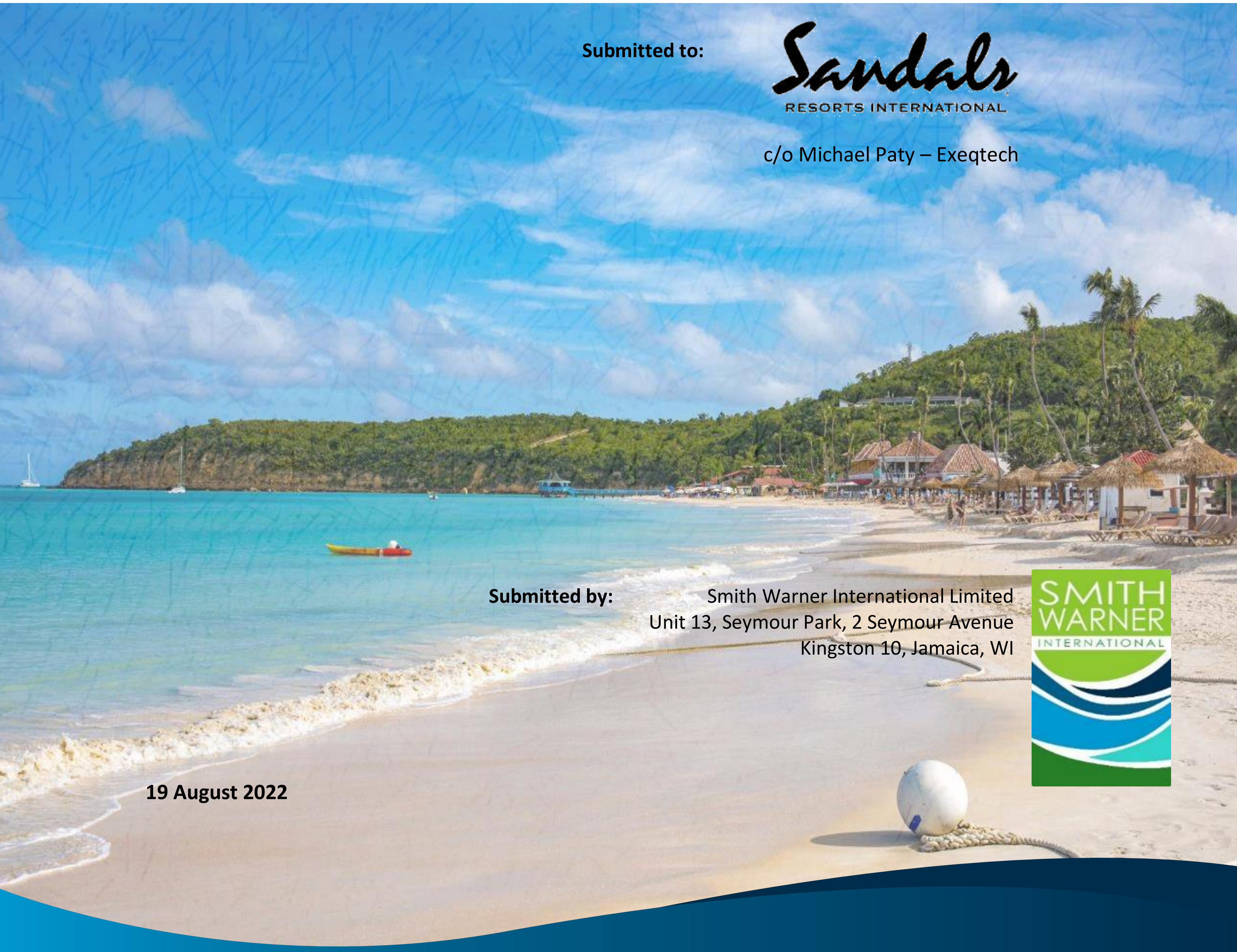
# Coastal Processes Analysis and Impact Assessment Report

# *for* Sandals Grande Antigua



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**19 August 2022**

# Table of Contents

# Executive Summary

The Sandals Resorts International (SRI) group of companies wants to construct overwater suites at their resort in Dickenson Bay, Antigua – the Sandals Grande Antigua. To facilitate the furtherance of the design along with its approval by the regulatory agencies, the following was necessary:

- Determination of the relevant design parameters to optimize functionality and avoid collapse during a storm, (such as wave forces on piles and floor levels to minimize overtopping); and
- Analysis of the potential marine and coastal impacts and provision of the baseline assessment and strategies to alleviate potential impacts to the marine environment.

To achieve these objectives, a review of data available for the site was undertaken as well as field data collection exercises. Specifically, bathymetric and shoreline profile surveys were undertaken along with sediment sampling exercises and benthic observations. The findings from the data collection activities are summarized in the graphic below.

***Bathymetry (Section 2.1)***

- Uniform and consistent across the length of the Bay. Notable exception by the headland to the north.
- Depths are shallow, ranging from 1m (at 11-14m from the shoreline) to 3m (at roughly 120m from the shoreline), which amounts to a roughly 2% slope.

***Sediment (Section 2.2)***

- Grain size on beach - medium-sized sand
- Grain size in nearshore - fine-grained sand.
- Fine sand in nearshore is contributing to high alongshore transport (shown by the LitDrift model)

***Benthos (Section 2.3)***

- No coral and/or reef habitat in the immediate project area.
- There are extensive seagrass beds in the immediate project area.
- Data suggests beds are retreating i.e. moving further offshore.
- These seagrass beds are within the impact zone of the proposed overwater suites (those most seaward).
- This seagrass will have to be removed and relocated to similar areas offshore prior to the start of construction.

Using the data collected, numerical models were built to simulate the behaviour of the existing coastal processes to form the baseline for all future impact analysis. The findings of the baseline conditions analysis are summarized in the graphic below.

| | |
|---|---|
| ***Operational Wave Climate (Section 3.1)*** | • Offshore waves approach the project site from the north-northeast.<br>• Incident wave heights are uniform along most of the shoreline.<br>• 50$^{th}$ percentile wave conditions are quite low ranging from 0.1m to 0.3m.<br>• 99$^{th}$ percentile wave condition wave heights range from 0.3m to 0.65m. |
| ***Hurricane Climate (Section 3.2)*** | • The worst case scenario of water level was used - high tide (0.20m), IBR of eye of storm passing over site (0.34m), GSLR of most extreme RCP (15mm/year consistently over a 50-year project life cycle).<br>• Of the 6 directional approaches simulated, the worst was determined to be the northwest. Northwest 50-yr return period became the "design storm".<br>• Hurricane simulations resulted in wave heights from 1.69m to 2.07m along the shoreline, and static storm surge levels of approximately +1.1m in the nearshore to +1.4m. |
| ***Shoreline Morphology (Section 4)*** | • Historical satellite imagery analysis pointed to an overall pattern of erosion, albeit at low rates: -0.06m/yr at the south and -0.64m/yr at the north.<br>• LitDrift analysis showed a distinct pattern in annual alongshore sediment transport to the south-southwest.<br>• The analysis also showed that the active zone of sediment transport extends typically from the shoreline out to 30m offshore at depths of 0m to -1.8m relative to MSL. |

From the simulations conducted, wave heights and storm surge values were extracted at the named extraction points for each scenario modelled. These results are summarized in Table 3-2 with additional results shown in Table 6-3.

The design deck elevation was developed based on deconstructing the water below the deck into its various components: Stillwater level (SLR, HAT), wave crest and freeboard. The analysis resulted in a recommended design elevation of +1.6m above MSL for the deck. This design level would not only minimize inundation in a storm event, but also allow for minimal splashing and disturbance from swell waves (up to the 99.99$^{th}$ percentile condition for operational waves) both now and into the future. However, this level is not high enough to withstand complete inundation in any storm event.

The design forces were calculated using this recommended design elevation along with the design parameters. Forces of slam, drag, and inertia on the piles and the sub-structure beam were considered, and vertical uplift pressures on the deck were calculated. These calculation values are outlined in Figure 6.10 and Figure 6.11 for the design storm events considering a 50-year project life cycle. It should be noted here that representational cross-sections were used in the analysis. Should the design cross-sections change significantly the design wave forces will have to be recalculated.

Having established the baseline conditions inclusive of day-to-day, swell and extreme wave events, along with sediment transport patterns, the pile layout of the proposed overwater suites were then input to the model and the scenarios re-run to determine impacts on currents, waves, and bed level change. Again, design assumptions were made based on previous Sandals' layouts, namely the piles were assumed to be 0.6m in diameter and spaced at 5m. This design was found to be suitable as it created little to no noticeable downdrift impacts on hydrodynamics or sediment movement along the Dickenson Bay shoreline. Therefore, it is envisioned that there will be no impacts on coastal processes as long as the suites are constructed as outlined. Should there be any change to the design that results in either an increase in pile diameter or a decrease in pile spacing, the new design would have to be remodelled and potential impacts noted. At this time, the only recommendation is to provide an allowance of 0.15m in pile depth to account for potential erosion of the seabed.

The most significant impacts to the marine environment will be on the seagrass beds beneath the most seaward portion of the structure, i.e., the heart shaped portion. Any construction done in the area will lead to deleterious effects on the seagrass beds both during the construction period (construction works leading to increased turbidity in the water and physical damage to the seagrass present) and afterwards (direct shading of the seagrass, changes in substrate regimes).

To mitigate the negative impacts to the seagrass, relocation is recommended. The beds should be removed using the modified Mat Method and replanted using the Staple Method to any of the numerous sites in the Dickenson Bay area that are in close proximity and have bare substrate. This is outlined in further detail in Chapter 8 of this report.

The owners should consider adjusting the proposed configuration to be slightly more landward. This is a key recommendation coming out of the analysis. As currently configured, the heart shaped portion of the suites will be constructed in water depths of roughly 3.3m (~11'), creating a construction pad and using land-based machinery for construction may not be feasible in these depths. Adjusting the layout so that this portion would be moved landward, would result in lower depths in which the suites would be placed. This would have several positive impacts:

- Shallower water would make for easier (and likely cheaper) construction.
- Shallower water would likely mean lower wave forces.
- Pulling the configuration more landward would remove it from the seagrass beds and therefore removal and relocation would likely not be required.

# 1 Introduction

## *1.1 Background*

The overall focus of this investigation is to assess the potential coastal impacts of the "Overwater Village" being proposed at the Sandals Grande hotel. The Sandals Grande hotel is in Dickenson Bay – "Antigua's best and most famous beach"[1]. The only flagged Sandals Resort in Antigua, shown in Figure 1.1, was opened in 1992 and has not been renovated in its 30-year history. To modernize and revitalize the hotel, expansion and renovation works are now in the works.

*Figure 1.1 View of existing layout of Sandals Grande Hotel, Dickenson Bay (left of frame)*

The draft Master Plan for the hotel indicates the plan for the creation of an 'Overwater Village' consisting of a total of sixteen (16) overwater suites of varying sizes connected by a boardwalk supported on piles. There are additional works also being considered; new keys are being added – an increase in total keys of roughly 72%; some elements are being relocated (the spa and gym) and some buildings / keys will see their interiors renovated.

## *1.2 Scope of Works*

Due largely to the potential for negative impacts from the overwater suites, many governments and corporations have now started to closely monitor the design and construction of these coastal elements to ensure that due regard is given to potential marine impacts. Smith Warner International (SWI) was contracted to develop this marine and coastal impact assessment (MCIA) for the proposed overwater suites at the Sandals Grande hotel. The MCIA will then be used as input to the larger Environmental Impact Assessments (EIA) for the expansion project.

[1] https://www.sandals.com/grande-antigua/

The overarching objectives of this project are to:

(1) Provide the relevant design parameters to optimize functionality and avoid collapse during a storm, (such as wave forces on piles and floor levels to minimize overtopping); and

(2) Conduct a Marine and Coastal Impact Assessment and provide the baseline assessment and strategies to alleviate potential impacts to the marine environment.

The first step in examining the potential impacts of changes to a system is understanding in detail the existing dominant processes in that system, so that changes can be easily quantified. To that end, Stage 1 of this project was focused on establishing baseline conditions in the nearshore of Dickenson Bay. That stage had been previously completed and submitted to the client. However, due to the significance of that work in forming the baseline for analysis, the results of Stage 1 are again presented herein as Part 1 of this report.

Part 2 of this report focuses on the proposed design of the suites, considerations of water level and overtopping, while determining the behaviour of the suites under swell wave conditions, specifically how the piles will disturb, if at all, the existing coastal processes.

The tasks undertaken throughout the entire marine and coastal impact assessment (MCIA) project are outlined in Figure 1.2 below.

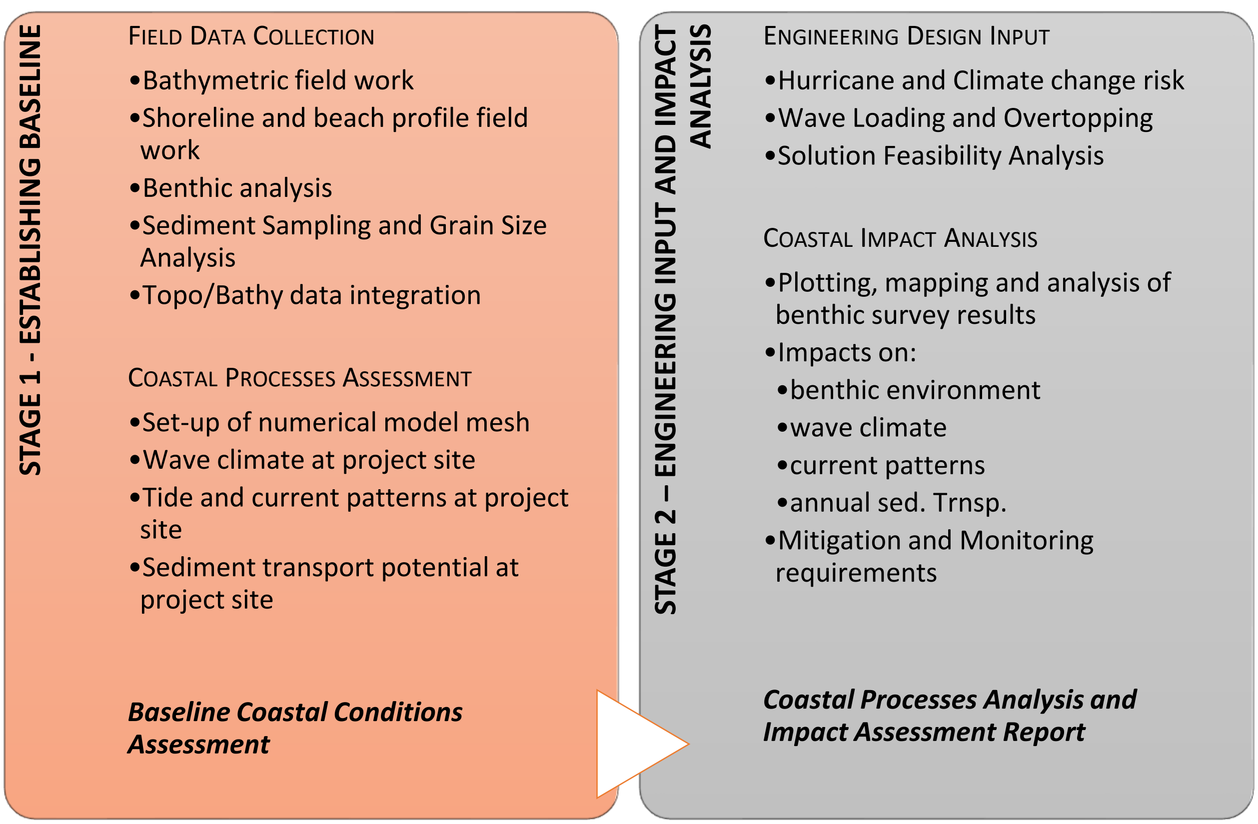



*Figure 1.2 Graphic outlining main stages, tasks and deliverables in approach to Scope of Works*

# PART 1 – BASELINE COASTAL CONDITIONS

# 2 Field Data Collection

## 2.1 Topography, Drone and Bathymetric Data

Water depths in the nearshore area are important baseline information when carrying out coastal investigations. They are used to:

- Determine baseline wave and hydrodynamic conditions and sediment transport rates through the computer model simulations for which they are the main input, and
- Determine suitability for the overwater suite locations as well as the depth that would impact functionality.

To generate accurate, high-resolution depth information of the sea floor, we obtained satellite-derived bathymetric (SDB) data from a provider (Figure 2.1 - top). This SDB data (available on a 10m grid spacing) was previously validated against a bathymetric dataset collected for a nearby area of the coast. The satellite data fit relatively well with the measured data offshore. However, in the nearshore, SDB data can become less reliable, interfered with by breaking waves and higher sediment loads, and therefore it was supplemented by survey data. A licensed marine surveyor visited the site on 30 April 2022 and conducted a bathymetric survey along with a shoreline profile survey; this data is shown in Figure 2.1 (bottom).

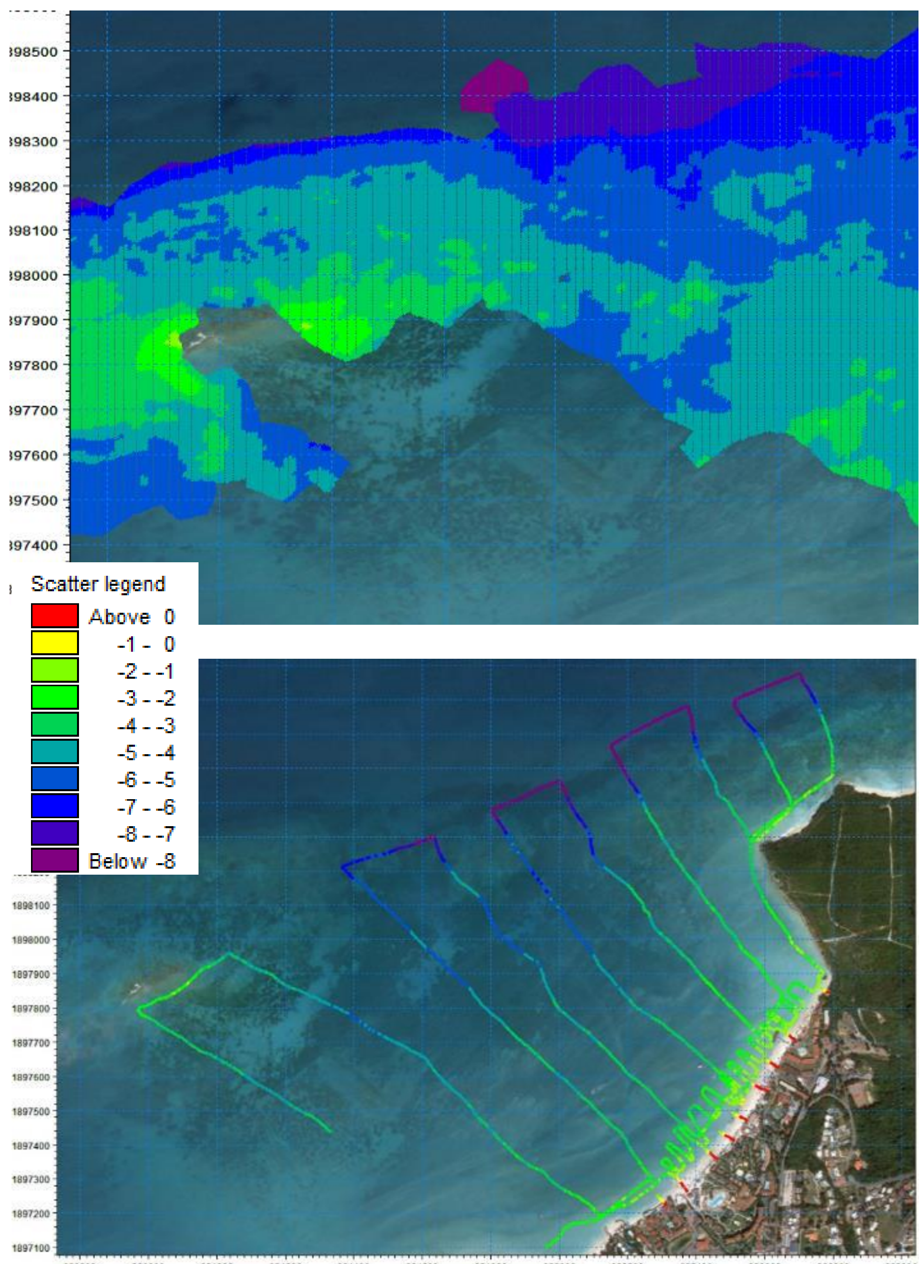



*Figure 2.1 Survey points from satellite derived bathymetry (top) and bathymetric and beach profile surveys (bottom)*

An aerial drone survey was also carried out. The goal of this survey was to obtain a high-resolution image of the nearshore and the shoreline, which will assist in placement of the proposed overwater suites.

To track offshore waves to the shoreline, offshore bathymetric data is also required. All the nearshore data collected was therefore merged with data from offshore nautical charts in the MapSource (Garmin HomePort) database, which includes spot elevations and contour lines in and around both the islands of Antigua and Barbuda. A screenshot of the available offshore data is shown in Figure 2.2.

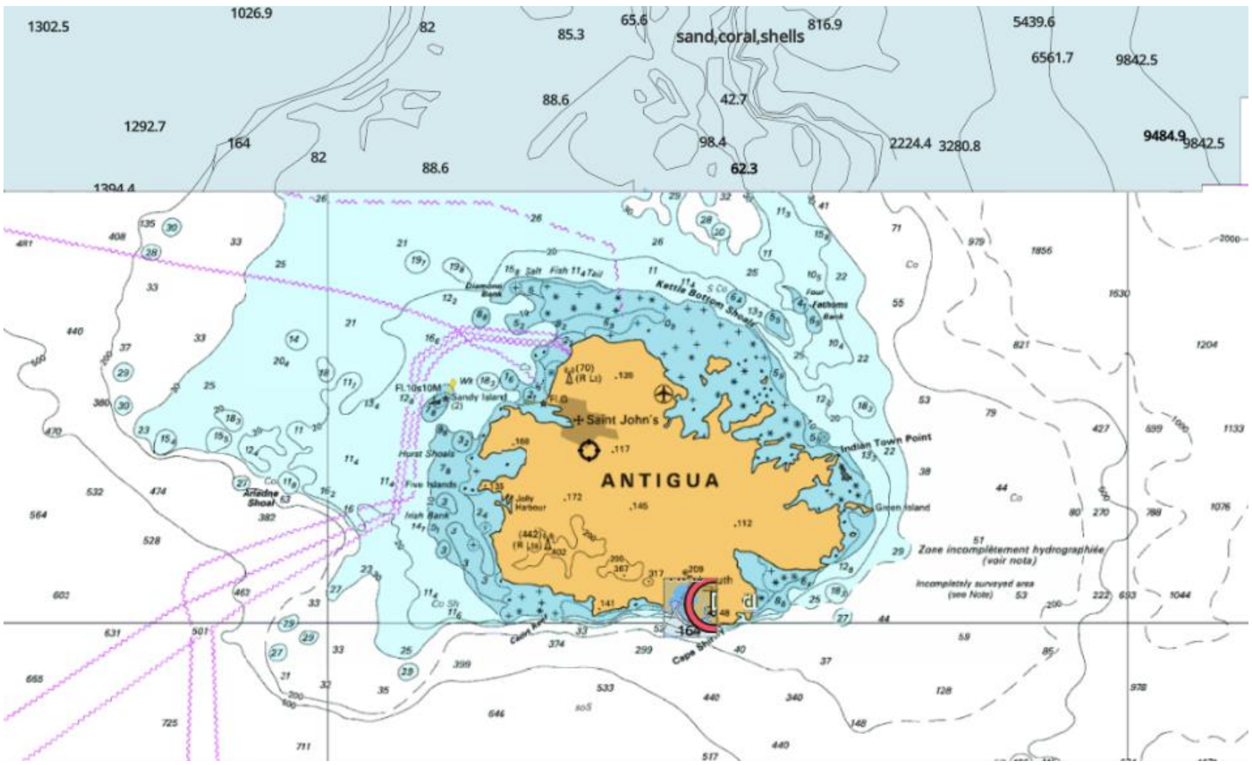



*Figure 2.2 Image of nautical chart showing offshore bathymetric data available*

Figure 2.3 depicts the final gridded digital elevation, which combines the beach profile and bathymetric surveys, the satellite-derived bathymetry in the nearshore area, and offshore nautical chart data.

The contour plot reveals that the water in Dickenson Bay is rather shallow, ranging from 1m (at 11-14m from the shoreline) to 3m (at roughly 120m from the shoreline). The plot also shows that the depth contours in the bay are relatively uniform across its length, except for a slightly shallower area towards the headland to the north.

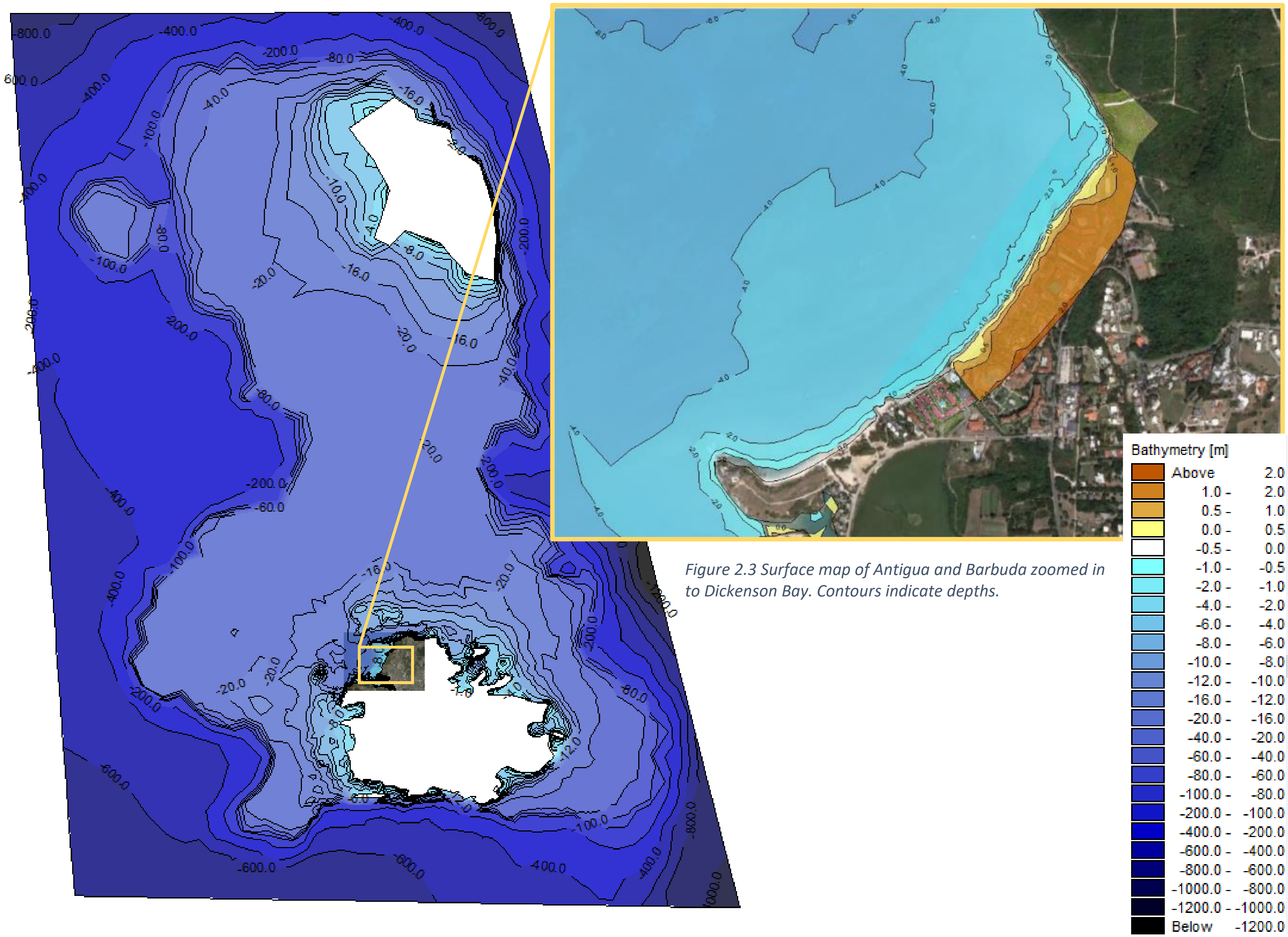



*Figure 2.3 Surface map of Antigua and Barbuda zoomed in to Dickenson Bay. Contours indicate depths.*

## *2.2 Sediment Analysis*

For the sediment analysis, sediment samples were collected at five (5) sampling locations. These sample locations are shown in Figure 2.4 and can be described as:

- the southern end of the Sandals beach (SS1),
- the approximate centre of the Sandals beach (SS2),
- the approximate footprint of the proposed suites** jetty (SS3) and centre (SS4), and
- the northern end of the Sandals beach (SS5),

**It should be noted that the footprint originally provided was used to select these locations; however, the footprint of the suites has since changed.


*Figure 2.4 Locations of samples collected from nearshore of project site.*

For sediment analysis, the five (5) sediment samples were visually inspected, air-dried and subjected to a standard dry sieve analysis to determine the grain size distribution as well as other characteristic parameters. Table 2-1 summarizes the results of the sieve analysis.

The results show that the sand is generally classified as fine to medium grained sand, with the onshore samples (SS1, SS2 and SS5) being medium-grained and the offshore samples (SS3 and SS4) being fine grained. This difference was also reflected in the silt content, with the onshore samples having very low silt content (≤0.1%) while the offshore samples were composed of 0.7% and 1.8% silt or clay at SS4 and SS3 respectively. Figure 2.5 shows the grain size in mm plotted against the percentage in the sample. The graph again confirms the disparity, showing clearly that all the onshore samples are coarser than the offshore samples.

*Table 2-1 Grain Size Results for sediment samples collected*

| Sample Number | GRAIN SIZES (mm) | | | | %Gravel | %Sand | %Silt / Clay | $C_c$ | $C_u$ | Wentworth Classification |
|---|---|---|---|---|---|---|---|---|---|---|
| | D60 | D50 | D30 | D10 | | | | | | |
| **SS1 - south** | 0.345 | 0.296 | 0.204 | 0.136 | 0.0 | 99.9 | 0.0 | 0.887 | 2.537 | Medium Sand |
| **SS2 - central** | 0.348 | 0.306 | 0.222 | 0.150 | 0.2 | 99.8 | 0.0 | 0.944 | 2.320 | Medium Sand |
| **SS3 - overwater** | 0.150 | 0.131 | 0.098 | 0.072 | 0.3 | 97.9 | 1.8 | 0.889 | 2.083 | Fine Sand |
| **SS4 - overwater** | 0.159 | 0.143 | 0.108 | 0.077 | 0.8 | 98.5 | 0.7 | 0.953 | 2.065 | Fine Sand |
| **SS5 - north** | 0.335 | 0.264 | 0.186 | 0.135 | 0.4 | 99.5 | 0.1 | 0.765 | 2.481 | Medium Sand |

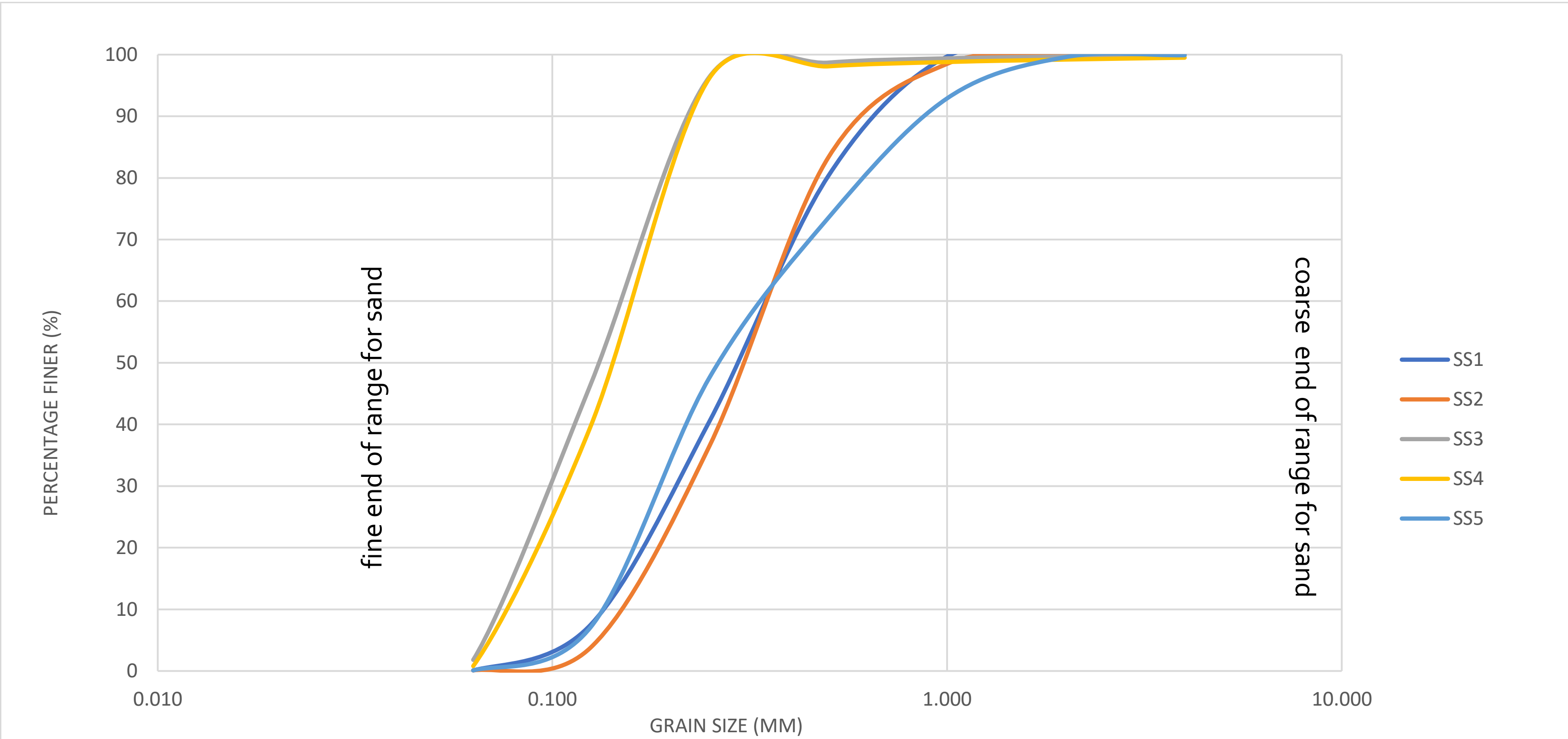



*Figure 2.5 Grain size analysis for collected sand samples*

## 2.3 Benthic Survey

A benthic analysis was conducted using several methodologies: desktop research of existing databases, aerial imagery interpretation and ground-truthing to define and map the character of the seafloor in the project area nearshore.

### 2.3.1 Existing Benthic Data

The Antigua Marine Life Project is a volunteer group with a mandate to record and catalogue “the marine species found in the inshore (+brackish waters) (to depths of ~30m/ 100ft) of Antigua. It employs photography to ensure the veracity of the records, held in a central database.” That database promises to “continue to catalogue the marine life found in [Antigua’s] coastal waters and provide an up-to-date, public resource for interested parties.”

As part of this investigation the database was searched for relevant information on the project area. The maps shown in Figure 2.6 indicate that while there is no coral reef habitat in Dickenson Bay, there are reportedly seagrass beds. Further, according to the map, the seagrass here is classified as “dense”.

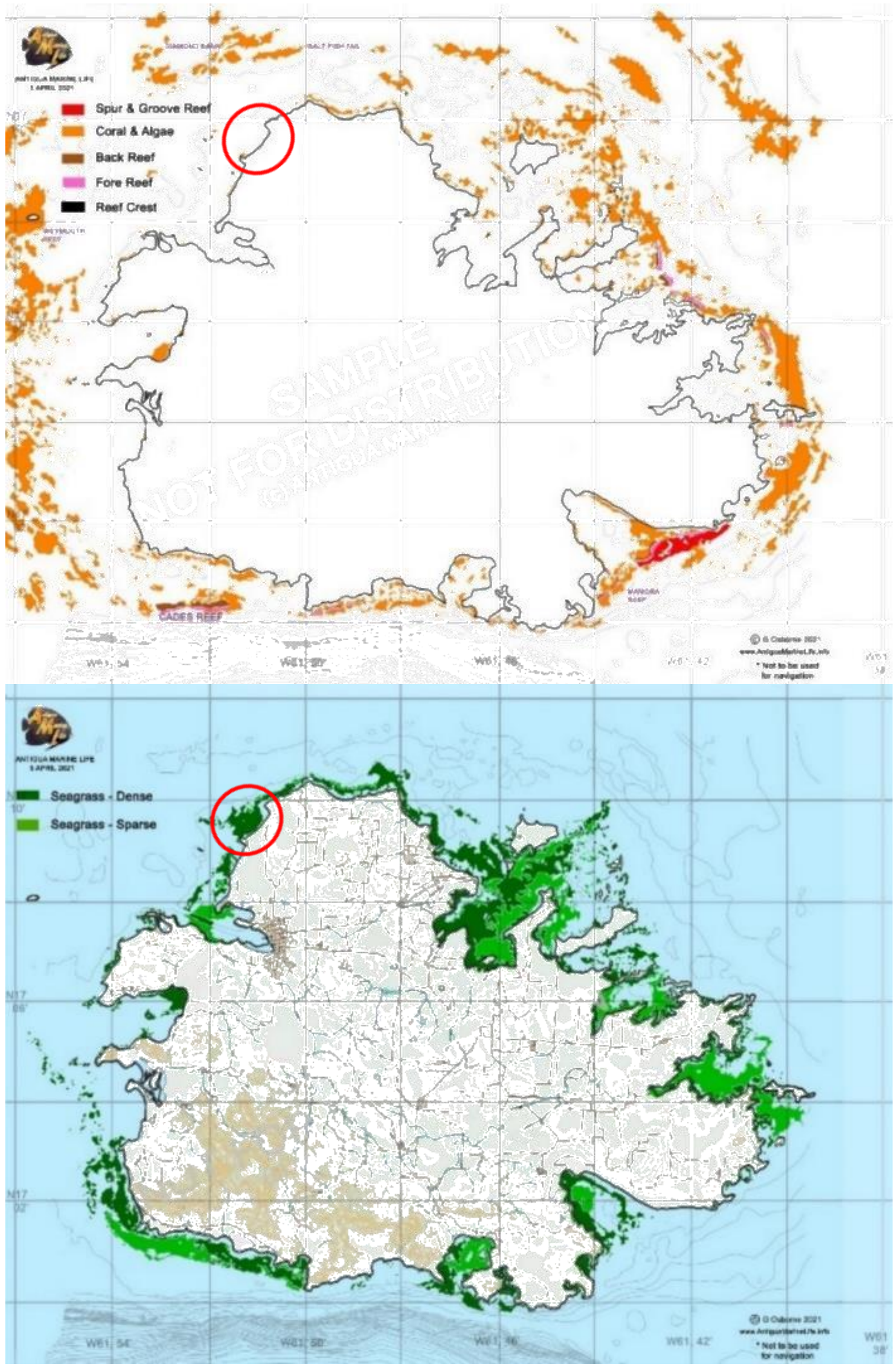



*Figure 2.6 Benthic substrate maps of Antigua’s coral and reef habitats (top) and seagrass habitats (bottom). In both maps the approximate site location is circled in red*

### 2.3.2 Satellite Imagery and Ground-Truthing

The mapping of seagrass beds by the Antigua Marine Life project was confirmed by Google Earth© satellite imagery captured as recently as February 2022 (Figure 2.7 - right). It is worth noting that the first 80-90m from the shoreline appears to be only sand; the seagrass appears to start roughly 85m from the waterline.

Interestingly, the drone image captured just three months later (30 April 2022) by our surveyor does not show any seagrass for the first 120m of the nearshore. It is not known if the seagrass closer to shore died naturally, was buried due to cross-shore sediment transport, or was physically removed. It is further unknown if the differences in the distances are caused by satellite image interpretation or by quality of the image – turbidity in the water during the drone image shot could translate to the sandier appearance of that image. Regardless of the limitations, the analysis points to no benthos (only sand) in the nearshore (the first 90-120m) but dense seagrass beds just beyond that. This observation was further corroborated by the field technicians and surveyors who conducted the bathymetric survey.

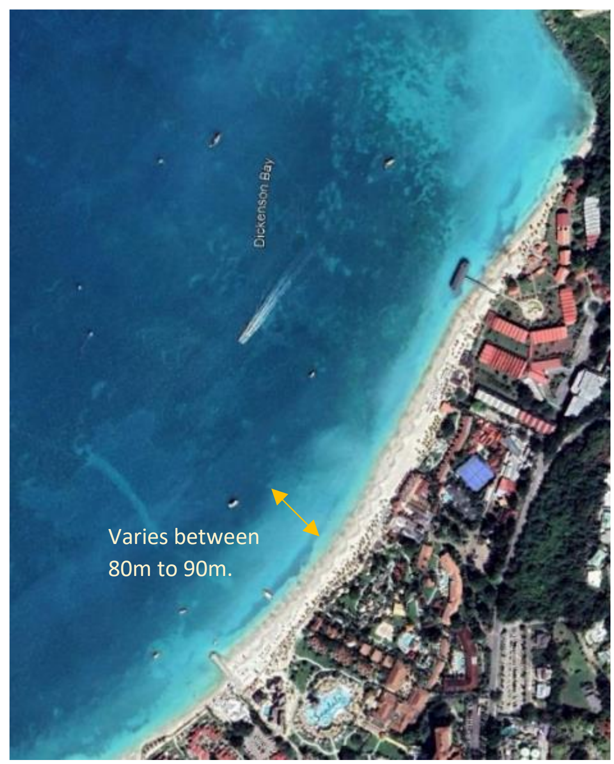




*Figure 2.7 Aerial Images of the project site. Left: Drone imagery of Dickenson Bay captured April 30, 2022. Right: Satellite image of Dickenson Bay from Google Earth© captured in February 2022*

# 3 Wave Climate Analysis

Once the data collection was complete, wave conditions were determined. This section describes existing coastal processes at the project site, including the prevailing operational wave climate and the extreme (hurricane) wave climate.

Antigua is subject to two distinct wave climates: (1) the operational wave climate, defined by day-to-day waves from the north-east Trade Winds and seasonal (winter) swell waves, and (2) the extreme wave climate, defined by storms that generate substantially higher waves.

The operational wave climate describes the day-to-day distribution of wave heights, periods, and directions for a specified location. These wave conditions contribute to sediment movement along the coastline and are responsible for long-term morphological changes. For coastal engineering design, and particularly for this project, the operational wave conditions are typically used to determine the most appropriate design solution in terms of types and layout of the structures.

The extreme wave climate describes waves associated with tropical storms and hurricanes, to which the Caribbean region is vulnerable each year from June to November. Dramatic and abrupt changes to the coastline can occur as a result of these storms. In general, coastal protection structures are designed to withstand wave attack from these extreme storm events; for example, the determination of design wave forces that may occur as a result of extreme waves. The severity of the design storm event (i.e., return period) is chosen in view of the acceptable level of risk of damage or failure that the owner is willing to assume. Normally a 50-year return period represents an acceptable balance between capital investment and maintenance costs.

To better understand the transformation of the wave conditions to the shoreline, numerical models were used. The design wave conditions (operational and extreme) were determined offshore in deep water and then transformed to the nearshore using the MIKE suite of computer models. This model, which was created by the Danish Hydraulic Institute (DHI) couples hydrodynamics (MIKE 21 or MIKE 3 HD), waves (MIKE 21 SW), particle transport (MIKE 21 TR) and sediment transport (MIKE 21 ST). See Appendix A for a detailed description of the models.

The first step in the modelling process was to create a computational mesh (described in Appendix B) where waves and currents are determined at each simulation time step. The second step is to execute the coastal process modelling to develop a clear understanding of the baseline coastal conditions within Dickenson Bay, but particularly at the Sandals Grande shoreline.

## *3.1 Operational Wave Climate*

The operational wave climate at the project site is characterized by day-to-day, relatively calm conditions and by seasonal winter swells (December to May). The day-to-day conditions are created by the north-east Trade Winds. The swells, however, are generated by North Atlantic cold fronts and these waves approach from the north to north-west. As such, the north-west coast of Antigua, where the project is located, can be exposed to these longer period and more aggressive wave conditions on an annual basis. It is these conditions that have the more profound impact on the shoreline of the project site even though, as a percentage of the year, their occurrence is relatively rare.

The data used to assess the operational wave climate of the site in deep water was procured from the ERA 5 global reanalysis model. The European Centre for Medium-range Weather Forecast produced the ERA5 reanalysis which, once completed, will embody a detailed record of the global atmosphere, land surface and ocean waves from 1950 onwards. Currently, data from 1979 to 2020 is available for use.

From the available datasets, a wave node east of Antigua (Node 55) was chosen; its location relative to the island is shown in Figure 3.1 along with a wave rose plot of the offshore waves. As shown, offshore waves 1-3m high approach the island from the east to northeast, according to the selected deep water wave node (Node 55).

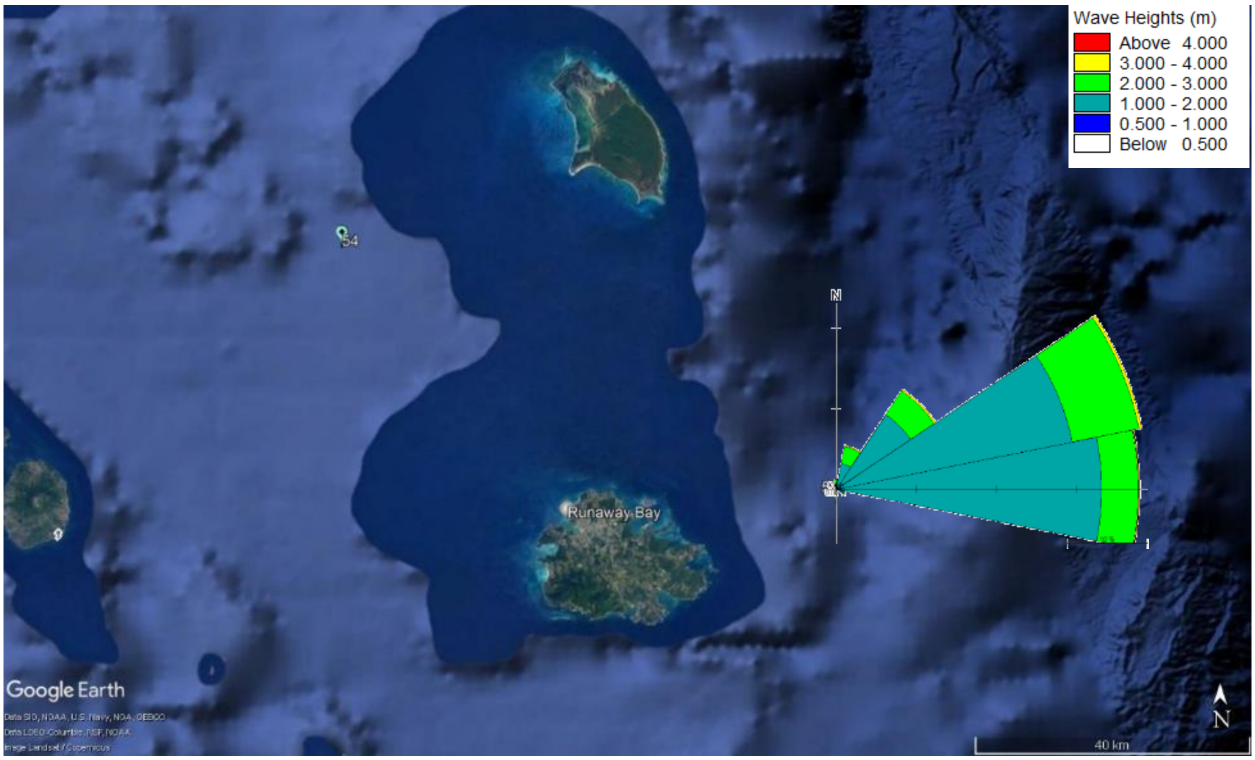



*Figure 3.1 Wave rose plot showing the wave heights and directions the waves come from at ERA 5 Node 55*

The wave data obtained from Node 55 were categorized using a tri-variate frequency analysis of wave height, period, and direction, also known as “binning”. This frequency analysis resulted in 1,769 different conditions or “events” representing a combination of wave height, peak period, and direction, each with a specific duration related to the number of occurrences in the 41-year period of the record analysed (1979-2020).

Data from the ERA 5 global reanalysis model is usually applied on spatial scales (grid increments) larger than 1-10 km and outside of the surf zone. As a result, the data is not at a sufficiently detailed scale to provide accurate nearshore wave data along the Dickenson Bay coastline. Therefore, the project area’s nearshore wave climate was developed using MIKE 21 SW spectral wave model (described in Appendix A) to transform the deep-water waves as they approach from the east, northeast and north and travel over the offshore bathymetry, wrapping around the island to reach the project site. The mesh that was developed for this modelling is described in Appendix A-1 – MIKE 21 Numerical Model Domain.

The wave climate can be represented through a wave rose plot (Figure 3.2), which displays the annual wave heights and their directions of approach at various points along the Dickenson Bay beach. The plots reveal the same general trends as previously observed – direction of approach and smaller wave heights (less pink) at the northern end due to the sheltering effect of the headland.


*Figure 3.2 Wave rose plots of annual wave climate at the project site*

A spatial representation of the nearshore wave climate is shown in Figure 3.3. The plots show the results of the annual wave climate transformation to the nearshore. This includes the mean annual wave climate (50th percentile; the mean wave conditions per year) as shown in Figure 3.3 (top) and the 99th percentile wave conditions (conditions only exceeded 1% of the time i.e. ~4 days per year) conditions are shown in Figure 3.3 (bottom).

The nearshore yearly wave climate results indicate that most of the time offshore waves approach the project site from the north-northeast, where they refract around the headland at the north. Under typical conditions (50th percentile), the bay's wave energy is quite small, with wave heights averaging between 0.1m and 0.3m; to the very north in the shelter of the headland, smaller waves are observed.

The 99th percentile wave condition, which represents swell wave conditions, shows significantly more wave energy entering Dickenson Bay. These waves enter the bay from the same direction (winter swell waves approach the Caribbean from the northeast to the north), as shown by the ERA 5 offshore database. They also behave similarly – wrapping around the headland to the north, thus creating a small shelter area there and resulting in slightly smaller waves. Besides this small area at the extreme north, the incident wave heights are uniform along most of the shoreline, like the 50th percentile conditions. The only difference is the value of the incident wave heights, where at the shoreline wave heights are 0.3m to 0.65m under swell wave conditions.

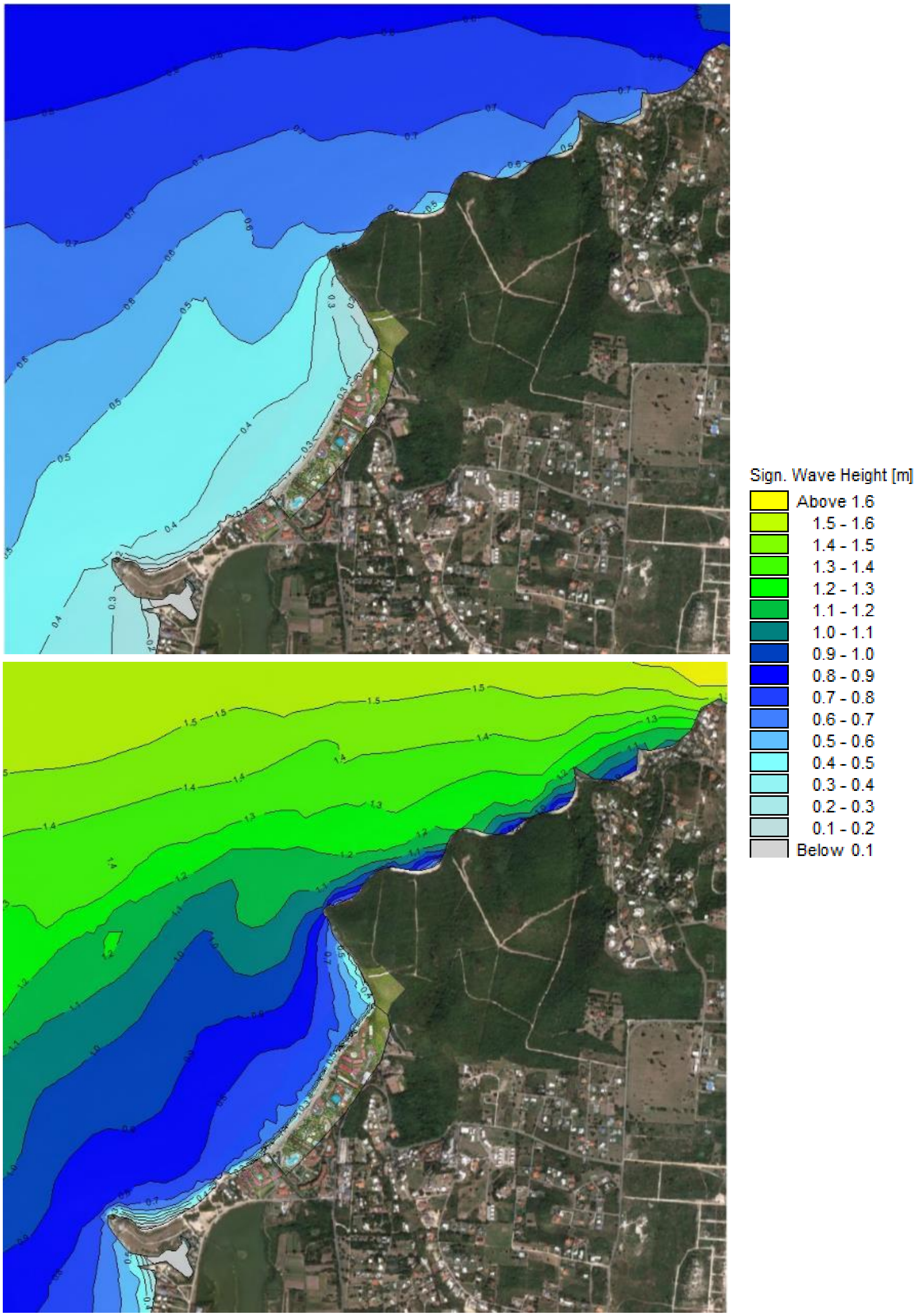



*Figure 3.3 50th percentile (mean annual) wave climate at the project shoreline (top) and 99th percentile (~4 days a year) wave condition at the project shoreline (bottom)*

## 3.2 Hurricane Wave Climate

The Caribbean region is vulnerable to tropical storms and hurricanes each year from June to November. The storms generate high-energy waves, impacting shorelines in dramatic and abrupt ways. In general, coastal protection structures are designed to withstand wave attack from these extreme storm events.

Antigua & Barbuda lies directly in 'Hurricane Alley', an area of water in the Atlantic Ocean within which hurricanes typically form because of the warmer sea surface temperatures there. Figure 3.4 shows the typical path of hurricanes in the north Atlantic basin, which tend to form between latitudes 5°N and 25°N off the west coast of Africa and then track across the Atlantic Ocean. Those formed at the lower latitudes are usually pushed on a westerly track by the north-east trade winds, whereas those of the higher latitudes track more to the north and north-west.

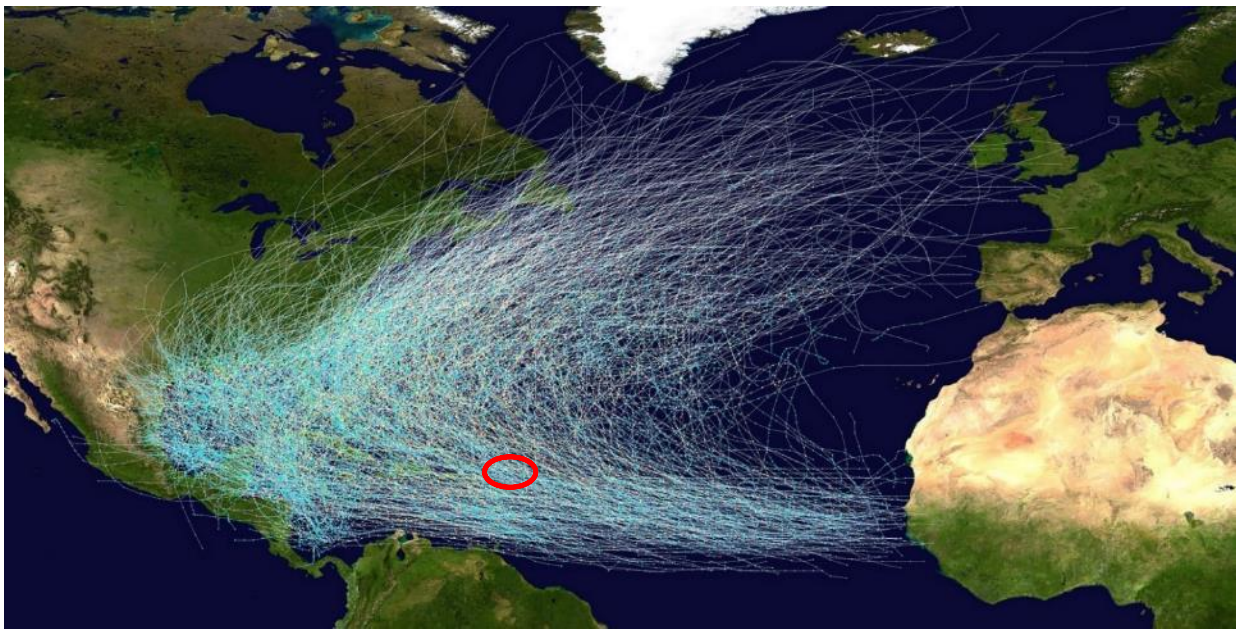


*Figure 3.4 Atlantic hurricane tracks since 1851, the sweeping shape of which is commonly called 'Hurricane Alley'. The approximate location of Antigua is highlighted by a red circle*

### 3.2.1 Historical Hurricane Activity

Extreme waves occur infrequently, and decades or centuries of data must be explored to adequately describe the statistics. For the Atlantic Ocean, detailed information on tropical cyclones, including all hurricanes, has been collected by the US National Oceanic and Atmospheric Administration (NOAA), specifically at the National Hurricane Centre (NHC). This database of historical hurricane information, dating from 1851 to 2019, contains storm tracks, wind speeds and several other parameters to accurately describe and simulate individual storms.

All hurricanes passing within a 300 km radius of the Sandals Grande site were extracted from the database and analysed using HurWave (an in-house computer program). The results show that since the year 1851 (over the past 169 years), 177 hurricanes and tropical storms passed within this distance. The total number of storms can be broken down according to the categories described by the Saffir Simpson scale. Figure 3.5 shows that the study area was more frequently hit by tropical storms (108) and was affected by major hurricanes (Category 3 and higher) less frequently (29).

Figure 3.6 shows the temporal distribution of storms. The graph shows that several years pass without a hurricane, but it also indicates that on many occasions more than one storm can impact the project site in any given year.

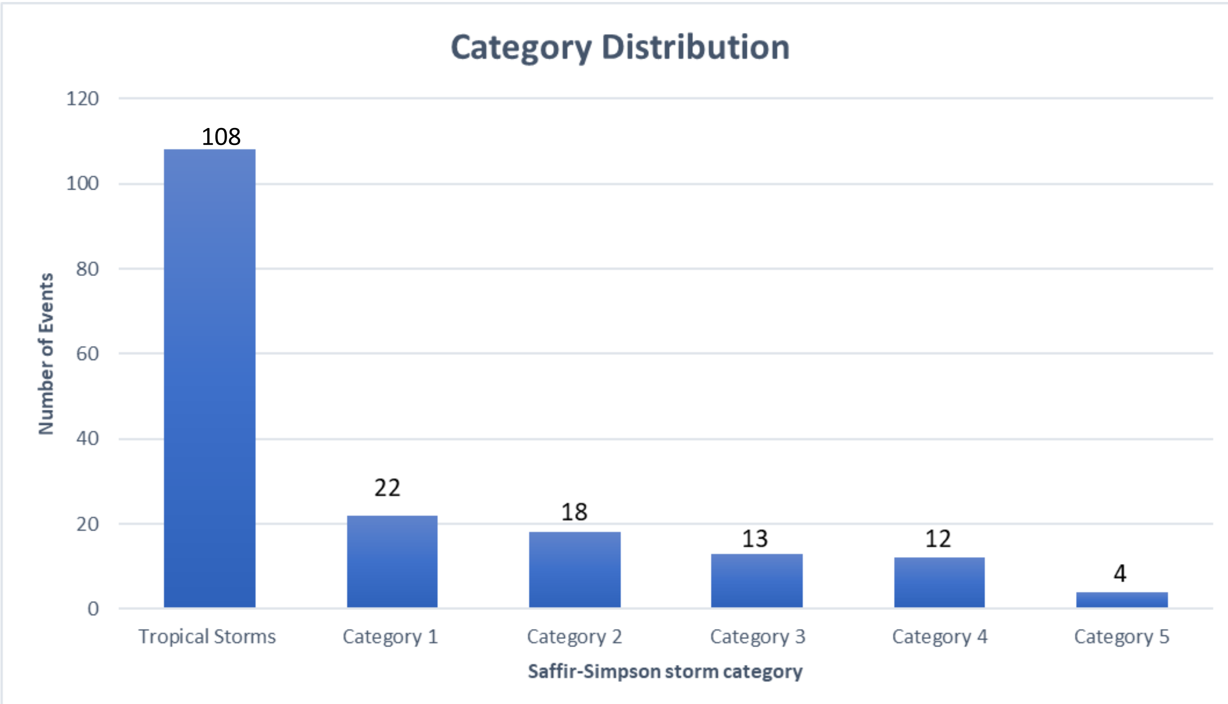



*Figure 3.5 Distribution of storm events according to the Saffir Simpson Scale over the past 169 Years*

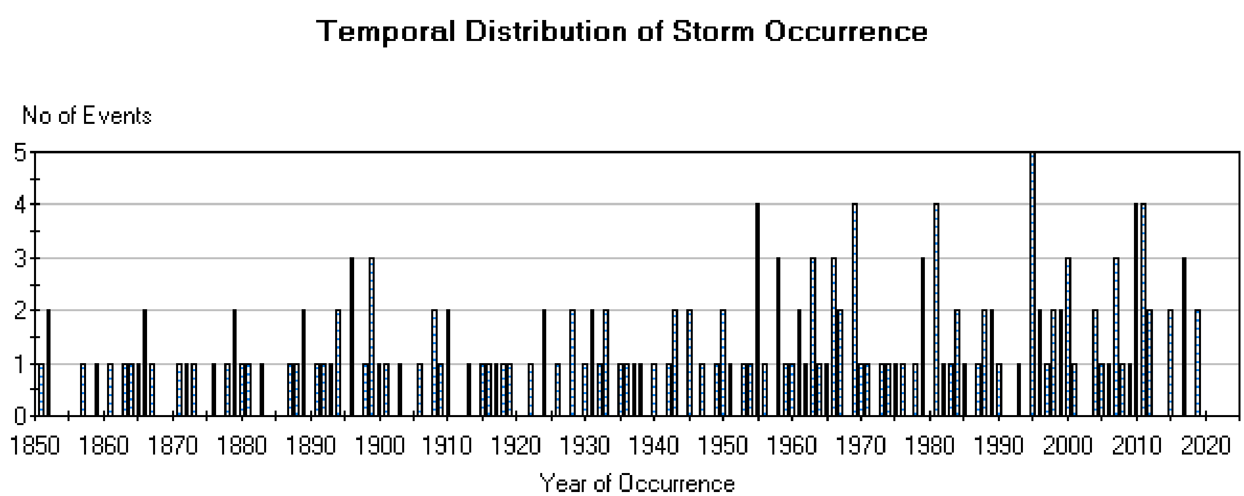



*Figure 3.6 Temporal distribution of storms passing near the project site (300 km radius) from 1851*

### 3.2.2 Hindcasting Hurricane Waves and Surge Levels

The initial water level estimated by hurricane wave climate modelling was determined using a combination of tide data, the storm event's expected inverse barometric pressure rise (IBR), and sea level rise. The highest astronomical tide (HAT) was used as the tide data, which represents the highest tide level. The HAT was obtained from the ADMIRALITY TotalTide (ATT) program and is 0.2m above MSL[2].

The HurWave program (described in Appendix B-1 - HurWave Model Description) was used to calculate the IBR, which was estimated to be 0.34m for a 50-year storm event.

The Intergovernmental Panel on Climate Change (IPCC) publishes guidelines for estimating the impacts of climate change on a regular basis. The IPCC 2019 release was used in this assessment. The global sea level rise (GSLR) used in the hurricane assessments below was calculated using a 50-year project life cycle and the rates specified in the IPCC 2019 guidelines for an RCP8.5 scenario. The GSLR was estimated to be 0.75m coming out of these IPCC 2019 guidelines.

When all the values are added together, the initial water level is 1.29m. Table 3-1 displays the deep-water wave results for a hurricane event with a 50-year return period for Antigua and Barbuda.

*Table 3-1 Wave parameters (significant wave height and peak period) and wind conditions used for the 50-year return period simulations*

| Direction | Direction (deg) | Windspeed (m/s) | Wave height (m) | Wave period (s) | | |
|---|---|---|---|---|---|---|
| **North** | 0 | 36.82 | 7.23 | 11.47 | | |
| **Northeast** | 45 | 38.21 | 9.86 | 13.95 | **Directional Spreading Factor** | 8.00 |
| **East** | 90 | 37.50 | 9.98 | 14.06 | **IBR (m)** | 0.34 |
| **Southeast** | 135 | 36.14 | 9.59 | 13.71 | **Highest Astronomical Tide (m)** | 0.20 |
| **South** | 180 | 34.21 | 8.61 | 12.81 | **Sea level rise (m)** | 0.75 |
| **Southwest** | 225 | 32.30 | 7.60 | 11.85 | **Static Storm Surge (m)** | 1.29 |
| **West** | 270 | 29.75 | 7.30 | 11.54 | **[IBR+HAT+SLR]** | |
| **Northwest** | 315 | 30.04 | 7.19 | 11.43 | | |

### 3.2.3 Nearshore Wave Transformation of Hurricane Waves

MIKE 21 (SW and HD modules) was used to simulate baseline conditions along the beach at Dickenson Bay. The model was used in a coupled mode to simulate the mutual interaction of waves and currents. The coupling of hydrodynamics and waves is an important aspect of storm surge computations, particularly in the Caribbean where wave set-up is a significant component of the total storm surge. As large waves approach shallow water (or a reef) and break, the water level increases, causing localized currents. These currents and changing water levels affect the waves by allowing them to travel further inland. The coupling of waves and currents in MIKE 21 allows these factors to be accurately simulated. For the shoreline at Sandals Grande, six different directional sector runs were statistically analyzed – east, northeast, north, northwest, west and southwest. The results of the hurricane simulations at the points from which data was extracted are shown in Appendix B-2 – Hurricane Modelling Results.

[2] Tidal variations in Antigua are known to be quite small compared to other locations in the Caribbean Sea.

Of all the directional sectors, northwest was found to create the highest values and therefore chosen as the design storm. Figure 3.7 shows wave heights (top) the water level (bottom) (i.e., storm surge) of the northwest 50-year design storm as simulated by the numerical model.

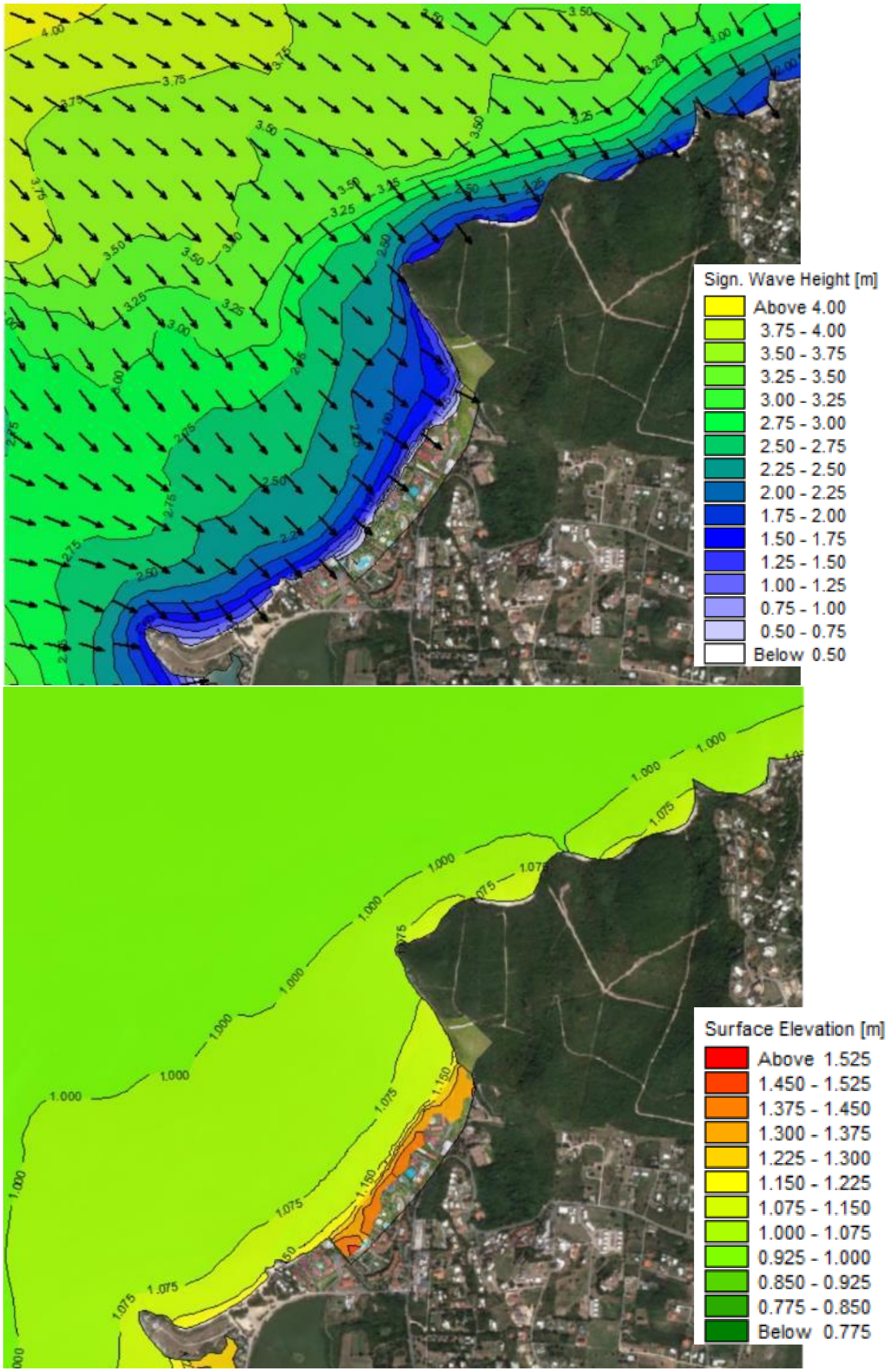



*Figure 3.7 Design storm (Northwest 50-year) plots of maximum wave height (top) and maximum water level (bottom) at project site*

As shown in Table 3-2 below, the results at the five extraction points (Appendix A-1 – MIKE 21 Numerical Model Domain Figure A.0.2) show wave heights from 1.69m to 2.07m are along the shoreline.

The results also indicate static storm surge levels of approximately +1.1m in the nearshore to +1.4m at the Dickenson Bay shoreline.

*Table 3-2 Resulting wave heights, wave periods, wave direction and surface elevation of hurricane simulations*

| | | East | North-East | North | North-West | West | South-West |
|---|---|---|---|---|---|---|---|
| Significant Wave Height (m) | Point 1 | 1.41 | 1.57 | 1.69 | 1.88 | 1.80 | 1.60 |
| | Point 2 | 1.46 | 1.60 | 1.67 | 1.85 | 1.80 | 1.63 |
| | Point 3 | 1.52 | 1.72 | 1.90 | 2.08 | 2.02 | 1.79 |
| | Point 4 | 1.36 | 1.48 | 1.56 | 1.69 | 1.65 | 1.53 |
| | Point 5 | 1.08 | 1.19 | 1.56 | 1.68 | 1.69 | 1.59 |
| Mean Wave Period (t02) (seconds) | Point 1 | 12.04 | 11.52 | 7.66 | 10.34 | 10.49 | 9.45 |
| | Point 2 | 12.16 | 11.70 | 7.62 | 10.36 | 10.46 | 9.49 |
| | Point 3 | 12.17 | 11.84 | 7.58 | 10.37 | 10.42 | 9.47 |
| | Point 4 | 12.19 | 11.86 | 7.78 | 10.44 | 10.39 | 9.55 |
| | Point 5 | 12.10 | 11.38 | 7.81 | 10.39 | 10.25 | 9.53 |
| Mean Wave Direction (degrees) | Point 1 | 328.3 | 329.1 | 327.1 | 316.5 | 303.9 | 300.6 |
| | Point 2 | 325.3 | 326.0 | 322.6 | 309.7 | 299.2 | 294.4 |
| | Point 3 | 325.7 | 326.7 | 320.3 | 308.3 | 293.2 | 285.4 |
| | Point 4 | 318.2 | 319.4 | 317.5 | 307.2 | 293.4 | 287.0 |
| | Point 5 | 306.5 | 308.9 | 313.2 | 303.1 | 289.3 | 282.4 |
| Surface Elevation (m) | Point 1 | 0.85 | 0.96 | 1.07 | 1.14 | 1.10 | 0.96 |
| | Point 2 | 0.85 | 0.95 | 1.07 | 1.14 | 1.10 | 0.96 |
| | Point 3 | 0.84 | 0.94 | 1.05 | 1.10 | 1.08 | 0.95 |
| | Point 4 | 0.84 | 0.94 | 1.07 | 1.12 | 1.10 | 0.97 |
| | Point 5 | 0.84 | 0.93 | 1.05 | 1.13 | 1.12 | 0.98 |

# 4 Shoreline Morphology

Waves and hydrodynamics have the biggest influence on sediment morphology. Having developed an understanding of the wave climate at the project sites, the next step is understanding if, how and why the sediment is moving in the bay.

## *4.1 Historical Shoreline Comparison*

Satellite images of the Dickenson Bay shoreline from Google Earth were extracted and geo-referenced into the project database. The geo-referenced satellite images were input to ArcGIS where they were overlaid on each other. For each available image of the area, the shoreline (which in this instance would refer to the high-water mark (HWM) or the 'wetted' area on the image) was traced over the base map and the location of the shoreline through the years was observed and compared.

There are some limitations to this methodology, uncertainties that mostly centre on the nature of the shoreline position at the time a satellite image is captured. Possible errors that could limit the analysis are outlined in the text box below.

- Seasonal error - Many beaches have seasonal cycles of erosion and accretion. Because high resolution satellite images are limited for the Caribbean islands, images cannot be selected on seasonal time frames.
- Tidal fluctuation error - The satellite images were obtained without regard to tidal cycles, which can result in inaccuracies on the digitized shoreline.
- Digitizing error - The error associated with digitizing the shoreline.
- Pixel error - The pixel size in orthorectified images is 0.5 m, which means anything within 0.5 m cannot be resolved.
- Rectification error – Satellite images are corrected, or rectified, to reduce displacements caused by lens distortions, earth curvature, refraction, camera tilt, and terrain relief using remote sensing software.

Even when considering the range of possible errors, the comparison of the variations between images is still regarded as helpful in quantifying the coastal changes (in a general sense), and an analysis was therefore still conducted.

Seven (7) satellite images of the shoreline were available for comparison. These images were captured between 2011 and 2021 from the Google Earth platform. In addition to simple visual observations of the shoreline locations, analysis regarding the extent of the change was made at two points along the shoreline where a distance change was first calculated and then the average rate of change (m/yr.) was calculated [distance change divided by time]. The results (Table 4-1) are colour coded: accretion along a profile line is shown in green and erosion is shown in red. The corresponding graph is shown in Figure 4.1 and the shoreline change map is shown in Figure 4.2. The table is perhaps the most instructive as clear links can be made between the dates of image capture and the calculated change.

The findings indicate that over the last 10 years there has been only slight shoreline variation along the Sandals Grande beach, although the overall trend is still one of erosion.

*Table 4-1 Distance between image high-water marks and calculated average rate of shoreline change over the period between images. The Table also shows Total and Average rates of change at each profile over the periods of analysis*

| Date of Image Capture | Time Change (years) | PROFILE 1 Distance (m) | PROFILE 1 Erosion Rate (m/yr) | PROFILE 2 Distance (m) | PROFILE 2 Erosion Rate (m/yr) |
|---|---|---|---|---|---|
| **3/11/2011** | | 95.89 | | 128.30 | |
| **4/9/2014** | 3.08 | 90.44 | -1.77 | 121.99 | -2.05 |
| **2/11/2017** | 2.84 | 98.63 | 2.88 | 126.45 | 1.57 |
| **8/8/2018** | 1.49 | 87.43 | -7.53 | 114.15 | -8.27 |
| **10/6/2019** | 1.16 | 91.33 | 3.36 | 120.72 | 5.66 |
| **8/6/2020** | 0.84 | 94.70 | 4.03 | 122.90 | 2.60 |
| **1/20/2021** | 0.46 | 95.34 | 1.41 | 121.98 | -2.01 |
| | | | | | |
| **Totals** | **9.86** | -0.55 | | -6.32 | |
| **Total Change over Total Time (m/yr)** | | | **-0.06** | | **-0.64** |
| **Average Rate of change (m/yr)** | | | **0.40** | | **-0.42** |

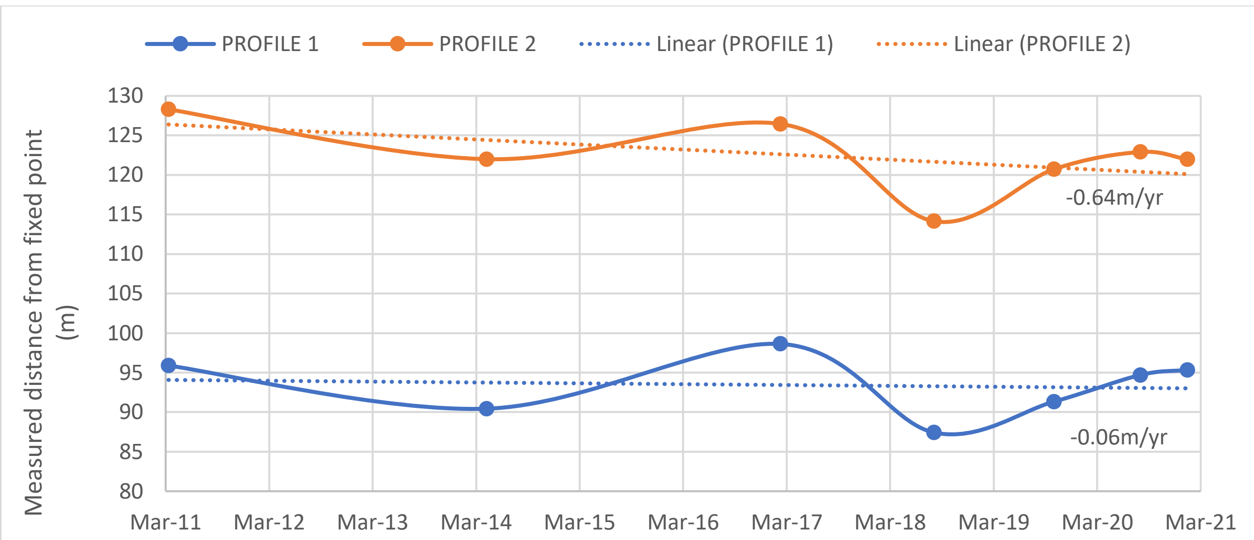



*Figure 4.1 Variation of shoreline width over the years of image capture for both the northern and southern profiles*

The more southern section of the beach has been relatively stable, only averaging -0.06 m/yr. of shoreline change. The northern end of the beach has undergone just slightly more erosion having lost over 6m of shoreline over the last 10 years, averaging -0.64m of shoreline loss per year.

For all sections of the beach the trendlines still point downwards indicating diminishing beach width.


*Figure 4.2 Historical Shoreline Change Map*

## *4.2 Patterns of Sediment Movement*

There are various ways in which sediment can move within an area; the two main movement patterns are described below.

### 4.2.1 Longshore Sediment Movement

Longshore sediment transport refers to the cumulative movement beach and nearshore sand parallel to the shore through the combined action of tides, wind and waves and the shore-parallel currents produced by them.

#### *LITDRIFT Analysis of Dickenson Bay*

We used a numerical model, LITPACK, developed by the Danish Hydraulic Institute, for this part of the modeling. The LITDRIFT module was used to determine sediment transport characteristics (see Appendix A – MIKE 21 (by DHI). The model calculates the rate and distribution of alongshore sediment movement at the project shoreline.

A profile in the centre of the beach stretching from the back of the beach (land) to a depth of about 3m below mean sea level was created (Figure 4.3). Throughout the simulation, the profiles are assumed to be constant, implying that the beach undergoes no morphological change or responds to the input wave conditions. The model simply shows how much sand moves and its distribution across the surf zone. Despite this limitation, LITDRIFT provides useful information about the coastal processes occurring at the site. The model's inputs include:

- The annual wave climate parameters at the project site (wave roses shown in Figure 3.2);
- Sediment distribution along the beaches, which incorporate the mean grain size from sediment sampling and sieve analysis. A $D_{50}$ of 0.13mm was used which corresponded to SS3 (Figure 2.5); and
- Computed nearshore wave parameters at the seaward end of the profile.

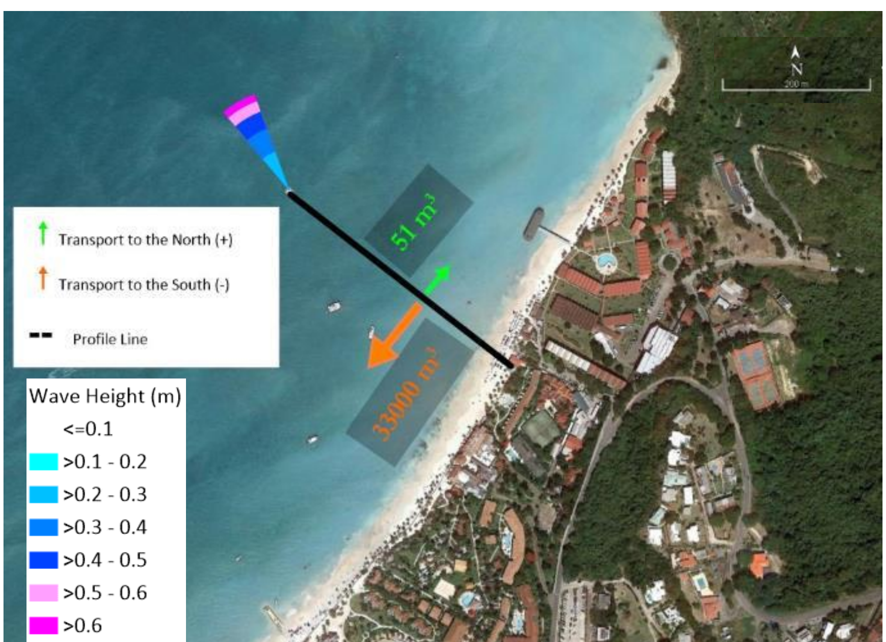



*Figure 4.3 LitDrift results showing dominant transport direction*

For the profile, LITPACK computed a relative indication of sediment quantities that would be transported to the north (+) and to the south (-). Figure 4.3 shows the computed annual alongshore sediment transport volumes and directions, which show a net sediment transport to the south-southwest. The transport value (33,000$m^3$) is quite high, likely because of the fine-grain size used as input.

Sediment transport distribution is also shown in Figure 4.4. The results indicate the following littoral zone characteristics:

- Overwhelmingly, the general movement of sediment is towards the south-southwest.
- The active zone of sediment transport extends typically from the shoreline out to 30m offshore, at depths of 0m to -1.8m relative to MSL (Figure 4.4, left);
- Small (negligible) amounts of sand move toward the north at certain times of the year (Figure 4.4, 3rd from the top on the right)

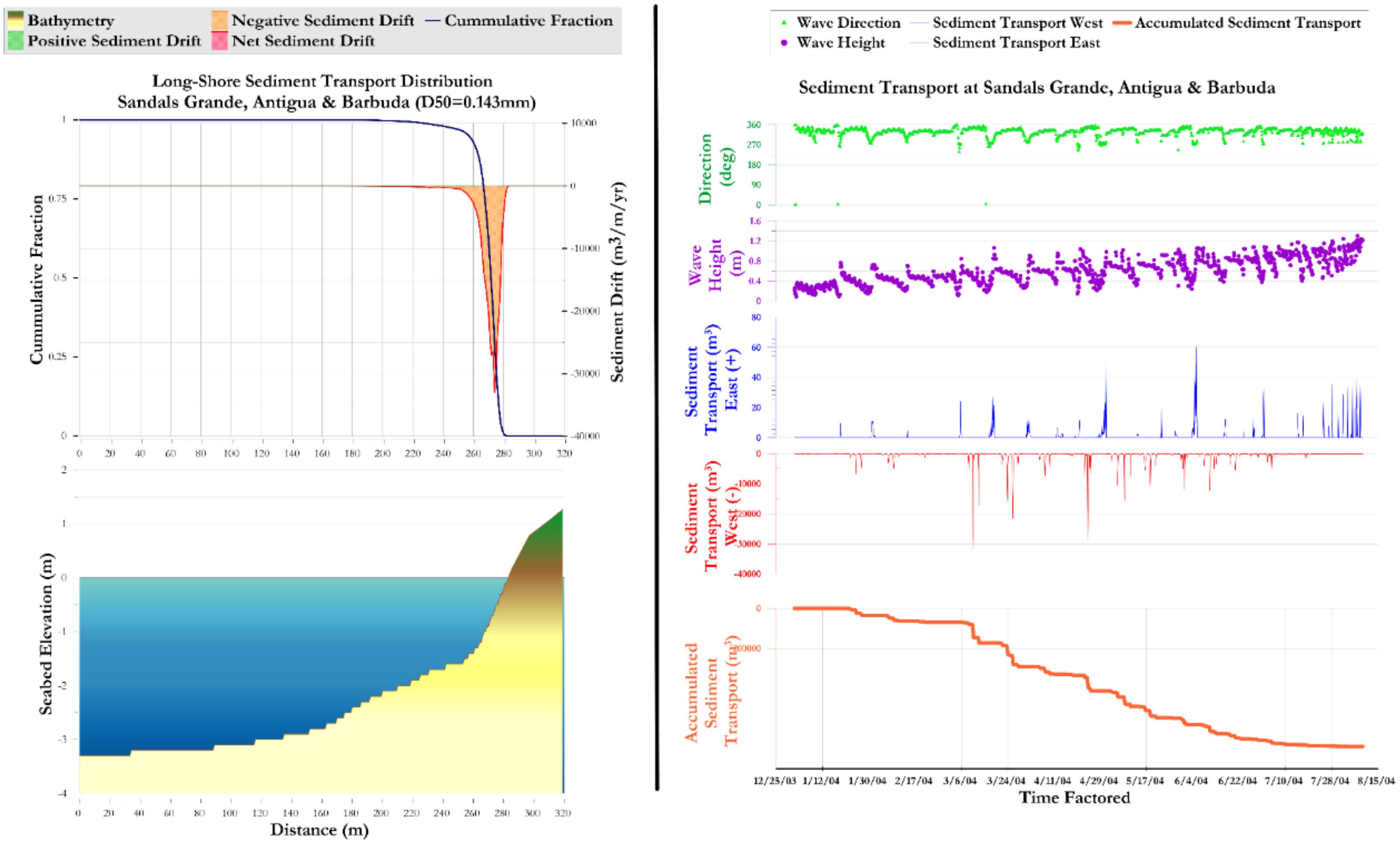



*Figure 4.4 Resulting plots from the LitDrfit model indicating sediment transport distribution at the Sandals Grande, Antigua*

### 4.2.2 Cross-shore sediment transport

Cross-shore sediment transport is the displacement of sediment perpendicular to the shore (onshore or offshore), usually into a berm (onshore) or into an offshore bar (offshore).

Whether through longshore or cross-shore mechanisms, erosion of the project shoreline is a cause for concern. To better understand any erosion issues at the shoreline, a distinction must be made between incidental coastal erosion and ongoing coastal erosion[3].

- *Incidental coastal erosion* (also called temporal coastal erosion) takes place mainly by cross-shore processes during extreme events (high water levels, high waves), which produce beach lowering or scouring. For stable coasts, incidental erosion is a reversible process; under average conditions the coastal profile is restored.
- *Ongoing coastal erosion* (also called long-term coastal erosion) is mainly due to a basic imbalance in the supply and export of material from a certain coastal section. Erosion takes place on the shoreface and on the beach if the export is greater than the supply of material. The deficit can be due to both cross-shore processes and longshore processes.

Incidental coastal erosion is likely a significant driver behind the erosion at the project site since it is exposed to hurricanes, tropical storms, and seasonal swell events. Incidental erosion can be a reversible process that describes the variations along the shoreline here, which tends to "come and go". Examination of the historical shoreline changes (shown in Table 4-1 and Figure 4.1) highlights this.

Some of the observed shoreline changes can be easily explained as being the result of passing storms. For instance, when comparing the satellite imagery captured in February 2017 to that captured in August 2018 there was significant erosion noted at both sections of the beach (roughly -8m / -26'). This erosion is likely linked to the extremely active hurricane season that occurred in September of 2017, which saw the island receiving the winds and waves from three major hurricanes: Hurricane Irma (Category 5), Hurricane Jose (Cat 4) and Hurricane Maria (Cat 5) one after another (Figure 4.5).

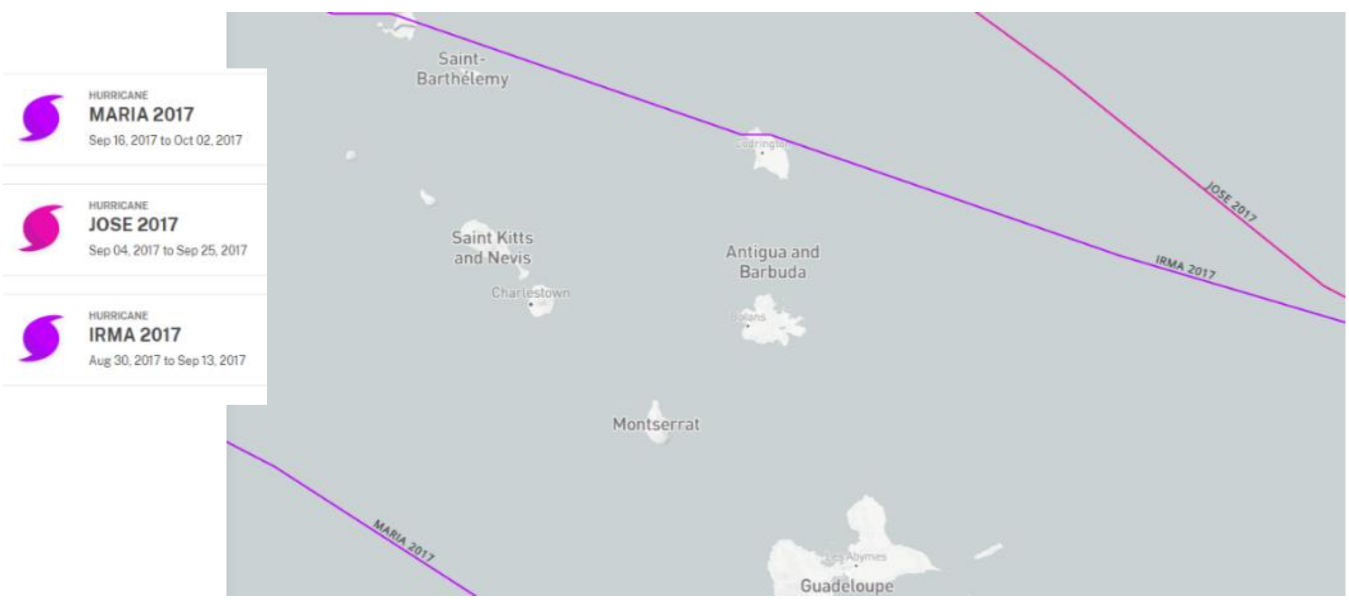



*Figure 4.5 Storms tracking past Antigua in September of 2017*

[3] Coastal Wiki: http://www.coastalwiki.org/wiki/Definitions_of_coastal_terms

# 5 Baseline Coastal Conditions Summary

## 5.1 Data Collection Summary

### Bathymetry

The bathymetry at the project site is quite uniform and consistent across the length of Dickenson Bay, except for a slightly shallower area towards the headland to the north. The bathymetric contour plot reveals that the water depths in Dickenson Bay are shallow, ranging from 1m (at 11-14m from the shoreline) to 3m (at roughly 120m from the shoreline), which amounts to a roughly 2% slope.

### Sediment

Sieve analysis indicates that the sand on the beach of Sandals Grande is medium-sized sand whereas the sand in the nearshore is fine (Table 4-1). Fine sand is easily transported and could be the reason why the volumes of transport shown by the LitDrift model are as high as they are.

### Benthos

There is no coral and/or reef habitat in the immediate project area. There are, however, extensive seagrass beds. Once dense and closer to shore (within 80-90m), it now appears as if the seagrass 'edge' is retreating (roughly 40m of retreat). Once the footprint of the overwater suites is finalized and approximate pile locations determined, the area will have to be surveyed in more detail and a seagrass relocation plan developed.

## 5.2 Coastal Processes Summary

### Operational Wave Climate

The nearshore annual wave climate results indicate that most of the time, offshore waves approach the project site from the north-northeast, where they refract around the headland at the north, creating a small, sheltered area with slightly smaller waves. Besides this small area at the extreme north, the incident wave heights are uniform along most of the shoreline. Under typical conditions (50$^{th}$ percentile), the wave energy in the bay is quite low, with wave heights averaging between 0.1m and 0.3m. Under the 99$^{th}$ percentile wave condition, wave heights are 0.3m to 0.65m.

### Hurricane Climate

Worst case scenarios were extracted to determine the input water level for the design storm. These were: highest astronomical tide (0.20m), inverse barometric rise associated with the eye of the storm passing (0.34m), and sea level rise on the order of 15mm/year (rate associated with the IPCC 2019 guidelines for an RCP8.5 scenario) consistently over a 50-year project life cycle.

Hurricane wave approaches from six directional approaches were simulated and the worst was determined to be the northwest. Therefore, the northwest 50-year return period event became the "design storm".

The results show wave heights from 1.69m to 2.07m along the shoreline, and static storm surge levels of approximately +1.1m in the nearshore to +1.4m at the Dickenson Bay shoreline.

### Shoreline Morphology

A comparison of seven dated satellite images of the shoreline was made by overlaying one on the other to determine changes in shoreline position. The analysis of the trendlines (Figure 4.1) pointed to an overall pattern of diminishing beach width, i.e., erosion, albeit at low rates: -0.06m/yr at the south and -0.64m/yr at the north.

LitDrift analysis showed a distinct pattern in annual alongshore sediment transport volumes and directions, which show a net sediment transport to the south-southwest. The analysis also showed that the active zone of sediment transport extends typically from the shoreline out to 30m offshore at depths of 0m to -1.8m relative to MSL.

There is also incidental cross-shore sediment transport occurring at the project shoreline, where significant periods of erosion such as that observed between February 2017 to August 2018 can be clearly linked to several major storms passing by the project site.

# PART 2 – WAVE FORCE AND IMPACT ASSESSMENT

# 6 Forces and Considerations for Proposed Overwater Structures

This section of the report provides a summary of the computations of preliminary wave loading characteristics for the proposed overwater suites, specifically the decking of the structure and its supporting piles.

It should be noted that based on usage and design life, different design boundary conditions were utilized for the sub-structure i.e., the supporting piles and the beams etc., versus the super-structure i.e., the finished floor level (FFL) of the suites. Namely the elevation / FFL was designed for swell wave conditions because it was assumed that evacuation could occur prior to a hurricane. Conversely, the piles will have to withstand hurricane waves, and so those design conditions were used. Considerations for the design deck elevation or finished floor level are made first, as that value is used in the subsequent wave loading analyses.

## *6.1 Proposed Suite Location and Layout*

The proposed design for the 'Overwater Village' consists of a total of sixteen (16) overwater suites connected by a boardwalk supported on piles, featuring a pool in the centre of the heart shape. As shown in Figure 6.1, the suites are organized along a main stem stretching roughly 135m from the sandy beach to the base of the heart shape. Along this stem there are eight (8) suites – four (4) on each side. The heart shape then extends roughly 50m offshore and features another eight (8) suites arranged seaward of a walkway approximately 215m long.

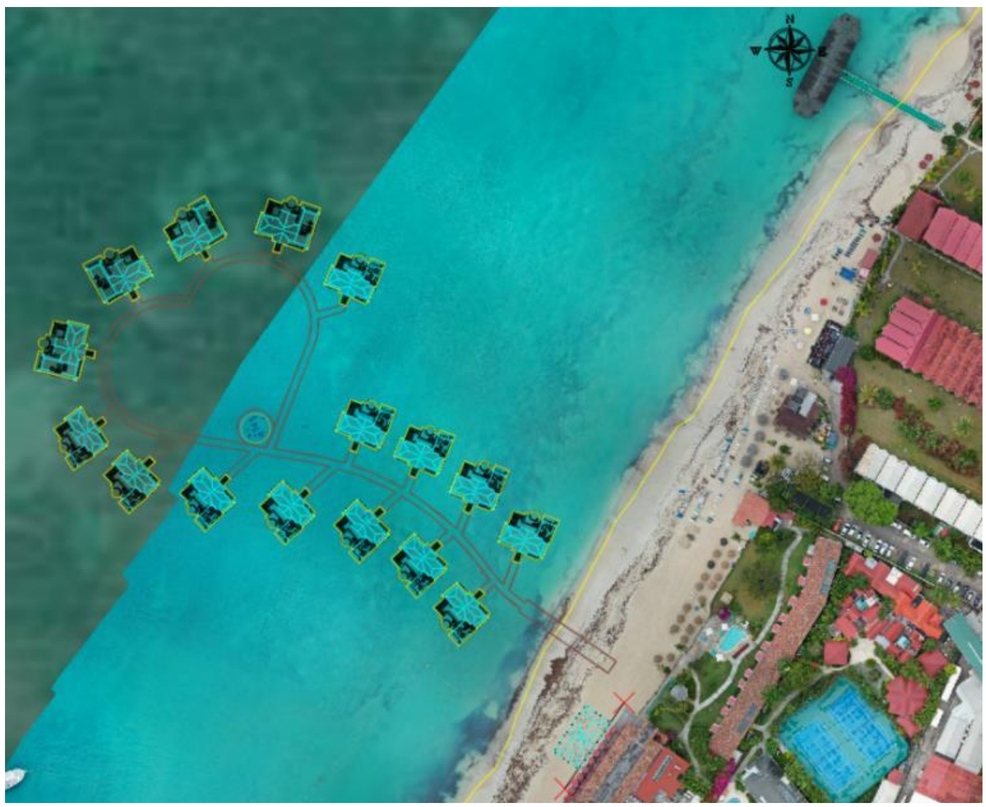
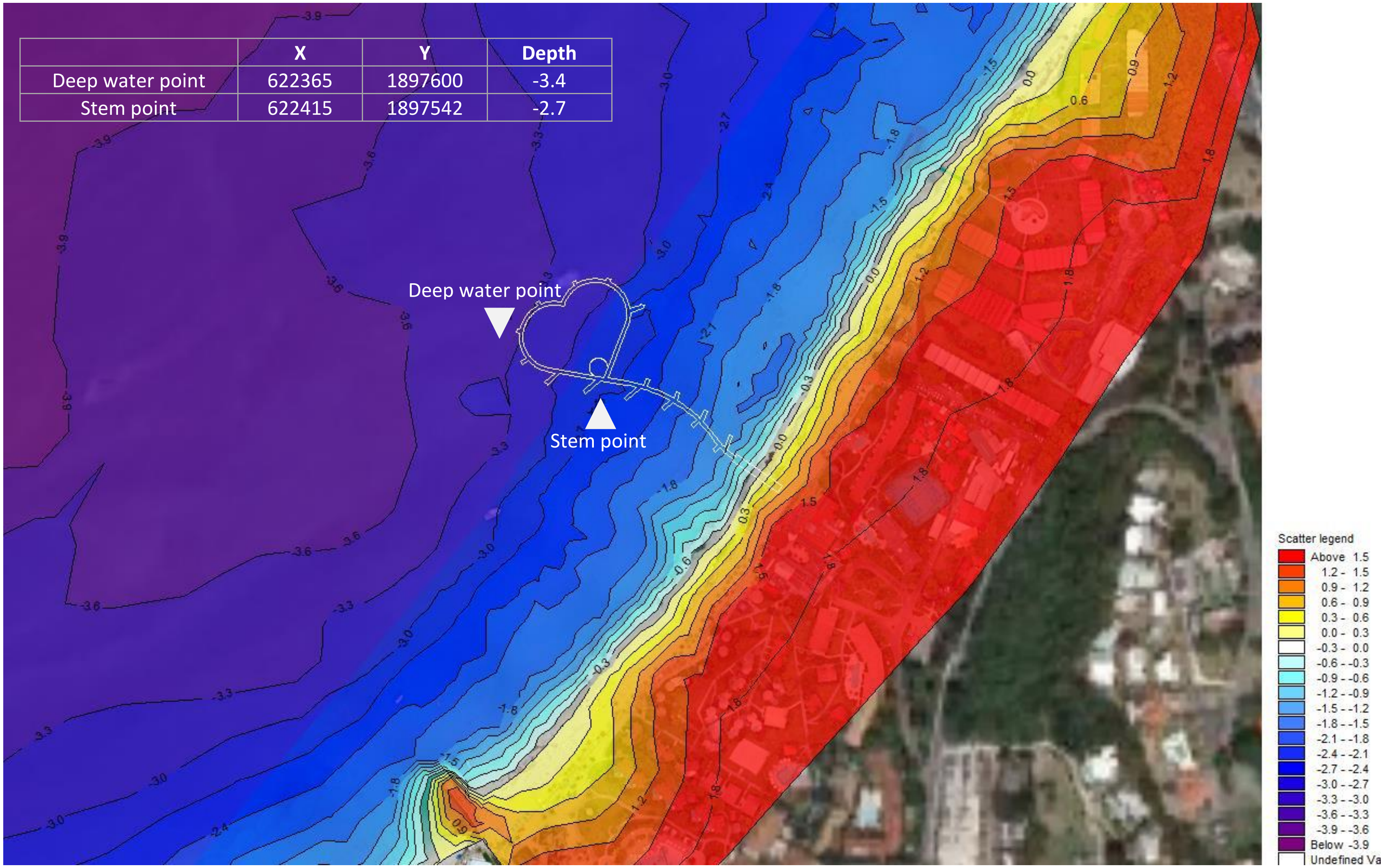


*Figure 6.1 Sketch of proposed overwater suites in proposed location just north of restaurant*

As is to be expected, and as shown in Figure 2.3 and Figure 6.2, the water depths increase moving seaward of the shoreline. Of note, the suites located around the heart are in deeper water (roughly 2.8 - 3.35m or 9' – 11') when compared to the suites along the main stem (water depths of 1.8 - 2.7m or 6' – 8.5'). As wave heights are very dependent on the local depth of water, two representational points were chosen. A deep point to represent the deepest / most seaward suites, and a stem point to represent the deepest of those suites on the walkway (i.e., suite #8). These representational points are shown below in Figure 6.2 along with their GPS coordinates and water depth below MSL.



| | X | Y | Depth |
|---|---|---|---|
| Deep water point | 622365 | 1897600 | -3.4 |
| Stem point | 622415 | 1897542 | -2.7 |



*Figure 6.2 Boardwalk of proposed overwater suites over contour map of area. Representational points highlighted with triangles and coordinate information shown in the table in insert.*

## 6.2 Design Structural Elevation

### 6.2.1 Parameters for Consideration

*Design Water Level*

In the analysis of the proposed deck area, it is important to strike a balance between limiting damage and optimizing functionality during a swell event with creating a 'closeness' to the water. Specifically, the suite floors should be placed at a level where splashing and overtopping will be minimized to allow safe usage by guests, even during winter swell events, but be sufficiently close to the water to create the desired aesthetic effect.

It should be noted that the superstructure of the suites i.e., the decking, has not been designed for a hurricane (unlike the sub-structure). Therefore, the overwater suites could become flooded in a hurricane's storm surge. It is thus recommended that the suites be evacuated if Antigua is placed on a hurricane watch or warning.

*Figure 6.3 Overwater suites showing differing deck levels to create aesthetic effect [Source: Sandals.com]*

*Sea Level Rise*

The optimum floor elevation must also account for potential sea level rise impacts from climate change, so that it is still useable even with sea level rise and similarly, under high tide. As mentioned previously in Section 3.2.2, the sea level rise (SLR) projected to our design horizon of 2070 is 0.75m and the highest astronomical tide (HAT) is 0.2m. Combining these results yields a baseline increased water level of 0.95m.

*Wave Profile*

The design should ensure that the area will still be calm and safe enough for everyday use of the suites i.e., under swell wave conditions. The 99$^{th}$ percentile wave, which would be exceeded only 1% of the year (roughly 3½ days of the year), was selected. This condition yielded a wave height value of 0.716m at the most seaward suite and 0.707m at the stem suite.

It should be noted that the waves that would be incident on the piles and underside of the deck would not be regular linear waves; rather, the waves would be non-linear. It is important to recognize the difference: *A linear wave has a sinusoidal surface profile with small amplitude and steepness, while a nonlinear wave has larger amplitude (finite-amplitude), sharper crests and flatter troughs than the linear wave. Nonlinear waves can be categorized into Stokes, cnoidal, and solitary waves, according to the wave characteristics.*

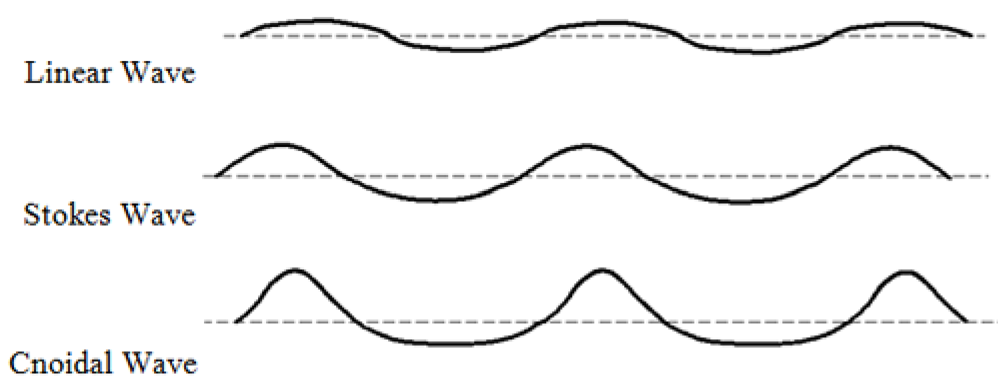



*Figure 6.4 Comparison of profiles of the different progressive wave types.*
*[Source: [https://www.flow3d.com/modeling-capabilities/waves/](https://www.flow3d.com/modeling-capabilities/waves/) ]*

Based on the depth of the water and wave period, it is most likely that the incident waves would be classified as cnoidal waves. A cnoidal wave is a long periodic wave in shallow and transitional water that has sharp crests and flatter troughs than regular linear waves, as shown in Figure 6.4. The asymmetry of the waves means that the crest and trough have unequal amplitudes, with more being carried in the crest as shown. Without site-specific measurement of swell waves, it was assumed that approximately 65% of the wave would be above the water line, with the remaining 35% in the trough. Applying this ratio to the design wave heights of 0.716 and 0.707m, results in crest heights of 0.4654m and 0.4595m respectively. The two values can be averaged and approximated to 0.46m.

### *Freeboard*

In coastal engineering terminology, the freeboard is defined as the height of the crest of a structure above the still water level.[4] In this case, it refers to that space between the water level and the underside of the suites (as shown in Figure 6.5). There are no published guidelines on the appropriate freeboard to use in coastal engineering design. However, hydraulic engineering adopts freeboards ranging from 15-25% depending on the structure being designed. In this case, a freeboard of 15% was adopted. The freeboard would be applied to the sub-total of the design water level and the wave crest.

[4] CIRIA (1996). *Beach management manual*. CIRIA Report 153.

### 6.2.2 Calculation

The values of these main components of the design elevation are summarized in Table 6-1 as well as displayed graphically in Figure 6.5. As shown, based on the summation of the individual components, the total recommended design elevation is +1.6m above MSL.

*Table 6-1 Values (in metres) of components used in determining the design deck elevation*

| Elevation Component | Value (m) [Calculation Steps] |
|---|---|
| Design stillwater elevation [sea level projection (2070) + highest astronomical tide] | 0.95 [0.75 + 0.2] |
| Design Wave Crest Elevation | 0.46 [65% of 99th percentile wave] |
| Freeboard | 0.211 [15% of subtotal 1.412] |
| **Total Design Elevation** | **1.624** |

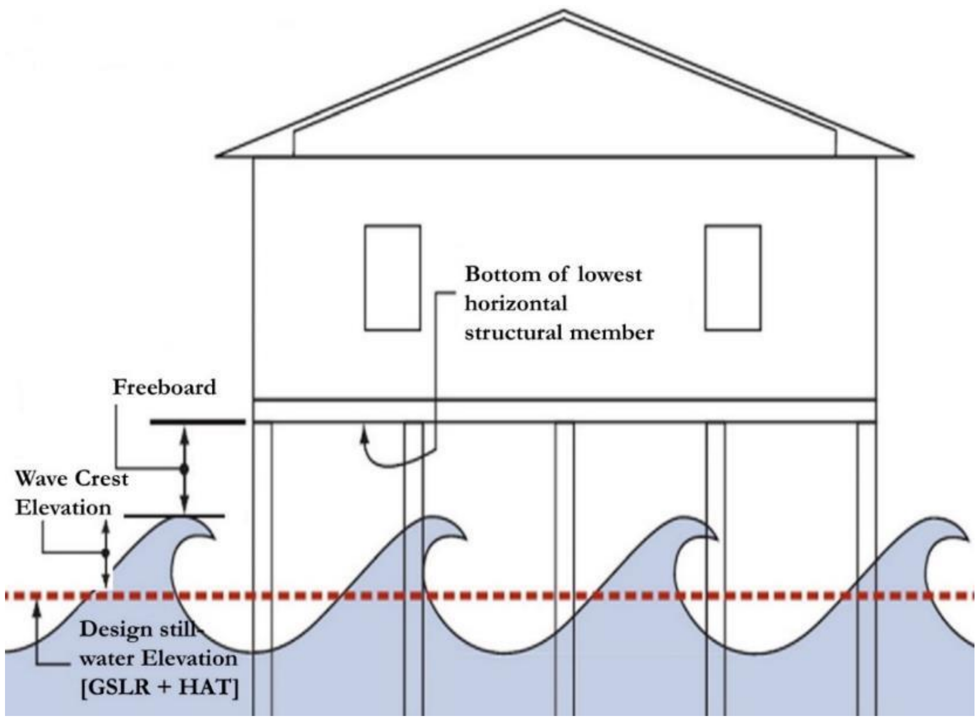



*Figure 6.5 Graphical representation of wave components contributing to elevation considerations*

## 6.3 Overwater Suite Cross-sectional Details

All forces will be acting on the structure face i.e., the portion of the structure that interacts with the wave. It should be noted that no cross-sectional design of the proposed suites at Sandals Grande Antigua were provided as the design has not yet progressed that far. Therefore, to assess the area of impact, an available cross-sectional detail for Sandals Dunns River was used, with the assumption that the Sandals brand of overwater suites would remain relatively similar across locations. This design, shown in Figure 6.6, was assessed and relevant dimensions extracted. It should be noted, however, that should this design cross-section actually differ significantly, or should it change, the forces will have to be recalculated. It should be further noted that the height of the deck above SWL was taken as the recommended 1.6m rather than the level of 1.0m as indicated in the available drawing.

The dimensions extracted from the provided sketch (Figure 6.6)are as follows:

| | | | |
|---|---|---|---|
| deck depth ($b_h$) | 0.35m | No. of piles on rear (first contact) | 3 |
| deck width ($b_w$) | 7.1m | Diameter of pile | 0.61m |
| deck length ($b_l$) | 6.5m | | |
| Height of deck above SWL | 1.6m | | |

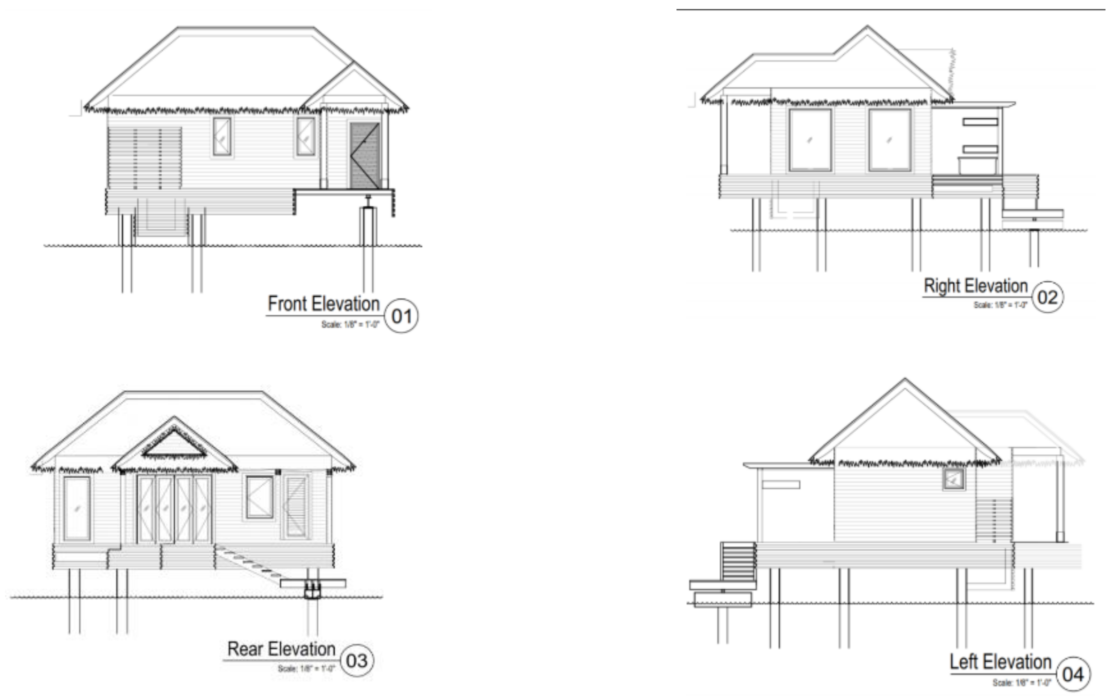



*Figure 6.6 Cross-sectional elevations of overwater suites from which structural dimensions were assumed*

It should also be noted that, as with the previous analysis, the proposed structure was divided into two main sections: Points 1 and 2 being representative of the deepest suite overall and the deepest suite of the stem respectively (Figure 6.2).

## 6.4 Design Wave Forces

### 6.4.1 Design Criteria

The use of a return period or design event such as the 1 in 50-year or 1 in 100-year essentially defines the kind of design conditions that will, on average, occur or be exceeded once every 50 years or every 100 years. It is important to understand risk and consider the chance of occurrence of a particular storm condition during the lifetime of a structure so that the associated risk of damage can be understood. Table 6-2 gives the exposure risk (probability) over a project lifespan for different return period events. For example, a project lifespan of 50 years (Design Life = 50) has a 99% chance of a 1:10-year event occurring and a 39% chance of a 1:100-year storm event occurring in 50 years.

*Table 6-2 Probability of occurrence for various return periods and design life*

| Storm Event Return Period (years) | Design Life (years) | | | |
|---|---|---|---|---|
| | 25 | 50 | 100 | 200 |
| 10 | 93% | 99% | 100% | 100% |
| 25 | 64% | 87% | 98% | 100% |
| 50 | 40% | 64% | 87% | 98% |
| 100 | 22% | 39% | 63% | 87% |
| 200 | 12% | 22% | 39% | 63% |
| 500 | 5% | 10% | 18% | 33% |

As the piles supporting the overwater suites will be exposed to wave breaking, cost savings can be made by adopting a lower return period as the design criteria. For the pile analysis therefore, the wave forces for the 1 in 50-year hurricane condition (Adopted Ultimate Limit State) were derived. The choice of 50 years is often the recommended design criteria because it is a good middle ground between the initial investment on the structure versus the estimated cost of repair in 50 years. The figures below are therefore applicable only to a 1/50 year return period event design condition and a 50-year (~2070) design horizon.

### 6.4.2 Slam, Drag and Inertial Forces

The slam force is created at the initial instantaneous point of contact between the wave and the structure. As a wave crest encounters a structure, there is a transfer of momentum from the water to the structure. Severe local damage, fatigue failure and local yielding are caused by this dynamic impact pressure over a small area and for a short duration (McConnell et al, 2004). Laboratory observations have shown that there are sometimes significant variations in the magnitude of this force for identical wave conditions. This is thought to be a result of air entrainment. The decking and supporting piles will be exposed to this slam force as the waves impact these structures.

As a wave inundates a structure, buoyancy, drag, and inertia forces develop. While the slam force is instantaneous, the drag and inertia forces pulsate between the wave crests and troughs. There is a phase difference between the drag and inertia forces and, as such, both forces do not act simultaneously. The British Standard Code for Maritime Structures (BS 6349 – 1:2000) recommends taking the design wave force as 1.4 times the dominant of the two forces. The force that dominates depends on the ratio of the width of the submerged part of the structure to the orbit width of the water particles in the wave. In this case, the drag forces will be the predominant load. The piles as well as the decking will be exposed to drag forces. The sum of the drag and inertial forces is called the total wave force.

BS 6349 recommends the use of Morrison's Equation (Morrison et al., 1950) to determine wave forces on objects that represent a narrow obstruction to waves. Jue (1993) reported that simple analytical models with Morrison equation-based slamming, drag, and inertial force components, were able to reasonably match the measured forces. The Morrison equation is given as:

**Equation 6.1 Morrison equation**

$$\mathbf{F = 0.5C\rho Au^2}$$

Where:

C = coefficient (slamming, drag or inertia) determined by characteristic of the structure

ρ = the mass density of the seawater.

A = Area of impact

In the case of the beam this is equivalent to the beam width multiplied by its depth

In the case of the pile, it is the horizontal width of the pile multiplied by its height above MSL

u = horizontal fluid velocity.

For the beam, the slamming coefficient recommended by Dean and Dalrymple is 4.71.

For the piles, theory and experiments indicate that the slamming coefficient can vary from C = π to C = 2π distributed vertically along the pile as shown in Figure 6.7 below (Veic & Sulisz, 2018). For this project the maximum force was assumed which would relate to a slamming coefficient of 2π.

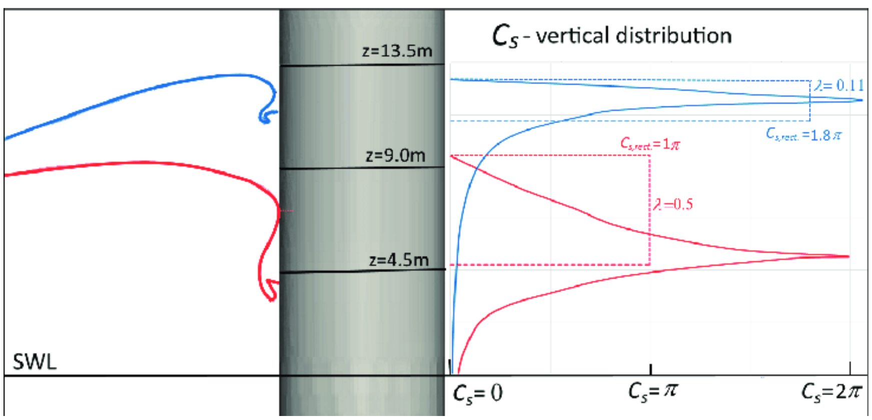



*Figure 6.7 Vertical distribution of slam coefficient along a pile. (Veic & Sulisz 2018)*

BS 6349-1:2000 provides drag and inertia coefficients of 2 and 2.5 respectively, for rectangular objects i.e., the deck. The coefficient for circular piles is dependent on the Reynolds number and wave characteristics. The Reynolds number is a function of the diameter and roughness of the pile. In this case, the applicable drag and inertial coefficients are 1.0 and 0.7, respectively.

## 6.4.3 Uplift Forces

When a wave hits the underside of the deck, the structure experiences an uplift force. The hydrostatic uplift force is defined as that force of water exerted on or underneath a structure tending to cause a displacement of the structure. The upward force is typically followed by a negative force (downward) as the wave passes through the structure and the water exits the underdeck area.

Equation 6.2 derives what is termed as the 'basic wave force' and is denoted as $F^*_v$. This 'basic wave force' is calculated for a wave reaching the predicted maximum crest elevation, $\eta_{max}$, whilst assuming no (water) pressure on the reverse side of the element. $F^*_v$. is defined by a simplified pressure distribution using hydrostatic pressures, $p_1$ and $p_2$, at the top and bottom of the particular element being considered (see Figure 6.8).

Uplift force should be calculated as:

*Equation 6.2 Uplift force*

$$F_v = (surge + \eta_{max}) * \rho * g * A$$

Where:

- surge = storm surge level during storm (increase in surface level)
- ηmax = maximum wave crest height
- ρ = the mass density of the seawater.
- A = vertical area of impact; this is dependent on the element being examined (deck, beam etc.)
- g = acceleration due to gravity

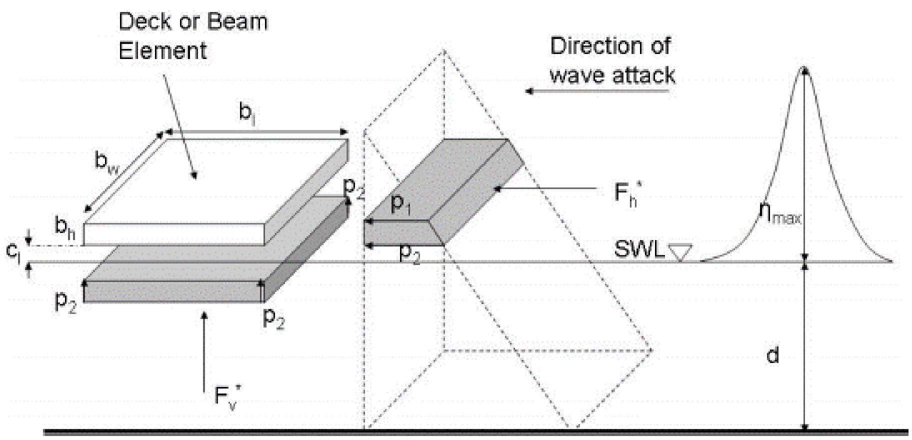



*Figure 6.8 Schematic showing vertical (uplift) and horizontal forces applied to a deck or beam element*

The total force acting on any element of the structure is equal to the sum of the buoyancy of the inundated deck and hydrostatic uplift minus the weight of the overtopping water. However, accurately calculating these forces on the individual members can be complex to attain with any accuracy. The well-known guidelines published in the text *"Piers, Jetties and Related Structures Exposed to Waves"* (McConnell et al., 2004) put forward some methods on the estimation of these forces on individual parts, beams, and decks of the structure. Should that level of detail determined to be required by the structural engineer, that text is strongly recommended as it prescribes methods and empirical coefficients through which the basic vertical force can be transformed into vertical quasi-static (slowly varying) forces acting on the underside of the deck.

For this project, to make the required calculations several assumptions had to be made regarding the direction of wave attack among other factors. The worst-case scenario was used to determine maximum forces, i.e., the basic wave force, not the quasi-static forces. Please note further, that the pressure is what is presented, which will then act across the surface areas of the various elements being considered.

### 6.4.4 Earthquake Forces for Consideration

The damaging effect of earthquakes on marine structures is essentially, but not exclusively, the result of horizontal oscillatory accelerations of the soil mass being transferred to structures above ground level through their foundations, base, or pile support. The response of a structure to these accelerations depends on its type, mass and dimensions and the failure modes to which it might be subject. It is therefore important in seismically active areas, such as Antigua and Barbuda, to select a type of structure that has as little sensitivity to seismic action as can be contrived. Fine sandy soils are especially vulnerable to liquefaction.

The geotechnical engineer will have to review borehole tests, determine the soft soil profile, determine the spectral acceleration for short periods and one second periods for the Maximum Considered Earthquake (MCE), and ultimately determine the potential susceptibility of the sandy layers to liquefaction in the event of the MCE. Based on the depth below seabed level that will liquefy in the event of the design earthquake, the geotechnical engineer will recommend the type of piles to be used to form the foundation system of the deck area and the depths to which the piles should extend below.

### 6.4.5 Design Conditions

As outlined previously in Section 3.2.3, of all the directional sectors, northwest was found to create the highest values and therefore chosen as the design storm. Figure 3.7 shows the wave heights and the water level of the northwest 50-year design storm as simulated by the numerical model. The results of various parameters at the two design points (Figure 6.2) are shown here in Table 6-3.

*Table 6-3 Parameters extracted from the northwest 50-year hurricane simulation at points of interest*

| Parameter | Point | Value |
|---|---|---|
| **Significant Wave Height (m)** | Deep point | 2.36 |
| | Stem point | 2.21 |
| **Mean Wave Period (t02) (seconds)** | Deep point | 10.30 |
| | Stem point | 10.33 |
| **Mean Wave Direction (degrees)** | Deep point | 312.5 |
| | Stem point | 310.4 |
| **Surface Elevation (m)** | Deep point | 1.09 |
| | Stem point | 1.11 |

### 6.4.6 Design Calculations

Using the equations, dimensions and assumptions as outlined above, the slam, drag, inertia and uplift forces impacting the entire sub-structure were calculated for the design event (1/50 yr storm) over a lifetime of 50 years.

The design input constants and wave parameters – some model results and others calculated, are shown below in Figure 6.9.

| **Constants:** | **Unit** | **Value** | |
|---|---|---|---|
| Water density ($\varrho_w$) | kg/m$^3$ | 1025 | |
| Acceleration due to gravity (g) | m/s$^2$ | 9.81 | |
| Density of reinforced concrete ($\varrho_c$) | kg/m$^3$ | 2400 | |
| Density of timber hardwood ($\varrho_t$) | kg/m$^3$ | 600 | |
| | | | |
| **Wave Parameters:** | **50yr** | Deep | Stem |
| Significant Wave Height (Hs) | m | 2.36 | 2.21 |
| Wave Period (Tp) | s | 10.30 | 10.33 |
| Max. water level (surge) | | 1.09 | 1.106 |
| Water Depth of wave (d) | m | 3.41 | 2.701 |
| Wave Length | m | 58.30 | 52.25 |
| Wave number (k) | | 0.11 | 0.12 |
| Anglar Wave frequency (ω) | | 0.61 | 0.608 |
| Height of wave crest above SWL | m | 1.18 | 1.105 |
| Wave Amplitude | | 1.18 | 1.11 |
| Horizontal wave velocity(u) | m/s | 1.23 | 1.29 |

*Figure 6.9 Input constants and wave parameters for the calculation of various wave forces*

The assumed values (highlighted in orange) as well as the resulting forces for the deck are shown in Figure 6.10 below. It should be noted that the horizontal forces are assumed to be acting on the most seaward beam of the suite's sub-structure, while the uplift force was assumed to act on the entire deck sub-structure.

| **Structure Parameters for the DECK:** | | | |
|---|---|---|---|
| beam depth ($b_h$) | m | 0.35 | |
| beam width ($b_w$) | m | 7.10 | |
| deck length ($b_l$) | m | 6.50 | |
| Slam area of impact (A) | $m^2$ | 2.49 | |
| Uplift area of impact (A) | $m^2$ | 46.15 | |
| Height of deck above SWL | m | 1.60 | |
| Weight of deck | KN | 27.16 | |
| | | | |
| **Wave Force Calculation:** | **50yr** | | |
| Slam, Drag and Inertia Coeffcients: | 4.71 | 2.00 | 2.5 |
| | | | |
| Slam, Drag and Inertia Forces: | | **Deep** | **Stem** |
| **Horizontal slam force (Fs)** | kN | 9.12 | 9.92 |
| **Horizontal drag force (Fd)** | kN | 3.87 | 4.21 |
| **Horizontal inertial force (Fi)** | kN | 4.84 | 5.26 |
| | | | |
| Vertical Pressures (Uplift) | | **Deep** | **Stem** |
| Pressure ($p_1$) | $kg \cdot m^{-1} \cdot s^{-2}$ | 11000.44 | 11121.11 |
| Pressure ($p_2$) | $kg \cdot m^{-1} \cdot s^{-2}$ | 22865.64 | 22232.16 |
| Uplift (Impact) | $kN/m^2$ | 22.87 | 22.23 |
| Uplift (pulsating) | $kN/m^2$ | 4.22 | 4.98 |

*Figure 6.10 Calculations of the horizontal forces (kN) on the beam and the vertical pressures ($kN/m^2$) on the deck.*

The calculation sheet for the horizontal forces on the piles is shown below in Figure 6.11.

| **Structure Parameters for the PILES:** | | | |
|---|---|---|---|
| diameter of pile | m | 0.60 | |
| height of pile above MSL | m | 1.60 | |
| Slam area of impact (A) | $m^2$ | 0.96 | |
| | | | |
| **Wave Force Calculation:** | **50yr** | | |
| Slam, Drag and Inertia Coeffcients: | 6.28 | 1.00 | 0.7 |
| | | | |
| Slam, Drag and Inertia Forces: | | **Deep** | **Stem** |
| **Horizontal slam force (Fs)** | kN | 4.70 | 5.11 |
| **Horizontal drag force (Fd)** | kN | 0.75 | 0.81 |
| **Horizontal inertial force (Fi)** | kN | 0.52 | 0.57 |

*Figure 6.11 Calculations of the horizontal forces on the piles*

## 6.5 Discussion and Recommendations

The owner and the architectural team in conjunction with the structural engineer should consider the following:

- Considering all the varying components of a design water level (such as SLR and HAT) along with a freeboard to account for the random nature of waves, a total design deck elevation of +1.6m above MSL is recommended.
    - It should be noted that an investigation was made into using different floor levels for the stem of the layout versus the heart shape. However, it was determined that the resulting levels were too similar to justify varying the floor levels.
    - It should be further noted that the floor level was determined using swell wave conditions, not hurricane conditions. Therefore, even with the recommended design deck elevation, the suites will likely be inundated during tropical storms and hurricanes. Therefore, when hurricane warnings are issued, the suites should be evacuated of all guests and staff and, where possible, electronic equipment and even valuable furniture should be removed or at least lifted off the floor.
- Climate change projections are forecasted based on current information; they are therefore limited (as most projections are) to using present information to predict future trends. The rate of SLR used in this report may accelerate, and the design water level may thus be attained before the 2070 projected horizon. If possible, water level measurements should be taken to make early observations to changes in this parameter, which is so critical to the design.
- The cross-sectional details for the proposed suites were not provided. Therefore, a cross-section from another Sandals location was used. Should this cross-section differ significantly from the proposed cross-section, the wave forces will have to be recalculated.
- The owners should consider the 'constructability' of the layout. The stem of the shape can be constructed using land-based heavy machinery, through the deployment of a construction pad on the sea floor to facilitate heavy machinery accessing the construction area. However, because the heart shaped portion of the suites will be constructed in water depths of roughly 3.3m (~11'), creating a construction pad and using land-based machinery may not be feasible. An offshore barge will perhaps need to be brought in and the piles driven from there. Due consideration should be given to the relative cost of both methodologies and perhaps even to reconfiguring the layout to shallower water.
- The owners should consider adjusting the proposed configuration to be slightly more landward. Doing so would result in lower depths in which the suites would be placed. This would have several positive impacts:
    - Shallower water would make for easier (and likely cheaper) construction as discussed above.
    - Shallower water would likely mean lower wave forces.
    - Pulling the configuration more landward would remove it from the seagrass beds and therefore removal and relocation would likely not be required. (This is discussed at length in the following chapter.)

# 7 Impact Analysis

The proposed overwater suites could have impacts on the coastal processes and the marine environment. These impacts could be either positive or negative. It is important to identify and, where possible, quantify the impacts to gain a better understanding of the development's role in the changes of its surroundings. The Government's Department of Environment will be particularly interested in the impacts of the suites, and their key concerns are likely to include:

- How will these structures affect the currents and waves in the bay and sandy shoreline? How will sediment movement be affected? Will there be downdrift sediment impacts?
- How will the suites affect benthos such as seagrass, corals, and marine fauna in the zone of direct impact? Are there expected impacts to the marine environment other than the benthos? What will these impacts be, both during construction and after? How will these negative impacts be mitigated?

Addressing these issues is the focus of this chapter.

## 7.1 Impacts on Coastal Processes

The presence of structures in the marine environment can affect long-term sedimentation (downdrift erosion), along with wave conditions and the current patterns in the downdrift areas. Due diligence in the engineering requires that the proposed designs do not adversely affect the environment and locations adjacent to structures. Along the Dickenson Bay coastline there are several properties with beaches that are of high value to their owners. The impact of the proposed overwater suites on the entirety of Dickenson Bay was therefore examined under swell conditions.

A typical swell event coming from the northeast to east sector was used to quantify the range of coastal related short-term impacts resulting from the implementation of the proposed Overwater Suites options.

The piles used in this analysis are circular piles with a diameter of 0.61m (24 inches). The pile orientation for the proposed concepts were extrapolated from a client-issued drawing and an assumption of 5m spacings between each pair of piles along the walkway. Piles were also assumed at the critical junctures in the building footprint. As such, the modelled impacts are valid under the condition that these assumptions are also valid. The assumed pile layout as it was input to the numerical model is shown in Figure 7.1.

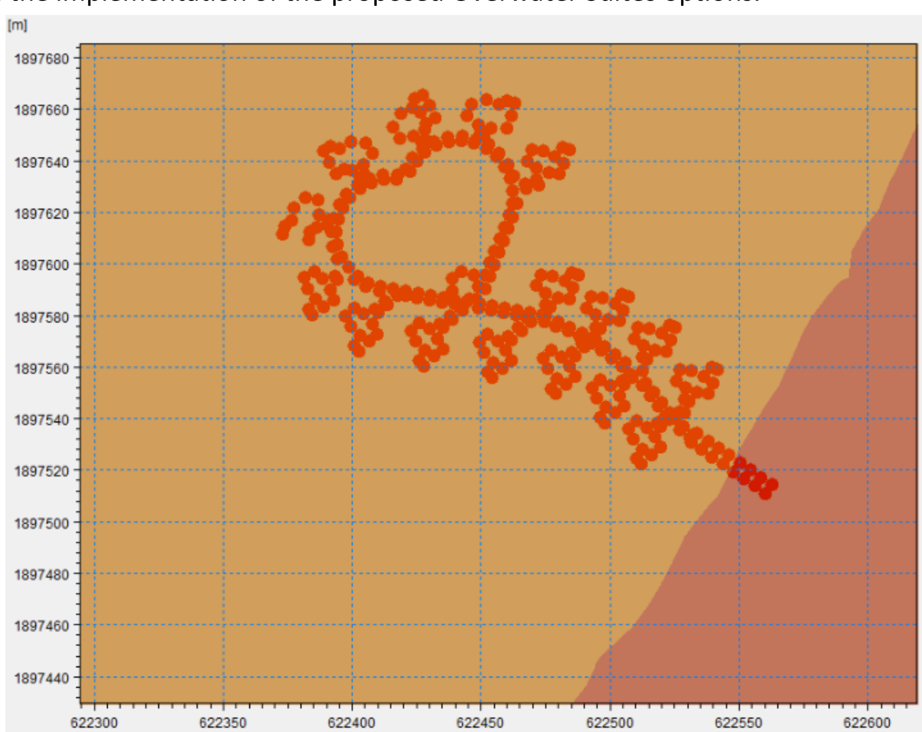



*Figure 7.1 Pile layout related to the proposed Overwater suites*

The calculated annual wave climate revealed that swell waves only arrive at the project site on rare occasions, primarily during the winter months. Nonetheless, because of the recommendation / assumption that the suites will be evacuated for hurricanes, swell waves will be the most signifcant wave incident on the suites. Further, since swell waves have the potential to substantially impact the shoreline, swell waves were examined. A time series of wave conditions representative of a winter swell event (Figure 7.2) was extracted from the deep-water ERA 5 database for a three-day period in January 2021.

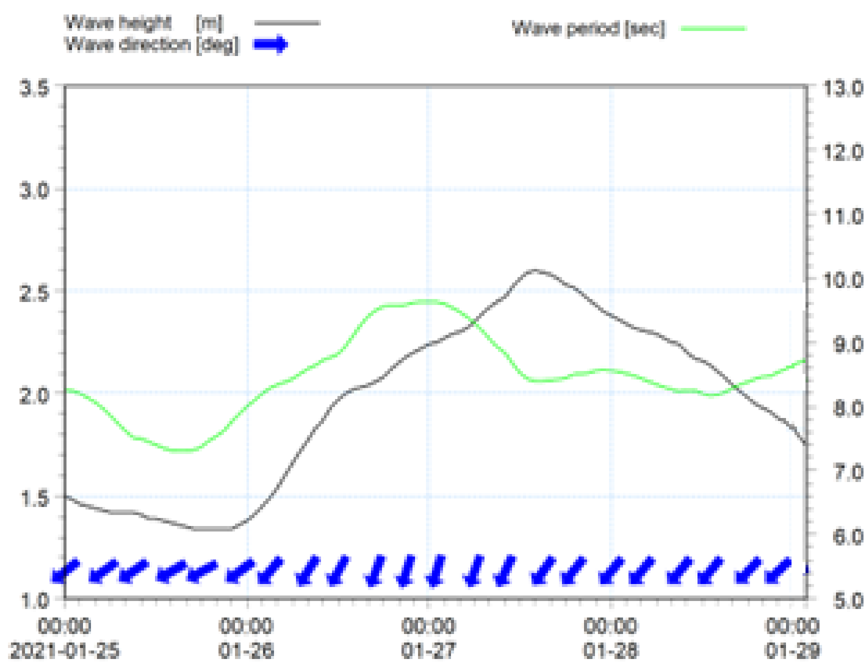



*Figure 7.2 Winter swell event time series over a 3-day period in January 2021*

This swell event was simulated numerically both with the proposed piles in place and without, to quantify the short-term impacts resulting from the implementation of the proposed concepts. The resulting significant wave heights and directions as well as the current speed and direction were plotted at the peak of the swell event, which occurred on January 27th at 10:50am. Comparisons were made between the existing conditions and the proposed layout and results are plotted in Figure 7.3 and Figure 7.4. Similarly, the resulting accumulated bed level change and direction of sediment transport was plotted at the end of the swell event and is shown in Figure 7.5.

Overall results indicate similar trends on beach response, current speed, and wave height when the existing and proposed scenarios are compared, i.e., very little change. The small impact observed in the models is likely due to the slenderness of each pile, which because of its size, blocks and reflects only a small percentage of incoming wave energy, allowing the remaining waves to converge, reform and freely propagate through the structure.

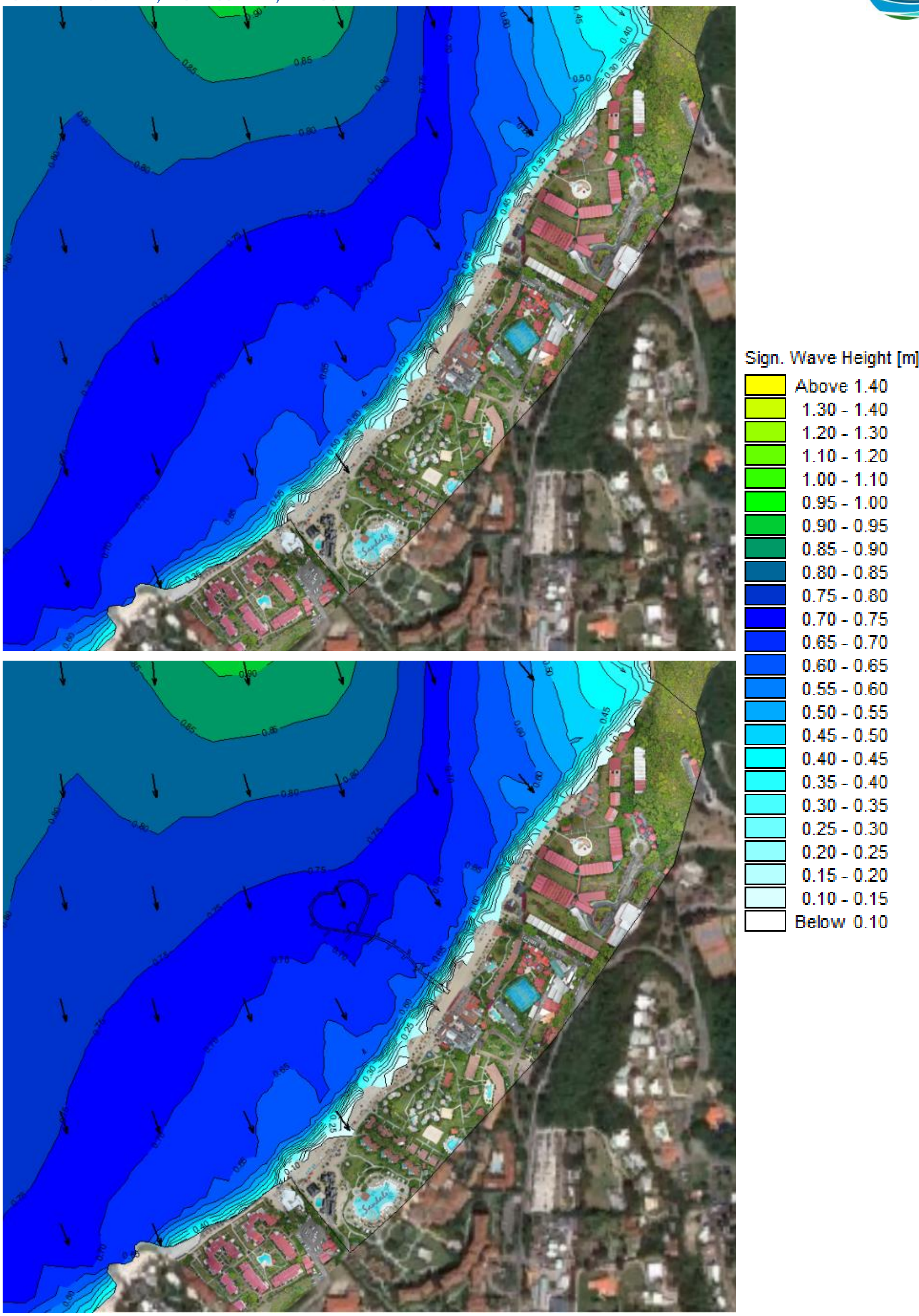



*Figure 7.3 Resulting wave height and direction at the peak of January 2021 swell event. Existing (top) versus Proposed (bottom)*

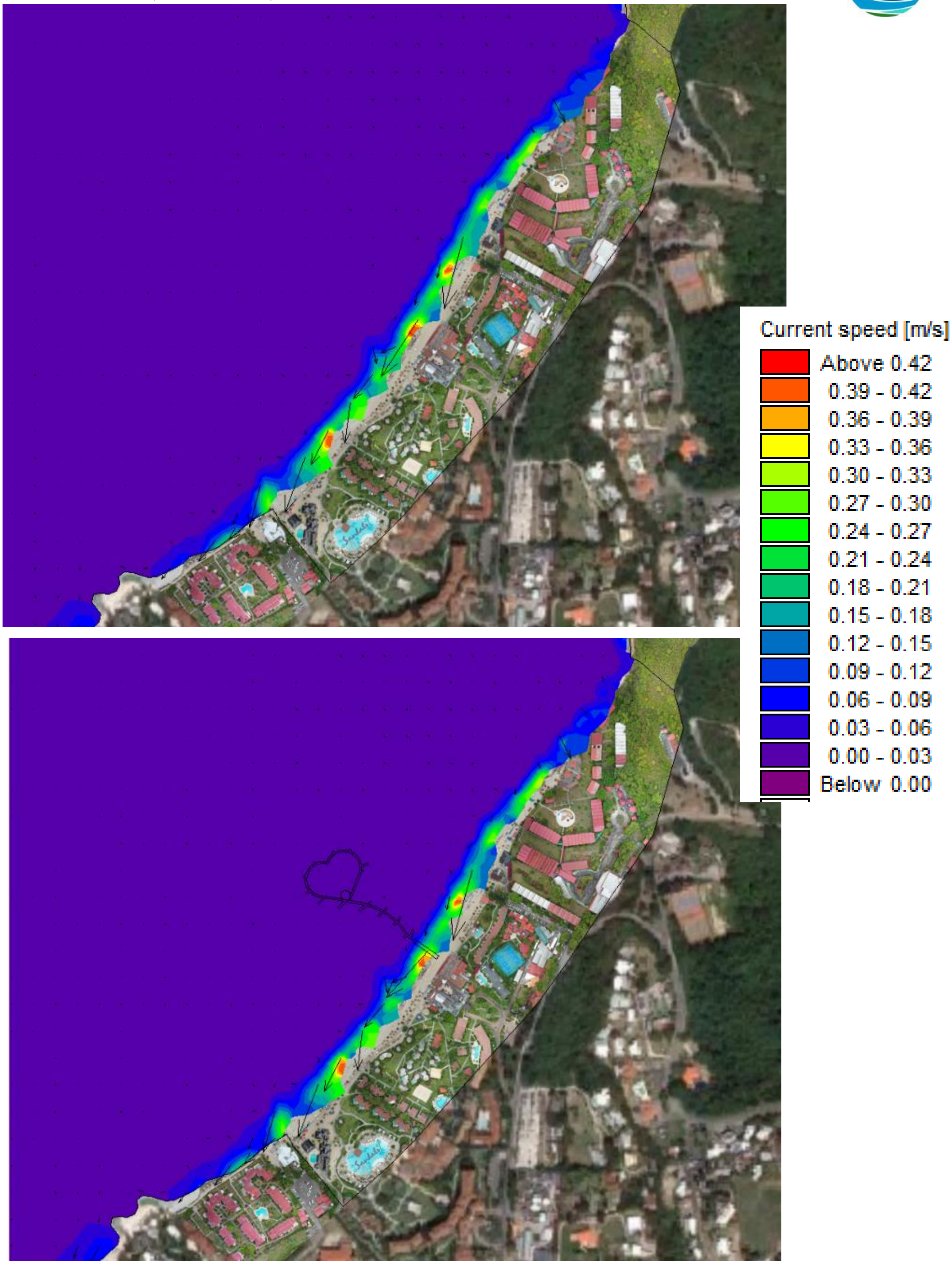



*Figure 7.4 Resulting current speed and direction at the peak of January 2021 swell event Existing (top) versus Proposed (bottom)*

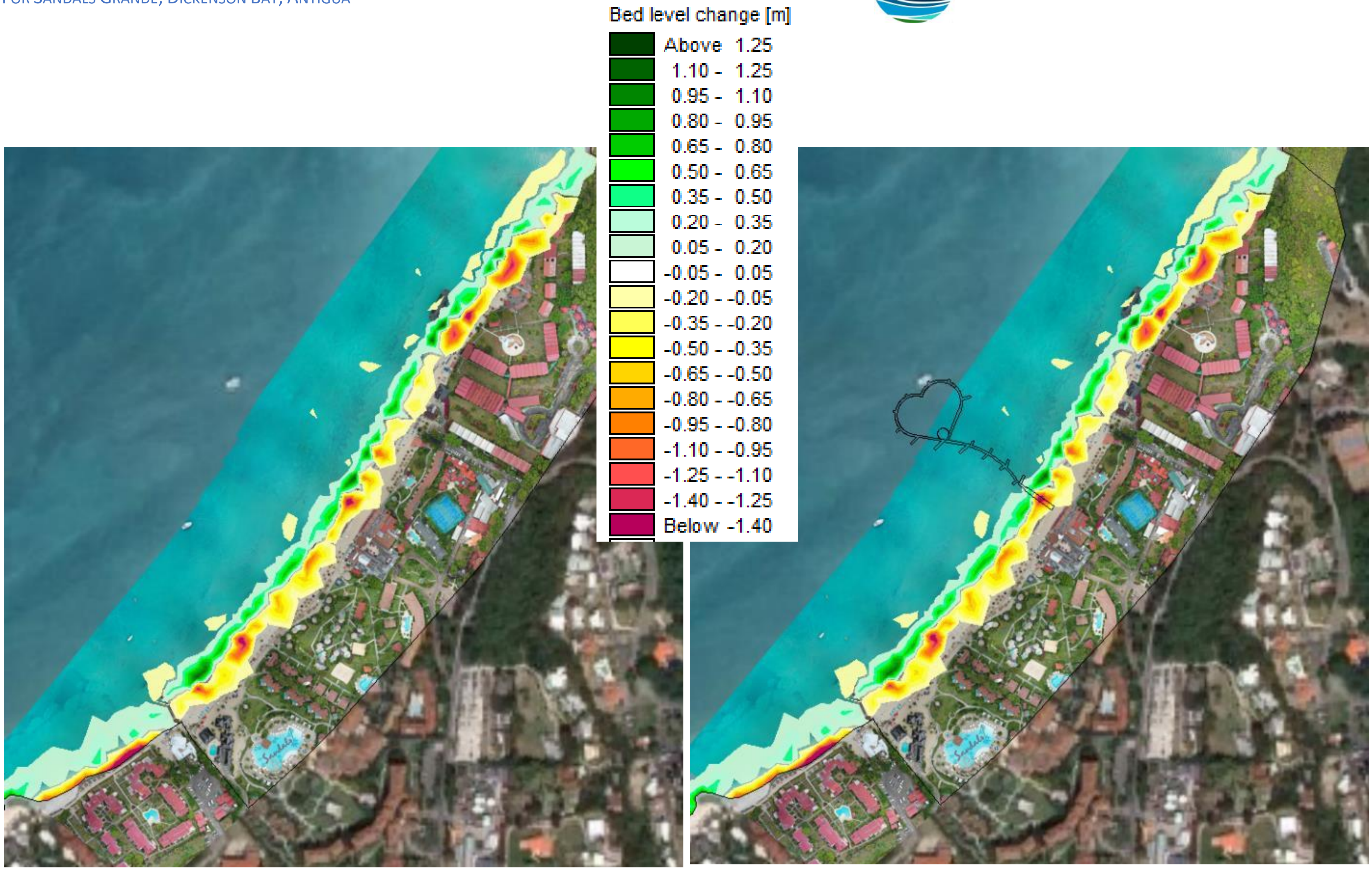



*Figure 7.5 Resulting bed level change at the end the January 2021 swell event. Existing (left) versus Proposed (right)*

The spatial comparisons shown above are so slight that it is difficult to discern any changes between the existing and proposed plots. Table 7-1 shows the comparisons made at specific extraction points (location of which are shown in Appendix A-1). Analysis of that table confirms there is no significant difference between the two scenarios. The most noteworthy difference is with the modelled bed level change, which shows increased erosion at three (3) of the extraction points under the proposed scenario and less erosion at point 2, which is right by the proposed walkway to the suites. This observation could point to an interruption of the alongshore sediment transport, however, the values are considered too small to draw any meaningful conclusions and point to the need for further study backed by measured field data.

*Table 7-1 Differences at extraction points between the proposed scenario and the existing scenario*

| | | Point 1 | Point 2 | Point 3 | Point 4 | Point 5 |
|---|---|---|---|---|---|---|
| **Significant Wave height (m)** | Proposed | 0.6348 | 0.6552 | 0.7070 | 0.6482 | 0.4710 |
| | Existing | 0.6325 | 0.6654 | 0.7068 | 0.6513 | 0.4609 |
| | **Difference** | **0.0022** | **-0.0102** | **0.0002** | **-0.0031** | **0.0101** |
| **Current Speed (m/s)** | Proposed | 0.0037 | 0.0100 | 0.0045 | 0.0088 | 0.0052 |
| | Existing | 0.0038 | 0.0098 | 0.0045 | 0.0090 | 0.0054 |
| | **Difference** | **0.0000** | **0.0002** | **0.0000** | **-0.0003** | **-0.0002** |
| **Bed Level Change (m)** | Proposed | -0.0207 | -0.0018 | -0.0214 | -0.0123 | -0.0090 |
| | Existing | -0.0206 | -0.0029 | -0.0213 | -0.0119 | -0.0090 |
| | **Difference** | **-0.0001** | **0.0012** | **-0.0001** | **-0.0003** | **0.0000** |

The overarching results are also summarised in the following table.

*Table 7-2 Identified short-term impacts*

| Parameter | Impacts from Easterly Swell (March 2017) on Option 1 |
|---|---|
| **Waves** | · Minimal impacts within the footprint or in the nearshore. Wave heights under proposed are ±0.01m of the existing wave heights.<br>· No noticeable downdrift changes. |
| **Currents** | · No observable impacts within the footprint<br>· Current magnitude and direction remain more or less the same as existing with the proposed suites in place.<br>· No noticeable downdrift impacts. |
| **Sediment Transport** | · No observable impacts within the footprint.<br>· There is a slight trend towards increased erosion west of the proposed jetty, but the differences are too small to make a conclusion. |

The overall findings of the analysis therefore indicate that the proposed layout has negligible impacts on local nearshore processes. The analyses also indicate no noticeable adverse downdrift impact further to the east or west of the proposed layout. The design comprising of 0.61m (24-inch) diameter piles at a 5m spacing was therefore found to be suitable in minimizing current, wave and sediment impacts on the environment. It should be noted, however, that any increase in pile diameter or a decrease in pile spacing could cause an exponential increase in the impact of the structure on the environment, and the impacts would thus have to be remodelled if there is any such change in the design or if the assumptions made are incorrect.

## 7.2 Impacts on the Benthic Environment

Analysis undertaken to date indicate that there is no coral in the footprint of the structure or in the nearshore that may become damaged in the construction of the suites. There is, however, dense seagrass in the footprint of the most seaward of the suites, approximately 140m from the shoreline.

The susceptibility of seagrass to negative biotic and abiotic influences is well documented. Anthropogenic influences include point and non-point sources of pollution, which result in diminished water clarity, excessive nutrients in runoff, and sedimentation. The effect these influences will have on seagrass depends on their type and severity. Seagrasses typically recover from some damage isolated to their leaves; however, should the roots be damaged, the ability of the plant to properly recover is severely diminished.

The productivity of seagrass communities is dependent on the ability of sunlight to penetrate the water column. The positioning of the overwater suites over these seagrass beds will undoubtedly affect how much light ultimately reaches them. Further, during construction itself, water clarity can be negatively affected by the movement of equipment and machinery. Additionally, the potential storm water runoff to the area is another influencer on light penetration through the water column. Various anthropogenic sources contribute to storm water runoff and may originate from both urban and agricultural areas. It is therefore necessary that storm water runoff is properly managed by Sandals. Seagrasses maintain water clarity by trapping silt, dirt, and other sediments suspended in the water column. These are incorporated into the benthic substratum, where they are stabilized by the seagrass roots. When sediment loading becomes excessive, the increase in turbidity of the water inhibits the penetration of sunlight. In extreme cases, seagrasses may be smothered by these excessive sediment loads.

Water with excessive levels of nitrogen and phosphorous can cause acceleration of the growth rate of phytoplankton. Microalgae grow at manageable levels under normal nutrient loadings and are an important food source for many filter-feeding and suspension-feeding organisms. However, excess nutrient loading in the seawater may cause algal blooms that reduce water clarity by blocking the amount of sunlight available. Reduction in light levels, as well as depletion of the nutrient supply, leads to the death and decomposition of these microalgal blooms. The process of decomposition further degrades water quality by depleting much of the dissolved oxygen available in the water column, sometimes leading to hypoxic conditions and fish kills. It is therefore critical that nutrient loading of the waters be avoided as much as possible, and generally monitored through regular water quality testing.

# 8 Recommended Mitigation Strategies

## *8.1 Mitigation Recommendations for Coastal Processes*

As previously stated, the design as it is currently presumed (0.6m diameter piles at a 5m spacing) was found to be suitable as it created no noticeable downdrift impacts on waves, hydrodynamics, or sediment movement along the Dickenson Bay shoreline. Because of the negligible impacts on coastal processes, no mitigation strategies have been recommended as long as the suites are constructed as outlined in the existing design, with an allowance of 0.15m to account for potential seabed erosion, not including localized scour. Should the assumptions as outlined herein not be true, or should there be any change to the design that would result in either an increase in pile diameter or a decrease in pile spacing, the new design would have to be remodelled and potential impacts noted. Appropriate mitigation strategies could be developed at that time.

It should be noted that design changes were recommended herein, although not having to do with pile spacing. These recommended changes are reiterated here:

- The deck elevation should be increased from +0.9m above MSL (as shown in the sketch) to +1.6m above MSL.
- The structural engineer should consider the design forces as presented herein.
- The geotechnical engineer should also consider an allowance for seabed change in the order of 0.15m in the pile design.

## *8.2 Mitigation Recommendations for Marine Benthos*

As outlined above, any construction done in the area will lead to deleterious effects on the seagrass beds present (towards the most seaward extent of the proposed suite layout) both during the construction period (construction works leading to increased turbidity in the water and physical damage to the seagrass present) and afterwards (direct shading of the seagrass, changes in substrate regimes). To mitigate this, there are two options which should be considered:

a) Altering the proposed layout to slant the heart shape and 'pull it in' towards the shoreline. In this manner, the seagrass beds would be avoided, and the suites would be located at a lesser depth which would also reduce the height of the waves incident upon the elements.

b) Removal and subsequent relocation of the seagrass that would be impacted (by specialists trained in this field). The approach to the seagrass relocation is outlined below.

### 8.2.1 Identification of Potential Recipient Sites

The seagrass present could be successfully relocated to sites with similar abiotic conditions and preferably already established seagrass. As shown in Figure 8.1, there is no shortage of these sites as they may be found to the left, right and further seaward beyond the boundaries of the proposed construction area, as seagrass was observed to be present there. Finding a suitable recipient site therefore should not be a difficult task.

The following criteria should be used in the assessing the suitability of any proposed recipient sites:

- Proximity to proposed overwater suites site to facilitate quick replanting so as to minimize stress on the beds;
- Similar exposure to waves and currents to facilitate efficient replanting and avoid displacement of replanted beds;

- Low turbidity and existing growth of seagrass to ensure that conditions are indeed suitable; and
- Type of substrate.

Based on the above criteria, it is believed that the "blowouts" in the existing seagrass beds near the proposed suite location would be ideal for the replanting and recollecting process. These areas would show similar wave and current patterns as the existing site, and they are  surrounded by existing seagrass beds now showing that the conditions there are favourable for growth.

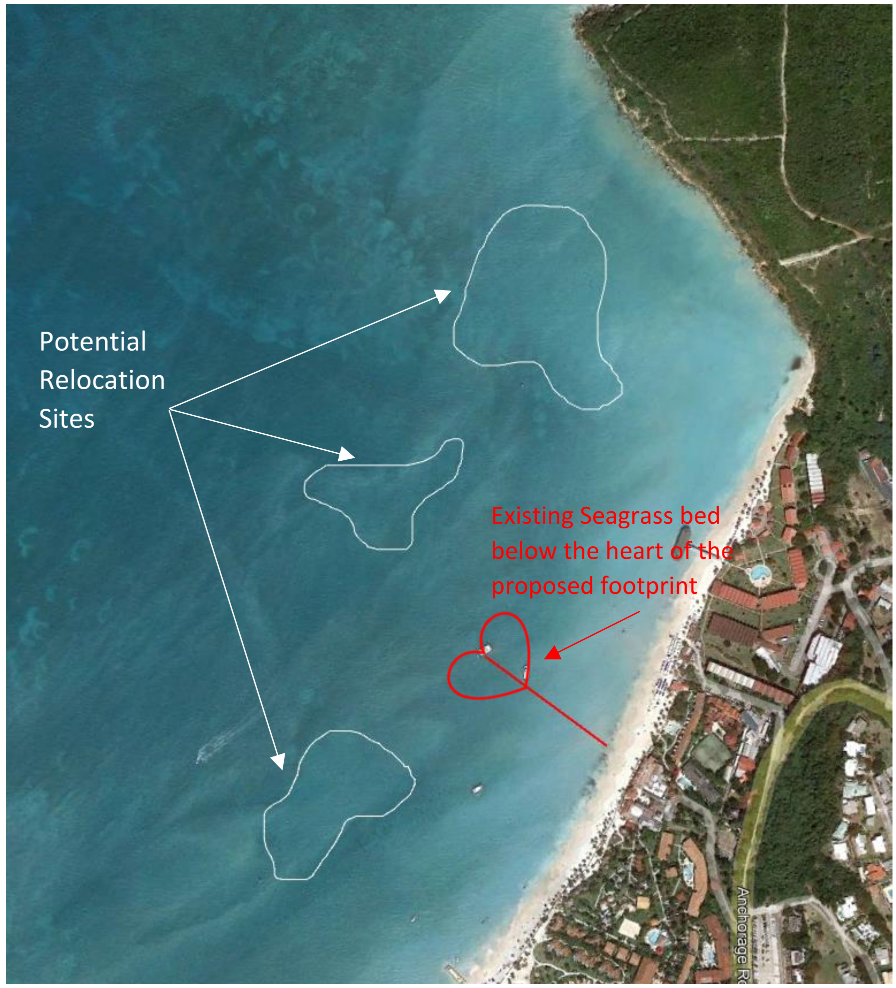


*Figure 8.1  Potential seagrass relocation sites in relation to existing seagrass for the proposed Sandals Dunn's River*

### 8.2.2 Recommended Methodology

#### *Harvesting*

The modified *Mat Method* should be used to harvest the seagrass if this is the decision moving forward. This entails using shovels and pitch forks to cut and extract the seagrass in mats. The sections will be removed with the rhizomes and soils attached. These sections will remain submerged at the harvesting site until they are needed for replanting.

Keeping the sections submerged at the harvesting site ensures that the leaves are covered with water thereby preventing desiccation as well as maintaining the same osmotic potential and temperature. The quantities reaped per day will be limited to that which can be replanted within the same day. Each mat is referred to as a 'Planting Unit'. Once harvested, the planting units will be placed on rafts in boats and transported to the replanting site.

#### *Replanting*

The replanting method is dependent on the type of substrate[5] at the recipient site. Generally, this could be either sand or a thin layer of sand covering rock. A sandy substrate is preferred and if not found, perhaps should not be used as a recipient site. If the substrate is sand, the method of replanting that will be employed

[5] The substrate is the term used to describe the physical surface that the seagrass grows on.

is the *Staple Method*. The planting units will be stabilised by "stapling" them into the seabed with the use of U-shaped rebars. Divers will manually place these mats on the seabed.

*Harvesting and Replanting Periodicity*

The harvesting and replanting will be a daily event. However, the amount harvested and replanted per day is dependent on the distance to travel from the reaping site to the replanting site, in addition to factors such as weather and visibility. The timeline will also be affected by the activities that have to be undertaken at the recipient site after the grass has been replanted.

*Preparation and Monitoring of Project Site*

The project site will be enclosed with turbidity curtains (screens) to minimize leakage of turbid waters to the surrounding environment. Points outside of the turbidity curtain will be monitored to determine if turbidity is escaping and impacting the marine environment outside of the isolated area.

*Removal and Relocation of Other Sensitive Organisms*

At the start of each day of operation, any observed sedentary or slow-moving marine fauna and/or flora (e.g., *Diadema antillarum*) that require removal and relocating will be done in an approved method, before harvesting commences. These flora and fauna will be placed in a bucket or a basket and immediately moved to areas outside of the harvesting area. The basket will be transported through the water so that the species remain submerged. There will be no holding time, therefore reducing the potential for them to become stressed. If a bucket is used, it will be filled with sea water from the site and the organisms will be submerged in the bucket and immediately relocated to a safer zone.

## 8.3 Construction Mitigation Plan

The potential negative impacts were examined in relation to the construction and operational phases of the development and are described within this section. The impacts of the structures are localized. Localized impacts mean that the effects are only felt within the area of the project site.

### 8.3.1 Construction

*Smothering:*

- The construction method for the driving of the piles and installation of the deck is undetermined at this time. If it is to be constructed using land-based heavy machinery, there will be a need to deploy construction pads on the sea floor to facilitate heavy machinery accessing the construction area. (It should be noted however that due to the depths of the suites, the contractor may opt to install the piles from an offshore barge, which would not result in smothering during its activity).
- Regardless of the methodology employed, the benthic resources in the footprint of the proposed overwater suites are likely to be impacted negatively by the physical disturbance resulting from the construction activities related to the pile and deck installation.
- To mitigate the effects, the benthic resources within the footprint of the structure will have to be relocated prior to construction. For the marine life outside of the footprint, a turbidity barrier should be used during construction to prevent fine material from going offshore.

*Turbidity:*

- The dominant component of the sediment in the project area is sand. The driving of piles, and the deployment and removal of construction pads will all generate turbidity. This turbidity can affect sensitive resources directly by smothering, or indirectly by occluding the water column in the vicinity of the construction. The limited circulation in this embayment makes it unlikely that the turbidity generated will lead to the formation of plumes affecting resources further alongshore.

- Similarly, a turbidity barrier will have to be used to lessen the spread of fines. A turbidity meter should be used to measure the turbidity outside of the construction area to ensure that the turbidity readings are within the acceptable range.

*Oil Pollution*

- There is the potential for fuel leaks or spills from equipment used during construction during refuelling or operation.
- Refuelling of the boat and sea-based equipment should only be done at anchor out at sea if the sea conditions are calm, otherwise, all refuelling should be done when docked at land. Appropriate refuelling equipment (such as funnels) and techniques should be used at all times.
- There should be appropriate minor spill response equipment (for containment and clean up) kept on site, to include oil absorbent pads and disposal bags.

### 8.3.2 Post-Construction

*Debris:*

- Any debris left on the seabed from the construction activity can become a projectile during severe wave activity, and this may cause damage to sensitive benthic resources.
- It is expected that a thorough swim-through will be done at the site after construction when the turbidity is back to normal. The inspection will ensure that all debris is removed and carted offsite.

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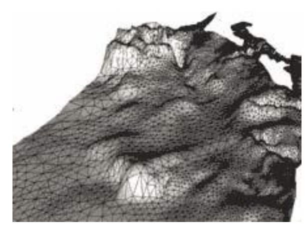
# Appendix A – MIKE 21 (by DHI) Model Description

**MIKE 21** is a professional engineering software package for the simulation of flows, waves, sediments and ecology in rivers, lakes, estuaries, bays, coastal areas and seas. The modelling system is designed in an integrated modular framework with a variety of add-on modules. This, in combination with the range of dedicated and easy to use tools and editors, allow you to customize your personal software package to suit your own specific needs, whether for simple or more complex 2D flow modelling needs.

**MIKE 21** provides

- A complete and effective design environment
- An advanced GUI combined with a series of highly efficient computational engines
- GUI facilities for easy applications
- GIS integration
- Free tools, e.g., for processing of model data in MATLAB
- Integration with urban and water resource models for flood modelling
- Modules for virtually any kind of 2D water modelling needs
- Open, flexible and easy ecology and water quality modelling
- Sophisticated tools for data handling, analysis and visualization
- Multiple computational grid options ensuring optimal model application
- Well-proven technology with 30+ years of track record
- Widely used by thousands of engineers and scientists worldwide

## *Flow Model Versions*



**MIKE 21 FM** is based on an unstructured mesh and uses a cell-centred finite volume solution technique. The mesh is based on linear triangular elements. The FM version is particularly well suited for modelling large complex areas that, at the same time, require a detailed resolution of specific features.

## *Hydrodynamics*

The hydrodynamic modules provide the basis for computations performed in many other modules but can also be used alone. They simulate the water level variations and flows in response to a variety of forcing functions on flood plains, in lakes, estuaries and coastal areas.

In MIKE 21 the HD modules solve the vertically integrated equations for the conservation of continuity and momentum, i.e., the Saint Venant equations on rectangular, flexible or curvilinear grids covering the area of interest, when provided with the bathymetry, bed resistance coefficients, wind field, hydrographic boundary conditions, etc.

The effect of waves on the currents can be included in various ways, e.g., by apparent bed roughness. Including wave-induced flow in the model is done by specifying wave radiation stresses, which then will enter the momentum equations. These can also be imported directly from the wave models MIKE 21 SW/NSW or PMS.

The effects of sources and sinks like precipitation and evaporation, river discharge, intakes and outlets from power stations, etc are included in the hydrodynamic equations. The impact of hydraulic structures (bridge piers or piles, weirs, etc) on the flow conditions can also be included. A valuable facility in MIKE 21 is its capability to compute the flow in an area that sometimes dries out and sometimes is flooded, e.g., tidal flats and flood plains.

MIKE 21 C, the flow model for the curvilinear version, includes helical three-dimensional flow that occurs in curved flows, especially in river bends. Helical flow is a principal secondary flow phenomenon in rivers that has a significant influence on the sediment transport direction and hence the morphological changes in the river channel.

The US Federal Emergency Management Agency (FEMA) has officially approved MIKE 21 HD and NHD for use in national flood insurance program studies (NFIS) for applications in both coastal and riverine environments.

### *SW Spectral Wave Module*

**MIKE 21 SW** is a new 3rd generation spectral wind-wave model that simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas. MIKE 21 SW solves the spectral wave action balance equation formulated in either Cartesian or spherical co-ordinates. At each element, the wave field is represented by a discrete two-dimensional wave action density spectrum.

The model includes the following physical phenomena; wave growth by action of wind, non-linear wave-wave interaction, dissipation by white-capping, dissipation by wave breaking, dissipation due to bottom friction, refraction due to depth variations, and wave-current interaction.

The discretisation of the governing equations in geographical and spectral space is performed using the cell-centred finite volume method. In the geographical domain an unstructured mesh is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action. **MIKE 21 SW** includes two different formulations:

- fully spectral formulation
- directional decoupled parametric formulation

**MIKE 21 SW** is used for the assessment of wave climates in offshore and coastal areas - in hindcast and forecast mode. A major application area is the design of offshore, coastal and port structures for which accurate assessment of wave loads is of utmost importance to the safe and economic design of these structures.

**MIKE 21 SW** is particularly applicable for simultaneous wave prediction and analysis on regional scale and local scale. Coarse spatial and temporal resolution is used for the regional part of the mesh and a high-resolution boundary and depth-adaptive mesh is describing the shallow water environment at the coastline.

**MIKE 21 SW** is also used for the calculation of the sediment transport, which, to a great extent, is determined by wave conditions and associated wave-induced currents. The wave-induced current is generated by the gradients in radiation stresses that occur in the surf zone. MIKE 21 SW can be used to calculate the wave conditions and associated radiation stresses. The long-shore currents and sediment transport are then calculated using the flow and sediment transport models available in the MIKE 21 package.

### *Coupled Model FM*

**MIKE 21/3 Coupled Model FM** is a truly dynamic modelling system for application within coastal and estuarine environments. It is composed of following modules:

- Hydrodynamic Module
- Spectral Wave Module
- Transport Module
- ECO Lab Module
- Mud Transport Module
- Sand Transport Module (only 2D simulations)

The Hydrodynamic Module and the Spectral Wave Module are the basic computational components of the MIKE 21/3 Flow Model FM. Using MIKE 21/3 Coupled Model FM it is possible to simulate the mutual interaction between waves and currents using a dynamic coupling between the Hydrodynamic Module and the Spectral Wave Module. The MIKE 21/3 Coupled Model FM also includes a dynamic coupling between the Mud Transport and the Sand Transport models and the Hydrodynamic Module and the Spectral Wave Module. Hence, a full feedback of the bed level changes on the waves and flow calculations can be included.

**Application Areas**

The application areas are generally problems where flow and transport phenomena are important with emphasis on coastal and marine applications, where the flexibility inherited in the unstructured meshes can be utilized.

MIKE 21/3 Coupled Model FM can be used for investigating the morphological evolution of the nearshore bathymetry due to the impact of engineering works (coastal structures, dredging works etc.). The engineering works may include breakwaters (surface-piercing and submerged), groins, shoreface nourishment, harbours etc. MIKE 21/3 Coupled Model FM can also be used to study the morphological evolution of tidal inlets.

It is most suitable for medium-term morphological investigations (several weeks to months) over a limited coastal area. The typical dimensions are about 10km in the alongshore direction and 2km in the offshore direction. The computational effort can become quite large for long-term simulations, or for larger areas.

**Computational features**

The main features of the MIKE 21 Coupled Model FM are as follows

- Dynamic coupling of flow and wave calculations
- Fully feedback of bed level changes on flow and wave calculations
- Easy switch between 2D and 3D calculations (hydrodynamic module and process modules)
- Optimal degree of flexibility in describing bathymetry and ambient flow and wave conditions using depth-adaptive and boundary-fitted unstructured mesh.

### *ST Sediment Transport Module*

**MIKE 21 ST** is mainly used to determine the sediment transport pattern (or changes in this pattern) and the initial rates of sedimentation/erosion due to the impact of engineering works. The simulations can be done for pure currents and combined currents and waves. Several formulations calculating sand transport in pure currents are implemented in the model. The STP (detailed sand transport model also used in LITPACK) and Bijker´s method is available for calculating sand transport rates in combined currents and waves.

It is an advanced sand transport model both for pure current or current and wave conditions, which includes influence of breaking and non-breaking waves, currents due to various driving forces, coastal structures, complex bathymetry, sediment gradation, etc. Some of the processes described in STP include waves propagating at an arbitrary angle with respect to the current, breaking/unbroken waves, effect of ripples, sediment grading, bed slope, wave asymmetry, undertow, etc.

Typical application areas for MIKE 21 ST are:

- Morphological optimization of port layouts, taking into consideration sedimentation at port entrance, sand bypassing and downdrift impact, etc

- Detailed coastal area investigation of the impact of shore protection structures on adjacent shoreline. Sand losses from bays due to rip currents, etc

Stability of tidal inlets -assessment of the ability of the tidal flows to maintain the entrance after sudden sedimentation due to littoral drift

## *Appendix A-1 – MIKE 21 Numerical Model Domain*

To investigate the coastal processes along the existing Dickenson Bay shoreline, a flexible mesh model was created. The goal being to numerically represent the existing environment by representing the project site's existing depths and elevations in a flexible mesh, which is accomplished using the DHI MIKE Suite of models.

Bathymetric and topographic data collected via satellite, transect, and bathymetric surveys are incorporated into the model. This information was entered into the MIKE Zero mesh developer, which was then used to generate the flexible mesh depicting the existing conditions, as shown in the figure below.

The baseline conditions for wave and sediment transport processes were established using the existing-condition flexible mesh.

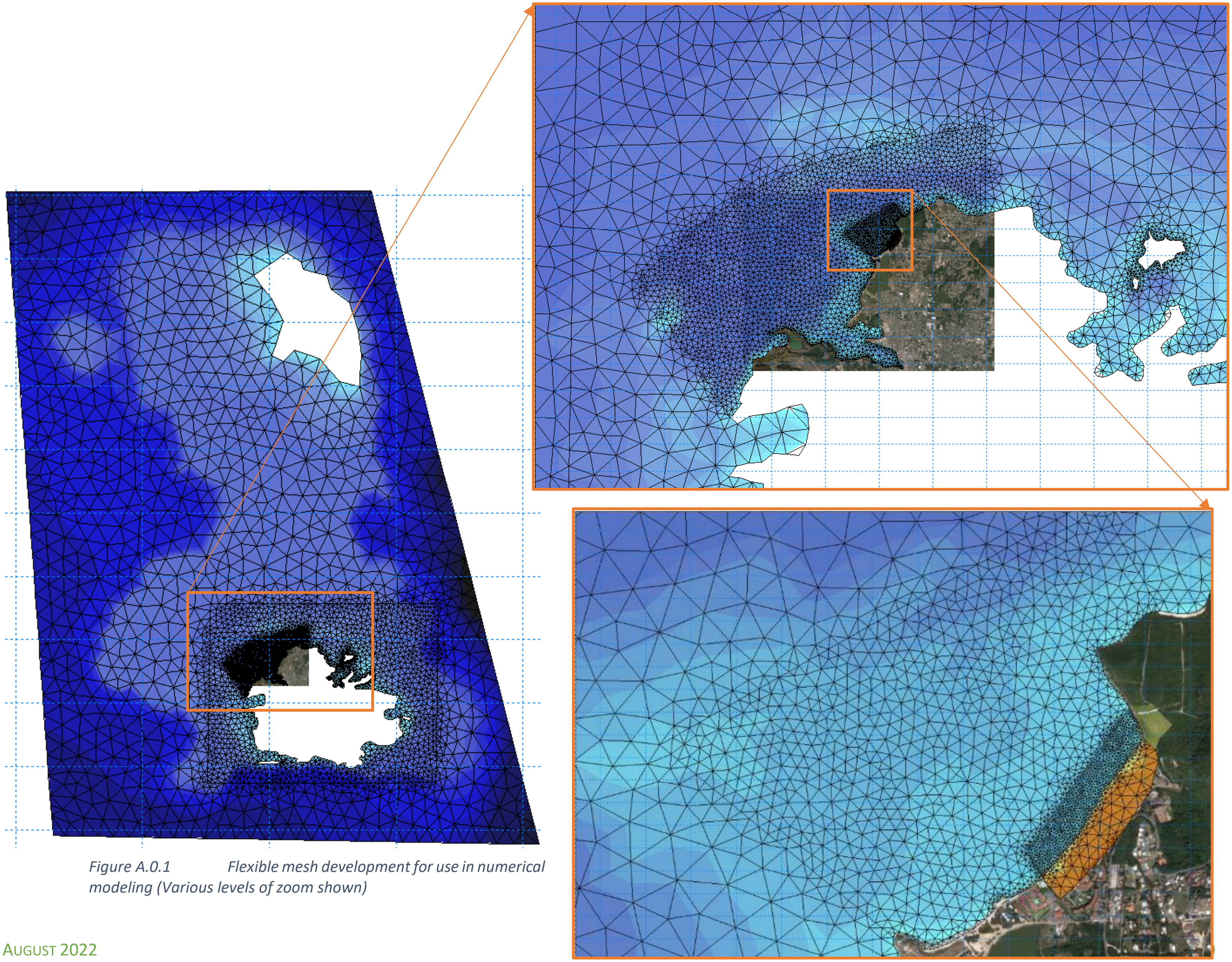


*Figure A.0.1 Flexible mesh development for use in numerical modeling (Various levels of zoom shown)*

| | Easting | Northing |
|---|---|---|
| Point 1 | 622350 | 1897350 |
| Point 2 | 622470 | 1897500 |
| Point 3 | 622500 | 1897650 |
| Point 4 | 622600 | 1897700 |
| Point 5 | 622700 | 1897850 |
| Deep Point | 622365 | 1897600 |
| Stem Point | 622415 | 1897542 |

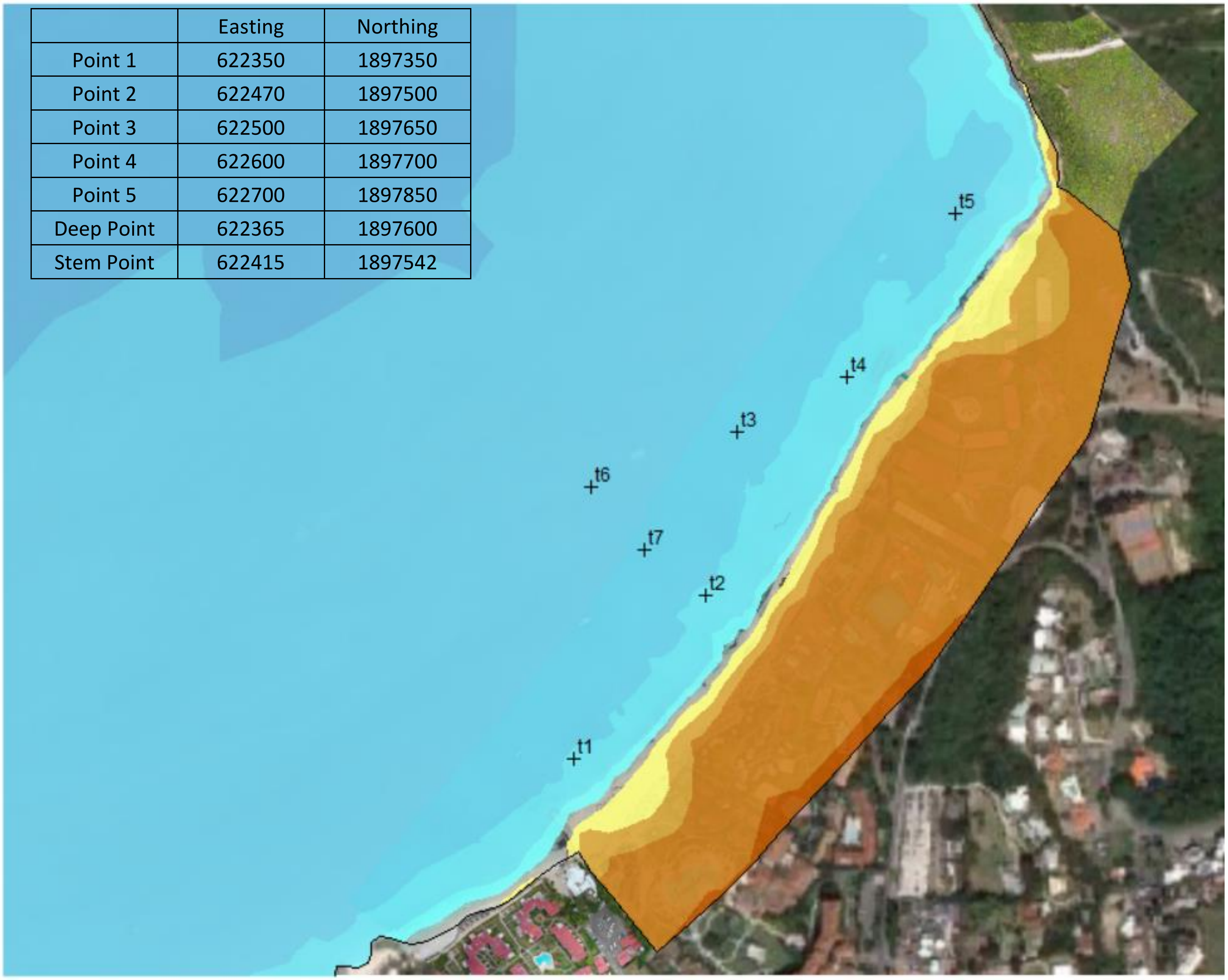



*Figure A.0.2 Extraction points for various parameters in numerical simulation*

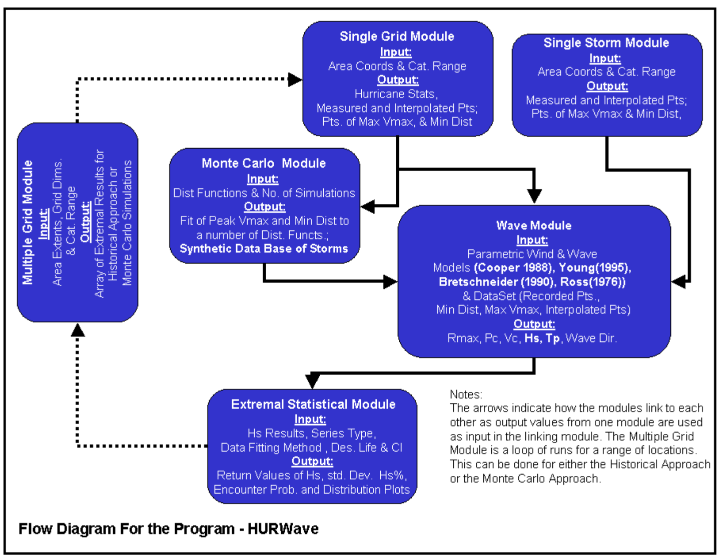
# Appendix B – Hurricane Modelling

## *Appendix B-1 - HurWave Model Description*

A package of Hurricane Parametric Wave Models and Extremal Statistical Analyses by Jamel D. Banton.


HURWave combines the database of the National Oceanic and Atmospheric Administration (NOAA), of hurricane tracks, with wind and wave distribution algorithms to statistically determine deep-water design wave conditions at any location within the Caribbean and the Gulf of Mexico.

The program consists of 6 main modules, namely: The Single Grid Module; The Single Storm Module; The Wave Module; The Extremal Statistical Module; The Monte Carlo Module; and The Multiple Grid Module. These are shown in the flow chart following.



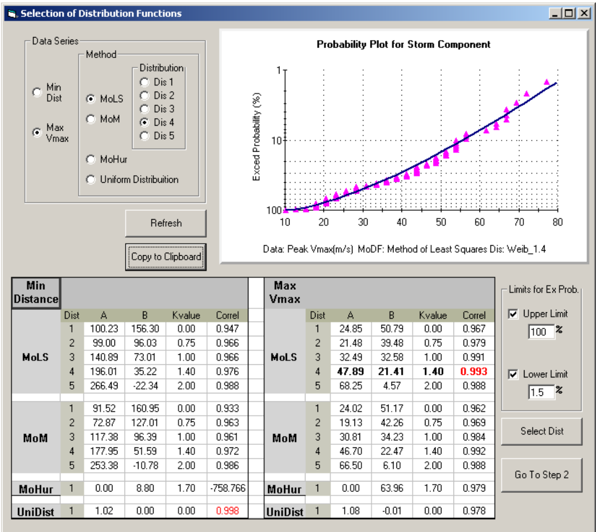
Flow Diagram For the Program - HURWave

**Program Capabilities:**

The NOAA database consists of Atlantic hurricane track positions along with wind and pressure conditions at 6-hour intervals, since the late 19th century. For any specified location within the North Atlantic Basin, HURWave searches this database for Tropical storms and hurricanes that have passed within a specified distance from the point of interest. The program produces a number of statistical descriptions for this result.

**The Monte Carlo Approach**

An alternative method to using just the NOAA database of storms is to generate a much larger synthetic database of storms from the statistical properties of those that actually occurred. This Monte Carlo approach is capable of generating hundreds of probable storms for a particular location, thereby simulating tracks that may occur in the future. This approach was developed from research observations of multi-decadal trends in hurricane frequency and intensity. The research and method are presented in the paper "Long term variability of hurricane trends and a Monte Carlo approach to design" by Smith, Warner and Banton, presented at The International Conference for Coastal Engineering (ICCE 2002).



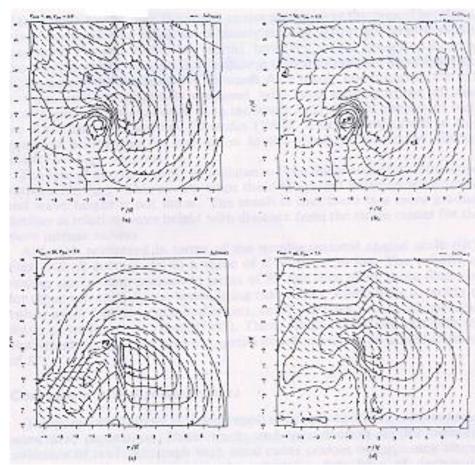

| Min Distance | Dist | A | B | Kvalue | Correl | Max Vmax | Dist | A | B | Kvalue | Correl |
|---|---|---|---|---|---|---|---|---|---|---|---|
| MoLS | 1 | 100.23 | 156.30 | 0.00 | 0.947 | MoLS | 1 | 24.85 | 50.79 | 0.00 | 0.967 |
| | 2 | 99.00 | 96.03 | 0.75 | 0.966 | | 2 | 21.48 | 39.48 | 0.75 | 0.979 |
| | 3 | 140.89 | 73.01 | 1.00 | 0.966 | | 3 | 32.49 | 32.58 | 1.00 | 0.991 |
| | 4 | 196.01 | 35.22 | 1.40 | 0.976 | | 4 | **47.89** | **21.41** | **1.40** | **0.993** |
| | 5 | 266.49 | -22.34 | 2.00 | 0.988 | | 5 | 68.25 | 4.57 | 2.00 | 0.988 |
| MoM | 1 | 91.52 | 160.95 | 0.00 | 0.933 | MoM | 1 | 24.02 | 51.17 | 0.00 | 0.962 |
| | 2 | 72.87 | 127.01 | 0.75 | 0.963 | | 2 | 19.13 | 42.26 | 0.75 | 0.969 |
| | 3 | 117.38 | 96.39 | 1.00 | 0.961 | | 3 | 30.81 | 34.23 | 1.00 | 0.984 |
| | 4 | 177.95 | 51.59 | 1.40 | 0.972 | | 4 | 46.70 | 22.47 | 1.40 | 0.992 |
| | 5 | 253.38 | -10.78 | 2.00 | 0.986 | | 5 | 66.50 | 6.10 | 2.00 | 0.988 |
| MoHur | 1 | 0.00 | 8.80 | 1.70 | -758.766 | MoHur | 1 | 0.00 | 63.96 | 1.70 | 0.979 |
| UniDist | 1 | 1.02 | 0.00 | 0.00 | 0.998 | UniDist | 1 | 1.08 | -0.01 | 0.00 | 0.978 |

## Parametric Wave Modelling

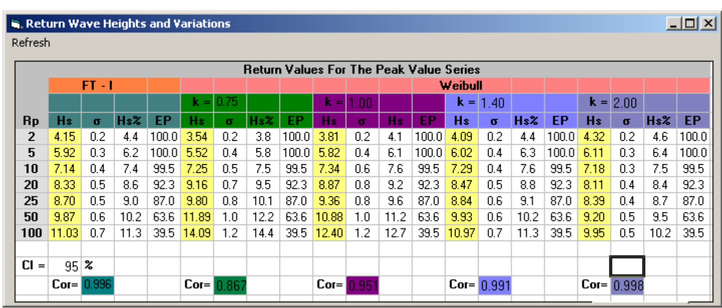
Several widely used wind and wave models are applied to produce a hindcast dataset of hurricane wave conditions at the point in question. These models include Cooper (1988) and Young (1995).

The Cooper model was developed by statistically analysing the output from numerical wind and wave models for 6 Gulf of Mexico hurricanes. The storms used covered a wide cross-section of hurricane conditions.

In the case of Young, he first developed an extensive synthetic database by running a numerical wave prediction model for a wide range of hurricane parameters. The data from these numerical experiments were then used to clarify the wave generation process within hurricanes and further to develop the parametric model suitable for wave prediction in deep water. This model was further calibrated with over 100 measurements made by the GEOSAT satellite.

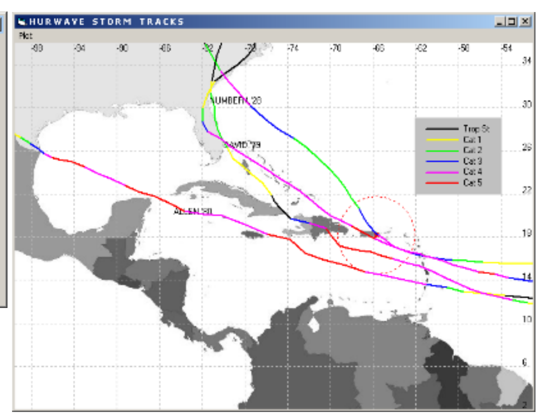


With the results of these models, a range of extremal statistical analyses may be carried out in HURWave. The extremal methods applied are based on work published by Yoshima Goda in 1988 for statistically analysing extreme events such as hurricane waves. Distribution functions such as Weibull and Fischer Tippet (Type I) are fitted to the model results and the best fit chosen. The results include the values for wind, wave and water level conditions for various return periods.




Return Values For The Peak Value Series

| | FT - I | | | | Weibull k = 0.75 | | | | Weibull k = 1.00 | | | | Weibull k = 1.40 | | | | Weibull k = 2.00 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Rp | Hs | σ | Hs% | EP | Hs | σ | Hs% | EP | Hs | σ | Hs | EP | Hs | σ | Hs% | EP | Hs | σ | Hs% | EP |
| 2 | 4.15 | 0.2 | 4.4 | 100.0 | 3.54 | 0.2 | 3.8 | 100.0 | 3.81 | 0.2 | 4.1 | 100.0 | 4.09 | 0.2 | 4.4 | 100.0 | 4.32 | 0.2 | 4.6 | 100.0 |
| 5 | 5.92 | 0.3 | 6.2 | 100.0 | 5.52 | 0.4 | 5.8 | 100.0 | 5.82 | 0.4 | 6.1 | 100.0 | 6.02 | 0.4 | 6.3 | 100.0 | 6.11 | 0.3 | 6.4 | 100.0 |
| 10 | 7.14 | 0.4 | 7.4 | 99.5 | 7.25 | 0.5 | 7.5 | 99.5 | 7.34 | 0.6 | 7.6 | 99.5 | 7.29 | 0.4 | 7.6 | 99.5 | 7.18 | 0.3 | 7.5 | 99.5 |
| 20 | 8.33 | 0.5 | 8.6 | 92.3 | 9.16 | 0.7 | 9.5 | 92.3 | 8.87 | 0.8 | 9.2 | 92.3 | 8.47 | 0.5 | 8.8 | 92.3 | 8.11 | 0.4 | 8.4 | 92.3 |
| 25 | 8.70 | 0.5 | 9.0 | 87.0 | 9.80 | 0.8 | 10.1 | 87.0 | 9.36 | 0.8 | 9.6 | 87.0 | 8.84 | 0.6 | 9.1 | 87.0 | 8.39 | 0.4 | 8.7 | 87.0 |
| 50 | 9.87 | 0.6 | 10.2 | 63.6 | 11.89 | 1.0 | 12.2 | 63.6 | 10.88 | 1.0 | 11.2 | 63.6 | 9.93 | 0.6 | 10.2 | 63.6 | 9.20 | 0.5 | 9.5 | 63.6 |
| 100 | 11.03 | 0.7 | 11.3 | 39.5 | 14.09 | 1.2 | 14.4 | 39.5 | 12.40 | 1.2 | 12.7 | 39.5 | 10.97 | 0.7 | 11.3 | 39.5 | 9.95 | 0.5 | 10.2 | 39.5 |
| CI = | 95 | % | | | | | | | | | | | | | | | | | | |
| | Cor= | 0.996 | | | Cor= | 0.867 | | | Cor= | 0.951 | | | Cor= | 0.991 | | | Cor= | 0.998 | | |



## *Appendix B-2 – Hurricane Modelling Results*

| | | East | North-East | North | North-West | West | South-West |
|---|---|---|---|---|---|---|---|
| Significant Wave Height (metres) | Point 1 | 1.41 | 1.57 | 1.69 | 1.88 | 1.80 | 1.60 |
| | Point 2 | 1.46 | 1.60 | 1.67 | 1.85 | 1.80 | 1.63 |
| | Point 3 | 1.52 | 1.72 | 1.90 | 2.08 | 2.02 | 1.79 |
| | Point 4 | 1.36 | 1.48 | 1.56 | 1.69 | 1.65 | 1.53 |
| | Point 5 | 1.08 | 1.19 | 1.56 | 1.68 | 1.69 | 1.59 |
| | Deep point | 1.53 | 1.76 | 1.98 | 2.36 | 2.29 | 1.92 |
| | Stem point | 1.51 | 1.73 | 1.92 | 2.21 | 2.14 | 1.84 |
| Mean Wave Period (t02) (seconds) | Point 1 | 12.04 | 11.52 | 7.66 | 10.34 | 10.49 | 9.45 |
| | Point 2 | 12.16 | 11.70 | 7.62 | 10.36 | 10.46 | 9.49 |
| | Point 3 | 12.17 | 11.84 | 7.58 | 10.37 | 10.42 | 9.47 |
| | Point 4 | 12.19 | 11.86 | 7.78 | 10.44 | 10.39 | 9.55 |
| | Point 5 | 12.10 | 11.38 | 7.81 | 10.39 | 10.25 | 9.53 |
| | Deep point | 12.09 | 11.61 | 7.44 | 10.30 | 10.39 | 9.40 |
| | Stem point | 12.12 | 11.66 | 7.50 | 10.33 | 10.43 | 9.41 |
| Mean Wave Direction (degrees) | Point 1 | 328.3 | 329.1 | 327.1 | 316.5 | 303.9 | 300.6 |
| | Point 2 | 325.3 | 326.0 | 322.6 | 309.7 | 299.2 | 294.4 |
| | Point 3 | 325.7 | 326.7 | 320.3 | 308.3 | 293.2 | 285.4 |
| | Point 4 | 318.2 | 319.4 | 317.5 | 307.2 | 293.4 | 287.0 |
| | Point 5 | 306.5 | 308.9 | 313.2 | 303.1 | 289.3 | 282.4 |
| | Deep point | 331.9 | 332.3 | 326.8 | 312.5 | 295.1 | 286.5 |
| | Stem point | 328.7 | 329.5 | 324.6 | 310.4 | 296.0 | 289.1 |
| Surface Elevation (metres) | Point 1 | 0.85 | 0.96 | 1.07 | 1.14 | 1.10 | 0.96 |
| | Point 2 | 0.85 | 0.95 | 1.07 | 1.14 | 1.10 | 0.96 |
| | Point 3 | 0.84 | 0.94 | 1.05 | 1.10 | 1.08 | 0.95 |
| | Point 4 | 0.84 | 0.94 | 1.07 | 1.12 | 1.10 | 0.97 |
| | Point 5 | 0.84 | 0.93 | 1.05 | 1.13 | 1.12 | 0.98 |
| | Deep point | 0.85 | 0.94 | 1.04 | 1.09 | 1.06 | 0.94 |
| | Stem point | 0.85 | 0.94 | 1.05 | 1.11 | 1.07 | 0.95 |