview of Catetgory 3 - Setup (m) with run up 4.81

(a) Category 3 Storm Event with runup

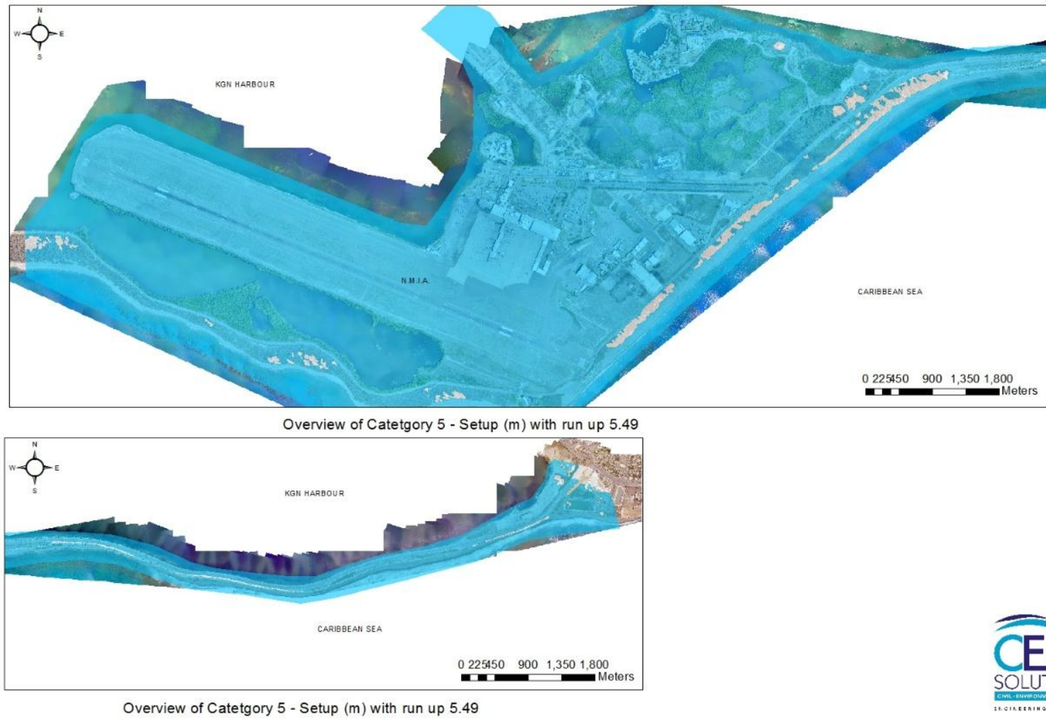


Overview of Catetgory 4 - Setup (m) with run up 4.95



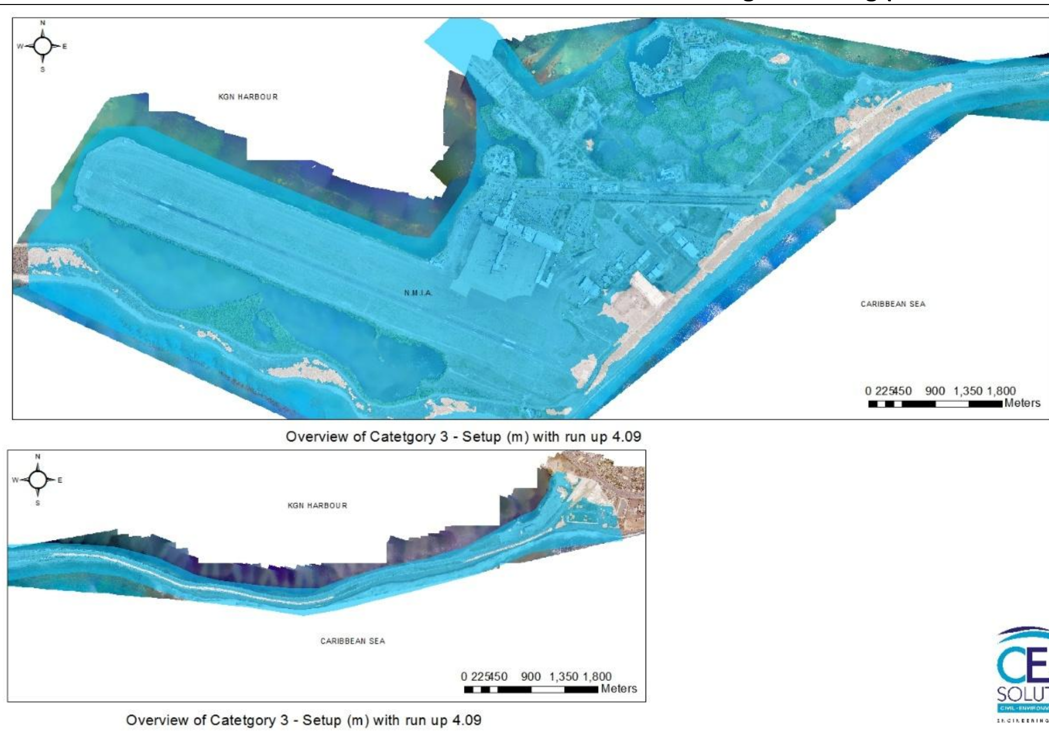
Overview of Catetgory 4 - Setup (m) with run up 4.95

(b) Category 4 Storm Event with runup

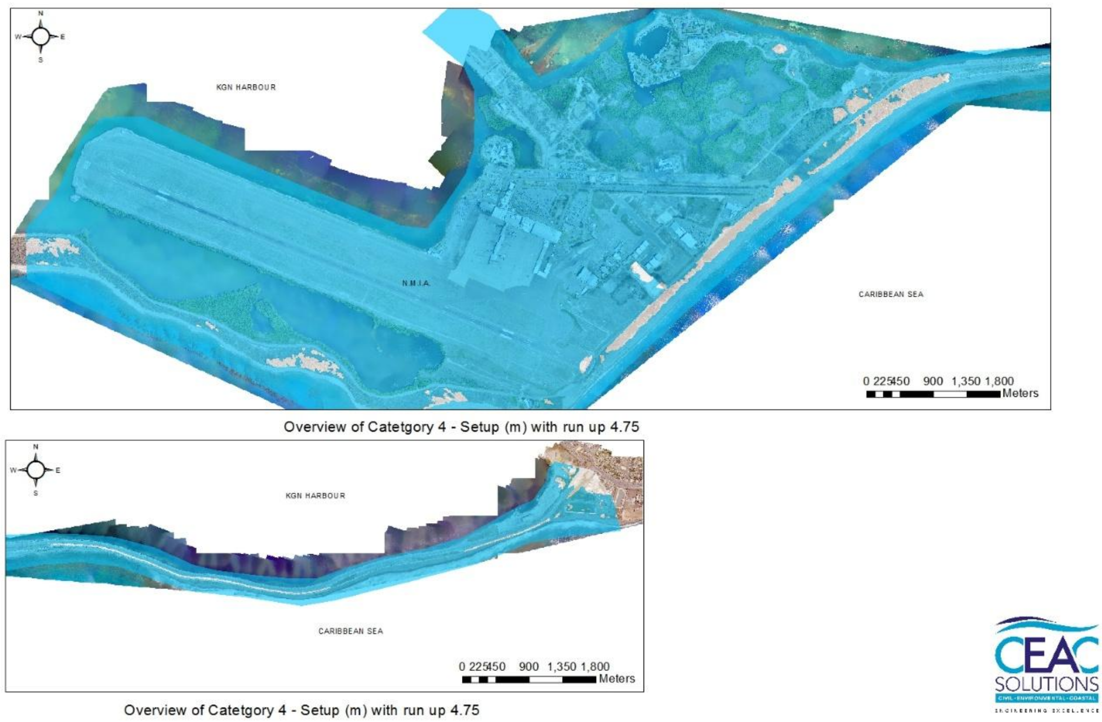


(c) Category 5 Storm Event with runup

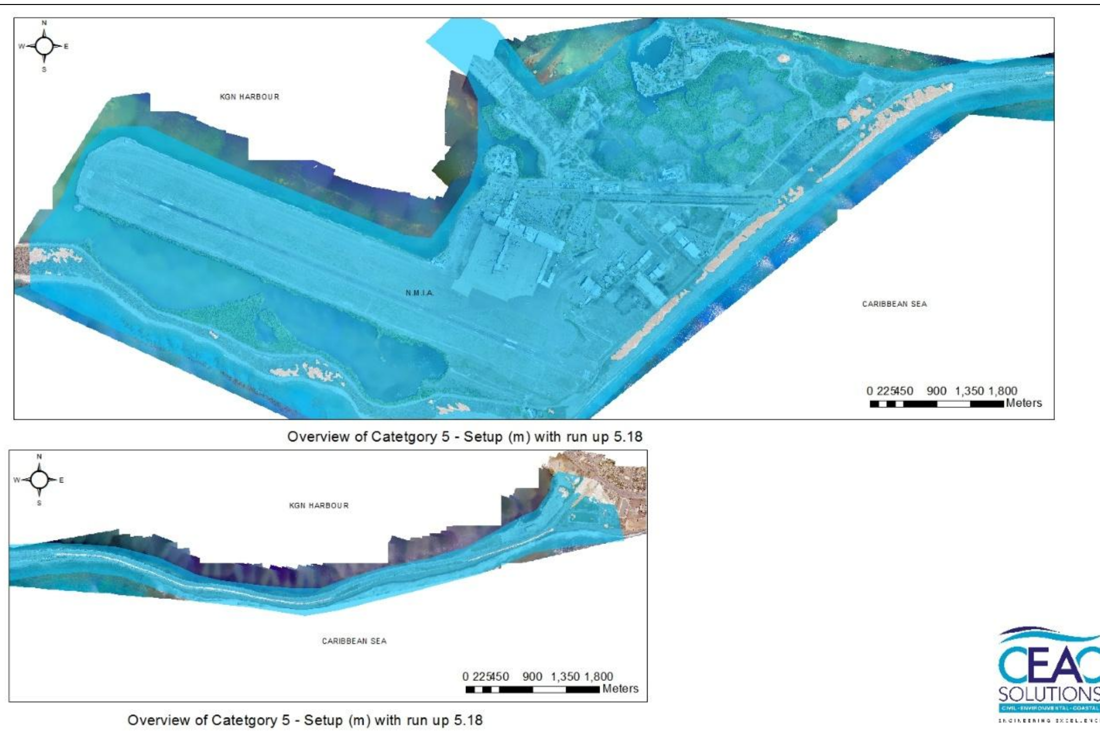
Table 11.5 Inundation levels associated with the worst case storm surge modeling (10 KM Parallel offshore of site)



(a) Category 3 Storm Event with runup

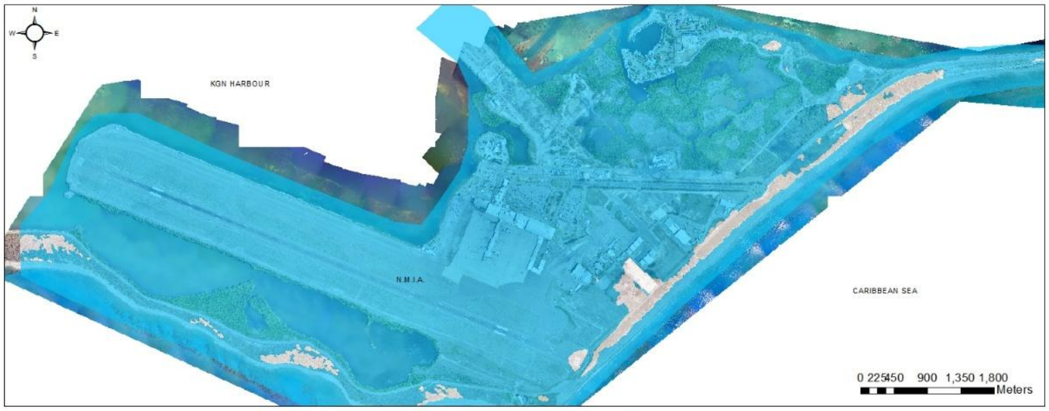


(b) Category 4 Storm Event with runup

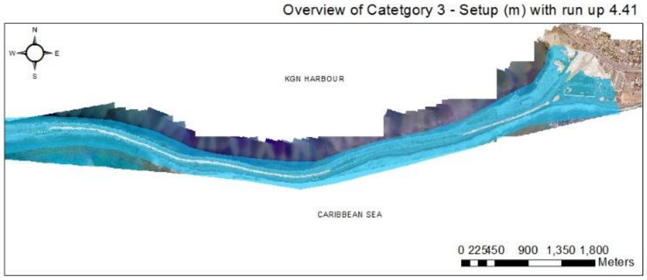


(c) Category 5 Storm Event with runup

Table 11.6 Inundation levels associated with the worst case storm surge modeling (40 KM Parallel offshore of site)



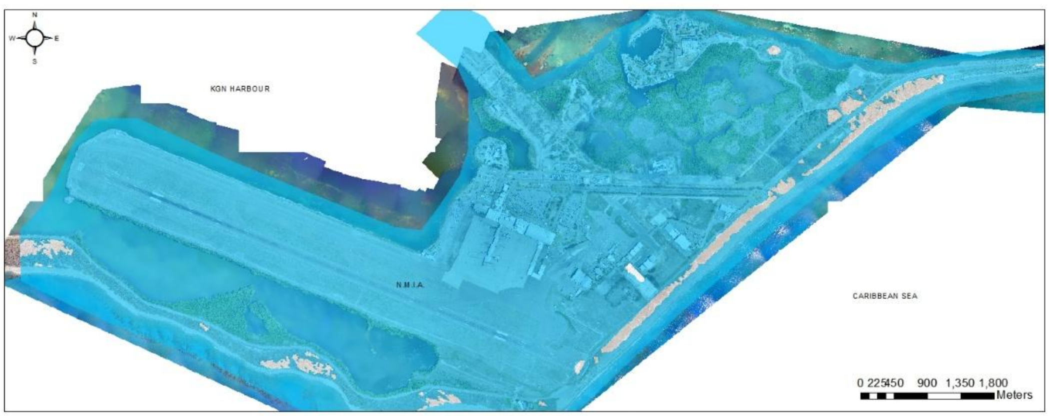
Overview of Catetgory 3 - Setup (m) with run up 4.41



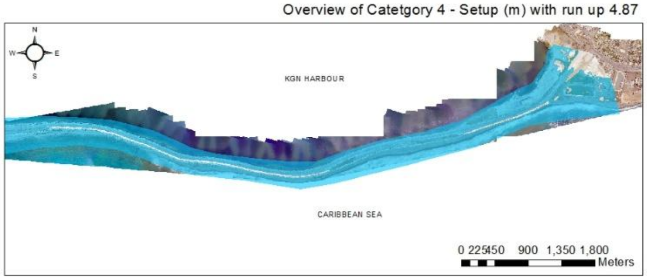
Overview of Catetgory 3 - Setup (m) with run up 4.41

CEAC SOLUTIONS

(a) Category 3 Storm Event with runup



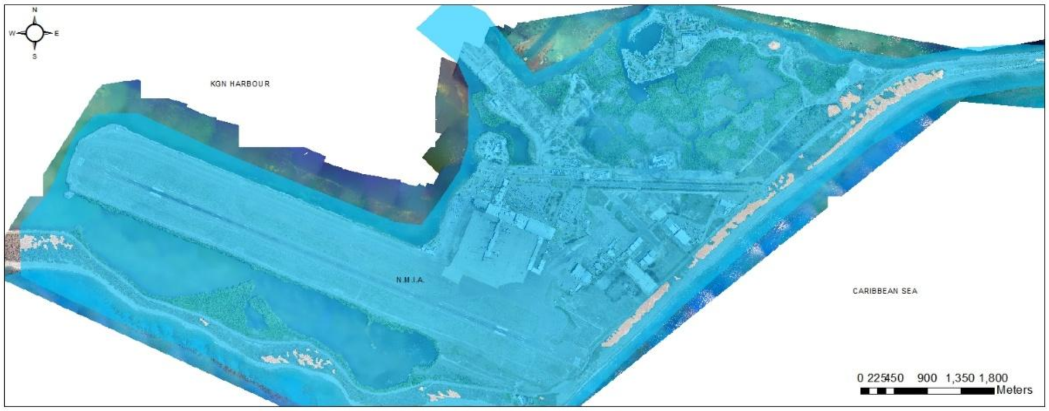
Overview of Catetgory 4 - Setup (m) with run up 4.87



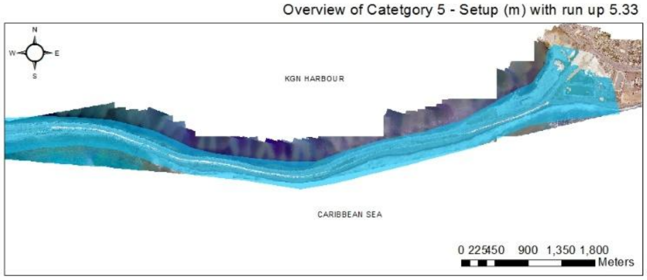
Overview of Catetgory 4 - Setup (m) with run up 4.87

CEAC SOLUTIONS

(b) Category 4 Storm Event with runup



Overview of Catetgory 5 - Setup (m) with run up 5.33

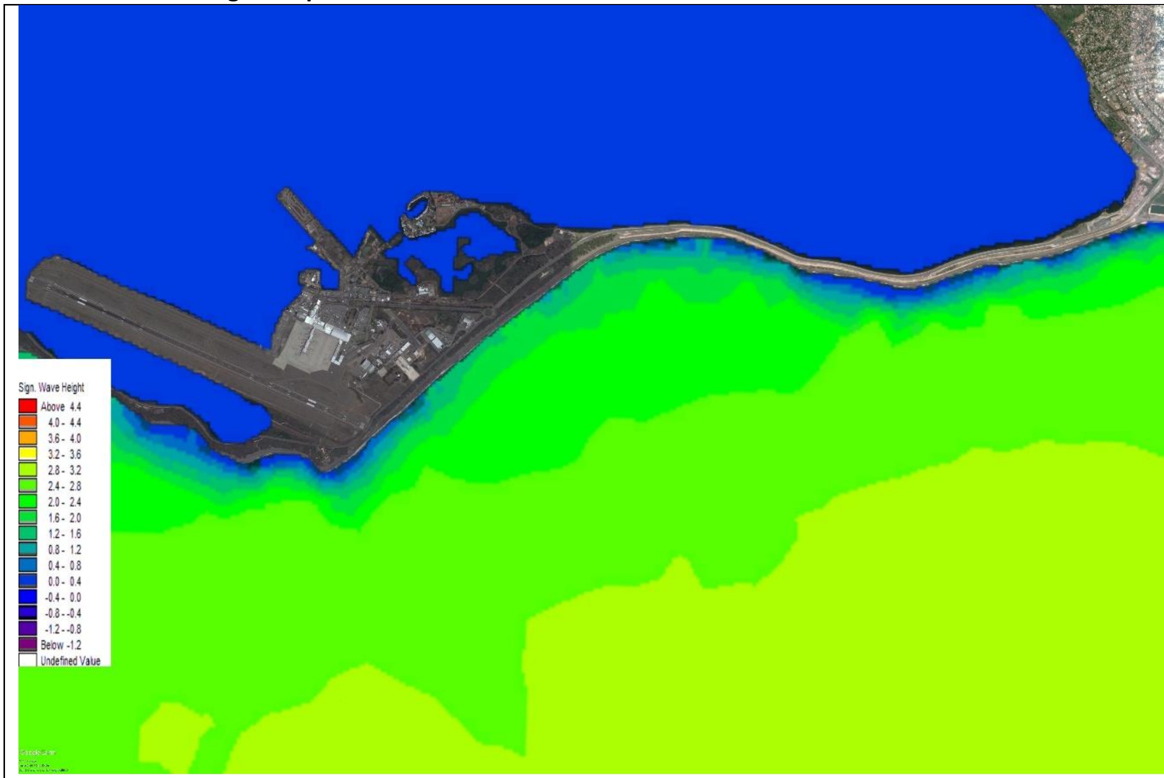


Overview of Catetgory 5 - Setup (m) with run up 5.33

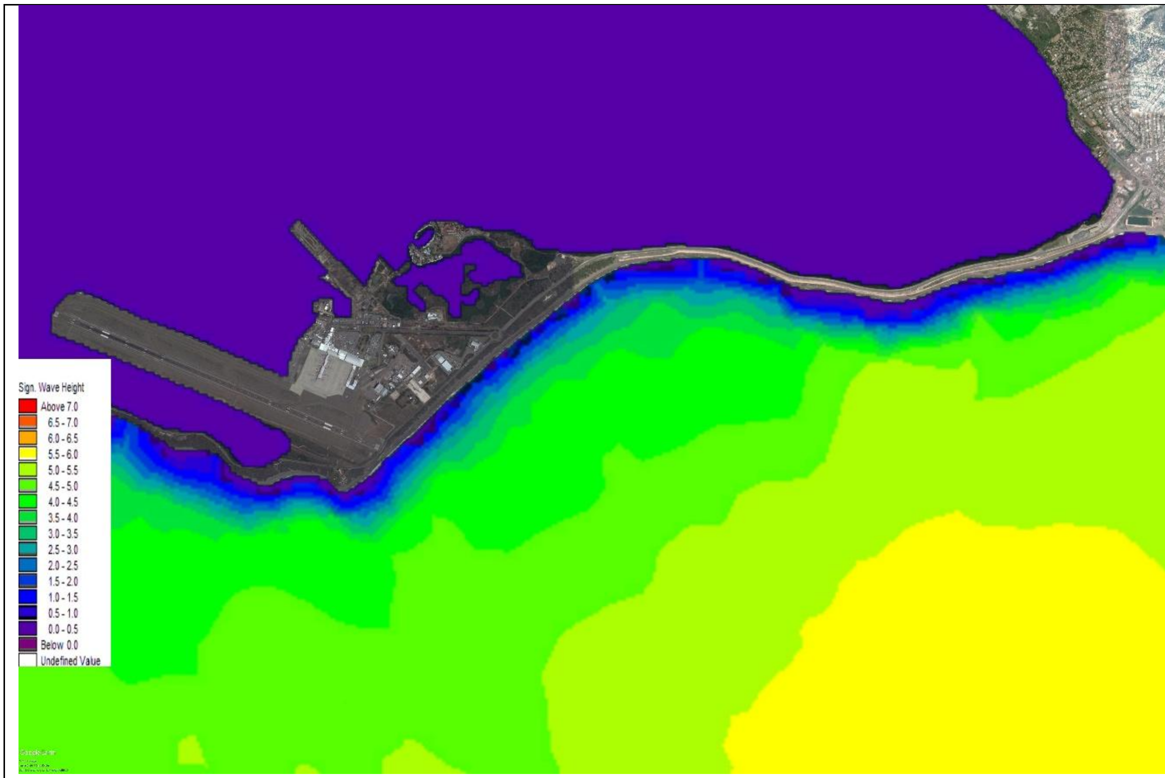
(c) Category 5 Storm Event with runup

11.5 Wave Modeling Plots

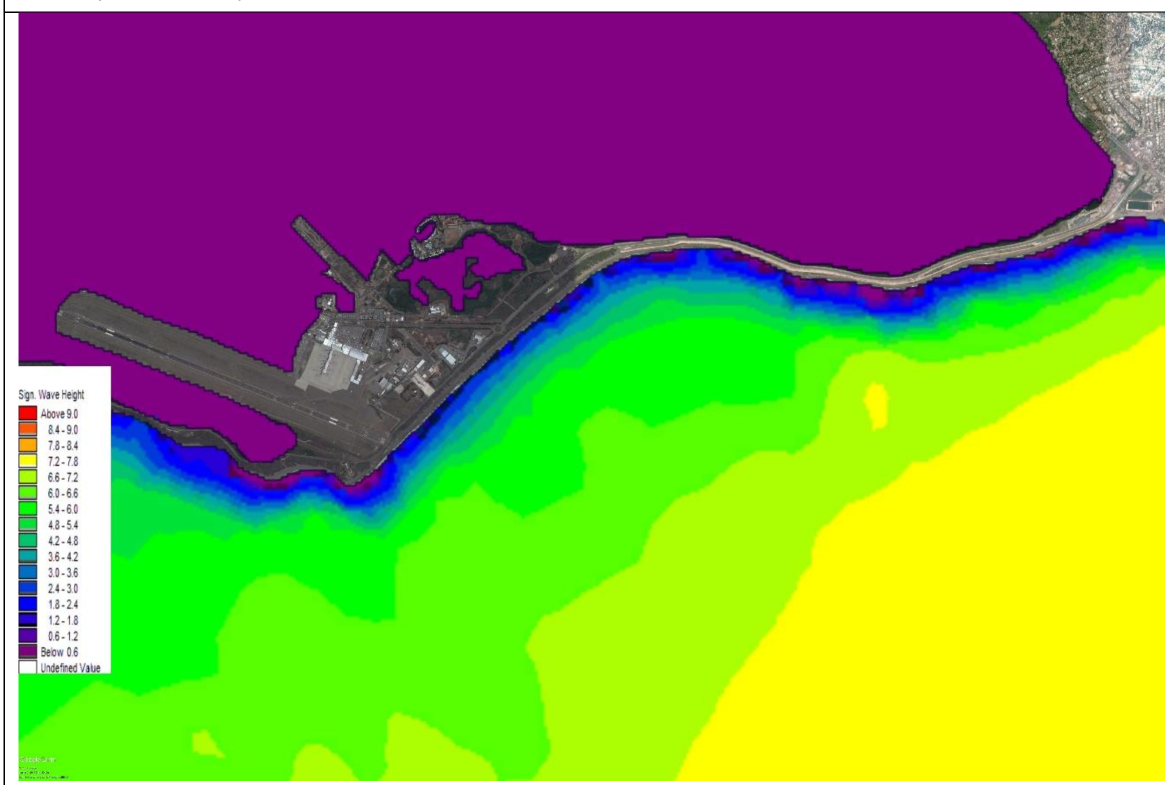
Table 11.7 Wave heights expected to reach shore under current climate scenario



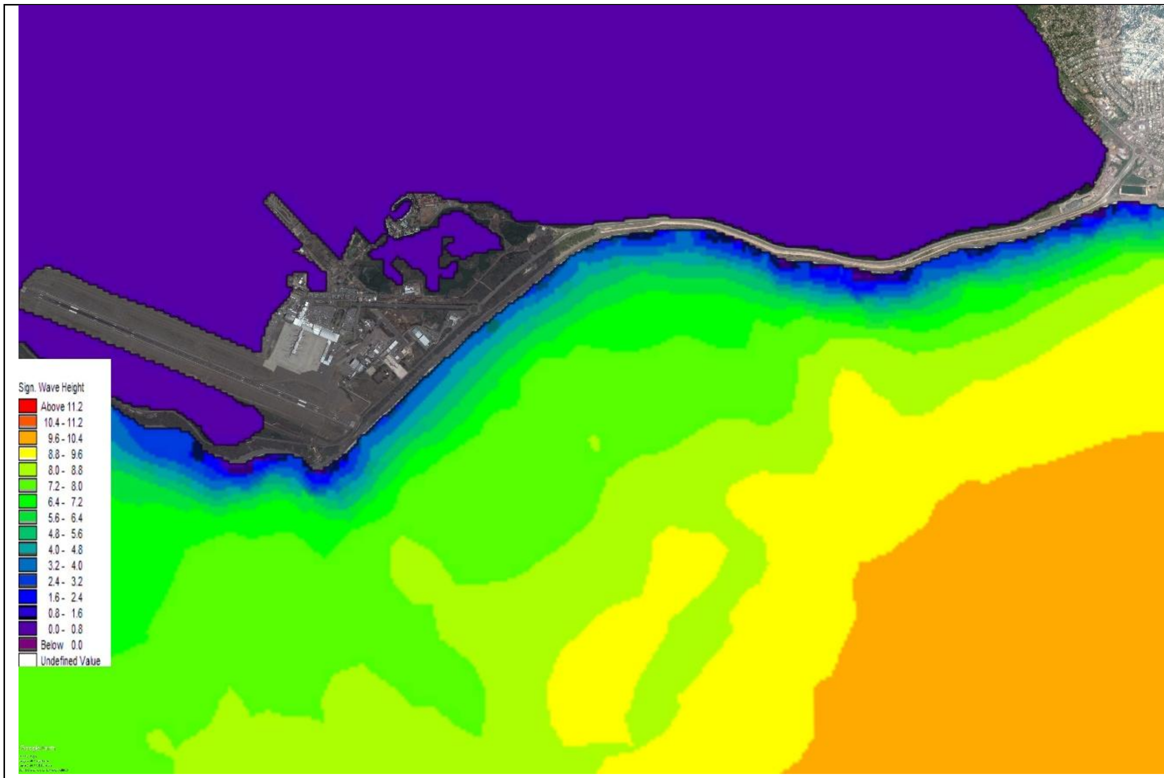
(a) 10 year return period



(b) 25 year return period

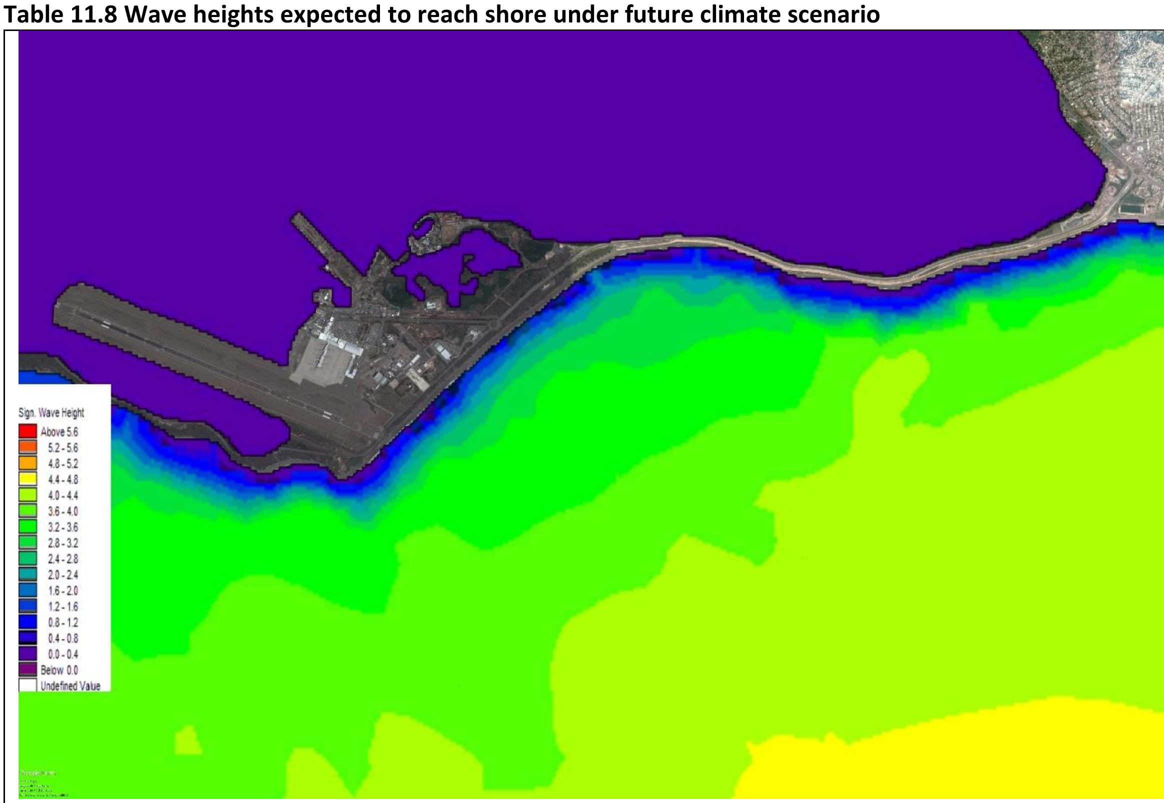


(c) 50 year return period

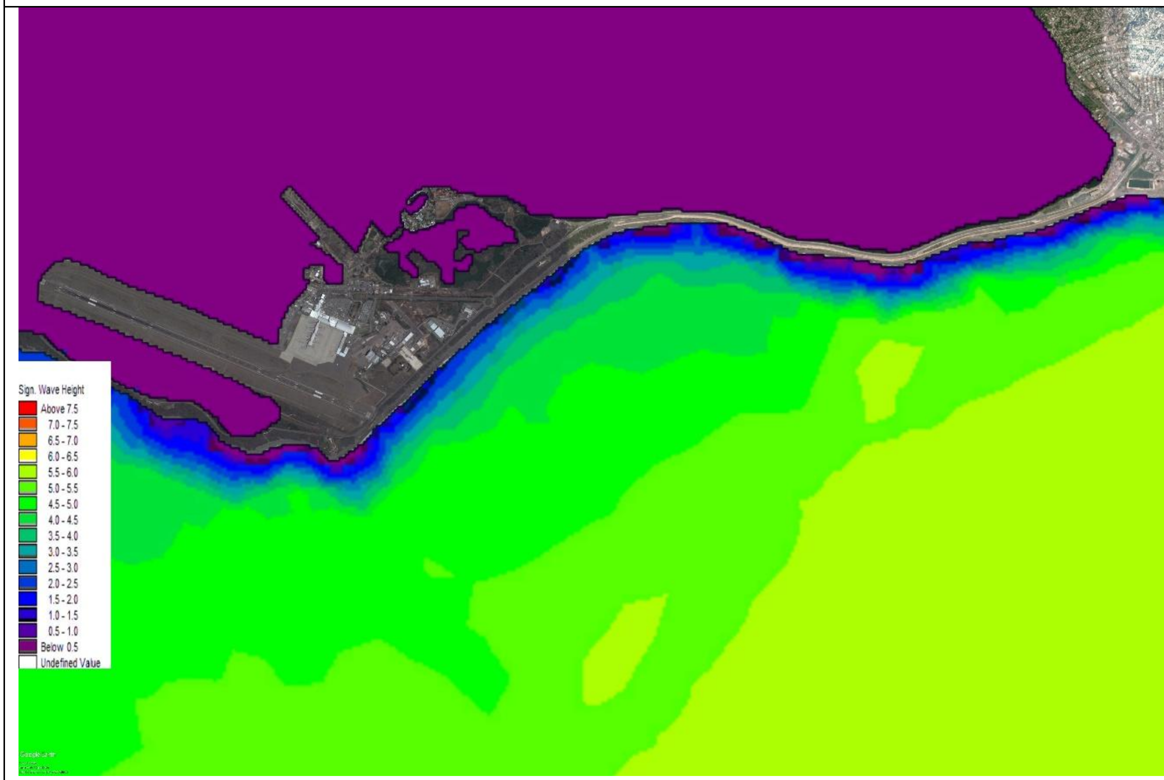


(d) 100 year return period

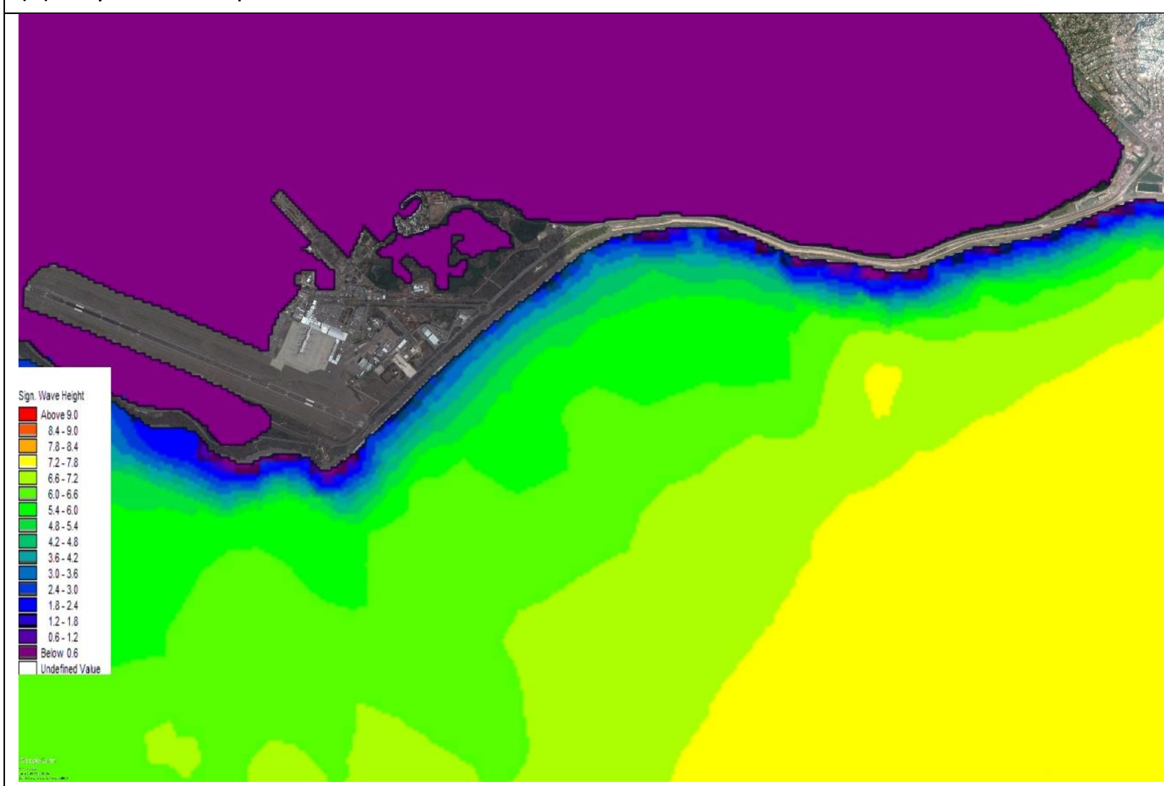
Table 11.8 Wave heights expected to reach shore under future climate scenario



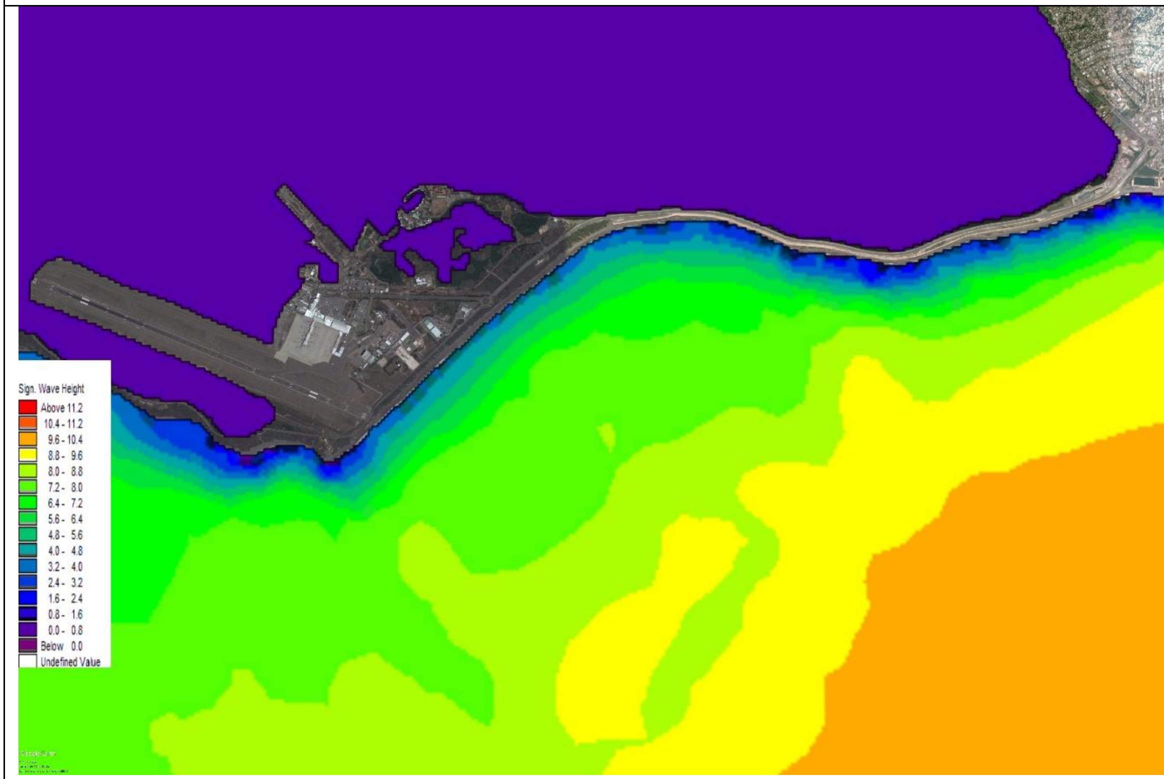
(a) 10 year return period



(b) 25 year return period



(c) 50 year return period



(d) 100 year return period

11.6 Shoreline Erosion

Table 11.9 Summary of results showing the expected shoreline erosion under pre-project conditions.

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (50 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse				
SSW	0	0	0	0
S	64	1.7	71	2.3
SSE	42	1.2	62	2.7
SE	52	1.4	62	1.1
ESE	40	0.6	62	1
End of Runway				
SSW	0	0	29	0.4
S	35	0.7	56	1.1
SSE	34	0.9	61	2.3
SE	42	1.3	45	2
ESE	42	1	44	2.3
Dune				
SSW	139	0.4	142	0.8
S	0	0	0	0
SSE	41	0.1	73	0.9
SE	40	0.2	75	0.6
ESE	38	0.2	63	0.4
Low Revetment				
SW	20	0.3	40	0.6
SSW	31	0.9	34	1.1
S	33	0.8	33	1.3
SSE	30	1.1	30	1.1
SE	29	1.3	32	1.4
ESE	0	0	0	0
High Revetment				
SW	0	0	0	0
SSW	0	0	0	0
S	0	0	0	0
SSE	0	0	0	0
SE	0	0	0	0
ESE	0	0	0	0

Table 11.10 Summary of results showing the expected shoreline erosion under post-project conditions.

Profile	Inland Reach of Erosion(50 year)(m)	Max Vertical Erosion (100 year)(m)	Inland Reach of Erosion(100 year)(m)	Max Vertical Erosion (100 year)(m)
Lighthouse				
SSW	24	0.8	24	0.9
S	26	0.7	26	0.9
SSE	13	0.9	13	1.1
SE	13	1	13	1.2
ESE	14	0.8	14	1
End of Runway				
SSW	32	0.2	31	0.4
S	17	0.3	17	0.5
SSE	13	0.4	13	0.7
SE	10	0.5	11	0.7
ESE	13	0.5	13	0.8

11.7 Overtopping Analysis

Table 11.11 summary of overtopping calculations for rock revetment along the Palisadoes

Overtopping Calculations on Revetment						
Parameter	NMIA-30 Dune Revetment	NMIA-30 Rock Revetment	Palisadoes High Revetment	Palisadoes Low Revetment	Proposed Palisadoes Dune Revetment	Unit
Revetment Section	**1**	**2**	**3**	**4**	**5**	
Crest Elevation	+5.00	+4.83	+6.40	+4.16	+5.70	m
Hs	3	3	3.5	3.5	3.5	m
Water elevation+setup	4.33	4.33	4.33	4.33	4.33	m
Slope (1:n)	2	2	2	2	2	
Berm Width, B	8	6	0	0	0	m
Slope (1:n)- upper slope	7	2	2	2	2	
Height of lower slope- H1	4.5	4.3	6.0	2.0	2.0	m
Height of upper slope- H2	0.50	0.30	0.40	2.16	3.70	m
Imaginery slope, S'	29.7	20.0	2.0	2.0	2.0	
Overtopping calculations-SIMPLE SLOPES						
A	9.39E-03	9.39E-03	9.39E-03	9.39E-03	9.39E-03	
B	21.6	21.6	21.6	21.6	21.6	
Overtopping calculations-BERMED SLOPES						
A	1.00E-02	1.00E-02	9.39E-03	9.39E-03	9.39E-03	
B	300	300	21.6	21.6	21.6	
Roughness Coefficient, r						
lower slope	0.55	0.55	0.55	0.55	0.95	
upper slope	1	0.55	0.55	0.55	0.95	
Weighted roughness	0.68	0.55	0.55	0.55	0.95	
Tp	9.18	9.18	9.92	9.92	9.92	sec
Tm	5.97	5.97	6.45	6.45	6.45	sec
R*	0.01	0.01	0.05	-0.01	0.03	
Q*						
simple slope	0.01	0.01	0.00	0.01	0.00	
bermed slope	0.00	0.00	0.00	0.01	0.00	
Q						
simple slope	**1.15**	**1.15**	**0.30**	**3.06**	**1.03**	m3/m.s
bermed slope	**0.03**	**0.01**	**0.30**	**3.06**	**1.03**	m3/m.s
Tide	**0.20**	**0.20**	**0.20**	**0.20**	**0.20**	

Design Crest elevation	**5.00**	**4.83**	**6.40**	**4.16**	**5.70**	m

11.8 Rock Armour Sizing

Table 11.12 Summary of proposed revetment rock sizing

ROCK ARMOUR STABILITY (NEARSHORE VAN DE MEER)				
	Project:	**NMIA Runway 30 End**		
	Location/Structure:	**High**	**Dune**	
		Revetment	**Revetment**	
1.00	**Design Parameters**	**Value**	**Value**	**Units**
1.01	Porosity, P	0.4	0.4	
1.02	Structure slope	2	2	
1.03	Structure slope (TAN)	0.5	0.5	
1.04	Number of waves	3000	3000	
1.05	Nearshore slope	2.41%	2.41%	
1.06	Offshore depth	5.0	5.0	
1.07	Nearshore depth	0.5	0.5	
1.08	Distance	187	187	
1.09	Angle of Seaward Slope (of structure)	31.3	31.3	
1.10	Density			
1.11	Armour stone	2450	2450	Kg/m3
1.12	Seawater	1024	1024	Kg/m3
1.13	Damage level	2.0	2.0	
2.00	**WAVES**	**Value**	**Value**	**Units**
2.01	**Deepwater Wave**			
2.02	Length	231.2	231.2	metres
2.03	Height	12.2	12.2	metres
2.04	Period	12.2	12.2	seconds
2.05	**Shallow water wave characteristics**			
2.06	Assume a value for L1	78	78	metres
2.07	Wave length, L2 =	78	78	metres
2.08	Depth (MSL)	0.0	0.0	metres
2.09	Nearshore Wave Height, Hs	1.8	1.8	metres

2.10	Hmax, H2%	2.8	2.8	metres
2.11	Steepness	0.023	0.023	
3.00	**DESIGN CONFIGURATION**			
3.01	Ibarren Number	3.282	3.282	
3.02	Ibarren comment	Plunging	Plunging	
3.03	Submergence (MSL) pre storm	0.0	0.0	metres
3.04	Height of structure (before storm surge)	0.00	0.00	metres
3.05	Storm surge elevation (above MSL)	4.33	4.33	metres
3.06	Depth of water (during storm surge)	4.33	4.33	metres
4.00	**ARMOUR SIZING**	**Value**	**Value**	**Units**
4.01	**Calculations**			
4.02	Delta	1.39	1.39	
4.03	Ns*	2.10	2.10	
4.04	**Dn50**	0.96	0.96	**metres**
4.05	D50-Toe	0.61	0.61	
4.06	Rc	4.33	4.33	metres
4.07	Reduction factor for submergence (Rd)	1.82	1.82	
4.08	**D50 (Rd)**	**1.75**	**1.75**	**metres**
ARMOUR				
4.08	**Grading**	1.5	1.5	
4.09	D15 (min)	0.8	0.8	metres
4.10	**D50**	**0.96**	**0.96**	**metres**
4.11	D85 (max)	1.15	1.15	metres
4.12	M15 (min)	1104	1104	Kg
4.13	**M50**	**2157**	**2157**	**Kg**
4.14	M85 (max)	3727	3727	Kg
4.15	Layer thickness	1.92	1.92	metres
FILTER				
4.13	**Grading**	1.5	1.5	
4.14	D15 (min)	0.4	0.4	metres
4.15	**D50**	**0.48**	**0.48**	**metres**
4.16	D85 (max)	0.58	0.58	metres

4.17	M15 (min)	138	138	Kg
4.18	**M50**	**270**	**270**	**Kg**
4.19	M85 (max)	466	466	Kg
4.20	Layer Thickness	1.44	1.44	metres

11.9 Engineering estimate