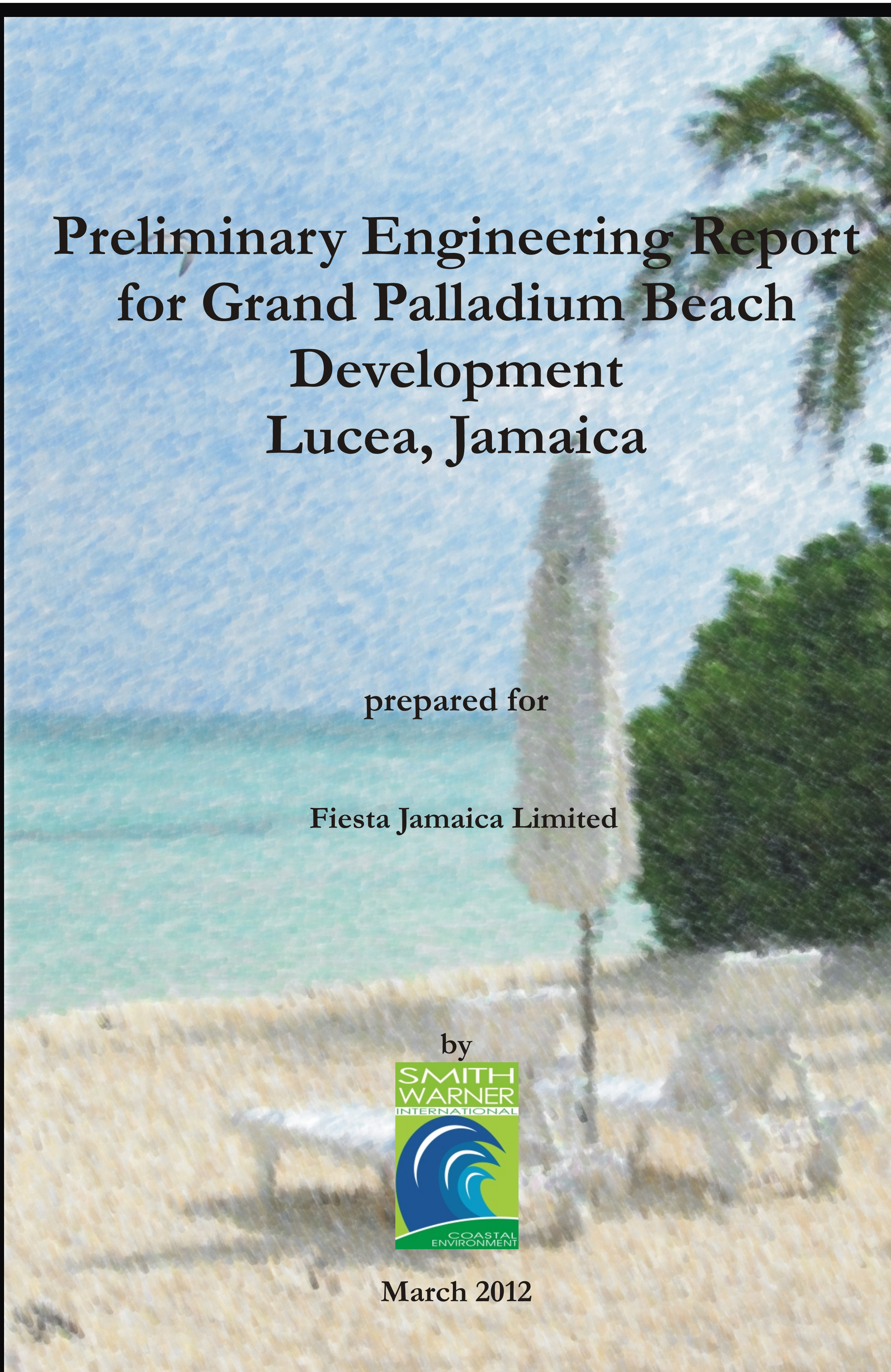

# Preliminary Engineering Report for Grand Palladium Beach Development
# Lucea, Jamaica

prepared for

Fiesta Jamaica Limited

by

SMITH WARNER INTERNATIONAL

COASTAL ENVIRONMENT

March 2012

**CLIENT`S REPRESENTATIVE:** **Mr. Dimitris Kosvogiannis**

**TITLE**: **Grand Palladium Beach Development – Lucea, Jamaica**

**ABSTRACT:**

**Phase I – Coastal Process Investigations and Preliminary Engineering Beach Design for New Royal Suites Expansion**

**REFERENCES:**

| **AUTHOR** | | **DATE** | **REVIEW** | | **APPROVED BY** | |
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- **FINAL PRELIMINARY ENGINEERING**

**PROJECT IDENTIFICATION: Grand Palladium Beach Development – Lucea, Jamaica**

## Contents

# 1. Introduction

## *1.1 Background Information*

The Grand Palladium Resort is situated on the north cost of Jamaica, on the eastern side of Lucea Harbour. The existing shoreline at the Grand Palladium (Figure1.1) is a mix of natural and man-made beaches. The man-made beaches are primarily along the northern stretches of the resort's shoreline and are retained with the aid of groynes and breakwaters (Figure 1.2). The main natural beach is within a cove at the southern end of the property. Development plans for the site include the addition of another 805 rooms, some of which will fall into a more upscale grouping known as the Royal Suites. Part of the plan for these suites includes the construction of two more pocket beaches and a boardwalk connecting the existing cove beach to these new beach areas.

Currently, a narrow beach exists for each of these areas (Figure 1.3) however the foreshore is quite rocky and/or covered with seagrass. The challenge for creating a proper swimming beach at each of these locations will be to have a wider and deeper beach coupled with a sandy foreshore that would allow guests to walk out and swim.


**Figure1.1 Project area showing existing beaches and proposed pocket beaches**

**Figure 1.2 Man-made beach on the northern resort shoreline (left) and natural beach cove on the southern resort shoreline (right)**

**Figure 1.3 Existing pocket beaches to be developed along the resort's southern shoreline**

## *1.2 Scope of Works*

A phased design approach was adopted for the beach development work. Phase 1(described in this report) describes the scientific background and analyses that have been carried out to the point where a concept for beach creation can be developed and preliminary costs prepared. Phase 2 will proceed to the final engineering design and construction documents to refine and quantify the various components of the project.

The beach enhancement concept will allow for the development of a wide, sandy beach, with a sandy seabed out to a water depth of 1.5 - 1.7m. Beyond this, it is anticipated that guests will start to swim. The investigations will examine the need, or not, for any structures to retain the beach. The primary challenge for this design will be to ensure that a wider, stable beach is created, with a sandy foreshore suitable for swimming. This design objective may be achieved through the excavation of existing foreshore, the nourishment of existing beach and perhaps the implementation of retaining structures.

The coastal process investigations presented in Phase 1, along with the environmental mitigation plan outlining strategies and donor areas for relocation of seagrass, will be used to facilitate the NEPA approval process. The report is intended to:

- Identify the optimum approach to achieving the design objectives;
- Identify potential impacts on the marine environment and surrounding coastline;
- Set out a mitigation strategy and environmental management plan dealing with the relocation of any sensitive benthic organisms such as seagrass and/or corals;
- Present the developed concepts in an Engineering Report; and
- Provide preliminary cost estimates.

The scope of works agreed to between Fiesta Jamaica Ltd. and Smith Warner International Ltd. (SWIL) to achieve the design objectives of Phase I include:

i. Carry out a rapid bathymetric and beach profile survey in the area of the two beach zones.

ii. Define the operational (day-to-day) wave climate. This will be obtained from the National Oceanic and Atmospheric Administration (NOAA) Wave Watch III model, or from UK Met Office data. This data will be modeled from offshore using the DHI's spectral wave model MIKE 21. This stage of modeling will output the wave climate in deep water offshore the project site. This wave climate will include day-to-day operational conditions as well as winter swell events.

iii. Define the wave climate attributable to hurricanes. The National Hurricane Centre archived database of hurricanes that have occurred since 1900 will be used to develop appropriate statistics of return period wave heights that the beaches and shoreline structures along the north-west coast of Jamaica will be exposed to. An in-house hurricane wave prediction computer model, HurWave, will be used for this evaluation. Design conditions will be developed for the 1 in 50 year hurricane.

iv. The deep water wave climates obtained above (operational and extreme) will be input to MIKE21 to give the appropriate wave climate at the project shoreline and at points adjacent to the property.

v. An analysis of storm surge will be undertaken to define flooding levels. This information will be used in the design of the structures proposed for beach creation and stabilization, and as well may be used to assess the back of beach wave interactions.

vi. The wave climate at the site will then be used to establish baseline alongshore and cross-shore sediment transport patterns and rates of sediment movement for both event-driven short periods (swell events) as well as for typical annual long term variations. This analysis is expected to provide insight into some of the seasonal beach fluctuations that can occur. The MIKE 21 model will then be used to give a 2-dimensional (plan) view of the sediment transport characteristics.

vii. Detailed mapping of the benthic features in the nearshore environment, particularly around the proposed location of the works, will be carried out. This mapping will help to dictate the mitigation strategies that will have to be adopted to facilitate the works. This may include the relocation of sensitive organisms. If it does become necessary to relocate sensitive benthic organisms, target locations will also be identified and an environmental management plan will be prepared giving relocation strategies, etc.

viii. The DHI model MIKE 21 will then be run in a coupled mode to incorporate the effects of wave and tidally driven currents on the local sediment transport regime. The proposed structures and foreshore deepening will be modeled and their performance in response to operational and swell wave conditions noted. This process of modeling will look particularly at the following:

a. Given the incident wave climate, a range of alignments and dimensions of any proposed structures will be investigated to provide the optimum degree of sheltering for the proposed swimming and created beach areas. The deepening of the foreshore area in front of each beach will also be investigated.

b. The performance of the proposed structures as wave attenuators and their ability to hold sand in place will be evaluated under varying wave conditions. These will include the daily Trade Wind wave conditions, typical seasonal swell events from the northwest and representative hurricane events.

# 2. Site Characteristics and Field Investigations

## *2.1 Site Location*

The proposed site for the two new beach coves is located on the eastern side of Lucea Harbour in northern Hanover, Jamaica. The project site (Figure 2.1) is bordered by the Caribbean Sea on its eastern, northern and western boundaries and by the North Coast Highway on its southern boundary.

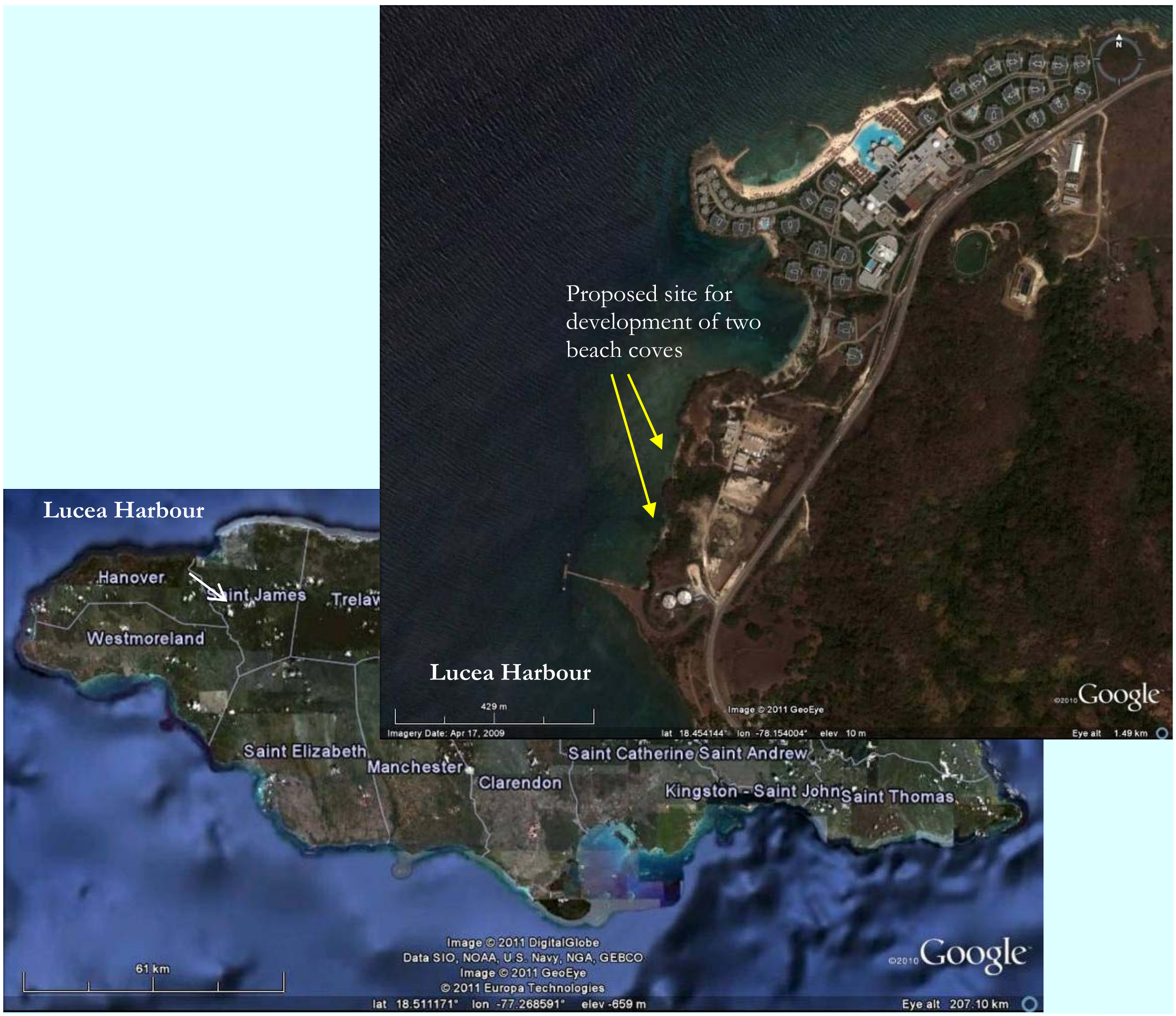



**Figure 2.1 Project location**

## *2.2 Site Characteristics*

The site has varying topographic and shoreline characteristics that have developed over time due to man-made and wave activity, as well as other geologic and hydrologic processes. The most dominant features include small and gently sloping pocket beaches connected to sections of steep rocky shoreline (Figure 2.2). The site is bordered by a reef just offshore the project site, dense vegetation and cliffs to the landward side, the main natural beach cove of the Grand Palladium to the north and an old Molasses Pier to the south.

The site is protected by an extensive reef system extending over a distance of 130-200m from the shoreline. The nearshore area has several seagrass patches and is very calm with virtually no wave activity during day-to-day conditions. In the winter period, when swells occur, the wave climate becomes more energetic. The shoreline has two small pocket beaches backed by steep cliffs. These pocket beaches have fine to coarse sand on them with minor build-ups of debris consisting mainly of bamboo and tree trunks. Access to this location is difficult due to the high cliffs and dense vegetation covering the entire backshore of the project site.

## *2.3 Bathymetry*

Detailed bathymetric and beach profile surveys were conducted and merged with previously surveyed topographic contours to produce a project base map (Figure 2.2). Water depths were collected using an Odom Echotrac sounding system, while spatial positions were recorded with a Trimble GPS. Beach profiles were surveyed approximately every 30m perpendicular to the shoreline. This data was used to assist in the computer modeling of waves from deep water into the nearshore regions, and in evaluating the degree of sheltering of the beach for the various options considered.

The bathymetric plot showed that the reef system at the project site has depths ranging from 0.5-2m over a distance of 200m. Beyond the reefs, the seabed deepens to 10m or more.

The natural beach cove at the northern boundary of the project site includes a sand channel with depths of 3-5m entering the cove and connecting to adjacent reef features on both headlands to the north and south.

The shoreline along the southern boundary of the property is steeper. This boundary is located within Lucea Bay, which has a relatively deep channel ranging from 8-10m in depth, with a 15-20m deep entrance.


**Figure 2.2** **Shoreline at the project site**

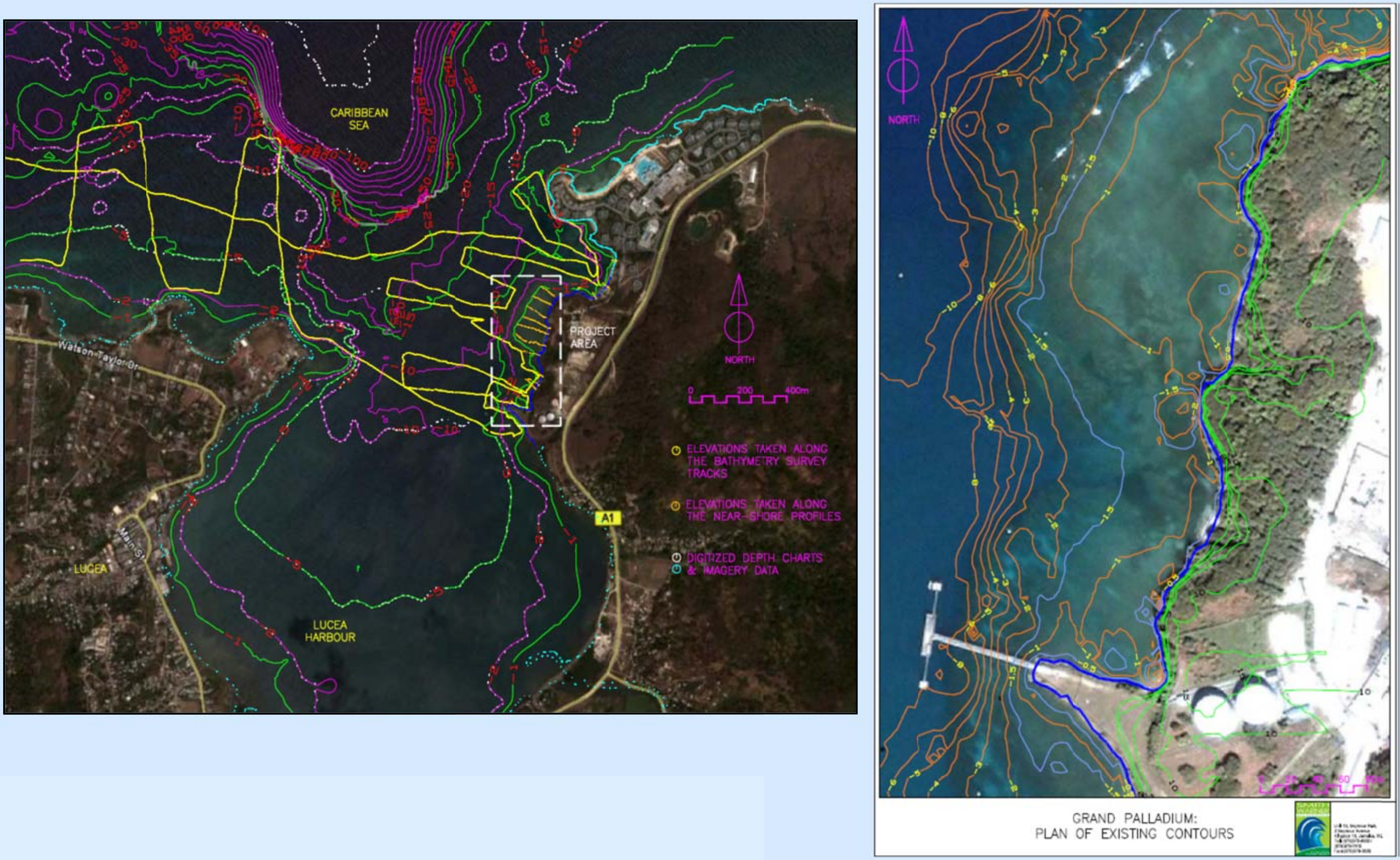



**Figure 2.3** **Merged bathymetric data (left) combined with beach profiles to form seabed and land contours (right) at the project site**

## *2.4 Sand Sample Analysis*

Sediment samples were collected at seven different locations along the project shoreline both in the swash zone and the higher berm of the beach (Figure 2.4). The 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.

Of particular interest were two sediment samples (3 and 6) located on each of the beach coves to be developed; these were chosen for a detailed analysis of grain size distribution and composition of sediments (Figure 2.5). The results of this analysis (Appendix A) indicated that both samples were comprised of gravel and sand with distinctive coarse and fine fractions and no or negligible amounts of silt. Sample 3 was classified as poorly sorted gravelly sand (medium to coarse) composed of 18% gravel and 82% sand with no silt. Sample 6 is moderately well sorted slightly gravelly sand (very fine and fine to medium) composed of 99.5% sand, 0.4%gravel and 1% silt. The larger portion of gravels found in Sample 3 indicates coral fragments, molluscs and lithics.

This information was used in the beach response modeling and as guidance for the grain size distribution to be used for the nourishment of the proposed beach coves to be developed.


**Figure 2.4 Sand sampling locations at the project site**

**Table 2-1 Sediment Sample Sieve Analysis Results**

| Name | Sand Type | D10 (mm) | D30 (mm) | D50 (mm) | D60 (mm) | % Gravel | % Sand | % Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|
| Main Beach Sample #2 | Poorly Graded Sand | 0.112 | 0.179 | 0.23 | 0.27 | 0.1 | 99.6 | 0.3 | |
| Main Beach Sample #3 | | 0.154 | 0.214 | 0.3 | 0.35 | 0.0 | 99.9 | 0.1 | |
| North Beach Cove Sample 1 | | 0.395 | 0.522 | 0.65 | 0.72 | 1.7 | 98.0 | 0.3 | |
| North Beach Cove Sample2 | | 0.271 | 0.519 | 0.71 | 0.82 | 3.2 | 96.6 | 0.1 | |
| North Beach Cove Sample3 | | 0.28 | 0.36 | 0.47 | 0.6 | 18.0 | 82.0 | 0.0 | |
| South Beach Cove Sample5 | | 0.159 | 0.287 | 0.49 | 0.60 | 1.9 | 97.7 | 0.4 | |
| South Beach Cove Sample6 | | 0.1 | 0.16 | 0.18 | 0.21 | 0.4 | 99.5 | 0.1 | |

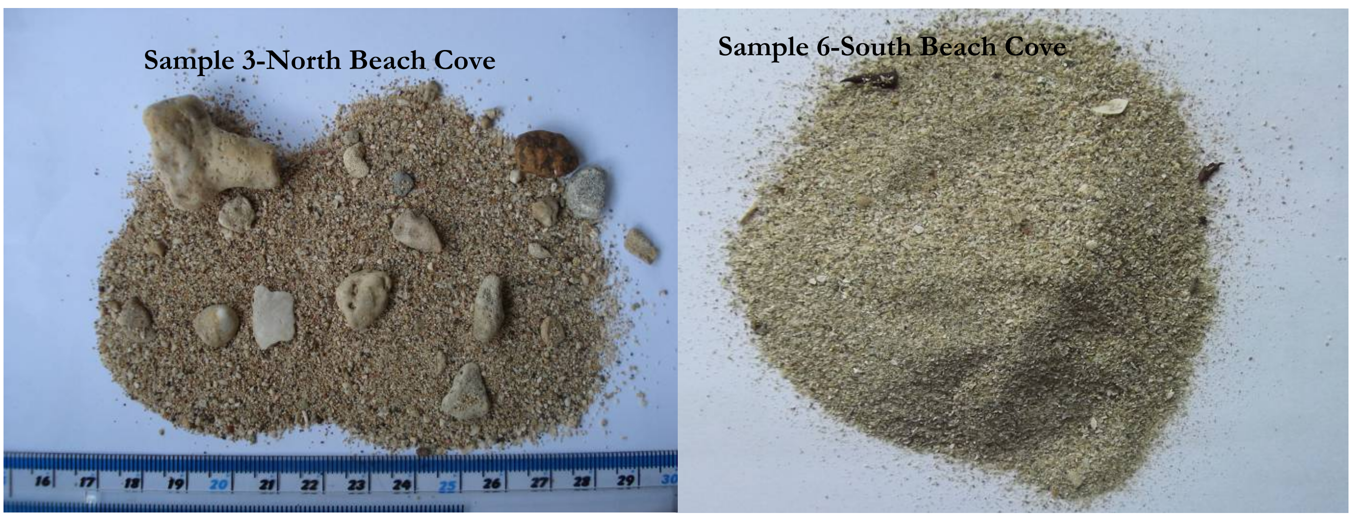



**Figure 2.5 Sample 3 and 6**

## *2.5 Benthic Survey*

To assess possible impacts of any proposed works on the marine environment, a benthic survey was conducted in the nearshore area of the project site. In order to focus this aspect of the works, the survey was carried out after a preferred concept was identified so that site-specific impacts could be investigated. The mapping helped to dictate mitigation strategies that would have to be adopted to facilitate the works as well as in the preparation of an environmental management plan in the event that sensitive benthic organisms have to be relocated. The results of the benthic survey are presented in Section 7 of this report.

# 3. Wave Climate Analysis

Once the data collection was complete, design 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.

The north coast of Jamaica is exposed to two very different 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, which is defined by occasional hurricanes that generate much higher waves. The analyses of both of these wave climates are described in this section.

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 within the beach system and are responsible for long-term morphological changes. For coastal engineering design, 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 selection of an armour stone size that would be required for a coastal structure or the determination of design wave forces that may occur as a result of extreme waves. The stability of beach nourishment would, however, not be designed for such wave conditions. 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 developer is willing to assume. Normally a 50-year return period represents an acceptable balance between capital investment and maintenance costs.

## *3.1 Operational Wave Climate and Transformation to the Nearshore*

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 sector. As such, the north coast of Jamaica 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 small.

The deep water operational wave climate was established using the global wave model WAVE WATCH 3 (WW3) developed by NOAA. The WW3 model archives wave parameters including wave height, period and direction as well as the wind speed and direction every three hours from July 01 1999 to November 31 2007 giving a total of 24,000 data points per parameter. This time series of wave conditions was extracted for a node located north of Lucea Harbour.

Figure 3.1 shows the wave height distribution and the location of the node (node 6) that was selected for the project. Note the majority of the waves come from the east sector, as dictated by the Trade Wind patterns.

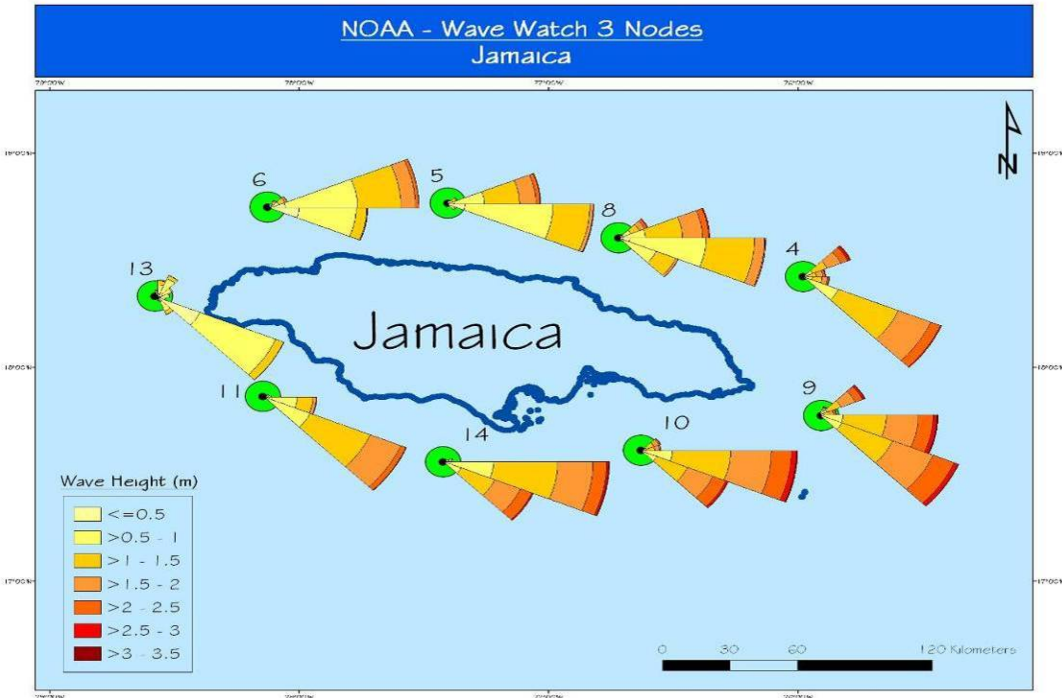



**Figure 3.1 NOAA Wave Watch 3 nodes in the vicinity of Jamaica**

The WW3 model is usually applied on spatial scales (grid increments) larger than 1-10km and outside the surf zone. As a result, the model is not at a sufficiently detailed scale to provide accurate nearshore wave data along the north coast of the island. The nearshore wave climate was therefore developed using a spectral wave model MIKE 21 SW to simulate waves as they approach from the east, north and west and wrap around the island to reach the project site. A description of the MIKE 21 numerical model capabilities is presented in Appendix B.

The seven years of wave data (1999-2007) obtained from the node 6 were categorized using a tri-variate frequency analysis of wave height, period and direction. This frequency analysis resulted in 300 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 seven year database. The MIKE 21 spectral wave model was run in a semi-stationary mode, with time-varying inputs of the previously binned deep water waves along the boundaries of the model domain. The model was set up on a flexible mesh to represent seabed depths from offshore to nearshore of the project site to a sufficient degree of detail.

Smaller mesh elements represent higher resolution in the representation of the bathymetry. Figure 3.2 shows the model domain, mesh and boundaries. The resulting data time-series, shown in Figure 3.3, were extracted at seven nodes in the nearshore of the project site, giving a representation of the annual wave climate. The nearshore annual wave climate generated at each node location was further used as input into the sediment transport model LitDrift, discussed in Section 4 of this report, to understand the long-term sediment transport at the project site. Results (Figure 3.4), giving the percentage of the annual wave climate, indicate that the site is very sheltered with waves less than 0.25m in height occurring 80% of the time.

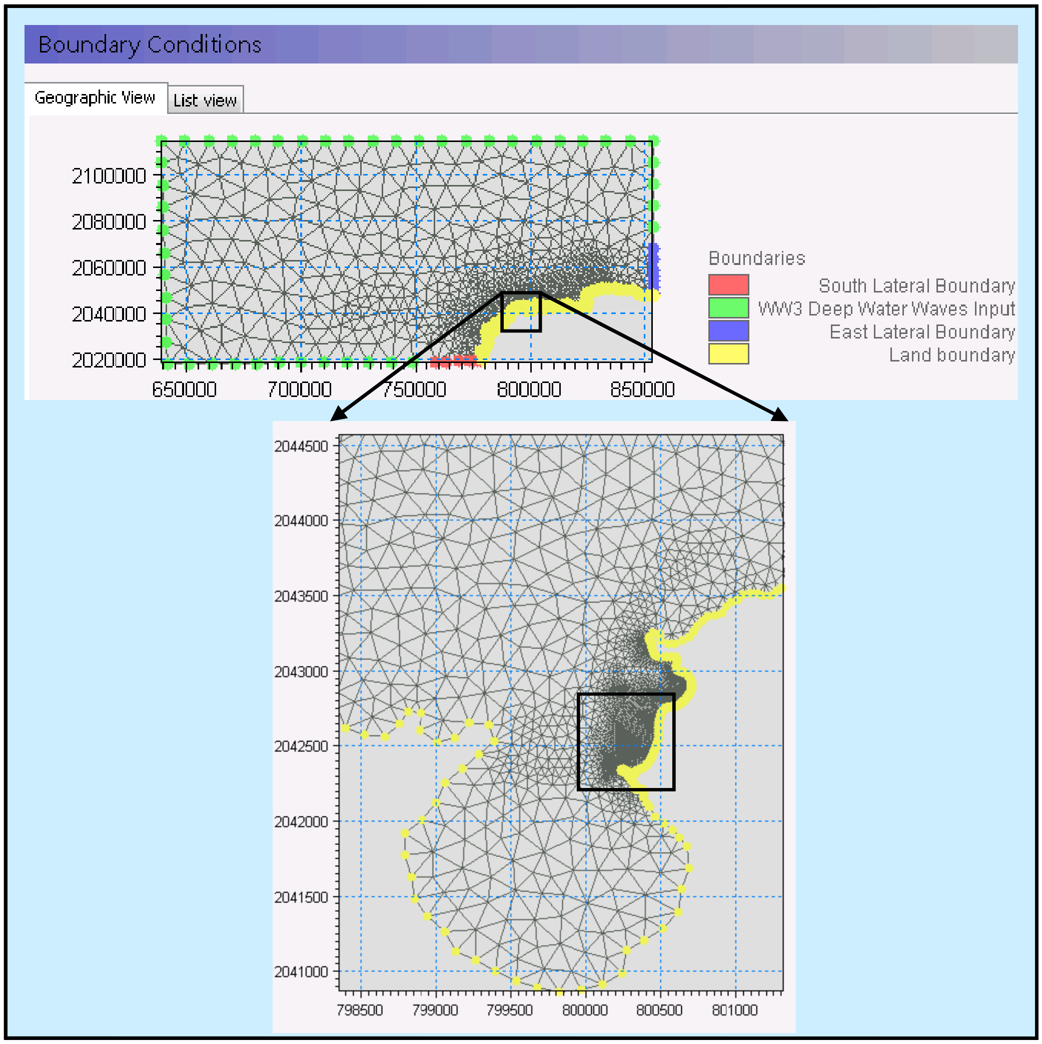
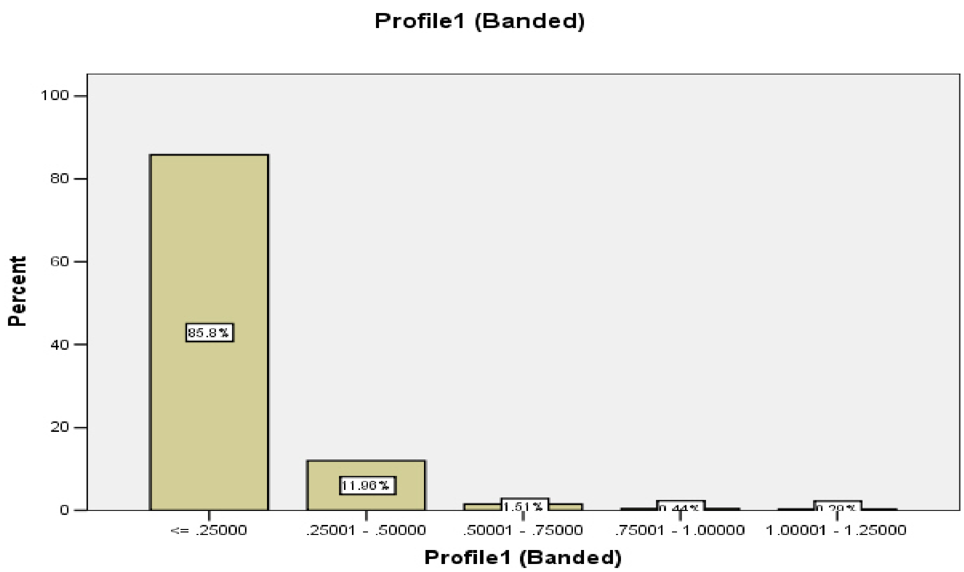



**Figure 3.2** **Computational mesh and boundaries over the model domain**


**Figure 3.3** **Resulting annual wave climate at the location of the selected nearshore nodes**




Wave height range (m)

**Figure 3.4** **Percentage of annual wave climate at project site**

## *3.2 Hurricane Wave Climate*

The first step in determining hurricane storm surge values for this shoreline involved the use of an in-house hurricane wave and statistical package, HurWave, to develop deep water wave heights for different design return periods. Details of HurWave are presented in Appendix B. The second step involved the use of the MIKE 21 Spectral Wave and Hydrodynamic modules, operating in a coupled mode, to transform hurricane wave conditions from deep water into the nearshore, and to give wave heights and storm surge levels at the site.

Jamaica is exposed to hurricane activity between June and November each year. During hurricanes, coastal areas (in particular those near the shoreline or low-lying lands) are exposed to high waves and increased water levels. The high waves are caused by the high wind speeds associated with the hurricane. The water level increase is caused by reduced atmospheric pressure in the central part of the hurricane (Inverse Barometric Pressure Rise – IBR) and by the wind stress acting on the water surface, causing the water level to be increased at the shoreline. In the Caribbean, these extreme conditions must be used to determine the design requirements for any coastal structure or beach enhancement works.

An analysis of historical hurricane data was carried out to determine these design conditions using an in-house computer program, HurWave. As part of the analysis:

- Historical hurricane information from the National Hurricane Center (NHC) database was reviewed (for storms between 1900 and 2007);
- Parametric wave prediction models were used to estimate wave heights and water levels in deep-water;

- Wave and flood hazards were evaluated, based on the statistical distribution of historical data.

All hurricanes passing within a 300km radius of the proposed site were selected from the NHC's database. Since the year 1900, 106 tropical storms and hurricanes have passed within this radius. This number can be broken down according to the categories described by the Saffir Simpson scale as shown in Figure 3.5. The figure shows that the area is more frequently hit by tropical storms, however it is also affected by major hurricanes. Of all the storms satisfying the search radius criterion, three storms that became Category 5 hurricanes at some point along their trajectory have passed within this radius: Allen (1980), Ivan (2004) and Emily (2005).

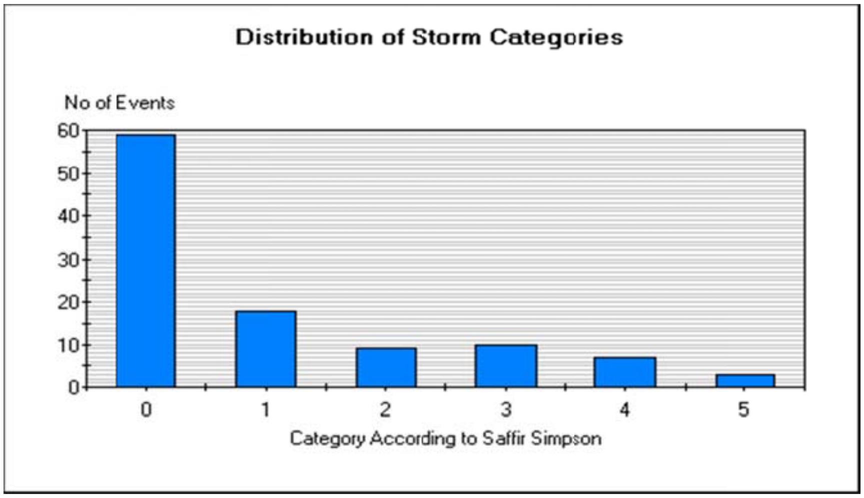



**Figure 3.5 Distribution of storms passing within 300km of the project site**

Using a parametric hurricane wave model and the historical hurricane data, a data series of deep water wave heights was computed for different directional sectors. The statistical method of Yoshima Goda (1990) was applied to each of these series and the best fit distribution was used to estimate various return period wave conditions, which are listed in Table 3-1, with the results for the 1 in 50 year event highlighted. Return periods express the probability in years, of at least one occurrence of a particular event. A 50-year return period is commonly used for the design of coastal structures in the Caribbean, as it provides a reasonable balance between capital costs and future maintenance requirements

Water levels increase during the passage of a hurricane due to inverse barometric pressure rise (IBR), which is caused by the low atmospheric pressure in the centre of the hurricane. As with the wave heights, water level was computed from each historical storm and the data fitted to various statistical distributions. Because of the non-directionality of the water level increase phenomenon, the analysis was not carried out on a directional basis.

The best-fit distribution was selected based on correlation and goodness-of-fit to the most extreme values. In addition to the extreme eventualities, it is important to consider the expected long-term trends from global sea level rise. Global Sea Level Rise (GSLR) has been predicted by scientists according to current rates of sea level rise and forecasting of the effects of global warming on the thermal expansion of the seas and the melting of glaciers and polar ice caps. The increase in sea level over the next century (until 2100) is estimated in the *Intergovernmental Panel on Climate Change –*

*Third Assessment Report* to range between 90 and 880mm, with a central value of approximately 480mm. This corresponds to approximately 0.25m over the next 50 years, which is the assumed design life of the project.

**Table 3-1 Wave Height-Hs (m), Peak Period-Tp(s) and Wind Speed-Vm(m/s) estimated for two return periods and the six main directional sectors**

| Directional Sector | Wave Parameter | Return Period (years) | |
|---|---|---|---|
| | | **50** | **100** |
| North | Hs (m) | 8.37 | 9.99 |
| | Tp (s) | 12.59 | 14.07 |
| | Vm (m/s) | 26.54 | 29.93 |
| North-east | Hs (m) | 11.40 | 13.71 |
| | Tp (s) | 15.29 | 17.17 |
| | Vm (m/s) | 32.87 | 37.69 |
| East | Hs (m) | 12.42 | 14.32 |
| | Tp (s) | 16.13 | 17.65 |
| | Vm (m/s) | 35.00 | 38.96 |
| South-west | Hs (m) | 7.64 | 9.07 |
| | Tp (s) | 11.89 | 13.24 |
| | Vm (m/s) | 25.02 | 28.01 |
| West | Hs (m) | 6.76 | 8.05 |
| | Tp (s) | 11.00 | 12.28 |
| | Vm (m/s) | 23.18 | 25.88 |
| North-west | Hs (m) | 8.20 | 9.81 |
| | Tp (s) | 12.42 | 13.91 |
| | Vm (m/s) | 26.19 | 29.55 |

Tidal variations were taken into account and, based on the tide measurements made adjacent to the project site; high tide above MSL was determined to be 0.25m. These effects were added to the IBR to produce final deep water levels for the various year return period storms, as shown in Table 3-2.

**Table 3-2 Values of basic storm surge components**

| Return Period (years) | IBR (m) | HAT (m) | GSLR (m) | Final Water Level (m) |
|---|---|---|---|---|
| 25 | 0.30 | 0.25 | 0.25 | 0.80 |
| 50 | 0.39 | 0.25 | 0.25 | 0.89 |
| 100 | 0.48 | 0.25 | 0.25 | 0.98 |
| 150 | 0.53 | 0.25 | 0.25 | 1.03 |
| 200 | 0.57 | 0.25 | 0.25 | 1.07 |

The deep water conditions were transformed to the nearshore regions and up to the project site using MIKE 21. A computational mesh extending to deep water (greater than 200m) was used for the transformation of the hurricane waves. The 50-year return period deep water waves and water levels were applied to the boundary of the model and transformed to the nearshore from the six main directional sectors with the most impact on the site. A constant wind field (magnitude and direction) was also applied over the entire model domain. The wind direction in a hurricane changes rapidly, therefore, the worst-case scenario for wind direction was used, with winds approaching from the same dominant direction as the waves.

The results for the six directions were then analysed to determine the envelope of maximum values. Figure 3.6 shows the resulting maximum wave heights and storm surge values over the project shoreline for the 50-year hurricane condition computed from all directional sectors. The results demonstrate that maximum waves from all directional sectors can be expected to reach heights ranging from 0.4m-1.5m in the vicinity of the project site. In the same way it was found that storm surge is expected to range from 1.4m-1.6m along the project site, *excluding wave run-up*.

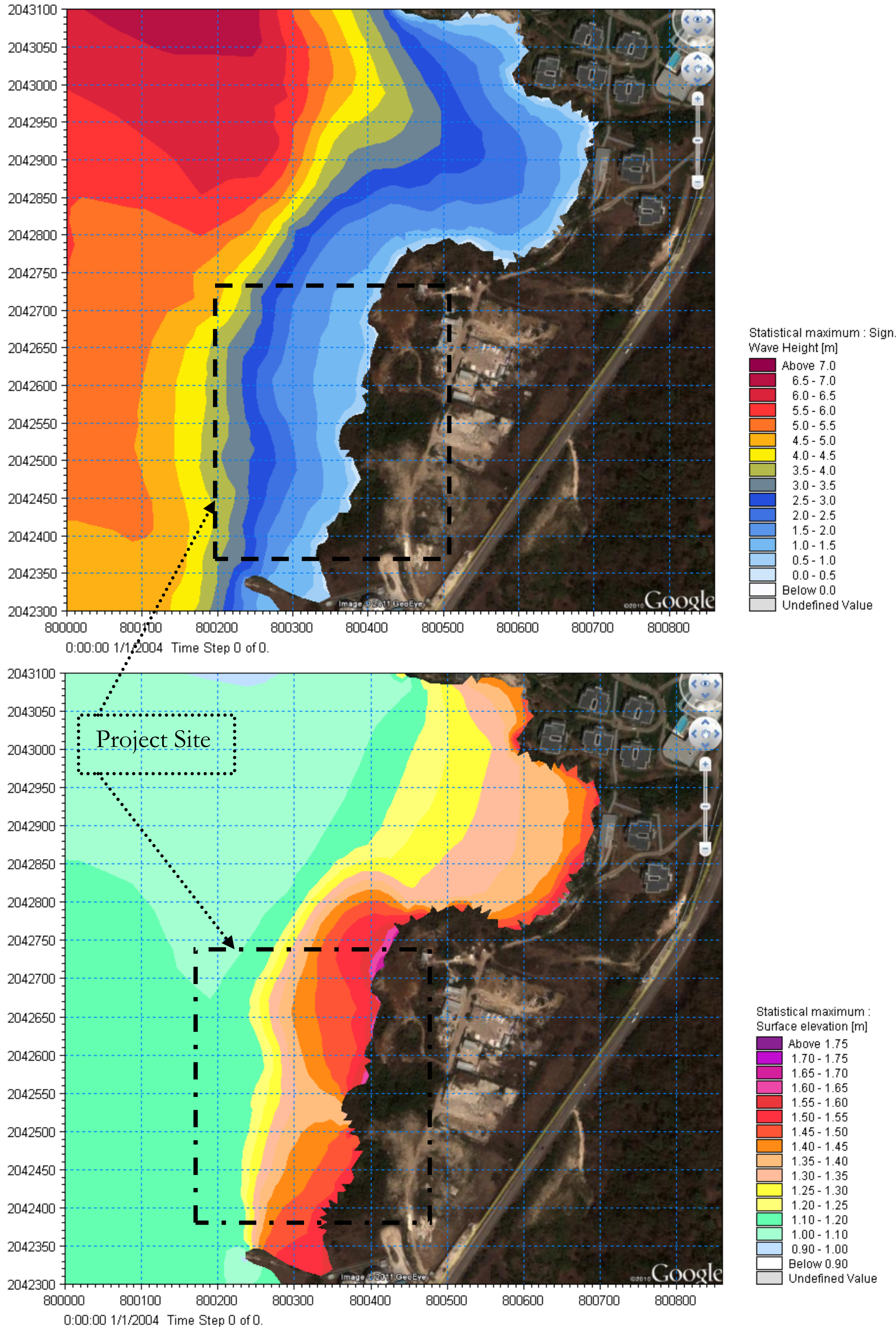



**Figure 3.6 Maximum wave heights and storm surge for the 1 in 50 year event from all directional sectors**

# 4. Sediment Transport Regime

## *4.1 Long-term Sediment Transport: 1-Dimensional Analysis*

Waves commonly approach the coastline at an angle, driving currents that have the potential to transport sediment in an alongshore or cross-shore direction. On most coastlines, waves reach the beach from different quadrants, producing day-to-day and seasonal fluctuations in transport magnitude and direction. In order to design beach enhancement works in harmony with the forces of nature, and that do not cause significant downdrift impacts, it is necessary to have a thorough understanding of the sediment transport regime at the site. This section describes the beach morphology and sediment transport characteristics at the Grand Palladium and provides insights into the littoral system at the project site.

Sediment transport characteristics were determined using the LITDRIFT module from the LITPACK suite developed by DHI. The model was used to track the movement of sediment across a coastal profile, calculate the amount of deposition or erosion over the domain and investigate the rate and distribution of alongshore sediment movement at the project shoreline. This analysis examines only the alongshore movement of sediment and does not consider onshore or offshore movement. Further, the profile is assumed to remain constant throughout the simulation; there is no morphology or response of the beach to wave conditions. Despite these limitations, LITDRIFT does provide valuable insight into the coastal processes occurring at the site.

The following points describe the different model input requirements in detail:

- *Cross-shore Profile and Bathymetry:* Using the bathymetric and beach profile data, seven profile lines, extending from about +2m above mean sea level (MSL) to a water depth of 5-10m were created along the north, central and south section of the project shoreline. Figure 3.3 presented in the preceding section shows the node locations and profile orientations selected for the sediment transport analysis. The profiles were oriented perpendicular to the shoreline and water depth contours in the bay.

- *Wave Climate:* The operational wave climates at the seven nearshore nodes described in Section 3.1 and shown in Figure 3.3 were used as input to the model.

- *Sediment Properties:* Sediment characteristics for the erodable sections of the profile were obtained from the sediment samples taken at the project site. The sediment properties are described in Section 2.4 and listed in Table 2-1. For each cross-section, the sediment characteristics of the closest sample to the profile location (i.e., fall velocity, geometrical spreading, bottom roughness, and several grain diameters - $D_{16}$, $D_{50}$, $D_{60}$, $D_{84}$ and $D_{90}$) were used as representative values for that profile.

Using these input conditions, LITDRIFT was run at the seven profile locations. The resulting distribution of alongshore sediment transport across the proposed northern beach cove (Profile 4) and southern beach cove (Profile 6) is given Figure 4.1, while other profiles are presented in Appendix C. The results indicate the following littoral zone characteristics:

- The main sediment transport zone is very dependent on the location of the existing reef. Most of the transport occurs within 10-25m of the shoreline, shoreward of the breaker zone, and beyond the reef, which is the region where waves dissipate most of their energy and where the alongshore currents are strongest.

- Along Profile 4 sediment transport also occurs seaward of the reef, 200m away from the shoreline. The presence of the reef induces wave breaking and dissipation of wave energy, indicating a potential pathway for sediment seaward of the existing reef.
- Transport to the south (blue) is dominant compared to the transport to the north (green), which is almost non-existent, resulting in a net sediment drift (negative) to the south (red).
- The computed annual potential sediment transport rates along the project site are plotted in Figure 4.2. It was found that rates of transport, along the selected profiles were relatively small (varying from 3,000-8,000$m^3$) to the south with almost no sediment transport to the north, resulting in a net potential transport to the south of 8,600$m^3$/year in the centre line of the proposed beach cove 1 (north) and 5,400$m^3$/year in the centre line of the proposed beach cove 2 (south).

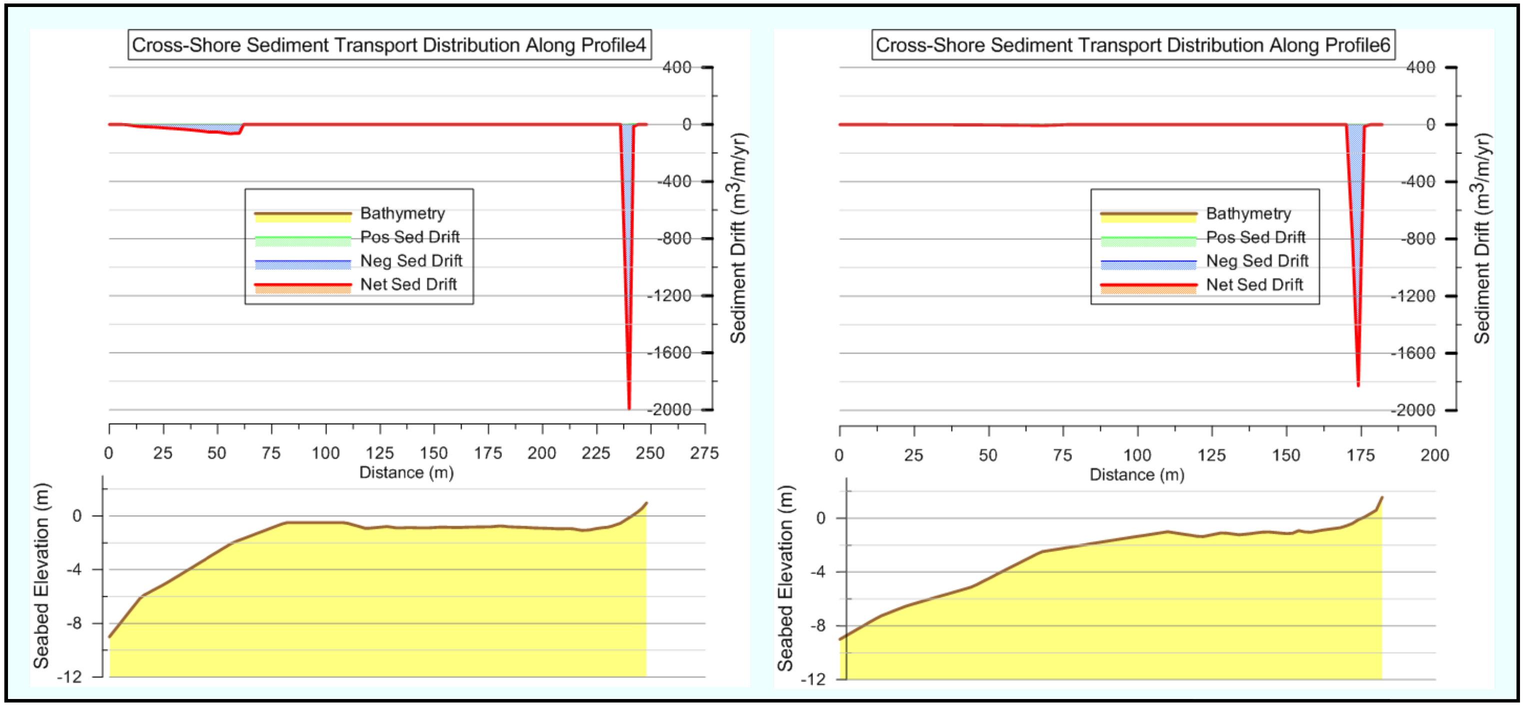



**Figure 4.1 Cross-shore sediment transport distribution along the central line of the proposed northern beach cove (left) and southern beach cove (right)**

From a morphological perspective, the difference in transport capacity from one profile to another is of particular importance. The results of the alongshore transport modeling, providing the accumulated sediment transport and direction over the year, is shown in Figure 4.2 for each profile. The results indicate the following littoral zone characteristics:

- The alongshore net transport is predominant to the south. The overall alongshore sediment transport rates are quite low and reflected by existing shoreline features, which show that the sand is produced locally by the existing reef system while the rocky headlands help contain the sand within each bay.
- In the northern section of the project site, between Profiles 2 and 3, there is a slight decrease in sediment transport capacity to the south from 7,500$m^3$ to 7,000$m^3$ followed by an increase from 7,000$m^3$ to 8,500$m^3$ between Profile 3 and 4. This indicates a potential for sediment to accumulate around the headland at the north end of the proposed northern beach cove 1, while the increase of sediment movement to the south along the northern beach cove suggests that sand is being produced by the reef system itself and deposited at the north end of the southerly beach cove.
- In the southern section of the project site, between Profiles 5 and 6, there is an increase in sediment transport capacity to the south from 3,100$m^3$ to 5,400$m^3$ followed by a decrease from 5,400$m^3$ to 3,200$m^3$ between Profiles 6 and 7. This indicates a potential for some erosion in the centre of that bay, with sediment accumulation at the south section of the southern beach cove. This process will be further investigated in the beach response modeling analysis.


**Figure 4.2 Total sediment transport ($m^3$) accumulated over one year with arrows/values (green/blue) indicating the sediment transport respectively to the north and south**

### *4.2 Wave Events Duration: Sediment Transport Analysis*

Sediment transport was also examined for each wave event, ranked in order of event duration from the longest to the shortest (Figure 4.3). The plot shows wave direction and wave height per event (top graph), sediment transport per event (middle graph), and cumulative transport relative to the event duration ranked in descending order (bottom graph).

These results suggest that:

- The longest duration events corresponding to the first 25 events (bottom graph) represent 10% of the annual wave climate. Among the 300 events registered, the longest first 100 events represent almost 80% of the annual wave climate. The remaining 200 events have a maximum duration less than 2% (7 days per year).
- The comparison of the wave height per event (top graph) and the sediment transport per event (middle graph) demonstrate that both long duration events (daily waves) and moderate duration events (occasional swells) create sediment transport to the south.
- The comparison of wave direction per event with the cumulative sediment transport per wave duration (top graph) also demonstrates that the events controlling sediment transport

are clustered within the north to northwest directional band, from 320 to 10 degrees from north. This observation suggests that sediment transports occurs when the waves come from a restricted offshore angle.

- The comparison of the wave height per event (top graph) with the cumulative sediment transport per wave duration (bottom graph) demonstrates that the sediment transport is mostly dependent on the occurrence of small waves (from 0.1m to 0.3m) and high waves (greater than 1.2m). This would imply that sediment transport is mostly generated during daily conditions but can also occur during swells.
- The comparison of the sediment transport per event (middle graph) with the cumulative sand transport (bottom graph) suggests that the dominant transport is negative (to the south). Numerical modeling of alongshore transport indicates a net annual transport to the south of approximately -8,000$m^3$ (bottom graph) over the northern beach cove (left) and -6,000$m^3$ along the southern beach cove (right).

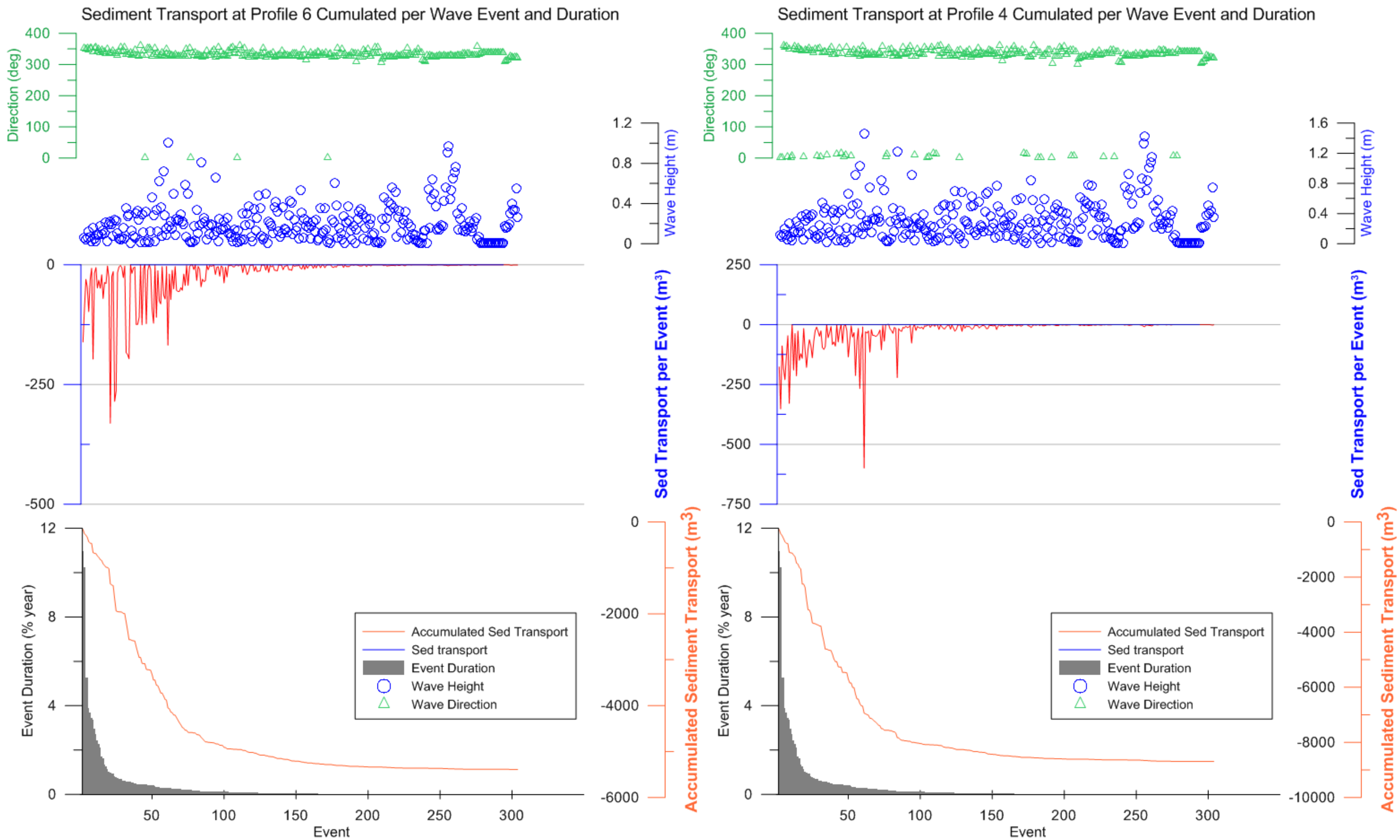



**Figure 4.3** **Sediment transport per wave event ranked by event duration along the northern (left) and southern (right) existing beach coves**

### *4.3 Sediment Transport from Swell Events: 2-dimensional Analysis*

The dynamics of the beach and its response to wave conditions were also modeled with the aid of MIKE 21. It was used for the hurricane storm surge analysis, but can also compute beach morphology using three main modules that work together in a coupled mode; results from one module are passed back and forth to the other modules in order to improve the efficiency and accuracy. The Spectral Wave (SW) module computes the wave conditions throughout the model domain; the Hydrodynamic (HD) module computes the water levels and current speeds, and is coupled with the SW module so that wave-induced currents are included. Water levels and currents affecting waves are also passed back to the SW module to improve the accuracy of the modeled wave conditions. The Sediment Transport (ST) module uses the results of the SW and HD modules to compute alongshore and cross-shore sediment transport rates. Finally, the model modifies the seabed depths based on the computed sediment transport rates and this modified seabed is used in subsequent time steps to compute the wave conditions and current patterns.

The basic starting point of the model is the creation of a computational mesh where waves, currents, sediment transport rates and the resulting morphological changes are determined at each simulation time step. The MIKE 21 model uses a flexible mesh, which represents the seabed by using a series of connected triangular and quadrangular elements. Quadrangular elements are preferred in areas where significant morphological changes are expected to occur, as the gradients in sediment transport can be more accurately represented. Along the project shoreline, the nearshore zone is expected to be very active from a sediment transport perspective and therefore, detailed resolution (using quadrangular elements) was required to adequately resolve and accurately represent the bathymetry in this area. The mesh and bathymetry used for beach response modeling of the existing conditions and proposed solution is presented in Figure 4.4. This mesh encompasses the study area and defines the water depth and sediment thickness at a series of connected points.

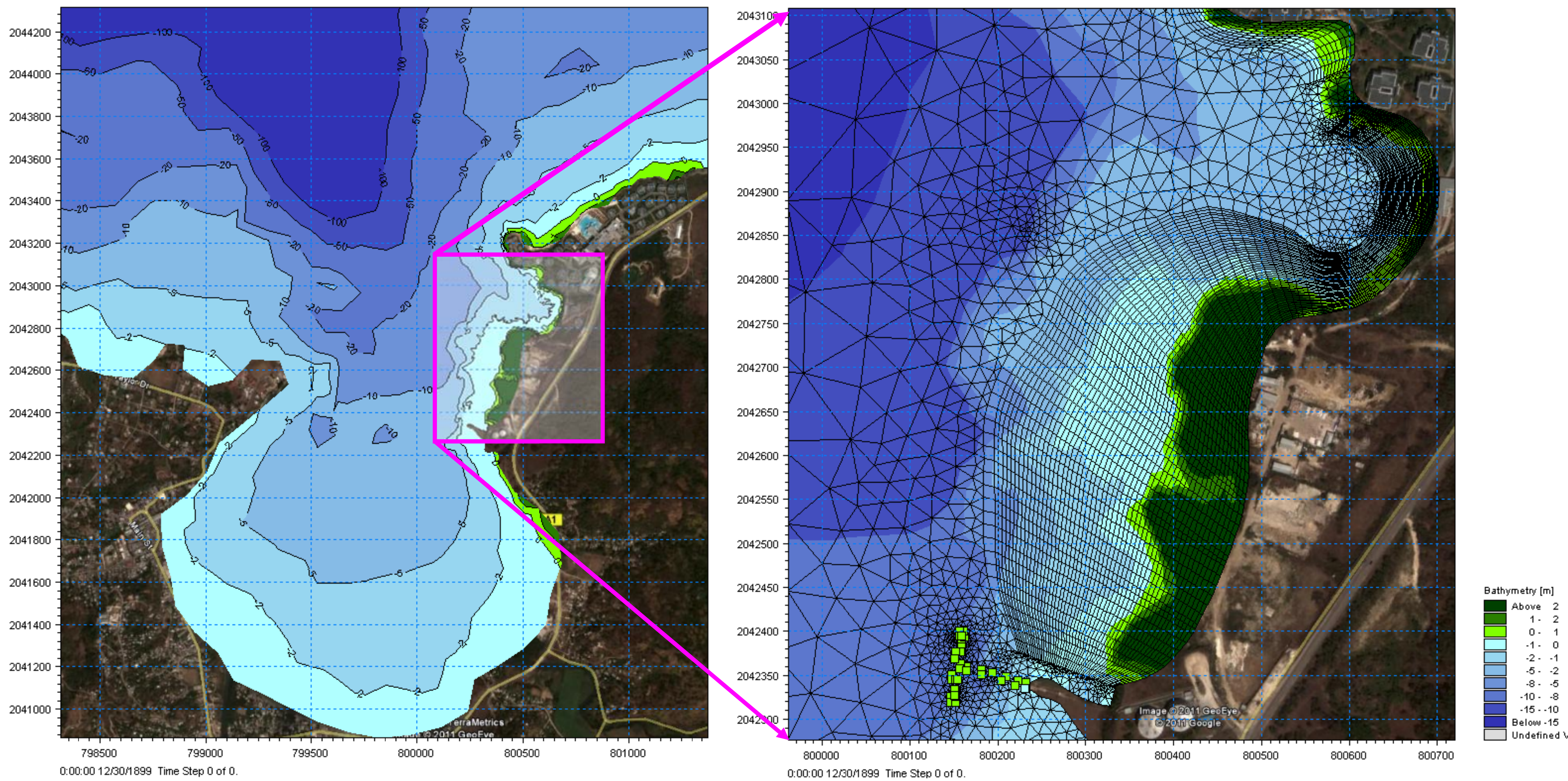



**Figure 4.4 Bathymetry offshore (left) and detailed (right) flexible mesh for existing condition**

The MIKE 21 model was used to investigate the beach response after a high swell event that occurred in November 2006 and a longer period swell event that occurred in January 2005. Time-varying wave heights, periods and directions characterising the swell events were input along the deep water boundary of the numerical model. Other points to note include:

- The SW module was run in a semi-stationary mode, with time-varying inputs of wave height, period and direction taken from the deep water Wave Watch 3 model.
- Wave-induced currents, which arise as waves break and dissipate energy, were included in the hydrodynamic (HD) calculations.
- Bed resistance was defined using a Manning coefficient, and the Smagorinsky formulation was used for eddy viscosity.
- Pre-calculated sediment transport tables were used to improve the model's efficiency. These transport tables were calculated using the Stokes 1$^{st}$order wave theory, which was found to be the best method to accurately reproduce the wave-induced near-bed velocities, both in the shoaling and the surf zones. In the shoaling region, wave asymmetry results in onshore-directed net sediment transport, which is typically small. In the surf zone, wave breaking and the associated undertow are the dominant mechanisms, which in most cases results in offshore-directed cross-shore sediment transport.
- A mean grain size diameter of 0.4mm and a grading coefficient of 1.1 were taken as constants for the sediment properties.
- Layer thicknesses varying from 0-2m were used in the locations assumed to contribute to the sediment transport, depending on the location of the node considered. Areas outside this had a sediment thickness of 0.0m, simulating either the existing reef and rocky shoreline features or an otherwise non-erodable substrate.

The resulting wave height and direction (a), current speed and direction (b) and the change in sea bed elevation and total sediment direction (c) are represented in Figure 4.5 for both the November 2006 swell (top) and January 2005 swell (bottom).

The plots show that the highest waves (up to 0.8m) occur during the November 2006 swell in the southern section of the project site, while the northern section (with waves up to 0.4m) appears to be sheltered due to the existing reef feature offshore of the project site.

The spatial distribution of currents shows the formation of a rip current within the existing Grand Palladium beach cove just north of the project site with speeds slightly higher during the January 2005 swell (0.3m/s) than the November 2006 swell (0.2m/s). At the project site currents flow towards the south with maximum currents, up to 0.7m/s, during the January 2005 swell in the northern and middle sections of the project site along the existing reef.

The plots of bed level change show erosion of the beach with maximum up to 0.5m measured just south of the northern beach cove and 0.6m measured on the headland just north of the project site. Erosion values are slightly higher during the January 2005 swell than the November 2006 swell. Finally, the sediment transport direction gives insight into the mechanism of beach loss from this bay. The plot shows that both beach coves erode with deposition offshore beyond the reef, indicating an alongshore movement of sediment over the reef and predominant cross-shore movement of sand over the southern beach cove beyond the existing reef and the northern headland just north of the northern beach cove.

Higher current and bed level changes were observed during the January 2005 swell despite the fact that the November 2006 swell generated higher waves at the project site. The January 2005 event included longer period waves coming from a slightly more northwest oriented swell. This suggests that the wave-induced sediment transport is very dependent on the wave period and the offshore wave angle.

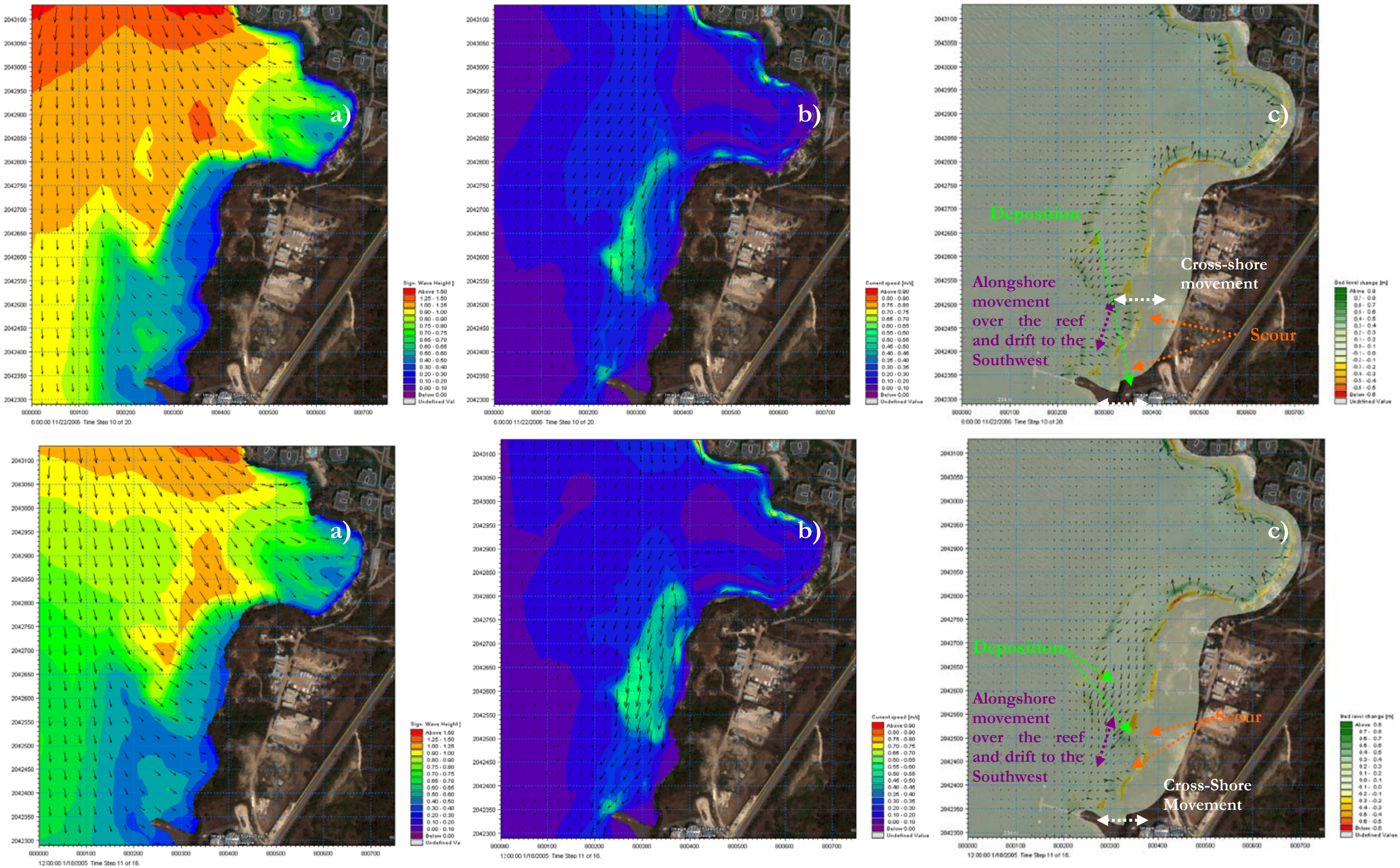



**Figure 4.5** (a) wave heights, (b) current speed, (c) sediment transport and direction at the peak of the November 2006 (top) and January 2005 (bottom) swell

## *4.4 Summary and Implications of Findings*

This section provides a summary of findings on the coastal process investigations and their implications, as well as recommendations for the design and implementation of protective structures at the project site.

A site visit and photos taken there revealed no obvious sediment transport trend along the two pocket beaches. However, the project site is protected by an extensive reef system that would be the main source of sediment input into the system. The nearshore area has seagrass patches and is very still with virtually no wave activity.

Water depths up to 200m from the shoreline are very shallow (ranging from 0.5-2m) because of the reef system. Seaward of the reef system water depths increase rapidly to 10m. The landward side of the project site is surrounded by dense vegetation and cliffs varying from 2-4m in height.

Sediment sample analyses indicate that samples were comprised of gravels and sand with distinctive coarse and fine fractions and no or negligible amounts of mud. Gravels found in some samples were indicative of coral fragments. This information was used in the beach response modeling and provided guidance for the grain size distribution to be used for the nourishment of the proposed beach coves. Note that for beach nourishment stability purposes, slightly coarser sand is recommended.

The operational wave climate describing the day-to-day wave conditions was obtained from the global wave model WAVE WATCH 3 (WW3) developed by NOAA. The data was extracted for a node north of Lucea Harbour and included descriptions of the wave height, period and direction as well as the wind speed and direction. Results were obtained every three hours from July 01 1999 to November 31 2007, giving a total of 24,000 data points per parameter.

The project wave climate was transformed from offshore to the nearshore area using the MIKE 21 Spectral Wave module. The model was run in a semi-stationary mode, with time varying inputs of spectral wave parameters $H_s$, $T_p$ and mean direction, taken from the WW3 wave database. The seven years of wave data were binned using a tri-variate frequency analysis of wave height, period and direction. The resulting wave heights in the nearshore of the project site were mostly lower than 0.25m (80% of the time); ranged from 0.25-0.5m (15% of the time); from 0.5-0.75m (3% of the time) while waves from 0.75-1m occurred 2% of the time and waves greater than 1m only occurred 1% of the time. An extreme wave climate was established using over 100 years of hurricane tracks and records. A return period of 50 years (recommended risk level for coastal construction projects) was used to compute nearshore wave conditions and storm surge heights from the six main directional sectors considered as having an impact on the project shoreline.

The deep water conditions were transformed across the shelf to the nearshore regions up to the project site using MIKE 21 in a coupled mode, to incorporate the effects of wave- and tidally-driven currents. The resulting maximum wave height and storm surge computed from all directional sectors showed that waves ranged from 0.4-1.5m and storm surge reached values up to 1.6m excluding wave run-up. Detailed results for each of the directional sectors are available in Appendix C.

An analysis of alongshore sediment transport was undertaken to determine the rates and distribution of sand movement. Alongshore sediment transport rates were determined along seven selected profiles using the resulting frequency analysis from the seven years of WW3 data. The resulting data was extracted at the location of the cross-shore profile and input to the alongshore sediment transport calculations.

The cross-shore sediment transport distribution demonstrated that sediment transport occurs seaward of the existing reef (200m away from the shoreline) and within 25m of the shoreline just shoreward of the reef. The presence of the reef (which induces wave breaking and dissipation of wave energy on its seaward side) was indicative of a potential sediment pathway.

The accumulated alongshore sediment transport was quite low (ranging from 3,000-8,000m$^3$ per year) with predominant net transport to the south. This was demonstrative of a fairly low annual production rate at the project site. The increase in sediment transport between both beach coves suggested that the sand is produced locally by the existing reef system while the rocky headlands help contain the sand within each bay. The results also suggest that the downdrift impact caused by the implementation of a protective structure would be very small. This is particularly relevant for the south cove, which is bound on its south side by a promontory that juts out approximately 100 metres into the sea. The Molasses Jetty is located off this promontory.

The comparison of wave height and direction per event and cumulative transport demonstrated that sediment transport is mostly generated during daily conditions but also occurs during swells with all events coming from the north to northwest directional band. These observations suggest that the beach tends to be stable, but sediment transport will occur when waves come from a restricted offshore angle.

MIKE 21 was used to assess beach response during swell activity. Results demonstrated that alongshore sediment transport was mainly over the reef with deposition offshore to a southwest direction while cross-shore movement of sediment was observed mostly in the southern beach south of the existing reef and on the northern beach during the 2006 swell.

Higher current and bed level changes were observed during the January 2005 swell despite the fact that the November 2006 swell generated higher waves at the project site. The January 2005 event included longer period waves coming from a slightly more northwest orientated swell.

Two-dimensional modeling results indicate that the waves inducing cross-shore and/or alongshore sediment transport are very dependent on the period and offshore angle of the waves. The November 2006 swell, with shorter period waves coming from a more northerly direction, generated cross-shore sediment transport along both coves while the January 2005 event (with longer period waves) coming from a more north-westerly direction, generated higher currents at the site and was responsible for alongshore sediment transport and drift to the southwest. Results also suggest that the beach will remain stable and protected by the existing Grand Palladium headland situated just north of the project site unless the waves come from a clustered northwest angle, in which case beach erosion is quite noticeable. This type of event is expected to occur in the winter period when swells are generated.

# 5. Hydrology

The surface runoff of any area can have a significant effect on the coastline onto which it empties. In the case of Grand Palladium, there are two natural drains that empty on to the areas designated for beach development in Phase II. Managing the flow of water that will rush on to the developed beach is critical in preserving the sediment placed there, as well as reducing scour effects, etc., on the shoreline. It is imperative therefore that a detailed drainage analysis be undertaken to understand the flows being generated in the catchment area as well as the capacity necessary to handle them.

## 5.1 The Watershed Runoff Process

Figure 5.1 is a diagram of the watershed runoff process[1].The watershed runoff process begins with precipitation, which can fall on the watershed's land surface, and water bodies. In the natural hydrologic system, much of the water that falls as precipitation returns to the atmosphere through evaporation and transpiration. During a rainfall event however, this evaporation and transpiration is limited. Some precipitation will fall directly on the land surface; depending on the soil type, ground cover, antecedent moisture and other watershed properties, it might infiltrate the soil to be stored temporarily in the upper, partially saturated layers of soil. From there, it rises to the surface again by capillary action, moves horizontally as interflow just beneath the surface, or it percolates vertically to the groundwater aquifer beneath the watershed. The interflow eventually moves into the stream channel. Water in the aquifer moves slowly, but eventually some returns to the channels as baseflow. Water that does not pond or infiltrate will move by overland flow to a stream channel. In other words, the stream channel is the combination point for the overland flow, the precipitation that falls directly on water bodies in the watershed, and the interflow and baseflow. Thus, resultant streamflow in the channel is the total watershed outflow.

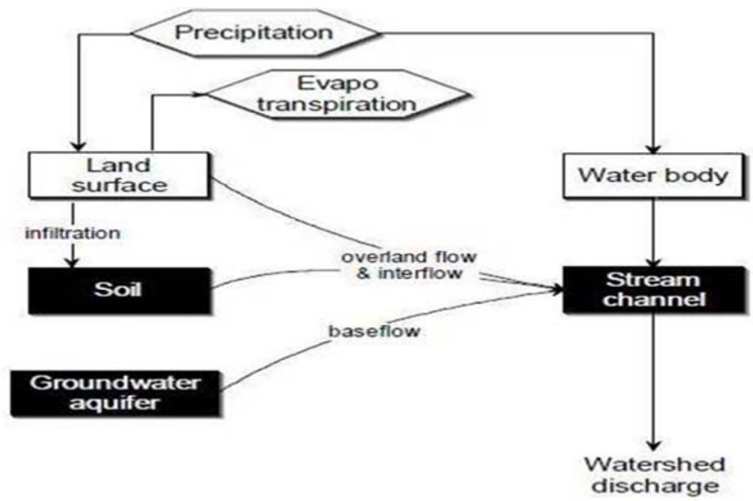



**Figure 5.1 Typical representation of watershed runoff (HEC-HMS Tech. Ref.2000**

## 5.2 Precipitation Description

The response of a watershed is driven by precipitation that falls on the watershed and evapo-transpiration from the watershed. The precipitation may be observed rainfall from a historical event or it may be a frequency-based hypothetical rainfall event.

[1] HEC-HMS Technical Reference Manual – March 2000

### 5.2.1. Extreme Rainfall Events

The 24-hour rainfall data provided by the National Meteorological Service of Jamaica is given in pre-calculated return period brackets. The values are either calculated by assuming a linear increase or decrease in intensity or by determining the probability of observing all the most extreme events within a given sub-period.

Values were given for thirteen gauge stations in the parish of Hanover. There was no gauge in the catchment being considered, (that of the Grand Palladium Phase II expansion) so a weighted average of all the gauge information was used, with the higher weighted importance being placed on the Lucea gauge, which was considered to be within a relatively close range. The values inputted were as follows:

**Table 5-1 24-hour Rainfall Data for Extreme Rainfall Events at the Grand Palladium expansion site**

| Rainfall Data (mm/24hours) | T2 | T5 | T10 | T25 | T50 | T100 |
|---|---|---|---|---|---|---|
| | 105 | 152 | 187 | 235 | 265.5 | 316 |

As shown, the data provided was the 24-hour rainfall for the 1 in 2, 5, 10, 25, 50 and 100-year return periods. However, general guidelines will point to the use of the 1:25 year design storm as a minimum design standard for drainage. Therefore, although all peak flows were calculated, it was the peak flow generated in a 1:25 year design storm that was used to determine the required drainage capacity.

### 5.2.2. Rainfall Distribution

The 24-hour rainfall distribution curve is essential input for hydrographic analysis as it determines the precipitation patterns, the saturation of the soil and river volumes for a typical rainfall event.

The United States Department of Agriculture Soil Conservation Service (USDA-SCS) developed rainfall distribution curves from historical analysis of four types: I, IA, II, or III. Curve III is most typically used in Jamaica because the island has the same climate and approximately the same altitudes as the area for which it was developed. However it is not tied to any analysis conducted on rainfall by Jamaican local authorities.

Two rainfall distributions for Jamaica, the Ja–A and the Ja–Bx have been developed by the Jamaican Water Resources Authority based on statistical analysis of hourly precipitation amounts through various rainfall events. The three rainfall distribution curves applicable to the island of Jamaica are plotted as a fraction of the 24-hr total distribution and are shown in Figure 5.2.

The Ja-Bx distribution has the most intense rainfall during the second quartile of the rainfall event duration. Applying this distribution to the published 24-hour data allows the basin to respond with a peak discharge that would be considered 'worst case' and was thus applied to the hydrograph.

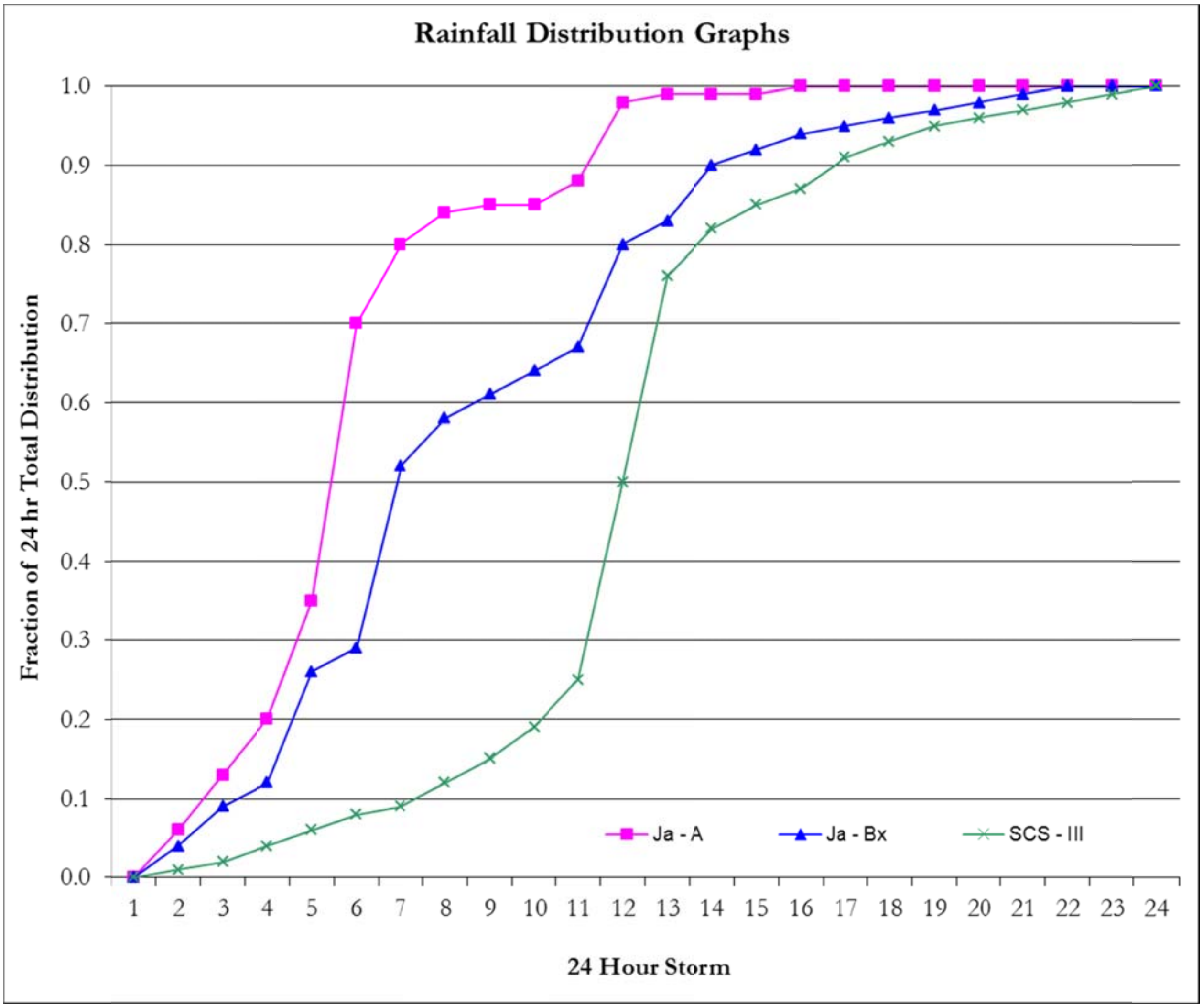



**Figure 5.2 Rainfall distribution graphs applicable to Jamaica**

## *5.3 Establishment of Stormwater Flows*

The computation of runoff values for a specific watershed requires an understanding of the limits and drainage features of the watershed. After delineating the catchment area being considered, calculations can be carried out from the application of a "Curve Number". This curve number is itself derived from two parameters, the Hydrologic Soil Group (HSG) and the Land Use.

### 5.3.1. Drainage Basin

There are several significant drainage features of the watershed area including: the high elevations of the hills above the Point area and the steep slopes leading to the drainage paths, which will create a high flow of water to and within the drainage paths. It is also significant to note that the main roadway into Lucea acts as a divider within the catchment and will break it in two.

The drainage basin that contributes surface water runoff to the area demarcated for the Phase II of the Grand Palladium hotel was delineated with Maps 21C, 21D, 22A and 22B of the 1:12,500 map series (Survey Department of Jamaica 1971 Edition). Grand Palladium Phase II is situated in an area known as

The Point, Hanover. The catchment area is bounded by the peak system of the hills to the southeast of Duhaney's Point. The peak system (outlined in red) is approximately 300 m above sea level and demarcates a boundary of the catchment stretching to Englin Town in the west and the coves dotting the shoreline between Anglin's Cove and Mosquito Cove to the north. The Grand Palladium and its larger catchment area of approximately 1374 acres are shown in Figure 5.3 along with the delineation of the main drainage sub-basins.

Based on the topography and drainage patterns of the entire catchment area, it can be divided into two sub-basins. The range which starts at Duhaney's Point continues in a south-easterly direction and divides the catchment area into a large catchment to the north which contains Phase I of the development and a smaller catchment to the south which contains Phase II. This latter catchment is the drainage basin of interest.

It should be noted that within the drainage basin of interest there are further catchment divisions. The roadway breaks the catchment, separating the upper catchment and hilly areas (referred to as Basin #1 herein) from the lower catchment, which contains the Phase II development and shall be referred to as Basin #2. Within the lower catchment, which is bound between the coastline and the roadway, a further division occurs along a small ridge in the area towards the molasses pier. Thus, within the drainage basin of interest containing the Phase II development area there are a total of three sub-catchments. These are highlighted in Figure 5.4.

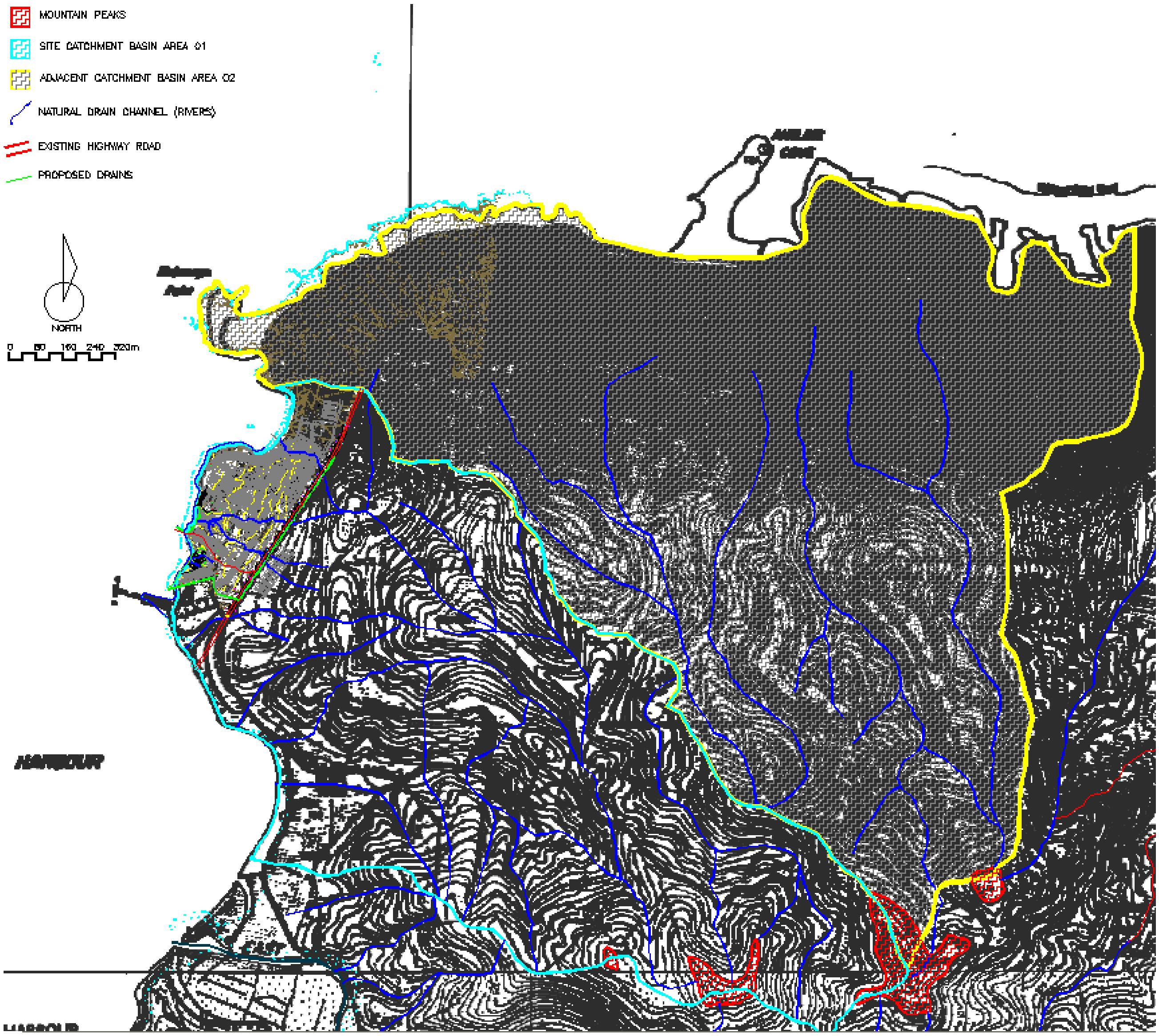



**Figure 5.3 Representation of the larger catchment area of relevance to Grand Palladium**

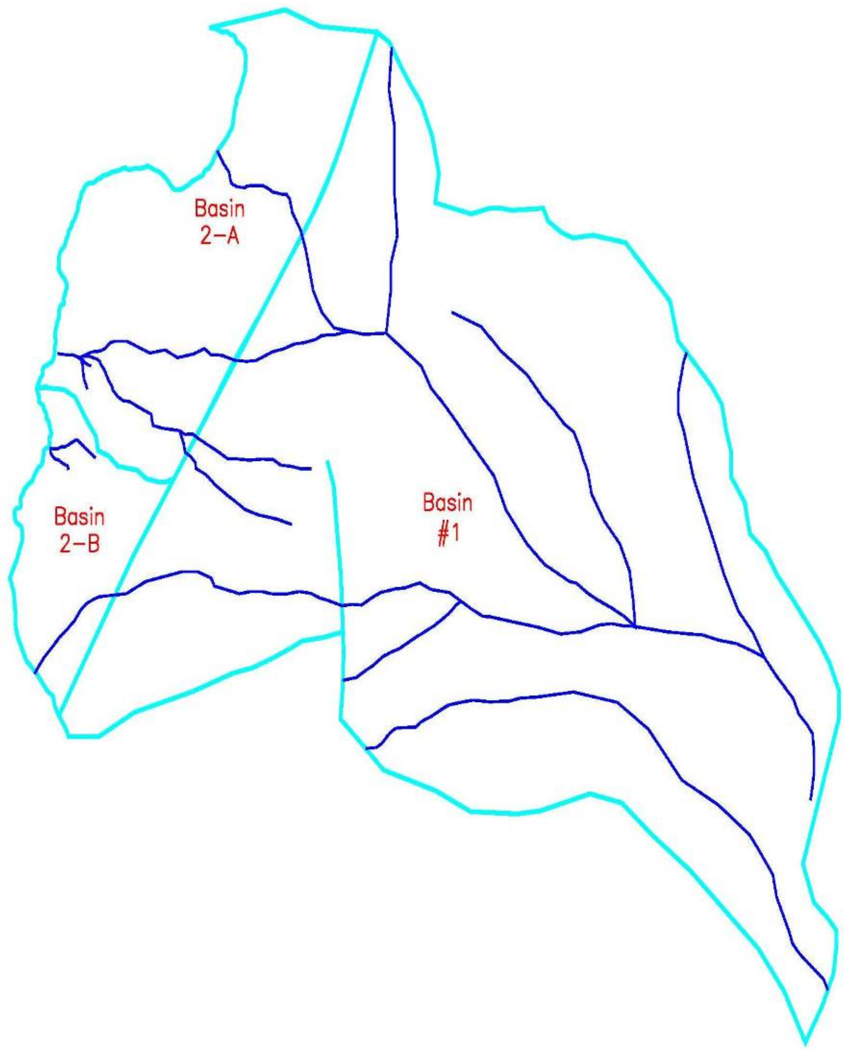



**Figure 5.4 Representation of the Grand Palladium three sub-catchments**

### 5.3.2. Definition of Hydrologic Soil Groups (HSG's)

The agricultural soil maps created by the Rural and Physical Planning Department of the Ministry of Agriculture were used to determine the soil types in the study area. The soil types identified within the drainage basin are #43 Highgate Clay and #46 Hall's Delight Channery Clay Loam; it is noted that the Channery Clay Loam is the predominant soil there. The parent material of these soils is made up of well-bedded layers of sandstones and claystones. Weathering of the shales takes place rapidly, forming new soil in quick succession[2]. If this were not so, the area would have been left barren of vegetation through soil erosion.

[2] Hillside Farming in Jamaica Training Seminar. Bib. Orton IICA / CATIE

Soils can be classified into four Hydrologic Soil Groups (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Within the Hydrological Soil Group: A = sand, B = a sandy loam, C = a loamy clay, and D = clay. As is expected, the runoff potential of the land increases as classification moves from A to D.

Based on the Hydrologic Soil Group classifications listed above, all of the drainage sub-basins can be classified as belonging to the Hydrologic Soil Group C.

#### 5.3.3. Definition of Land Use

Land use data for the analysis was taken from published land-use maps prepared by the Forestry Department of the Ministry of Agriculture and Fisheries - Jamaica, aerial photography and various sources of satellite imagery. The upper part of the catchment was defined as hilly, with mixed brush/vegetation, while the two lower sub-catchments were defined as urban residential based on the proposed hotel development in those areas.

#### 5.3.4. Composite Curve Number

As previously noted, the Composite Curve Number (CN) values are derived from two parameters, the Hydrologic Soil Group (HSG) and the Land Use. Based on the aforementioned categorizations of Land Use and HSGs in the basin, the CN values were assigned (Table 5-2).

**Table 5-2 Composite Curve Numbers used in basins and sub-basins**

| Basin / Sub-Basin | CN value assigned |
|---|---|
| Basin 1 – hills to roadway | 74 |
| Sub-Basin 2A | 80 |
| Sub-Basin 2B | 80 |

#### 5.3.5. Calculation of Storm water Runoff and Peak Flows

In the examination of storm water runoff and peak flows, three calculation methods can be used: (1) the Rational Formula, (2) the Jamaica II method, and (3) the SCS method. Each method is discussed briefly below.

*The Rational Method:* The Rational Formula is the simplest and most general method for calculating stormwater runoff in a catchment area. It is the product of the intensity of the rainfall falling on a specified area, with some allowance for the soil and usage type which is embedded in the Runoff coefficient. The Rational Formula reads:

$$Q_p = 0.28 \cdot C \cdot I \cdot A$$

where:

$Q_p$ = Peak runoff rate
C = Runoff coefficient
I = Rainfall intensity
A = Drainage area

*The Jamaica II Method:* The Jamaica II (metric) method of determining the peak surface runoff was developed locally for Jamaican conditions by Mr. R Harrison during his extensive time at the Public

Works Department (now defunct). It is based on trends observed over the course of his experience there. The equations involved in the calculation for peak runoff using the Jamaica II method are set out below:

$$Q = \frac{0.505 \cdot A \cdot R}{1.6746T_c + t}$$

$$T_C = \left[\left(4.7815 \cdot L^3 \cdot \left(\frac{101.4 - CN}{70}\right)^2\right)\Big/_H\right]^{0.234}$$

$$R = \frac{[p \cdot CN - 50.8(100 - CN)]^2}{CN[(CN \cdot p) + 203.2(100 - CN)]}$$

$$p = i \cdot t$$

where:

| | |
|---|---|
| Q | = Flow, $m^3/s$ |
| A | = drainage area, hectares, ha |
| R | = depth of runoff, mm |
| CN | = Curve Number for AMC III |
| p | = rainfall amount, mm |
| i | = rainfall intensity, mm/hr |
| Tc | = Time of concentration, minutes |
| t | = duration of rainfall and t > Tc, min |

The rain data collected from the National Meteorological Service's estimates of maximum 24-hour rainfall for various return periods is converted to rainfall intensities using the following equations developed by Mr. R. Harrison (based on Sangster International Airport's recording rain gauge) and are shown in Table 5-3.

**Table 5-3 Rainfall intensities in mm/hr**

| $T_R$ Years | Rainfall Intensity in mm/hr (Sangster Int'l ) | |
|---|---|---|
| | t < 60 min | t > 60 min |
| 2 | $i = 5.6559Pt^{-0.5171}$ | $i = 24.888Pt^{-0.879}$ |
| 5 | $i = 6.4753Pt^{-0.5704}$ | $i = 20.5852Pt^{-0.8529}$ |
| 10 | $i = 6.7976Pt^{-0.5893}$ | $i = 19.281Pt^{-0.8439}$ |
| 25 | $i = 7.063Pt^{-0.6047}$ | $i = 18.2178Pt^{-0.8361}$ |
| 50 | $i = 7.1923Pt^{-0.6123}$ | $i = 17.6826Pt^{-0.832}$ |
| 100 | $i = 7.2901Pt^{-0.6181}$ | $i = 17.2759Pt^{-0.8288}$ |

*The SCS Hydrologic Method*: The SCS method was developed by the United States Department of Agriculture (USDA). It can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The SCS method is a widely accepted model for predicting storm flow volumes from rural catchments. Although it was derived for rural catchments with uniform conditions, it can also be used for rural catchments contributing to an urban system.

The SCS hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. However, the SCS approach is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses due to interception and depression storage, and an infiltration rate that decreases during the course of a rainfall event. The SCS equation for stormwater runoff is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

where:

$Q$ = Stormwater Runoff
$P$ = Rainfall
$S$ = Potential Maximum Retention after runoff begins
$I_a$ = Initial Abstraction

From the hydrograph analyses (SCS 1983), the Graphical Peak Discharge Method for computing peak discharge from rural and urban areas was developed. The peak discharge equation is:

$$q_p = q_u \cdot A_m \cdot Q \cdot F_p$$

where:

$q_p$ = peak discharge
$q_u$ = unit peak discharge
$A_m$ = drainage area
$F_p$ = pond and swamp adjustment factor

*Method Used, Assumptions and Limitations*

Each of the aforementioned methods was used in the determination of the peak discharge and the resulting values for each rainfall event compared.

Typically it was found that the Jamaica II method underestimated the values, whereas the SCS method and the Rational method gave reasonably similar results, with the Rational method tending to be larger. It is thought that the applicability of the formulas is largely dependent on the size, topography and land usage within the catchment area.

It is to be noted that there is no exact method of determining the storm water runoff for any given watershed area. The selection and definition of land usage, soil groups, drainage basins and sub-basins requires an in-depth knowledge of the terrain and otherwise may be somewhat subjective. There is therefore the potential for errors to be involved in the calculations. Errors may be further exacerbated by the assumptions inherently involved in the calculation steps. These assumptions are:

- Consideration of the entire drainage area as a single unit;
- Estimation of flow at the most downstream point only; and
- The assumption that rainfall is uniformly distributed over the entire drainage area.

In addition to the assumptions governing all of the methods, there are limitations to each method applied:

- The rational method is good for small, homogenous catchment areas and is not to be used on areas larger than 200 acres. This severely restricts its usage on any of the areas being examined.
- The Jamaica II method, although developed specifically for local conditions, is applicable to the rural areas of Jamaica where significant and suitable soil cover exists. It has not been updated to handle the significant urbanization of many towns across Jamaica. This is why the Jamaica II method may underestimate the runoff values calculated, as it assumes a higher initial infiltration into the soils than actually exists. Additionally, the intensity formulas are based on 24-hour rainfall and rainfall duration data collected at the Sangster International Airport. As the intensity patterns vary dramatically across the island, these intensity formulas should not be considered relevant to the entire island.
- The SCS method is not limited by catchment size; and since it was developed as a general formula applicable to many land use areas, it can be used for rural, urban and mixed catchment areas. The SCS calculation method was therefore used in the determination of the peak discharge value.

The resulting peak discharges are as shown in Table 5-4.

**Table 5-4 Peak flows ($m^3$/sec) calculated from the Graphical Peak discharge method. The design return period is highlighted**

| | 1/2 yr | 1/5 yr | 1/10 yr | 1/25 yr | 1/50 yr | 1/100 yr |
|---|---|---|---|---|---|---|
| **Roadway Basin** | 5.441 | 11.085 | 15.392 | 21.609 | 25.817 | 32.863 |
| **Sub-basin 2A** | 1.378 | 2.411 | 3.220 | 4.358 | 5.092 | 6.321 |
| **Sub-basin 2B** | 0.489 | 0.856 | 1.143 | 1.546 | 1.807 | 2.243 |

## *5.4 Hydraulic Analysis*

The area's natural topography and soil type facilitate the relatively speedy drainage of the site during rainfall events. In addition, it is to be expected that with the construction of hotel buildings and road infrastructure, much of the water that would have infiltrated into the ground will now contribute directly to the run-off from the site. There are two natural drainage features running through the area designated for the expansion, through which stormwater is being carried to the sea. Further, above the Phase II area the roadway acts as an interceptor that channels water from the hills into an existing gully that in turn discharges to the sea south of the Molasses Jetty. This site can therefore be classified as a well-drained site in its natural state.

Drainage infrastructure must however, be installed within the property even though the site is well-drained. This is because heavy rains can cause scouring of the roadway, deterioration of the pathways within the proposed hotel, erosion of hotel lawns and gardens, and can negatively affect the beach area by bringing debris down the channel as well as eroding the scarp and coastline there. As a result, proper site drainage details to handle the storm water runoff must be developed.

The outline site drainage plan that has been developed proposes to channel the runoff along the natural land contours and will rely primarily on three main open channel drains that will cross the site from the main road and discharge to the sea. These main drains will in turn be fed by secondary open channel drains that will double as water features on the site and are intended to intercept overland flow moving towards the western (seaward) boundary of the project site. Finally, at a third level, HDPE pipes will be used to collect water from buildings and roads to feed into the primary or secondary drainage systems as necessary. The recommended layout is shown in **Error! Reference source not found.**. The detailed design of these drainage elements will be carried out in the Final Design stage of the project. It should be noted that a geotechnical team will be employed to carry out drill investigations underneath the axes of the proposed new buildings. It is recommended to have the owners include in the scope of works for that team, an investigation into the permeability of the rock in the vicinity of the outfall for the central main open channel drain. The objective here would be to see whether or not a water detention and soak-away feature could be incorporated, with discharge to the sea occurring only during times of very heavy rainfall.

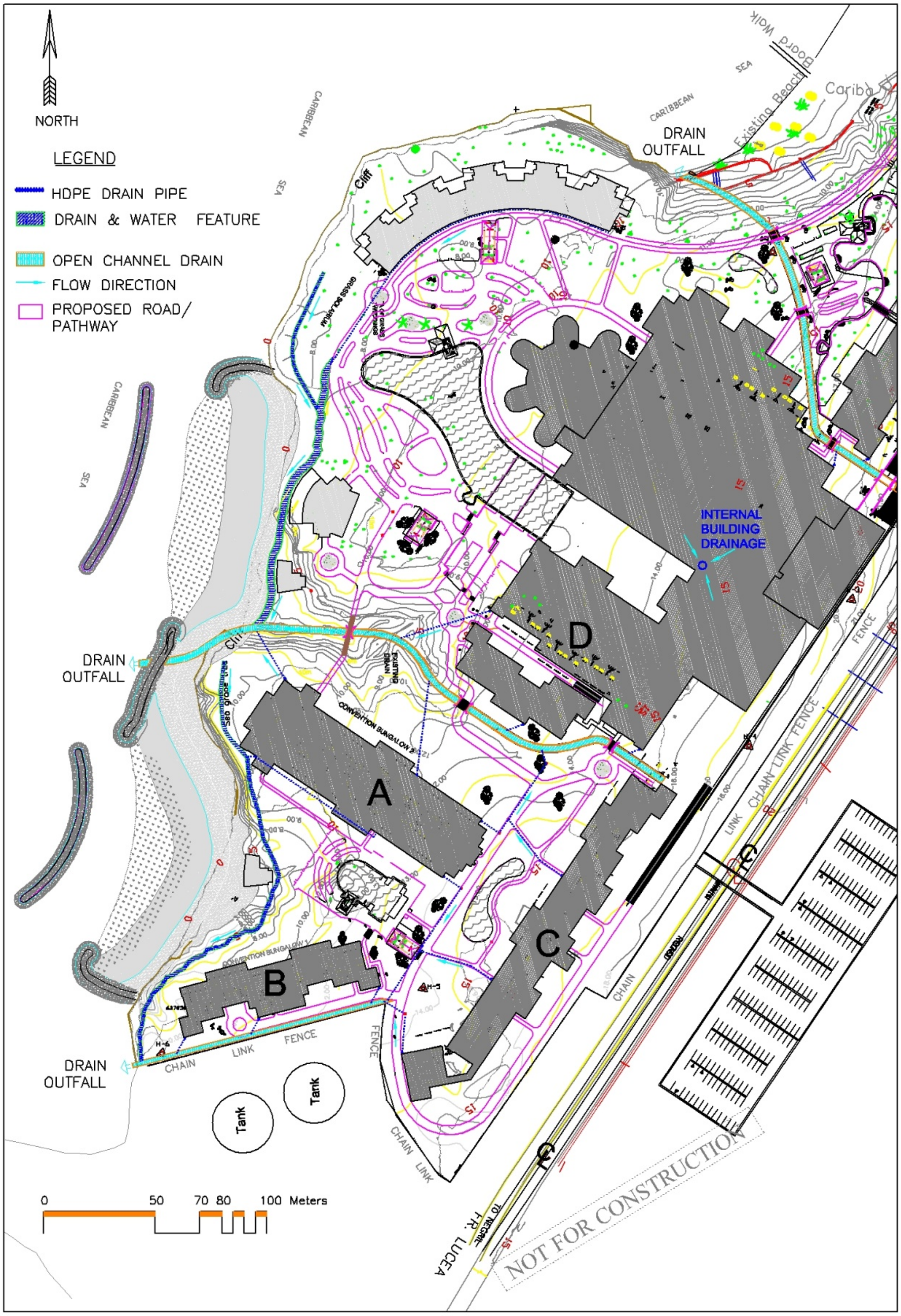



**Figure 5.5 Proposed Drainage Layout**

### *5.5 Evaluation of Storm Water Discharge*

The primary objectives of this work were to:

- Determine the effect of storm water discharge from the drains on the water quality in the nearshore of the project site.
- Review and revise the discharge locations of the existing storm water drains.
- Formulate an engineered solution to remedy the erosion of the shoreline due to storm water discharge.

The MIKE 21 hydrodynamic and pollutant transport modules were used to investigate and determine the location of the drainage outfall. The approach involved the evaluation of the existing current speeds and patterns as well as the pollutant decay over a tidal cycle including both neap and spring tide. The model domain and boundary conditions are represented in Figure 5.6

The hydrodynamic model computes nearshore currents forced by water level variations, such as tidal height variations. These values were obtained from a global tide prediction model and were applied along the east and west boundaries of the model, including Coriolis corrections. The global tide prediction model is part of MIKE 21 and is based on the superposition of numerous sinusoidal tidal constituents, each with an amplitude and phase lag that varies depending on time and position (DHI, 2009d). A simulation period of 7 days from the 13th to the 20th of January 2005 was chosen for the simulation and included a neap and spring tide, as shown in Figure 5.6

The pollutant transport module calculates the transport of materials or pollutants based on the flow conditions as determined in the hydrodynamic calculations. A base flow from the stormwater drain with an average discharge of $1m^3/s$ was used. An arbitrary pollutant was introduced at the central drain outfall seaward of the central T-Groyne and its dispersion and decay was mapped over the selected tidal cycle. This outfall was considered as the most critical of the three proposed outfall and was therefore used to evaluate any negatives impact of the storm water discharge.

An analysis was made to determine the impact the proposed beach enhancement works would have on the pollutant dispersion and also to examine the effect of the proposed drainage outfall. Figure 5.7 show the maximum (right) and mean (left) pollutant concentration from the central drainage outfall. The pollutant dispersion is represented by a graded colour scale with a starting concentration of 100.

The maximum concentration plot (Figure 5.7-right) show that 100% of the pollutant concentration plume is directed towards the northwest sector. While the 5 to 10% pollutant concentration plume is dispersed 20m seaward 95% of the pollutant concentration plume is dispersed over a distance of 300m. The mean concentration plot (Figure 5.7-Left) show that 100% of the pollutant concentration plume is directed towards the northwest sector. While the 5 to 10% pollutant concentration plume is dispersed 15m seaward 95% of the pollutant concentration plume is dispersed over a distance of 250m.

Generally, the mean and maximum concentrations show the same concentration cloud extent in as straight direction towards the west to northwest sector and away from the proposed swimming area. Overall it is seen that the pollutant does not affect the project site.

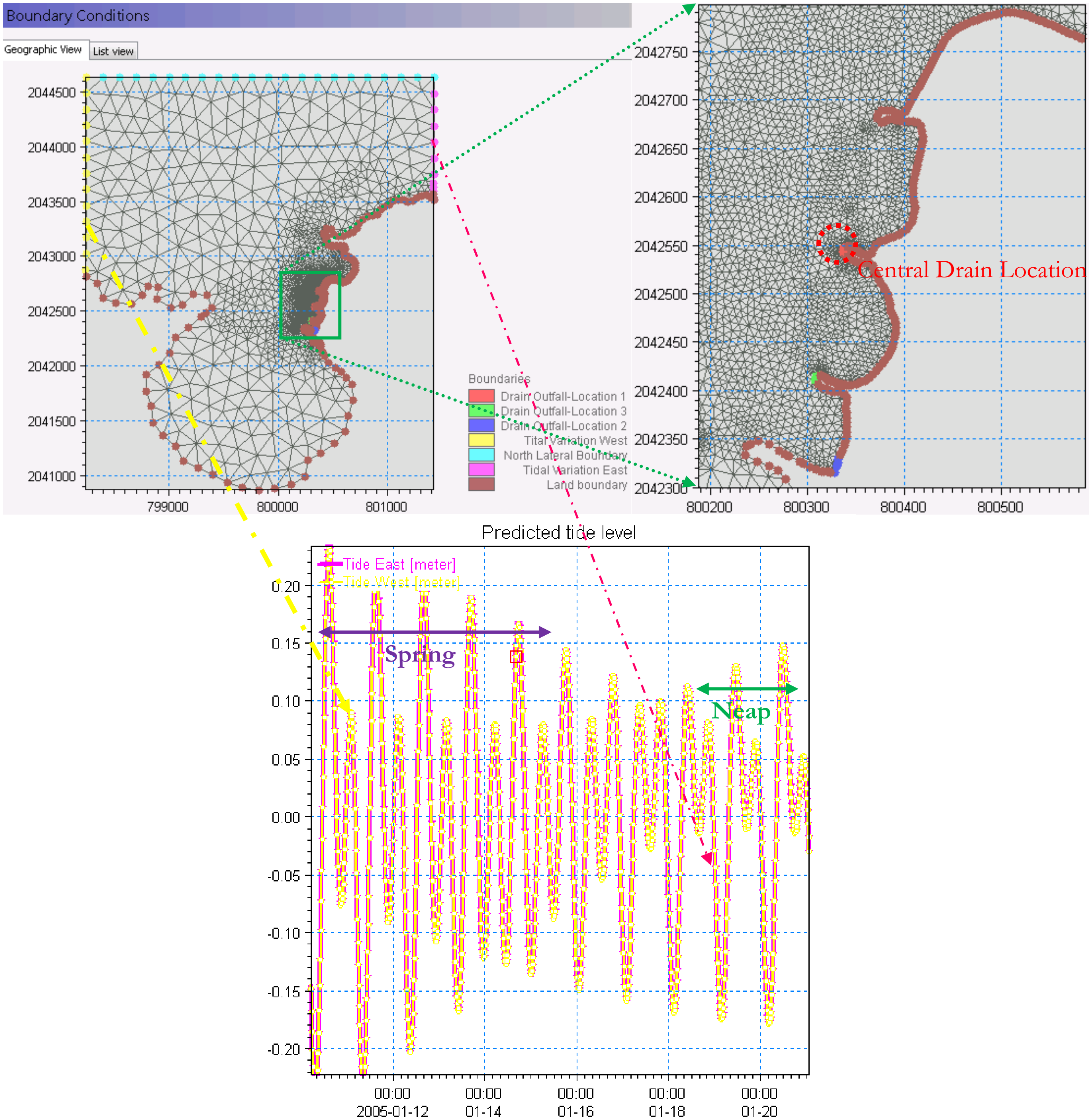



**Figure 5.6 Model domain and boundary along computational mesh**

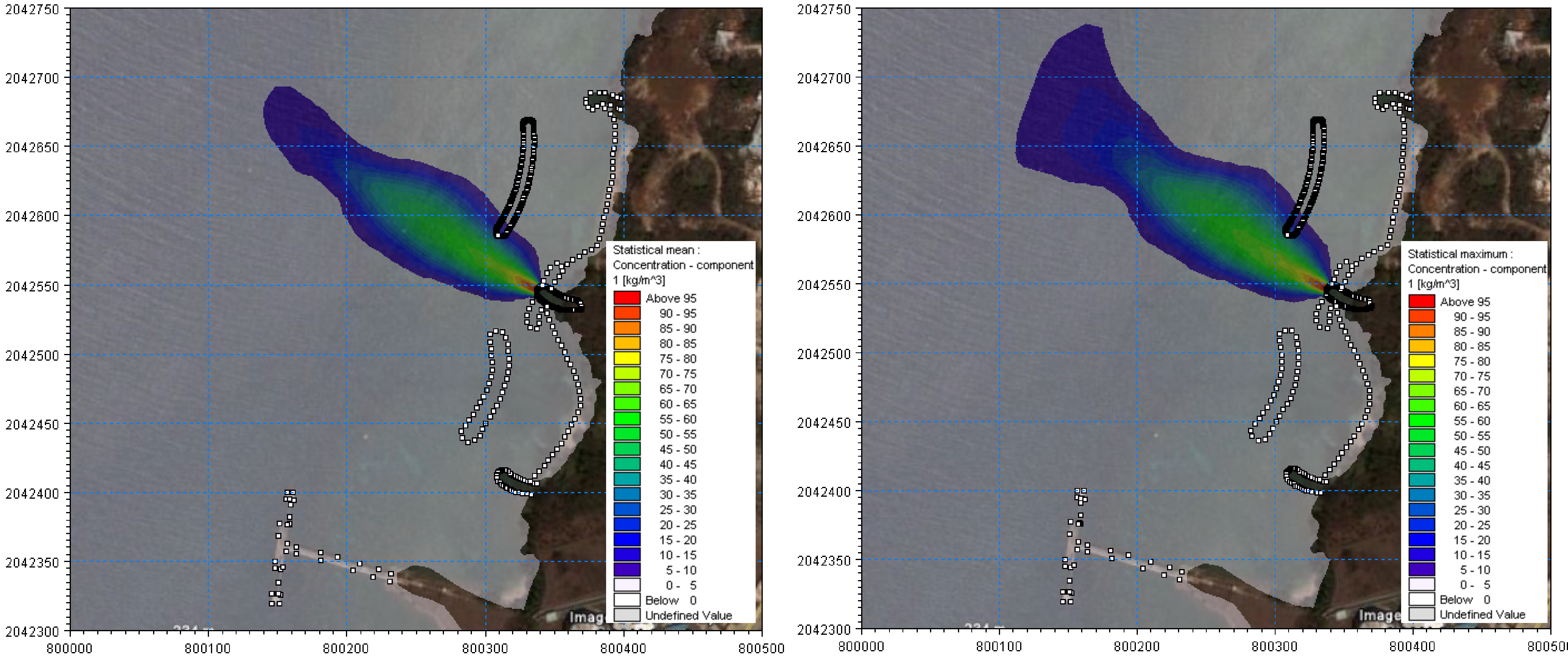



**Figure 5.7 Mean (left) and maximum (right) pollutant concentration for the final recommended option at the location of the central outfall**

# 6. Design of Coastal Structures/Beach Enhancement

The coastal engineering requirements for Phase 1 of the work were set out with the primary objectives being to:

- Identify the optimum approach to create two additional beach coves for the development of the planned Royal Suites;
- Produce an engineering solution to enhance and stabilize the newly created beach coves;
- Identify potential impacts on the marine environment and surrounding coastline;
- Set out a mitigation strategy and environmental management plan dealing with the relocation of any sensitive benthic organisms such as seagrass and/or corals;
- Present the developed concepts in an Engineering Report; and
- Provide preliminary volumes and cost estimates.

This section describes the design of the coastal structures proposed for the beach enhancement concept, allowing for the development of two wide sandy beach coves with a sandy seabed out to a water depth of 1.5-1.7m. Preliminary designs were prepared, including footprints and cross-sections for the considered option. Likely construction techniques and environmental impacts were also considered.

## *6.1 Design Specifications*

The primary challenge for this design is to ensure that a wider, stable beach is created, with a sandy foreshore suitable for swimming. This design objective can be achieved through the dredging of the existing foreshore and nourishment of the existing beach. Investigations also considered the need for the implementation of retaining structures to protect the newly created beach coves.

A preliminary concept design was developed based on existing literature, collected field data, as well as on an understanding of the waves, tides, currents and alongshore sediment transport characteristics at the project site. As waves and currents are the main driving force of sediment movement in the nearshore zone, these forces must be reduced and/or redirected to ensure the stability of the beach at the site.

The design of any protective structure is highly dependent on the day-to-day (operational) wave climate experienced in the nearshore. Given the incident wave climate, a range of alignments and dimensions of the proposed structures were investigated, with numerical modeling results detailed in Appendix C, leading to a total of eight preliminary concept solutions. Hurricane wave conditions were used to assess the structural stability requirements.

*Option1* consisted of the implementation of a T-Groyne to the north and a groyne to the south (as reinforcement) of both existing headlands set at 1.5m above mean sea level, along with two swimming beach coves totalling an area of 16,400m$^2$. Numerical modeling results indicate that this option would impair the natural flushing of the beach coves, which could lead to water quality problems in the future. This option was also considered as being too invasive from an environmental point of view due to the amount of coral and seagrass replanting. It was also considered to be too expensive due to the volume of seabed excavation and required beach nourishment.

*Option 2* consisted of the same north T-Groyne but a shorter south groyne, as well as an additional submerged breakwater further offshore (in 2.5m of water) with the same swimming and beach coves. Numerical modeling results revealed that this option helped in improving flushing of the southern beach coves but was also considered to be too invasive with regards to environmental impacts and too costly given the required amount of dredging and excavation of the seabed at 13,500m$^2$.

*Option 3* consisted of a shorter T-Groyne and submerged breakwater with the same beach and swimming dimensions. The modeling results showed satisfactory wave sheltering and flushing in the south beach cove despite the fact that a shadow zone was created in the lee of the T-Groyne. This option was created essentially to try and reduce the footprint and volume of structures to be implemented and to come up with a more environmentally friendly solution. This option was further revised to reduce the swimming and excavation areas to try and obtain a more cost-effective solution.

*Option 3 revised* consisted of the shorter T-Groyne and submerged breakwater from Option 3 along with considerably reduced swimming beach coves totalling a new area of 8,500m$^2$. This solution was found to be much less invasive to the environment as well as being more cost-effective. However, the numerical modeling results revealed flushing issues around the T-Groyne between both beach coves. This investigation led to the final recommended Option 4, created to improve flushing conditions within both coves.

*Option 4* consisted of the implementation of two emergent breakwaters located 64m from proposed beach cove 1 (north) and 98m from beach cove 2 (south) along with two retaining emergent groynes as reinforcement of the existing rocky headlands separating the beach coves defined in Option 3, and as an anchor at the south end of the works. The implementation of a timber walkway was also added to allow the guests lateral access along the back shore zone of this beachfront property.

The latest suggestion from the Fiesta Hotel Group – the creation of a single and extended dry beach spanning both coves – led to the investigation of three additional options.

*Option 5* consisted of the implementation of two emergent breakwaters located 64m and 60m respectively from the proposed northern and southern sections of the beach along with the north and south retaining emergent groynes presented in Option 4. For beach stability, an additional central emergent groyne built with two spears (to create a T-shape) was placed along the central section of the beach to help keep the sand in place. Numerical modelling results showed satisfactory wave sheltering in the lee of both breakwaters with the creation of two shadow zones in the nearshore of both coves. The resulting morphological results indicated the beach would be very stable. Note that the south emergent breakwater was moved closer to shore than in Option 4 to try and minimize the impact to the benthic environment.

*Option 6* consisted of the implementation of one single central emergent breakwater placed 60m from the central section of the beach along with two extended southern and northern emergent groynes as an anchor to the south and north end of the works. This option was investigated to try and implement both aesthetics and efficiency of the single extended beach, however numerical modelling results showed waves entering the southern section of the beach in the deeper sections located south of the cove and consequently lead to the investigation of Option 7.

*Option 7* consisted in the shifting to the south and the lengthening of the single central emergent breakwater with the objective of reducing wave energy entering the deeper sections while preserving the aesthetics. This option included the south and north emergent groynes proposed in Option 6.

Numerical modelling results showed satisfying wave sheltering in the lee of the proposed breakwater however some amount of erosion was observed in the central section of the beach. From a stability point of view this option was therefore not as efficient as Option 5. Note that from a beach stability point of view, small beach coves are always preferred to a longer beach, as these tend to become unstable over time.

*Option 5 was finally recommended as the most attractive and efficient solution for the beach enhancement.*

For the preliminary design of the proposed structures, the placement, length and crest elevation of each of the protective breakwaters was determined using the operational wave conditions to optimize the protection provided to the newly created beaches. The results from the coastal process investigations showed that the impact on the adjacent shorelines would be minimal due to the existing headlands north and south of the project site. Plan layout and typical cross-sections of the proposed beach enhancement are presented in Figure 6.1 and Figure 6.2, while design specifications are described following.

The design specification of *the Final Recommended Option 5* is as follow:

- A northern protective emergent breakwater 82m long (Breakwater A) to be built in a water depth of approximately 1m.
- A southern protective breakwater 73m long (Breakwater B) to be built in a water depth of approximately 1.5m.
- The crest elevation at 0.3m above MSL was adopted to decrease the permeability of the structures in shallow water while minimizing the footprint of the structure. Note that increasing the crest elevation of a breakwater above mean sea level significantly improves its effectiveness in blocking the waves and reducing wave energy in its lee, however, this creates a greater visual imposition. For this project, a higher, less permeable structure in shallower water depths would have the advantage of reducing the footprint of the structure and therefore reducing the area of seagrass and natural habitat that would be disturbed.
- To ensure impermeability of the breakwaters, a reinforced concrete wall is to be placed in the core of the structures.
- The protective breakwater will be composed of armour stone, with respective crest widths of 3m and a slope of 1(V):1.5(H).
- The construction of two retaining emergent groynes set with a crest elevation set a +1.5m above MSL and implemented as reinforcement of the existing rocky headlands as an anchor at the central, south and north end of the works. These structures would prevent loss of the nourished beach sand at the south and north end of the beach while promoting beach stabilization.
- The creation of one central emergent T-Groyne set at +1.5m above MSL, to provide stability in the central section of the beach, and sloping shoreward below the beach crest in order to allow the guest to walk along the entire length of the shoreline.
- The creation of one beach cove with 9,600m$^3$ of sand nourishment, the purpose of which is to enhance the beach in front of the development between the proposed structures and the shoreline. The finished elevation of the beach was set to +1m above MSL with a slope of 1 in 8 giving a total beach of approximately 300m (length) by 20m (width).

- To complement the creation of the beach coves, a deepened swimming area has been proposed between the proposed breakwaters and the shoreline. The total surface area ($6,900m^2$) will be created by deepening the seabed to a depth of 1.6m below MSL (mean sea level) totalling a dredging volume of $4,900m^3$. The area will serve the dual purposes of providing a swimming area and reducing the current flow across the beach and thereby encouraging sand to deposit in the general beach area.

- Between the groyne headlands, the upland area of the beach is to be cleared of existing rock rubble, debris and vegetation. The beach cove will be further extended by excavation of the foreshore over a total surface area of $4,600m^2$. It was estimated that a total volume of $4,200m^3$ would have to be cleared and excavated from the site.

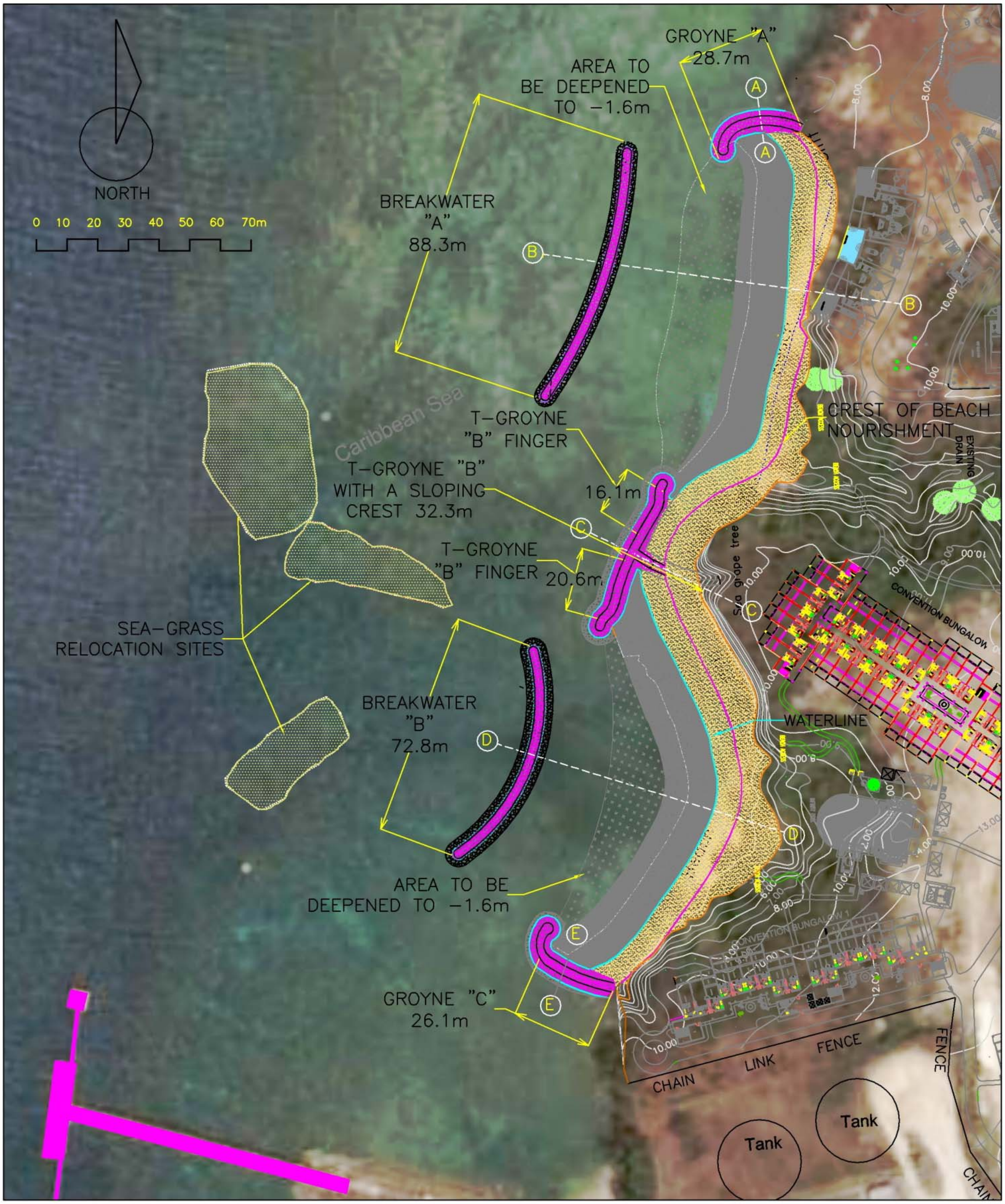



**Figure 6.1 Proposed beach enhancement concept for the Grand Palladium Royal Suites with extended dry beach and swimming are as recommended by Fiesta Hotel Group**

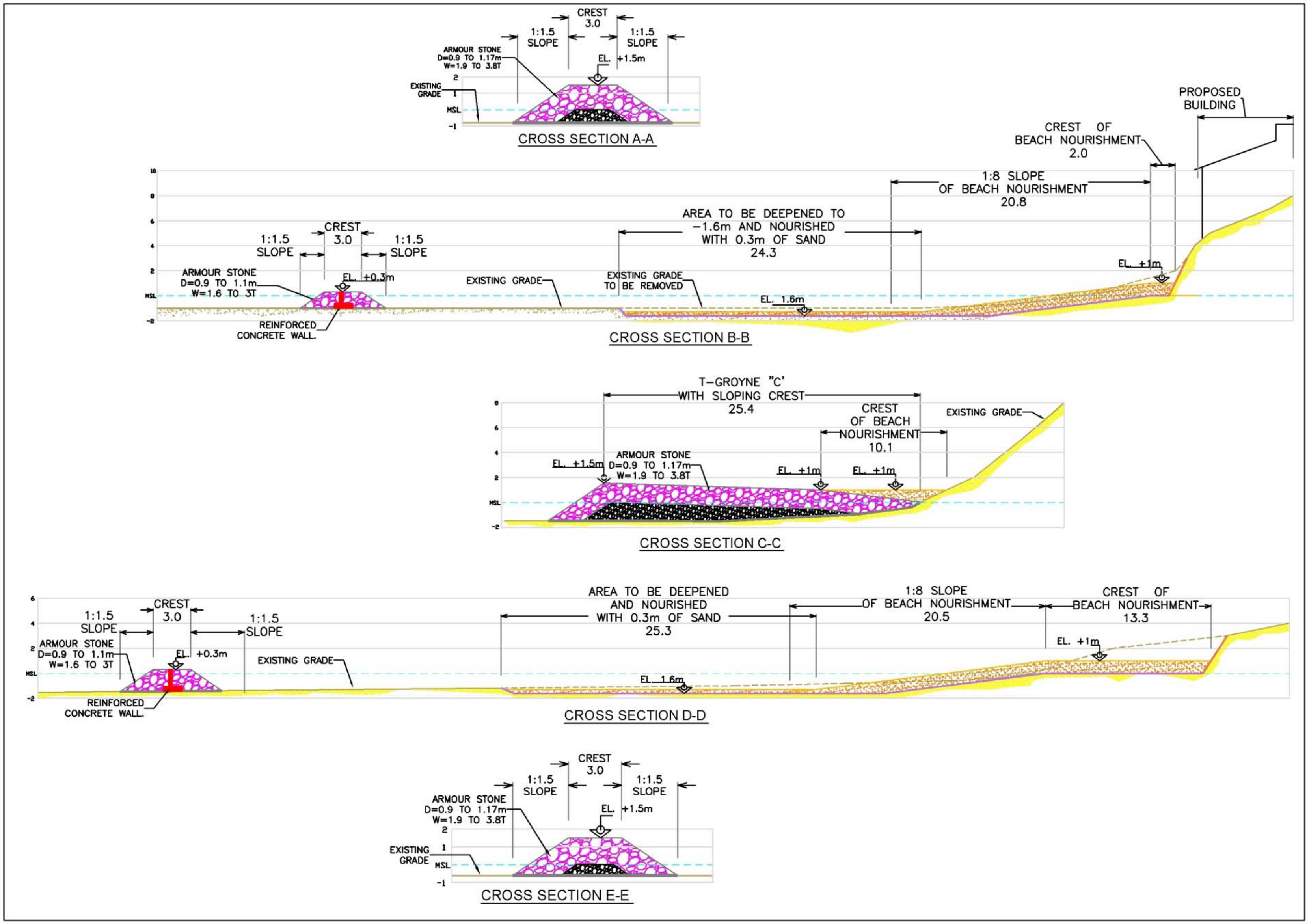



**Figure 6.2** **Typical cross-section of the proposed breakwater/groynes and beach nourishment**

## 6.2 Beach Response and Structure Performance during Swell Events

The method and model set up previously used to develop an understanding of the existing conditions was also adopted to evaluate the effectiveness of the proposed breakwater/groyne system. The wave, hydrodynamic and morphological modeling investigated the existing and proposed shoreline response under a typical two-day swell from the northwest and two-day swell from the north. Table 6-1 presents the data from deep water swell events used as input to the MIKE 21 coupled model.

Given the incident wave climate, a range of alignments and dimensions of the proposed structures were investigated, leading to a total of height preliminary concept solutions. Results for the final recommended design solution (described in the preceding Section6.1) are represented following, while results for the other investigated concepts are shown in Appendix C. Overall, these results helped in determining the critical aspects in the design where scouring and/or deposition could be expected, and the likely impacts of the proposed solution on the shoreline of the Grand Palladium extension.

**Table 6-1 Details of selected swells**

| Details at Swell Peak | Date &Time (mmm,dd,yyyy) (hh) | Wave Height (m) | Wave Period (s) | Wave Direction (Deg) |
|---|---|---|---|---|
| Typical North Swell | Jan,18, 2005 09am | 1.9 | 10 | 40 |
| Typical North Swell | Nov,22, 2006 03pm | 2.2 | 7.5 | 50 |

Figure 6.3 represents the resulting *significant wave heights and directions* at the peak of the 2005 swell event (top) that occurred on January 18th 2005 at 09am, and the 2006 swell event (bottom) that occurred on November 22nd 2006 at 3pm, with comparison between the existing scenario (left), and proposed option (right). Observations from the plots are as follows:

- A maximum wave height of 0.7m occurs just seaward of the northern breakwater and 0.5m seaward of the southern breakwater. Waves penetrate to the toe of the northern emergent groyne between both breakwaters with a height of 0.6m, however this is not due to the effect of the proposed structures as the wave heights are also 0.6m at this same location in existing conditions.
- Leeward of the proposed breakwaters, the wave height is reduced from 0.7m to 0.3m leeward of the northern breakwater and from 0.65m to 0.2m just leeward of the southern breakwater, which demonstrates the effectiveness of the proposed breakwaters in reducing wave heights by at least 40% and up to 70%.
- The November 2006 swell generates slightly higher waves at the project site compared to the January 2005 swell. This concurs with the fact that the November 2006 swell was a stronger swell event.

- At the project site, waves are found to range between 0.1m and 0.3m in the lee of the proposed structures and between 0.6m and 0.4m within the gap between the structures. The higher wave heights observed within the structure gaps are not a function of the structures themselves, as those values were also observed under existing conditions
- No downdrift impact to the adjacent property was observed with the structures in place.

Overall, the proposed structures are efficient in reducing the waves at the project site by up to 50%. The efficiency of the emergent structures is slightly dependent on the swell strength with a larger sheltered area in the lee of the structures observed with the moderate swell of January 2005.

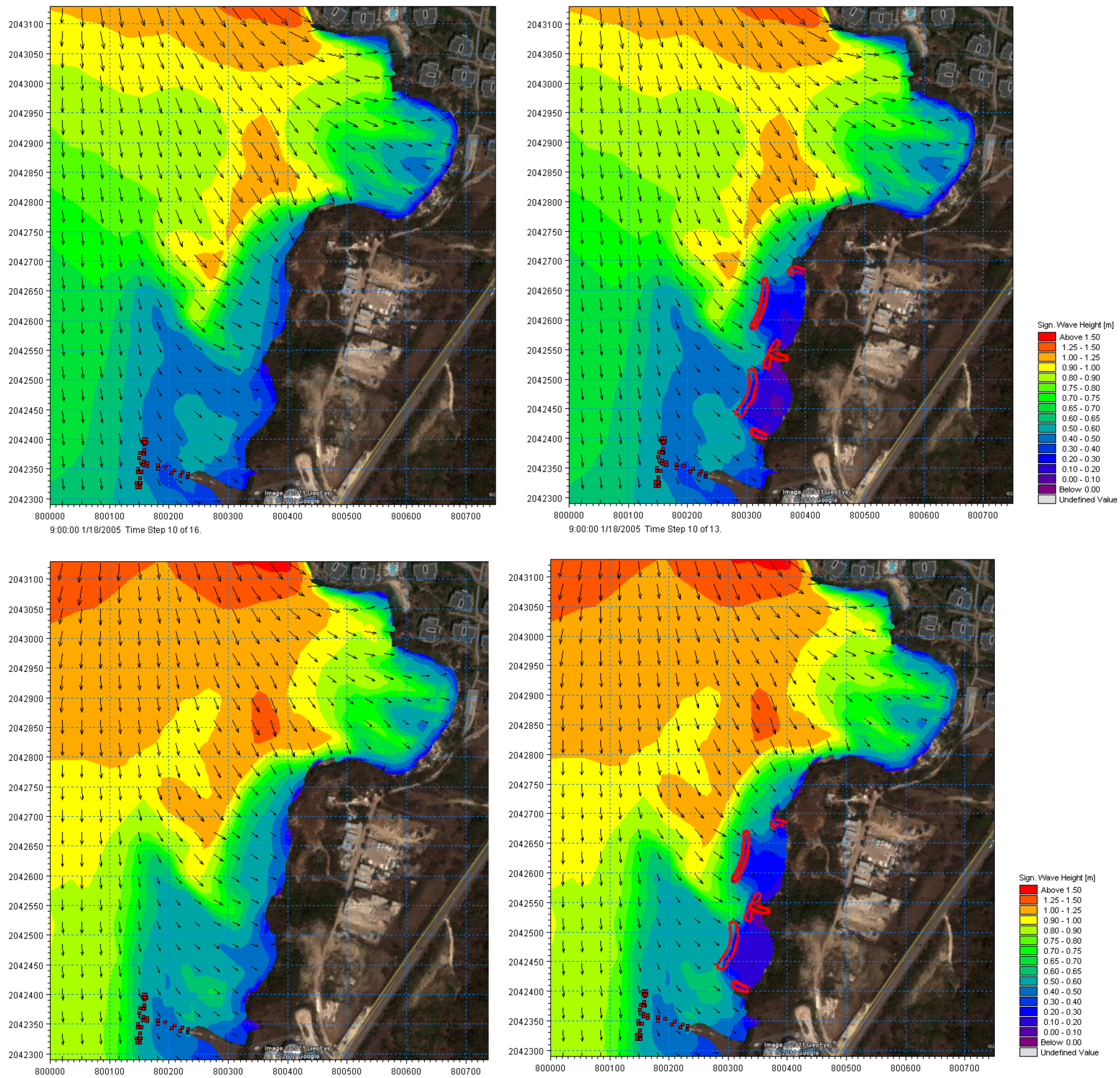


**Figure 6.3 Significant wave height and direction at the peak of the winter 2005 swell event(top) and winter 2006 swell event(bottom) with existing (left), and proposed option (right)**

Figure 6.4 represents the resulting *current speed and directions* at the peak of the 2005 swell event (top) and 2006 swell event (bottom), with comparison between the existing scenario (left), and proposed option (right). Observations from the plots are as follows:

- Currents flow toward the south in the nearshore of the project site, which coincides with the chosen swell direction coming from the north to north-northwest sector.
- Maximum currents speeds (top, left) exceed 0.5m/s in the area of the existing reefs during the 2005 swell event. However, currents appear to be slightly smaller with a maximum up to 0.4m/s over the existing reef with the 2006 swell. Currents at the project site are small: of the order of 0.3m/s along the northern beach cove; to 0.1m/s along the southern beach coves; and with maximum up to 0.4m/s along the headland that separates the two beach coves.
- With the proposed breakwaters partially emergent there appears to be higher current speeds flowing through the structures with speeds up to 0.6m/s through the southern breakwater and 0.4m/s through the northern breakwater. There is a reduction of current speeds to almost nil in the lee of the north sector of the northern breakwater and the southern lee of the southern breakwater (bottom, right).
- Leeward of the northern breakwater, currents are found to range from 0.2m/s to almost nil along the proposed beach compared to the maximum of 0.3m/s found in existing conditions. Leeward of the southern breakwater currents are even smaller than in the lee of the northern breakwater and are found to range between 0.1m/s and almost nil up to the project shoreline along the southern beach cove. Currents at the toe of the proposed groyne that separates both beach coves is reduced from 0.4m/s to 0.25m/s, which will be sufficient to allow for natural flushing of the beach coves.
- These observations demonstrate the effectiveness of the proposed breakwater in reducing maximum alongshore current speeds from 30% leeward of the proposed northern breakwater, to 40% leeward of the southern breakwater, and up to 100% along both of the proposed beach coves. Overall, currents are also reduced over the existing reef.
- Note that there is no increase in current speeds between the breakwaters; this shows the effectiveness of the breakwaters in allowing currents to flow around them without the formation of rip currents. An increase in current speeds around nearshore structures often leads to partial scouring around the structure, which is seen as a negative impact.

These observations demonstrate that the proposed structures will decrease alongshore current speeds while still allowing for natural flushing of the bay to occur and promote a stable beach and safe swimming conditions along both of the proposed beach coves. The fact that stronger currents were observed during the moderate January 2005 long period north-westerly swell compared to the stronger but shorter period November 2006 northerly swell demonstrates that nearshore currents are very dependent on the wave period and the orientation of the offshore wave angle.

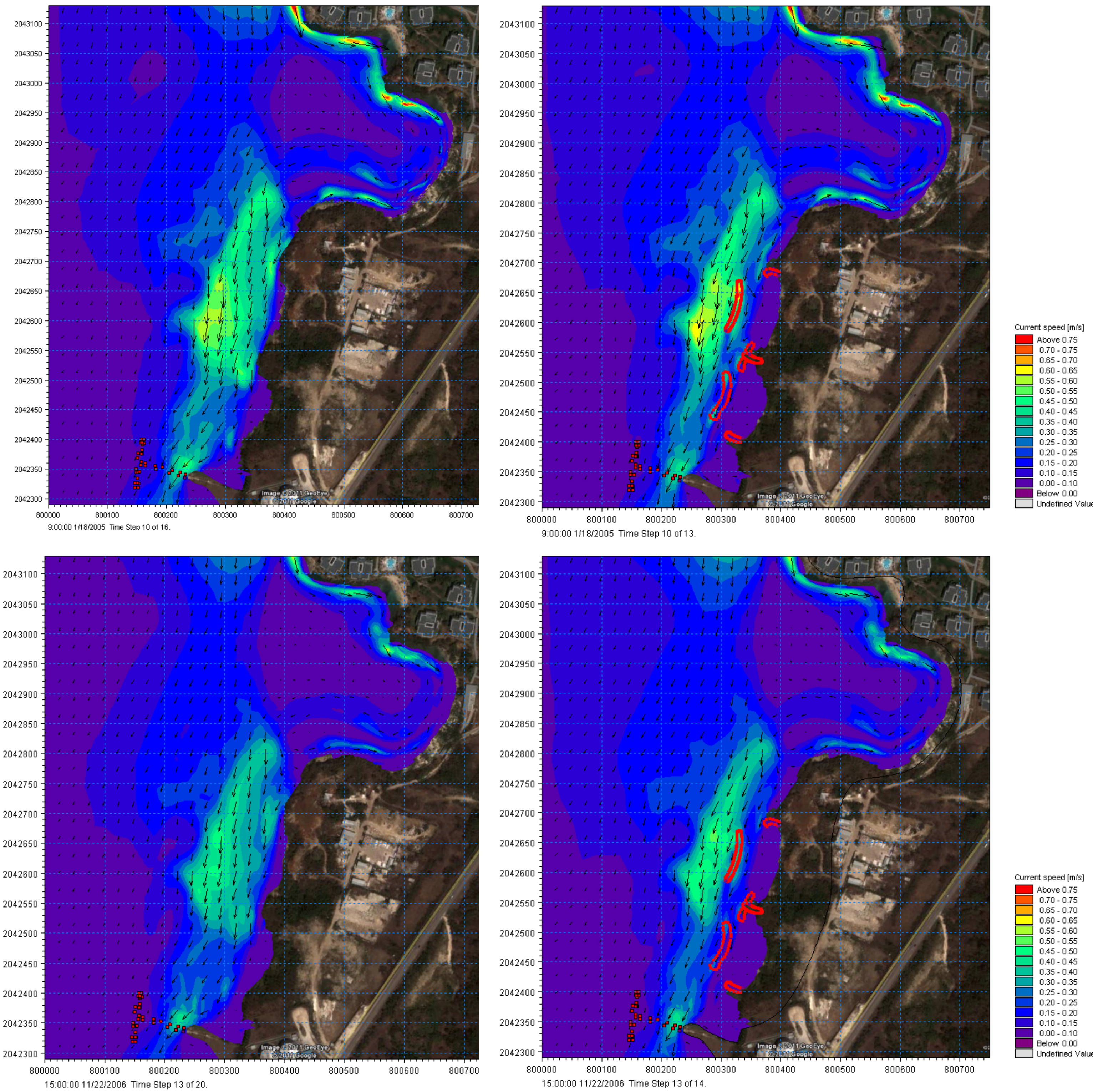



**Figure 6.4 Current speed and direction at the peak of the winter 2005 swell event(top) and winter 2006 swell event(bottom) with existing (left), and proposed option (right)**

Figure 6.5 represents the resulting *bed level change and magnitude of directions* at the peak of the 2005 swell event (top) and 2006 swell event (bottom), with comparison between the existing scenario (left), and proposed option (right). The plots show the intensity and locations where sediment is moving and therefore identifies important sediment pathways. Observations made from the plots are as follows:

- Under existing conditions, the November 2006 swell (bottom left) comes from a more northerly direction and generates cross-shore sediment transport along both coves while the January 2005 event, coming from a more north-westerly direction, was responsible for alongshore sediment transport and drift to the southwest (top, right).
- In a general sense, it can be seen that along the beach at the project site, sediment has been eroded (approximately 0.4m) from the high berm and has been deposited seaward of the beach slope (top/bottom left side).
- An important inflow of sediment appears to be over the reef in a south-west direction. Sand is deposited seaward of the existing reef, which could be an indication of both cross-shore sediment transport over the reef and alongshore sediment from the Grand Palladium existing beach just north of the project site.
- With the breakwaters in place (top/bottom, right side), the intensity of the potential sediment movement is substantially decreased in the lee of the proposed structures but the pathways and trend of sand movement remains the same. The mechanism of erosion and deposition is reduced by almost 100% along both of the proposed beach coves. The implementation of the structures and beach nourishment seem to cause localized scouring of 0.2m south of both swimming areas and at the southern toe of the northern emergent groyne.

These observations suggest that the proposed emergent structures are very efficient in reducing the swell-induced erosion and contribute to beach enhancement along both beach coves.

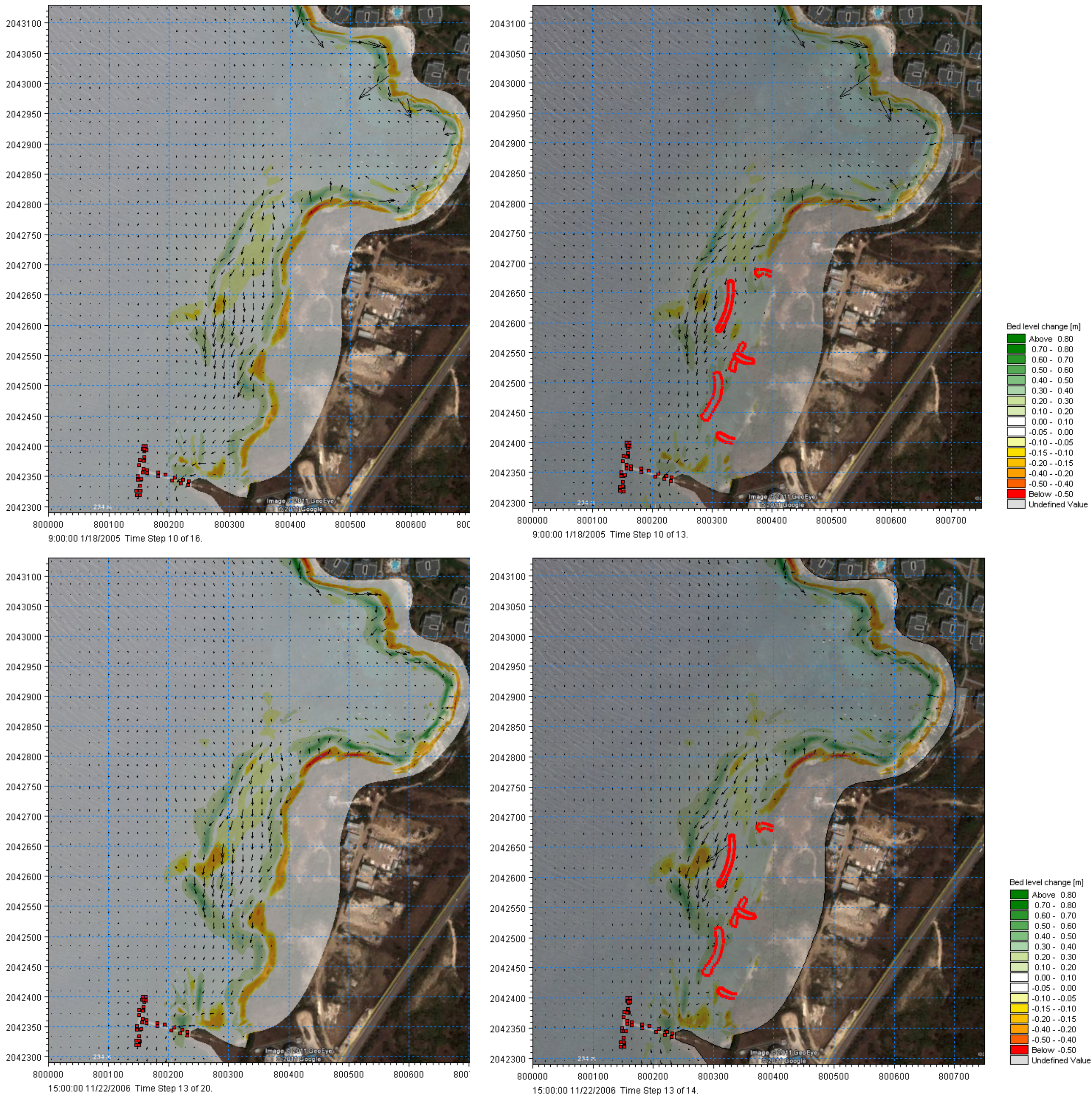



**Figure 6.5 Sediment transport and magnitude of direction at the peak of the winter 2005 swell event(top) and winter 2006 swell event(bottom) with existing (left), and proposed option (right)**

## *6.3 Beach Stability after Hurricane Dean*

The performance of the proposed option and its ability to hold sand in place was simulated for the passage of Hurricane Dean (2007). The wind velocities, pressure field and storm surge components along the hurricane track were extracted from the HurWave database and input every 15 minutes into a large scale wave model to generate wave conditions over the entire model domain.

The wave climate at the project site was developed using a spectral wave model to simulate the generated waves as they approach from offshore to the east and north coasts and wrap around the island to reach the project site. The model was set up on a triangular mesh, represented in Figure 6.6, encompassing the entire island of Jamaica and extending out to water depths greater than 1000m and a distance of more than 100km from the coastline.

The time series of wave conditions calculated in the large scale model were input along the boundaries of a smaller scale model that was used to compute waves, hydrodynamic and morphological changes for both the existing and proposed options. Figure 6.6 shows the computed hurricane wave heights over the large-scale mesh, while Figure 6.7 shows the wave inputs into the small scale mesh over the existing (a) and proposed option(b) in order to compute the corresponding current speeds and bed level changes as Hurricane Dean passed from east to west of Jamaica in August 2007.

Results show that the spatial distribution of wave heights is unidirectional, coming from the north at the project site with values up to 1.6m at the location of the proposed breakwaters. With the breakwater in place, hurricane waves in that area are reduced from 1.6m seaward to 0.6m shoreward of the breakwater; this demonstrates the efficiency of the proposed breakwater in reducing wave energy for this event by over 60%.

Figure 6.8 (top) shows the spatial distribution of current speeds and bed level changes with 20 hours of hurricane conditions. Currents are directed towards the south and vary from an average of 1.8m/s, to a low of 1.0m/s with a maximum of 2.2m/s south of the existing reef. With the structures in place, currents are reduced in the lee of the proposed structures with a maximum reduction from 1.6m/s to 0.4m/s in the lee of both breakwaters. The area over which the current is reduced is greater in the southern beach cove and currents are reduced to almost zero along the shoreline. Overall it can be seen that the proposed structures promote a sheltered and calmer area in their lee up to the project shoreline.

The resulting bed level changes [Figure 6.8 (bottom)], show that alongshore sediment transport to the south (over the existing reef) dominates. After 20 hours of hurricane conditions the high berm of the beach is eroded by up to 0.5m and seems to be deposited just offshore of the beach slope in a similar manner as during the swell condition, however the magnitude (shown by arrows) of cross-shore deposition is very small. With the proposed structures in place the high berm of the beach is still being eroded, with sand deposited further down the beach slope, however the amount of sediment lost is significantly reduced, with values up to 0.2m along the northern beach cove and up to 0.35m along the southern beach cove.

Results indicate that localized erosion is generated during a hurricane however the breakwaters contribute to maintaining the beach shape in an acceptable condition without any downdrift impacts.

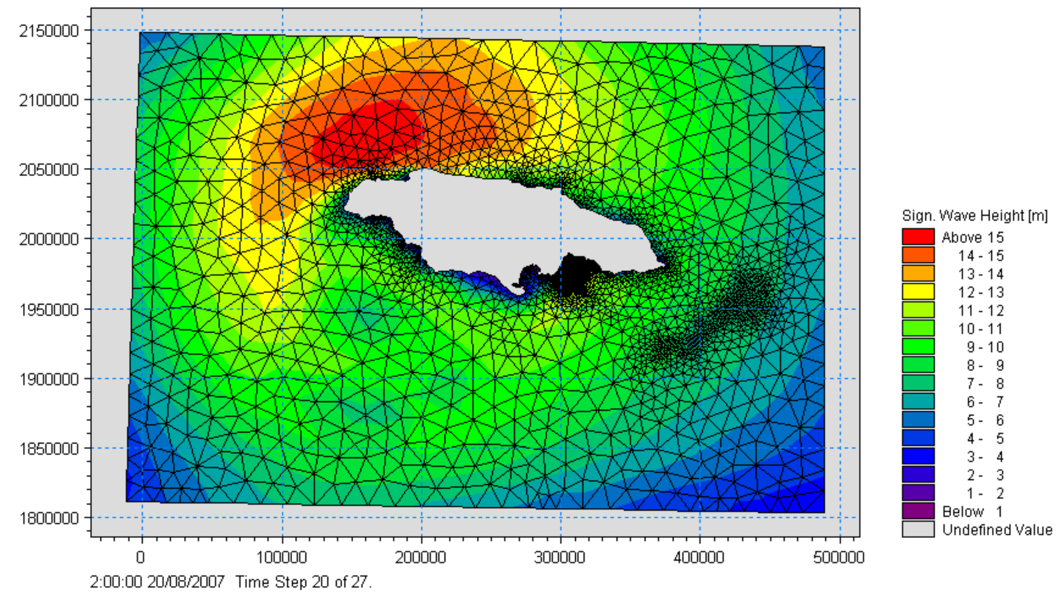



**Figure 6.6** Large scale triangular finite element mesh encompassing the island of Jamaica

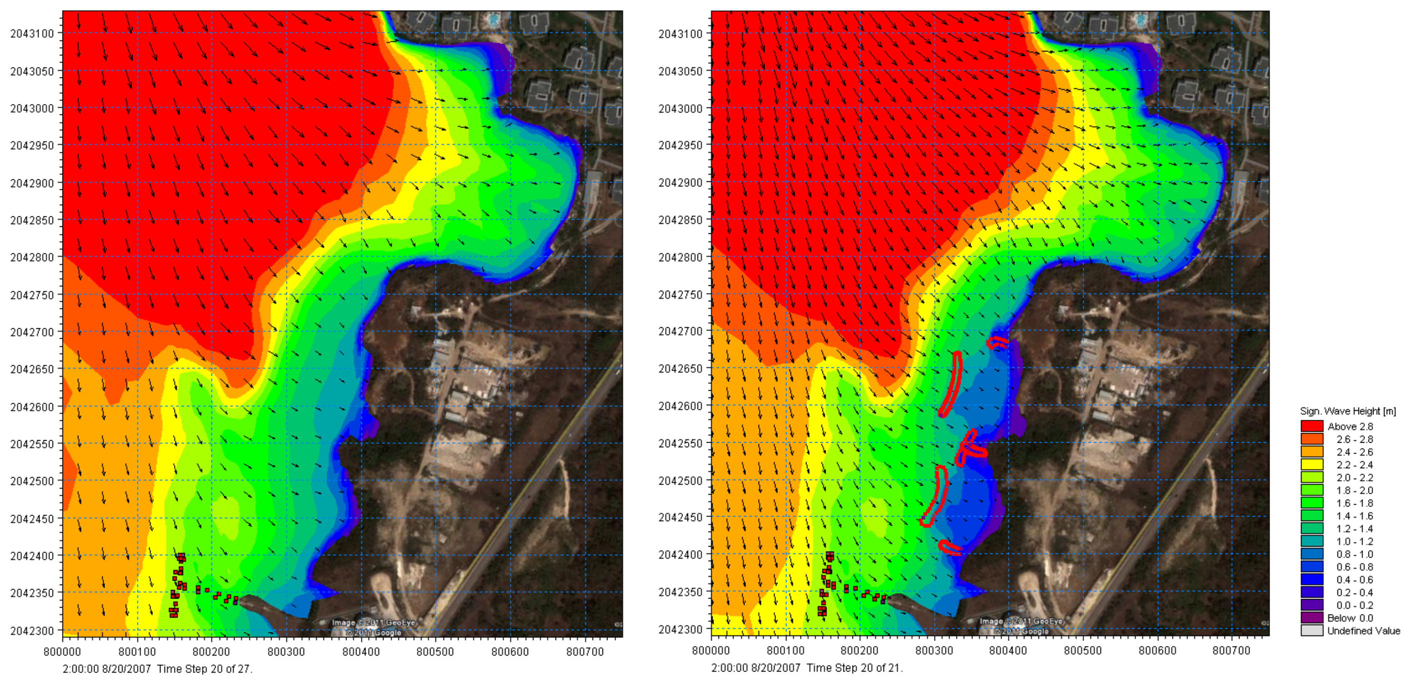



**Figure 6.7** Simulated wave height at the project site after 20 hours of Hurricane Dean conditions over the nested model with details for(a) existing and (b)proposed design option

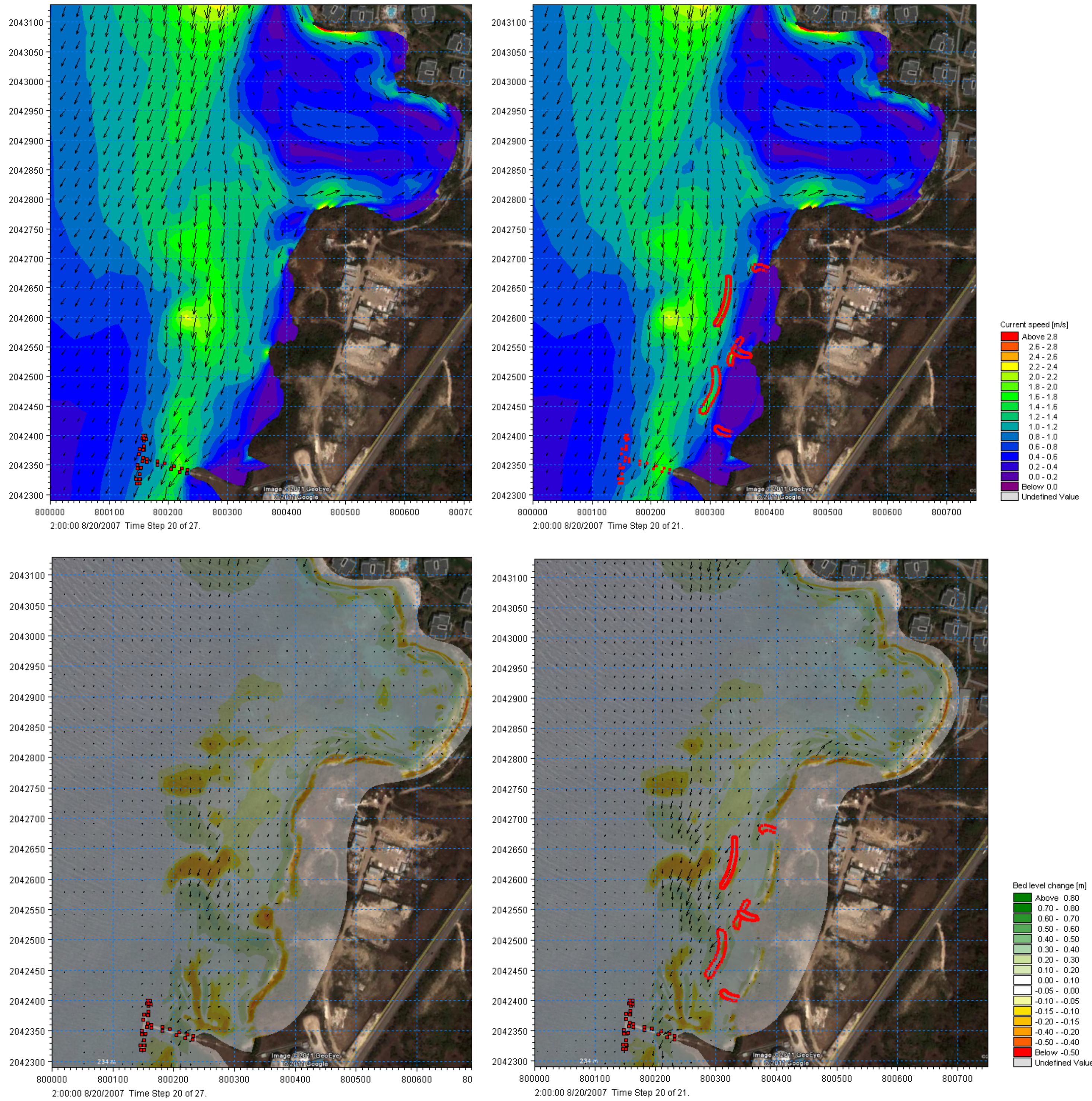



**Figure 6.8 Current speed/direction and sediment transport/magnitude of direction after 20 hours of Hurricane Dean conditions with existing condition (left) and proposed design option (right)**

## *6.4 Structural Stability*

This section describes the analysis carried out to determine the stability of the proposed structures using proper stone sizes and, if necessary, core and armour layer thickness. Using the results of the wave modeling, the structures (breakwater and groyne) were designed to provide adequate wave sheltering to create a safe swimming area and to encourage accretion of sand along the project shoreline. The use of armour stone is proposed to provide protection against wave forces for the structures, which should be designed to withstand the 1 in 50-year hurricane condition.

### 6.4.1. Stone size for Partially Emergent Breakwater

The structures were designed using an in-house program for determining armour stone sizes from the design wave conditions and on work by van der Meer, Pilarczyk and Hudson.

A sensitivity analysis was carried out on the water levels at the structure to determine the wave height and water level combination with the greatest impact on the structure. It was found that waves occurring at the shallowest water depths generated the most force on the structure and therefore this condition was adopted as being the worst-case scenario that the armour stones would have to sustain.

The design worksheet used to calculate the armour stone size for the partially emergent breakwaters(Figure 6.9) shows that stones with a density of 2400 kg/m$^3$ and a mass from approximately 1100 - 2200kg ($D_{50}$ of 0.77m to 0.97m) would be required to sustain only minimal damage during the 50-year storm event.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | STATICALLY STABLE SUBMERGED BREAKWATERS | | | | | | | |
| Wave Conditions | | Symbol | | units | | | | |
| | Wave Height | Hs* | 1.24 | m | Results from Kamphuis Formula: | | | |
| | Wave Period | Tp | 16 | s | Kamphuis, J.W., 1991. Incipient Wave Breaking, Coastal Eng., 15: 185-203 | | | |
| | Wave Length | Lp | 61.11 | m | | | | |
| Stone Parameters | | | | | | Rock Gradation factor | | |
| | Stone Density | rho a | 2400 | kg/m^3 | | | 1.25 | |
| | Water Density | rho w | 1025 | kg/m^3 | | Range | | |
| | Damage Level | S | 2 | - | | Lower | Upper | |
| | Stone Size | Dn50 | 0.87 | m | | 0.77 | 0.97 | m |
| | Stone Weight | W50 | 1574.87 | kg | | 1106 | 2160 | kg |
| Structure | | | | | | | | |
| | Water Depth | h | 1.5 | m | | | | |
| | Crest height | h'c | 2 | m | | | | |
| | Submergence | | -0.5 | | measured positive down from Water Line to Crest | | | |
| | | | | | Emergent struct has a Negative Submergence | | | |
| Calc's | | | | | | | | |
| | Stability Number | Ns* | 3.89 | - | van der Meer and Pilarczyk, 1990, "Stability of Low Crested and Reef Breakwaters", Proceed 22nd Coastal Conference, Vol.2, pp.1375-1388, 1990 | | | |

**Figure 6.9 Design table used to determine armour stone size for the proposed slightly emergent breakwater**

In order to keep the crest elevation as close to mean sea level as possible, while ensuring the effectiveness of the breakwater, an internal concrete wall will need to be included in the design of this structure. This essentially eliminates permeability through the breakwater and therefore reduces the transmission of waves through the voids in the armour stone layer. As a result, larger stone sizes are necessary to withstand the wave forces at the structure. The van der Meer formula for Static Stability of Rocks (Figure 6.10) was used to calculate the armour stone sizes, which indicates that stones of 1600 - 3200kg in mass ($D_{50}$ of 0.9 to 1.1m) are necessary to achieve stability for this arrangement. It should be noted that a damage level of 1.1 has been used, and this represents the "Start of Damage", with less than 2% of stones displaced.

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**Armour Stone Design Worksheet**

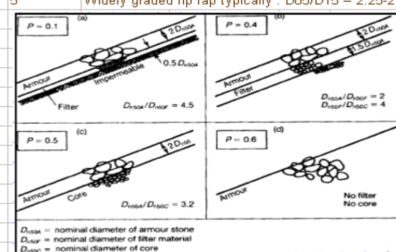
| | |
|---|---|
| Client: | Fiesta Jamaica,.Ltd |
| Project: | Grand Palladium Beach Development |
| Designer: | EM |
| Date: | Jun-11 |

| Design Parameters (Structure) | | | Design Parameters (Wave) | | |
|---|---|---|---|---|---|
| armour unit density (phor) | 2400 | kg/m3 | design wave height (Hs) | 1.24 | m |
| water density (phow) | 1025 | kg/m3 | design wave period (Tp) | 16.00 | s |
| buoyant density (s) | 1.34 | | mean wave period (Tm) | 13.33 | s |
| structure slope (cot alpha) | 1.5 | >=1.5 | duration of storm | 8 | hrs |
| stability coefficient (Kd) | 2 | - | no. of waves (N) | 2160 | no |
| no. of units displaced per Dn, Nod | 1.1 | - | wave steepness | 0.004 | - |
| damage number (Sd) | 1.76 | needs no repair | iribarren no. | 9.97 | 9.9701 |
| perm of structure (P) | 0.1 | - | critical iribarren no. | 4.54 | wave is surging |
| | | | wave attack angle (deg) | 350.00 | |
| | | | | 10 | |

| filter and core layer specifications | |
|---|---|
| User specified ratio? *(yes/no)* | yes |
| User specified value : | 2 |
| D50A/D50F | 2 |
| D50F/D50C | 2 |

**Notes**

| | |
|---|---|
| 1 | Rounded rock is less stable than equant rocks |
| 2 | Damage generally greater than predicted by |
| | van der Meer for widely graded armouring |
| 3 | Narrow graded armour typically : D85/D15 = 1.25-1.5 |
| 4 | Widely graded armour typically : D85/D15 = 2 - 5 |
| 5 | Widely graded rip rap typically : D85/D15 = 2.25-2.5 |




**Rock Sizing Table**
**Van der Meer - Rock (Static Stability)**

| Primary Armour | | | Range (Rock Gradation factor 1.25) | | Filter Layer | | | Range (Filter Gradation factor 1.25) | | Core Material for Rock | | | Range (Core Gradation factor 3) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Lower | Upper | | | | Lower | Upper | | | | Lower | Upper |
| **Dn50** | **0.81** | **m** | 0.72 | 0.91 | D50F | 0.41 | m | 0.36 | 0.45 | D50F | 0.20 | m | 0.08 | 0.25 |
| **M50** | **1298** | **kg** | 912 | 1781 | M50F | 162 | kg | 114 | 223 | M50F | 20 | kg | 1.4 | 39 |
| ***D5n0 max*** | ***1.00*** | **m** | *0.89* | *1.11* | *D50F max* | *0.50* | m | *0.44* | *0.55* | | | | | |
| ***M50 max*** | ***2376*** | **kg** | *1668* | *3259* | *M50F max* | *297* | kg | *209* | *407* | | | | | |
| **Layer thickness** | **2.2** | **m** | | | | | | | | | | | | |

**Figure 6.10 Design table used to determine armour stone size for the proposed emergent breakwaters**

### 6.4.2. Stone Size for Emergent Groyne

The wave height for the 1 in 50-year event was found to be 2m in the area of the proposed groyne. The emergent groyne was designed based on the same in-house program used to determine armour stone sizes from the design wave conditions and on work by van der Meer, Pilarczyk and Hudson.

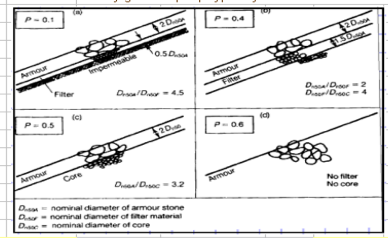
Three different design methods were computed and compared with varying input conditions: (1) van der Meer, (2) van der Meer method accounting for a thin armour layer, and (3) the Hudson formula. The computations of the van der Meer formula for Static Stability of Rocks (Figure 6.11) showed that stones of 1900kg to 3800kg in mass ($D_{50}$ of 0.94 to 1.17m) are required to sustain only minimal damage during the 50-year storm event.

**Armour Stone Design Worksheet**

| | |
|---|---|
| Client: | Fiesta Jamaica,.Ltd |
| Project: | Grand Palladium Beach Development |
| Designer: | EM |
| Date: | Jun-11 |

| Design Parameters (Structure) | | | Design Parameters (Wave) | | | |
|---|---|---|---|---|---|---|
| armour unit density (phor) | 2400 | kg/m3 | design wave height (Hs) | 2.00 | m | |
| water density (phow) | 1025 | kg/m3 | design wave period (Tp) | 16.00 | s | |
| buoyant density (s) | 1.34 | | mean wave period (Tm) | 13.33 | s | |
| structure slope (cot alpha) | 1.5 | >=1.5 | duration of storm | 8 | hrs | |
| stability coefficient (Kd) | 2 | - | no. of waves (N) | 2160 | no | |
| no. of units displaced per Dn, Nod | 1.5 | - | wave steepness | 0.007 | - | |
| damage number (Sd) | 2.40 | intermediate damag | iribarren no. | 7.85 | 7.8505 | |
| perm of structure (P) | 0.5 | - | critical iribarren no. | 4.08 | | wave is surging |
| | | | wave attack angle (deg) | 45.00 | | |
| | | | | 315 | | |

| Notes | |
|---|---|
| 1 | Rounded rock is less stable than equant rocks |
| 2 | Damage generally greater than predicted by |
| | van der Meer for widely graded armouring |
| 3 | Narrow graded armour typically : D85/D15 = 1.25-1.5 |
| 4 | Widely graded armour typically : D85/D15 = 2 - 5 |
| 5 | Widely graded rip rap typically : D85/D15 = 2.25-2.5 |




| filter and core layer specifications | |
|---|---|
| User specified ratio? *(yes/no)* | yes |
| User specified value : | 2 |
| D50A/D50F | 2 |
| D50F/D50C | 2 |

**Rock Sizing Table**
**Van der Meer - Rock (Static Stability)**

Rock Gradation factor 1.25 · Filter Gradation factor 1.25 · Core Gradation factor 3

| Primary Armour | | | Range Lower | Range Upper | Filter Layer | | | Range Lower | Range Upper | Core Material for Rock | | | Range Lower | Range Upper |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Dn50** | **0.68** | **m** | 0.61 | 0.76 | D50F | 0.34 | m | 0.30 | 0.38 | D50F | 0.17 | m | 0.07 | 0.21 |
| **M50** | **767** | **kg** | 539 | 1053 | M50F | 96 | kg | 67 | 132 | M50F | 12 | kg | 0.9 | 23 |
| ***D5n0 max*** | ***1.06*** | **m** | *0.94* | *1.17* | *D50F max* | *0.53* | m | *0.47* | *0.59* | | | | | |
| ***M50 max*** | ***2834*** | **kg** | *1990* | *3887* | *M50F max* | *354* | kg | *249* | *486* | | | | | |
| **Layer thickness** | **2.3** | **m** | | | | | | | | | | | | |

**Figure 6.11 Design table used to determine armour stone size for the proposed emergent groyne**

### 6.4.3. Transmission Coefficient of the Proposed Breakwaters

The transmission coefficient is a parameter used to measure how effective breakwater structures are in reducing the wave energy in their lee, and thus sediment transport. Careful design is required in setting the transmission coefficient of the proposed breakwater in order to adequately represent the wave energy passing the structure without causing the formation of a tombolo or initiating downdrift erosion. Values for transmission coefficients can range from 0.0, meaning that all the wave energy is blocked and no transmission of wave energy occurs, to 1.0, where all the wave energy passes through the structure. A lower transmission coefficient means that less wave energy is transmitted and the wave heights in the lee of the structure will be smaller. The reduction of wave energy in the lee of breakwaters usually causes the shoreline to accrete, as this allows sediments to be deposited, causing the beach to build seaward. Figure 6.12 shows the typical beach response to the placement of a breakwater. Figure 6.12 (A) shows a salient, which is a partial build-up of sand behind the breakwater while B shows a tombolo formation, which occurs when the beach becomes connected to the structure.

The transmission coefficient of the proposed breakwaters was obtained using the formula from the US Army Corps of Engineers, WES, Vicksburg, Miss, 44p., 1987 and the Technical Report CERC-87-7, Ahrens, "Characteristics of Reef Breakwaters".

The crest elevation at 0.3m above MSL was adopted to decrease the permeability of the structures while reducing their footprint and reduce the area of seagrass and natural habitat to be excavated. It was found that the transmission should be set to 0.34 to represent the fact that the structure is emergent. The design worksheet used to calculate the required transmission coefficient for the emergent breakwater is shown in Figure 6.13. Numerical model results showed that, with a structure set at 0.3m above MSL, the wave height of 1.31m seaward of the proposed breakwater was transmitted through the breakwater with a resulting average wave height of 0.45m in the lee of the structure.

Increasing the crest elevation of a breakwater above mean sea level significantly improves its effectiveness in breaking the waves and reducing wave energy in its lee, however, this creates a greater visual imposition. The water depth at the location of the proposed breakwater is very shallow and the footprint of a submerged structure would have to be much wider to obtain the same wave sheltering efficiency as an emergent structure. For this project, however, increasing the footprint of the structure is not recommended as the benthic environment is very sensitive. In this case the solution is to increase the emergence of the structure rather than widen the structure and increase the area of seabed excavation.

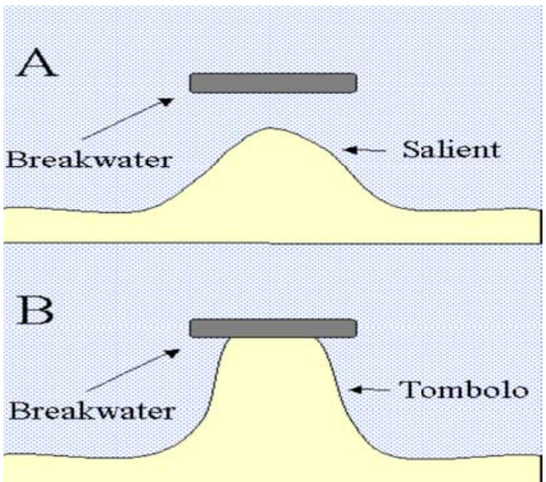



**Figure 6.12 Beach responses to breakwaters (Salient - A and Tombolo - B)**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **REEF BREAKWATERS: Transmission Coefficients** | | | | | | | | | | |
| **Wave Conditions** | | Symbol | | units | | | | | | |
| | Wave Height | Hs | 1.31 | m | | | | | | |
| | Wave Period | Tp | 16 | s | | | | | | |
| | Surge | surge | 0 | m | | | | | | |
| | Wave Length | Lp | 65.97 | m | | | | | | |
| **Stone Parameters** | | | | | | | | | | |
| | Stone Density | rho a | 2400 | kg/m^3 | | | | | | |
| | Water Density | rho w | 1025 | kg/m^3 | | | | | | |
| | Stone Weight | W50 | 2300 | kg | | | | | | |
| | Stone Size | Dn50 | 0.94 | m | | | | | | |
| **Structure** | | | | | | | | | | |
| | Water Depth | water depth | 1.75 | | | | | | | |
| | Water Depth+surge | h | 1.75 | m | | Top Width | Bottom Width | Side Slope | | |
| | X-Sectional Area | At | 18.26 | m^2 | | 5 | 11.6 | 1.5 | | |
| | Crest height (end) | hc | 2.2 | m | | | | | | |
| **Calc's** | **d'Angremond, van der Meer, De Jong (ICCE 1996)** | | | | | | | | | |
| | Irrib No. | 4.7 | Ht | | | | | Ht before | Ht after | Increase |
| | Kt (Impermeable) | kt | **0.34** | - | Ht | 0.45 | m | 1.31 | 0.45 | -88% |
| | Relative Freeboard | F/Hmo | 0.36 | | | | | | | |
| | Ahrens, "Characteristics of Reef Breakwaters", Technical Report CERC-87-7 | | | | | | | | | |
| | US Army Corps of Engineers, WES, Vicksburg, Miss., 44p., 1987. | | | | | | | | | |

**Figure 6.13 Transmission coefficient required of the proposed emergent breakwater**

## 6.5 Sand Characteristics

When specifying sand for beach nourishment projects it is preferable that the sand have a mean grain size larger than the existing beach sand. If this is not the case an "overfill" ratio must be computed to account for sand that will be quickly carried away by waves. The mean grain size was found to vary between a maximum of 0.65mm along the north beach cove and a minimum of 0.18mm along the south beach cove (Table 2-1). It is therefore recommended that the sand used to enhance the beach at the Grand Palladium have a mean grain size ranging from 0.3mm to 0.5mm. In addition, the silt content should be low, ideally less than 0.5%. Higher silt content will result in cloudy water as the waves gradually clean the sand, and can create a hardened surface over time. Other characteristics, such as carbonate content and colour are generally aesthetic, and are subject to preference. In this case, however, the existing beach sand is white/brown in colour, and it is recommended that the sand placed for beach enhancement be selected to either match this or to provide a specific desired visual aesthetic.

## 6.6 Estimated Material Quantities

Estimates of the material volumes required to implement both the beach enhancement and drainage works are summarized inTable 6-2 following.

**Table 6-2 Estimated Material Quantities for the proposed beach works including sand nourishment**

**Jamaica - Grand Palladium Beach Extension**
**Preliminary Bill of Quantities**

**Date:** 14-Mar-12

| No. | Description | Quantity | Unit |
|---|---|---|---|
| 1.0 | BILL NO. 1 - GENERAL ITEMS | | |
| 1.01 | Site Supervision and Management, Variation and Additional works, Demobilization, Contract Requirements (Insurance), Material Testing, Mobilization of Equipment and Workforce, Services, Utilities and Security, Supply Turbidity Barrier, Surveying and setting out. | 1 | item |
| 2.0 | BILL NO. 2 - BREAKWATER "A" | | |
| 2.01 | Supply Boulders to site for Breakwater "A" | 481 | $m^3$ |
| 2.02 | Supply and Place Filter Fabric | 1,141 | $m^2$ |
| 2.03 | Form and place reinforced concrete sections for sea wall | 84 | $m^3$ |
| 2.04 | Install reinfoced concrete sections for seawall | 84 | $m^3$ |
| 2.05 | Place and Shape Boulders for Breakwater "A" | 481 | $m^3$ |
| 3.0 | BILL NO. 3- BREAKWATER "B" | | |
| 3.01 | Supply Boulders to site for Breakwater "B" | 700 | $m^3$ |
| 3.02 | Supply and Place Filter Fabric | 1152 | $m^2$ |
| 3.03 | Form reinforced concrete sections for sea wall | 89 | $m^3$ |
| 3.04 | Install reinfoced concrete sections for seawall | 89 | $m^3$ |
| 3.05 | Place and Shape Boulders for Breakwater "B" | 700 | $m^3$ |
| 4.0 | BILL NO. 4- T-GROYNE "B" | | |
| 4.01 | Supply Boulders to site for T-Groyne "B" | 1,046 | $m^3$ |
| 4.02 | Supply Filter material to site | 389 | $m^3$ |
| 4.03 | Place and shape filter material for T-Groyne "B" | 389 | $m^3$ |
| 4.04 | Supply and Place Filter Fabric | 1,720 | $m^2$ |
| 4.05 | Place and Shape Boulders for T-Groyne "B" | 1,046 | $m^3$ |
| 5.0 | BILL NO. 5 - GROYNE "A" | | |
| 5.01 | Supply Boulders to site | 305 | $m^3$ |
| 5.02 | Supply Filter material to site | 62 | $m^3$ |
| 5.03 | Place and shape filter material for Groyne "A" | 62 | $m^3$ |
| 5.04 | Supply and Place Filter Fabric | 305 | $m^2$ |
| 5.05 | Place and Shape Boulders for Groyne "A" | 305 | $m^3$ |
| 6.0 | BILL NO. 6 - GROYNE "C" | | |
| 6.01 | Supply Boulders to site | 322 | $m^3$ |
| 6.02 | Supply Filter material to site | 50 | $m^3$ |
| 6.03 | Place and shape filter material for Groyne "C" | 50 | $m^2$ |
| 6.04 | Supply and Place Filter Fabric | 371 | $m^3$ |
| 6.05 | Place and Shape Boulders for Groyne "C" | 322 | $m^3$ |
| 7.0 | BILL NO. 7- SEABED AND LAND EXCAVATION | | |
| 7.01 | Coral and Seagrass Transplantation | 5,921 | $m^2$ |
| 7.02 | Supply material for construction pad | 1 | LS |
| 7.03 | Excavate land side from native vegetation and rocks | 4,202 | $m^3$ |
| 7.04 | Excavate seabed as per design profiles. | 4,948 | $m^3$ |
| 7.05 | Dredging of seabed in preparation for Nourishment | 288 | $m^3$ |
| 7.06 | Cart Away material to off-site location | 9,438 | $m^3$ |
| 8.0 | BILL NO. 8- SAND NOURISHMENT | | |
| 8.01 | Supply and Stockpile Sand to site for Nourishment | 9,628 | $m^3$ |
| 8.02 | Place and grade 0.3m layer for Nourishment | 8,137 | $m^3$ |
| 8.03 | Place and grade 1m layer for Nourishment | 1,491 | $m^3$ |

# 7. Potential Environmental Impacts and Mitigation Measures

The modeling results showed that the proposed structures are effective in sheltering the shoreline during swell and hurricane events as well as promoting sand accretion along the newly created beach coves. This section of the report describes the impacts of the proposed structures on the marine environment (benthos and the physical environment). In addition, a construction methodology is proposed and its impacts outlined.

## *7.1 Benthic Survey*

To aid in the determination and evaluation of the most appropriate solutions for the beach works, a qualitative assessment of the sea floor was carried out in June 2011 in the nearshore areas of the proposed project site, through snorkelling and wading. The main objectives of the assessment were to:

- Ascertain the extent of the seagrass beds/coral life;
- Evaluate the composition and relative health of the seagrass beds/coral life;
- Determine the potential impacts of the proposed activities on the seagrass beds and any other notable benthic species.

### 7.1.1. Methodology

A desktop review of the available material was conducted prior to the field investigation.

The environmental impact assessment[3] (EIA) for the existing Phase 1 of the hotel development was reviewed. The EIA resulted in an approval and the subsequent grant of a number of Environmental Permits, Environmental Licenses, and Beach Licences. These consents included permission for the modification of the foreshore and floor of the sea, which included the need to relocate sensitive benthic organisms in connection with the deployment and maintenance of coastal encroachments and the enhancement and operation of beach bathing areas.

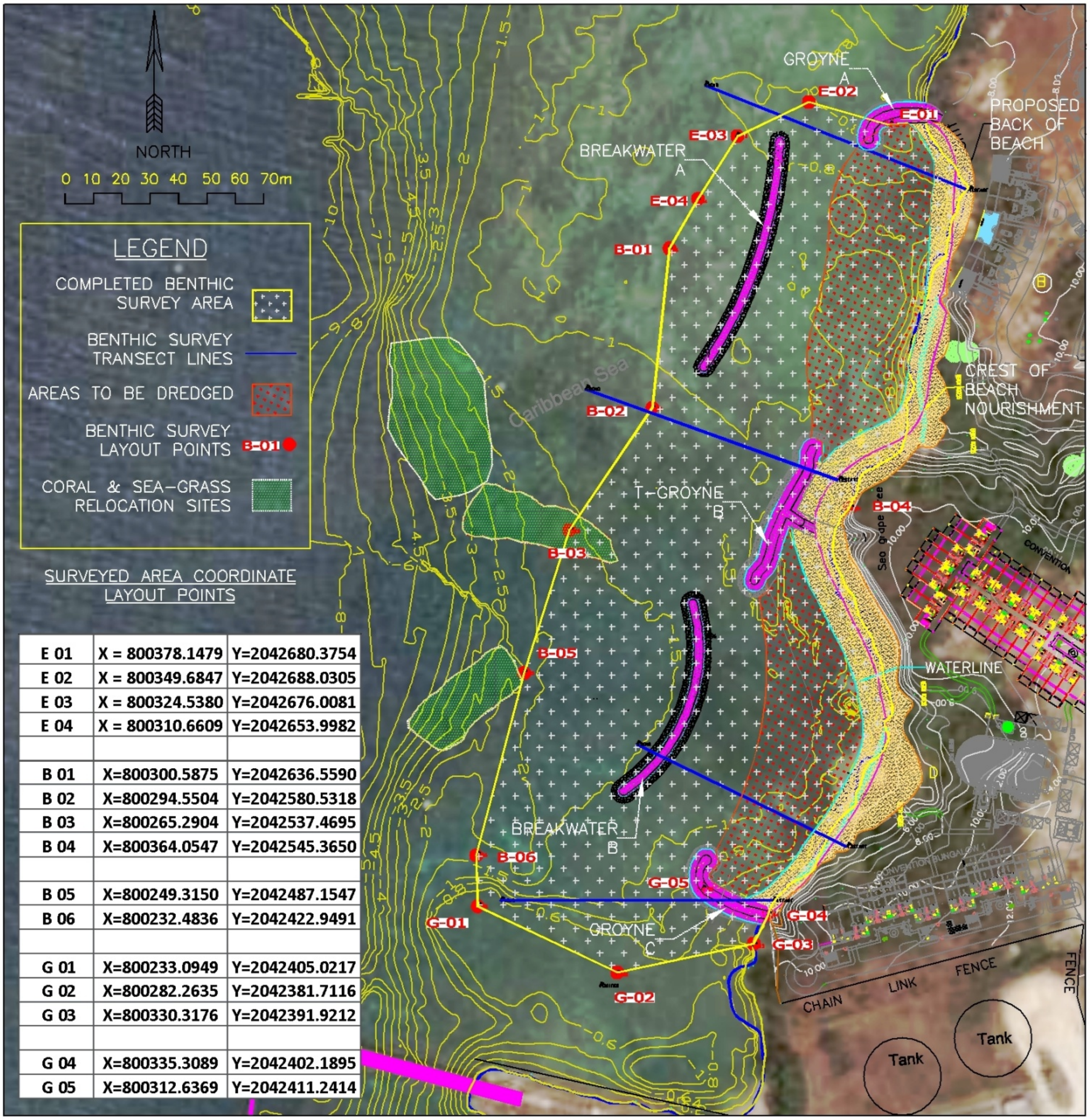
A more detailed study[4] that followed the EIA and investigated suitable relocation sites for sensitive benthic organisms that would be impacted by the coastal works was also reviewed.

The plan of the site showing the coastal works for the initial proposed option 1 (i.e., beach enhancement areas, T-groyne, finger groyne, and breakwater) were examined and Global Positioning Satellite (GPS) coordinates were extracted from the plan and uploaded to a GPS instrument.

A field trip was conducted to the site on the 11-12$^{th}$ of June 2011 and the GPS instrument was used to identify the approximate location and extent of each of the proposed coastal works on the ground.

[3] Environmental Impact Assessment for Grand Palladium Lady Hamilton Resort & Spa at Point, Hanover (December 2005) by Environmental Science and Technology Limited.

[4] Proposed Seagrass Relocation and Replanting and Coral Relocation Methodology (2006) by CL Environmental and CEAC.

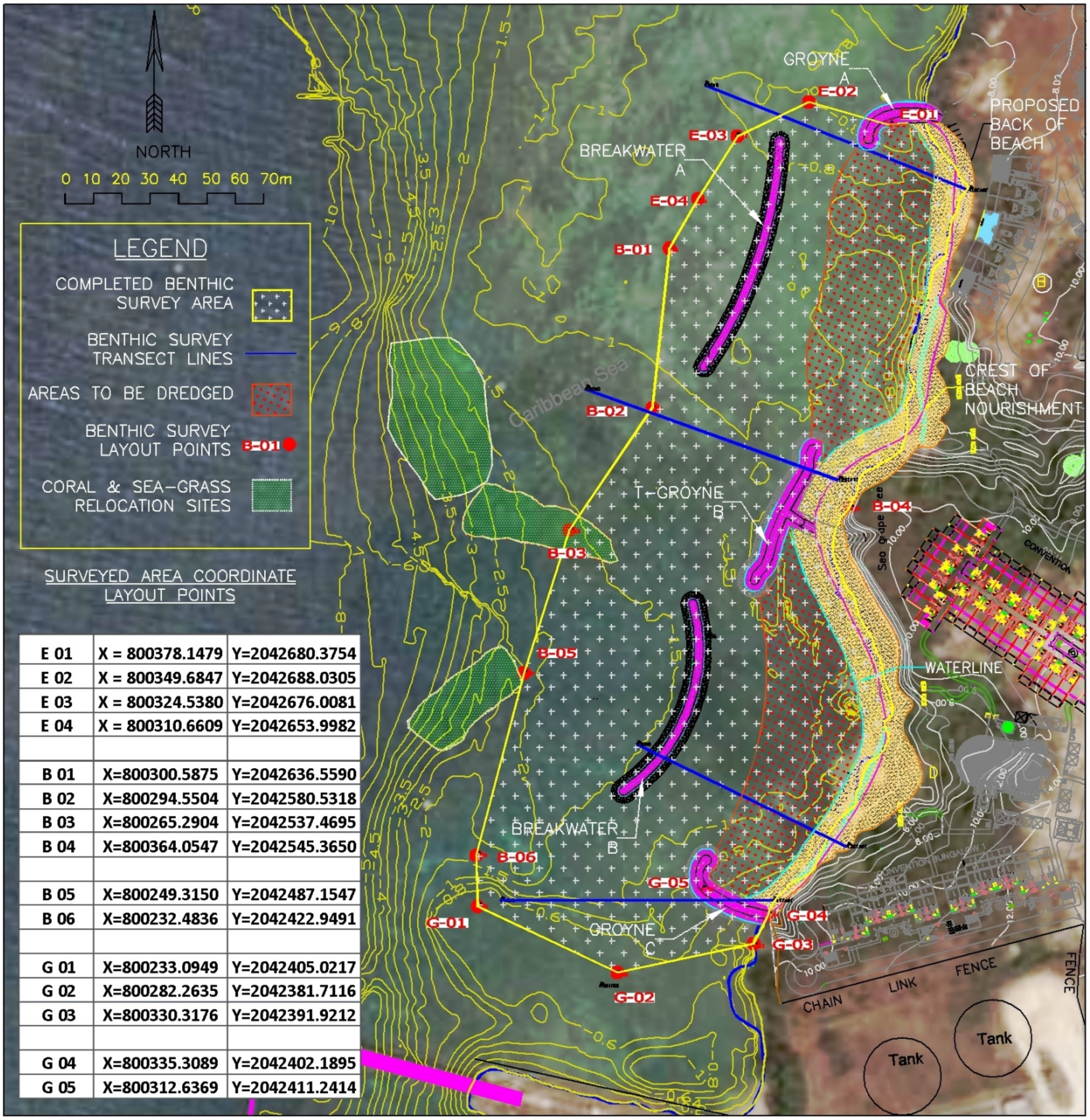



| | | |
|---|---|---|
| E 01 | X = 800378.1479 | Y=2042680.3754 |
| E 02 | X = 800349.6847 | Y=2042688.0305 |
| E 03 | X = 800324.5380 | Y=2042676.0081 |
| E 04 | X = 800310.6609 | Y=2042653.9982 |
| | | |
| B 01 | X=800300.5875 | Y=2042636.5590 |
| B 02 | X=800294.5504 | Y=2042580.5318 |
| B 03 | X=800265.2904 | Y=2042537.4695 |
| B 04 | X=800364.0547 | Y=2042545.3650 |
| | | |
| B 05 | X=800249.3150 | Y=2042487.1547 |
| B 06 | X=800232.4836 | Y=2042422.9491 |
| | | |
| G 01 | X=800233.0949 | Y=2042405.0217 |
| G 02 | X=800282.2635 | Y=2042381.7116 |
| G 03 | X=800330.3176 | Y=2042391.9212 |
| | | |
| G 04 | X=800335.3089 | Y=2042402.1895 |
| G 05 | X=800312.6369 | Y=2042411.2414 |

Figure 7.1 hereunder gives the overall extent of the benthic survey along with the points coordinates and transects used for the data collection as well as the plan for relocation of resources.

A rapid survey of the general area around the location of each of the proposed coastal works was made to quickly assess the area and to generate a species list. Transects (100m in length) were laid from shore outwards to the sea and 0.25m quadrat was used to guide the data collection process. The quadrat was placed at the beginning of each metre and observations recorded. Detailed results of the benthic analysis are included in Appendix D and E.

At the time of the field investigation the sea was calm and the visibility in the water was excellent.




| | | |
|---|---|---|
| E 01 | X = 800378.1479 | Y=2042680.3754 |
| E 02 | X = 800349.6847 | Y=2042688.0305 |
| E 03 | X = 800324.5380 | Y=2042676.0081 |
| E 04 | X = 800310.6609 | Y=2042653.9982 |
| | | |
| B 01 | X=800300.5875 | Y=2042636.5590 |
| B 02 | X=800294.5504 | Y=2042580.5318 |
| B 03 | X=800265.2904 | Y=2042537.4695 |
| B 04 | X=800364.0547 | Y=2042545.3650 |
| | | |
| B 05 | X=800249.3150 | Y=2042487.1547 |
| B 06 | X=800232.4836 | Y=2042422.9491 |
| | | |
| G 01 | X=800233.0949 | Y=2042405.0217 |
| G 02 | X=800282.2635 | Y=2042381.7116 |
| G 03 | X=800330.3176 | Y=2042391.9212 |
| | | |
| G 04 | X=800335.3089 | Y=2042402.1895 |
| G 05 | X=800312.6369 | Y=2042411.2414 |

**Figure 7.1 Extent of benthic survey along with transects/observation points location and coral/seagrass relocation sites**

### 7.1.2. Findings

*Beach area 1*

The foreshore of beach area 1 is comprised primarily of white sand. The floor of the sea at beach area 1 is part of a shallow embayment consisting of a seagrass meadow dominated by *Thalassia testudinum* (no *Syringodium filiforme* or *Halodule wrightii* were seen). The seagrass provides a habitat to a variety of invertebrates (such as urchins) and fish (rays and schools of juvenile stages of finfish).

There are frequent occurrences of small corals (less than 10cm in diameter) that lie loosely attached to sand within the seagrass bed. The benthos is a mix of sand, sand overlaying rock, and rocky seafloor. Where there are rocky areas, these are often colonized by small corals (such as *Siderastrea* species) that are affixed to the rock. Within the seagrass beds, *Lytechenus* species of sea urchins were common and there were species of algae such as *Dictyota*, *Penicillus*, and *Halemeda* present.

The area to be dredged is roughly 2,100m$^2$ and, of this area, approximately 60% is covered by seagrass. The seagrass bed in this area is growing over relatively shallow sandy sediment and is more or less continuous with moderate density and short blade lengths. There is evidence of grazing on the blades of the seagrass.

There were some areas of the floor of the sea close to the foreshore that had loose rolling mats of debris consisting of dead algae, seagrass blades and other detritus from terrestrial vegetation.

This embayment is part of the larger nearshore ecosystem and is comprised of seagrass beds behind a back reef that, with the associated invertebrate and fish community, comprise relatively healthy, well-developed and ecologically significant marine resources. The data collected from the field work is presented in tabular form in Transect 1, Appendix E.

*Beach area 2*

Of the two beach areas to be enhanced this beach area is closest to the molasses pier.

As with beach area 1, the foreshore of beach area 2 is comprised primarily of white sand. The floor of the sea in this shallow embayment consists of a seagrass meadow that is dominated by *Thalassia testudinum* (no *Syringodium filiforme* or *Halodule wrightii* were seen). The sea grasses provide a habitat to a variety of invertebrates (such as *Diadema* species, sea eggs) and fish (rays and schools of juvenile stages of finfish). There are frequent occurrences of small corals (less than 10cm in diameter), which lie loosely attached to sand within the seagrass bed. The benthos is a mix of sand, sand overlaying rock, and pavement. Where there are rocky areas of the seafloor these are often colonized by small corals (such as *Siderastrea* species) which are affixed to the rock. Within the seagrass bed there are algae such as *Dictyota*, *Penicillus* and *Halemeda* species present.

The foreshore of beach area 2 is predominantly sandy, and the floor of the sea from the shore to 20m seaward is sandy, followed by a coral rubble zone (at 21-26m), then a sparse *Thallasia* species seagrass bed (at 27-40m), followed by a dense *Thallasia* species seagrass bed (at 41-55m), and thereafter (from 56m onward) it is a predominantly hard bottom with algal communities.

The footprint of the area to be dredged in beach area 2 is roughly 2,100 m$^2$, and of this area, approximately 40% is covered by seagrass.

There were *Diadema* sp. sea eggs present on these pavement areas and also small coral colonies including *Porities* species and *Siderastrea* species affixed to the sea bottom.

There were a few large monument/massive coral colonies growing in the shallows of the embayment amongst the seagrass.

As with beach area 1, this embayment is part of the larger nearshore ecosystem of seagrass beds and back reef which, along with the associated invertebrate and fish community comprise relatively healthy, well-developed and ecologically significant marine resources. The data collected from the field work is presented in tabular form as Transect 3, Appendix E.

*The northern groyne*

Between the shallow embayment where beach area 1 and beach area 2 are to be developed there is small rocky headland on the coast where a roughly 36 meters long finger groyne is proposed to be deployed.

The foreshore in this area is rocky, and the floor of the sea is initially sandy where after it alternates between bands of *Thallasia* species seagrass bed (at 3-24m, 36-46m, and 61-100m from the shore) with rocky or pavement bottom between those seagrass areas. There were *Diadema* species, and sea eggs present on these pavement areas and also small coral colonies including *Porities* species and *Siderastrea* species affixed to the sea bottom.

The average cover, density, and blade length of the seagrass in the first band of sea grasses (at 3-24m) was less than in beach area 1. While the seagrass in the second (at 36-46m) and third (at 61-100m) bands was similar to that found in beach area 1. Even where the sea grasses are growing in sandy areas there is a large amount of coral rubble and other rubble on the floor of the sea. The data collected from the field work is presented in Appendix E.

The footprint of the groyne is approximately 400m$^2$ and of this approximately 25% is covered by seagrass.

*The southern groyne*

A 31.7m long finger groyne is proposed to be constructed at the western end of beach area 2. The foreshore at the area where the proposed groyne is to be constructed is rocky and the floor of the sea is a hard bottom or pavement. There is very little seagrass in the footprint of the proposed finger groyne and where there is seagrass it is sparse, with very short blade length, and it is growing in very shallow sediment or among rubble.

Growing on the hard bottom are various types of algae, such as *Enteromorpha*, *Dictyota*, *Halimenda*, and *Penicillus* species. There are also small colonies of corals such as *Siderastrea* species affixed to the hard bottom and *Diadema* species sea eggs grazing on the hard bottom.

The footprint of the groyne is approximately 316m$^2$ and, of this area, approximately 10% is covered by seagrass.

*Northern breakwater*

One 87m long shore-parallel breakwater is proposed to be constructed (via a construction pad from the groyne) in a water depth of less than 1m. The footprint of the breakwater is approximately 691m$^2$ and, of this area, approximately 100% is covered by seagrass.

*Southern breakwater*

A 77m long shore-parallel breakwater is proposed to be constructed (via a construction pad through beach area 2) in a water depth of less than 2m.

Most of the floor of the sea in this area is the framework of a dead coral reef that has very little live coral cover and is severely degraded although the three dimensional structure is visible. There are, however, several dozen large “massive” growth form coral colonies within the footprint of the breakwater and just outside of the footprint of the north-eastern end of the breakwater.

## 7.2 Potential Impacts to Benthic Resources

The potential negative impacts to benthic resources were examined in relation to the construction phase and the operational phase of the development and are described in the following sections.

### 7.2.1. Construction

*Smothering:* The area of sea floor to be dredged and then nourished in beach area 1 and 2, the groynes, and the breakwaters are all to be constructed using land-based heavy machinery. There will be the need to deploy construction pads on the sea floor to facilitate heavy machinery accessing the construction area for each breakwater.

All the benthic resources in the footprint of the coastal structures (groynes, breakwaters), beach areas 1 and 2, and the construction pads will be impacted negatively by the physical disturbance resulting from the dredging and from the deployment of boulders that make up the breakwaters and the groynes. It is estimated that approximately 5,000$m^2$ of seagrass bed and all invertebrates will be lost from these physical disturbances. The numerous small (less than 10cm diameter) coral colonies and the few large massive growth form coral colonies within the seagrass bed along with the large massive growth form coral colonies at the southern breakwater will be impacted negatively.

*Turbidity:* The dominant component of the sediment in the project area is sand, however there is also some amount of fines present in the sediment. The deployment of boulders for the breakwaters and the groynes, the dredging of each beach area, the deployment and removal of construction pads, and the nourishment of the beach 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 these embayments makes it unlikely that the turbidity generated will lead to the formation of plumes affecting resources further alongshore.

### 7.2.2. Post-Construction

*Debris:* Any debris left on the seabed from the construction activity can become projectiles during severe wave activity, and this may cause damage to sensitive benthic resources.

## 7.3 Mitigation and Environmental Management Plan

An impact is defined as any change to the existing condition of the environment arising from project implementation. Impacts may arise during two phases of project implementation: (1) construction and (2) post-construction (operation). Understanding the nature of the impact can be assisted by categorizing the effect of the potential impact as being either:

- Positive or negative,
- Reversible or irreversible,
- Of short or long duration,
- Of small or large magnitude, and

- Being local or wide in extent.

Where the effect of an impact is negative, consideration should be given to implementing mitigation measures. It is important to design mitigation measures carefully so that potential negative impacts are minimized as much as possible, so that any damage to the environment is reduced. Mitigation measures are especially important when the nature of the impact has been identified as being irreversible, or being of long duration, or being of large magnitude, or where the expression is likely to be wide in extent.

A summary of the potential negative impacts and the proposed mitigation measures is presented in

Table 7-1 following. All of the impacts identified are of small magnitude and are likely to be expressed in the vicinity of the proposed coastal works, however, some of the impacts identified were found to be irreversible and of long-lasting duration.

For all of the impacts identified, regardless of their nature, appropriate mitigation measures have been proposed. These mitigation measures involve known techniques related to relocating resources, the use of silt screens, and visual inspections. These mitigation measures are outlined below.

### 7.3.1. Relocation of Ecosystem Resources

The area of benthic resources that will be impacted during construction and operation are easily identified. Based on the existing environmental conditions it would be appropriate to relocate these resources (comprised mainly of *Thallasia* species seagrass).

Where the sediment type allows, harvesting of seagrass as mats/planting units can be done for the material to be relocated and used in re-turfing. Additionally and where the sediment characteristics are such that harvesting seagrass as mats/planting units is not practical (due to depth of sediment, presence of rubble, etc.), the apical meristems may be harvested allowing for the restoration of the seagrass bed in other areas. The combination of relocation and restoration will minimize the impact of this development proposal on the seagrass bed.

The sea grasses harvested from within the footprint of the proposed coastal works can be relocated to nearby areas of the seagrass bed that have experienced 'blow-outs', bed erosion, or which have been otherwise damaged from human activity. These areas should be appropriately prepared by filling the depressions with sand and replanting donor mats or meristems.

As these areas are within the broader seagrass meadow, the replanting will be in conditions (e.g. substrate, light penetration, water quality, flushing, depth) that have been established to be similar to the harvest site.[5]

It is proposed that an area to be replanted and/or restored should be approximately 4,200m$^2$, which is an area equivalent to 120% of the area that will likely be disturbed by the coastal works.

[5]Proposed Seagrass Relocation and Replanting and Coral Relocation Methodology (2006) by CL Environmental and CEAC p36.

Where the seagrass beds have been relocated, both the donor sites and the recipient sites may need to have appropriate stabilization treatments to the edge of the beds to prevent any erosion of the bed edges. This stabilization may be carried out using mesh and pins.

All invertebrates (such as sea urchins and sea eggs) and all small corals in the seagrass bed can be collected by hand and transported underwater where they will be relocated to adjacent seagrass beds.

Where there are large "massive" growth form coral colonies present in the footprint or the vicinity of the proposed works these should be removed and relocated to adjacent areas of the reef and properly anchored to the substrate. Suitable relocation areas are identified in the plan layout of the recommended options (Figure 6.1).

In order to monitor the success of the relocation exercise there should be appropriate long-term monitoring of the relocated resources and the ecosystem generally.

#### 7.3.2. Turbidity Screens

Areas of coastal construction should be surrounded by silt curtains where the depth of water is sufficient to allow deployment. Properly deployed and maintained turbidity screens can significantly reduce the transportation of sediment-loaded waters along the coast and offshore.

#### 7.3.3. Debris Surveys

During the construction phase and immediately after construction is completed, the seabed around the proposed coastal works should be examined for any debris, which could have the potential to become a projectile in severe weather. This debris should be removed and appropriately disposed of.

Table 7-1 Summary of Potential Impacts and Proposed Mitigation

| Phase | Potential Impact | Impact reversible? | Duration | | Magnitude | | Extent | | Proposed Mitigation |
|---|---|---|---|---|---|---|---|---|---|
| | | | Long | Short | Large | Small | Wide | Local | |
| **Construction** | Smothering of benthic resources in footprint of the groynes, breakwaters, and beach areas | No | X | | | X | | X | Relocation of resources within footprint of structures and dredging to adjacent sand patches or coral reef, and restoration of disturbed areas of adjacent seagrass bed. |
| | Turbidity of water column | Yes | | X | | X | | X | Deployment and maintenance of silt screens, carrying out of work only when sea conditions are suitable. |
| **Post-construction** | Damage to benthic resources by debris on the seabed after storm damage | Yes | X | | | X | | X | Post-storm survey of seabed and removal of debris |

## 7.4 Conclusion

The site proposed for the construction of the coastal works is located within a seagrass meadow and a shallow back reef. The coral reef is extremely degraded, however there are dozens of large coral colonies within the footprint of the proposed works, and the seagrass beds and the associated invertebrate and fish community comprise relatively healthy and ecologically significant marine resources.

The potential negative impacts identified from the construction of the breakwaters and groynes and the dredging and nourishment of the beach areas will have a significant negative impact on the seagrass habitat of the area. The associated impacts were identified to be of long lasting duration.

However, for all of the impacts there are appropriate mitigation measures available to reduce the damage to the environment. If the proposed mitigation measures are carried out in a sensitive manner, the benthic resources in the vicinity of the construction and those resources of the wider seagrass meadow can be minimised.

# 8. Construction Methodology and Recommendations

The method for building the breakwaters, groynes and beach coves involves relatively standard practices of construction, excavation and dredging. Given the findings of the field visits, site historical characteristics and beach response modeling, the following points should be observed during the construction phase of the project:

- In an effort to minimize possible major impacts to the marine environment, it is recommended to transplant the corals, sponges and sea grasses that will be directly impacted by the structures and their construction.
- Turbidity barriers must be installed and maintained to prevent/control silt entering the water column.
- The groyne and breakwaters can be constructed onsite using mainly conventional land-based equipment such as loaders, excavators and cranes. The armour stones for the breakwaters and groynes will be stockpiled at locations on land within the project site. The stones will be carried out to the breakwater/groyne locations and placed either with a crane or excavator.
- The packing should be done such that each boulder is in contact with two or three other boulders. The voids between the boulders should be left as voids and not filled with smaller rocks. It is critical to the structural stability of the marine structures that the specified range of boulders be used and that they are packed and shaped to the specified slopes. The design is based on the use of rocks with a minimum density of 2400kg/m$^3$.
- Access to the nearshore structures will be via temporary construction pads that will be removed once each structure is built.
- Boulders can be used in the works without further scale model testing or field investigations, although specific physical, chemical and structural laboratory tests will be required for the stone material. Once an appropriate quarry is located, the required numbers and sizes of stones can be sourced and stockpiled.
- A reinforced concrete wall is to be placed in the core of the proposed emergent breakwaters to reduce the permeability of the breakwaters to wave transmission, which is important in order that sand is not washed out from behind them.
- The deepening of the seabed for swimming will be accomplished by use of an excavator. Material taken up will be placed in a truck and taken offsite for disposal.
- Approximately 9,600m$^3$ of sand will be required for this project. Sand will be dredged from either selected offshore or land sources, to be placed in the lee of the proposed breakwaters and along the beach in between the groynes.
- The sand sourcing for the beach enhancement will have to be further investigated in the final engineering design stage of this study.
- It is recommended that sand for beach nourishment have a mean grain size larger than the existing beach sand. If this is not the case, an overfill ratio must be computed to account for sand that will be quickly carried away by waves. The mean grain size varies between a maximum of 0.65mm along the north beach cove and a minimum of 0.18mm along the south beach cove; consequently it is recommended that the sand used to enhance the beach at the Grand Palladium have a mean grain size ranging from 0.3mm to 0.5mm.

- Test digs and detailed topographic surveys should be conducted prior to the final engineering design to get a better understanding of soil characteristics for land excavation techniques and to compute correct quantities for the excavation volumes. Existing land elevations were estimated from an assumed datum and therefore uncertainties regarding land elevations remain and quantities may be subject to revision. This will affect the total cost of the project.
- It is recommended that an investigation for the best location of the southern breakwater be done during the final engineering stage of this project to minimize the impact on coral disturbance and relocation while enhancing the beach accretion.

# 9. Summary

This report describes the work carried out for the Preliminary Engineering designs of the beach enhancement concept at the Grand Palladium, Lucea, Jamaica. The design objectives for the beach enhancement of the proposed Royal Suites included the following:

- Identify the optimum approach to create two additional beach coves;
- Produce an engineering solution to enhance and stabilize the newly created beach coves;
- Support the preliminary designs of the recommended solution by demonstrating its capacity to promote accretion of sand at the project shoreline as well as to provide protection from daily and seasonal wave conditions while being environmentally friendly;
- Identify the potential impacts on the marine environment and surrounding coastline;
- Set out a mitigation strategy and environmental management plan dealing with the relocation of any sensitive benthic organisms such as seagrass and/or corals;
- Present the developed concepts within an Engineering Report; and
- Provide preliminary volumes and cost estimates.

Field investigations including nearshore bathymetric surveys, beach profiles and sand sampling were conducted and the results were presented in Section 2.

Operational and extreme wave climates, presented in Section 3, were established using the best available information. The operational wave climate was based on seven years of 3-hourly global wave model results, and the extreme waves based on over 100 years of hurricane tracks and records. These climates were transferred from deep water to the project site using MIKE21. Storm surge and nearshore wave heights were defined for use in structure design.

The operational wave climate was defined to determine sediment transport characteristics, detailed in Section4 of this report. Alongshore sediment transport was analysed and found to be very small (less than 8,000m$^3$ per year) to the south, demonstrative of a fairly low annual sand production rate. A more detailed morphological analysis was conducted, which identified the sediment transport pathway in and out of the beach coves. The analysis revealed that the central portion of the beach coves is subject to cross-shore transport while alongshore transport to the southwest occurs over the existing reef. The increase in sediment transport between both beach coves suggested that the sand is produced locally by the existing reef system while the rocky headlands help contain the sand within each bay. Results suggested that the beach will remain stable and protected by the existing Grand Palladium headland situated just north of the project site while sediment transport tends to be more predominant with longer period waves coming from a clustered northwest angle.

Using this information, various solutions were proposed, leading to a final recommended solution presented in Section 6. The recommended solution consists of two partially emergent breakwaters, and northern and southern emergent groynes. These groynes are designed to delineate both coves and used as headland reinforcement to promote the sand accretion along the beach coves and avoid loss of sediment to the south. The proposed structures are to be implemented along with foreshore excavation and beach nourishment to create two wider and stable beach coves and enhance swimming conditions.

Computer simulated beach response modeling was carried out (and described in Section 6)to test the various concepts and evaluate the effectiveness of the proposed solution. Using two different swell conditions as well as Hurricane Dean statistics, hydrodynamics, waves and sediment transport were simulated using MIKE21. The model was set with existing conditions and then configured with the proposed optimum solution to predict what would happen during the same swell/storm conditions if the proposed solution had been built.

The comparison of the results between the existing and proposed beach enhancement solution revealed that the breakwaters reduced wave heights in their lee by up to 70% while current speeds were also reduced by up to 40%. The breakwaters were found to reduce erosion of the beach by up to 90% with a structure crest elevation set at 0.3m above MSL.

The intensity of the potential sediment movement was also decreased in the lee of the proposed structures. While the pathways and trend of sand movement remained the same, but with a lower intensity, it was found that the proposed breakwater will contribute to diminishing the sediment drift to the south while still allowing for natural current flow to the south. This will promote natural sand accretion along both of the beach coves. Results also suggested that the downdrift impact caused by the implementation of the protective structures would be very small.

The model revealed significant decreases in the alongshore sediment transport rates at the existing shoreline in the lee of the proposed breakwaters due to the wave and current sheltering effects of these structures. In addition, the proposed groyne structure was efficient in reducing currents up to 20% and enhancing accretion along the proposed beach. It was concluded that the proposed structures would be beneficial to the shoreline.

Preliminary designs of the structures were prepared using established methods to determine stable armour stone sizes. The transmission coefficient of the emergent breakwaters was investigated using available methods so that the crest width and elevation could be established. Given the site characteristics and environmental constraints, the solution favoured an emergent structure (with its visual impacts) rather than a submerged structure with a wider footprint.

Characteristics for sand recommended to increase the beach width and enhance the cove have been evaluated and a median grain size ranging from 0.3-0.5mm was recommended for the project site. Volumes of different materials were estimated based on the plans and cross-sections. The different materials include armour stone, filter or core material, geotextile filter fabric and beach sand. Using established unit rates for similar construction projects in Jamaica and other Caribbean islands, cost estimates were prepared for the proposed works.

Finally the proposed construction methodology was presented along with the environmental impacts and appropriate mitigation methods. It was found that the construction of the breakwaters, groynes and the dredging and nourishment of the beach areas will have a significant negative impact of long lasting duration on the seagrass habitat of the area. However, if the proposed mitigation measures, defined for all of the negatives impacts and identified to reduce the damage to the environment, are carried out in a sensitive manner, the benthic resources in the vicinity of the construction and those resources of the wider seagrass meadow can be minimised. The total costs for the beach enhancement works were estimated to vary between US$2,510,000 andUS$3,200,000.

# 10.References

Danish Hydraulic Institute, 2009a.*MIKE 21 Flow Model FM – Hydrodynamic Module* User Guide. Hørsholm,Denmark

Danish Hydraulic Institute, 2009b. MIKE 21 Flow Model FM – Sediment Transport *Module User Guide*. Hørsholm,Denmark

Danish Hydraulic Institute, 2009c.*MIKE 21 SW Spectral Waves FM Module User* Guide.Hørsholm,Denmark

Danish Hydraulic Institute, 2009d. MIKE 21 Tide Analysis and Prediction Module *Scientific Documentation*. Hørsholm,Denmark

Kamphuis, J.W., 1991. *Incipient Wave Breaking*,Coastal Eng., 15:185-203

Payo, A., Kobayashi, N. and Kim, K.H. (2006).*Beach Nourishment Strategies*.Proceedings of 30th Coastal Engineering Conference, World Scientific, Singapore, 4129-4140.

Smith Warner International (2007).*NegrilBeach Restoration Works*

Van Der Meer et al,(1996). *Wave Transmission at Low-Crested Structures*.Coastal Engineering Manual

Van Der Meer and Pilarczyk, 1990.*Stability of Low Crested and Reef Breakwaters*. Proceed 22nd Coastal Conference, Vol.2, pp.1375-1388.

**Data report: analysis of grain size distribution and composition for sediments collected from north coast of Jamaica**

Prepared by Shakira Khan

Sediment characteristics for were undertaken at the request of Smith Warner International Ltd. for a north coast site. Samples were collected at the High water mark from two sites on the eastern side of Lucea Harbour (Figure 1). The sediments from two samples were examined to determine grain size variations and composition. The analyzed samples were supplied by Smith Warner International Ltd. Statistical parameters such as mean, sorting and skewness, were determined and sediment textural classification given for each sample is based on Folk 1954. Biogenic constituents were identified using a qualitative approach and proportions of the constituents present determined. Detailed description of approach and findings are outlined below.


**Figure 1** Location of samples analyzed in this report. Collected from the High water mark (HWM) of two beaches along the north eastern coast of Lucea harbour

Methods:

Both samples were dried before analysis. Each sample was emptied into a tray, labelled and placed in an oven for 24 hours to dry. The sample was subsequently poured through a sample splitter to ensure random selection of the portion of the sample used for grain size and compositional analyses.

*Grain size analysis*

Sample splits of 100.08g (S3H) and 100.09g (S6H) were sieved through a nest of seventeen (17) sieves, decreasing in half phi (½ Φ) intervals from 16 mm to 62 μm. The nest was placed on a mechanical shaker for 10 minutes [2]. The initial weight of each sample was recorded prior to sieving. The sediment retained in each sieve was weighed, using a digital balance with precision of 0.01 g, and the data recorded.

Table I. Size scale adopted in the GRADISTAT program, compared with those previously used by Udden (1914), Wentworth (1922) and Friedman and Sanders (1978)

| Grain size | | Descriptive terminology | | | |
|---|---|---|---|---|---|
| phi | mm/μm | Udden (1914) and Wentworth (1922) | Friedman and Sanders (1978) | GRADISTAT program | |
| | | | Very large boulders | | |
| −11 | 2048 mm | | | | |
| | | | Large boulders | Very large | |
| −10 | 1024 | | | | |
| | | | Medium boulders | Large | |
| −9 | 512 | Cobbles | | | |
| | | | Small boulders | Medium | Boulders |
| −8 | 256 | | | | |
| | | | Large cobbles | Small | |
| −7 | 128 | | | | |
| | | | Small cobbles | Very small | |
| −6 | 64 | | | | |
| | | | Very coarse pebbles | Very coarse | |
| −5 | 32 | | | | |
| | | | Coarse pebbles | Coarse | |
| −4 | 16 | Pebbles | | | |
| | | | Medium pebbles | Medium | Gravel |
| −3 | 8 | | | | |
| | | | Fine pebbles | Fine | |
| −2 | 4 | | | | |
| | | Granules | Very fine pebbles | Very fine | |
| −1 | 2 | | | | |
| | | Very coarse sand | Very coarse sand | Very coarse | |
| 0 | 1 | | | | |
| | | Coarse sand | Coarse sand | Coarse | |
| 1 | 500 μm | | | | |
| | | Medium sand | Medium sand | Medium | Sand |
| 2 | 250 | | | | |
| | | Fine sand | Fine sand | Fine | |
| 3 | 125 | | | | |
| | | Very fine sand | Very fine sand | Very fine | |
| 4 | 63 | | | | |
| | | | Very coarse silt | Very coarse | |
| 5 | 31 | | | | |
| | | | Coarse silt | Coarse | |
| 6 | 16 | Silt | | | |
| | | | Medium silt | Medium | Silt |
| 7 | 8 | | | | |
| | | | Fine silt | Fine | |
| 8 | 4 | | | | |
| | | | Very fine silt | Very fine | |
| 9 | 2 | Clay | | | |
| | | | Clay | Clay | |

**Figure 2** Comparison of grain size scale used by GRADISTAT (Folk and Ward, 1957)[3] with those defined by Udden (1914)[4], Wentworth (1922)[5] and Friedman and Sanders (1978)[6]

# Results

## Grain size

Sedimentary parameters described are based on cumulative weight % retained in each sieve (Appendix 1) and overall sediment classification is based on the Folk and Ward scheme.

## Sample: S3H

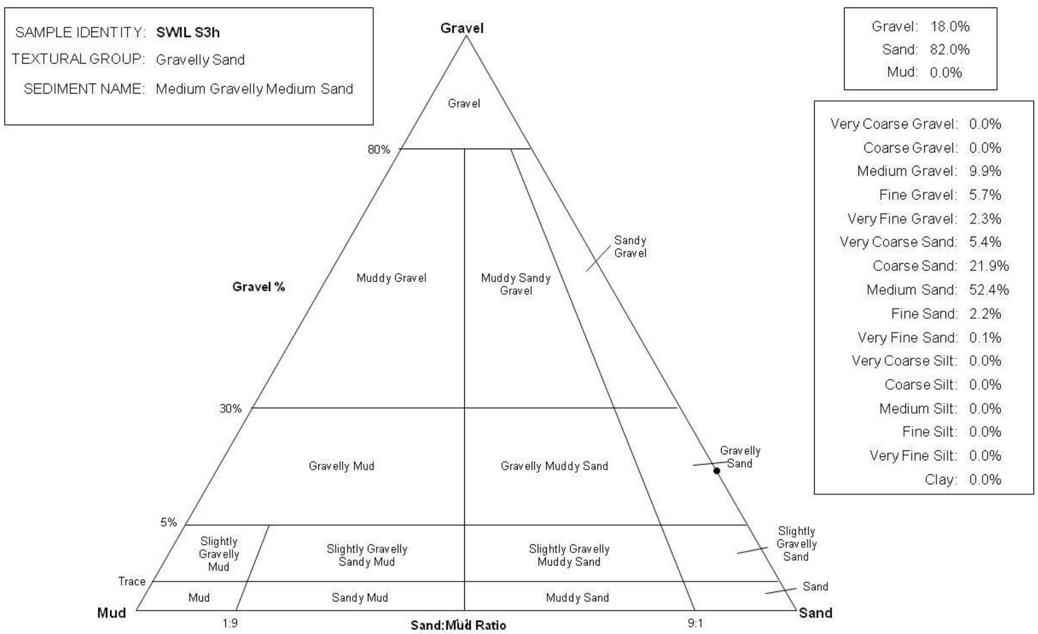



**Figure 3** Ternary diagram illustrating proportions of Gravel-Mud -Sand present in sample S3H and textural classification

This is a bimodal (Figure 4), very coarsely skewed (0.657 μm /-0.657 Φ) poorly sorted (3.2 μm /1.7 Φ) sediment; with a mean grain size of 789.3 μm (0.341 Φ) (Coarse sand). It is classified as a gravelly sand which consists of 18% gravel; 22% sand and no mud (Figure 3). A detailed break down of the sand and gravel categories shows that the sample consists primarily of medium (52.4%) and coarse (21.9%) sand. From Analysis the following descriptive parameters have been calculated:

D50=0.47mm
D16= 0.32 mm
D 90= 7.95 mm
D86 = 5 mm
D84= 4.5 mm

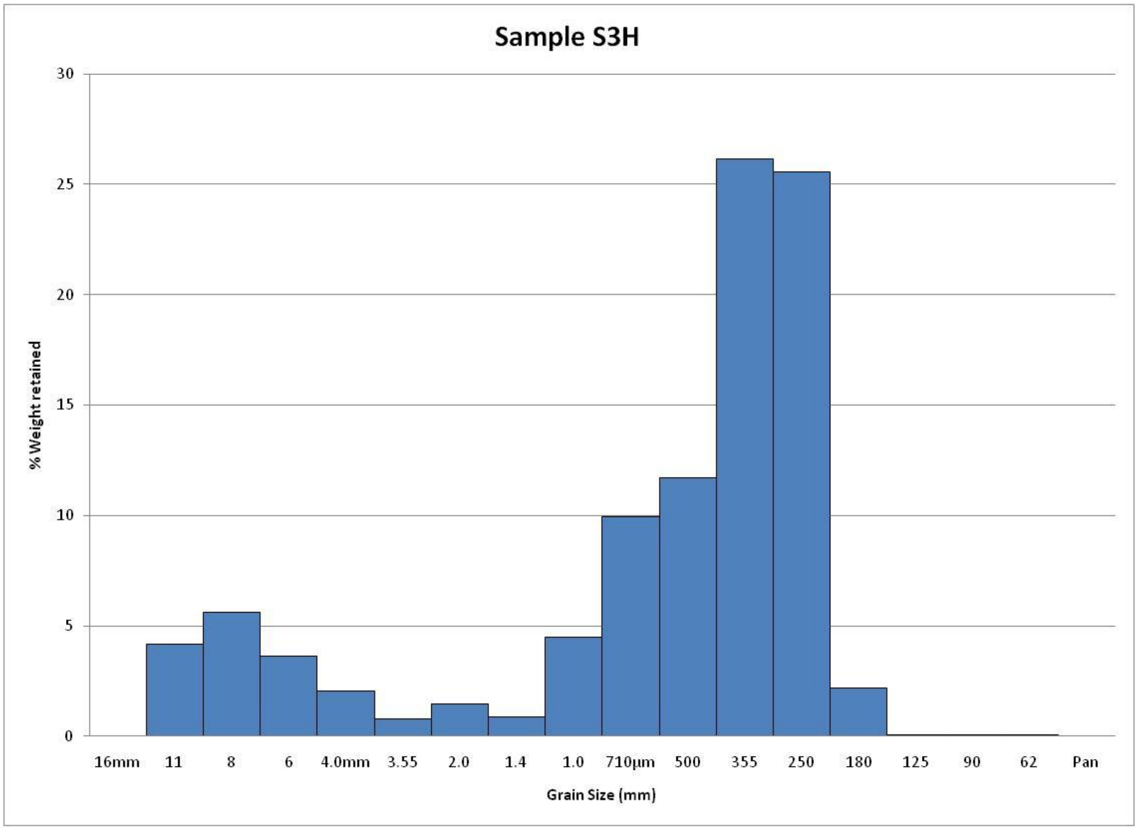



**Figure 4** Histogram illustrating bimodal property of Sample S3H. Coarse fraction is dominated by corals and lithics

Sample: S6H

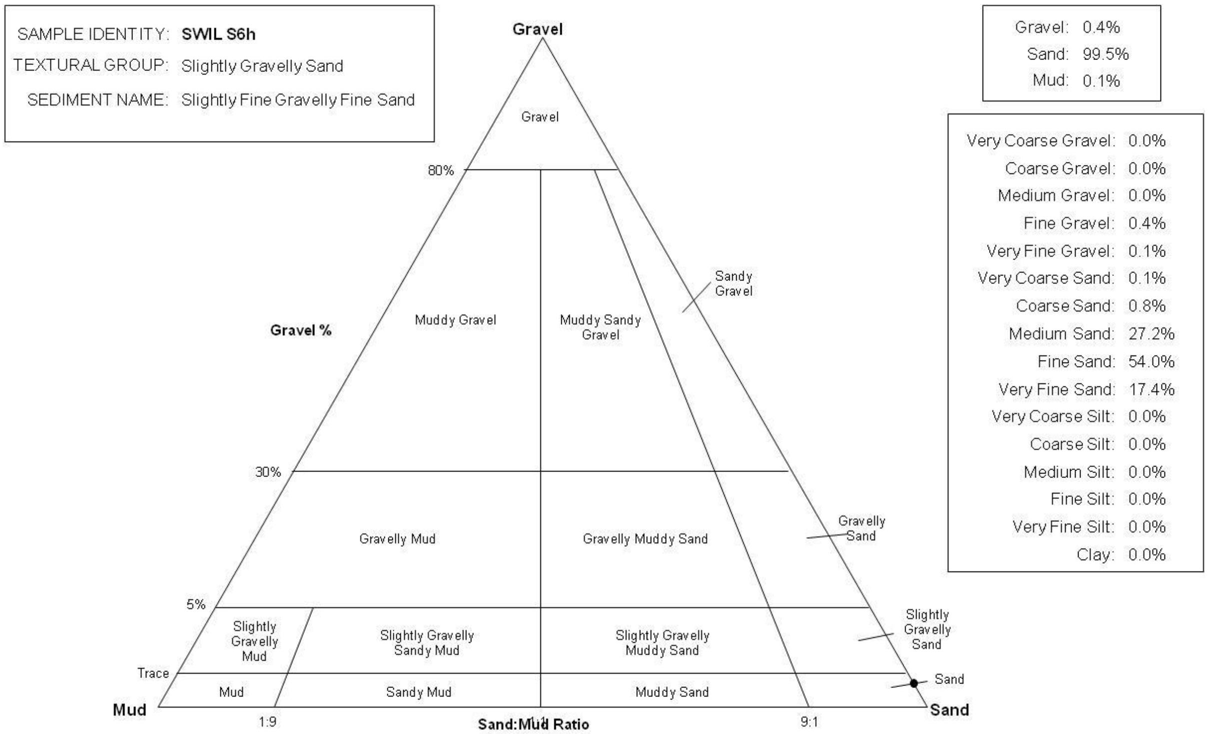



Figure 5 Ternary diagram illustrating proportions of Gravel-Mud -Sand present in sample S6H and textural classification

S6H is a symmetrically distributed sample (figure 6) which has been classified as a slightly gravelly sand (Figure 5). It is moderately well (0.607 Φ) sorted with mean grain size of 187.5 μm (fine sand) and consists of 99.5% sand; 0.4% Gravel and 0.1% mud. The bimodal property of this sample is a result of shell fragments occurring in the sample. A detail breakdown of the sand and gravel categories shows that the sample consists primarily of fine (54%), medium (27.2%) and very fine (17.4%) sands. The remaining 10% of the sample is distributed amongst fine, very fine gravels, very coarse and coarse sands. From Analysis the following descriptive parameters have been calculated:

D90=0.23 mm
D84=0.29mm
D50= 0.18mm
D16=0.12mm
D86= 0.3 mm

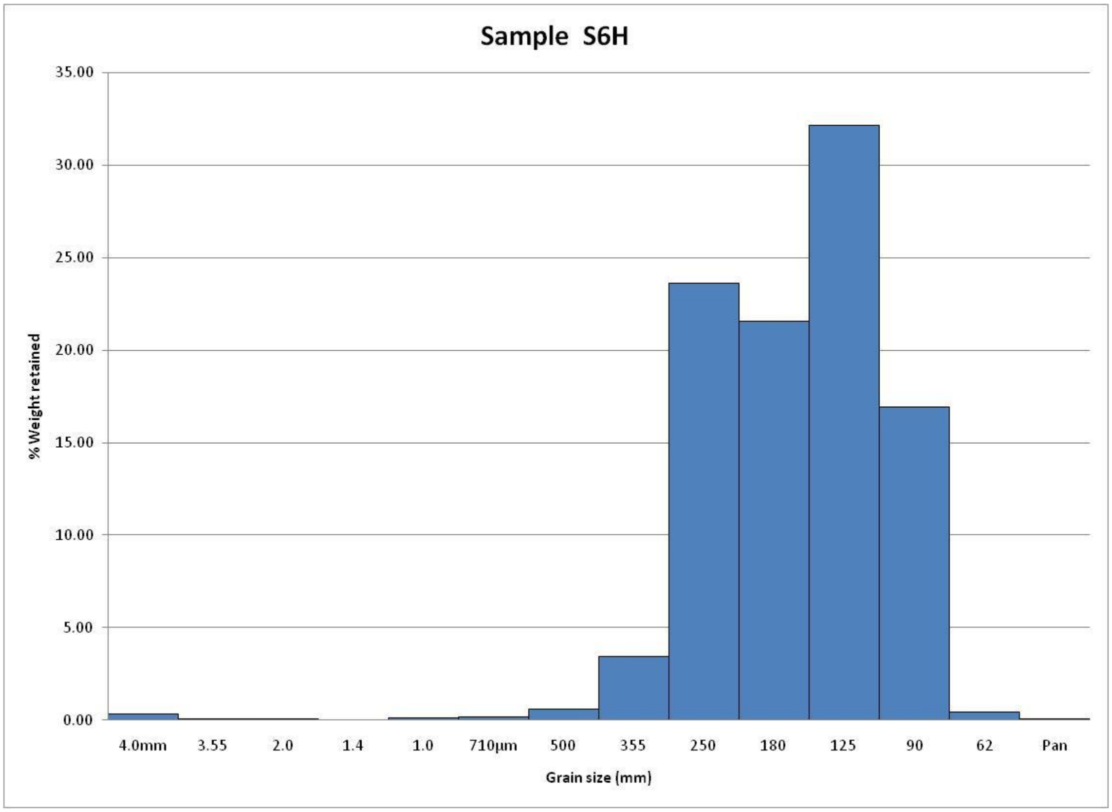



**Figure 6** Histogram illustrating symmetrical distribution of sample S6H. The low percentage of clasts greater than 4mm makes the sample bimodal but does not affect the symmetry of the distribution.

## Qualitative Compositional analysis

From compositional analysis of sediments, six categories of constituents were identified: mollusc (Bivalve & gastropods); algae (red & green); foraminifera; coral; crystalline grains and echinoid fragments. Proportions of each constituent present in the two sediment samples analyzed were determined by a qualitative approach utilizing incident light and a binocular microscope. These are shown in Table 1.

| Constituent | Proportions of constituents in Sample (%) | |
|---|---|---|
| | S3H | S6H |
| Algae ( Red & Green) | 5% | 10% |
| Crystalline Grains | 60% | 60% |
| Molluscs (Bivalve & Gastropod) | 15% | 15% |
| Foraminifera | 5% | 5% |
| Coral fragments | 10% | 5% |
| Echinoid fragments | 5% | 5% |

**Table 1** Proportions of the constituent contribution to the carbonate fraction of the sediments identified using a qualitative approach.

*Carbonate Contributors*

The fine grained nature of the sediment made distinction of gastropod and bivalve contributions difficult; they have been grouped in the mollusc category in most cases. Some large fragments were identified, and this shows that both marine gastropods and bivalves contribute to the sediment. Bivalves were distinguished by their distinctive shape and ornamentation on valves and presence of an articulated hinge line which exhibits teeth and sockets. Gastropods are uni-valved, and have an unchambered shell that is usually coiled resulting in a conical shaped tube, closed at the pointed end and open at the wide end (aperture). The exterior of the shells may be smooth or ornamented with ribs, nodes and spines. Shells from this category are composed of aragonite or of a mixture of aragonite and calcite in alternating layers. Details of their occurrence, common habitat and range[1] are detailed below. For the purposes of this report range is as follows: Shallow water- tidal area to a depth of about 30 feet; Moderately shallow water- 30 to 80 feet; moderately deep water- 80- 200 feet and deep water- greater than 200 feet.

The presence of green algae was determined based on identification of characteristic pits on the outer surface of grains and/or the presence of randomly oriented utricles. The distinctive shape and composition of red algae allowed for easy identification in sediment.

Foraminifera are small single celled protozoan organisms which occur predominantly in marine environments, although some species may occur in brackish settings. The benthic species identified produce porcelaneous calcareous tests with distinctive shapes and chambered structures used to determine species present.

Crystalline grains exhibit no discernable characteristics, such as diagnostic external shape or internal features that could be identified using a binocular microscope. These exhibit a dense microcrystalline internal structure and are most likely eroded fragments of the coral reef.

## Sediment composition by sample

### Sample S3H

This sample consists of both lithic clasts and carbonate biogenic clasts (Figure 7. However the lithics are a minor contributor accounting for approximately 10% of the sample.

The carbonate fraction of this sample consists of crystalline grains; echinoid fragments; foraminifera; algal fragments (red and green), coral and molluscs. Approximately 20% of this sample consists of clasts larger than 4mm. This coarse (gravel) fraction is dominated by coral and molluscs fragments (Figure 7). Coral fragments were rounded compared to the mollusc fragments which imply that the molluscs may have a nearer shore source relative to the corals.


**Figure 7.** Sample S3H showing both gravel and sand fractions with lithic clasts and coral and mollusc fragments

The finer (sand) portion of the sediment (Figure 8) is comprised of crystalline grains 60%; algas (Red & Green) 5%; echinoid 5%; coral 10%; foraminifera 5% and molluscs 15% (Table 1).

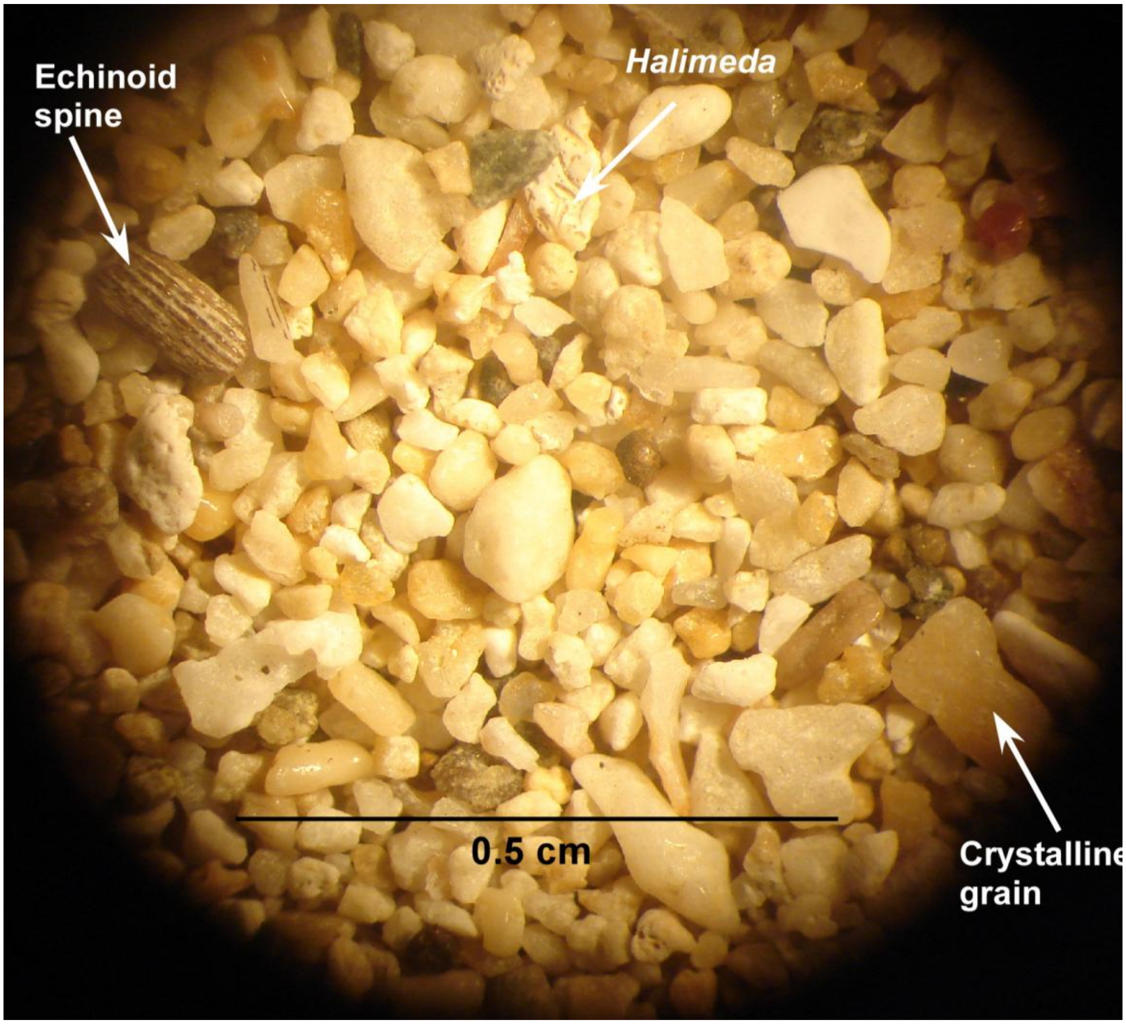



**Figure 8** Microscopic view of sample S3H showing carbonate constituents (Scale 0.5cm)

Sample S6H

This sample is comprised of carbonate biogenic clasts (Figure 9); primarily crystalline grains (60%); echinoid fragments (5%); foraminifera (5%); algal fragments (red and green) (10%), coral (5%) and molluscs (15%).

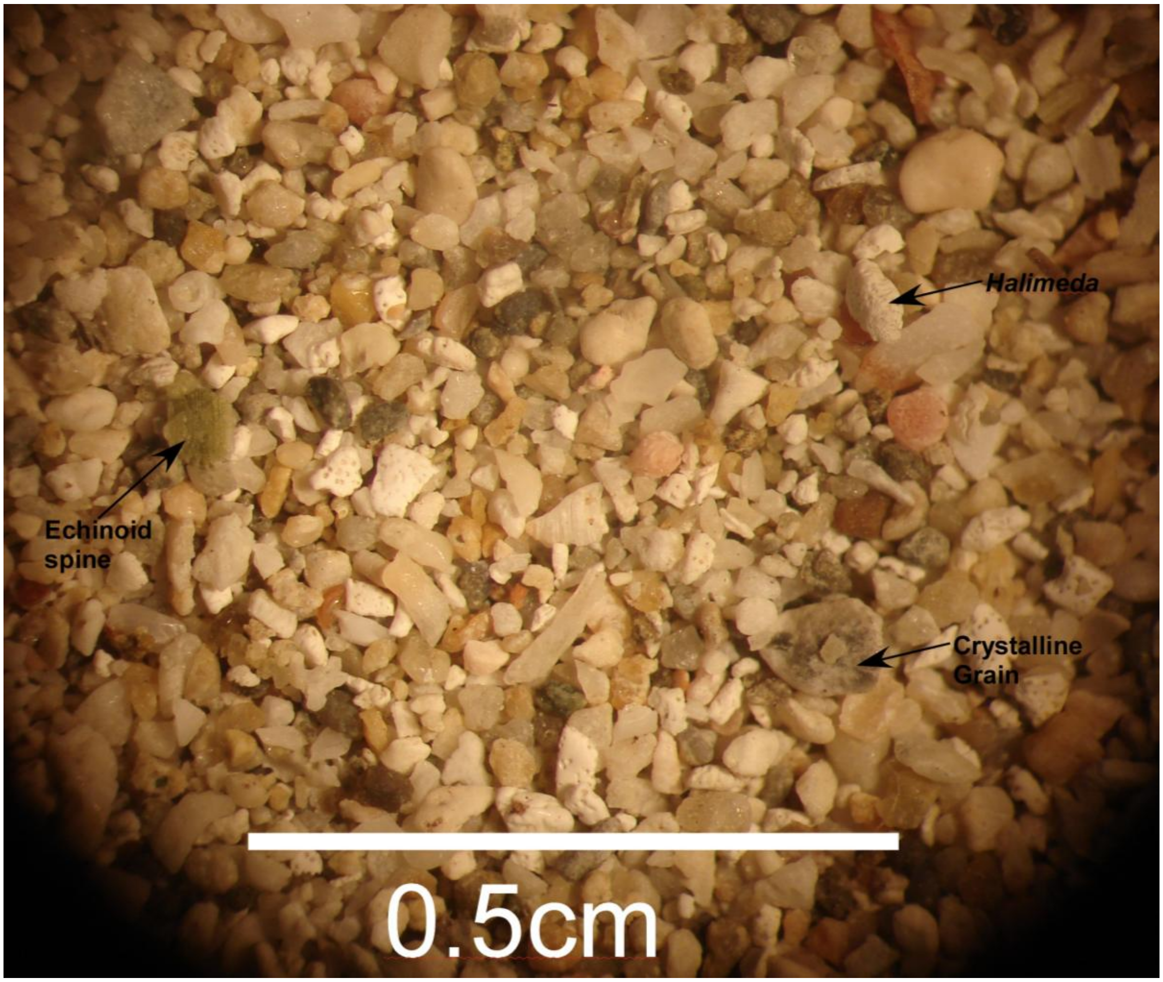



**Figure 9** Microscopic view of sample S6H showing carbonate constituents (Scale 0.5cm)

## Constituents contributing to the sediment

### *Crystalline Grains*

Crystalline grains appear to be the most abundant grain identified in the sediments, accounting for 60% of sample S3H and sample S6H. These grains are of carbonate origin which are highly crystalline and have no distinctive or characteristic shape. They are often rounded in shape with a smooth exterior. Fresh broken surfaces show no distinguishing internal structure and so these grains cannot be placed in any of the skeletal categories discussed. Possible sources of these grains are reworked coral fragments.

*Halimeda*

The green algae *Halimeda* was found to be the most abundant sediment contributor with proportions ranging between 30-45% of the total sample. 3 species of *Halimeda* were identified from the samples: *Halimeda opuntia, Halimeda incrassata* and *Halimeda monile.* All grow on sand and gravel flats and are typically associated with sea grass beds, these species are commonly found at depths of up to 12 m (*Halimeda incrassata* and *Halimeda monile*) and 25 m (*Halimeda opuntia*).

*Molluscs*

Mollucs are also important contributors to the sediment accounting for 10- 30% of the grains identified (Table 1). Seven species of gastropods were identified (Table 2) during compositional analysis. These species are common in sandy areas most are common in waters up to 80ft; however some can be found in waters up to 600 ft deep[1].

Gastropods:- Two species of gastropods (Common sundial and Orange-banded Marginella) were identified, they are typically shallow water sessile species found in water depths of up to 30ft.

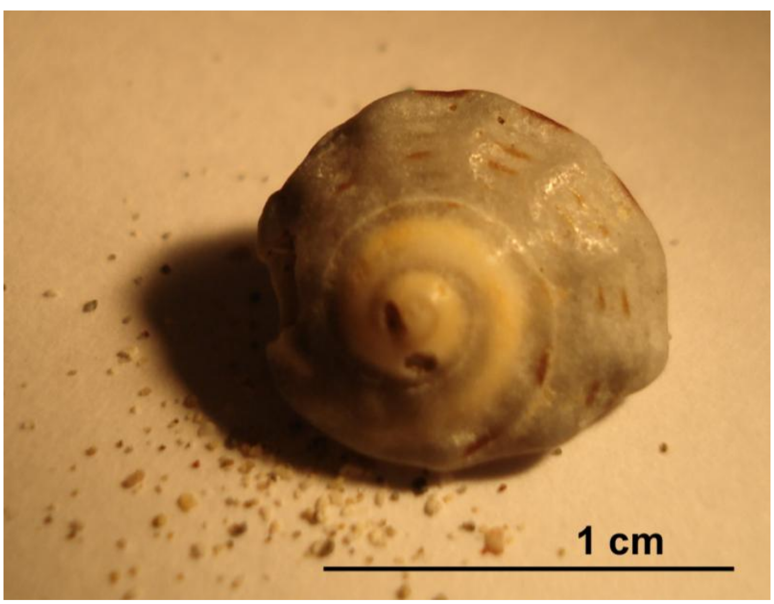



**Figure 10.** This common sundial is one species of marine gastropod identified from sediments analyzed. These are typically found in shallow water and the mostly intact condition of shell indicates short transport distance to the shore.

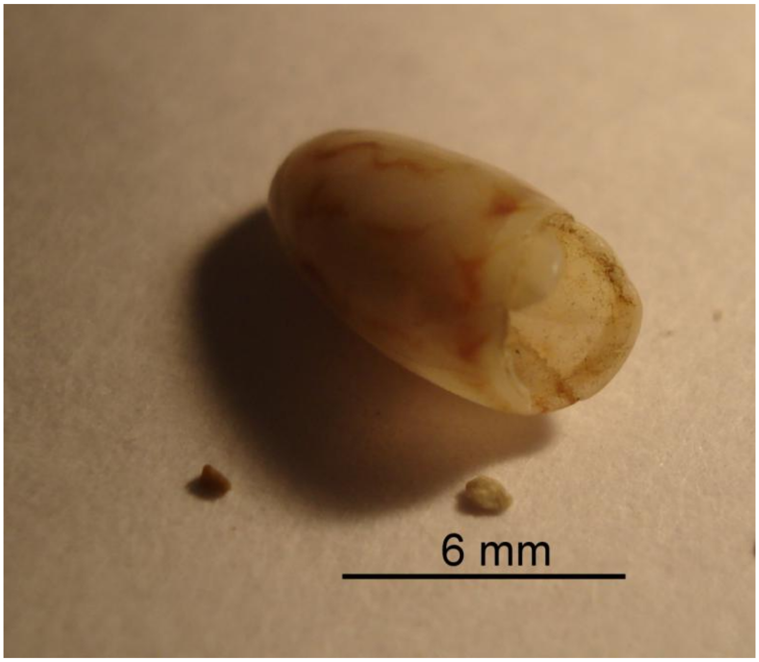



**Figure 11** The orange-banded marginella is another marine gastropod identified from sediments analyzed. These are typically a shallow water fauna and the mostly intact condition of shell indicates short transport distance to the shore.

| Species | Range | Common habitat |
|---|---|---|
| Common sundial<br>*Architectonica nobilis* | Shallow water<br>Tidal area – 30ft | In sand and on rocks |
| Orange Banded Marginella<br>*Hyalina avena* | Shallow water<br>Tidal area – 30ft | Typically found on Sandy bottoms |

**Table 2 . Common habitat, depth and depth range of gastropods identified in sediment samples S3H and S6H**

## *Bivalves*

Although mollusc fragments were identified in the sediments they were not large enough to be distinguished as bivalves or to determine the species.

*Foraminifera*

A single species of benthic foraminifera was identified: *Archais angulatus*. This foraminifera is a common carbonate sediment producer and is abundant on shallow reefs and open shelf sites and is commonly found attached to plants in quiet back reef environments. (Figure 12).


**Figure 12** Example of *Archais angulatus* identified in S3H scale shown is in microns

*Echinoid fragments*

Fragments of echinoid plates and spines of two common species were identified (Figure 10 & 11); *Diadema antillarum* and *Lytechinus variegates*. *Diadema antillarum* is most often found in areas with hard surfaces as this species is often found in rock crevices however some are free roaming on sandy substrates. They can be found in the subtidal zone to water depths of 400m. *Lytechinus variegates* is typically found in calm, clear waters, not typically deeper than 50m, often associated with sea grass beds but can also be found on un-vegetated sandy substrates.

*Coral Fragments*

Some small corallites were identified contributing to the sediment. Fragments of coral were identified based on carbonate composition, presence of septa and shape of fragments.

Lithics

Non biogenic clasts were identified in sample S3H (Figure 13 & 14). These clasts are not carbonate in composition and do not exhibit any internal characteristics of the bioclasts know to be carbonate sediment contributors. These clasts have not been further classified as this would require petrographic analysis.

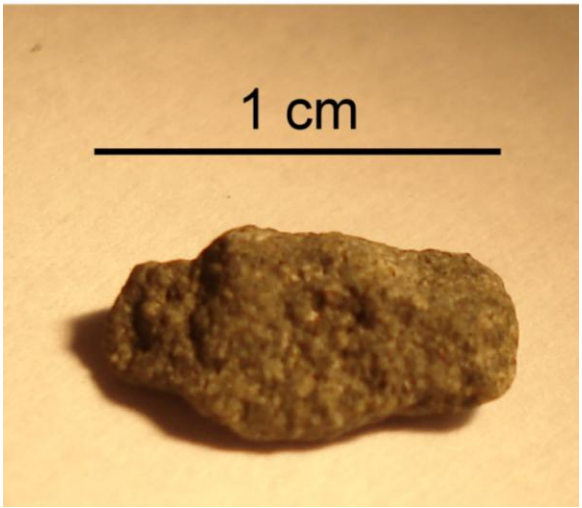



**Figure 13 Lithic clast identified in sediments**


**Figure 14 Unknown lithic clast identified in sediments**

## Conclusions

Grain size analysis conducted on samples S3H and S6H show that sediments along the north east section of the Lucea harbor are similar; both samples are gravelly sands with distinctive coarse and fine fractions. There is however a larger portion of gravels (19.4%) in sample S3H than S6H which makes it clearly bimodal; this coarser fraction is made up of coral fragments, molluscs and lithics. Both samples are comprised of sand and gravels; with no or negligible amounts of mud. Sample S3H is classified as a poorly sorted coarse sand. Texturally it is comprised primarily of sands (82%) and gravels (18%). Sample S6H is a moderately well sorted sample dominated (99.5%) by sand sized grains and is classified as fine sand. Variations in mean grain size and sorting between the samples may be a function of sample point as well as wave dynamics at the two sites.

Compositional analysis shows that crystalline grains appear to be the major producers of sediment in these two samples. Both samples consist of 60% crystalline grains with molluscs being the next most abundant biogenic contributor (15%). Other common contributors are algas (red & green); foraminifera; coral and echinoid fragments. Although lithic fragments were identified in Sample S3H they account for only 10% of the total sample and should be considered minor contributors.

Bibliography

[1] Folk RL. 1954. The distinction between grain size and mineral composition in sedimentary-rock nomenclature. *Journal of Geology* **62**:344–359.

[2] Twenhofel, W. H., and S. A. Tyler. 1941. *Methods of Study of Sediments*. New York and London: McGraw-Hill Book Company Inc

[3] Folk RL, Ward WC. 1957. Brazos River bar: a study in the significance of grain size parameters. *Journal of Sedimentary Petrology* **27**: 3–26.

[4] Udden JA. 1914. Mechanical composition of clastic sediments. *Bulletin of the Geological Society of America* **25**: 655–744.

[5] Wentworth, C. K. 1922. A scale of grade and class terms for clastic sediments. *Journal of Geology* 30:377-392.Abbott, R. Tucker, and Percy A. Morris. 1995. *Shells of the Atlantic & Gulf Coasts & the West Indies*. Fourth ed, *Petersons Field Guide*. New York: Houghton Mifflin Company.

[6]Friedman GM, Sanders JE. 1978. *Principles of Sedimentology*. Wiley: New York.

APPENDIX 1- sieve data

Sample: S3H Initial sample weight- 100.03g

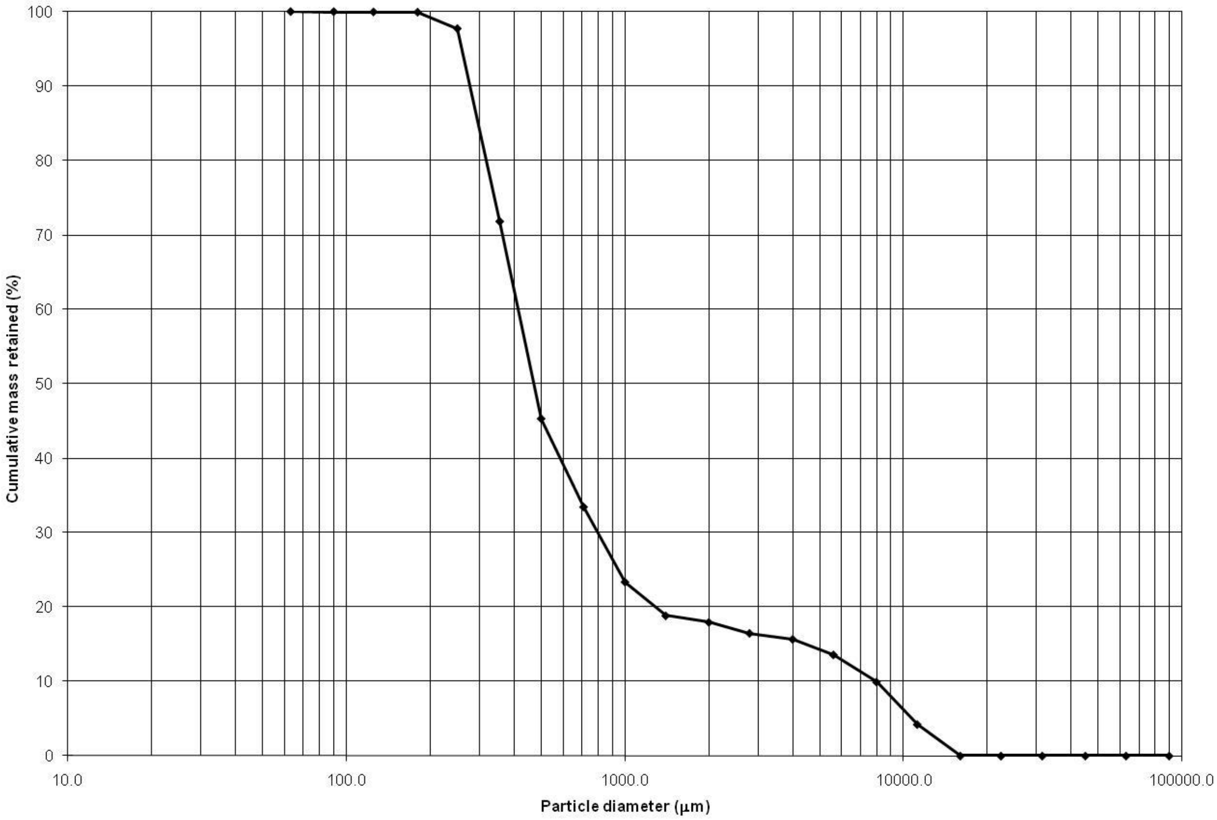
| **Aperture (microns)** | **Class Weight Retained (g or %)** |
|---|---|
| | |
| 11200 | 4.18 |
| 8000 | 5.63 |
| 5600 | 3.61 |
| 4000 | 2.06 |
| 2800 | 0.8 |
| 2000 | 1.45 |
| 1400 | 0.87 |
| 1000 | 4.48 |
| 710 | 9.95 |
| 500 | 11.72 |
| 355 | 26.16 |
| 250 | 25.58 |
| 180 | 2.17 |
| 125 | 0.03 |
| 90 | 0.04 |
| 63 | 0.01 |
| | 0 |

Sample : S6H Initial sample weight- 100.09 g

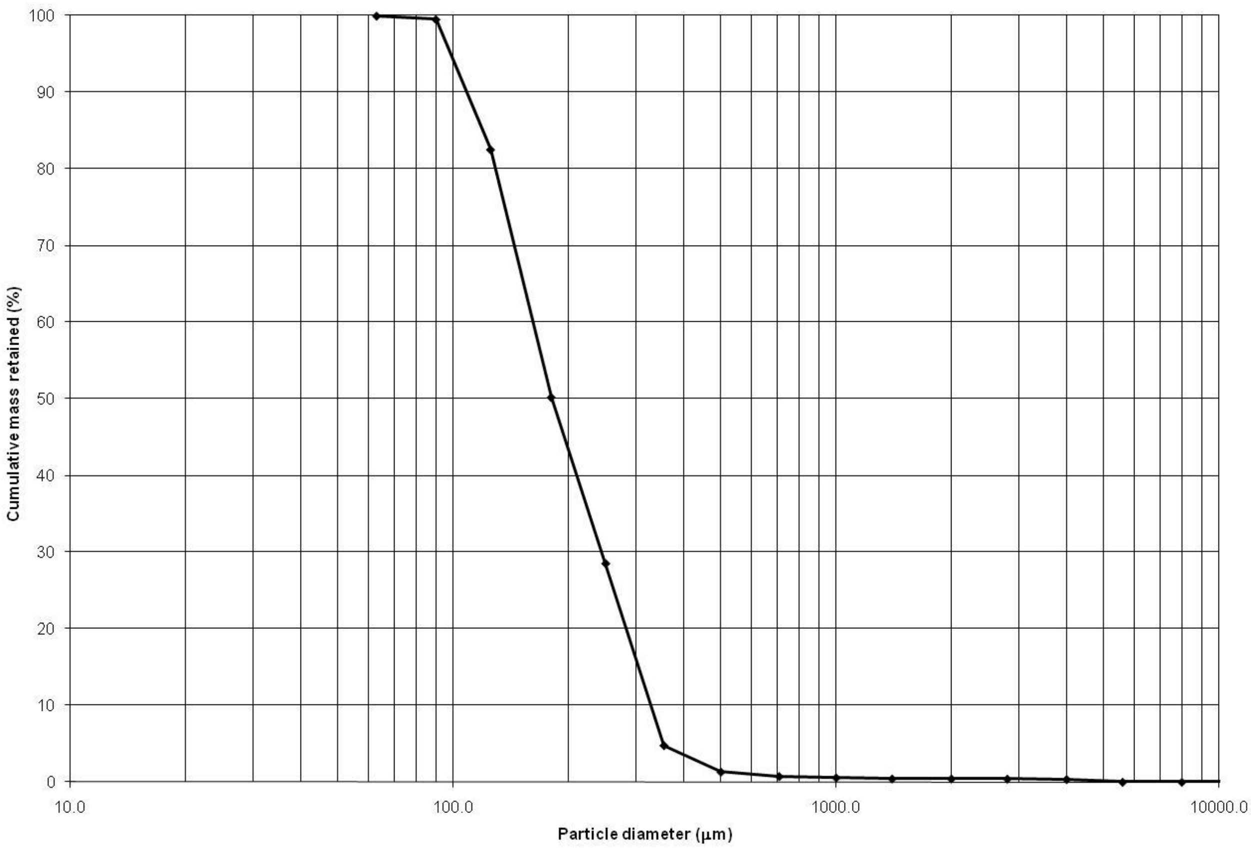
| **Aperture (microns)** | **Class Weight Retained (g)** |
|---|---|
| 4000 | 0.35 |
| 2800 | 0.06 |
| 2000 | 0.02 |
| 1400 | 0.00 |
| 1000 | 0.14 |
| 710 | 0.17 |
| 500 | 0.58 |
| 355 | 3.44 |
| 250 | 23.63 |
| 180 | 21.57 |
| 125 | 32.16 |
| 90 | 16.90 |
| 63 | 0.45 |
| | 0.06 |

APPENDIX 2

Sample: S3H



Sample S6H



100
90
80
70
60
50
40
30
20
10
0
Cumulative mass retained (%)
10.0
100.0
1000.0
10000.0
Particle diameter (µm)

# MIKE 21

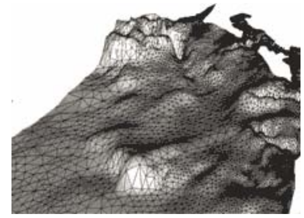
**MIKE 21** is a professional engineering software package for the simulation of flows, waves, sediments and ecology in rivers, lakes, estuaries, bays, coastal areas and seas. The modelling system is designed in an integrated modular framework with a variety of add-on modules. This, in combination with the range of dedicated and easy to use tools and editors, allow you to customise 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, eg 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 modeling
- 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, ie 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, eg 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 are 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

**MIKE 21 ST** has full compatibility with LITDRIFT

## MT Mud Transport Module

**MIKE 21 MT** is a combined multi-fraction and multi-layer model that describes erosion, transport and deposition of mud or sand/mud mixtures under the action of currents and waves.

Processes that can be included in the simulation are forcing by waves, sliding, salt-flocculation, detailed description of the settling process, layered description of the bed, and morphological update of the bed.

For example, waves calculated by one of the MIKE 21 wave modules may be used to include the wave or wave-current induced effect on shear stresses. Flocculation in the water column is taken into account through optional descriptions of the settling velocity dependency of salinity and concentration. Furthermore, hindered settling and consolidation in the fluid mud and under-consolidated bed are included in the model. Bed erosion can either be non-uniform; i.e. the erosion of soft and partly consolidated bed, or uniform; i.e. the erosion of a dense and consolidated bed. The bed is described as layered and characterized by the density and shear strength.

**MIKE 21 MT** is typically applied to the study of the following engineering problems:

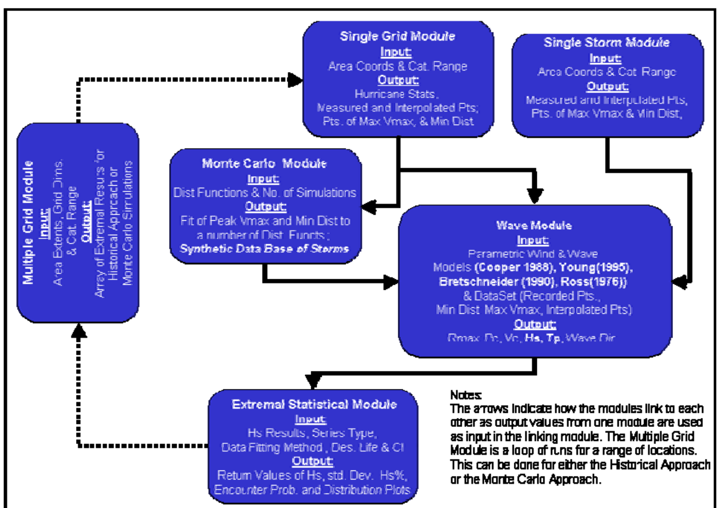
- sediment transport studies for fine cohesive materials or sand/mud mixtures in estuaries and coastal areas in which environmental aspects are involved and degradation of water quality may occur
- siltation in harbours, navigational fairways, canals, rivers and reservoirs
- dredging studies
- morphology

# HURWAVE

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.



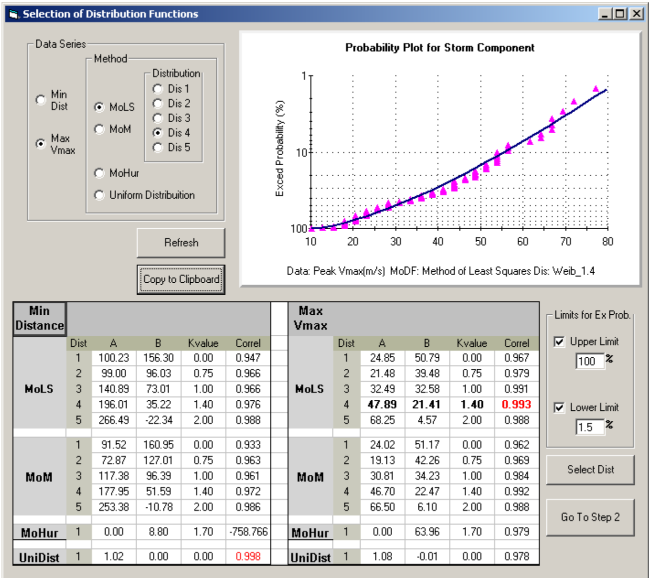

**Flow Diagram For the Program -HURWave**

### 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).

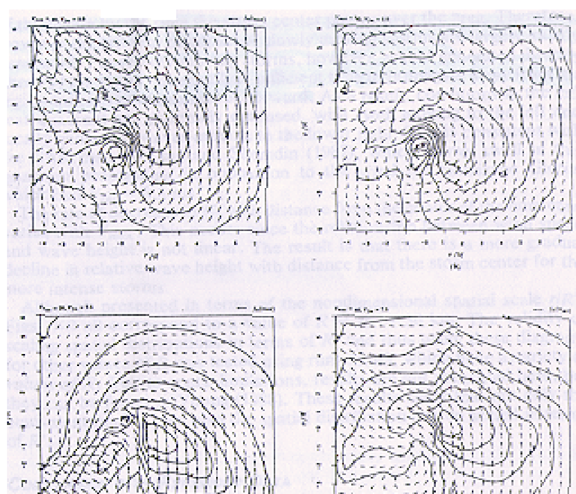



## Parametric Wave Modeling

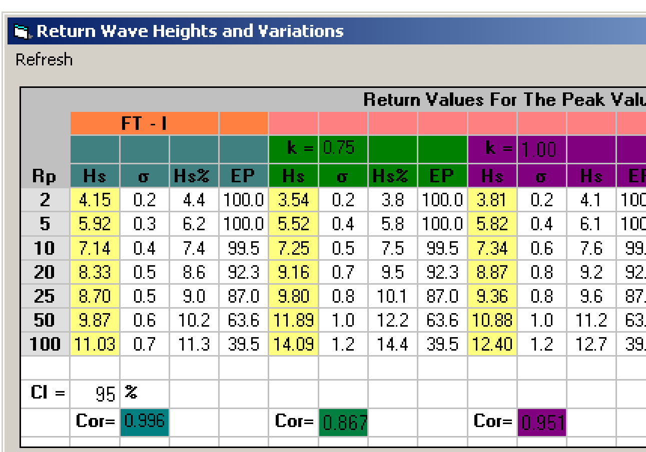
A number of widely used wind and wave models are applied to produce a hindcast dataset of hurricane wave conditions at the point in question. These models include Cooper (1988) and Young (1995).

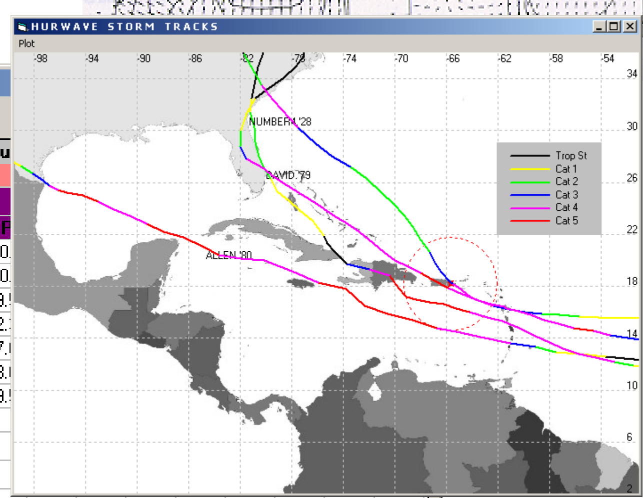
The Cooper model was developed by statistically analyzing the output from numerical wind and wave models for 6 Gulf of Mexico hurricanes. The storms used covered a wide cross-section of hurricane conditions.

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.

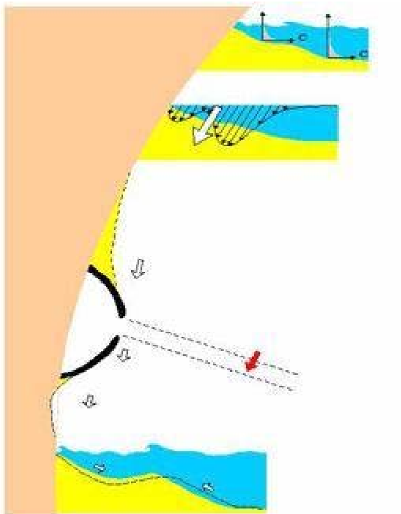


With the results of these models, a range of extremal statistical analyses may be carried out in HURWave. The extremal methods applied are based on work published by Yoshima Goda in 1988 for statistically analyzing extreme events such as hurricane waves. Distribution functions such as Weibull and Fischer 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.



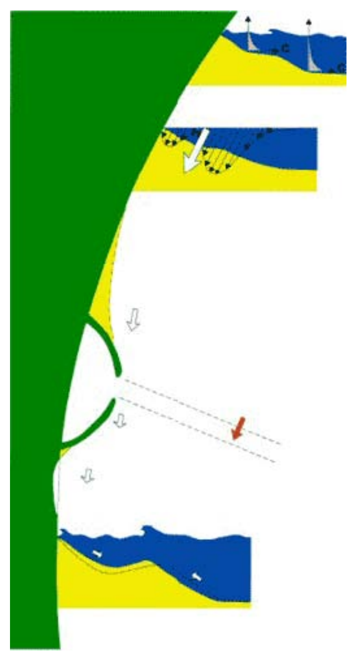
# LITPACK



**LITPACK** -The DHI Software Package for Littoral Processes and Coastline Kinetics

Design and implementation of effective shoreline management strategies, either locally or regionally, require detailed knowledge of the ongoing physical processes controlling the transport and sedimentation of beach materials. LITPACK applies a unique deterministic approach to give you a very powerful tool for a wide range of coastal zone management applications.

**LITPACK** combines a technically very strong deterministic sediment transport model with user-friendly facilities for the simulation of a large number of wave/current scenarios and for the combination of these simulations into predictions of the net littoral drift, developments of coastal profiles, and long-term coastline evolution.



**LITPACK** is a MIKE Zero based product and requires the PP Module. LITPACK consists of the following modules:

Main modules:

- **LITSTP** the deterministic sediment transport model for non-cohesive sediment in waves and current **LITDRIFT** the littoral sediment module for littoral drift along a uniform coastline with an arbitrary coastal profile

Add-on modules:

- **LITLIN**E the coastline evolution model for coastline development due to changes in transport capacity
- **LITTREN** the trench sedimentation model for channel back-filling due to non-equilibrium sediment transport mechanism
- **LITPROF** the profile evolution model for profile development due to cross-shore transport

Where the MIKE 21 sediment transport modules operate in the time domain and simulate the sediment transport potential in area with complex bathymetries, LITPACK assumes a long uniform coastline and stationary sediment transport during each simulation. Thus, the two tools supplement each other very well.

LITPACK is fully compatible with MIKE 21. Many of the pre-and post processing facilities are common for the two packages, and transfer of results between the two systems is easy. All LITPACK modules are currently operated through an efficient interactive menu system.

Context sensitive online help is provided. LITPACK includes the possibility of comprehensive run time graphics simplifying model calibration and application. Simulations may be executed interactively, or the user may setup batch execution of one or more simulations.

## LITSTP Module

The **LITSTP** module is one of the basic modules and comprises a deterministic non-cohesive sediment transport model – STPQ3D. This is a quasi 3D model that calculates the non-cohesive sediment transport in combined waves and currents. The LITSTP Module forms the basis for all sediment transport calculations made in other LITPACK modules - and in MIKE 21 ST.

STPQ3D solves the vertical sediment diffusion equation on an intrawave period grid to provide a detailed description of the non-cohesive sediment transport for breaking/non-breaking waves and current.

STPQ3D accounts for:

- Waves and currents at arbitrary angles
- Breaking waves
- Plane/ripple-covered bed
- Uniform/graded bed material or shingle
- Effect of bed slope
- Effect of streaming

The outcome of a simulation is the time-varying as well as the time-averaged profiles of eddy-viscosity and bed and suspended load in two directions. Also the instantaneous values of internal parameters, eg near-bed velocity, shields parameter and bed concentration, are given as output.

### Typical applications

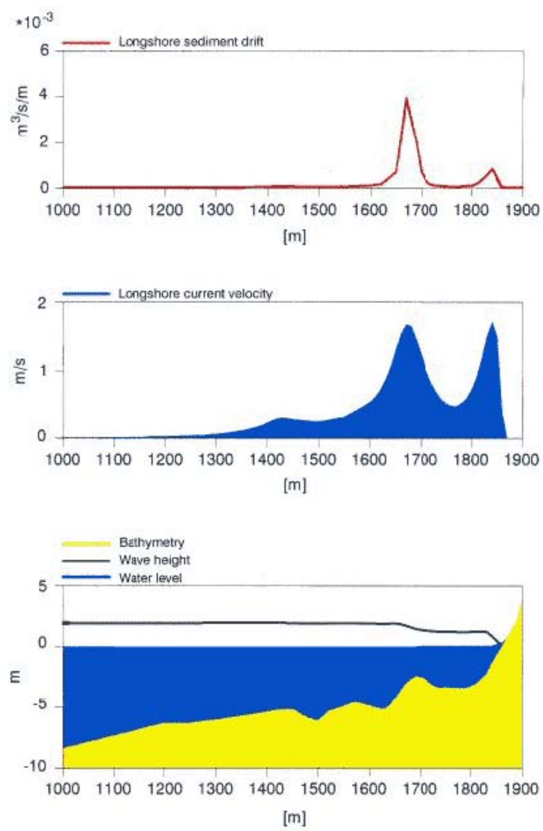
LITSTP can be applied to the study of non-cohesive sediment transport in waves and currents, where the input parameters are the wave, current and sediment properties for the specific point. The assessment of transport rates is essential for the estimation of littoral sediment drift (may be calculated by the module LITDRIFT). The concentration profiles become vital information when the vertical position of water intakes is planned and when investigating harbour siltation. Having the possibility to include undertow effects in the calculations, the formation of bars on a cross-shore profile may be investigated (by the module LITPROF). A facility in LITSTP enables the user to obtain the inter-period parameters in an output file in order to investigate the sediment transport mechanisms in detail.

**Main Features**

The main features of LITSTP sediment transport module are as follows:

- Deterministic Approach
- Output of transport rates in user-defined directions
- User defined sediment descriptions
- Options to include convective terms
- User defined calculation parameters

# LITDRIFT Module




The **LITDRIFT** module combines the sediment transport model STPQ3D with a coastal hydrodynamic model to give a deterministic description of the littoral drift. **LITDRIFT** provides a powerful tool for sediment budget analysis, which is of paramount importance to all coastal morphology studies.

The **LITDRIFT** module simulates the cross-shore distribution of wave height, setup and longshore current for an arbitrary coastal profile. It provides a detailed deterministic description of the cross-shore distribution of the longshore sediment transport for an arbitrary bathymetry for both regular and irregular sea states.

**LITDRIFT** solves the longshore and cross-shore momentum balance equation to give the cross-shore distribution of longshore current and setup. Important factors, such as wave decay due to breaking and linking of the water level and the profile to the incident sea state are included.

**LITDRIFT** accounts for:

- regular/irregular waves
- water levels
- tidal currents

- wind shear stresses
- non-uniform bottom friction
- wave refraction and shoaling
- breaking waves
- non-uniform sediment distribution

The outcome of the simulation of one single wave event is the cross-shore distribution of water level, longshore current, wave height and wave angle, water flux, bed load and suspended load transport, total load and cumulative total load transport. For annual drift calculations, or over a specified design period, the total drift is found as the weighted sum of contributions from all events in the hydrographic database or from a time series of hydrographic boundary conditions.

### Typical applications

**LITDRIFT** can be applied to the study of wave driven currents and longshore sediment transport of non-cohesive sediment on a long uniform beach. The assessment of the wave conditions - wave heights, wave periods and wave directions - is essential for the estimation of the wave forces at a shoreline. Another important problem in coastal engineering is the simulation of the sediment transport, which for a large part is determined by the wave-induced littoral current. The wave-induced current can be generated by the strong gradient in radiation stresses which occur in the surf zone. LITDRIFT can be used to calculate the radiation stresses; the wave generated longshore current and the longshore sediment transport rate.

A facility in **LITDRIFT** enables the user to calculate the annual sediment budget for the location, based on time series as input. Another facility enables the user to transfer the wave climate from deeper water to a point in the profile or to obtain the conditions at a specific single point in the profile for the entire time series with the purpose to calculate the sediment transport later by the STP Model Type Multiple STP Calculations.

### Main Features

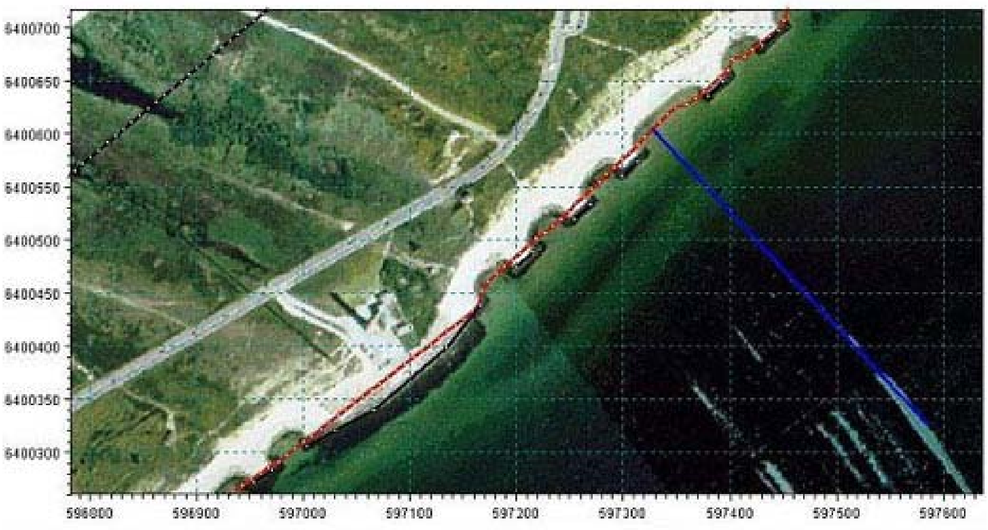
The main features of LITDRIFT are as follows

- Deterministic Approach
- Local hydrodynamics for sediment transport
- Measured time series as input
- Graphics of results while calculating

# LITLINE Module

**LITLINE** is used for studying the evolution of coastlines when influenced by various structures or sources and sinks. LITLINE is therefore a powerful and reliable tool for impact assessment and the design and optimization of many coastal engineering projects.

Based upon the results from LITDRIFT, LITLINE simulate the coastal response to gradients in the longshore sediment transport capacity resulting from natural features and a wide variety of coastal structures.




LITLINE calculates the coastline evolution by solving a continuity equation for the sediment in the littoral zone. The influence of structures, sources and sinks are included.

The structure types in **LITLINE** include:

- groynes
- jetties
- revetments
- offshore breakwaters

With jetties and breakwaters the influence of diffraction on the wave climate is included.

The results of the simulation are the coastline position, longshore sediment transport rates and the depth in from of revetments, if they are present. The accumulated volume of material deposited and bypassed is also given.

The evolution of the coastline can be calculated, either based on a yearly net littoral drift or from a time series of wave events. The latter option offers the capability of studying coastline movements during a winter season (for instance maximum retreat) or coastline changes on a coast with monsoon climate.

**Application Areas**

Some specific applications for LITLINE are:

- Coastline impact arising from coastal works
- Functional optimisation of coastal protection
- Design of beach recovery through nourishment

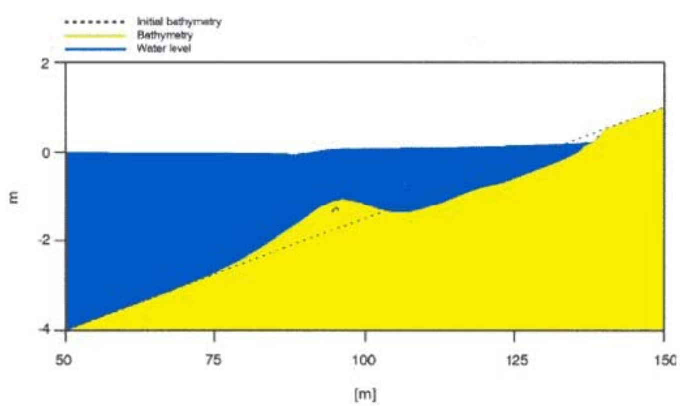
**Main Features:**

The main features of the LITLINE coastline evolution module are:

- Deterministic description of transport rates and distribution over the profile
- Influence of structures automatically included
- Measured or pseudo time series as input
- Time varying sediment sources
- Wide range of coastal structures
- Graphics of results while calculating

# LITPROF Module

**LITPROF** -Cross-shore Profile Evolution




Storm profile response and the response of beach nourishment to storm conditions can be investigated with the profile evolution model, LITPROF.

**LITPROF** is therefore an aid for understanding the mechanisms of cross shore sediment transport and a tool for estimation of on-offshore transport rates.

**LITPROF** describes the cross-shore profile changes based on a time series of wave events. LITSTP provides the deterministic basis for quantifying the cross-shore transport distribution.

**LITPROF** describes cross-shore profile changes by solving the bottom sediment continuity equation, based on the sediment transport rates calculated by LITSTP. b, being a time-domain model, includes the effects of changing morphology on the wave climate and transport regime. This enables a simulation of profile development for a time-varying incident wave field.

**LITPROF** has the possibility to include structures to the profile, modelling non-erodible areas.

The effects and facilities include:

- Time series of wave heights and water levels
- Shoaling and refraction of waves breaking
- Transport, including the effects of undertow, Lagrangian drift, streaming and bed slope
- Possibility of fixed bed, submerged breakwater and revetment.

## Application Areas

Some specific applications for LITPROF are:

- Profile response to various conditions
- Fate of nourished material
- Null point location of structures

## Main Features

The main features of the LITPROF cross-shore profile evolution module are:

- Heuristic transformation of the deterministic description of transport rates and distribution across the profile
- Measured or pseudo time series as input
- Time domain model
- Graphics of results while calculating

# APPENDIX C

# DETAILED MODELING RESULTS

# HURWave Transformation and MIKE 21 Modeling Results for Hurricane Waves

## *Historical Hurricanes*

Jamaica is exposed to hurricane activity each year between the months of June and November. From the database of the US National Hurricane Center (NHC), 122 tropical storms and hurricanes have passed within 300km of the project site since 1900. The numbers of occurrences within each category, as well as the wind speed classifications were broken down according to the categories described by the Saffir Simpson scale and is presented in Figure 1 Scheme a). Figure 1 Scheme (b) shows the temporal distribution of the storm events that have passed within the 300km radius of Jamaica since 1900. Figure 1 Scheme (c) show tracks of notable hurricanes that have affected the project site. Three Category 5 hurricanes and seven Category 4 hurricanes have passed within the 300km of the site. The Category 5 included Allen (1980), Ivan (2004) and Emily (2005). The site has been exposed, on average, to one to two tropical storms or hurricanes per year.

## *Deep Water Wave Conditions*

Several models are available to estimate the generated deep water wave conditions and water levels given certain basic parameters of a hurricane, which can be obtained from historical data. The term "parametric models" means that the models require the input of a few specific parameters. These parametric models, in most cases, rely on the simplification or the parameterization of numerical formulations related to wind-wave generation theories in combination with results of complex spectral models.

The high waves experienced during a hurricane are caused by the high wind speeds associated with the hurricane. The water level increase in deep or intermediate water depths comes about largely from the phenomenon called Inverse Barometric Pressure Rise (IBR). This is caused by the low pressure system in the eye of the hurricane (the low pressure causes the water level to rise). The IBR can also be computed using a parametric model.

### Wave Heights

The parametric model of Young1  was used to calculate the deep water extreme wave conditions. The NOAA database of hurricane records, which dates back to 1900, was used in this analysis. All hurricanes passing within a 300 km radius of Grand Palladium were selected from the larger database. The directional distribution of deep water wave heights calculated in HurWave is shown in Figure 1 Scheme d). It can be seen from this plot that waves approach most frequently from the east and north-east directions; however there are contributions from waves from all other directions. This is because of the typical west-north-westerly tracks of the hurricanes and the anticlockwise rotating wind field.

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[1] Young, I.R., 1988. *A Parametric Model for Tropical Cyclone Waves.* Research Report No. 28, University of new South Wales.

The plot also shows that waves from 2m to 4m are most frequent. The waves between 4m and 6m have the second highest rate of occurrence. Also of note are a few exceptional waves having extreme heights of over 14m; these approach from the east. The north western coast of Jamaica, which includes our area of interest, is typically exposed to the waves approaching from the east to southwest counter clockwise. Therefore the deep water wave conditions were filtered into the following directional bins:

Sector 1 – Waves from the north (337.5º – 22.5º)

Sector 2 – Waves from the north-east (22.5º– 67.5°)

Sector 3 – Waves from the east (67.5º – 112.5º)

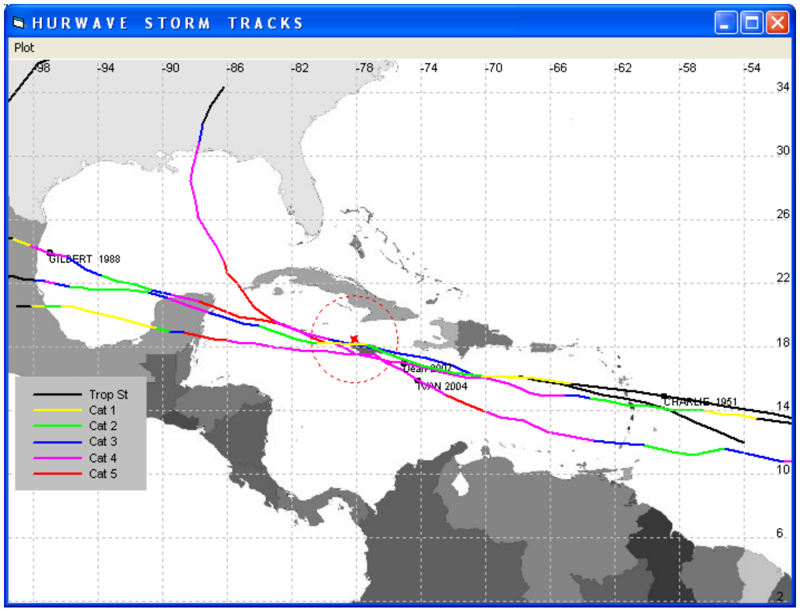
Sector 4 – Waves from the south-west (202.5º–247.5°)

Sector 5 – Waves from the west (247.5º–292.5°)

Sector 6 – Waves from the north-west (292.5º–337.5°)

Using the parametric model and the historical hurricane data, a data series of deep water wave heights was computed. A statistical analysis was carried out according to the method of Yoshima Goda (1990). The data was fit to various statistical distributions, and the best fit distribution was determined from the correlation as well as the goodness of the fit to the most extreme values in the distribution. Figure 1 Scheme e) shows a plot of the data fitted to the Weibull distribution.

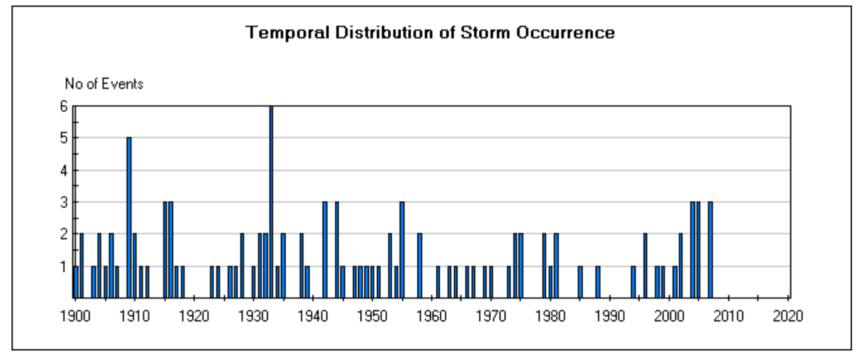
c)



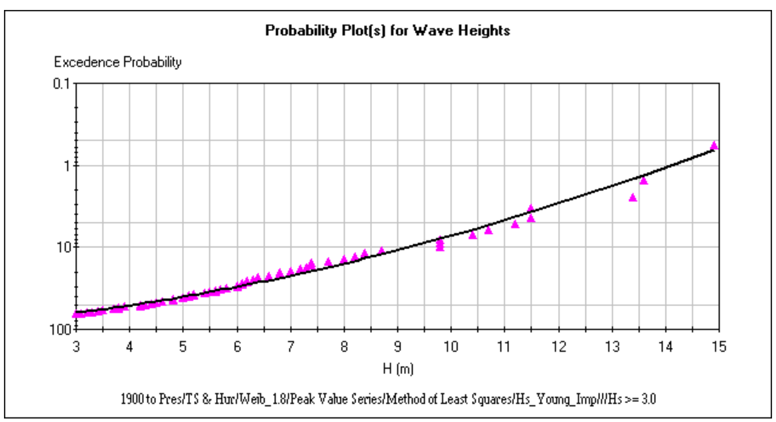

a)

| Cyclone Category | Wind Speed (m/s) | Wind Speed (km/h) | No. of Events |
|---|---|---|---|
| Tropical Storm | 18 – 33 | 64 – 118 | 59 |
| 1 | 33 – 43 | 119 – 154 | 18 |
| 2 | 44 – 49 | 155 – 178 | 9 |
| 3 | 50 – 58 | 179 – 210 | 10 |
| 4 | 59 – 70 | 211 – 250 | 7 |
| 5 | > 70 | > 250 | 3 |

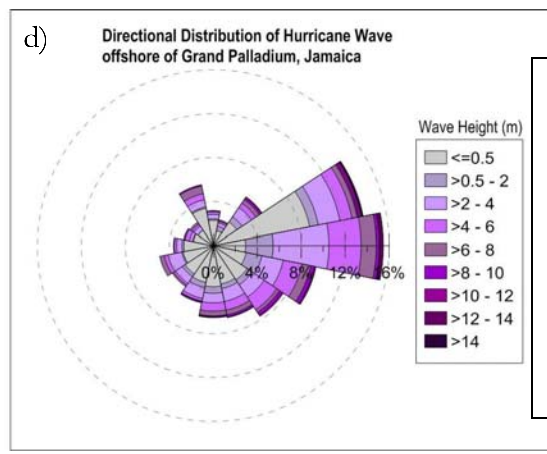
b)




e)




d)




a)- Events in each Saffir Simpson hurricane category

b)- Events per year since 1900

c)- Tracks of notable hurricanes

d)- Direction distribution of offshore waves

e)- Curve fitting of Extreme Events

Figure 1 Summary of HurWave Analysis at Jamaica, within 300km of the project site

**Water Levels**

The elevated water level that accompanies hurricanes and creates flooding and causes damage to coastal infrastructure is known as storm surge. The rapid rise in water level that accompanies an intense hurricane is mainly due to the effects of strong winds and low pressure as the storm passes a given point in shallow water. The water level rises above mean sea level to create the static storm surge. The static surge is made up mainly of five components, namely:

1. *The Inverse Barometric Rise (IBR)* – The IBR is the rise in the water surface elevation caused by the low pressure centre of the hurricane. It has its peak at the eye of the storm, decreasing with increased distance from the centre or eye.

2. *Highest Astronomical Tide (HAT)* – The HAT is the highest level that daily tidal variations may reach. This level can be accurately predicted and is available from tide charts. It is important to include this water level, as it is possible for the storm to occur while the sea level is already at this elevation.

3. *Global Sea Level Rise (GSLR)* – The GSLR has been predicted by scientists according to past and present rates of sea level variations and forecasting of the effects of global warming on the melting of polar ice caps. This present rate of increase for this part of the Caribbean is predicted to be approximately 0.25 m for the next 50 years.

4. *Wind Setup* – Wind setup is a result of intense winds blowing over the water surface that causes shear stresses at the water surface. This will tend to push water towards the land. This water will rise more steeply in areas where the water depth is shallow, and therefore further add to the water level rise in nearshore areas.

5. *Wave Setup* – Wave setup includes the increase in water elevation due to the dissipation of wave energy as waves approach the shoreline and start to break. During wave breaking, wave heights steepen as the wave velocity slows due to the effects of bottom friction on the seabed.

The final component of inundation is termed wave run-up and occurs at the landward limit of the nearshore zone in an area called the swash zone. This is the area where waves run up and down the shore, wetting and drying with each wave. The elevation of the run-up is dependent on the surface characteristics of the "swash zone". If this area is a smooth impermeable beach, a higher run-up can occur; conversely if a rough armour stone slope or a vegetated surface is encountered, then the run-up would be reduced. Because of the localized variability of wave run-up and the fact that it is a dynamic component, storm surge computations do not usually include wave run-up. It is, however, calculated and used in the design of coastal structures. Figure 2 is a diagrammatic representation of these components of storm surge and their spatial extents of impact.

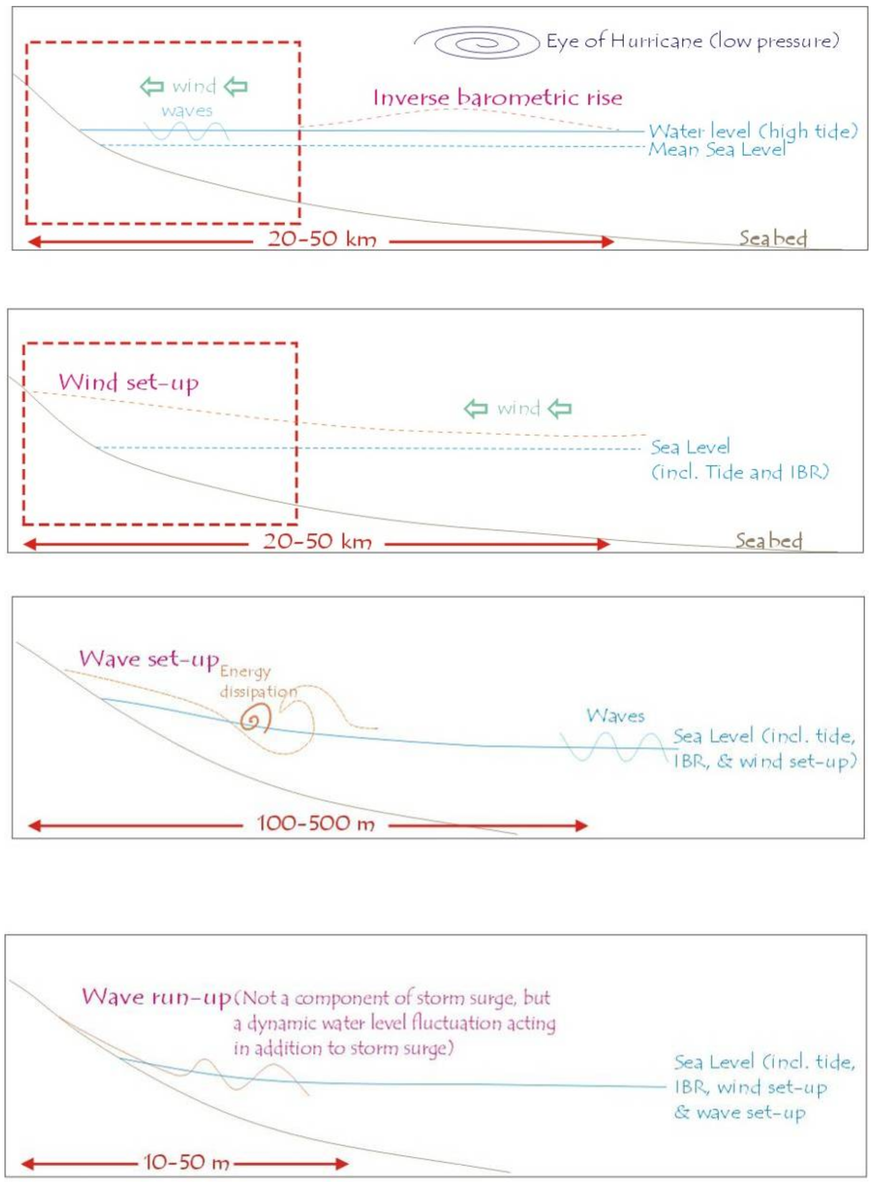



**Figure 2 Component of Storm Surge (Static and Dynamic)**

Storm surge is considered to be static during the hurricane event, as the time period over which it occurs is far greater than the period of the waves generated from the hurricane winds. The HAT and GSLR are assumed to have constant values, as the maxima are used for design purposes. The IBR is dependent on the pressure drop of the storm system; therefore it varies for different return periods. The summation of the IBR, HAT and GSLR were presented in the main report and were used to determine the values of the wind and wave setup parameters. This is so because the wind and wave setup are a function of the water depth at the time of the hurricane, to which the IBR, HAT and GSLR are contributing factors. Following is a summary of the parameters considered.

- IBR levels were computed from each storm hindcast by HurWave and the data fitted to various statistical distributions. The best-fit distribution was selected based on correlation and goodness of fit to the most extreme values. Because of the non-directionality of this phenomenon, the analysis was not carried out on a directional basis.

- Tidal variations were considered based on the output generated from MIKE 21. The results showed that high tide above MSL for Jamaica was 0.25m.
- The expected long-term trends on local and global water levels were also considered and the global sea level rise over the next 50 years as predicted by the UNDP to be 0.25m was used.

These effects were added to the IBR to produce the final static storm surge predictions for the 5 to 150-year return periods presented in the main report.

## *Nearshore Transformation of Waves and Storm Surge*

The preceding section outlined the offshore (deep water) extreme wave conditions, which will now be used as input or boundary conditions to determine nearshore wave conditions.

The parametric models are limited to determining the conditions in deep water (greater than 200m depth). Thus, as with the operational wave conditions, the hurricane deep water wave conditions were transformed to the nearshore area using MIKE 21. The 50-year return period hurricane waves and water levels were used as the design conditions for any proposed coastal structures.

While the worst case direction northwest, was presented in the main report, the following plots show the results of the 2-D wave modeling, with respectively showing the variations in wave height (a) and storm surge (b) over the project area for the 50-year hurricane conditions coming from north, northeast, east, southwest and west.

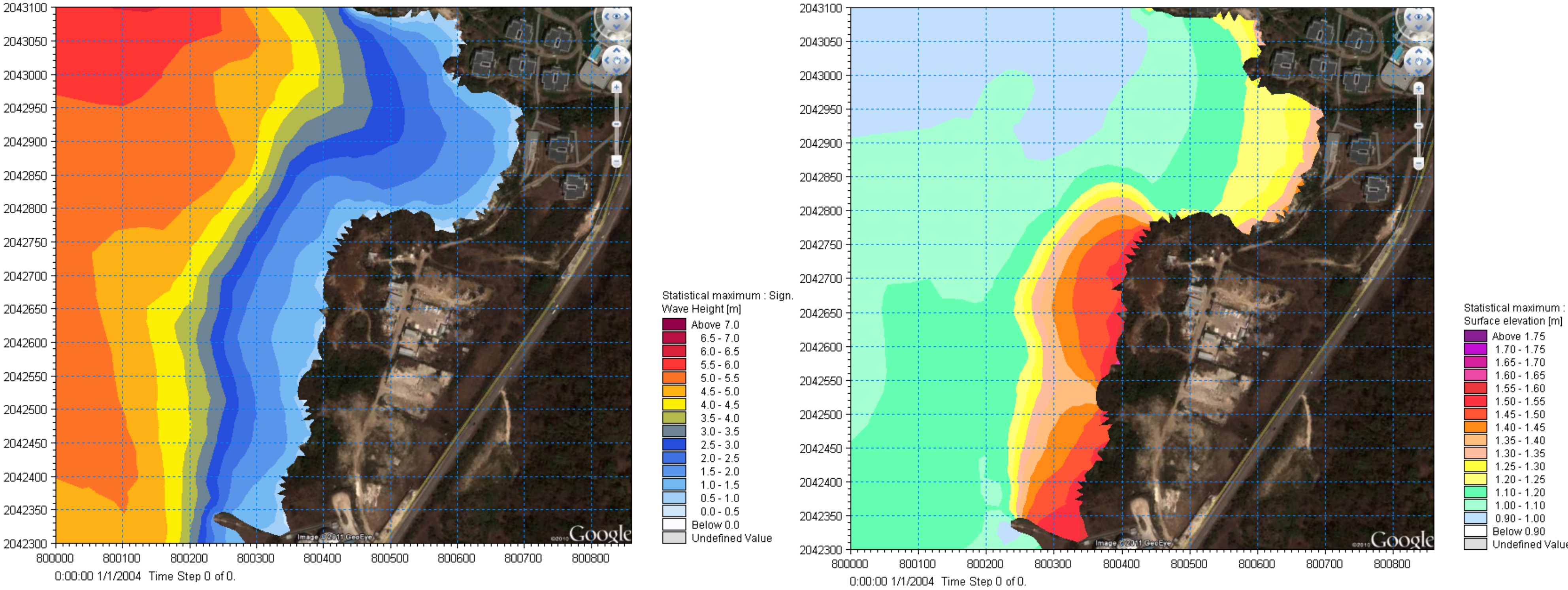
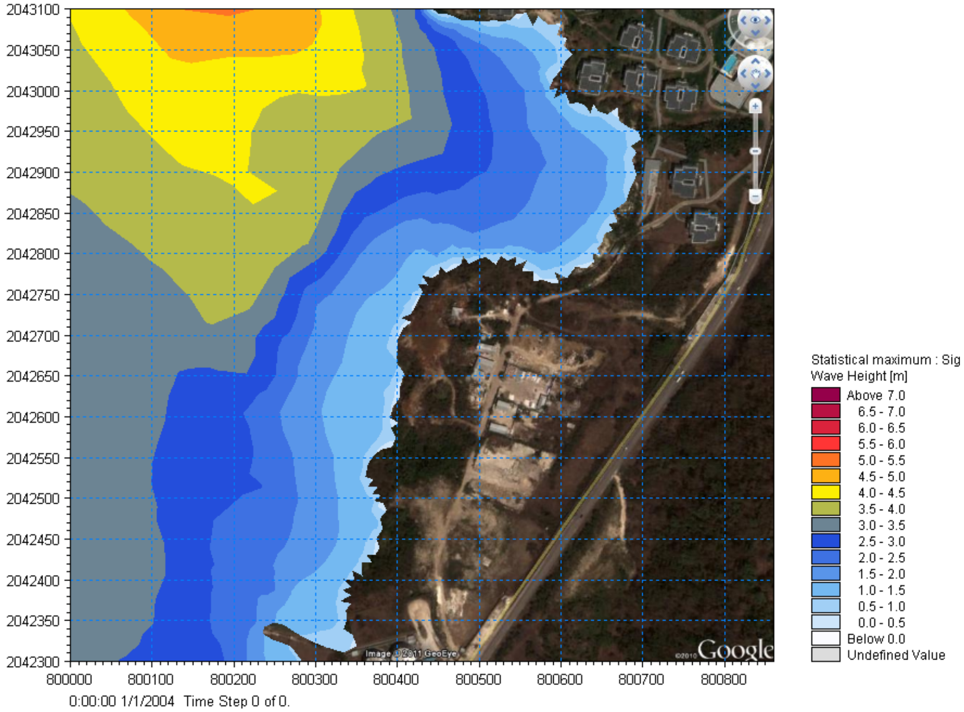



**Figure 3 Maximum wave heights (L) and storm surge (R) for the 50 year hurricane event coming from north**



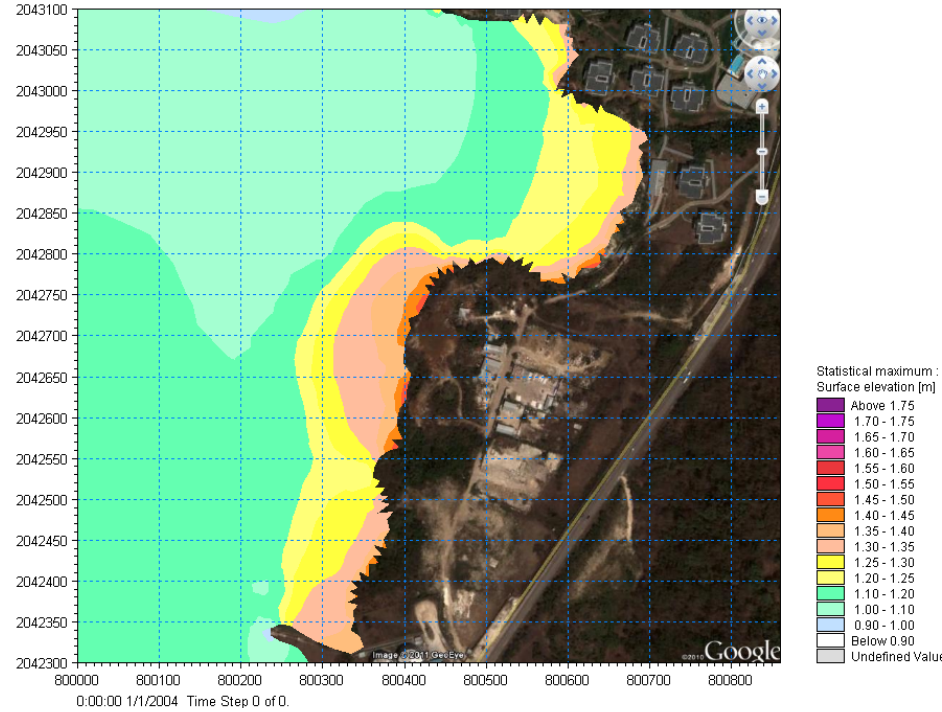




**Figure 4 Maximum wave heights (Left) and storm surge (Right) for the 50 year hurricane event coming from northeast**

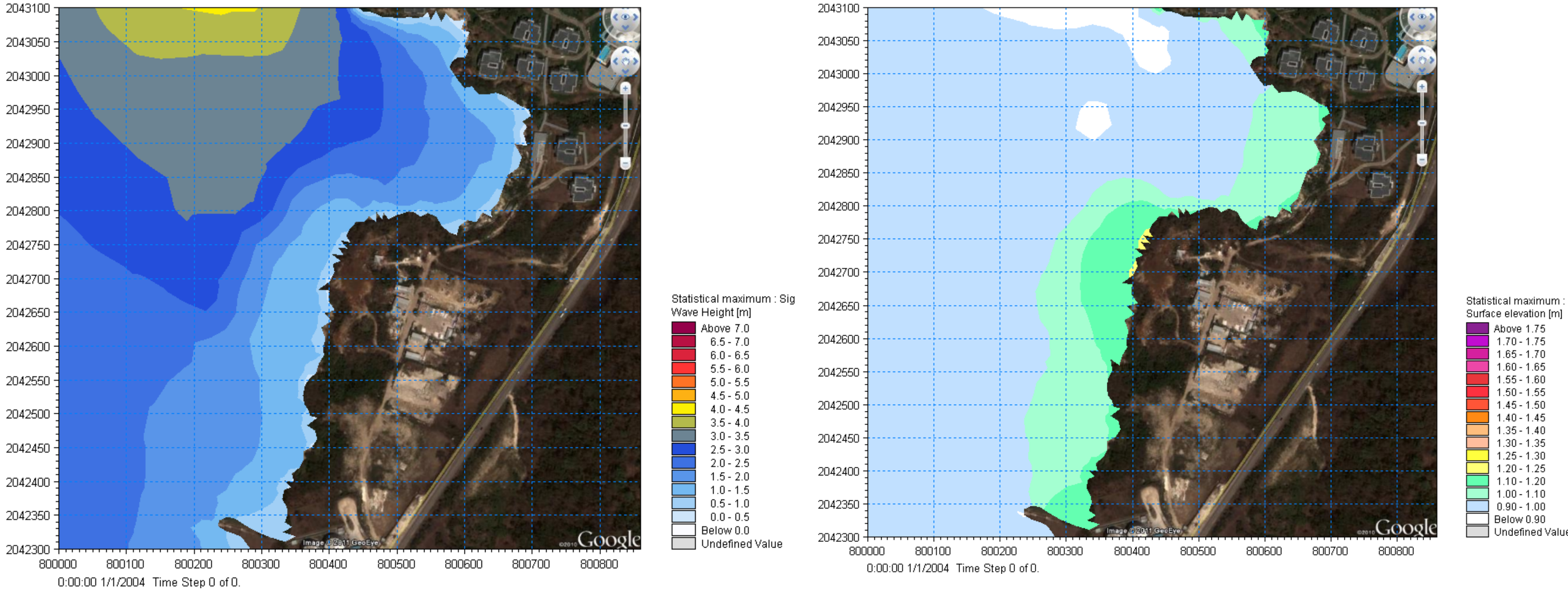
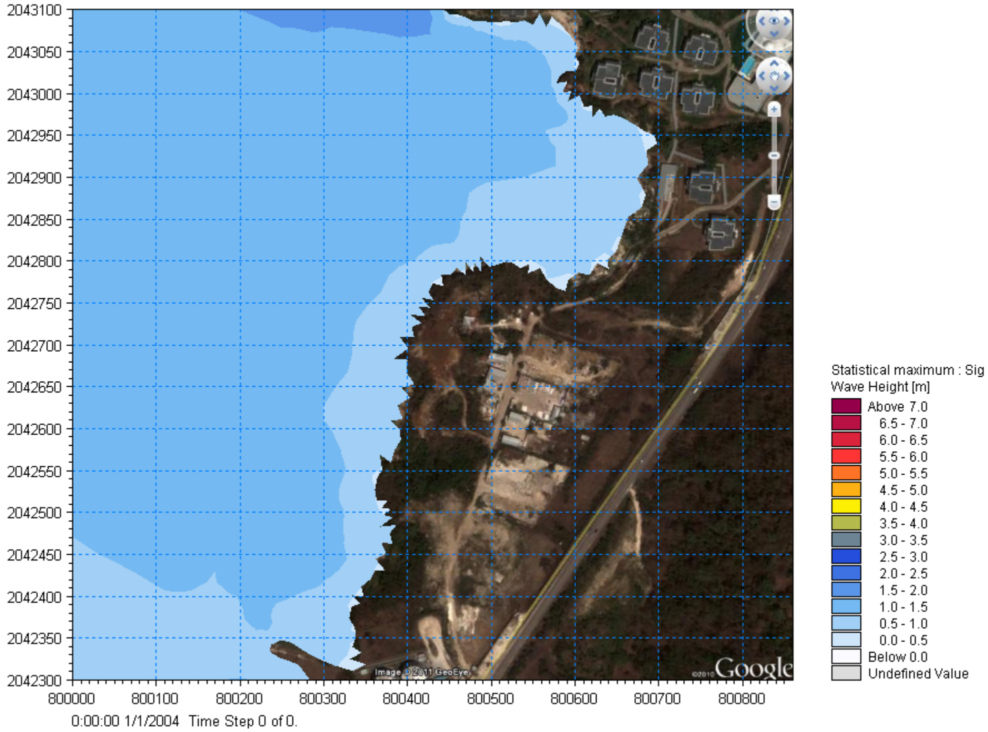



**Figure 5 Maximum wave heights (Left) and storm surge (Right) for the 50 year hurricane event coming from east**



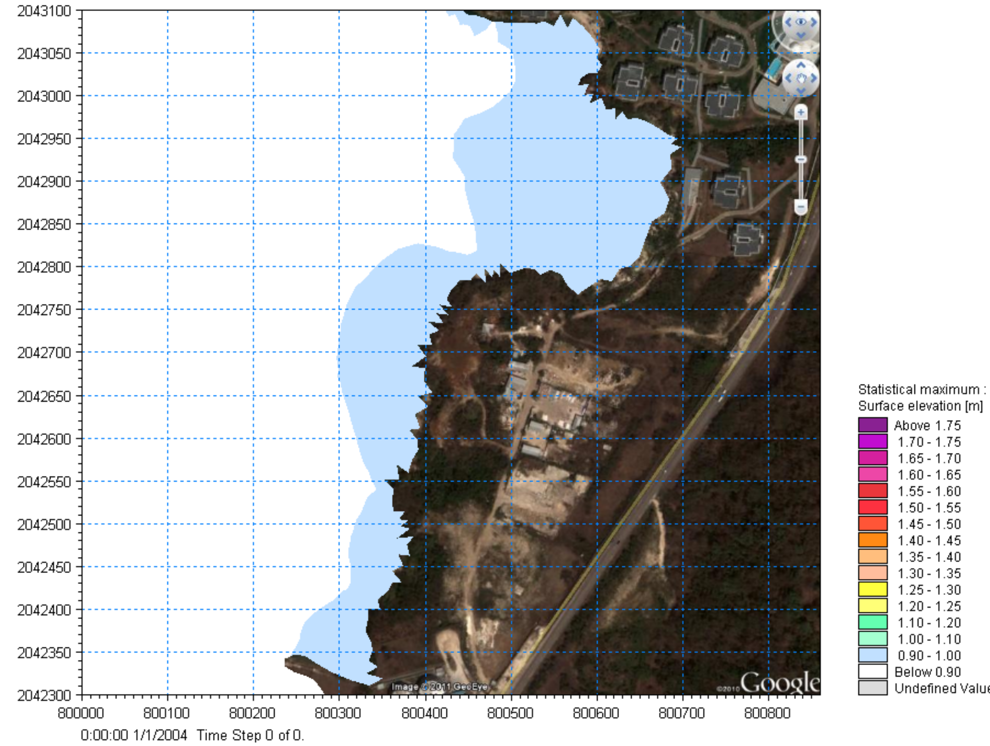




**Figure 6 Maximum wave heights (Left) and storm surge (Right) for the 50 year hurricane event coming from southwest**

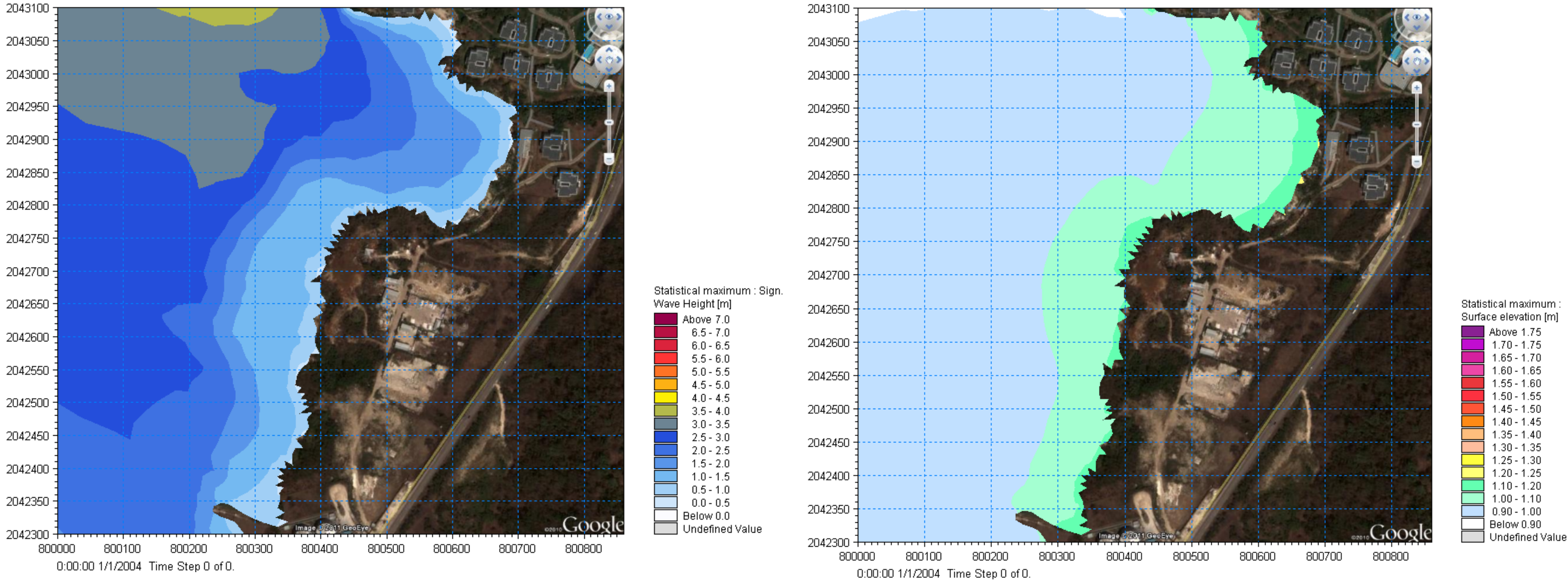
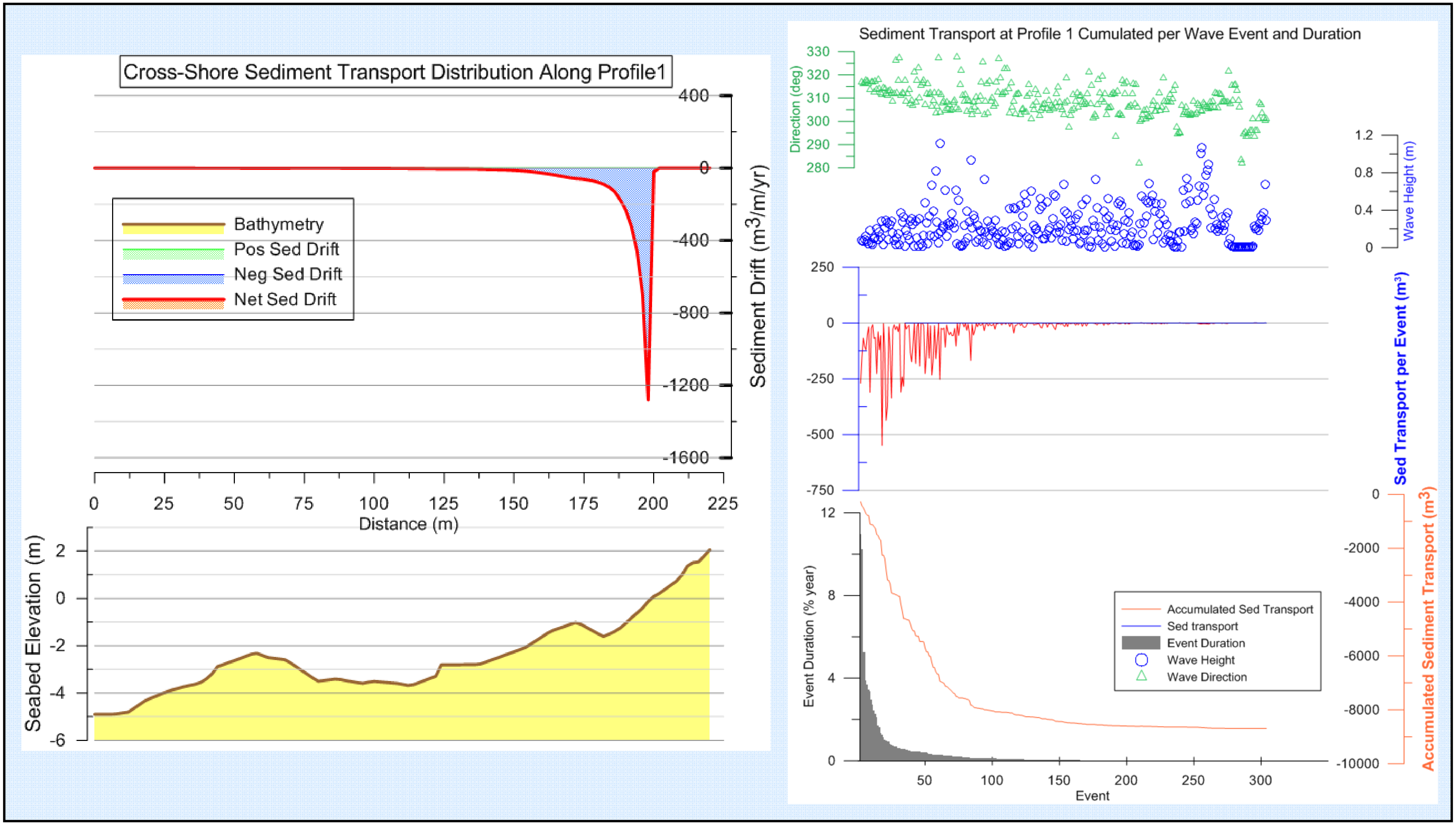
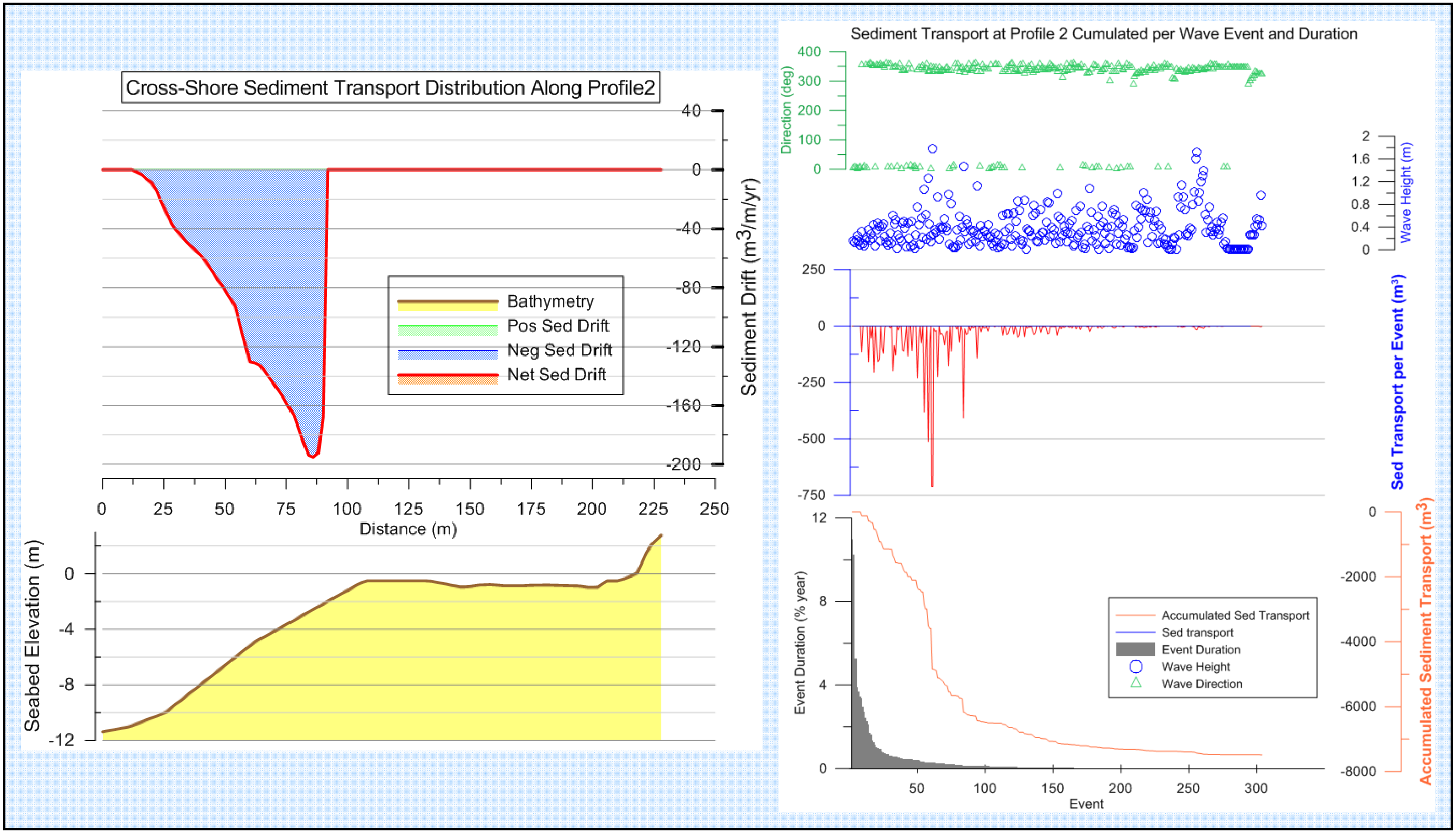


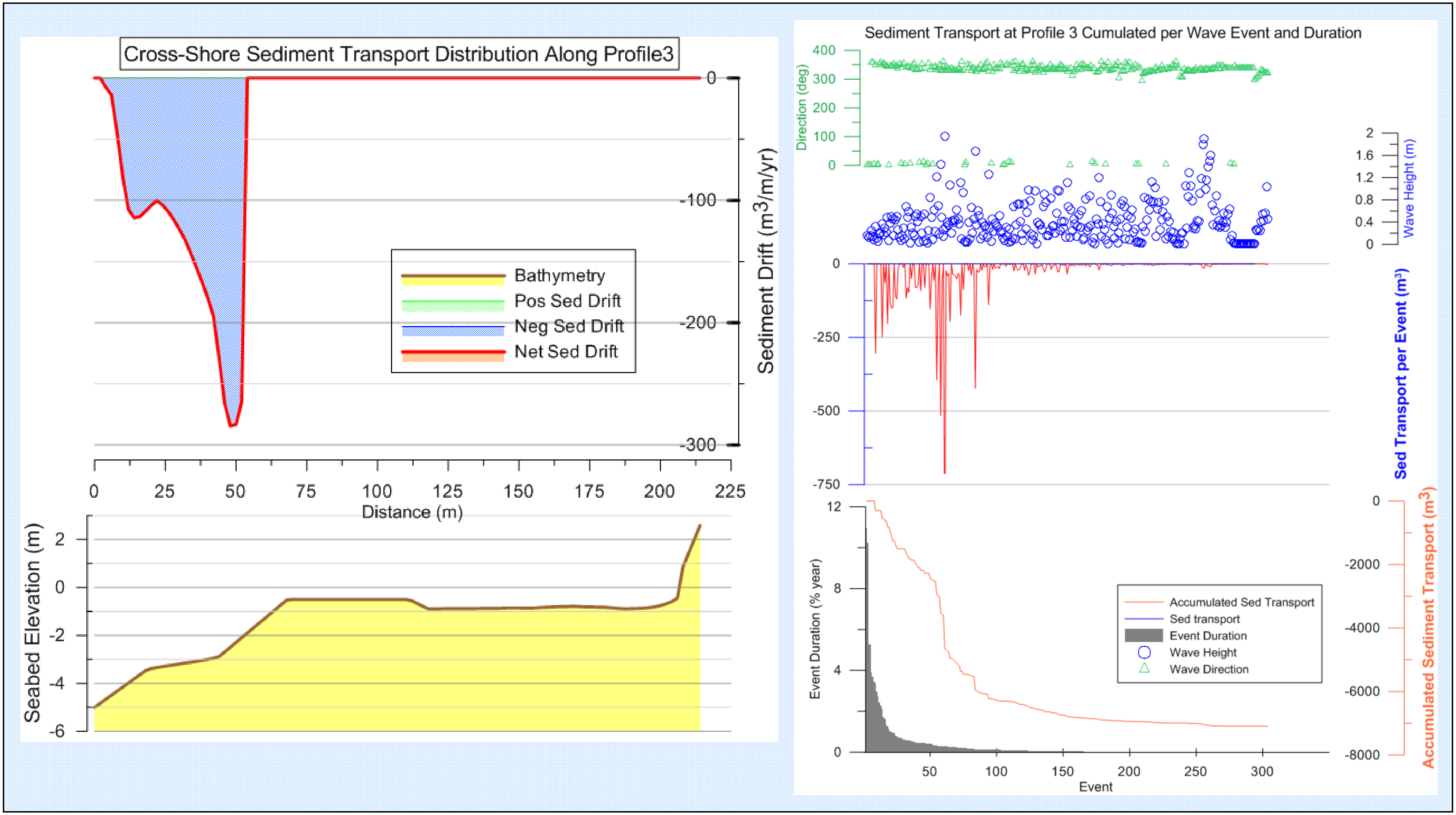

**Figure 7 Maximum wave heights (Left) and storm surge (Right) for the 50 year hurricane event coming from west**

# LITDRIFT Modeling Results

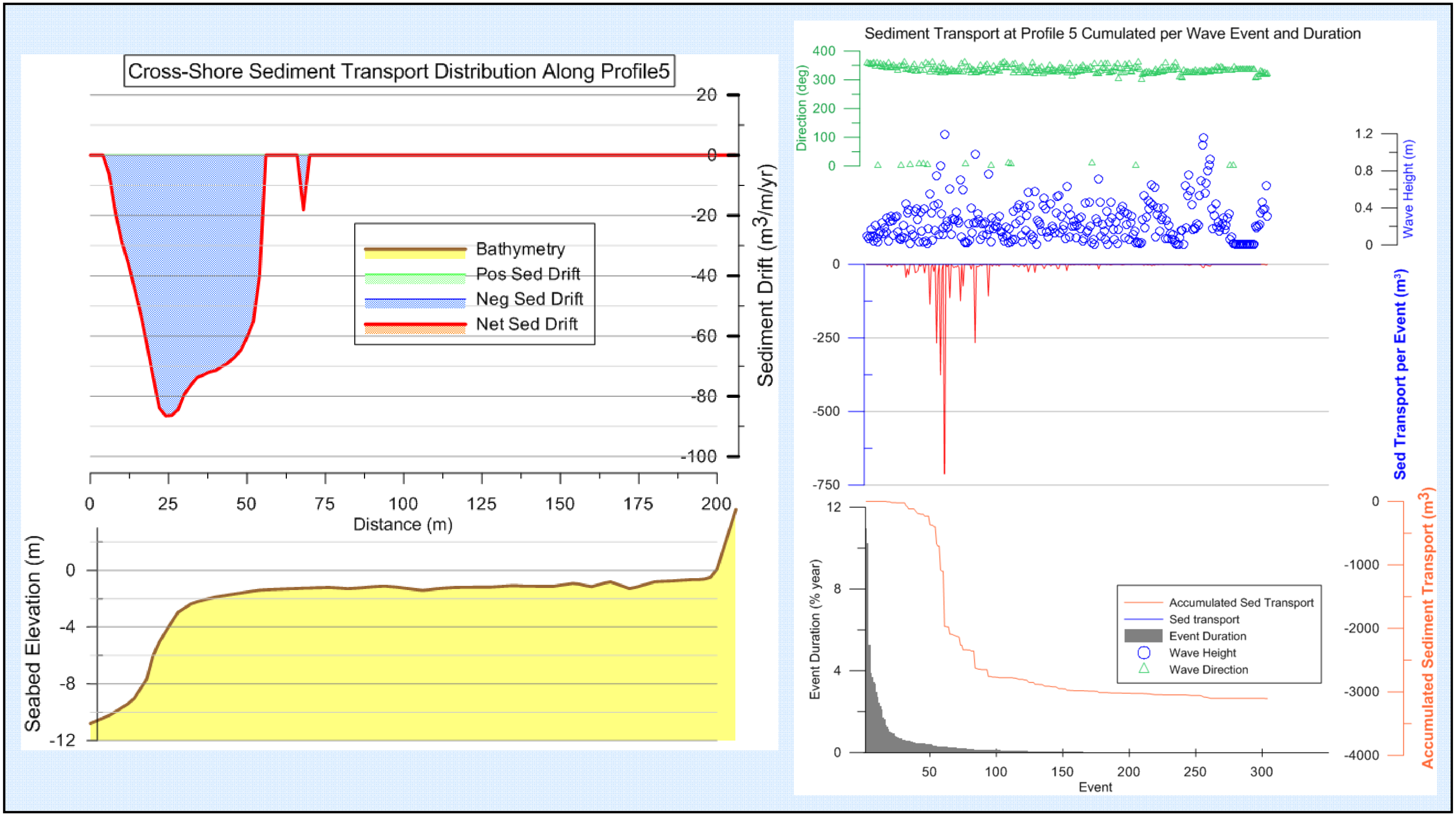




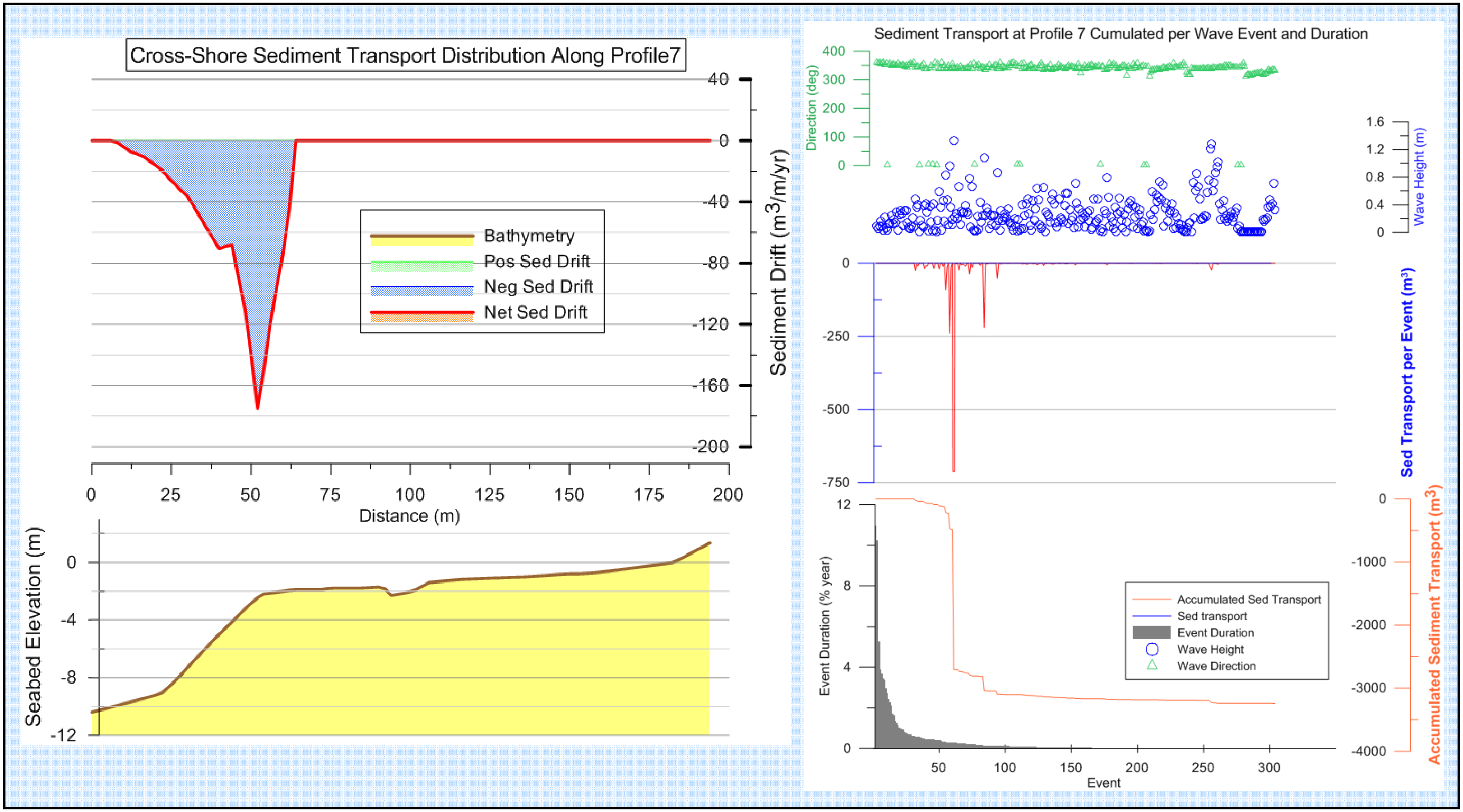
Cross-Shore Sediment Transport Distribution Along Profile2
Bathymetry
Pos Sed Drift
Neg Sed Drift
Net Sed Drift
Sediment Drift (m3/m/yr)
Distance (m)
Seabed Elevation (m)
Sediment Transport at Profile 2 Cumulated per Wave Event and Duration
Direction (deg)
Wave Height (m)
Sed Transport per Event (m3)
Event Duration (% year)
Accumulated Sediment Transport (m3)
Accumulated Sed Transport
Sed transport
Event Duration
Wave Height
Wave Direction
Event



Cross-Shore Sediment Transport Distribution Along Profile3
Bathymetry
Pos Sed Drift
Neg Sed Drift
Net Sed Drift
Sediment Drift (m3/m/yr)
Distance (m)
Seabed Elevation (m)
Sediment Transport at Profile 3 Cumulated per Wave Event and Duration
Direction (deg)
Wave Height (m)
Sed Transport per Event (m3)
Event Duration (% year)
Accumulated Sediment Transport (m3)
Accumulated Sed Transport
Sed transport
Event Duration
Wave Height
Wave Direction
Event

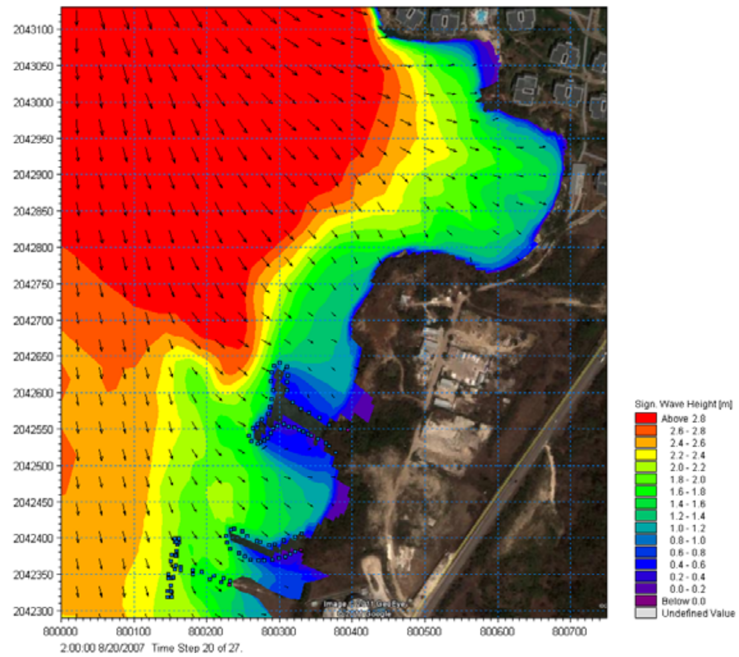


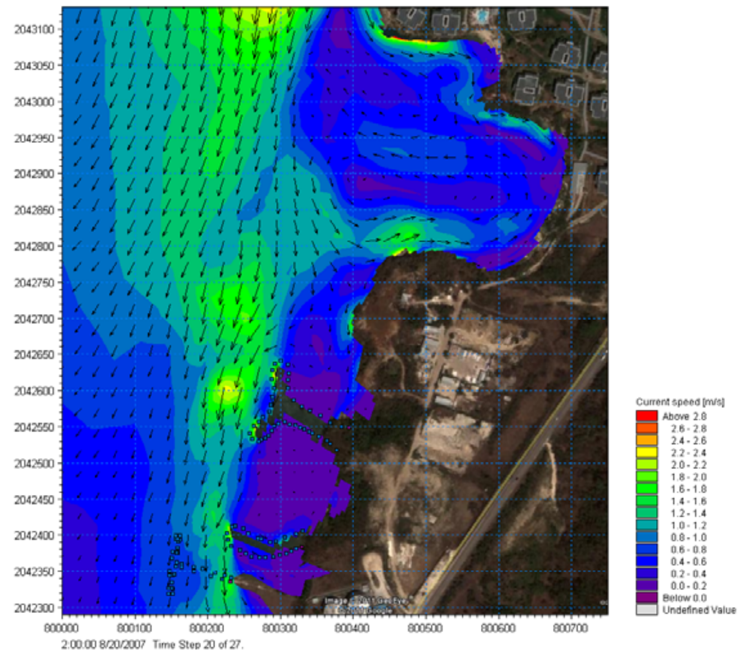
Cross-Shore Sediment Transport Distribution Along Profile5
Bathymetry
Pos Sed Drift
Neg Sed Drift
Net Sed Drift
Sediment Drift (m3/m/yr)
Distance (m)
Seabed Elevation (m)
Sediment Transport at Profile 5 Cumulated per Wave Event and Duration
Direction (deg)
Wave Height (m)
Sed Transport per Event (m3)
Event Duration (% year)
Accumulated Sediment Transport (m3)
Accumulated Sed Transport
Sed transport
Event Duration
Wave Height
Wave Direction
Event

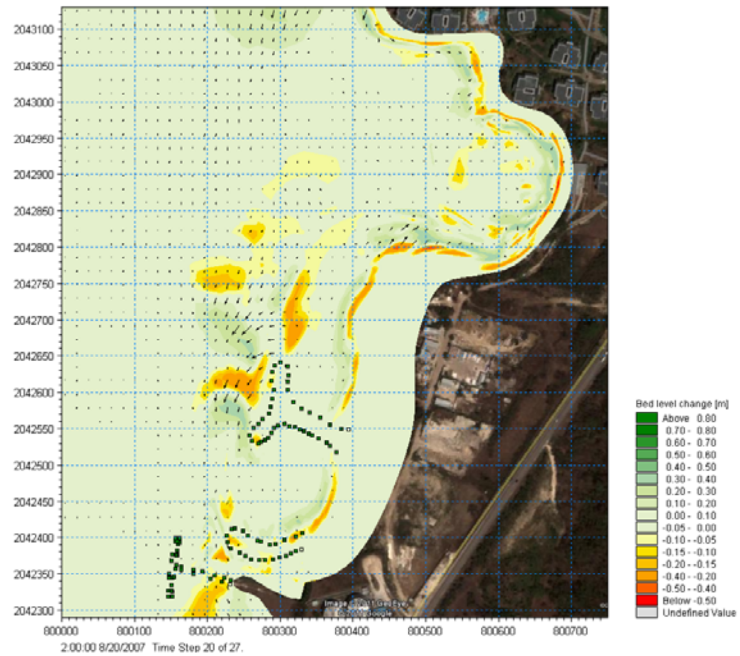


Cross-Shore Sediment Transport Distribution Along Profile7
Bathymetry
Pos Sed Drift
Neg Sed Drift
Net Sed Drift
Sediment Drift (m3/m/yr)
40
0
-40
-80
-120
-160
-200
0
25
50
75
100
125
150
175
200
Distance (m)
Seabed Elevation (m)
0
-4
-8
-12
Sediment Transport at Profile 7 Cumulated per Wave Event and Duration
Direction (deg)
400
300
200
100
0
Wave Height (m)
1.6
1.2
0.8
0.4
0
Sed Transport per Event (m3)
0
-250
-500
-750
Event Duration (% year)
12
8
4
0
Accumulated Sediment Transport (m3)
0
-1000
-2000
-3000
-4000
50
100
150
200
250
300
Event
Accumulated Sed Transport
Sed transport
Event Duration
Wave Height
Wave Direction

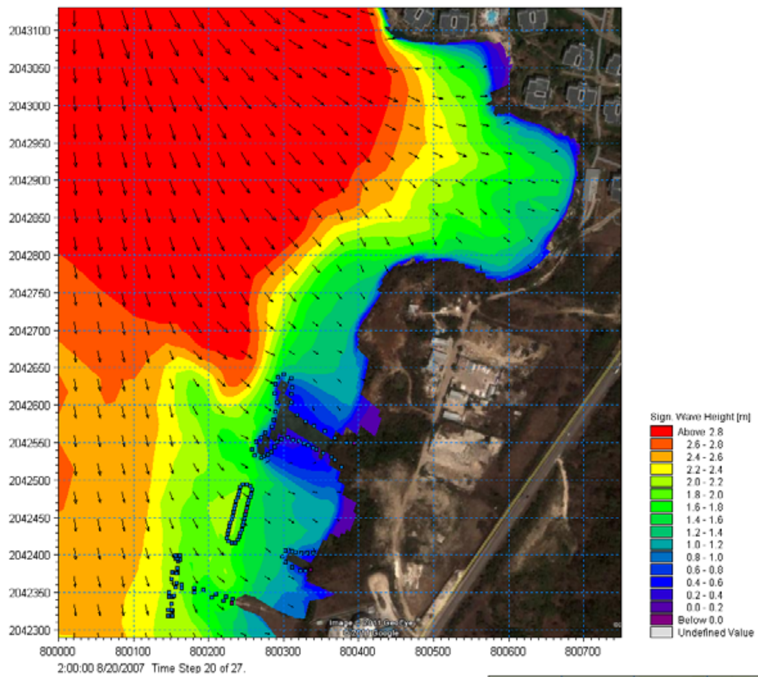
# MIKE 21 Simulation Results: Proposed Option 1 at peak of Hurricane Dean

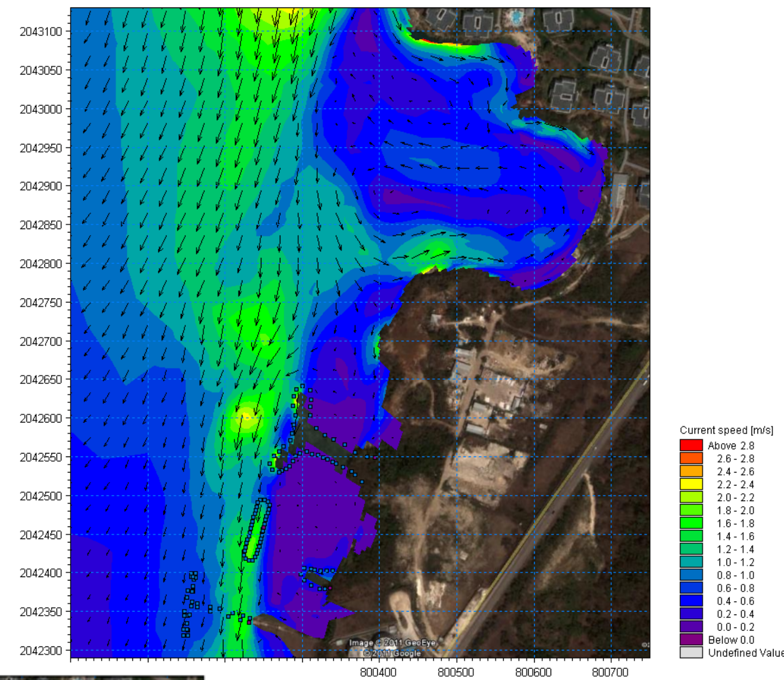


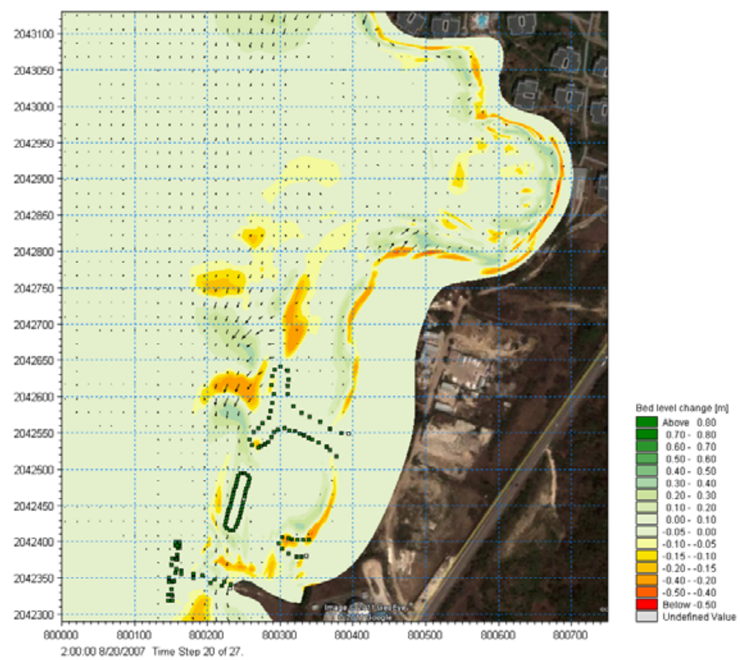



# MIKE 21 Simulation Results: Proposed Option 2 at peak of Hurricane Dean



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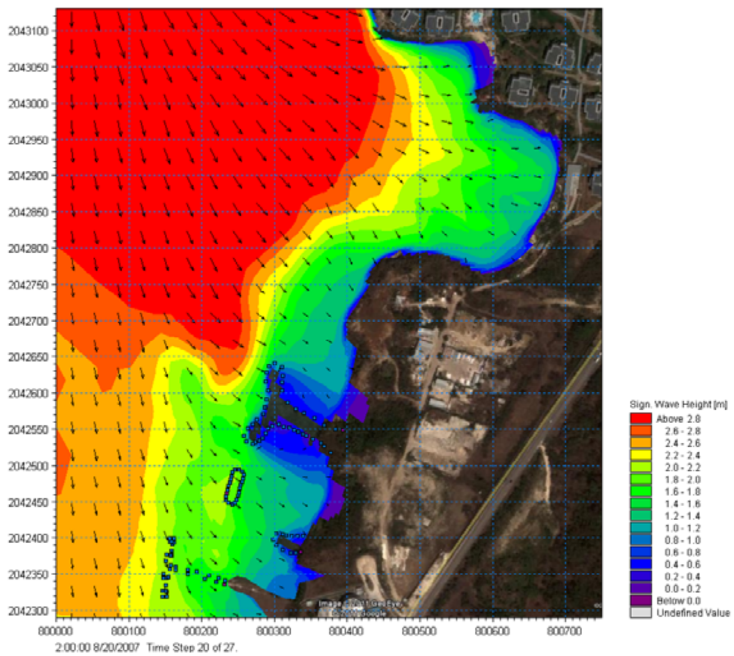


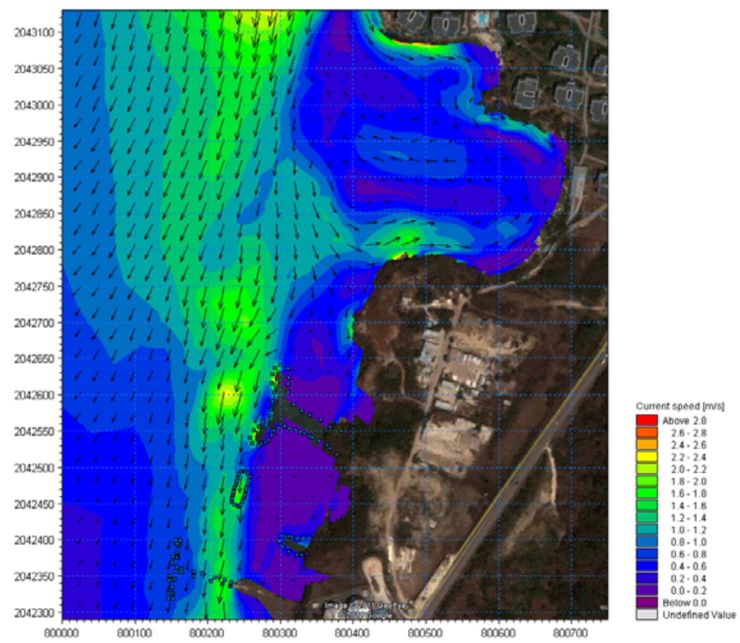


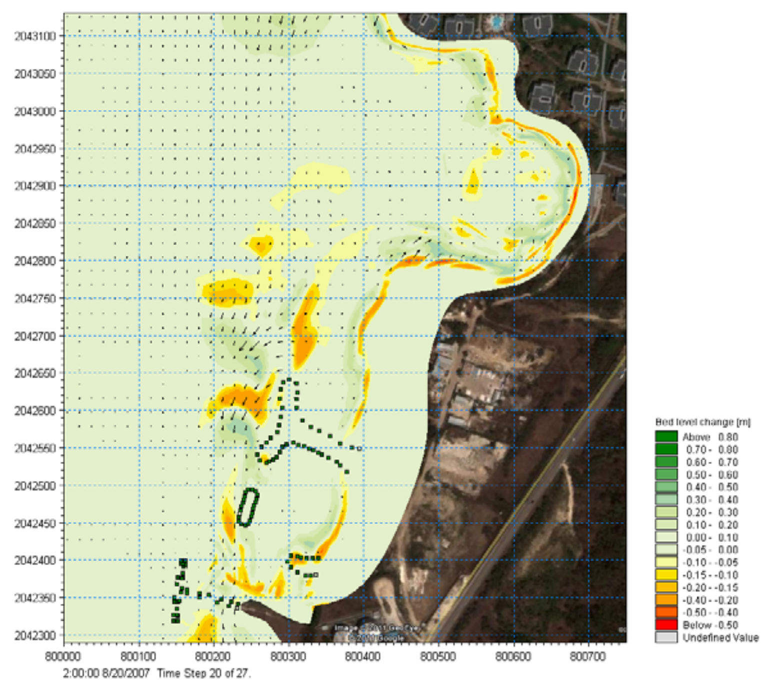

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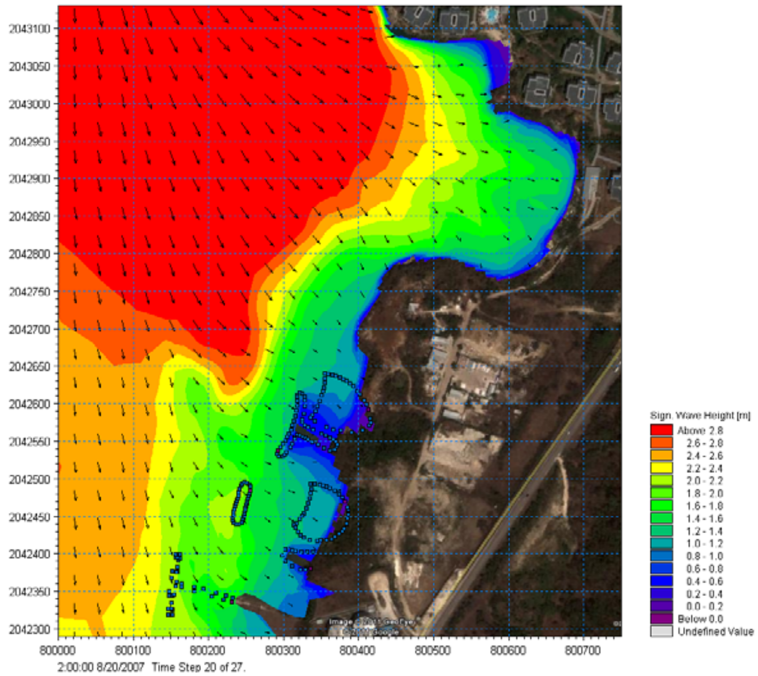


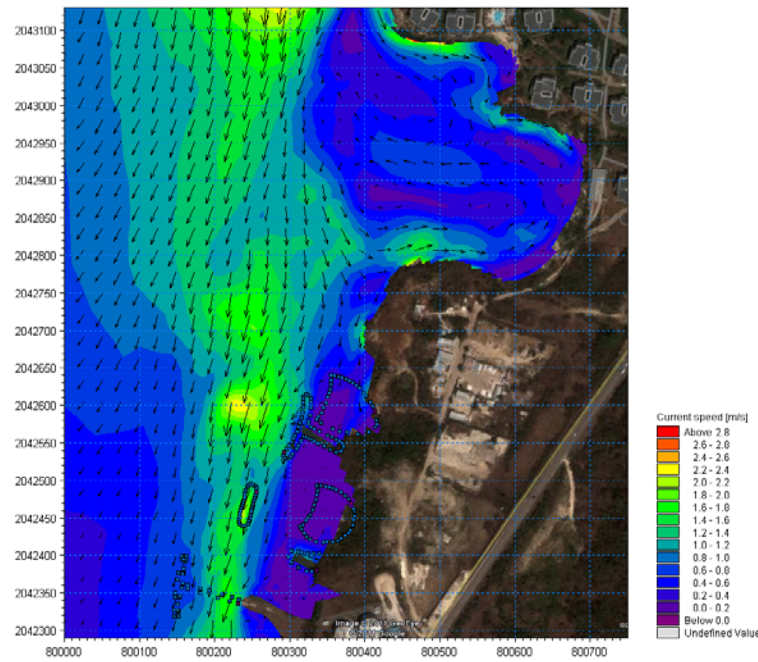


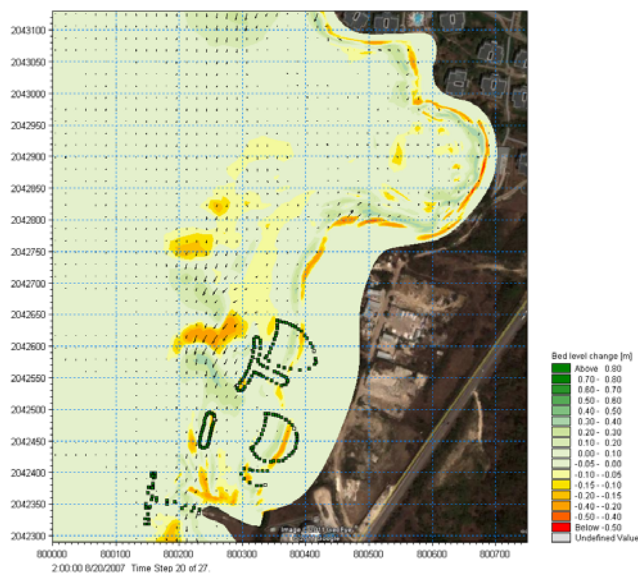

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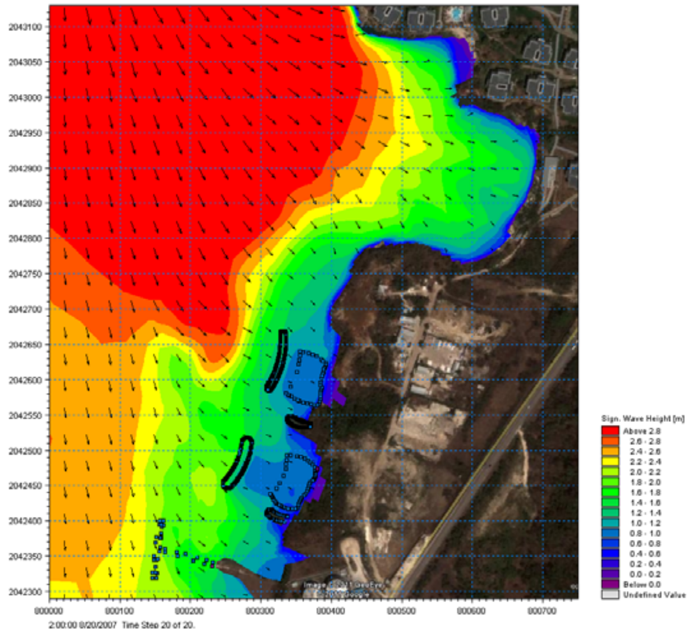


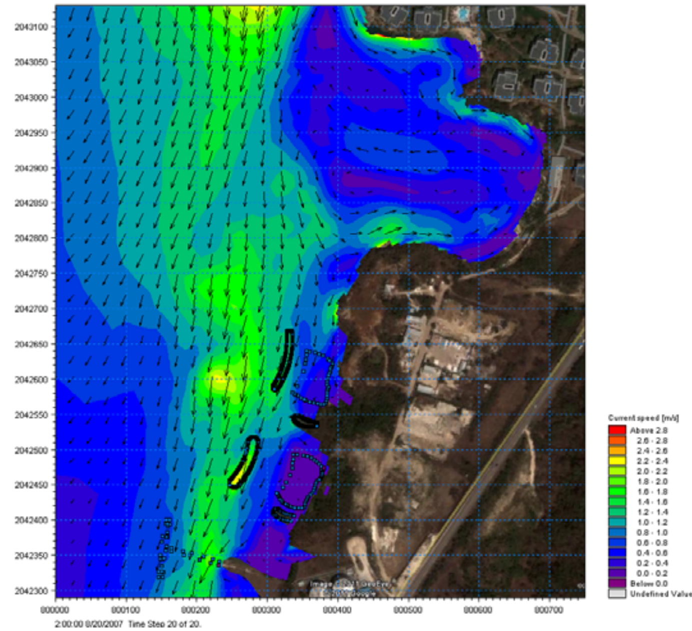


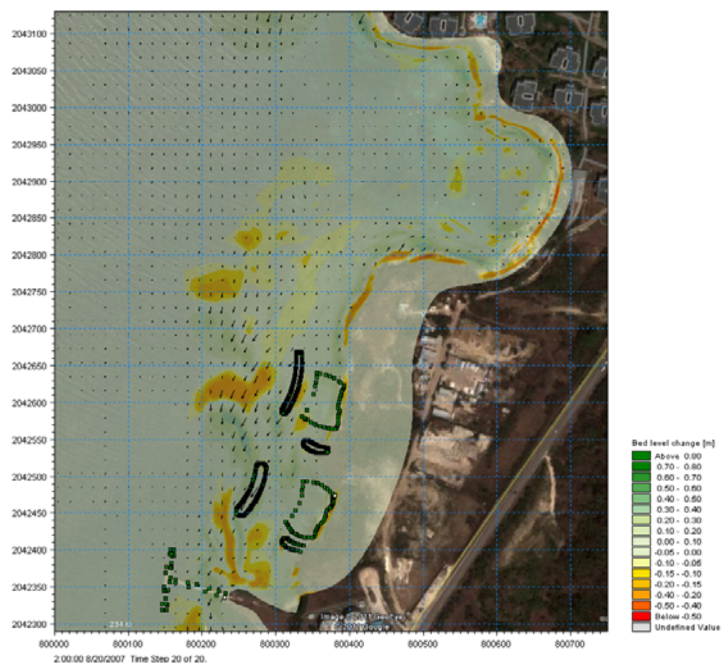

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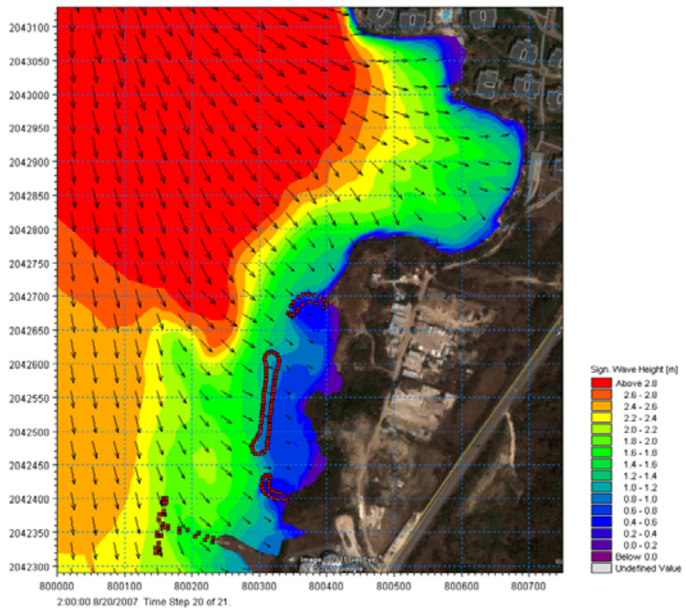
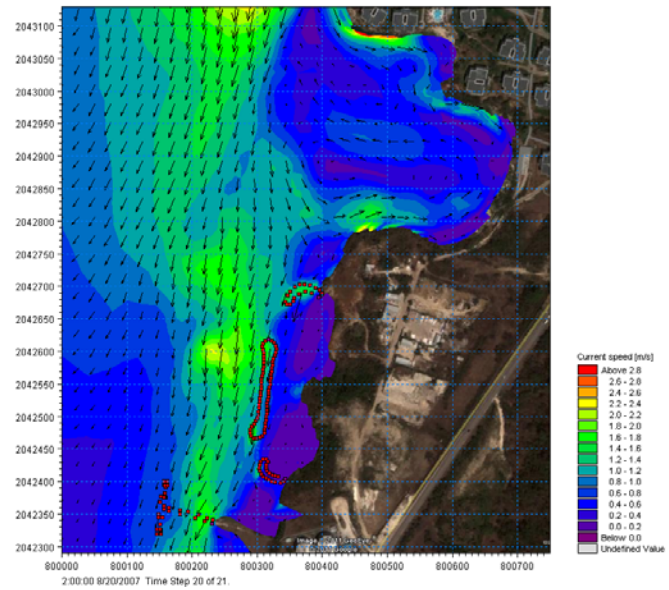
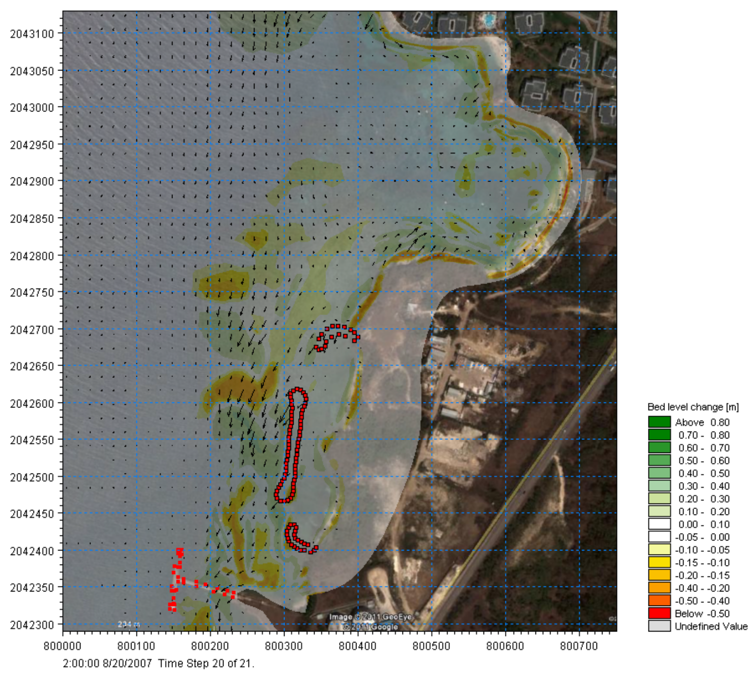
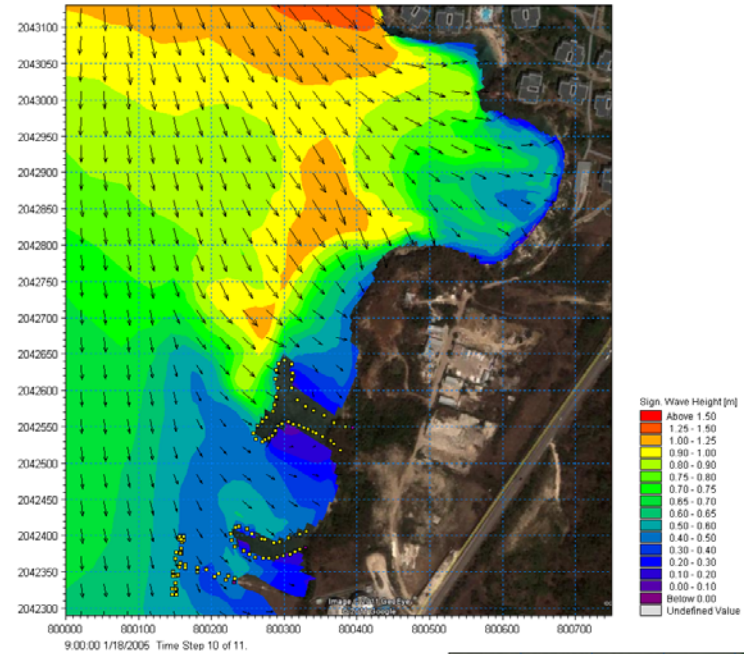
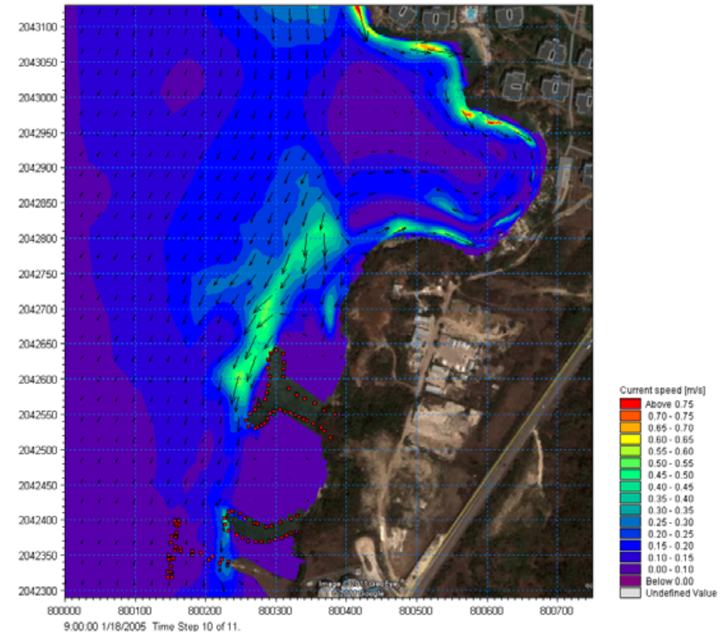


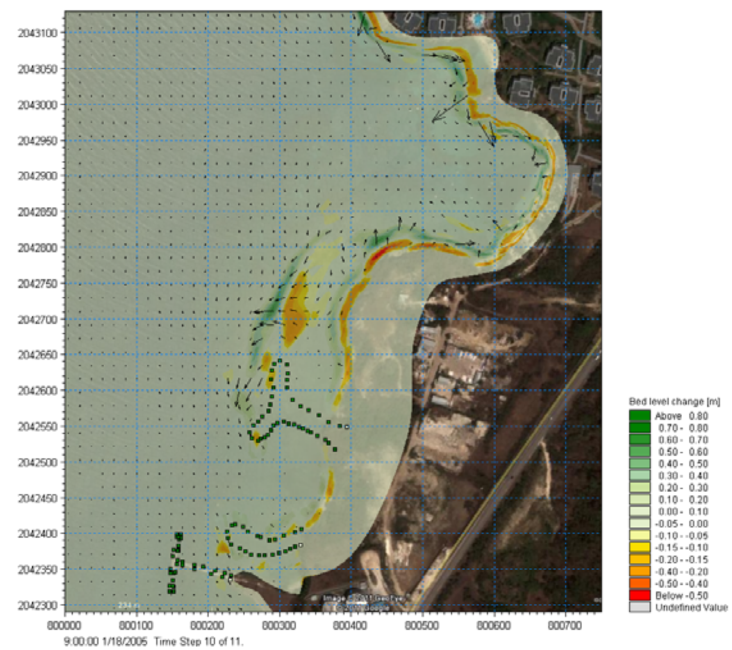



## MIKE 21 Simulation Results: Proposed Option 1 at peak of the January 2005 Swell



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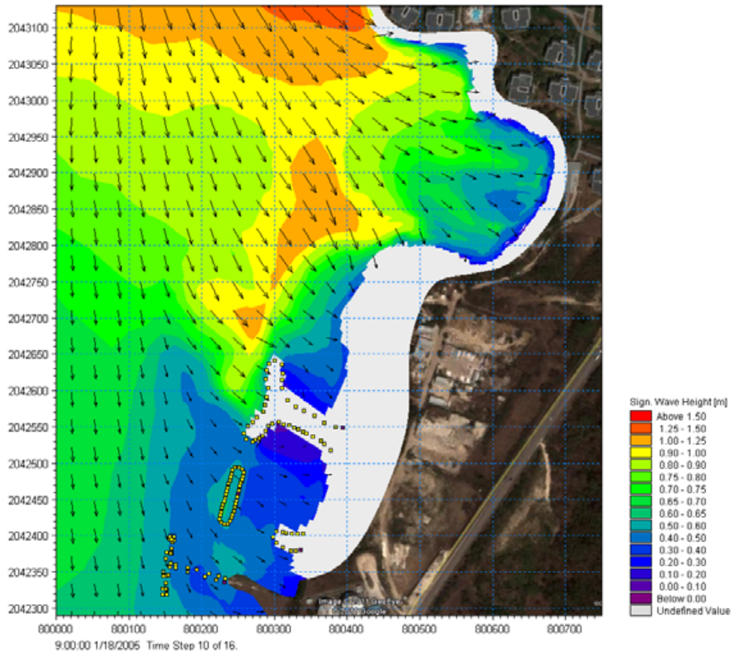


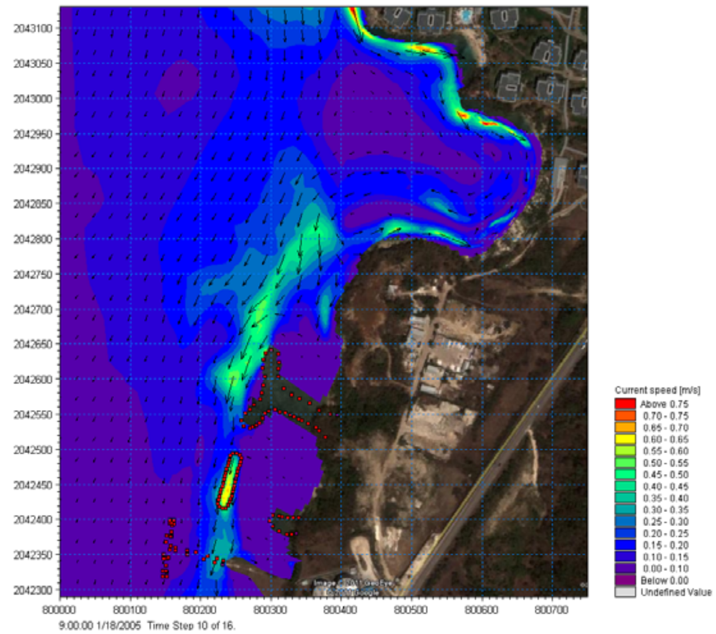


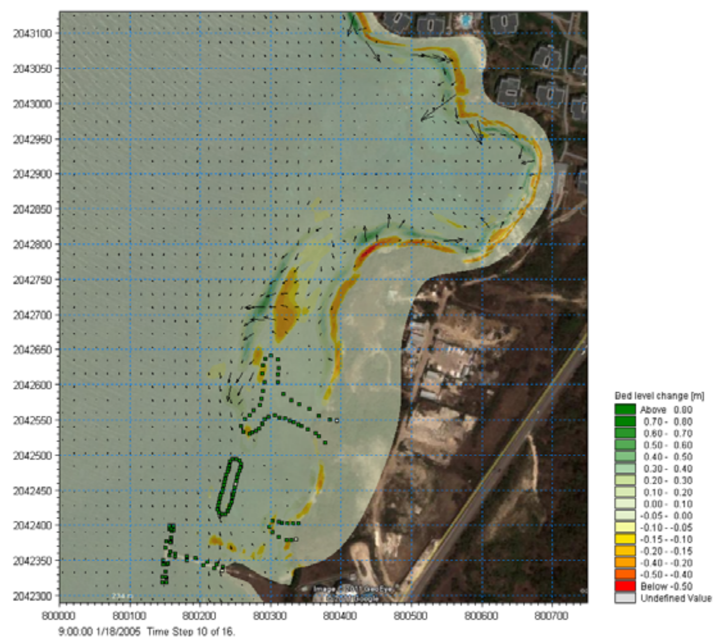

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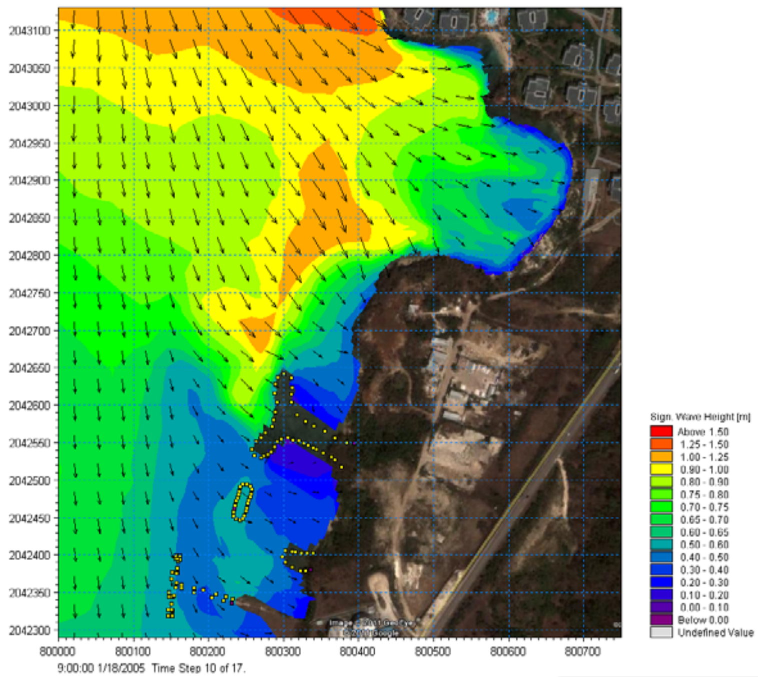


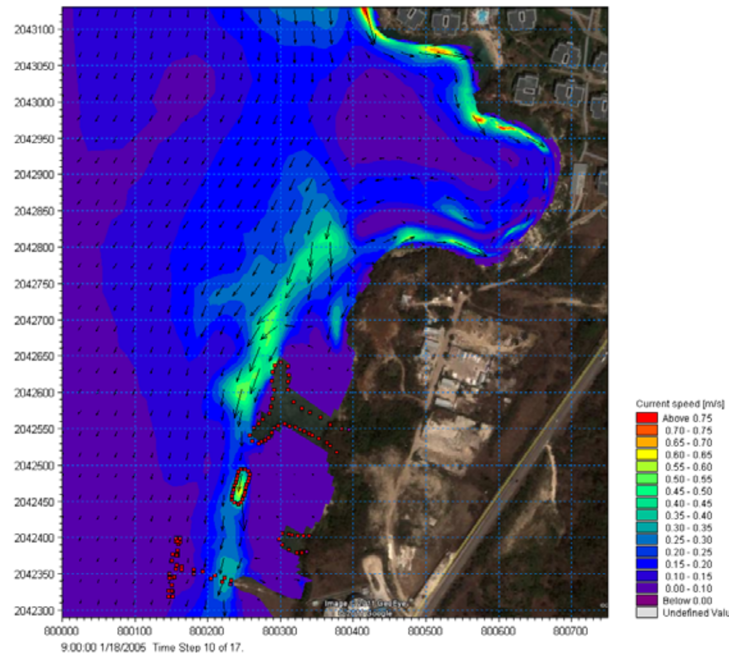


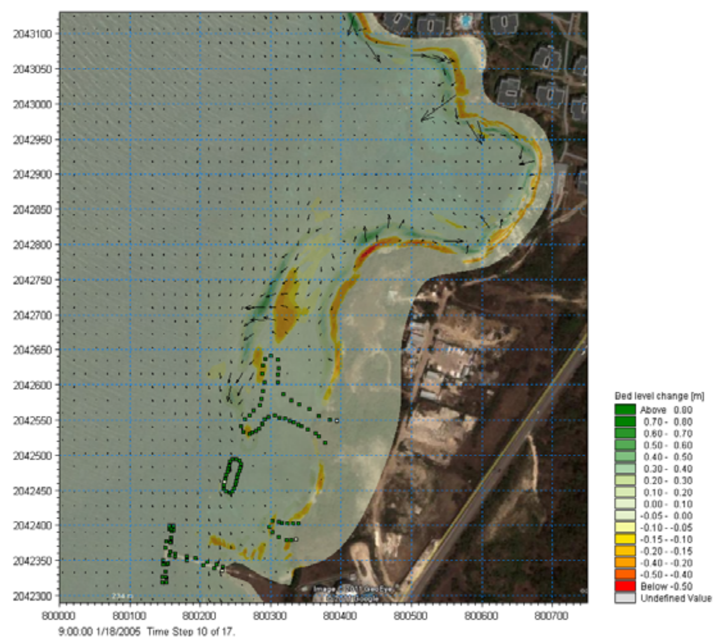

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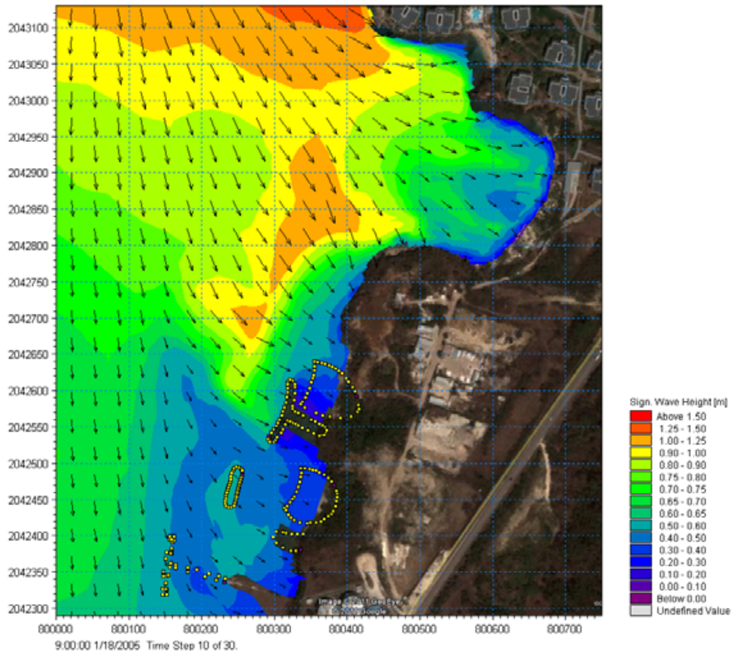


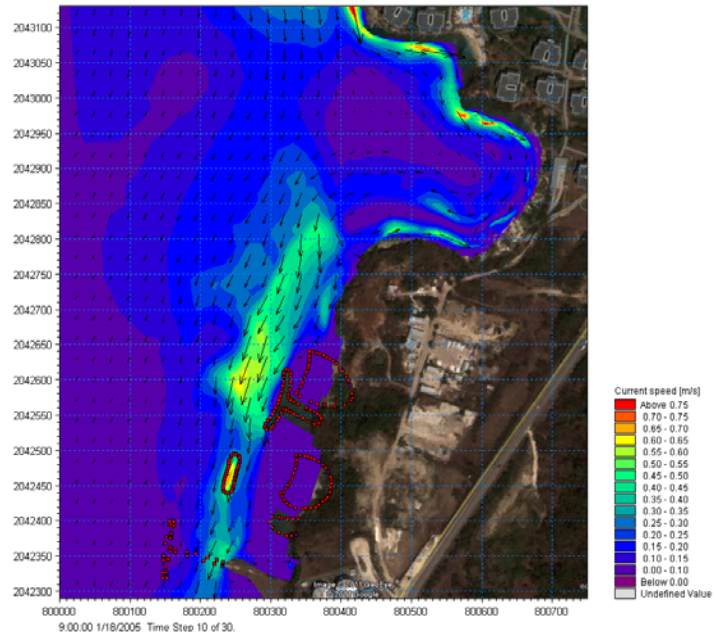


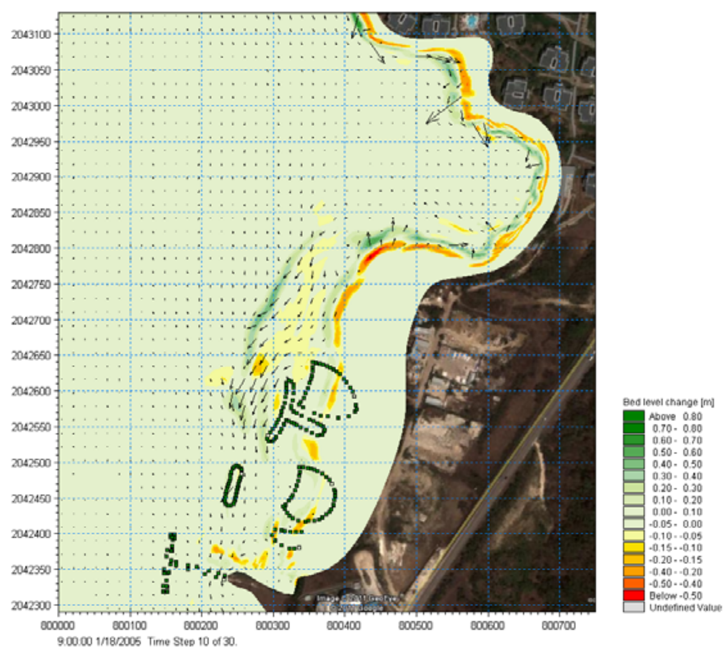

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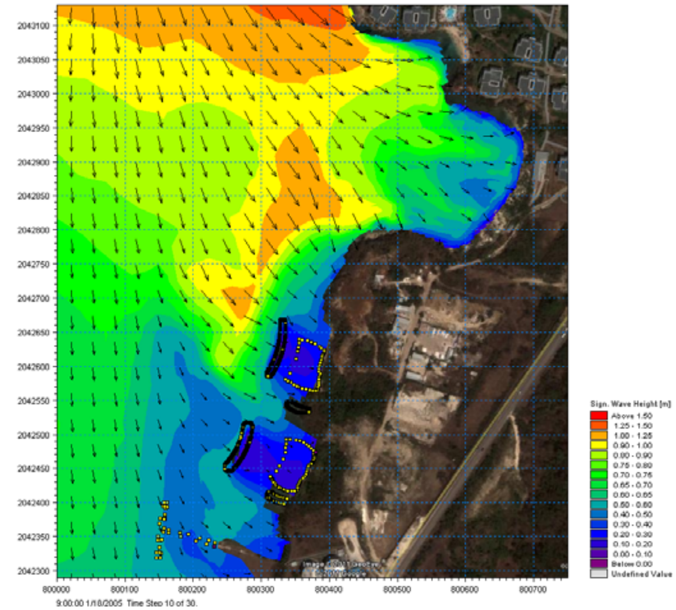


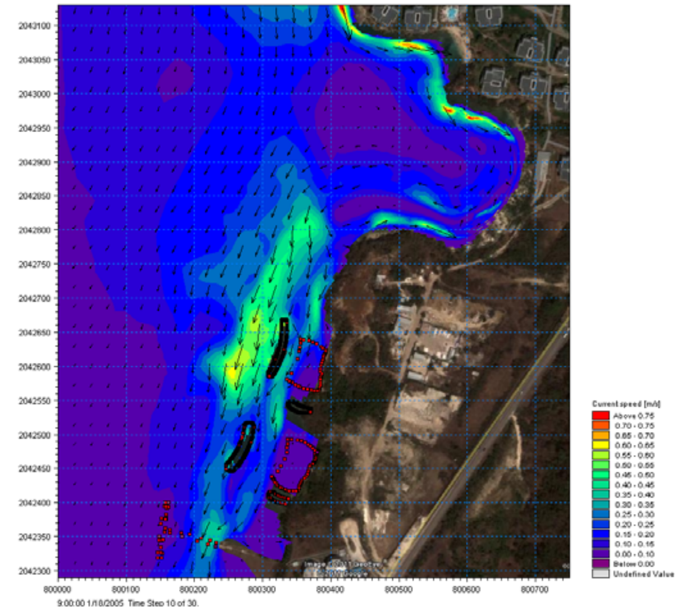


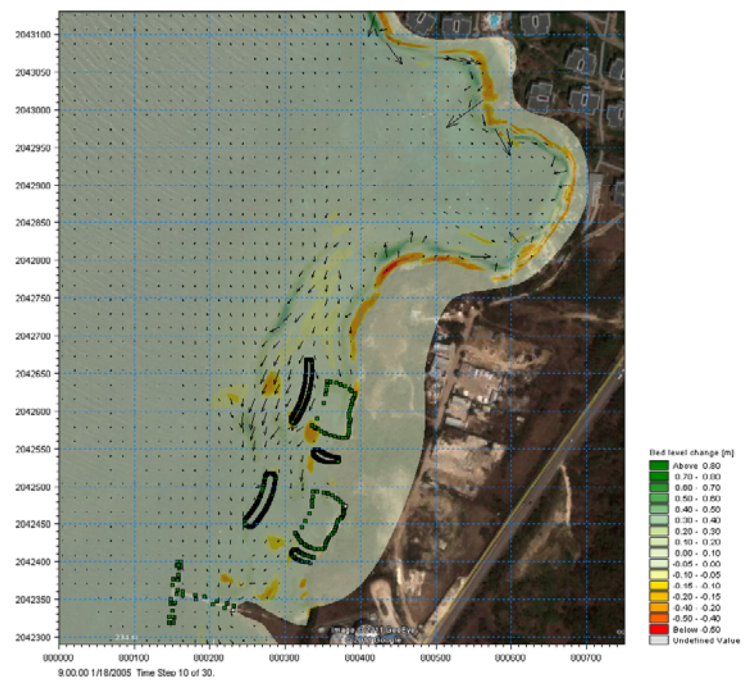

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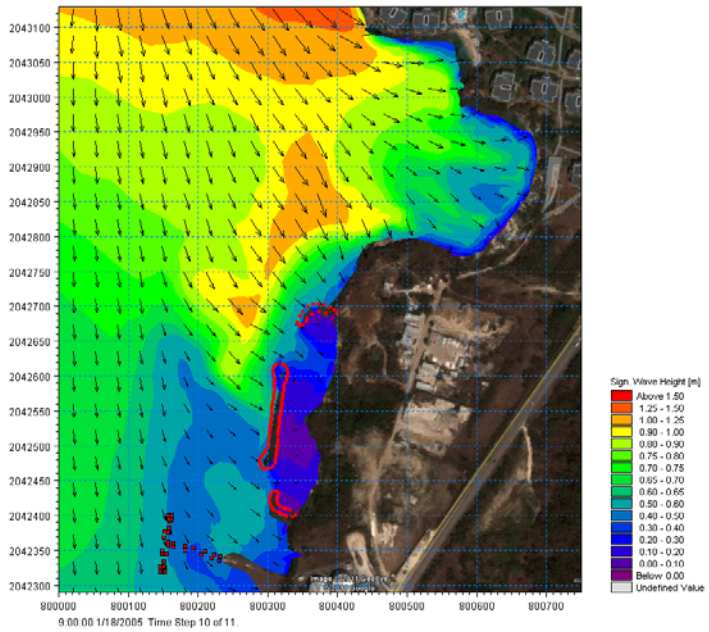
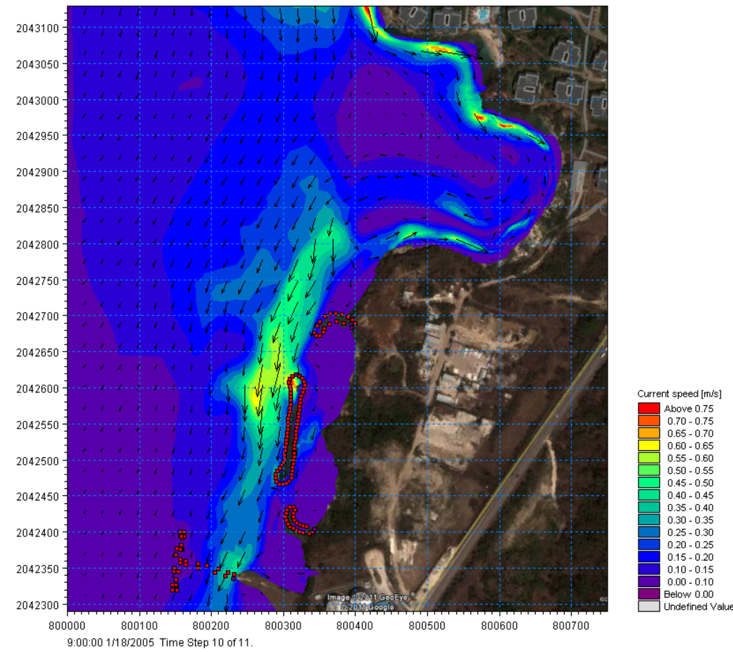
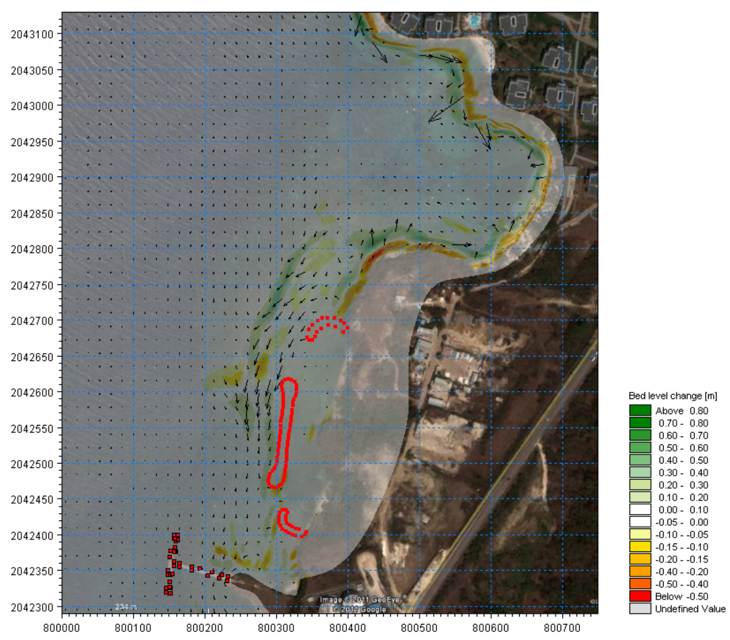
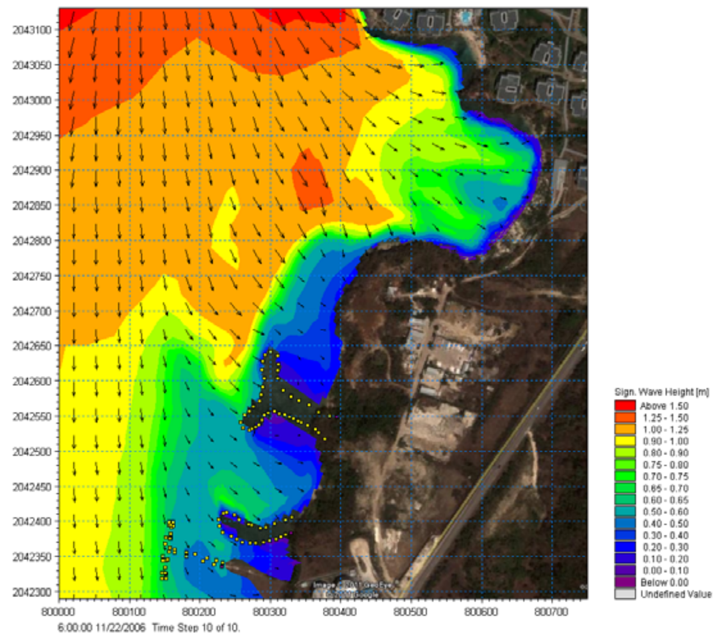
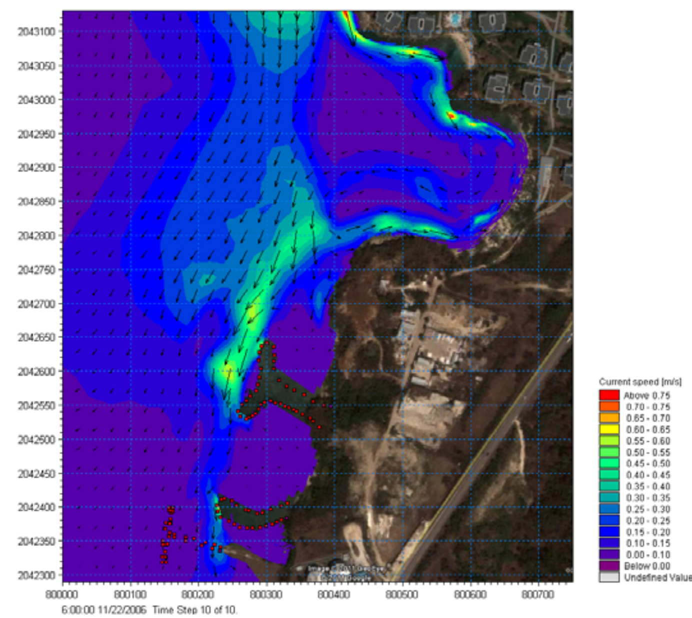


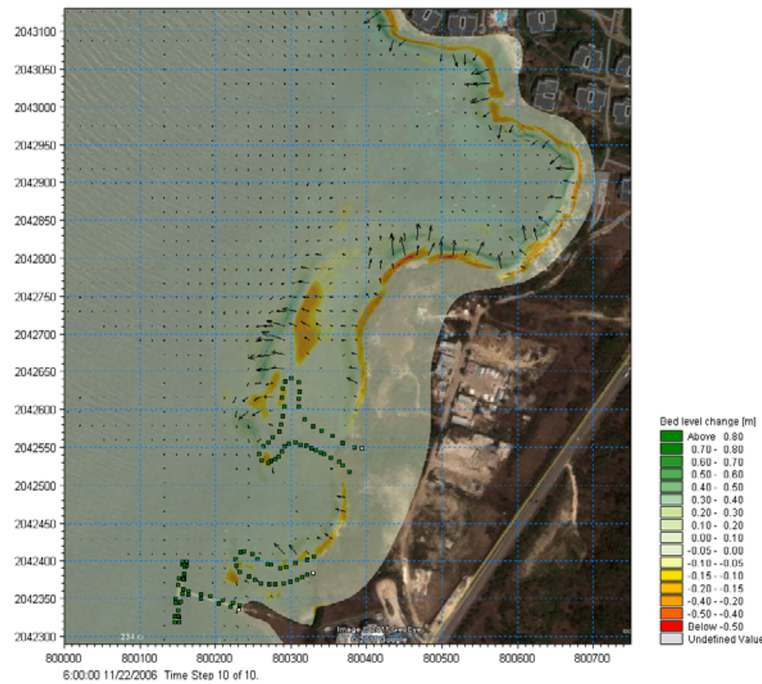



# MIKE 21 Simulation Results: Proposed Option 1 at peak of November 2006 Storm



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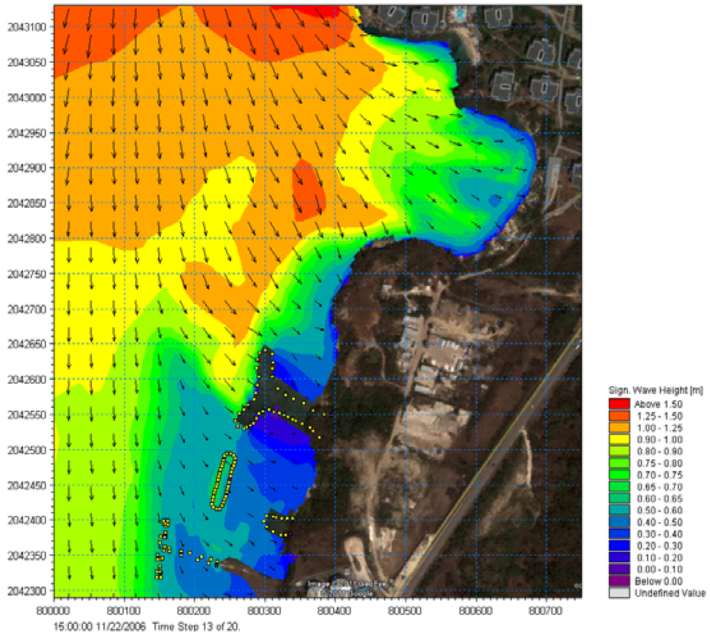


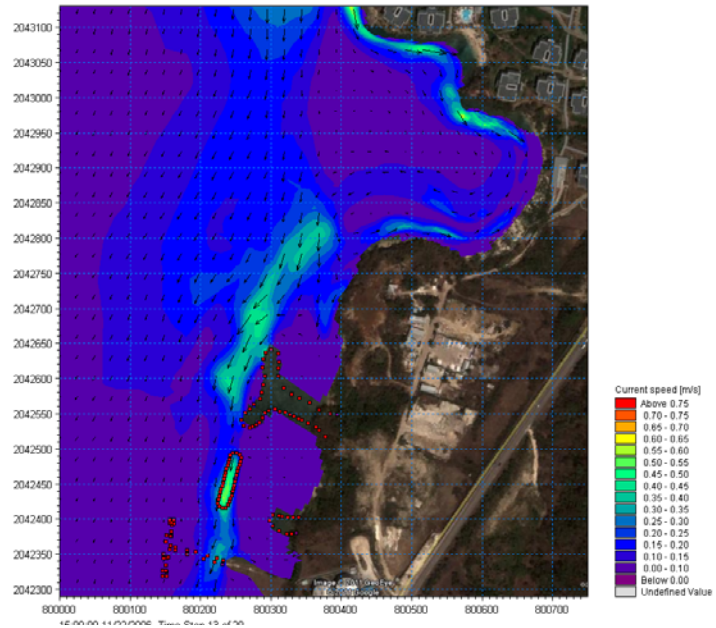


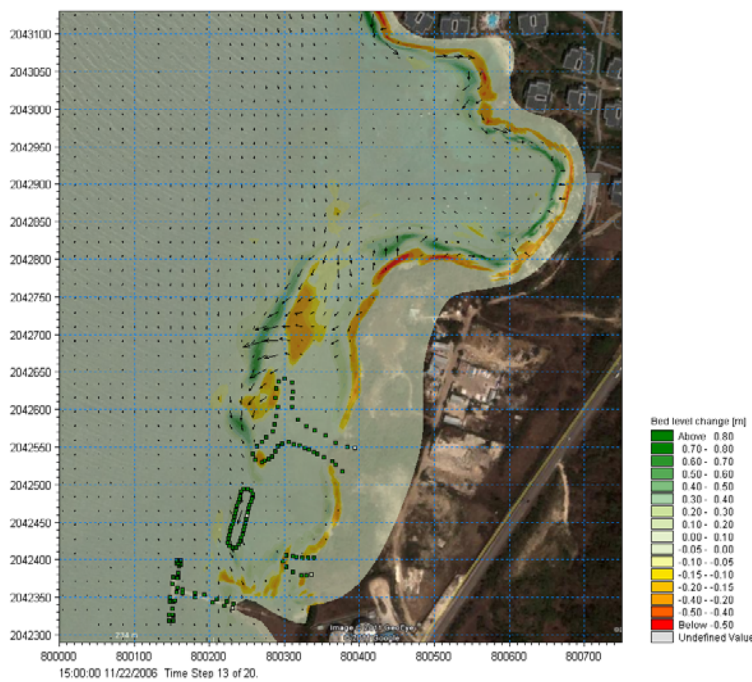

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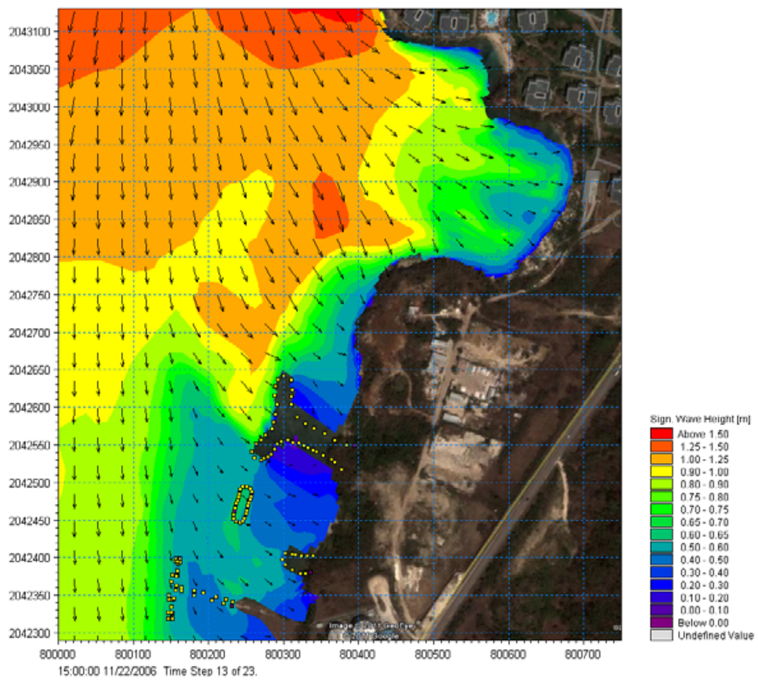


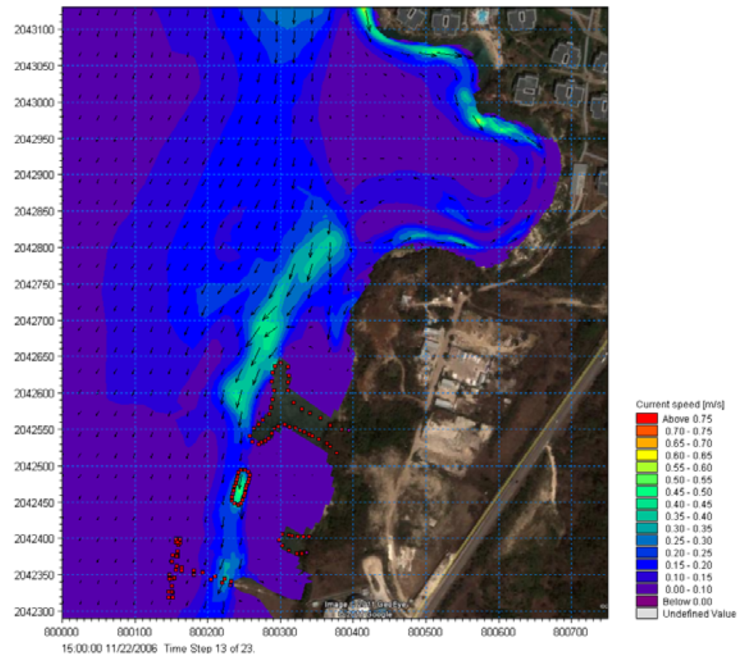


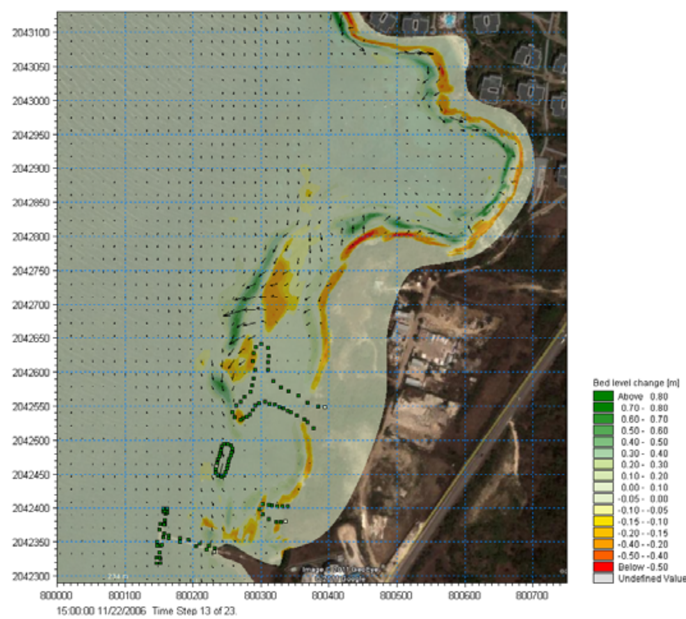

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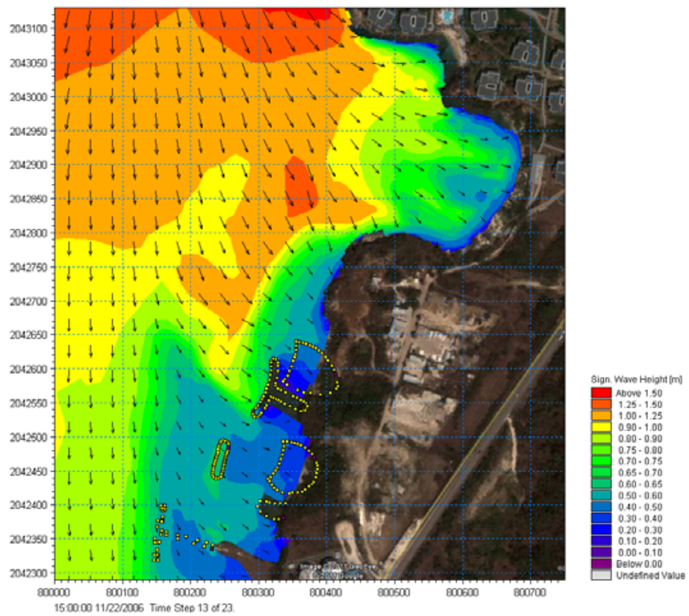


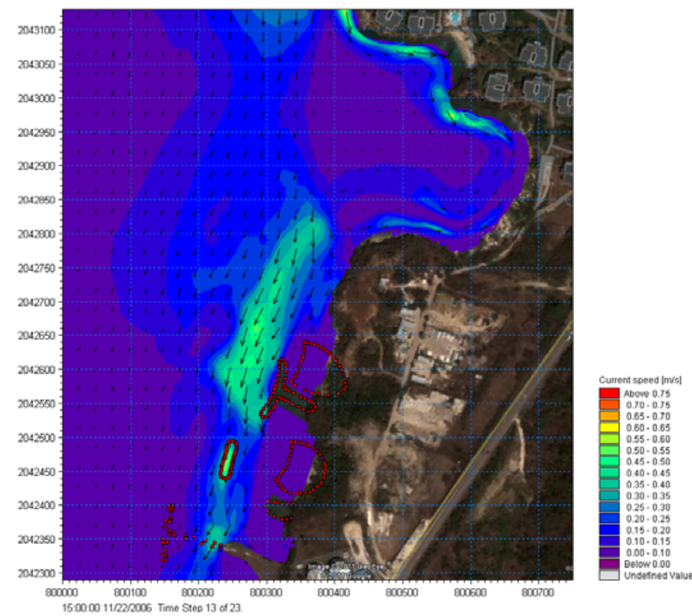


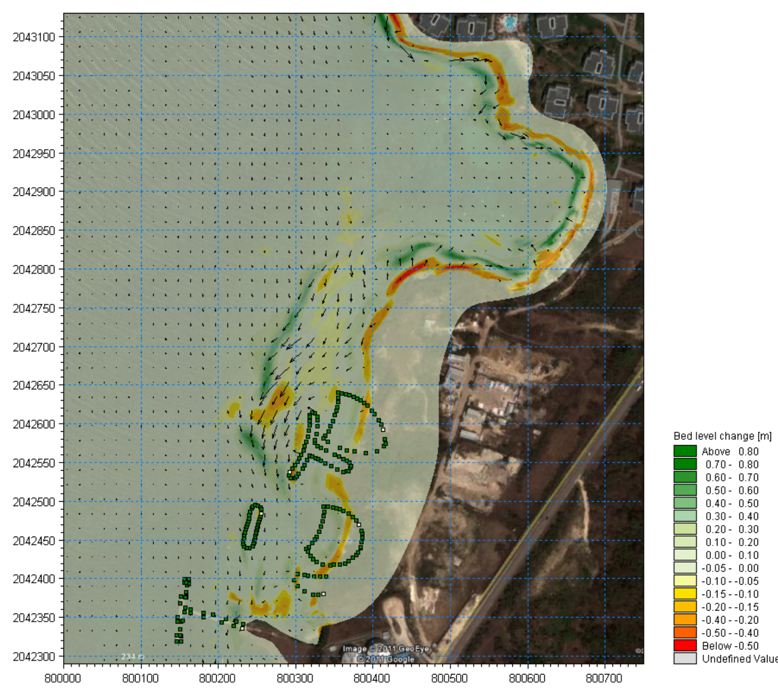

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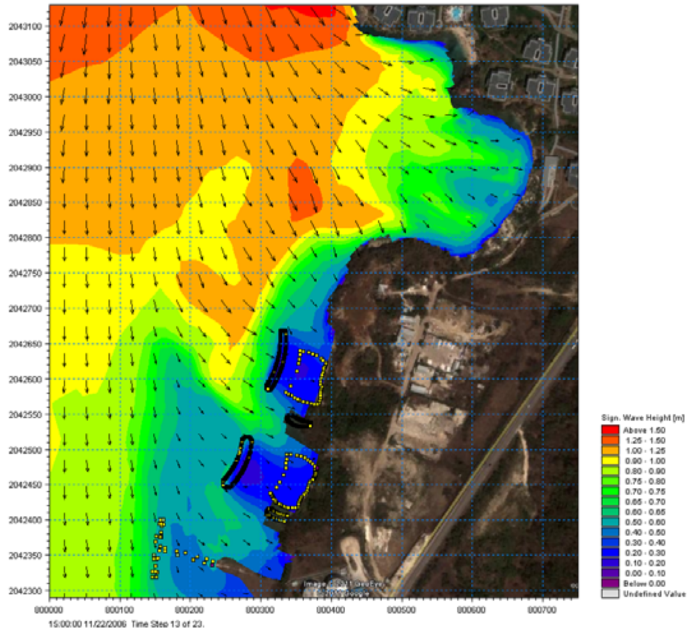
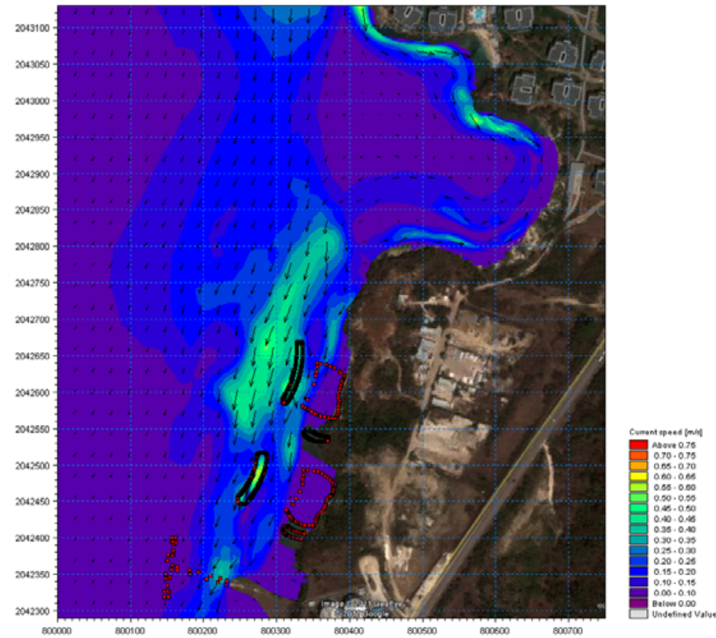
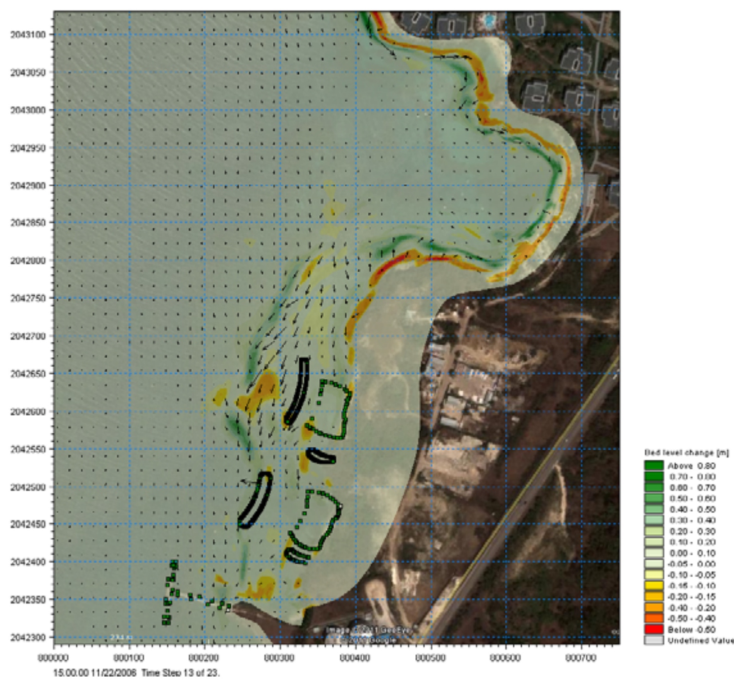
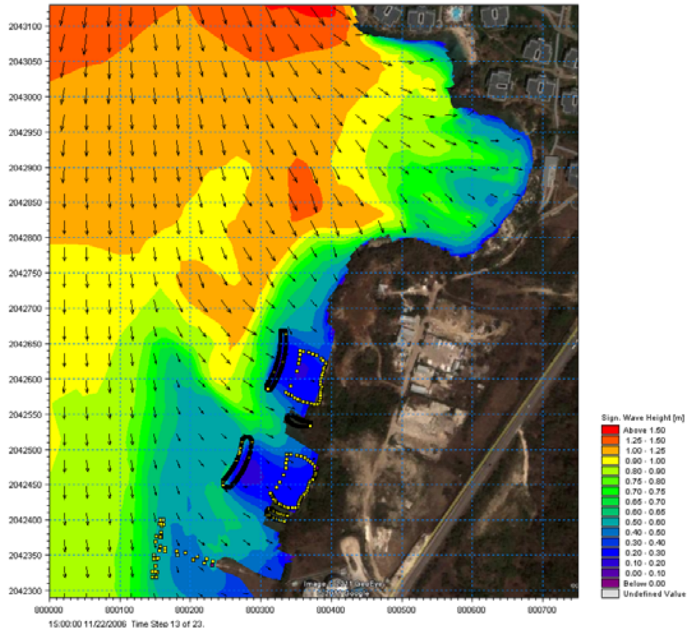
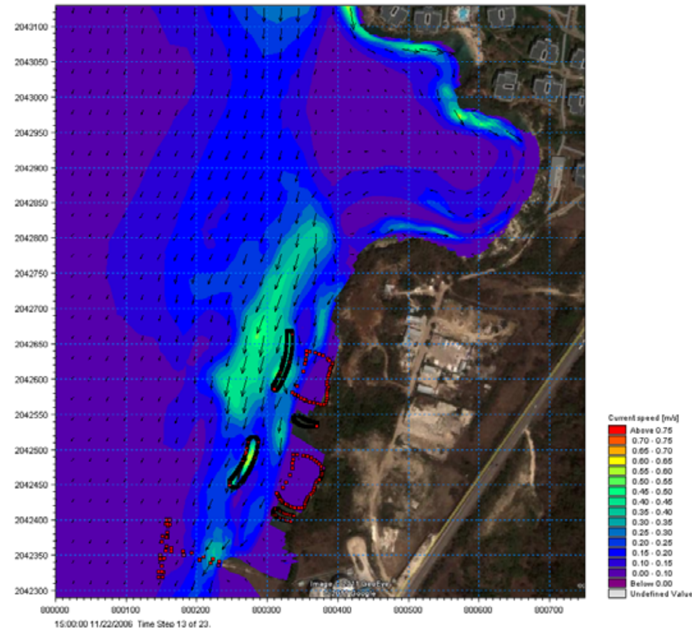


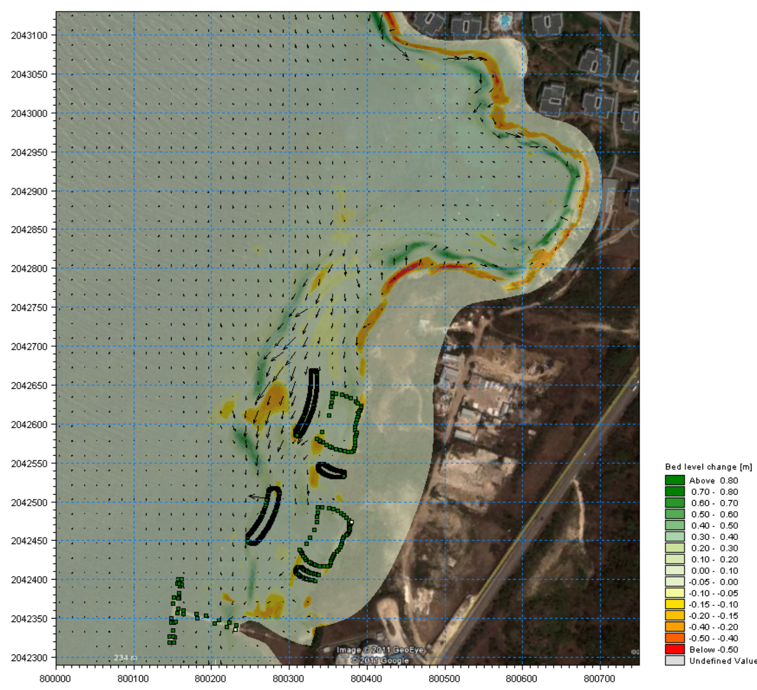



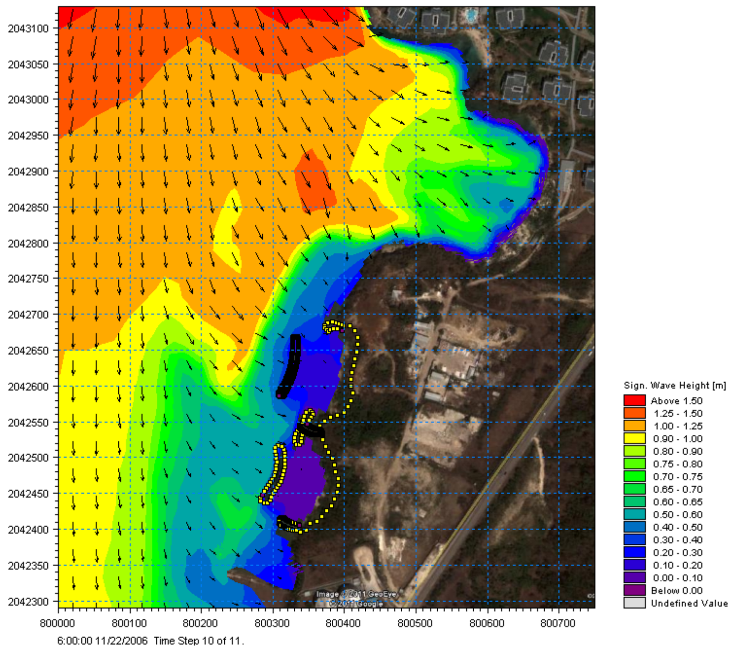
# MIKE 21 Simulation Results: Proposed Option 4 at peak of November 2006 Storm

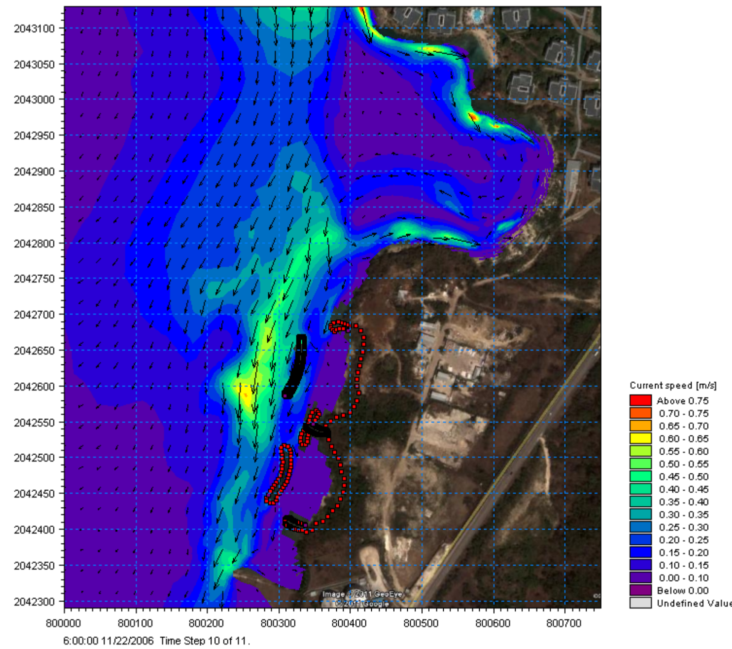


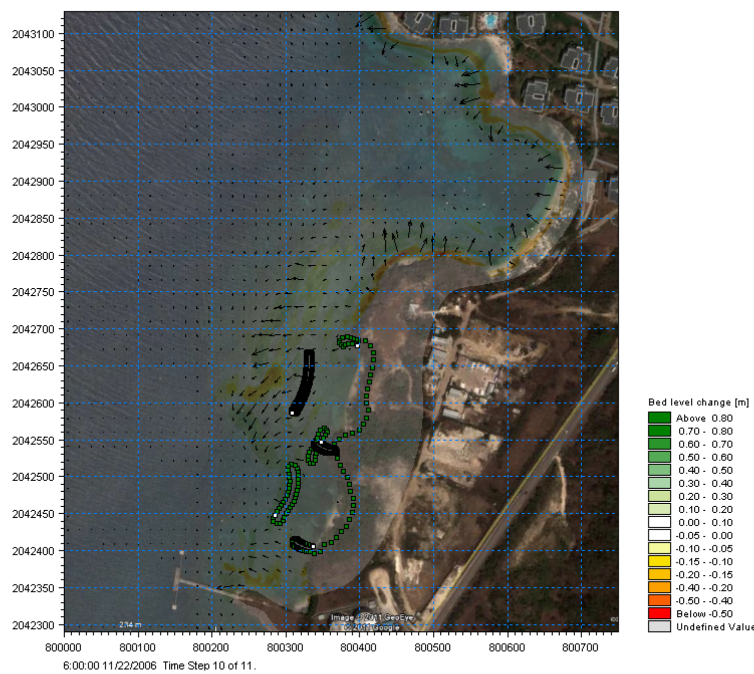



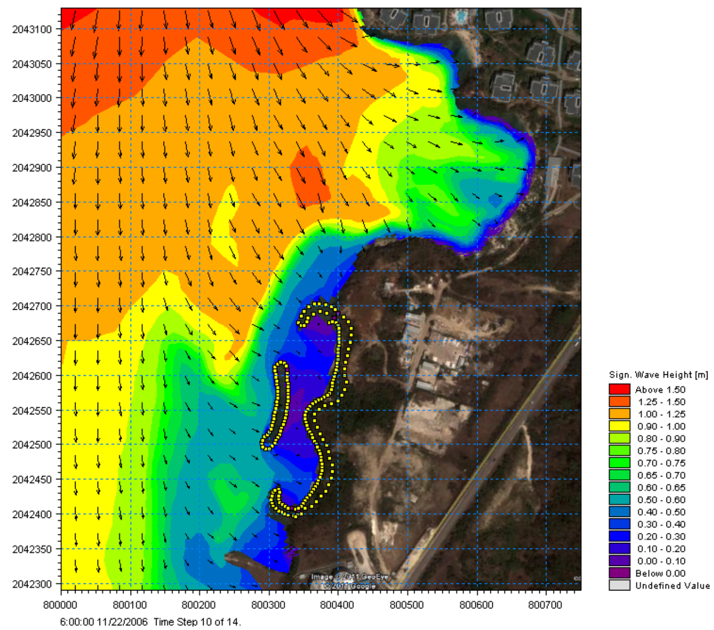
# MIKE 21 Simulation Results: Proposed Option 5 at peak of November 2006 Storm

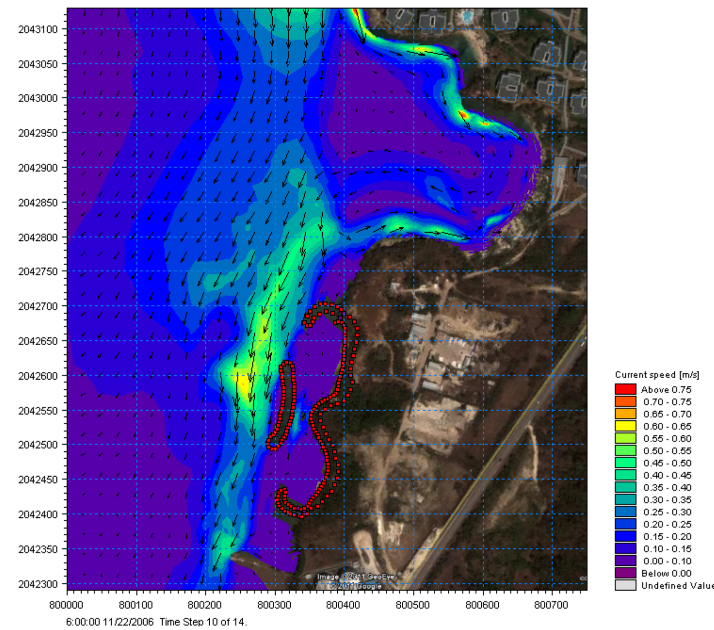


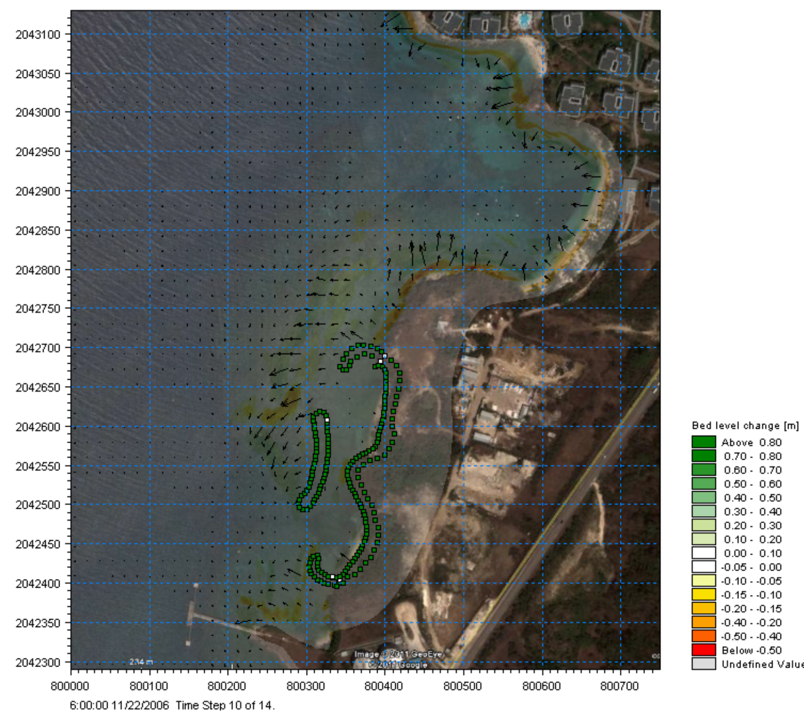



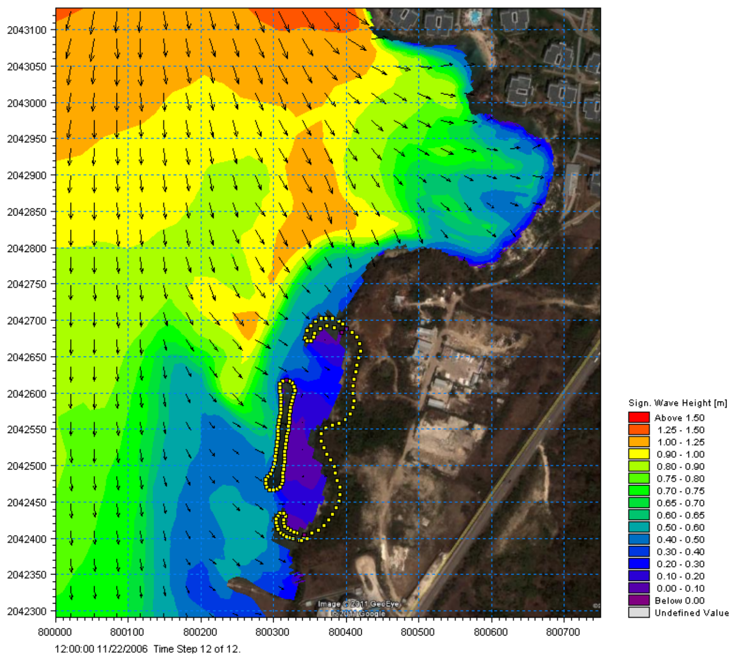
# MIKE 21 Simulation Results: Proposed Option 6 at peak of November 2006 Storm

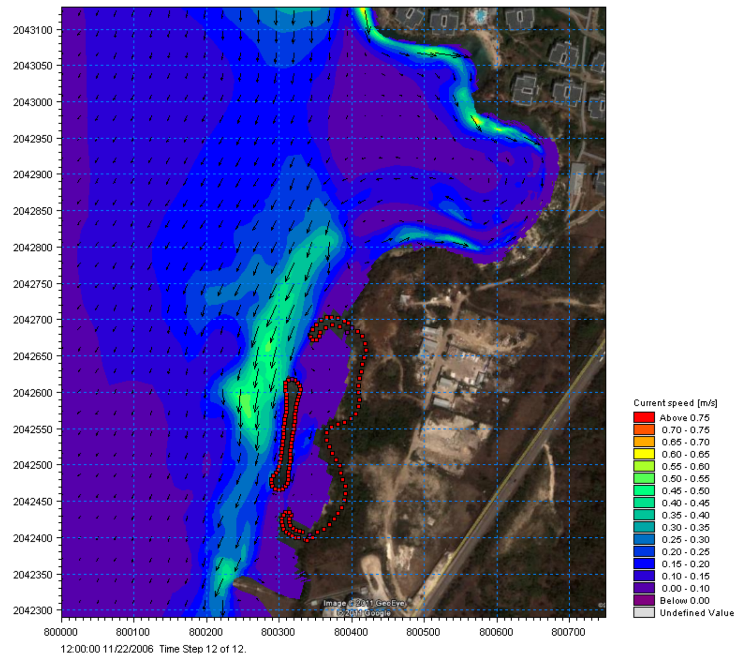


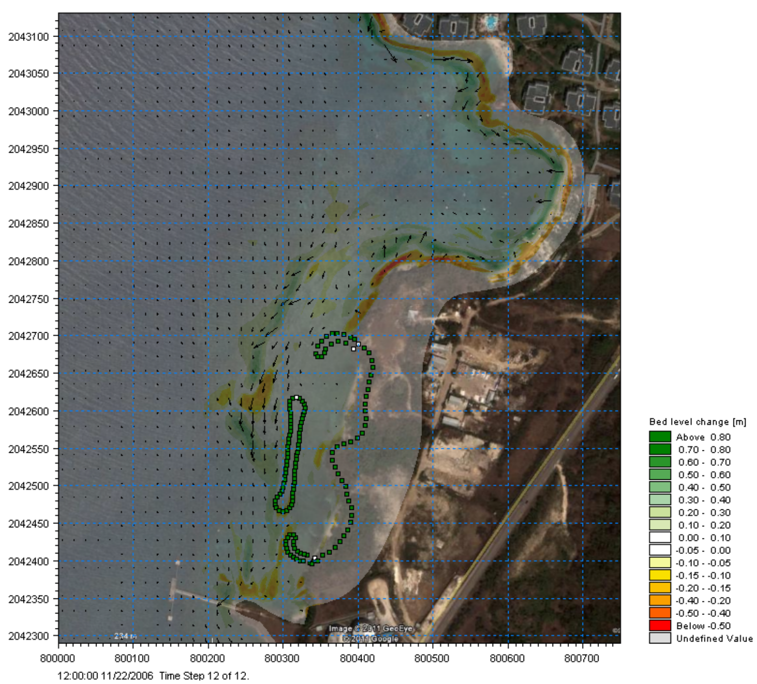



# MIKE 21 Simulation Results: Proposed Option 7 at peak of November 2006 Storm



## APPENDIX D

### ROVING SNORKEL – SPECIES LIST

### Fish and Rays (vertebrates) Data

| Common Name | Scientific Name | Economic/ Ecological Value | Occurrence/ Frequency |
|---|---|---|---|
| School Master | *Lutjanus apodus* | Jamaican (reef) Fin Fishery | F |
| Stoplight Parrot | *Sparisoma viride* | Jamaican (reef) Fin Fishery, Herbivore | D |
| Princess Parrot (juv) | *Scarus taeniopterus* | Jamaican (reef) Fin Fishery, Herbivore | O |
| Other Parrots | Scaridae | Jamaican (reef) Fin Fishery, Herbivore | F |
| Spanish Grunt | *Haemulon macrostomum* | Jamaican (reef) Fin Fishery | A |
| Spanish Hogfish | *Bodianus rufus* | Dive attraction/ Fin Fishery | R |
| Doctor Fish (juv) | *Acanthurus chirurgus* | Jamaican (reef) Fin Fishery | O |
| Blue Tang (Juvenile) | *Acanthurus coeruleus* | Ornamental/Dive attraction | F |
| Beaugregory (Juvenile) | *Stegastes leucostictus* | Ornamental/Dive attraction | F |
| Banded Butterfly Fish | *Chaetodon striatus* | Ornamental/Dive attraction | F |
| Four Spot Butterfly Fish | *Chaetodon capistratus* | Ornamental/Dive attraction | F |
| Dusky Damsel | *Stegastes adustus* | Ornamental/Dive attraction | F |
| Cocoa Damsel | *Stegastes variabilis* | Ornamental/Dive attraction | F |
| Squirrel Fish | *Holocentrus adscensionis* | Jamaican (reef) Fin Fishery | A |
| Spotted Goat Fish | *Pseudupeneus maculatus* | Jamaican (reef) Fin Fishery | O |
| Needlefish (Juvenile) | *Ablennes hians* | | R |
| Balloonfish | *Diodon holocanthus* | Dive attraction | O |
| Lionfish (adult) | *Pterois volitans* | Alien invasive species – threat | O |
| Unidentifiable Juveniles | Reef | Potential fishery, | |
| Unidentifiable Juveniles | Pelagics | Potential fishery, bait | A |
| Yellow Sting Ray | *Urolophus jamaicensis* | Dive attraction | R |

**D – Dominant; A – Abundant; F – Frequent; O – Occasional; R – Rare**

### Invertebrates Data

| Common Name | Scientific Name | Economic/ Ecological Value | Occurrence / Frequency |
|---|---|---|---|
| Sea Egg | *Tripnuestes ventricosus* | Potential fishery | F |
| Rock Urchin | *Echinometra viridis* | Herbivore | O |
| Pencil Urchin | *Eucidaris tribuloides* | Herbivore | O |
| Long-Spined Urchin | *Diadema antillarum* | Primary Herbivore | F |
| Green Urchin | *Lytechinus varietgatus* | Herbivore | O |
| Rock Boring Urchin (red or black coloured) | *Echinometra lucunter lucunter* | Herbivore | F |
| Sand Dollar | *Clypeaster species* | | O |
| Red Heart Urchin | *Meoma ventricosa ventricosa* | | O |
| Sea Cucumber | *Actinopygia sp* | | O |
| Sea Star | *Oreaster reticulatus* | | O |
| Hermit Crab | *Paguristes sp* | | O |
| Blue Crab | *Callinectes sp* | | O |
| Lettuce Sea Slug | *Elysia crispata* | | O |
| Reef Squid | *Sepioteuthis sepioidea* | | R |
| Octopus* | *Octopus sp* | | R |
| Conch | *Strombus gigas* | Very important Jamaican Fishery | R |
| Hydroid | *Thyroscyphus ramosus* | | O |

| | | | |
|---|---|---|---|
| Chiton | *Acanthopleura granulata* | | F |
| Sea Anemone | *Condylactis gigantea* | | O |
| Tube Dwelling Anemone | Unidentified | | |
| Feather Duster | *Sabellastarte magnifica* | | R |
| Mustard Sponge | *Pseudoceratina crassa* | | R |
| Encrusting sponge | *Plakortis angulospiculatus* | | O |
| Fireworm | *Hermodice carunculata* | | O |

**D – Dominant; A – Abundant; F – Frequent; O – Occasional; R – Rare**

**Coral Data**

| Common Name | Scientific Name | Economic/ Ecological Value | Occurrence/ Frequency |
|---|---|---|---|
| Finger Coral | *Porites porites* | | O |
| Thin Finger coral | *Porites divaricata* | | F |
| Lesser Starlet coral | *Siderastrea radians* | Reef building corals; biodiversity; shoreline protection; habitat | F |
| Fire Coral | *Millepora alcicornis* | | R |
| Great Star coral | *Montastrea cavernosa* | | R |
| Rose coral | *Manicina areolata* | | O |
| Tube Coral | *Cladocora arbuscula* | | R |
| Sea Fan | *Gorgonia ventalina* | | R |

**D – Dominant; A – Abundant; F – Frequent; O – Occasional; R – Rare**

**Algae and Plant Data**

| Scientific Name | Group | Economic/ Ecological Value | Occurrence/ Frequency |
|---|---|---|---|
| *Caulerpa racemosa* | Green Algae | | O |
| *Padina jamaicensis* | Brown Algae | | A |
| *Udotea sp.* | Green Algae | Pioneer species | R |
| *Penicillus spp.* | Green Algae | Pioneer species | A |
| *Dictyota sp.* | Brown Algae | | A |
| *Amphiroa rigida* | Red Algae | | R |
| *Ventricaria ventricosa* | Green Algae | | O |
| *Dictyosphaeria cavernosa* | Green Algae | | O |
| *Enteromorpha sp.* | Green Algae | Pioneer species | F |
| *Halimeda tuna* | Green Algae | | O |
| *Halimeda incrassata* | Green Algae | | F |
| *Acetabularia crenulata* | Green Algae | | R |
| *Turbenaria sp.* | Brown Algae | | R |
| *Codium sp.* | Green Algae | | R |
| *Wrangelia penicillata* | Red Algae | | O |
| *Thalassia testudinum* | Flowering Plant | | D |

**D – Dominant; A – Abundant; F – Frequent; O – Occasional; R – Rare**

Transect 1 view toward sea

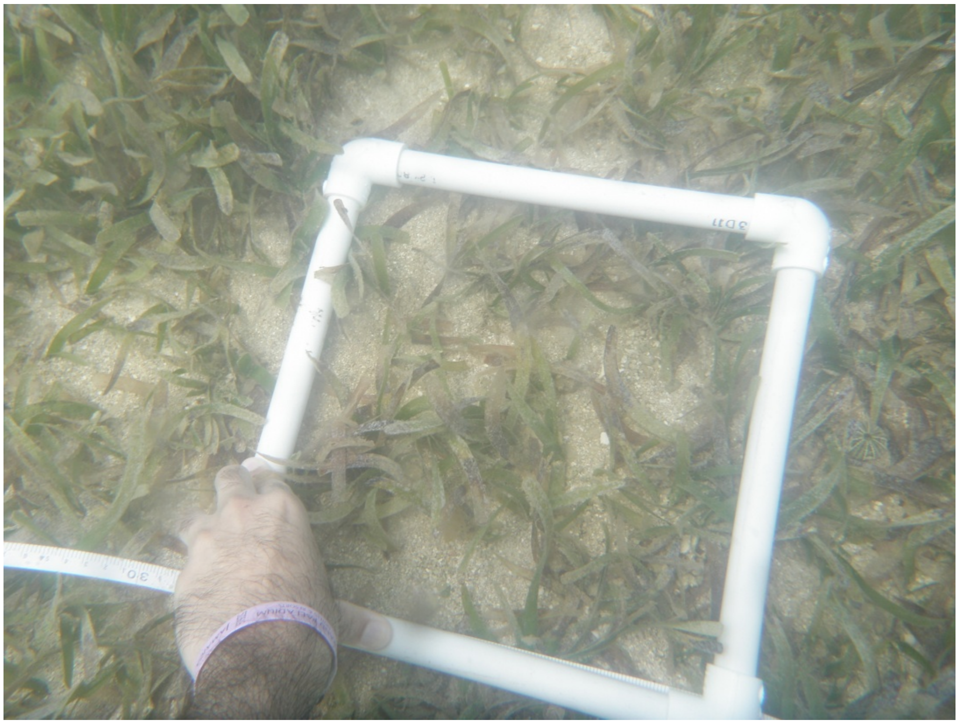
Transect 1 view toward beach

Transect 1 Foreshore

Transect 1 Seagrass and small coral

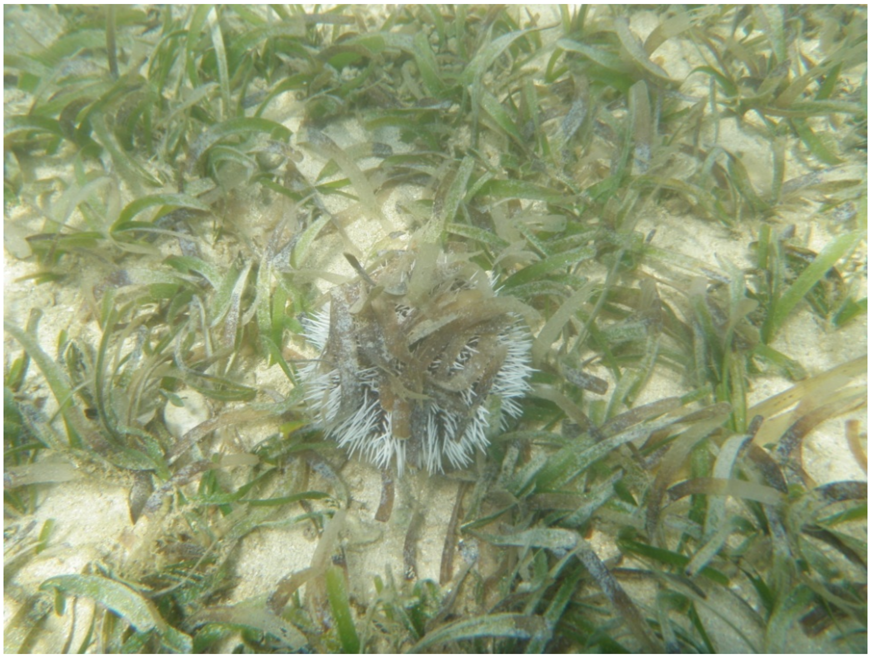
Transect 1 Close up of small coral

Transect 1 small coral amongst seagrass



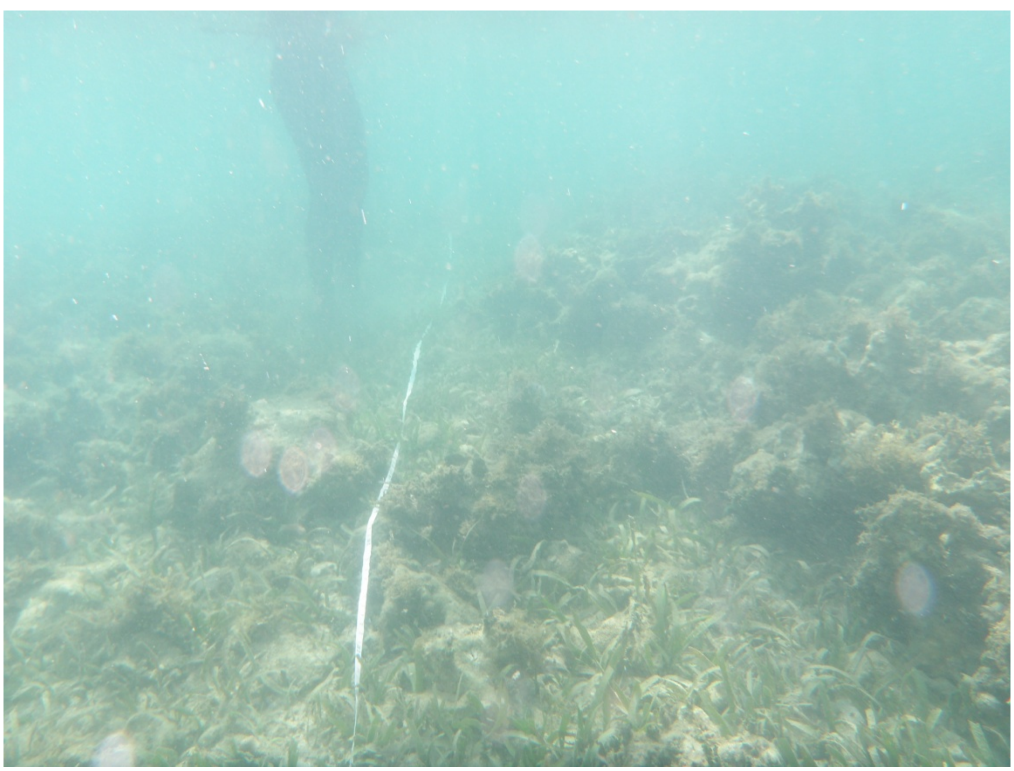
Transect 1 Seagrass

Transect 1 Seagrass



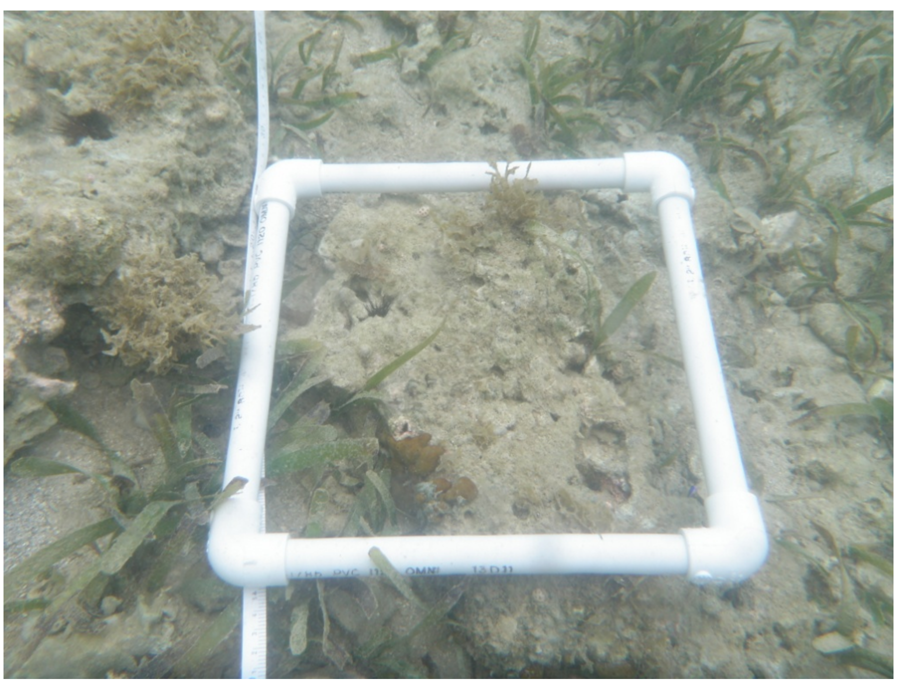
Transect 1 sea urchin

Transect 2 shoreline

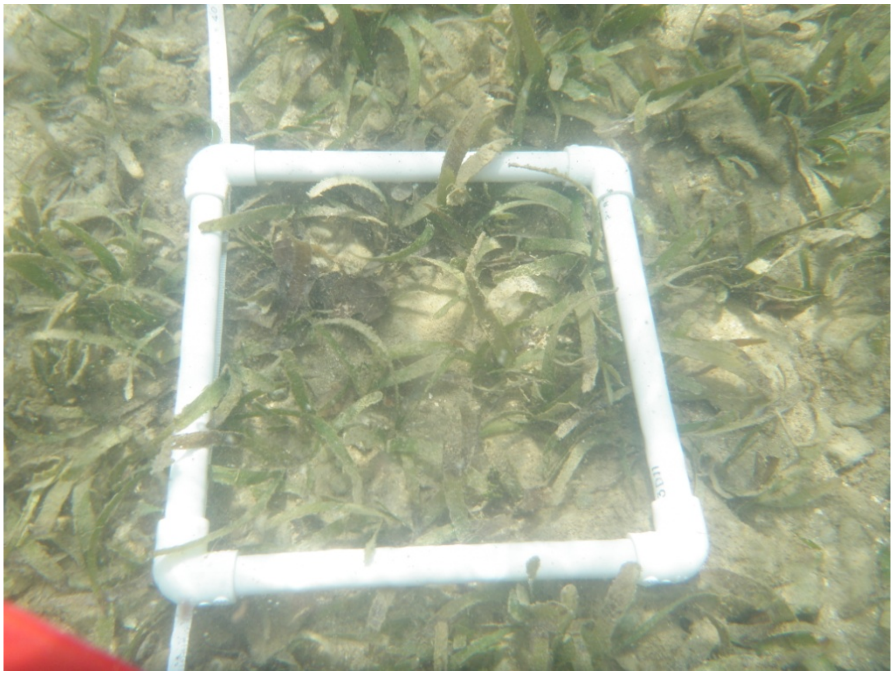


Transect 2 seagrass and rubble

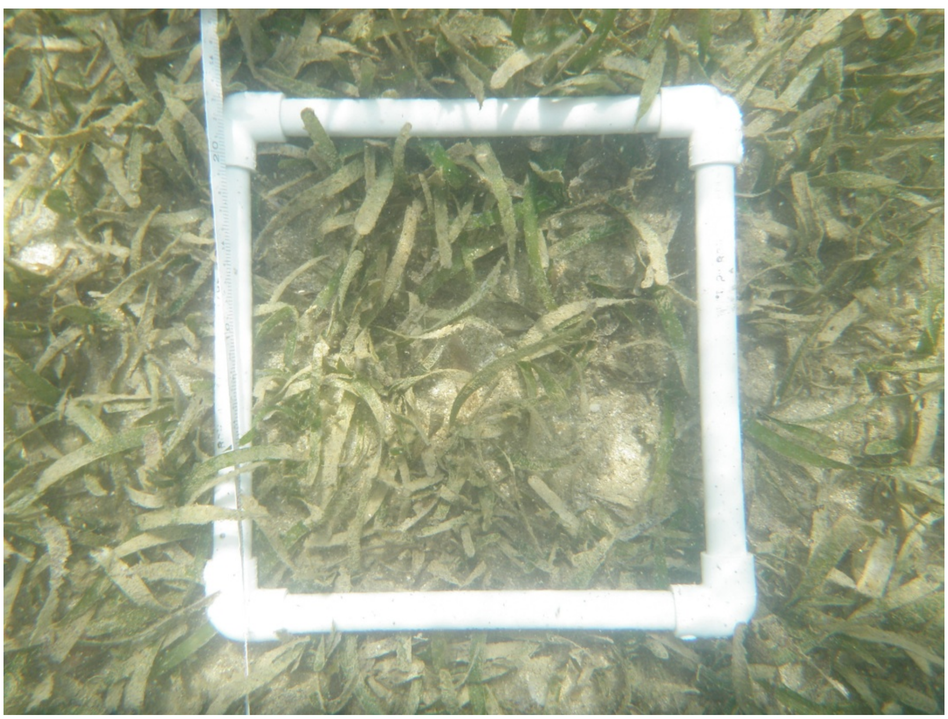
Transect 2 small corals



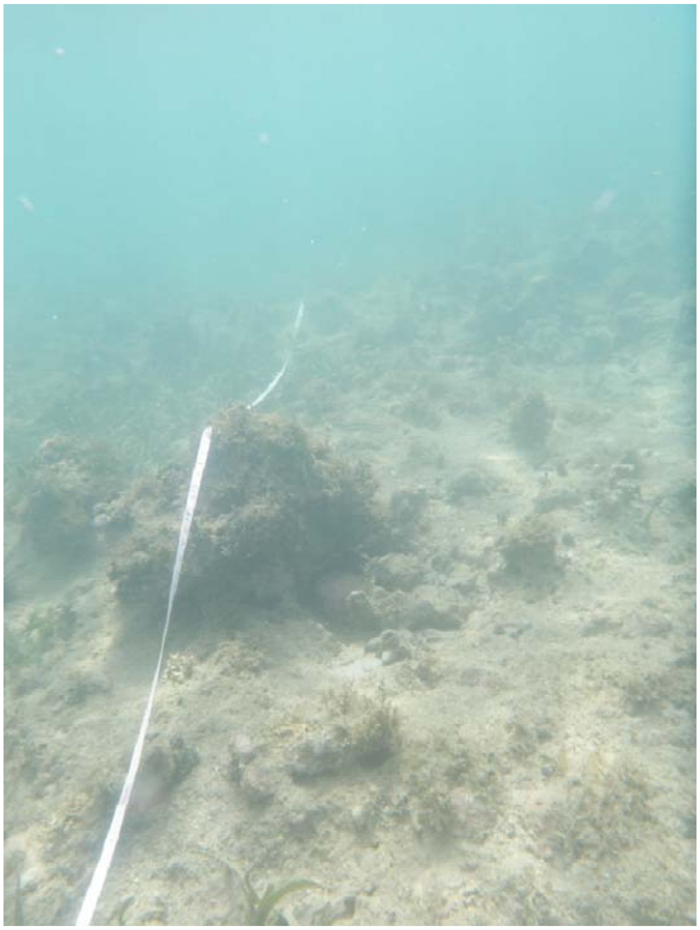
Transect 2 pavement and rubble



Transect 2 seagrass



Transect 2 seagrass



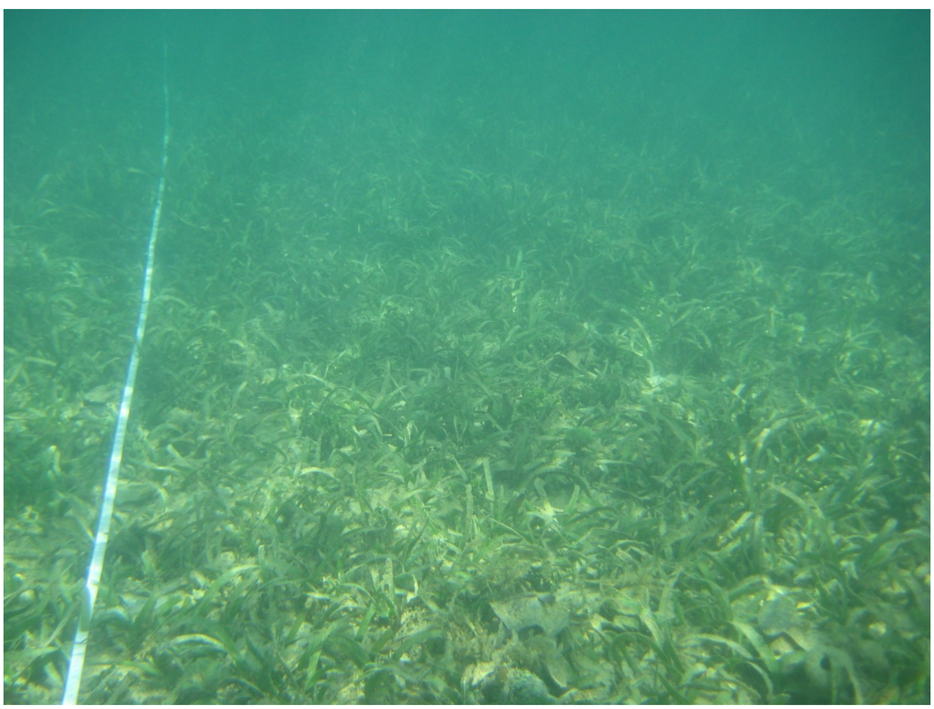
Transect 2 pavement and rubble

Transect 2 *Diadema* sp.

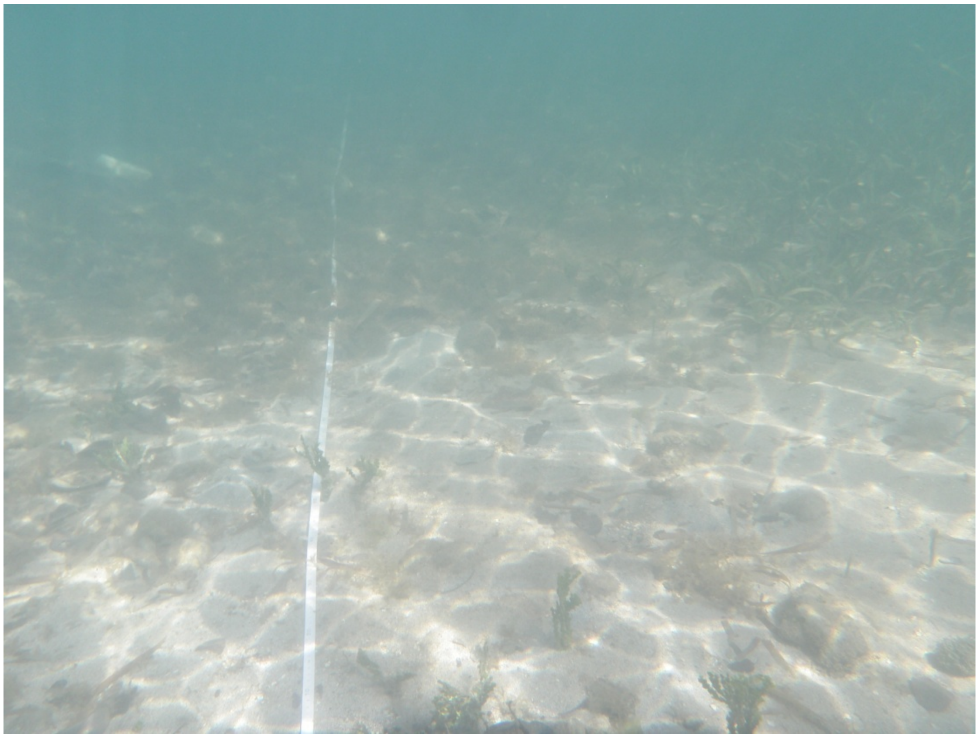
Transect 2 *Diadema* sp.

Transect 2 showing sea grass bed

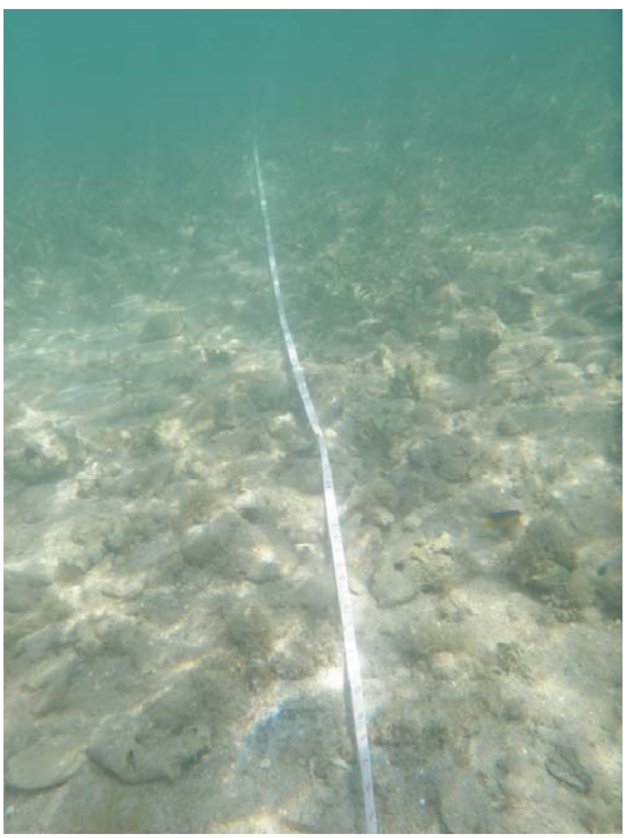
Transect 3 showing seagrass bed at 40m along transect



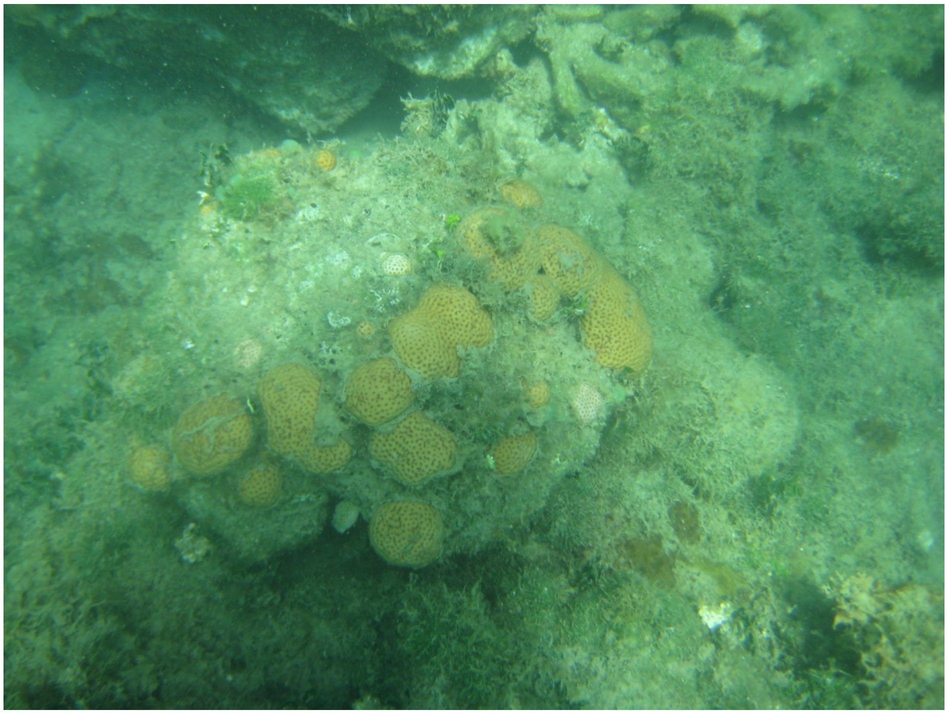
Transect 3 seagrass bed



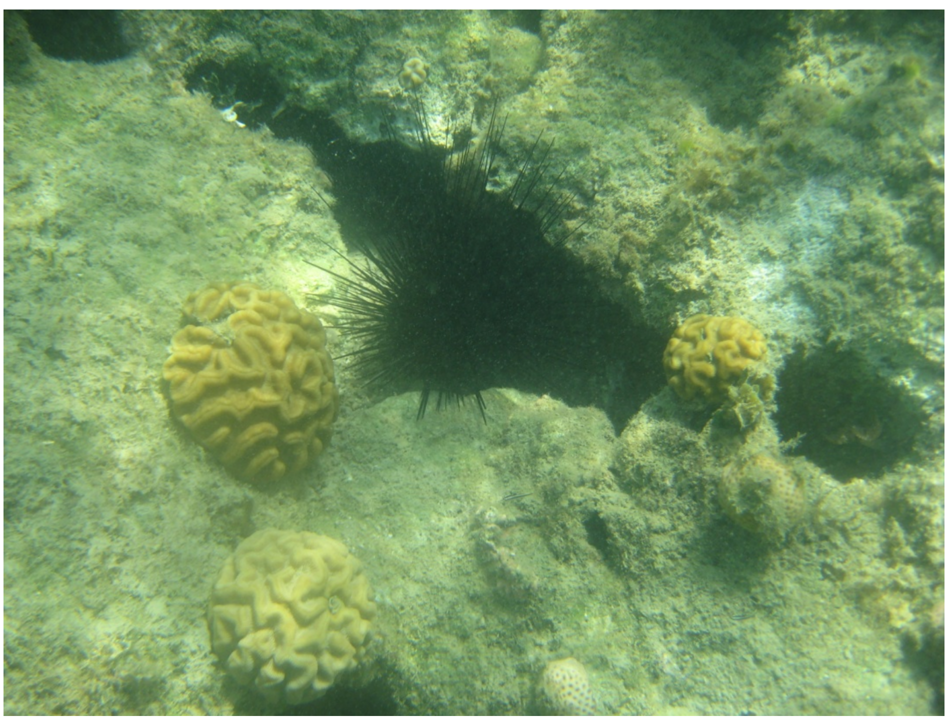
Transect 3 rubble and pavement



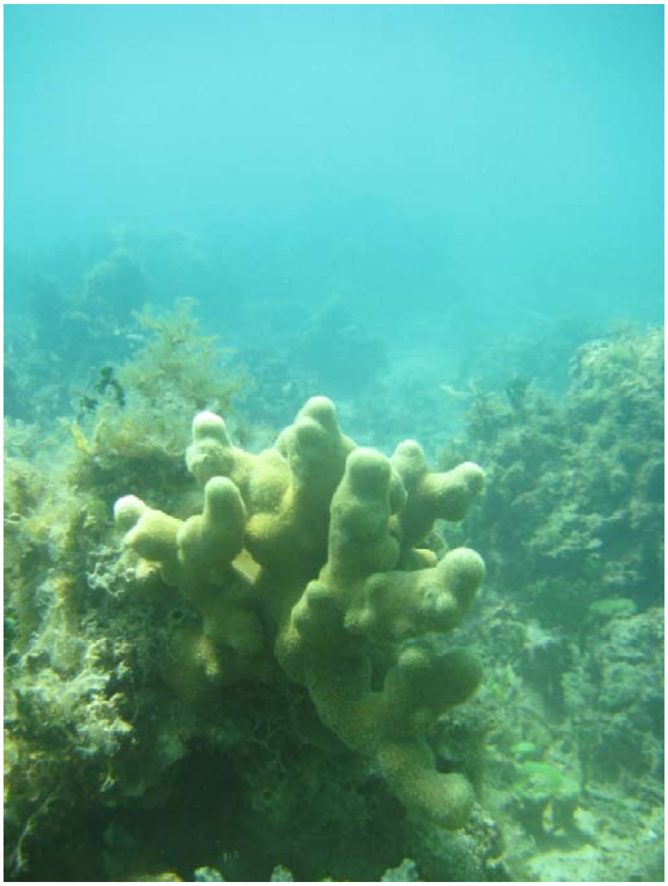
Transect 3 rubble and pavement



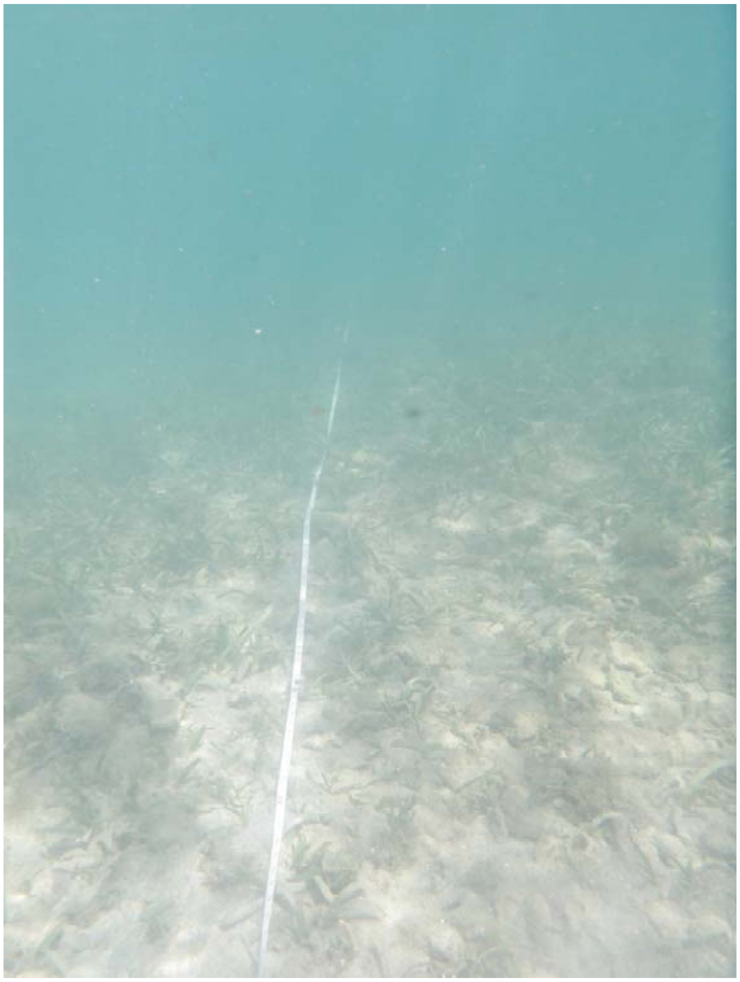
Transect 3 small coral adjacent to transect



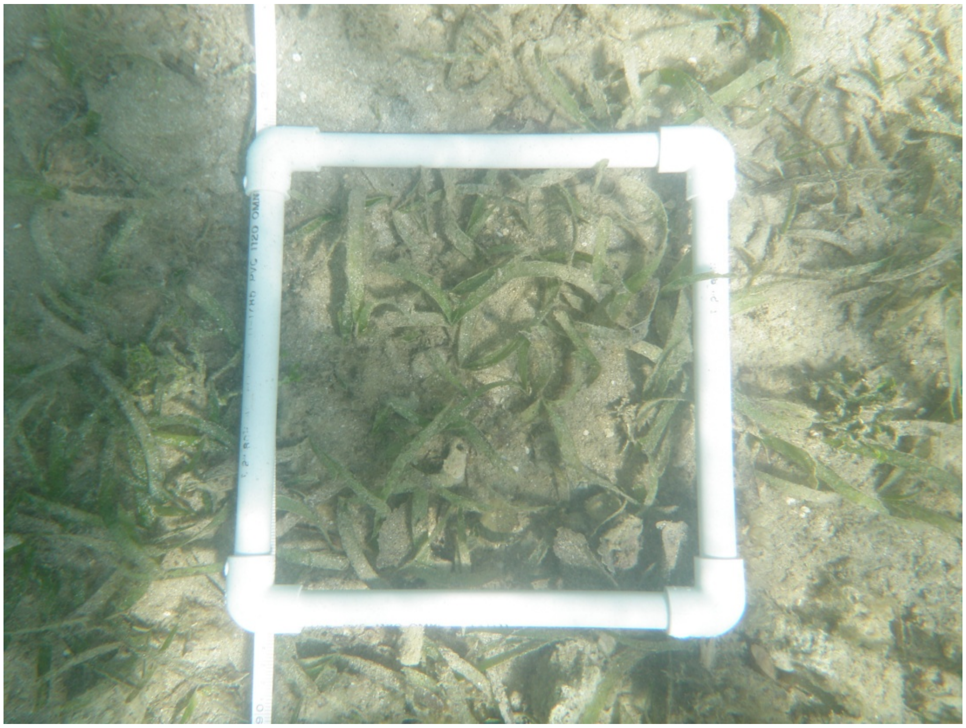
Transect 3 small corals on pavement and *Diadema* sp. adjacent to transect



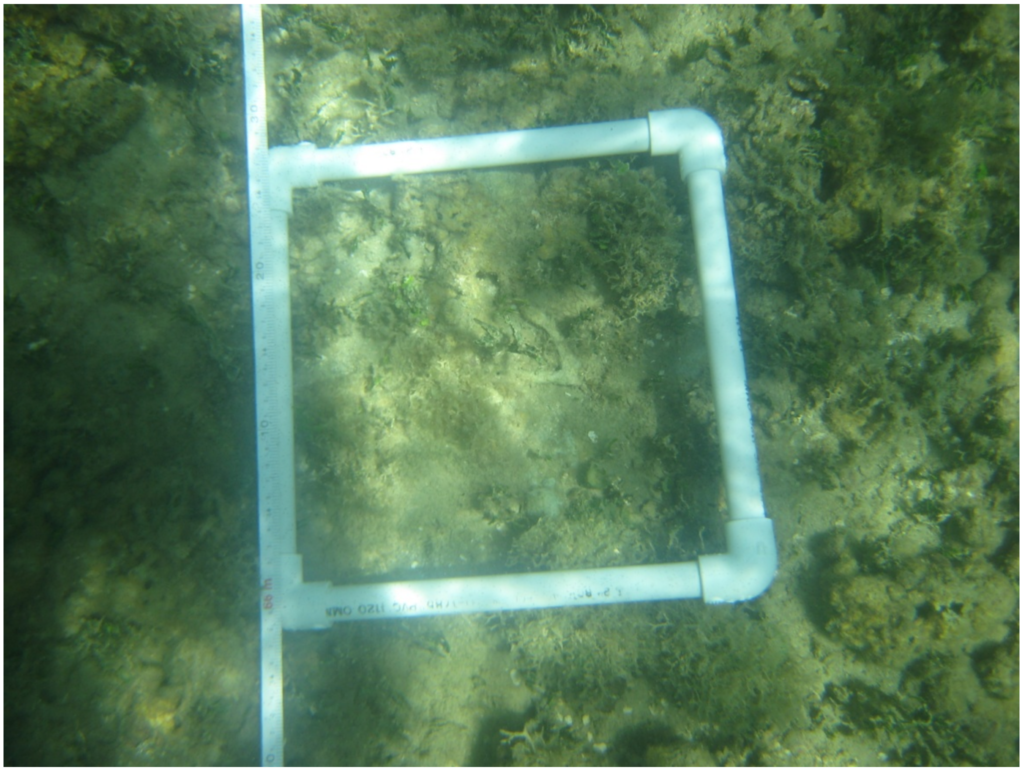
Transect 3 coral adjacent to end of transect



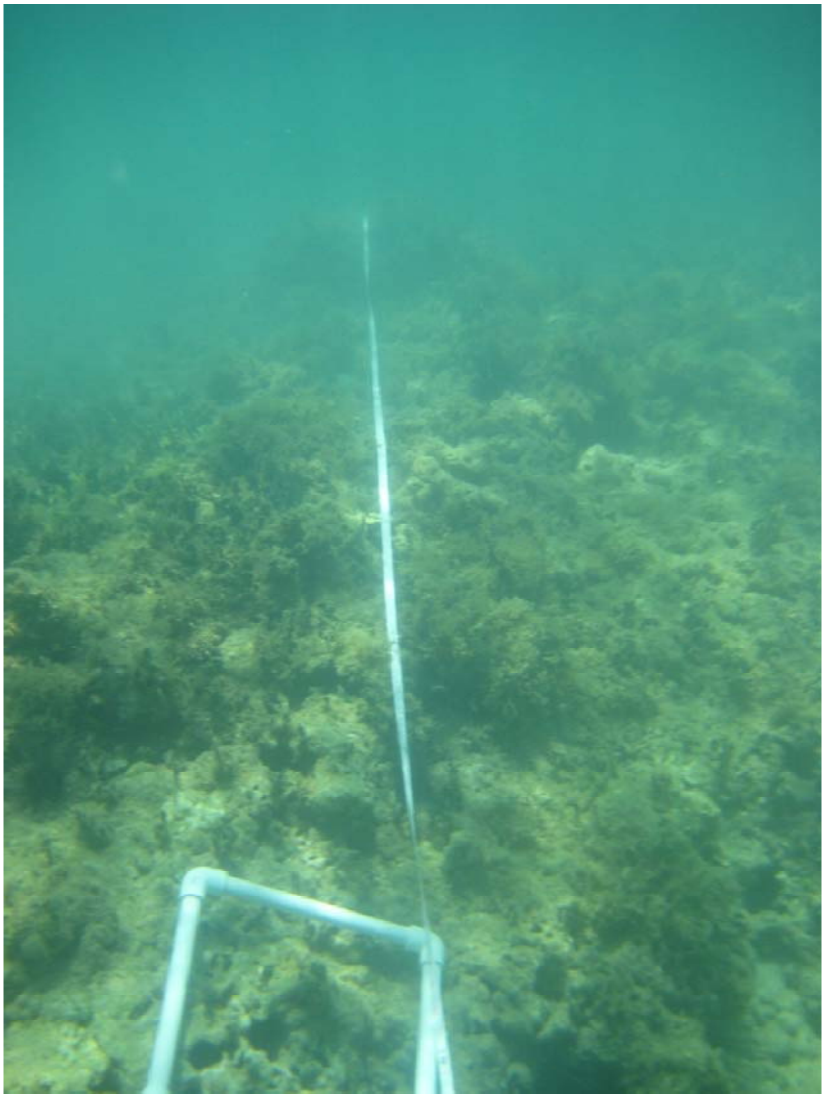
Transect 3 rubble and seagrass bed



Transect 3 seagrass

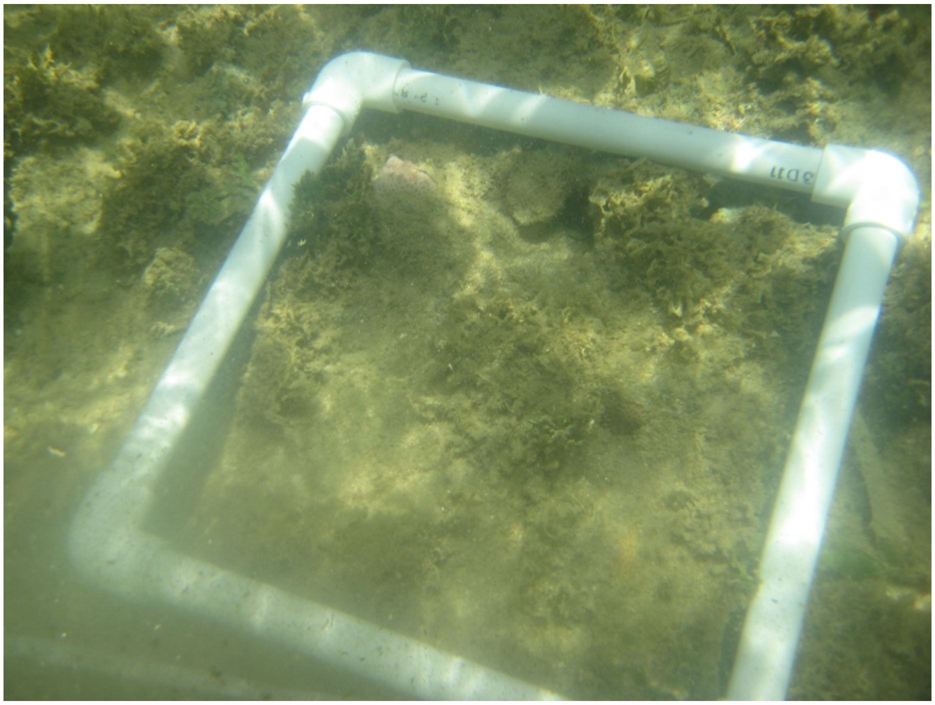


Transect 4 pavement and algae

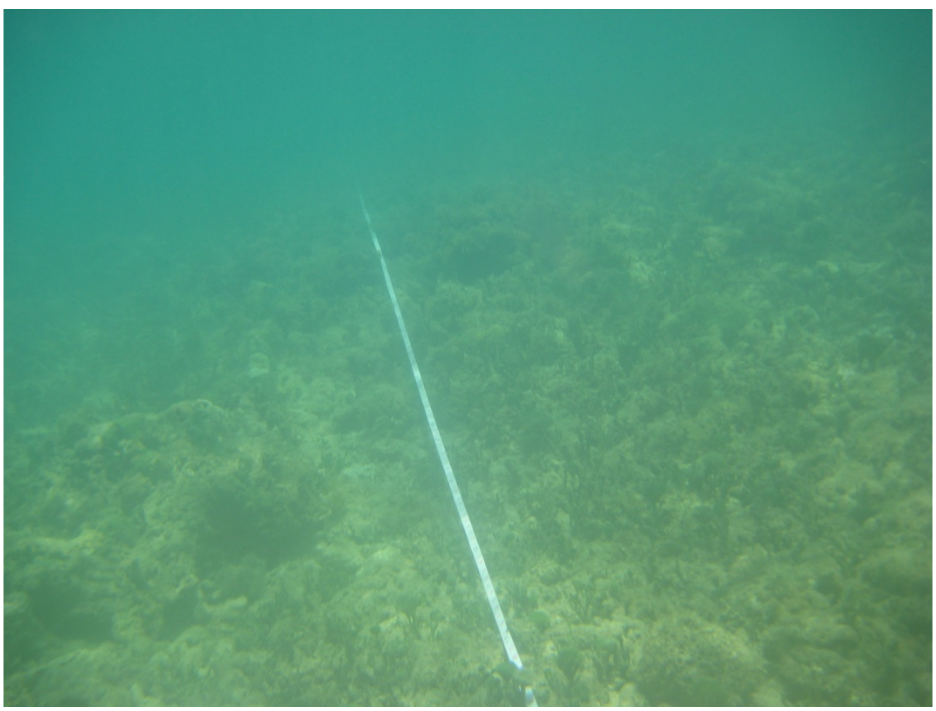


Transect 4 pavement, relic reef, algal bed

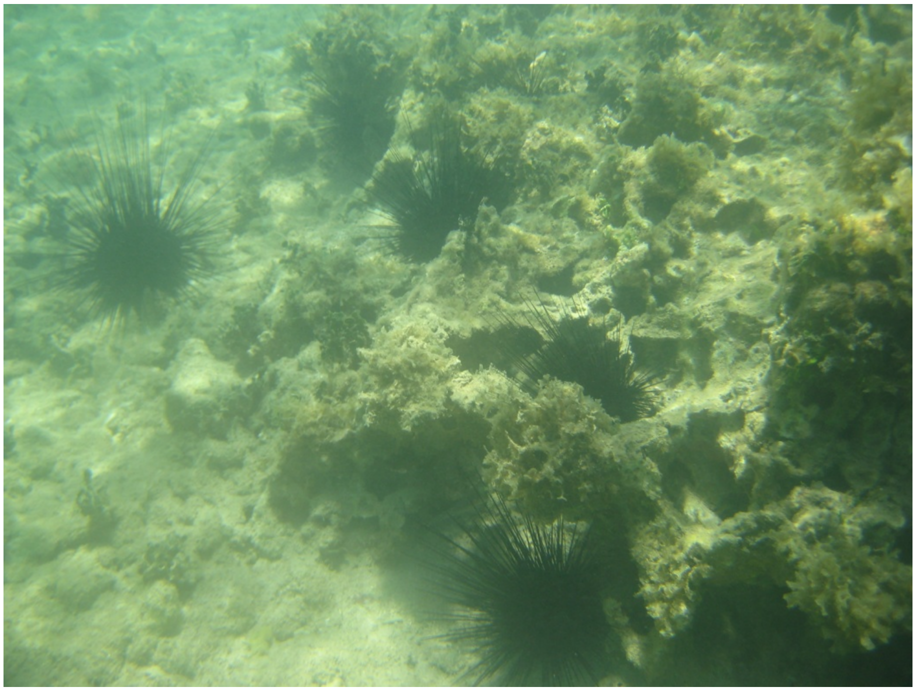
Transect 4 toward the sea



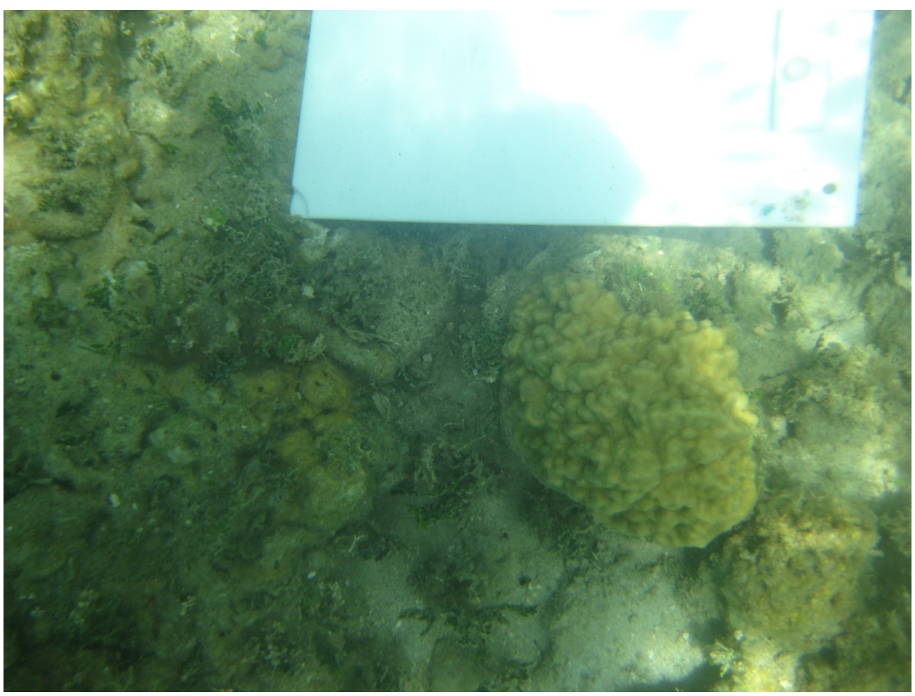
Transect 4 pavement



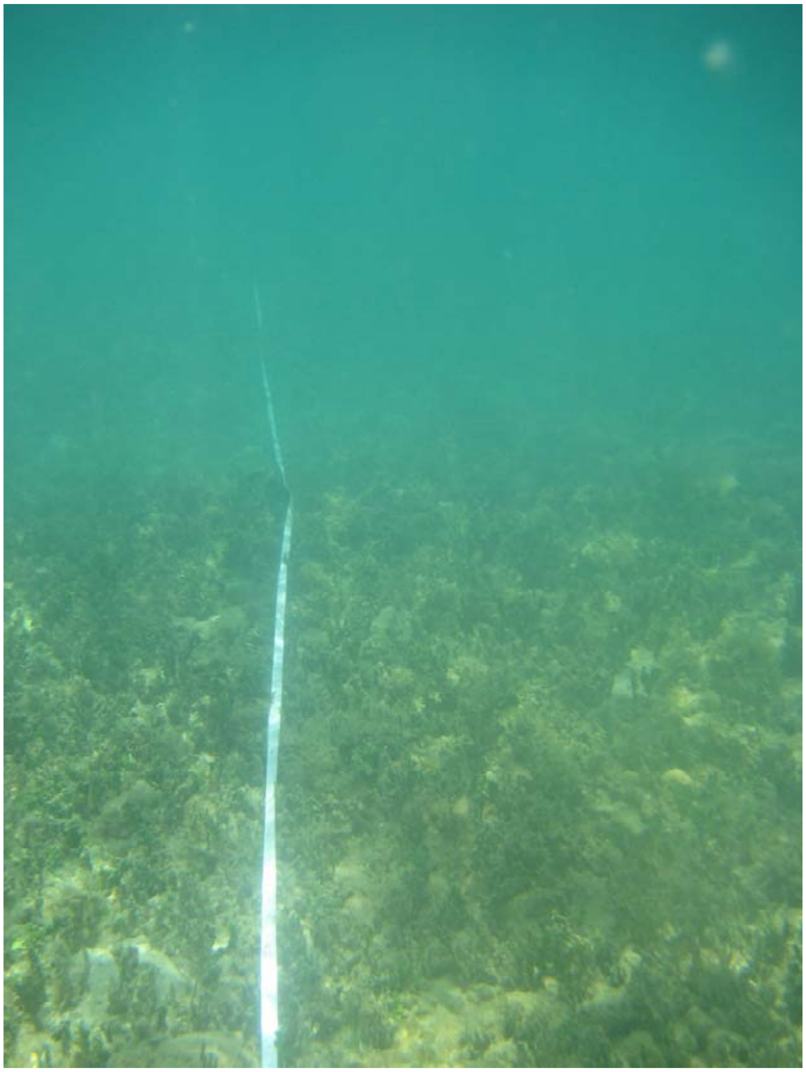
Transect 4 rubble, pavement and algal bed



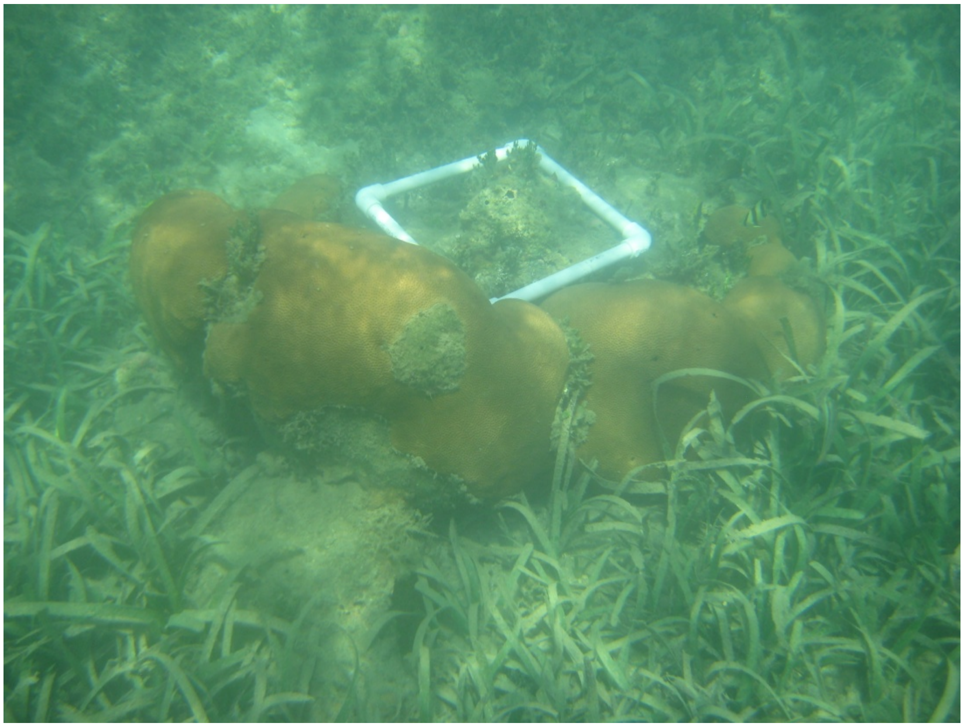
Transect 4 relic reef and *Diadema* sp.



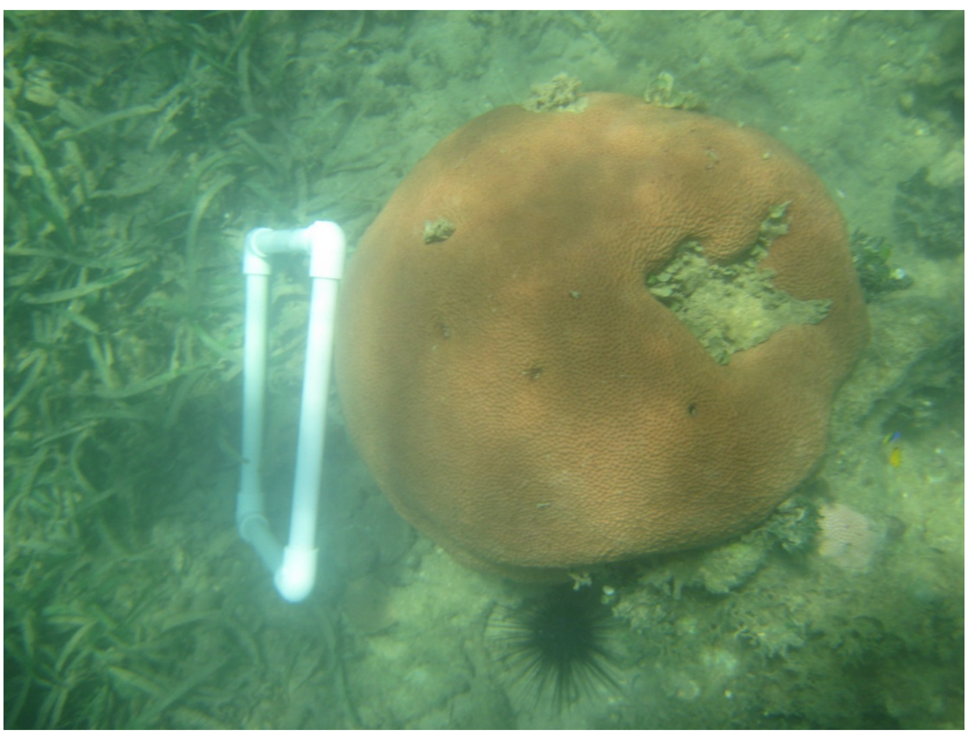
Transect 4 coral colony



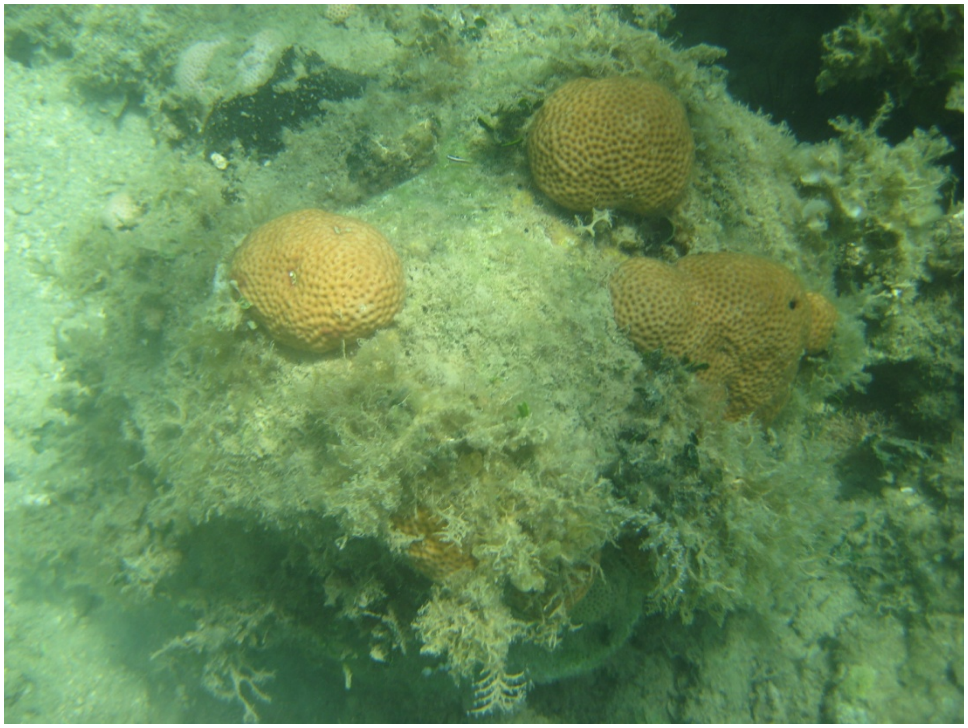
Transect 4 pavement and algal bed



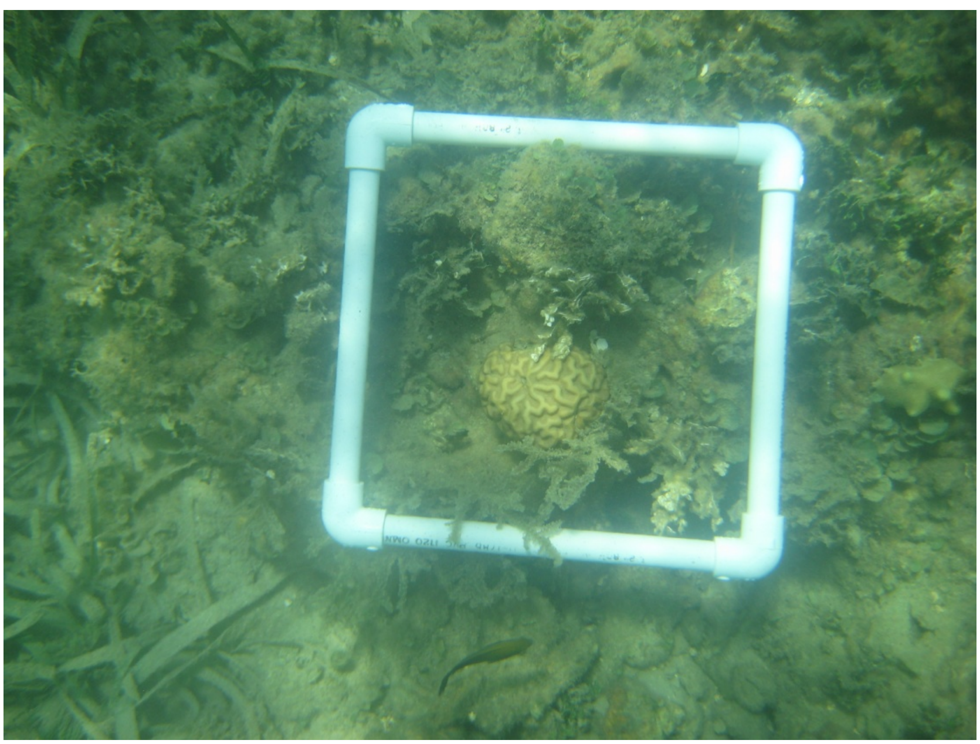
South Beach large coral colony



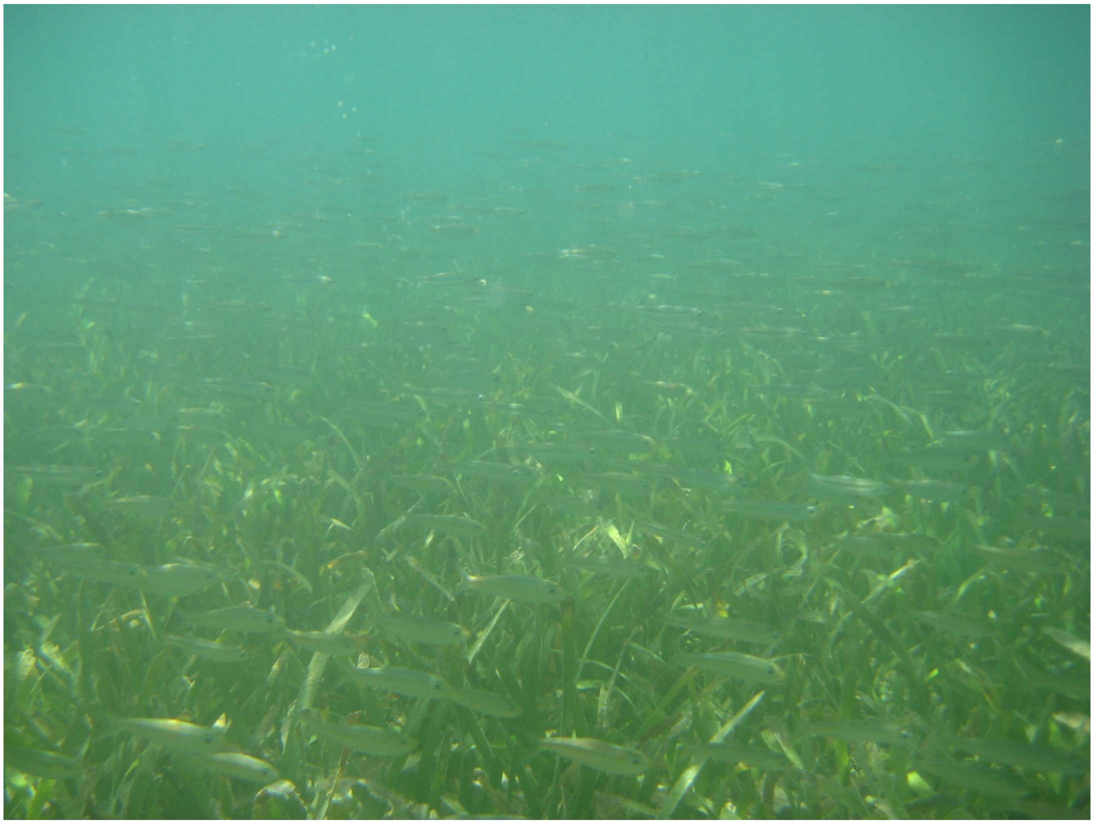
South Beach large coral colony



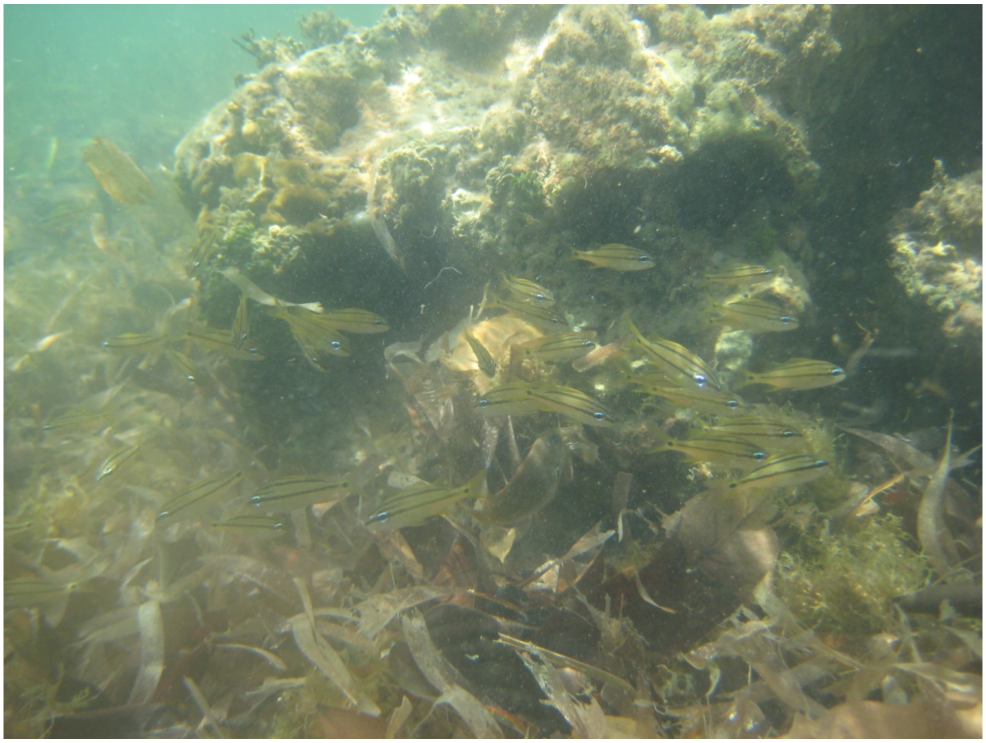
South Beach small coral colony



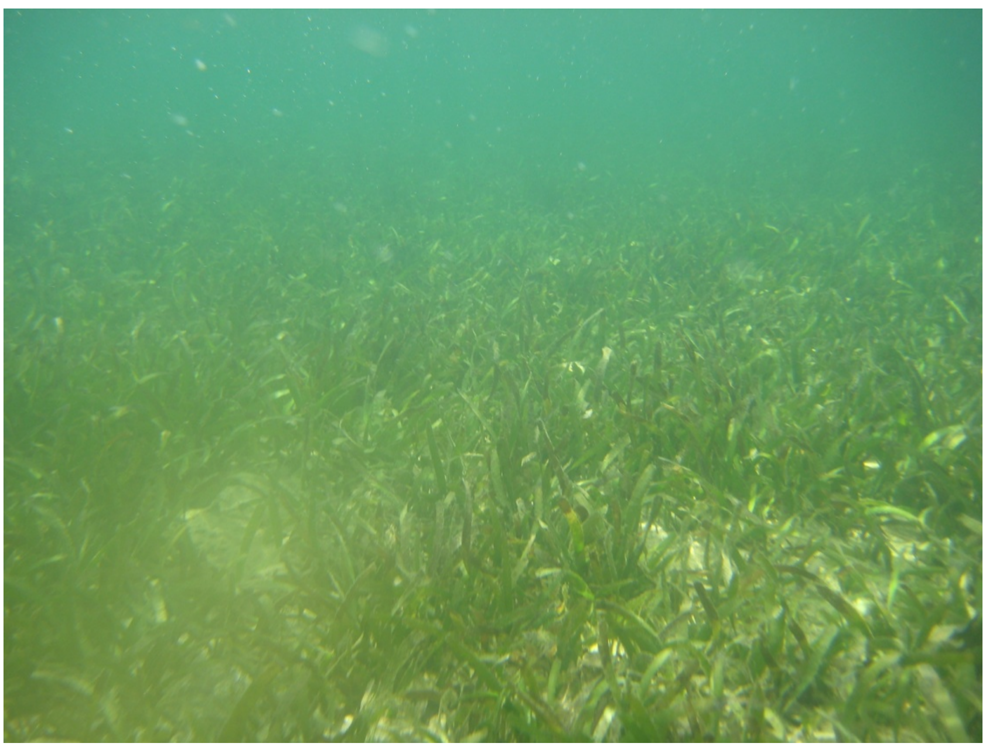
South Beach small coral colony



South Beach school of juvenile fish



North Beach school of juvenile fish



North Beach seagrass bed