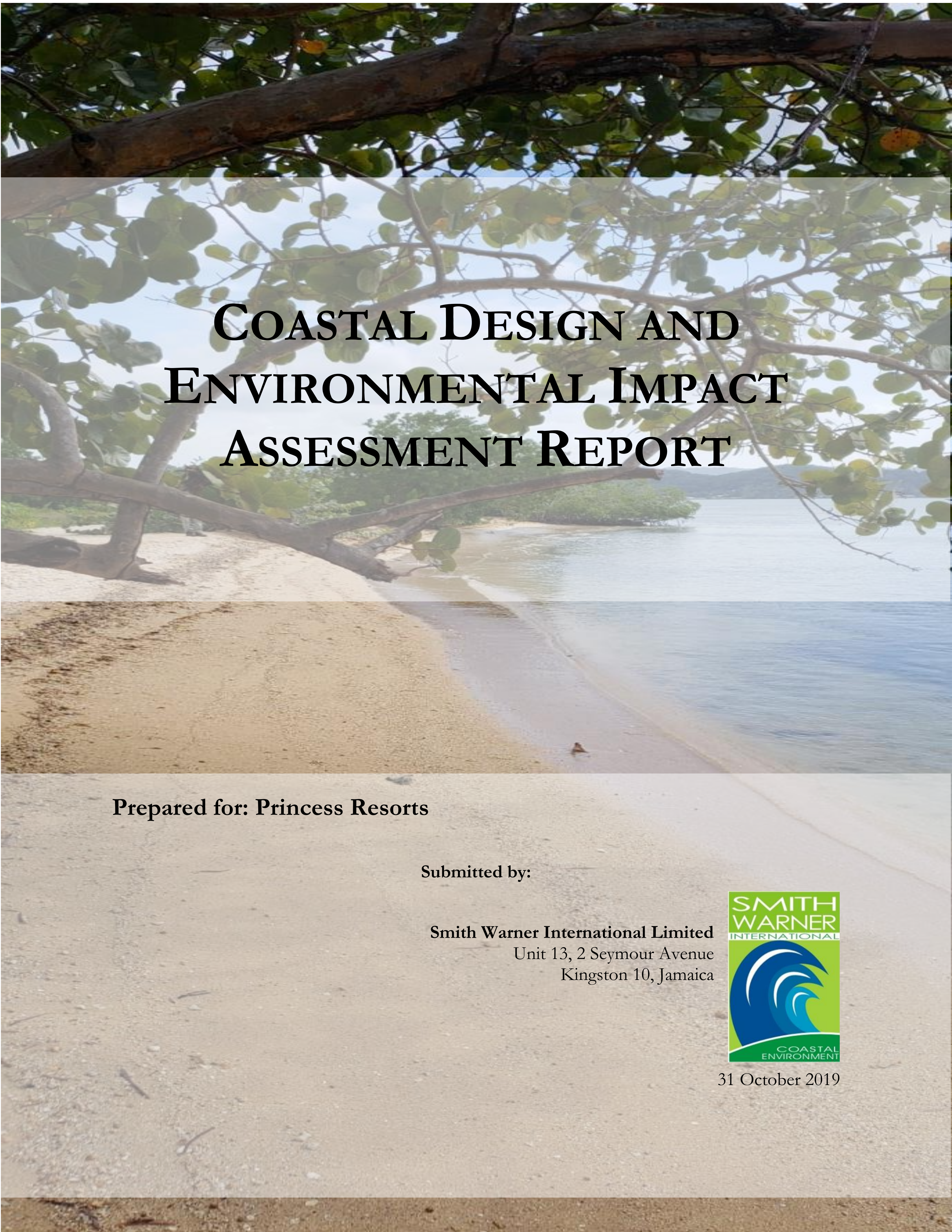

COASTAL DESIGN AND ENVIRONMENTAL IMPACT ASSESSMENT REPORT

Prepared for: Princess Resorts

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Contents

Executive Summary

The Princess Resorts Group acquired over 180 acres of land in the Green Island area of Hanover. Princess Resorts Group intends to use a portion of this area to construct a 2,042-room eco-resort that serves a varying clientele. The hotel will have two phases as follows:

- Phase 1
  - Hotel 1 (Adults Only)
  - Hotel 2 (Family resort)
- Phase 2
  - Hotel 3 (Family resort)
  - Hotel 4 (Adults Only)

The resort requires resort-grade beach along several areas of the shoreline to serve the hotel properties. The eastern bay where Hotel 1 is proposed has a natural beach, but its current state requires significant improvement to reach resort standards. The shoreline along Hotel 2 and Hotel 3 is a rocky shore exposed to rough seas daily and, as such, there is no significant natural beach formation. A beach along this area will no doubt require creative intervention. The shoreline at Hotel 3 has a natural sandy beach area but the nearshore is shallow and the seabed is rocky. As such, there is no opportunity for wading. This part of the shoreline is also eroding. Another significant feature of the hotel along the coastline is the proposed Sea Room development within the eastern bay. The area proposed for this development is ideal from the point of view that it is naturally sheltered from large waves due to the presence of reefs offshore. The area, however, has good seagrass coverage and is close to a band of mangroves.

Another component investigated by SWI is the storm water drainage for the resort. The master plan for the drainage does not require any direct drainage to the shoreline and the drainage plan is discussed in detail in a separate report (attached as Appendix A).

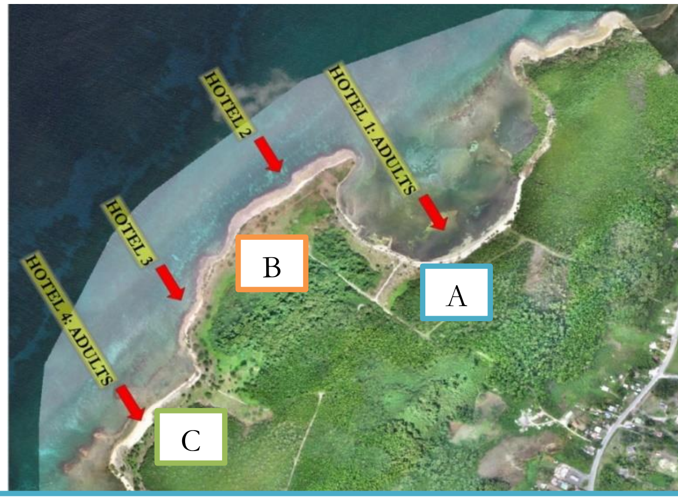
This report presents the baseline and impact assessment for beach enhancement within the bay areas, beach creation along the rocky coast and the Sea Rooms in the nearshore area. The report describes existing coastal conditions at the site, presents concepts for improving the shoreline areas, and discusses anticipated impacts and proposed mitigation measures.

The investigations included field surveys, data collection and numerical modelling of wave, current and sediment patterns. Some of the findings include:

- The site is located within the *Negril Marine Park/Protected Area* and is an environmentally sensitivity area. Based on an assessment of current site conditions we found the area to have a highly functioning ecosystem. Mangroves are acting as an erosion and coastal flood protection system and the benthic environment is thriving, creating a buffer for the shoreline.
- The shoreline positions of Hotels 1 and 4 have retreated significantly since 2003. Findings suggest, however, that the retreat was due to the removal of the mangroves in the area and not daily wave and current action.
- The water depths within the bay of Hotel 1 are very shallow, causing low current speeds (less than 0.1 m/s) and circulation issues. The circulation issues must be addressed to increase the beach's viability for resort use. At a minimum, the groynes currently in place must be removed.

- Hotels 2 and 3 are along a rocky shore that is bombarded by wave heights up to 2m. This suggests the area will not naturally support the formation of beach. Along the rocky shore the currents are fast (over 0.7 m/s) and have the potential to move a lot of sediment.
- The shoreline along Hotel 4 is protected by a shallow reef system. The reef, however, has gaps through which waves penetrate and reach the shoreline.
- The sediment transport modelling shows that sediment movement near Hotel 1 is minimal. Along the shoreline of Hotels 2-4 sediment movement is mainly in a westerly direction. Most of the sediment movement occurs between 70 and 80m of the shoreline.
- Under extreme conditions (i.e. a hurricane occurring once every 50 years and accounting for sea level rise due to climate change) the site is expected to be flooded by up to +2.6m MSL. At some sections of the site this represents more than 2m of water. The buildings and critical infrastructure must be raised above this flood level. We recommend that the building floor levels be at least +3.0m MSL and ground level be at least +2.8m MSL.

Objectives and constraints for each section of the shoreline are summarised in the image below. The main aim of the coastal enhancement is to create resort quality beaches at Hotels 1 and 4 and create more sunbathing area for occupants of Hotels 2 and 3.



A: Hotel 1

Aim: To create resort type beach with a nearshore that has good water circulation.
Constraints: The beach is eroding and shows areas of stagnation.
Options: Beach nourishment. Offshore structures to attenuate waves. Create a channel across headland to improve flushing.

B: Hotels 2 and 3

Aim: To create sunbathing area with a pocket beach for swimming.
Constraints: Aggressive wave conditions on a rocky shoreline. No natural sand deposits and corals in the nearshore.
Options: Create beach by excavating the land instead of going offshore. Create perched/dry beaches for sunbathing only along the rocky coastline.

C: Hotel 4

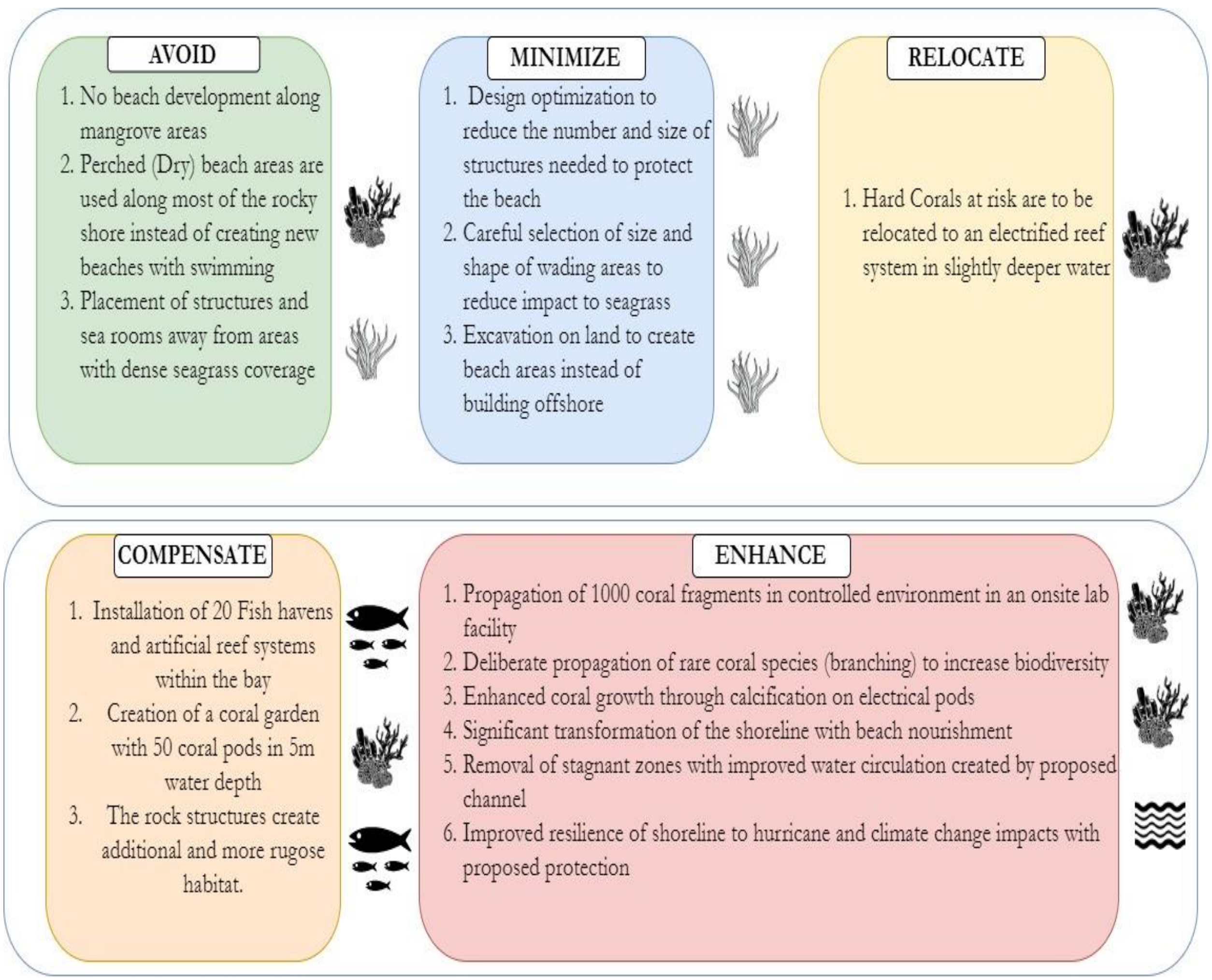
Aim: To create a stable sandy beach for the resort with area for wading.
Constraints: Very shallow nearshore with hard pavement seabed. Significant beach erosion noted over several years.
Options: Nourish to advance shoreline. Move further inland with the beach. Excavate to create pocket beaches. Groyne and nourishment to create beaches.

The final plan selected is described in the following table and shown in the figure below. The main addition to the bay at Hotel 1 is a flushing channel to encourage faster water exchange within the bay, getting rid of any contaminant within 1.5 days (less than international guidelines). Hotel 2 and 3 will share a 100m length pocket beach created by excavating and sloping the land. Additionally, the rocky shoreline will be a perched beach. The gaps in the reef at Hotel 4 will be filled with two submerged breakwaters.

Activities	*Structure Seafloor Area*
At Hotel 1	
• -1.5m MSL flushing channel through western headland • Two groynes at +2m MSL • 500m long nourished sandy beach at a 1:14 slope • 79m of shoreline nourishment with a revetment at +2.8m MSL • +2m MSL groyne to be used as a sport jetty	*2049m²/0.50ac*
Sea Rooms	*3300m²/0.81ac*
Hotel 2 and 3	
• Perched beach at +2.8m MSL retained by a revetment at +2.8m MSL • 130m long nourished pocket beach at a 1:14 slope • Two spur groynes at +3.0m MSL • One submerged breakwater at MSL	*2789m²/0.68ac*
Hotel 4	
• Two submerged breakwaters at -0.3m MSL • 450m long nourished sandy beach at a 1:14 slope • One groyne at +2m MSL. • Nearshore dredging of up to 6500m²	*1789m²/0.44ac*

The construction will be land-based and has several associated impacts on the marine benthos. The impacts of the proposed works will spread over the construction phase as well as the operational phase of the project. The strategies used to reduce and mitigate impacts can be grouped as shown in the figure below. The proposed works will directly impact a total of 2.43 acres (0.98 hectares) of seabed habitat. The impacted areas are currently covered with seagrass, sand, rubble and corals. During construction, measures are proposed to reduce the negative impacts associated with smothering, turbidity, oil pollution and post-construction debris. These include the use of turbidity barriers as well as washing the stones and other materials to remove silt. Coral relocation is advised, to remove all corals from the footprint of this impacted area.

The design proposes the addition of 20 fish havens with high rugosity and a total of 50 coral pods to host transplanted and propagated corals. Both measures should span over 2.5 acres offshore, thereby increasing the total available habit in the area. Mitigation and compensation measures will also include the propagation of corals via microfragmentation measures, and the removal of stagnant zones in the area to improve water quality. The multi-tier approach to mitigation is shown in the image below.



Marine impact mitigation strategies

1 Introduction

The Princess Resorts Group acquired over 180 acres of land in the Green Island area of Hanover. Princess Resorts Group intends to use a portion of this area to construct a 2042-room eco-resort that serves a varying clientele. The Green Island site (Figure 1.1) has a natural cove in the north washed by the Caribbean Sea as well as a long rocky shore to the west. Our understanding is that the objective of our scope of work is to significantly enhance the existing shoreline to create resort-grade beaches. Princess Resorts wishes to develop sustainable beach areas that will provide an excellent guest experience at the hotels.

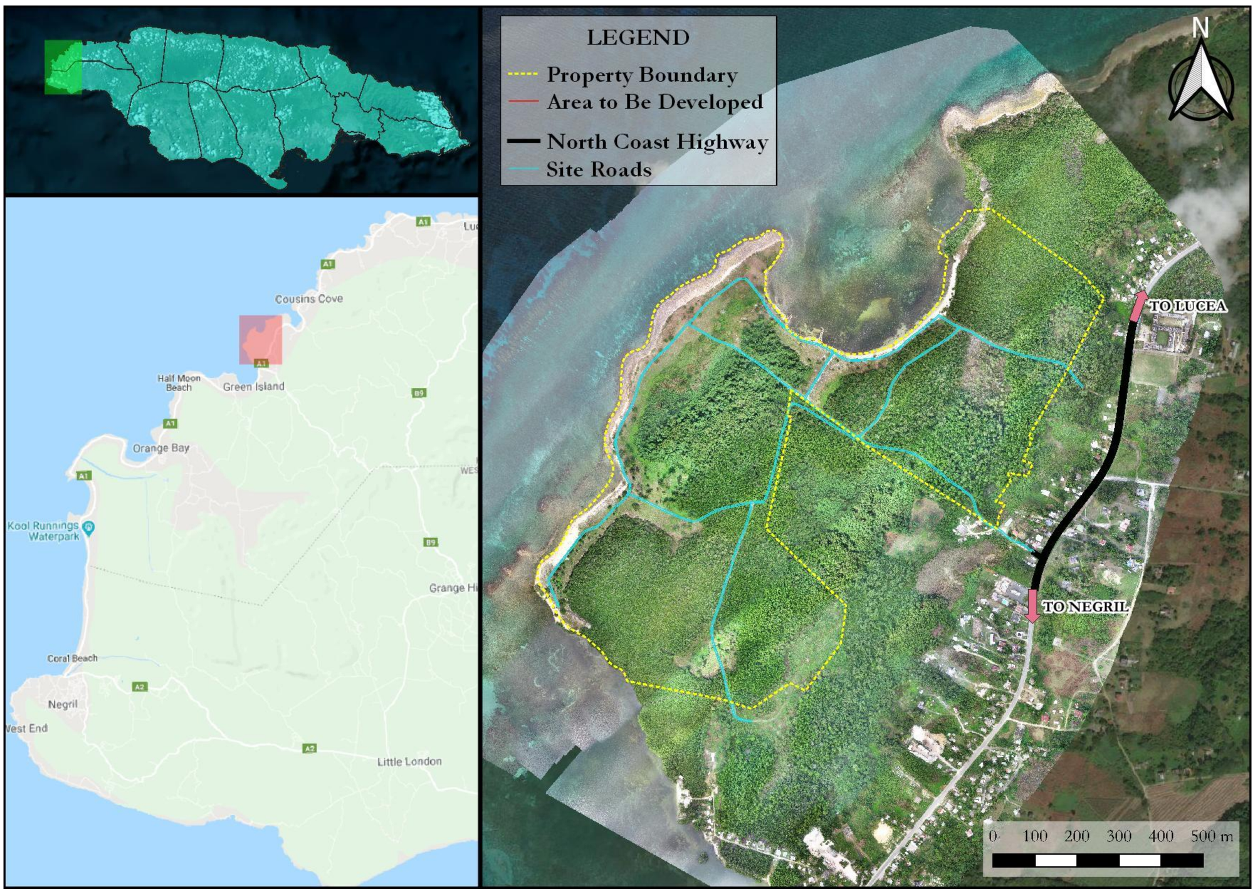


Figure 1.1 Location plan for the Princess Resorts in Green Island, Hanover

Princess Resorts will create four luxury hotels on the property using a phased approach. Each hotel will serve a different clientele, from adults only packages to family fun parks and public beach clubs.

The hotel will have two phases as follows:

- Phase 1
  - Hotel 1 (adults only)
  - Hotel 2 (family resort)
- Phase 2
  - Hotel 3 (family resort)
  - Hotel 4 (adults only)

1.1 Scope of Work

The hotel buildings will be located along the shoreline, which will be bordered by a dense mangrove forest and the Caribbean Sea. The shoreline along the property has sections of rock outcrops as well as narrow sandy beaches. A hotel of this size and occupancy will need a large enough beach area to comfortably accommodate the number of guests that will be using the beach daily. The beach is undoubtedly a very important part of the guest experience for the planned resort. For a comfortable resort, a dry beach area of 8-12m^2 per room is recommended; this is a recommendation that is not currently met by the existing conditions at the site, as only a half of the required beach area is available. Further, the seafloor immediately in front the shoreline has a high density of corals and seagrass, making it an environmentally sensitive area. Some sections of seafloor at the site, especially in the natural cove, have a significant amount of silt, which makes it unpleasant for wading and swimming. Therefore, while the shoreline presents several opportunities for creating a resort, additional work is needed to create the product sought by Princess Resorts.

Further inland, a wetland covered by dense mangrove forest plays a significant role in reducing flooding in the area. The low-lying nature of the site makes it easy for rainfall runoff to pond in the forest and reduces the potential for landward flooding from the sea. Placing a hotel resort in these areas has two immediate implications: (i) the development area to be used for buildings in the naturally existing floodplain is at risk of flooding; and (ii) the floodplain area will be reduced and may result in flooding of nearby communities. It is therefore necessary to investigate methods for reducing the flood risk to the development and mitigation measures to alleviate flood risk to the ambient areas.

This report presents findings from baseline and impact assessment studies carried out by Smith Warner International Limited (SWI) for the Princess Resorts Group. The report discusses existing coastal and drainage conditions for the site and presents concepts for improving the area for the resort.

1.2 Technical Approach

Developing an appropriate solution requires a staged process that allows the client to provide meaningful feedback. The approach is summarised in the following figure. Summary reports of Stages 1 and 2 are presented in Appendix A of this report. The main text of this report addresses the requirements of Stage 3.

Figure 1.2 Stages of work to be done

1.3 Format of Report

The table below presents the format of the report and brief summary of the chapter content.

Table 1-1 Report format

Chapter 2 **Physical Site Conditions**	This section of the report describes physical conditions at the site: observations from the site visit, preliminary analysis from a desktop review of the conditions, and results from the topographic and bathymetric surveys to be used throughout the study are shown.
Chapter 3 **Biological Site Conditions**	A detail description of the physical environment from the climate to the water quality as well as the biology of the site.
Chapter 4 **Socio Economic Context**	This section of the report presents the social and economic context of the project. It presents an understanding of the community that surrounds the site, the economy of the area and as well as give legal framework surrounding work in the marine environment for the area.
Chapter 5 **Baseline Coastal Processes**	This section describes the baseline coastal process conditions such as waves in the nearshore, current patterns, and extreme wave conditions the structures will be designed to withstand, as well as flooding potential from the sea.
Chapter 6 **Concept Development**	This chapter summarizes considerations for the selection of the final plan for the coastal and drainage design.

Chapter 7 **Coastal Design**	This section shows the response of the structure to extreme operational waves (i.e. swell waves), the response to hurricanes, the changes in waves and currents, as well as flushing potential in the area.
Chapter 8 **Structural Engineering Design**	This section presents considerations and calculations made in the structural design of the structures.
Chapter 9 **Impacts and Mitigation**	This section presents the overall scope of the project as well as the mitigation measures proposed to offset any impact identified.
Chapter 10 **Conclusions and Recommendations**	The final thoughts on the project are presented here as well as recommendations for further study.

2 Physical Site Conditions

The site shoreline is approximately 2.3km in length and includes rocky shoreline, sandy beach, and a mangrove coastline in some areas. The shoreline at each hotel location is as follows:

Hotel 1:	Sandy shoreline
Hotel 2:	Rocky shoreline
Hotel 3:	Rocky shoreline
Hotel 4:	Sandy shoreline and mangrove coast

Figure 2.1 shows the location of the hotels on the site.

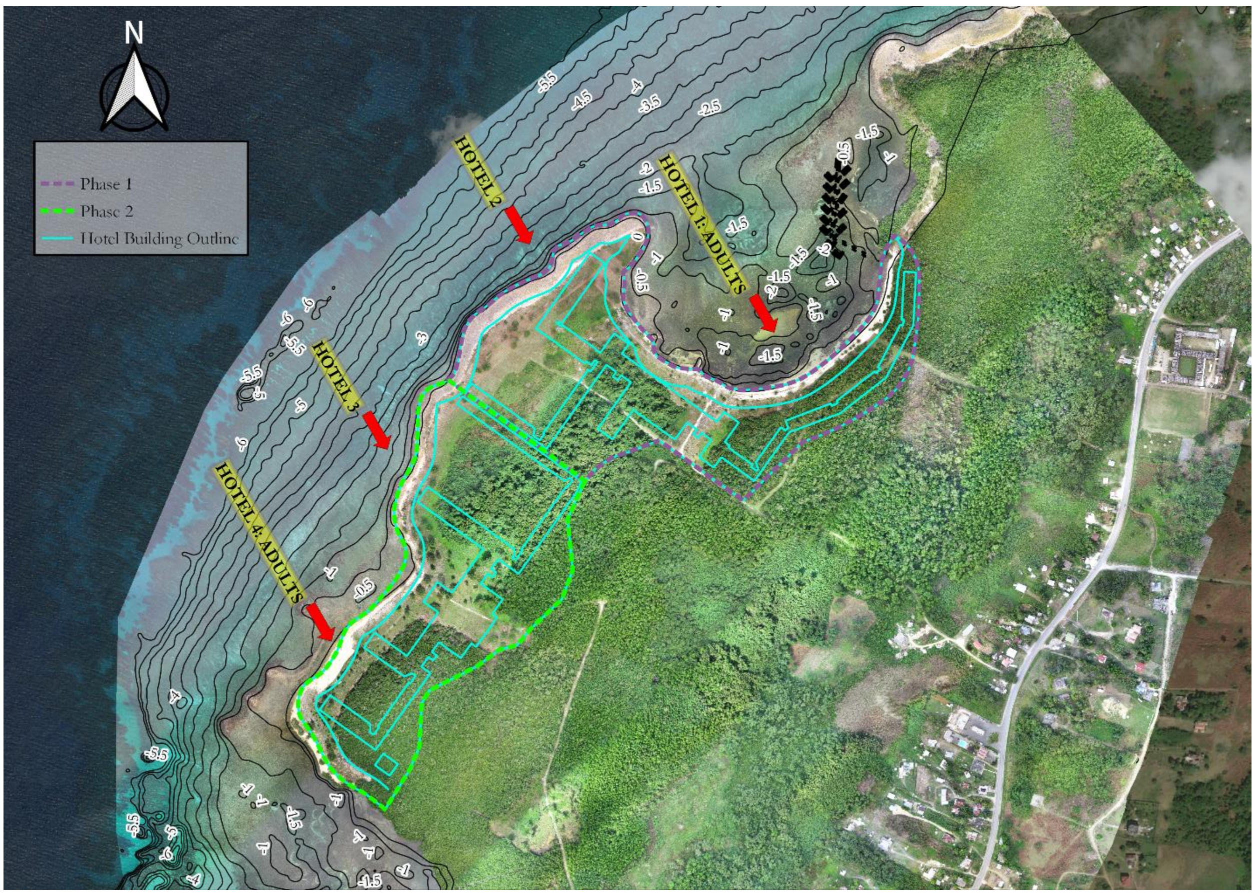


Figure 2.1 Location of the hotels along the Princess Resorts shoreline

The variations in shoreline are due to several factors: ambient wave conditions, topography of the foreshore; and the wetland that borders the site. This section of the report presents conditions observed at the site and the consideration involved in developing a shoreline solution for the Princess Resorts property.

2.1 Site Location

An overview of the site is presented in the Figure 2.2. The underlayer image was selected from Google Earth to show what occurs at the shoreline under high energy wave conditions. The image clearly shows bands of white where waves break as they approach the shoreline. There is a reef system at the entrance of the bay that causes significant wave breaking. Along the shoreline of Hotels 2 and 3 the white band is right on the shoreline. This indicate that the shoreline in this section is bombarded by large waves. At Hotel 4 there is an offshore reef system that creates a sheltered swimming area.

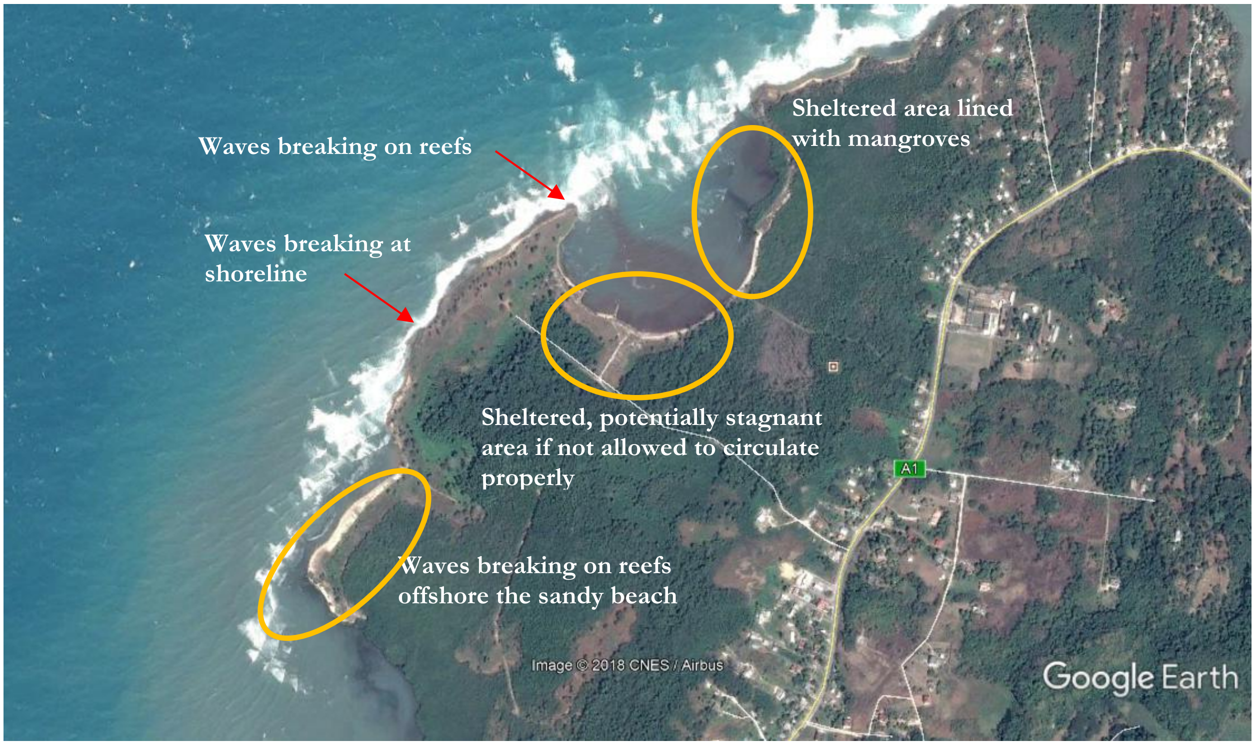


Figure 2.2 Aerial image of site showing waves breaking on reefs along the shoreline

2.2 Shoreline Characteristics

2.2.1 Hotel 1: Negro Bay, Green Island

Main observations regarding the shoreline in this area are as follows:

- There is a shallow pavement with reef system providing protection from waves that get to eastern section of the bay.
- The seafloor is covered with dense seagrass, suggesting the area is healthy environmentally.
- There is relatively low water circulation in this area, which can cause large amounts of organic matter to settle at the shoreline.

- The natural beach is eroding. The depth of water in front of the bay in less than 1.0m and has a **very gentle slope**. This suggests the need to protect this beach under daily conditions. It appears that tires were used in the past to try to stabilize this shoreline or at least to break the waves coming in.

Figure 2.3 Hotel 1 proposed location

- The shoreline has **two** manmade structures, both located in the cove beach. The two groynes create a calm area on both sides of the groynes, which contributes to poor circulation and results in the settling out of small sediments and silty material. The water is not very clear in this area.
- There is natural rocky headland at the west of the bay that contributes to the breaking of waves and creates currents that flow into the bay

Figure 2.4 shows areas inside the bay in photographs.

A: Looking to east from the east existing groyne showing the sandy beach and dense mangrove.
B: Looking to the west showing the western groyne
C: On the groyne
D: Sandy beach showing dark brown waves running up to the shoreline.
E: Seagrass and shallow depths from walking out to sea from the shoreline.

Figure 2.4 Photos taken inside the bay at Hotel 1

2.2.2 Hotels 2 and 3: Negro Bay

The main observations about shoreline in this area are as follows:

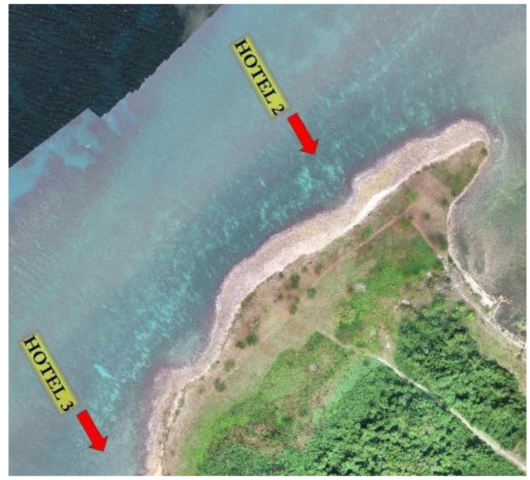
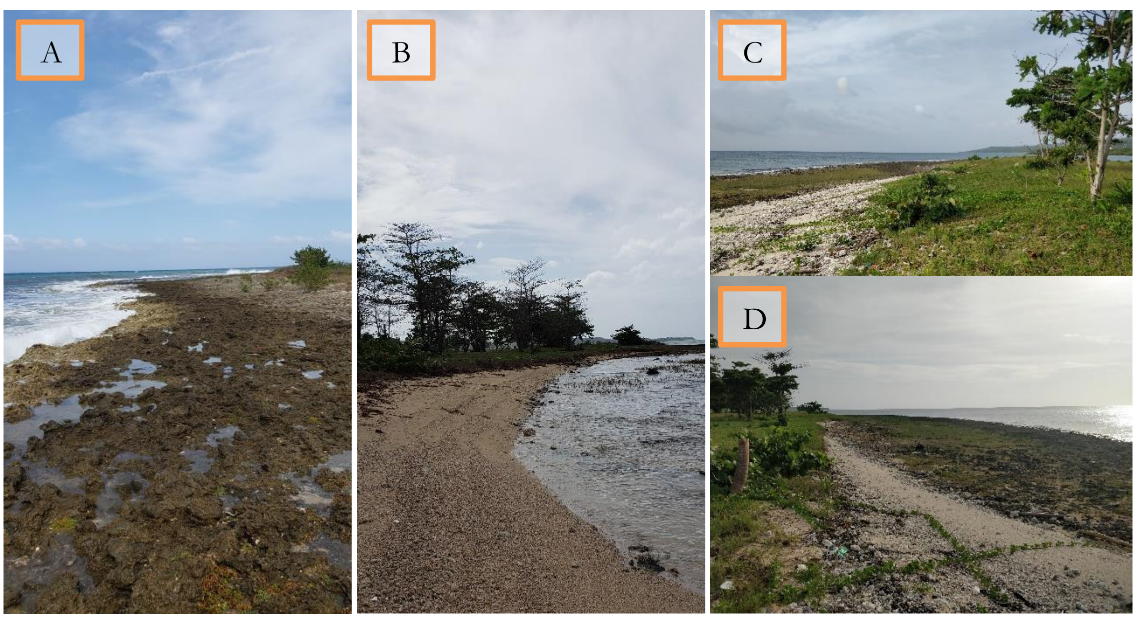


Figure 2.5 Shoreline at Hotels 2 and 3

- Conditions outside of the bay are aggressive: waves are high and currents are strong, contributing to the ongoing removal of sediments from the shoreline. As a result, the site is very rocky in this section.
- Along Hotel 2, the shoreline is rockier than the other section of shoreline. The rocky shore makes it difficult to walk and would not be suitable for a resort.
- Immediately seaward of the rocky shore the water gets deep quickly, which is indicative of a steeply sloping shore. A steep slope makes the wave breaking process more energetic and aggressive.
- Along some sections of the Hotel 3 shoreline there are sandy pockets as shown in Figure 2.6(b). These sandy pockets occur when the shoreline is indented. The depth of sand in this area is very shallow.
- The elevation of the backshore was notably higher than the shoreline. At Hotel 2, the backshore was the highest point along the entire shoreline.



A: Looking east showing sharp rocky outcrop.
B: Looking west showing sandy pockets.
C and D: Higher elevation above the shoreline.

Figure 2.6 Hotels 2 and 3

2.2.3 Hotel 4: Negro Bay

The points below highlight the main observations about shoreline in this area.

- This section has a sandy beach. The reef system in this section protects the beach, however the area is very shallow. It is possible to walk from the shore to the reef system without water passing knee height.
- If this area is to be used as a beach that supports swimming or wading, the seabed would have to be deepened.
- The seafloor in front of the beach at the south-west is densely vegetated with seagrass. Several large corals are also in the nearshore of the beach.
- The best approach to create a swimming area in this location is to move the beach inland.

A: View looking to east showing sandy beach.
B: View looing to the west showing scarping due to erosion.
C: At southwestern most point of the site showing a sandy beach with a gentle sloping shoreline.

Figure 2.7 Hotel 4 site

2.3 Bathymetry and Topography

Topography and bathymetry are the starting points to understanding coastal processes in the area and they are the major boundary condition for geometric and structural designs. This section describes the offshore (deep water) bathymetry, the nearshore bathymetry (from boat surveys), and finally the terrestrial elevation from profiles and digital elevation model (DEM) data.

2.3.1 Data Sources

Topography and bathymetry data were collected from four sources for this project. These sources are discussed below. All nearshore data was combined and is presented in Figure 2.8.

1. Client supplied topography (data for the mangrove forest)
A topographic survey of the mangrove forest was conducted by Andre Fiffe using a theodolite survey.
2. Boat-based bathymetry and beach profiles
The survey was done using an echo-sounder and GPS device aboard a small boat. The Odom EchoTrac sounding device was mounted in an over-the-side configuration and was used to provide the water depth information. While the echo sounder recorded water depths, the Trimble RTK GPS simultaneously recorded spatial coordinates for each reading. As the boat traversed the nearshore area to create bathymetric tracks the measurements were automatically stored to a field computer. The measurements were converted to the national coordinate system and depths were referenced to mean sea level (MSL). The reduced depths and positions were then used to aid in the generation of contours for the nearshore of the project site. Using a theodolite, a total of 41 beach profiles were done along the shoreline of the Princess Resorts. The survey measures the depths to which a person could comfortably walk out to sea.
3. Digital Elevation Models (DEMs) from Unmanned Aerial Vehicle (UAV)
An Unmanned Aerial Vehicle (UAV) was used to collect data of the ground elevation of sections of the bay. The data was limited to only areas where there was a clearing in the mangrove forest.
4. Satellite-derived bathymetry from Danish Hydraulics Institute (DHI) and Earth Observation and Environmental Services (EOMAP)
Data obtained from DHI was at a 90m x 90m spacing. This data set extended over 8km seaward of the site. More refined data was also collected from EOMAP that had bathymetric data in a 1x1m spacing. The collected bathymetric data was used to validate the EOMAP satellite data.

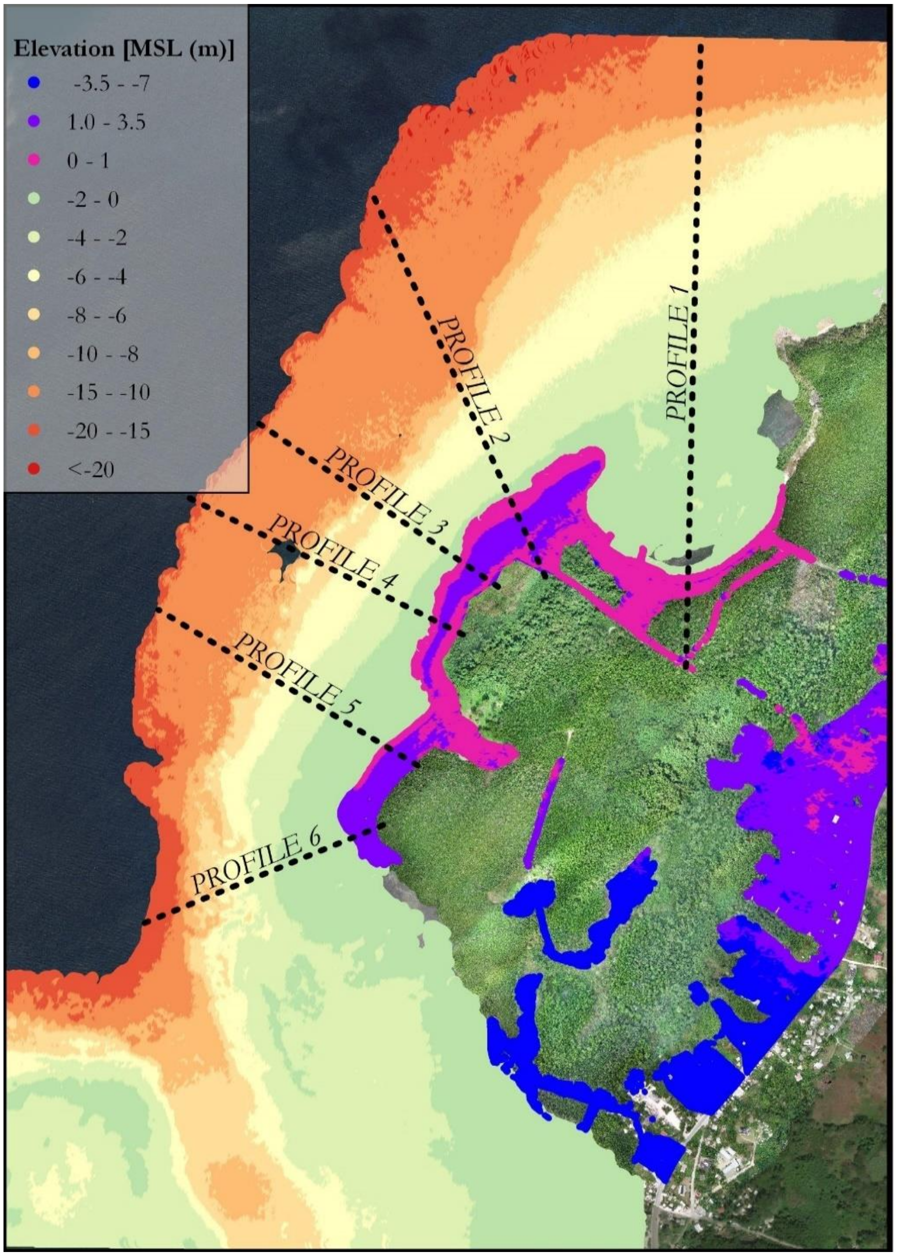


Figure 2.8 Collation of nearshore and terrestrial elevation points referenced to mean sea level (MSL)

2.3.2 Results of Topographic and Bathymetric Assessment

Hotel 1

There is a clearly a reef system offshore of Hotel 1 (20-30m wide). Some sections of the reef are only submerged by approximately 0.5m of water. At Hotel 1, the backshore does not go above 1.5m MSL. A typical 1 in 50-year hurricane usually has a static surge greater than 1m. It can therefore be assumed this site can easily become inundated under hurricane conditions.

Hotels 2 and 3

Along this section of the bay the backshore is above 2.5m. This section of the shore could be protected from storm-related flooding, but the low landward section could still face flooding.

Hotel 4

Along Hotel 4, the nearshore is only about 0.5m deep. Further out, water depths increase significantly. Like Hotel 1, this section of the site would also be submerged under the 1 in 50-year hurricane condition.

Figure 2.9 shows profile survey results along various sections of the site shoreline.

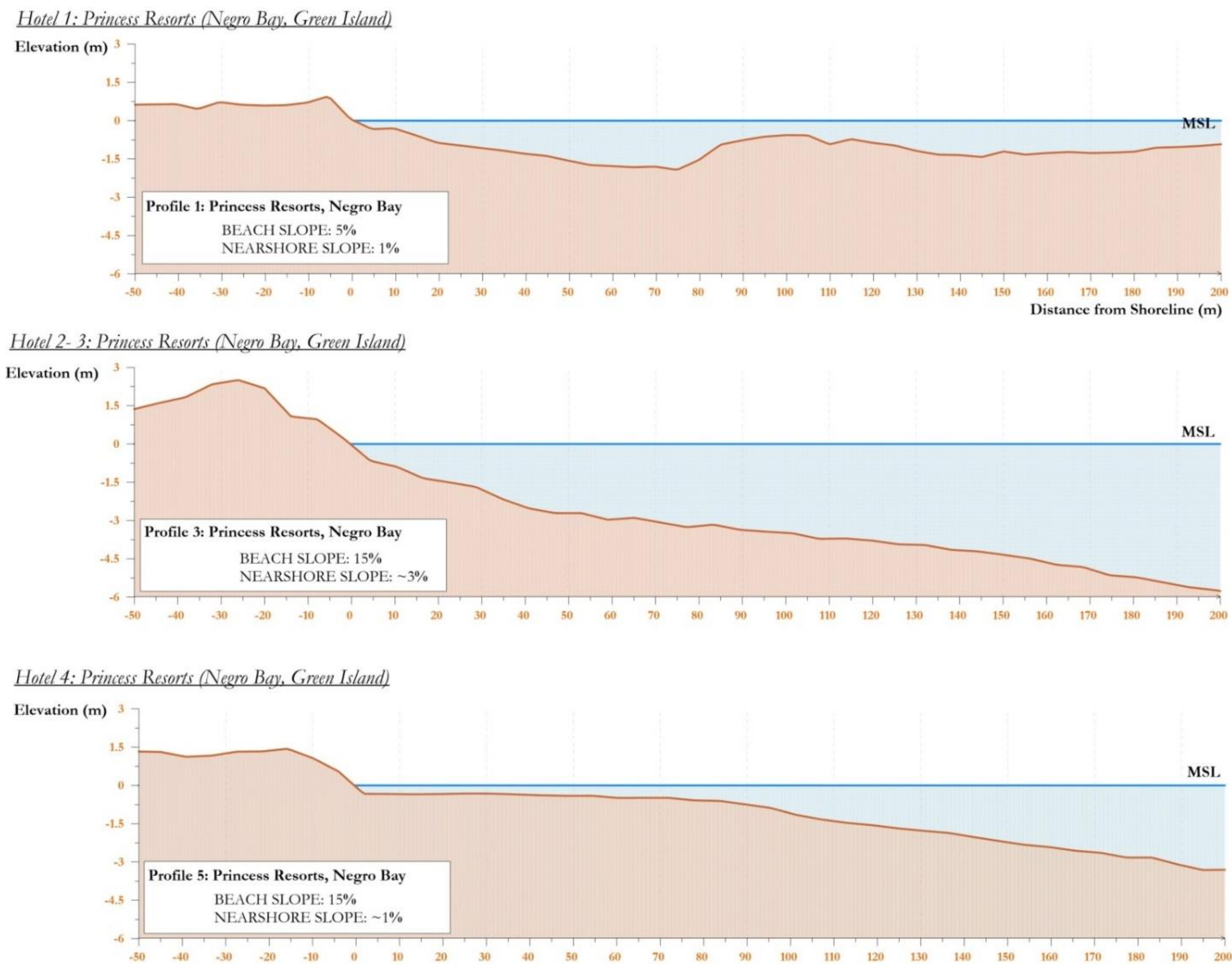


Figure 2.9 Typical sections along the shoreline of Princess Resorts Hotels

2.4 Sedimentology and Seafloor Conditions

2.4.1 Sieve Analysis

We collected five sand samples from the beach and performed sieve analysis on each sample. All samples were light brown coralline sand indicating high calcium content. Samples 1, 3, and 5 are suitable for use on the beach but there is only a small amount of sand available: results of the sand probing exercise indicate only 0.6m of sand is available in these areas. Figure 2.10 shows where samples were collected and Table 2-1 summarises the sieve analysis results.

Figure 2.10 Locations of sand samples

Table 2-1 Sediment sample analysis results

Sample	Mean Grain Size (mm)	Description	Percentage Silt (%)	Uniformity Coefficient	Standard Deviation
H1 – EAST	0.656	*Coarse Sand* Well Sorted	**0**	1.777	0.404
H1 – WEST	0.315	*Medium sand* Moderately Sorted	0	2.383	0.765
H2	0.722	*Coarse Sand* Well Sorted	0.11%	1.710	0.403
H3	1.165	*Very Coarse Sand* Moderately Well Sorted	0	2.123	0.654
H4	0.979	*Coarse Sand* Moderately Sorted	0	1.971	0.865

2.4.2 Hydraulic Probing

We used a water jet to probe the sea bottom at several locations along the shoreline and in the nearshore. This procedure determines the depth of silty or sandy material below the seabed. The probe is pushed into the seabed until it reaches hard/rocky material. The investigation informs areas that can be easily deepened and, in some cases, potential sources of sand. It also gives a preliminary indication of the kind of foundation possible for marine structures, especially for piling. The probing exercise led to the following conclusions:

- The eastern end of the cove has a thick layer of loose sediment (probably silt). In this area, currents are low and therefore fine sediments settle.
- Eastward the probe went 0.6-1.5m before hitting hard material. This suggests that piling will encounter a hard layer just below the seabed. Detailed geotechnical borings will be necessary.
- Along the sandy beach, loose sediment is 0.6-1.8m thick. This is a suitable amount of sand for a resort however the beach is quite narrow. More sand will be required to widen the beach.
- Moving towards the west end of the property (Hotels 2-4), the shoreline gets harder with less than 0.6m of loose sediment. These areas will have to be excavated and filled with suitable sand. Unless sand can be found landward of the beach, the sand needed for nourishing the proposed beaches will have to be sourced externally.

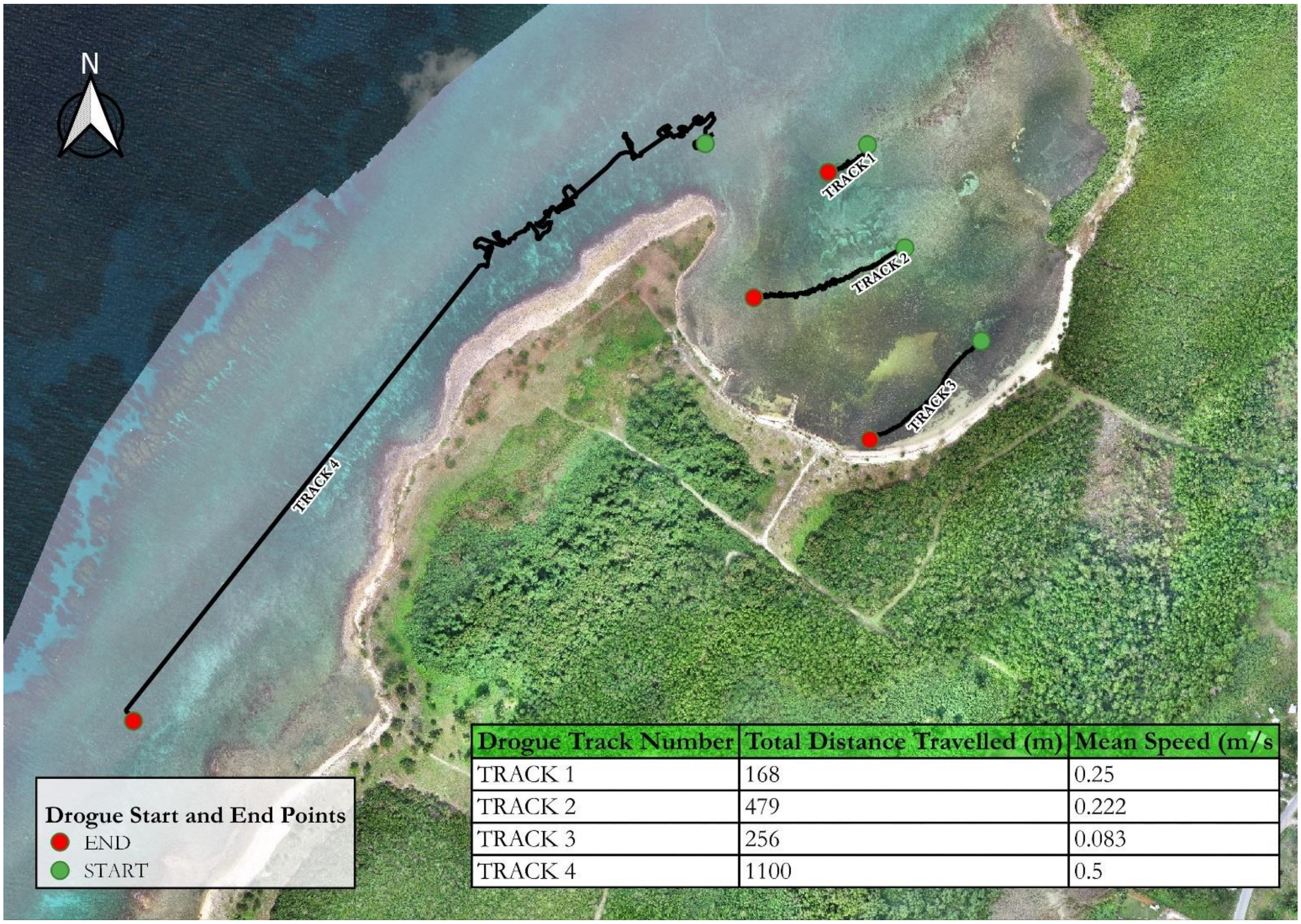
Figure 2.11 shows locations and results of the sand probing exercise.

Figure 2.11 Probe Depths

2.5 Current Measurements

Four drogue tracks were done for the area. This involves placing a submerged floating device in the sea and tracking it with an onboard GPS. This is a measurement of the surface currents in the area. The drogues were deployed at four different locations within the cove (Figure 2.12) and all moved toward the west as expected. All the drogues deployed inside the cove ran into the shoreline and stopped. This indicates is a tendency for any floating material to be brought into this area.

This also implies that the current moves slowly and has contributed to the deposition of debris such as black/brown dead seagrass. The currents in these areas will have to be improved via the proposed channel. **Track 4**, which is outside of the bay, showed current speeds more than 50% higher than currents in the bay. Creation of a channel connecting the cove to this current could help with reducing stagnation in the bay.



Drogue Track Number	Total Distance Travelled (m)	Mean Speed (m/s
TRACK 1	168	0.25
TRACK 2	479	0.222
TRACK 3	256	0.083
TRACK 4	1100	0.5

Figure 2.12 Current measurement with the drogue

3 Biological Site Conditions

This section describes baseline biological conditions at the site. The site is a known wetland with an extensive mangrove forest; a summary of stable of mangroves is therefore presented. Marine conditions are then presented along with water quality from the area.

3.1 Terrestrial Ecology

The proposed site can be described as a wetland floodplain and drainage collection area for major storm drains along the Green Island to Negril "Highway" (Figure 3.1). The mature mangrove forest system has a traditional and expected Caribbean mangrove forest tree zonation and a high presence of mangrove and golden ferns (*Acrostichum aureum*; regarded as mangrove plants worldwide). The forest is interspersed with other emergent wetland vegetation (Typha sp, Dalbergia sp, Spartina sp.), with these species replacing the true mangrove species (Red, Black and White mangroves).

Figure 3.1 Conditions in the mangrove wetland

The site displayed various important ecological traits that are expected of a large wetland system. These traits included but are not limited to:

- Habitat for birds, reptiles, crustaceans, molluscs, etc.;
- Nutrient filter and water absorption;
- Spawning/breeding and nesting grounds for birds and fishes;
- Habitat for numerous juvenile creatures, especially reef fishes of commercial importance;
- Buffer to coastal energy (storms, high wave energy).

Despite the impressive occurrence of large undisturbed areas of mangrove forest, the site shows strong evidence of human disturbance and fragmentation. Based on satellite images, this work occurred over 10 years prior to the site visits and does not seriously hinder the wetlands ecological functions and shows minimal habitat fragmentation as the disturbances were of a small scale and localized in some sections Figure 3.2).

Figure 3.2 Time-lapse of aerial photos at the site

The following points can be made about the implications for coastal design and drainage design for the area:

- The topographic surveys provide ideal supporting evidence for the theory that roadways act as physical impediments for tidal influence of the whole forest. Several culverts need to be implemented to improve the flow in the mangrove forest.
- A tidal inlet was identified to the north-east of the site. This inlet needs to be enhanced to ensure that the flow of brackish water is continued.

3.2 Benthic Resources and Fish Population

To assess the benthic resources in the area, rapid benthic and fish surveys were done. The detailed report is presented in Appendix B. A modified Reef Check® Method was used to assess the substrate in the survey area and a modified Visual Fish Census of **A**tlantic and **G**ulf **R**apid **R**eef **A**ssessment (AGRRA) Protocol was used to understand the fish population of the area.

3.2.1 Benthic Survey

Modifications to this method were made and these included the following:

- Only the Substrate transect was executed. No Fish or Invertebrate transects were conducted.
- The category Sponge (SP) was replaced by Seagrass (SG). Seagrass is usually recorded under other (OT) but was disaggregated due to sizeable presence at this site.
- Figure 3.3 shows the transect lines used within survey area Section A. Substrate type was identified and recorded at 0.5m intervals along each transect line. Substrate types were categorized as:

Hard Coral (HC)	Rock (RC)
Soft Coral (SC)	Rubble (RB)
Recently Killed Coral (RKC)	Sand (SD)
Macroalgae (MA – including Nutrient Indicating Algae)	Silt/clay (SI)
Seagrass (SG)	and Other (OT)

Figure 3.3 Survey area and transects

Hotel 1

The composition of the substrate along this site is varied. The Eastern Bay (at **Hotel 1**) was dominated by seagrass cover (Figure 3.4 and Figure 3.5).

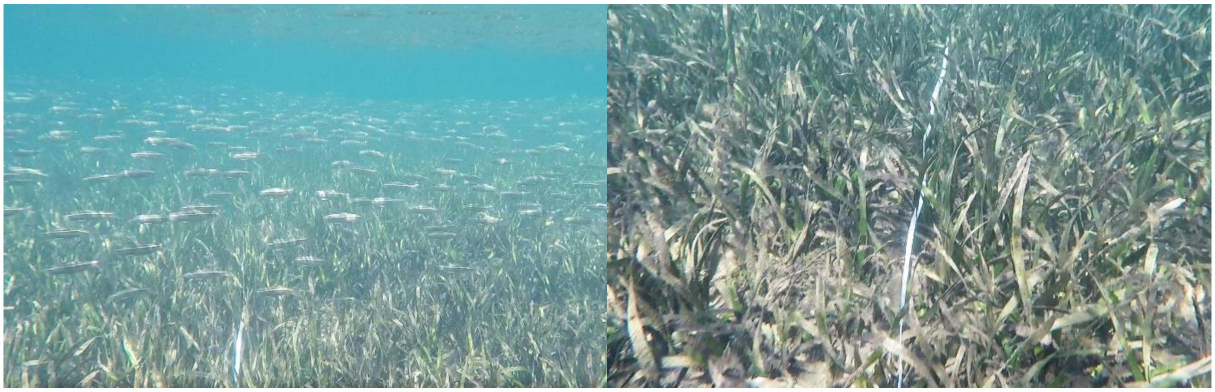


Figure 3.4 Example of substrate observed along Line 1

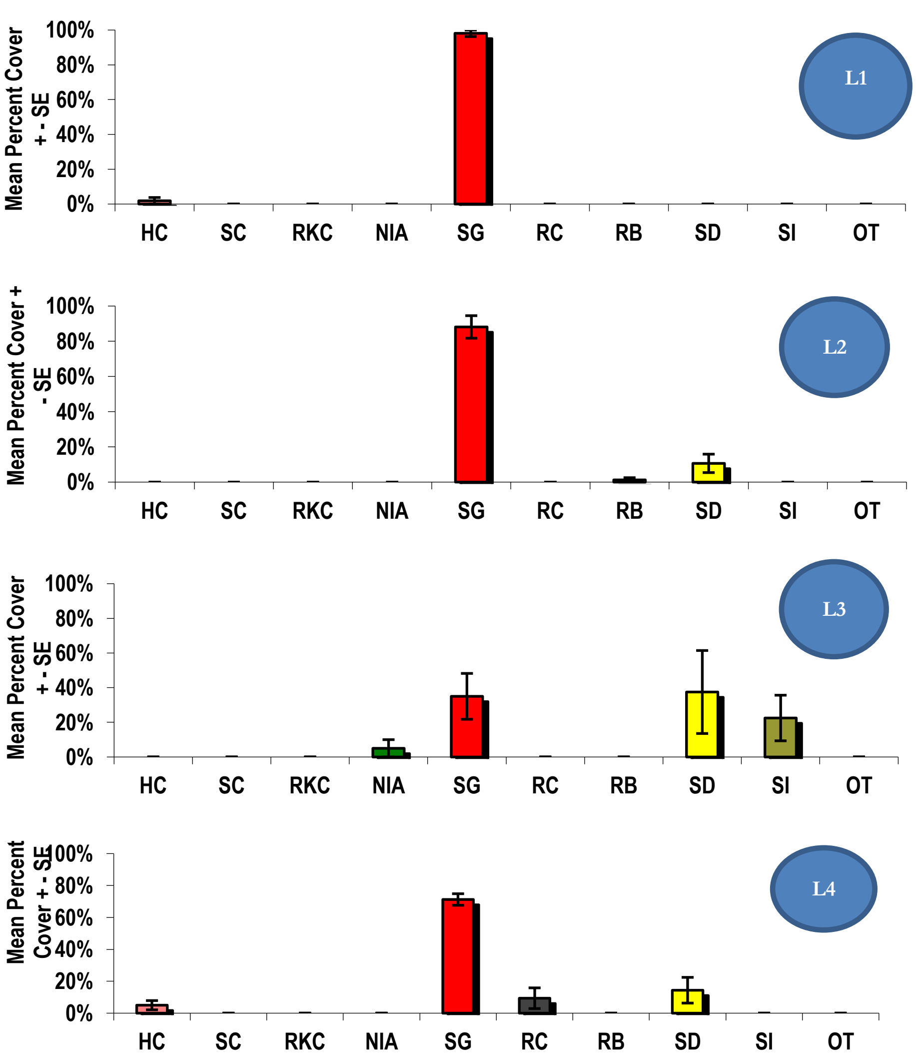


Figure 3.5 Percentage coverage of seafloor along Line 1 – 4 within the bay

Hotels 2 and 3

Along the central section of the site (**Hotels 2 and 3**), the substrate was characterised by limestone rock/pavement and corals (Figure 3.6 and Figure 3.7).

Figure 3.6 Example of substrate at Line 6

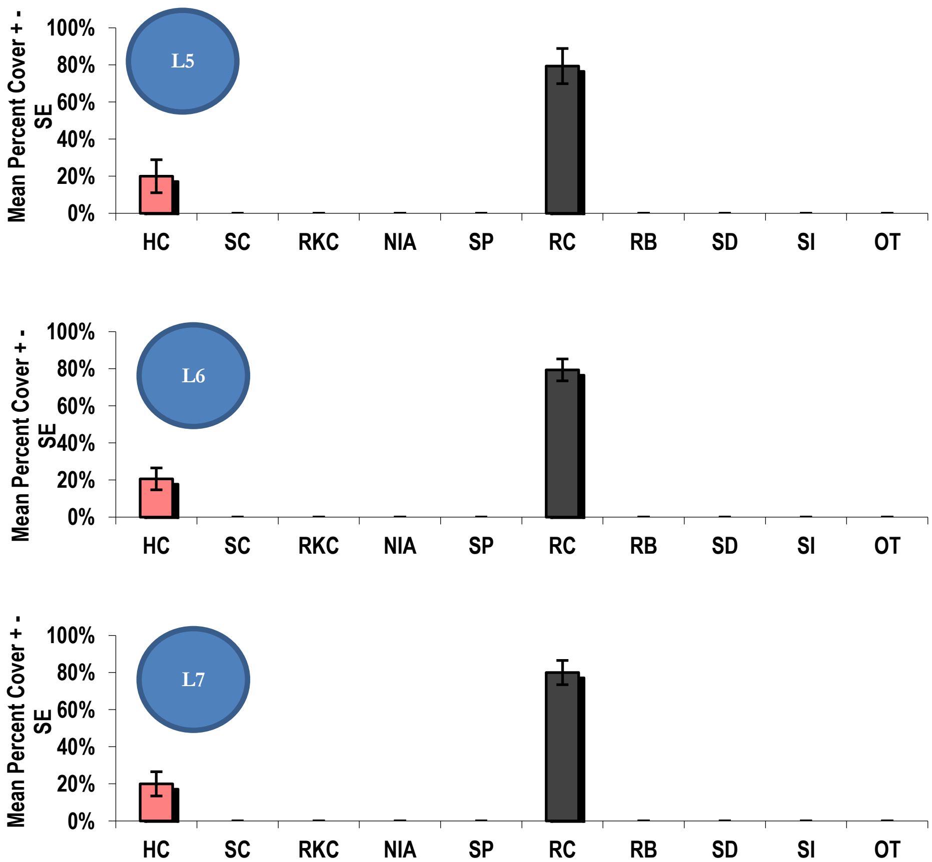


Figure 3.7 Percentage coverage of seafloor along Lines 5 – 7 along the Ironshore

Hotel 4

The Western section (**Hotel 4**) of the site was characterised by a mix of limestone pavement and seagrass (Figure 3.8).

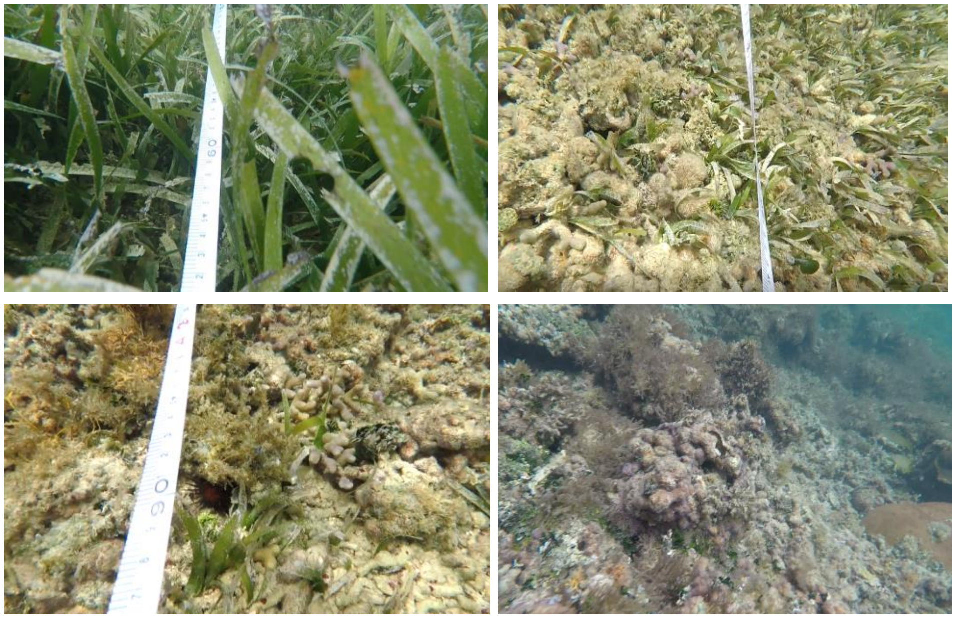


Figure 3.8 Example of substrate along Line 8

Summary and Implications for Coastal Design

Figure 3.9 shows the composition of the seafloor according to the benthic survey. The ecosystem is represented by well-developed and ecologically significant marine resources (reefs and seagrass meadows). Seagrass meadows are essential coastal ecosystems that provide many ecosystem services, such as improved water quality and light availability, increases in biodiversity and habitat, and sediment stabilization. Seagrass beds are also highly productive habitats that provide important ecosystem services in the coastal zone, including carbon and nutrient sequestration, therefore acting as a carbon sink.

Impacts on marine resources could include excessive sedimentation during the nourishment/ construction works and settling of spoil, physical damage from heavy equipment on site, and loss/disruption of habitat. Corals, seagrasses and other valued ecosystem components are present within the footprint of the proposed works, and it will be important to mitigate potential impacts.

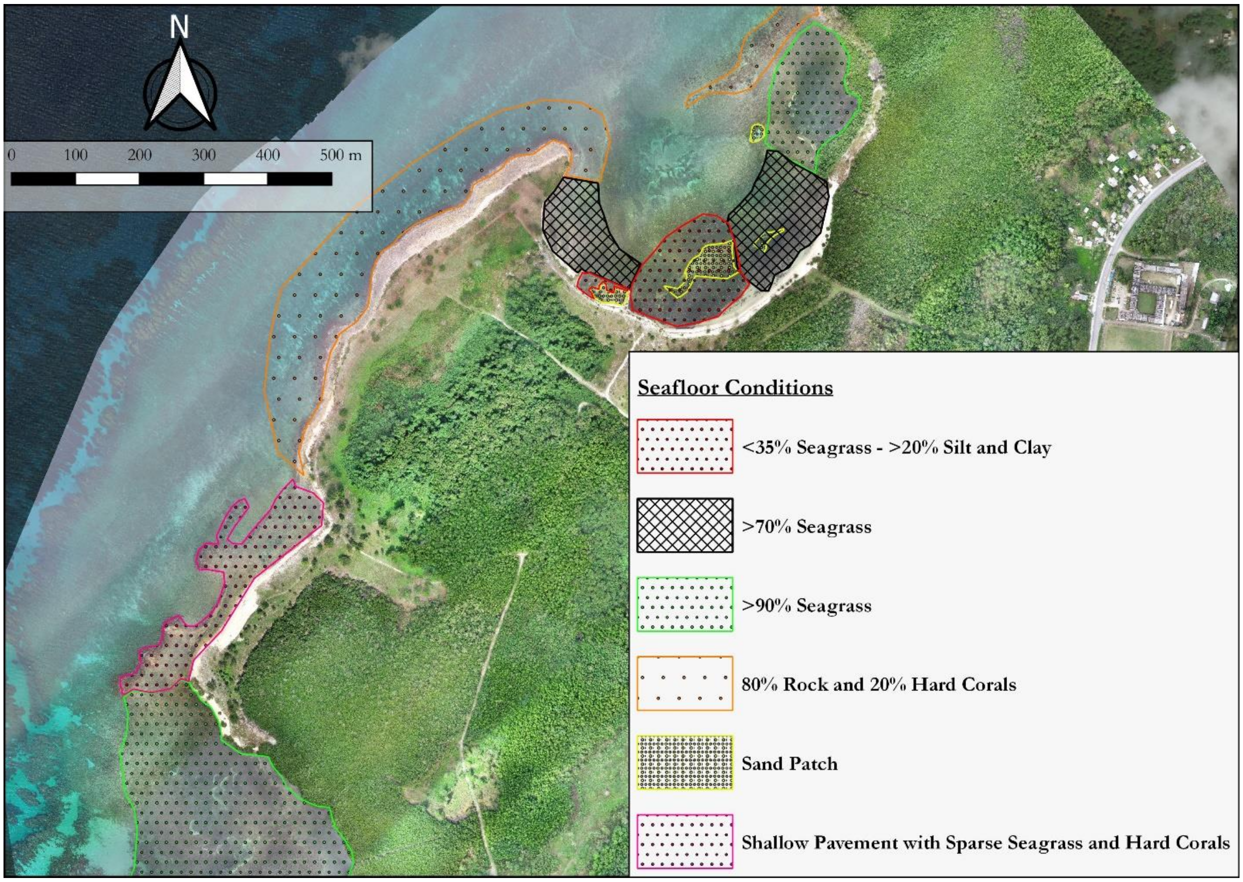


Figure 3.9 Distribution of benthic resources in the nearshore of the project site

3.2.2 Visual Fish Surveys

A modified Visual Fish Census of **A**tlantic and **G**ulf **R**apid **R**eef **A**ssessment (AGRRA) Protocol was employed to capture fish data (presence/absence and frequency). All fish observed were identified and given a frequency rating (based on occurrence) of Single (S = single individual), Few (F = 2-10 individuals), Many (M = 11-100 individuals), or Abundant (A = >100 individuals).

The fish population observed throughout the entire area was relatively low both in total number of fish and number of fish per species. The reef fish observed were typical and small in size (5-10cm size class). Twenty-one species of fish were observed. The full benthic assessment is attached as Appendix B.

3.3 Water Quality

In April 2019, we collected five water samples that were sent to the lab to assess six parameters to show the quality of the water in the marine environment. The analysis indicated good water quality on a typical day (no rainfall) and relatively calm wave conditions. This is not surprising considering the filtering provided by the mangrove/wetland forest. This justifies the need to minimize direct drainage from the resort to the marine environment. Figure 3.10 shows where the water samples were collected and Table 3-1 provides the results of the water quality analyses.

Figure 3.10 Water quality sampling locations

Table 3-1 Water quality sampling results

Sample	Phosphate (Mg PO_4^{3-}/L)	Nitrate as Nitrogen (NO_3^- N/L)	Biochemical Oxygen Demand (Mg O_2/L)	Total Suspended Solids (Mg/L)	Faecal Coliform (MPN/100ml)	Enterococci (MPN/100ml)
S2 – 1013	<0.02	<0.01	1.7	8.3	2	<1.8
S2 – 1015	<0.02	<0.01	0.9	7.5	<1.8	<1.8
S2 – 1017	<0.02	<0.01	1.1	6.1	<1.8	<1.8
S2 – 1019	<0.02	<0.01	0.8	4.8	<1.8	<1.8
S2 – 1021	<0.02	<0.01	0.6	7.1	<1.8	<1.8

4 Socio-Economic Context

This section of the report presents the social and economic context of the project. It describes the community around the site, the economy of the area, and the legal framework relating to work in the marine environment.

4.1 Stakeholders and Potential Concerns

Potential concerns of direct and indirect stakeholders in the area are summarised as follows.

- The main stakeholders along the coastal area are fishermen. Within the bay to the west of the site there are currently several aquaculture farms. Additionally, the area is known to be frequented by spear fishermen who sometimes use the property's beaches. While the resort cannot stop the use of the beach by locals, access to the land would be restricted as it is private land.
- Other stakeholders for this project are the members of the Green Island community. Green Island has an estimated population of 3100 living in an estimated 1100 households. According to the Social Development Commission[1], the community has high levels of youth and adult unemployment, high levels of high school dropouts and low levels of literacy and numeracy. It is expected, therefore, that community stakeholders would be interested in the development of the area. However, the growth of the community could lead to the development of informal settlements nearby. From discussion with the locals in the area, the beach is not used for recreation.
- The area also has several small businesses set up along the roadway. These include shops, stalls, gas stations, and restaurants. Potential concerns are related to the development of a hotel in the area.
- Indirect stakeholders may also include:
  - Forestry Department
  - Fisheries Division
  - NRCA/NEPA
  - Negril Environmental Protection Trust
  - Jamaica Environmental Trust
  - National Works Agency
  - Green Island High School and other educational institutions

The area has high environmental value. The site has developed mangroves, seagrass and corals. Therefore, the major concern is the lack of support for the project from environmental groups with an interest in the area.

4.2 Regulatory Context

This section presents the legal framework for working within the Green Island Area.

[1] https://sdc.gov.jm/communities/green-island/

4.2.1 Natural Resources Conservation Authority Act 1991

The Natural Resources Conservation Authority Act provides for the management, conservation and protection of the natural resources of Jamaica. The functions of the NRCA include ensuring the effective management of the physical environment, which includes the establishment of marine parks and protected areas.

NRCA Act 1991 – Marine Park Regulations

The NRCA Act of 1991 stipulates the regulations for marine parks. Pertinent sections of the regulations under the Natural Resources Conservation Authority Act 1992 (under Section 38) are summarized below.

Table 4-1 Pertinent regulations from the NCRA Act 1992

Regulation	Extract from the NRCA Act 1991
No mining for minerals within the marine park	*A person shall not, except with the written permission of the Authority and in accordance with the provisions of a licence or permit granted under any other enactment, carry out any operation for the extraction or mining of minerals in a marine park.*
No removal or destruction of natural features and marine life	*A person shall not* *(a) destroy, injure, deface, move, dig, harmfully disturb or remove from a marine park any sand, gravel or minerals, corals, sea fans, shells, shellfish, starfish or other marine invertebrates, seaweeds, grasses, or any soil, rock, artifacts, stones or other materials;* *(b) cut, carve, injure, mutilate, move, displace or break off any bottom formation or growth;* *(c) attach any rope, wire or other contrivance to any coral, rock or other formation, whether temporary or permanent in character or use;* *(d) use, sell or otherwise dispose of any seaweed, coral, mineral, gravel, sand or other substance or thing, knowing it to have been stolen or unlawfully removed from a marine park.*
No dredging or filling of marine areas	*A person shall not in a marine park* *(a) dredge, excavate or carry out any filling operations or deposit any material in the waters thereof; or* *(b) erect any building or other structure or any public service facility, without the written permission of the Authority.*
No discharge of pollutants in the marine area	*A person shall not discharge or deposit in or on the waters of a marine park any refuse, oily liquids or wastes, acids or other deleterious chemicals or any toxic or polluting substance of any kind injurious to plant or animal life.*

4.2.2 The Beach Control Act 1956

This Act was passed in 1956 to ensure the proper management of Jamaica's coastal and marine resources by means of a licensing system. This system regulates the use of the foreshore and the floor of the sea. The Act speaks to other issues including access to the shoreline, rights related to fishing and public recreation and establishment of marine protected areas. Under section 5 of this act, it is an offence to encroach on the foreshore or floor of the sea for a public or commercial purpose without a license. Key sections of the Act are presented below:

> *"5 (1) From and after the 1st June, 1956, no person shall encroach on or use, or permit any encroachment on or use of, the foreshore or the floor of the sea for any public purpose or for or in connection with any trade or business, or commercial enterprise, or in any other manner (whether similar to the foregoing or not) except as provided by sections 3, 4 and 8, without a licence granted under this Act."*

> *"6 (1) Where at the 1st June, 1956, any person is encroaching on or using or permitting any encroachment on or user of the foreshore or the floor of the sea except as authorized by this Act, such person may continue or*

may continue to permit such encroachment or user for a period not exceeding six months after the 1st June, 1956, but such person shall, if he intends to continue or to permit the continuance of such encroachment or user for any longer period, apply to the Authority for a licence under this Act within the aforesaid period of six months."

"9.-(1) Subject to the provisions of section 8, no person shall erect, construct or maintain any dock, wharf, pier or jetty on the foreshore or the floor of the sea, or any structure, apparatus or equipment pertaining to any dock, wharf, pier or jetty and encroaching on the foreshore or the floor of the sea, except under the authority of a licence granted by the Minister on behalf of the Crown."

4.2.3 Other Related Acts

The following are other acts that may be pertinent to the development of a resort in Green Island:

- The Town and Country Planning Authority (TCPA) Act of 1957;
- The Watersheds Protection Act of 1963; and
- The Wildlife Protection Act of 1945.

The project site is located within the Negril Environmental Protection Area and the Negril Marine Park. The Negril Environmental Protection Area was established in November 1997 and the Negril Marine Park in March 1998. The protected area is just over 37,100 hectares and includes a large watershed area. The regulations as outlined in the Regulatory Context above are therefore in effect for the project site.

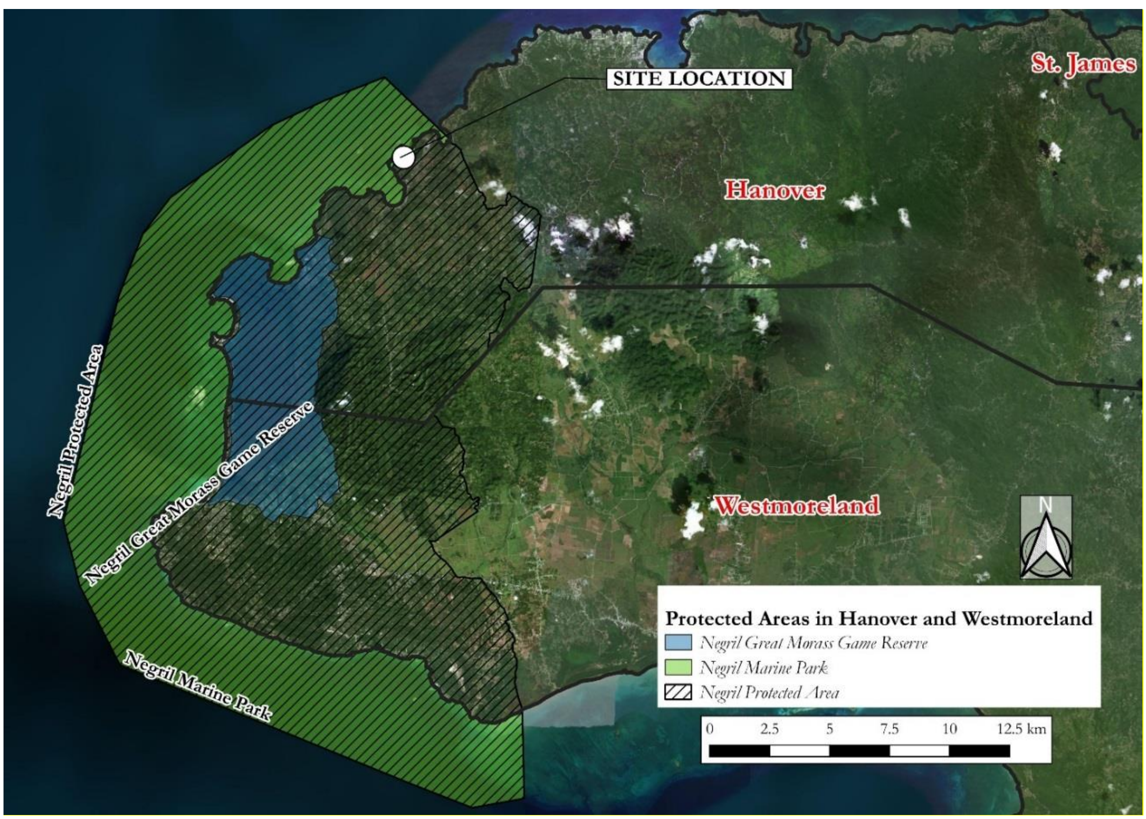


Figure 4.1 Conservation and protected areas in Hanover and Westmoreland

4.3 Socio-Economic Impacts

The Gleaner reported that the Princess Resorts chain intends to invest USD150-500 million in the construction of a 2000-room hotel. This investment would add growth to the community of Green Island and, by extension, Jamaica. The coastal enhancement at the site will also benefit the site and the wider community. During construction, employment opportunities will increase as will the circulation of money in area. Further impacts are shown in the figure below.

Current conditions	During construction	Future stable beach
Low property value	Local employment opportunities	Increased property values
Limited space for recreational activity for the Green Island community	Limited beach access for fishermen	Added recreational space
Low levels of employment	Noise and dust	Added appeal to investors
Few opportunities for public integration		Positive downstream taxes and fees from property sales
		Increased potential for tourist and local use
		Contribution to local economies (e.g. craft and food)

Figure 4.2 Socio-economic impacts at different stages of implementation

4.4 Climate Change

Climate change is defined as a change in the state of the climate that can be identified (e.g. using statistical tests) by changes in the mean and/or the variability of its properties, and that persists for an extended period, typically decades or longer. It also refers to any change in climate over time, whether due to natural variability or as a result of human activity.[2]

Main components of climate change such as rising sea levels and increased storminess may prove damaging for small low-lying developing countries like Jamaica. Vulnerabilities that may be further exacerbated include:

- Increased storm surge and coastal erosion from more intense hurricane activity;
- Long-term shoreline erosion from higher waves due to higher sea levels;
- Changes in trends of shoreline morphology.

Designing with climate change in mind presents some difficulties because most guidelines are projections that may change in the coming years. Guidelines have been summarized in the *Inter-Governmental Panel on Climate Change (IPCC) Summary for Policy Makers* publication. The main section that is applied in our coastal work is the projection for mean global sea level rise. The IPCC RCP8.5 guideline was chosen as it represents the highest level of radioactive forcing by the year 2100. The global mean increase in sea level is projected to be 0.63m.

[2] Intergovernmental Panel on Climate Changes (IPCC) usage

5 Baseline Coastal Processes

Baseline coastal zone modelling is required to gain an understanding of the coastal processes acting along the shoreline of the project site. Waves, currents and sediments all interact to affect shoreline morphology, such as erosion or accretion. This section of the report describes the additional work done to enhance the previous work.

Coastal hazards that affect the coast include flooding from storm surge due to hurricanes, and chronic shoreline erosion from daily and swell waves. Shorelines may show signs of dynamism, as they build up during the summer months when the conditions are calmer and erode during the winter months when the wave conditions are stronger due to ocean swells. To understand the daily wave climate and storm surge potential of the area, detailed numerical modelling was carried out and the results are presented in this section. Potential erosion at the site due to swells and hurricanes is also presented.

5.1 Numerical Model Domain

MIKE21, a coupled wave-hydrodynamic-sediment model developed by the Danish Hydraulic Institute, was set up for this project area. The model was used to transform wind and wave fields from deep water to the nearshore at the project site and to determine both operational wave conditions and hurricane wave conditions, including storm surge and inundation. Figure 5.1 shows the MIKE21 flexible mesh representing the seabed that used for the modeling.

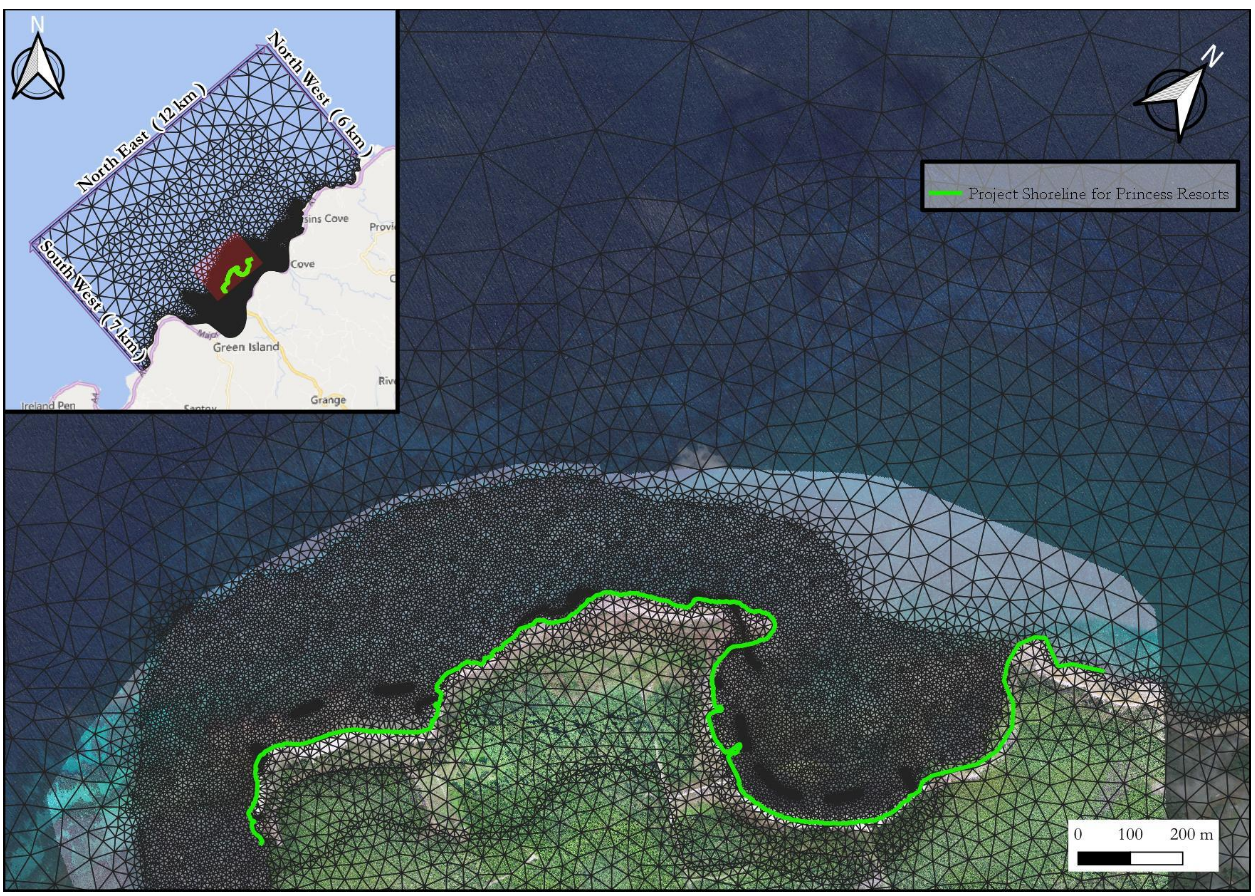


Figure 5.1 Numerical model domain

5.2 Summary of Model Validation

Before beginning the numerical modeling, the first step is to validate its performance by comparing it to actual measurements in the field. We tested the model to see if it reflected the results obtained from the drogue tracking. We used the MIKE21 Transport module combined with the Wave (SW) and Hydrodynamic (HD) modules. This set-up shows what would happen to a dispersive pollutant introduced into the sea. We treated the floats as pollutants and introduced them to the model at the same time it was deployed during the actual field investigation. The black line shows what happened in the field and the red is what the model predicted, which is a very close match.

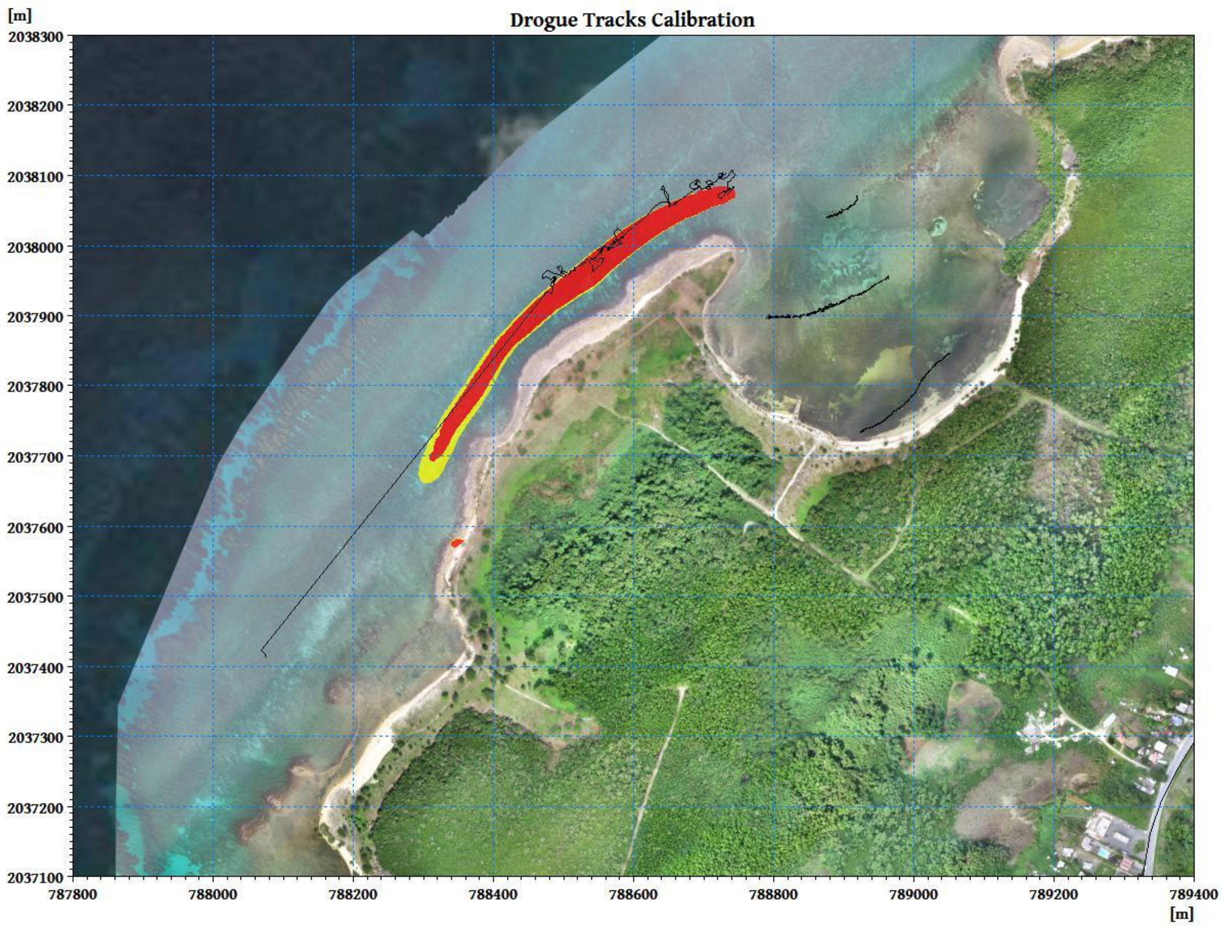


Figure 5.2 Pollutant track overlain on drogue tracks

The model shows the introduced pollutant moving in the same direction as the drogue (Figure 5.3). The speed of the drogues was also close to the modelled currents. Track 4 had a mean speed of ~0.5m/s while the model predicted a mean current speed of approximately 0.43m/s. This validation implies that the model can reliably predict current speeds and directions for the area and is therefore suitable for the investigations to be carried out.

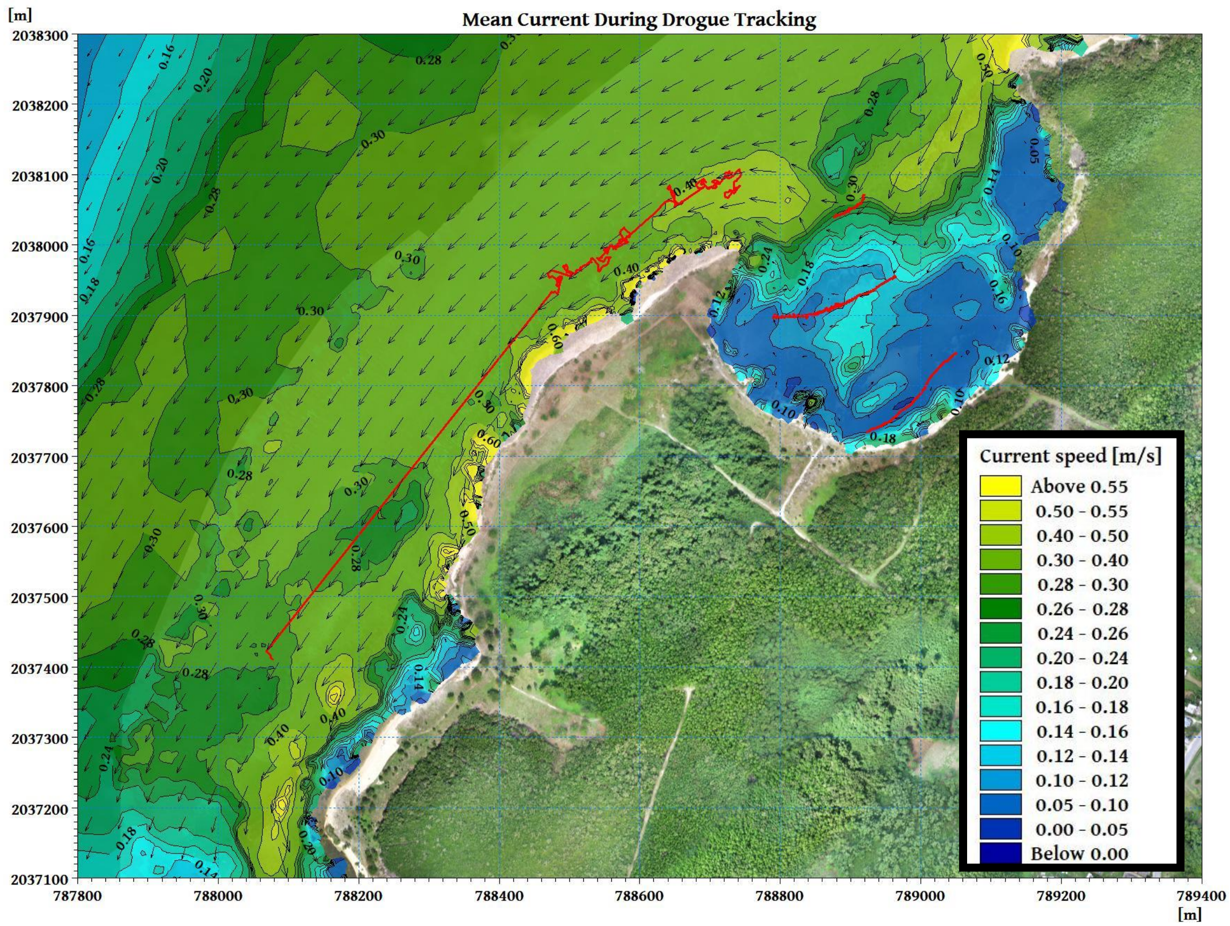


Figure 5.3 Mean current speed during the drogue measurement period

5.3 Offshore Wave Climate

The operational wave climate at the project site is characterized by (a) day-to-day, relatively calm conditions and (b) seasonal winter swells (December to May). The day-to-day conditions are created by the north-east Trade Winds. The north coast of Jamaica is especially vulnerable to these wave conditions because of its location. The swells, on the other hand, are generated by north Atlantic cold fronts and these waves can approach from the north to north-west sector.

The deep-water operational wave climate describing the day-to-day wave conditions was obtained from the global wave model Wave Watch 3 (WW3) developed by the US National Oceanic and Atmospheric Administration (NOAA). The WW3 model archives wave parameters including wave height, period and direction as well as the wind speed and direction. Data is available for every three hours from July 1999 to April 2015, giving a total of over 46,000 data points per parameter and covering almost 16 years. This time series of wave conditions was extracted for a node located north of Jamaica. Figure 5.4 shows the wave height distribution and the location of the node (Node 8) that was selected for the project. Note that most of the waves come from the east sector, as dictated by the Trade Wind patterns.

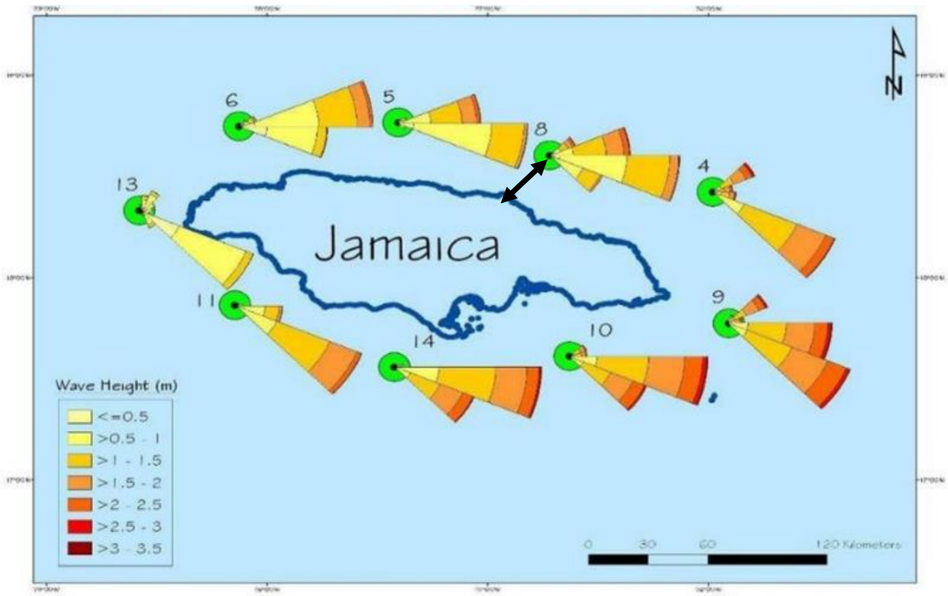


Figure 5.4 NOAA Wave Watch 3 nodes near Jamaica

5.4 Nearshore Wave Climate

The WW3 model used by the NOAA 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. The nearshore wave climate for this project was therefore developed using a spectral wave model MIKE21 SW to simulate waves as they approach the nearshore of the project site. The basic starting point of the model is the creation of a computational mesh where waves and currents are determined at each simulation time step. The MIKE21 model uses a flexible mesh that represents the seabed using a series of connected triangular and/or quadrangular elements. The bathymetric data is then interpolated on the flexible mesh to create the model domain. As shown in Figure 5.1, the model domain extends out to water depths of at least 1000m and captures the changing contours, which tend to run parallel to the shoreline.

We used the validated model to investigate mean wave conditions within the bay. We used 20 years of wave data to predict the conditions at the site and found that over the past 20 years the average wave heights within the bay were less than 0.3m. These wave heights are good for comfortable wading/swimming and other recreational activities in the bay. In contrast, along the shoreline of the proposed Hotels 2 and 3, the average waves were shown with values over 0.7m. This will be unsuitable for recreational activities year-round. This confirms that the proposed revetment for this area will have to be elevated in this section to prevent uncomfortable sea spray from the crashing daily waves. This also justifies the need for the proposed structures to create the new beach between Hotels 3 and 4.

Nearshore reefs along the shoreline of Hotel 4 create a sheltered area with waves less than 0.2m in height. The sudden breaking of the waves, however, results in a strong current flow that has been causing erosion of the shoreline. Figure 5.5 shows the 20-year mean wave heights and directions for the site.

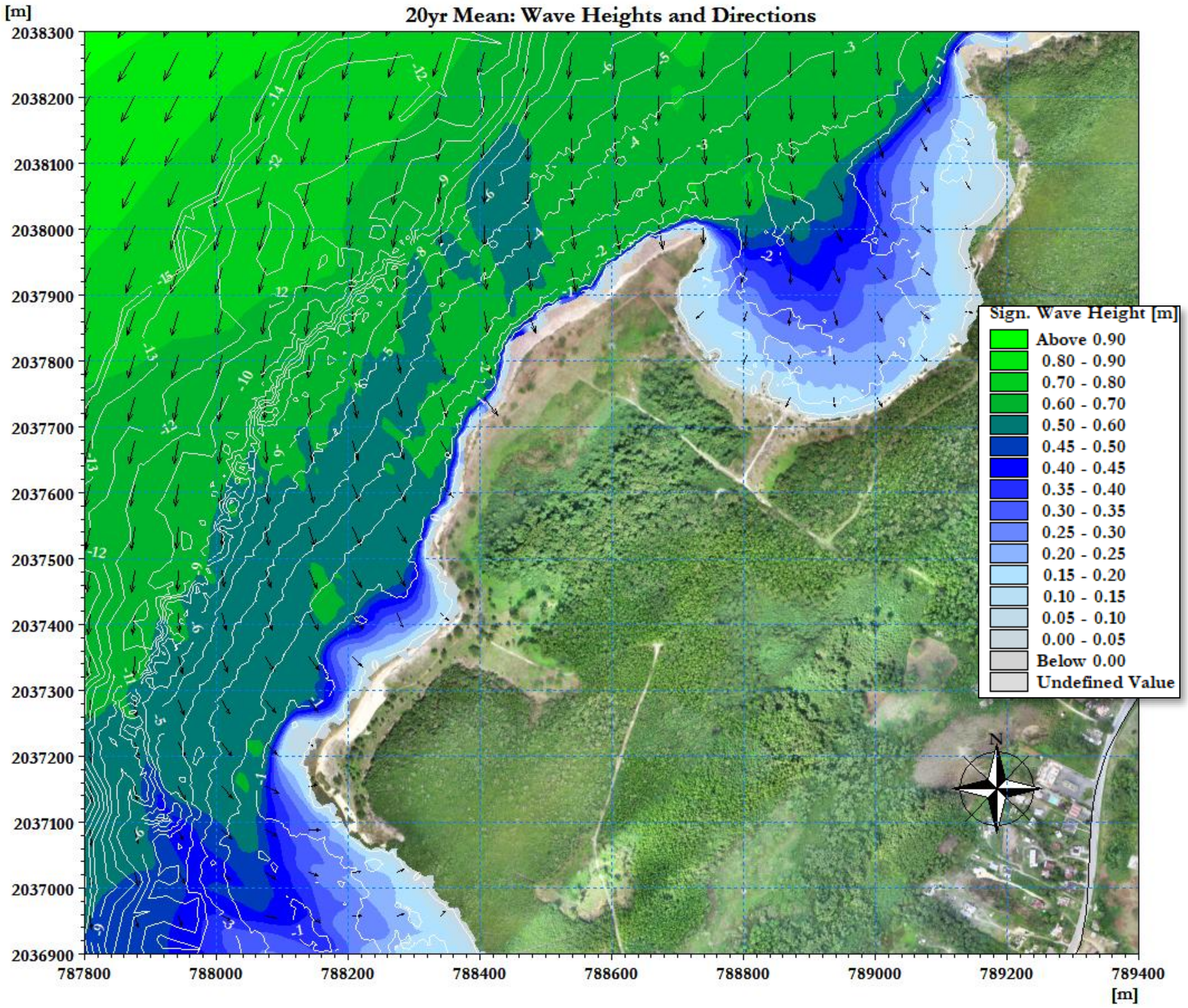


Figure 5.5 20-year mean wave heights and directions

5.5 Currents and Tides

In general, currents at the site flow in a westerly direction. The currents are strongest in areas where waves break, i.e. along the rocky shore of Hotels 2 and 3 and the shallow area of Hotel 4. The breaking of waves on the reef at the opening of the cove also causes these strong currents to pull water out of the cove and cause significant movement of any sediment in the area. This is why the shoreline of Hotels 2 and 3 is rocky; there is no opportunity for sand to stay in place. A perched beach in this area (as currently proposed) will create valuable sunbathing areas that would not be eroded by the strong currents. The cove beach in front of Hotel 1 has slow-moving currents that will cause stagnation in the area. Runoff into the bay must therefore be limited to preserve good swimming conditions. Figure 5.6 shows 20-year mean current speeds and directions at the project site.

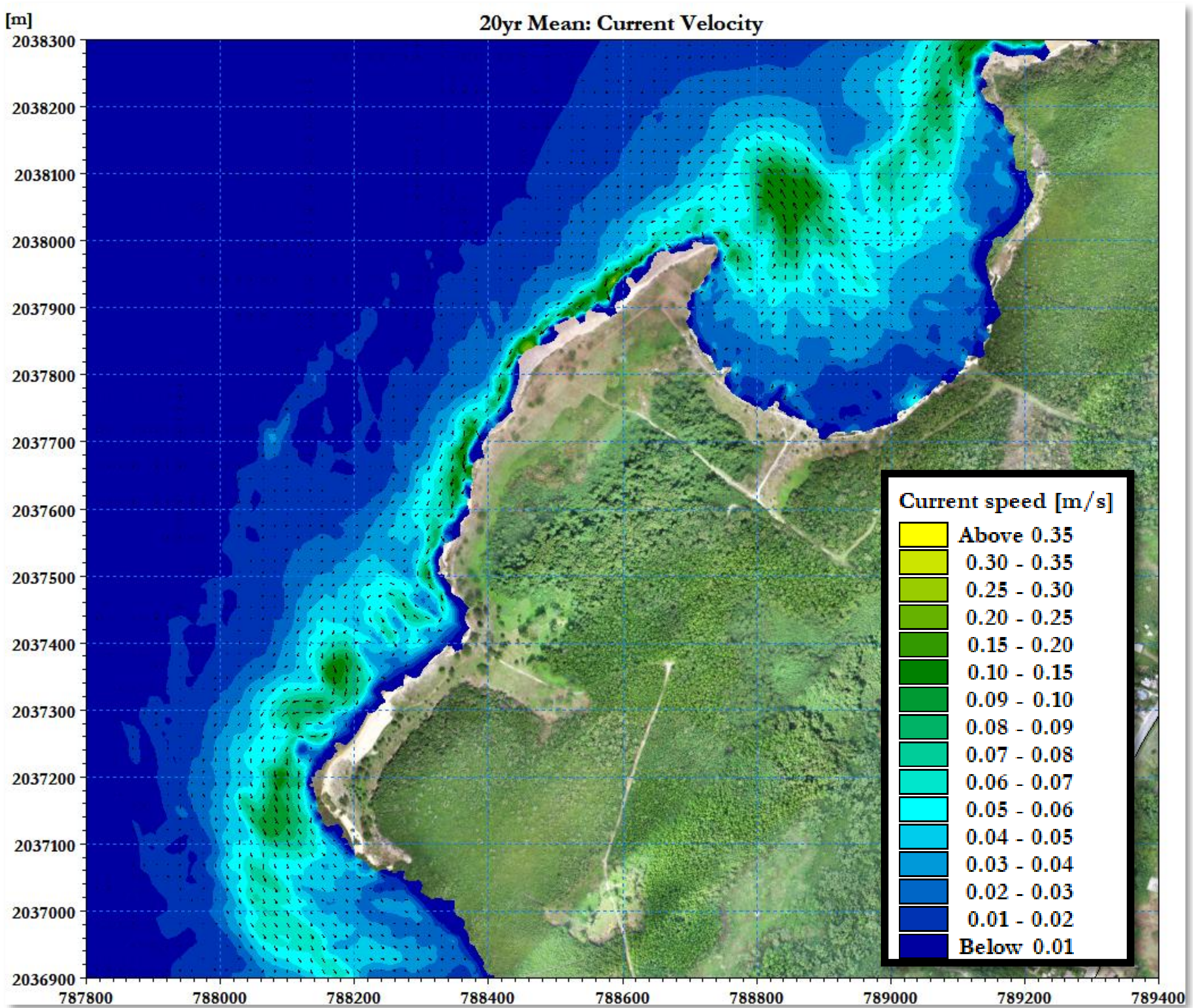


Figure 5.6 Current speeds and directions at the project site

5.6 Hurricane Wave Climate

The Caribbean region is vulnerable to tropical storms and hurricanes each year from June to November. Dramatic and abrupt changes to the coastline can occur because of these storms. In general, coastal protection structures are designed to withstand wave attack from these extreme storm events (e.g. 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 because of extreme waves). Extreme waves occur infrequently, and decades or centuries of data must be explored to adequately describe the statistics.

For the Atlantic Ocean, detailed information on tropical cyclones, including all hurricanes, has been collected by the US National Oceanic and Atmospheric Administration (NOAA), specifically at the National Hurricane Center (NHC). This database of storm tracks and other parameters was the main source of information describing individual storms. Hurricane tracks in the North Atlantic basin can often be characterized by a parabolic sweep. These typically form between latitudes 5°N and 25°N off the west coast of Africa and then track across the Atlantic Ocean. Those formed at the lower latitudes are usually pushed on a westerly track by the north-east Trade Winds, whereas those of the higher latitudes track more to the north and north-west.

A tropical cyclone is classified as a hurricane only after it has attained one-minute maximum sustained near-surface (10m above ground level) winds of 33m/s or more. Below this, these cyclones are referred to as tropical storms. The Saffir-Simpson Scale is commonly used to classify hurricanes into five different ranges based on the maximum wind speed attained.

5.6.1 Storm Occurrence

Using HurWAVE (Banton 2002), an in-house hindcasting program, we determined that a total of **118 hurricanes and tropical storms** have passed within a 300km radius of the Princess Resorts property since 1850. The number of occurrences within each storm category (per the Saffir-Simpson scale) is shown in Figure 5.7. The bar chart shows that many of the storms that pass the project area are weaker than a Category 3 cyclone. Only four (4.2%) of the recorded storms were Category 3 or stronger. The graph shows that the study area was more frequently hit by tropical storms and was rarely affected by major hurricanes. Figure 5.8 shows the storm tracks of the hurricanes that came closest to the site, the most intense of which was Hurricane Gilbert (a Category 3 hurricane in 1988). The storm track data shows that the major hurricanes pass Jamaica (and by extension the site) along the south. It therefore holds that, based on the anticlockwise nature of hurricanes, the winds and waves experienced at the site will be from the east (Figure 5.9).

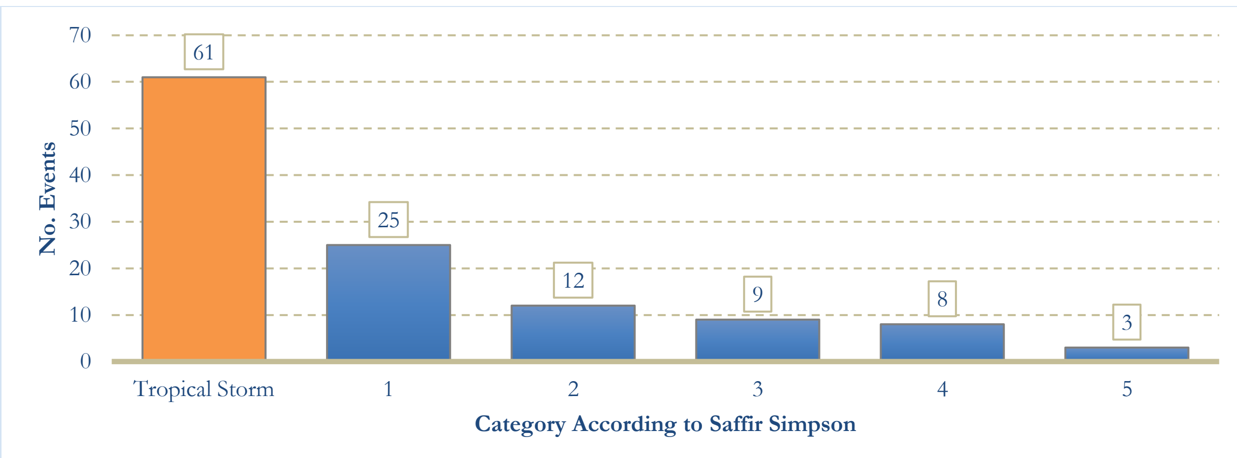
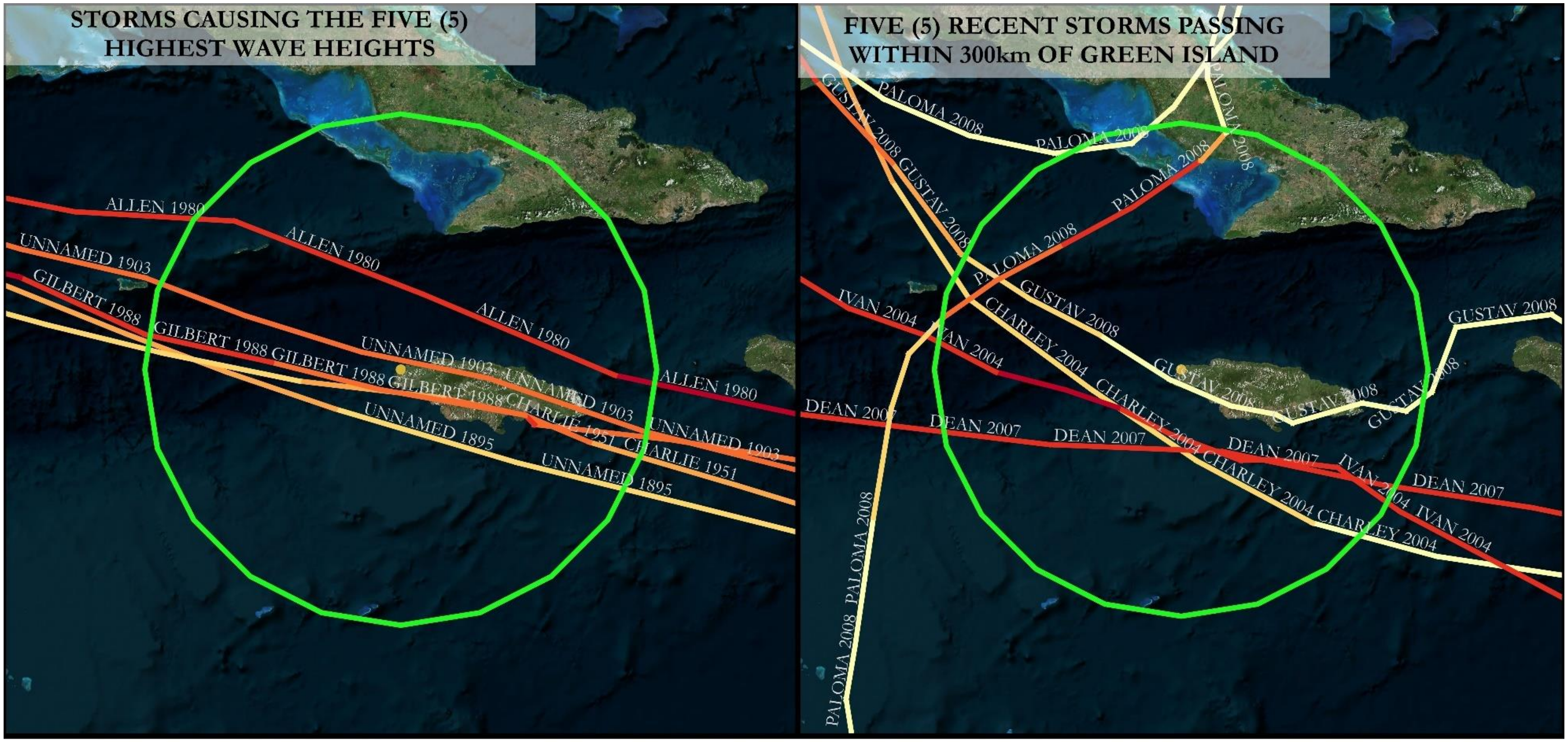


Figure 5.7 Distribution of storms according to the Saffir Simpson scale



Name	Season
UNNAMED	1895
UNNAMED	1903
CHARLIE	1951
ALLEN	1980
GILBERT	1988

Legend

- Hurricane Radius of Influence
- TROPICAL DEPRESSION OR TROPICAL STORM
- CATEGORY 1 HURRICANE
- CATEGORY 2 HURRICANE
- CATEGORY 3 HURRICANE
- CATEGORY 4 HURRICANE
- CATEGORY 5 HURRICANE

Name	Season
CHARLEY	2004
IVAN	2004
DEAN	2007
GUSTAV	2008
PALOMA	2008

Figure 5.8 Storm tracks of the closest passing hurricanes

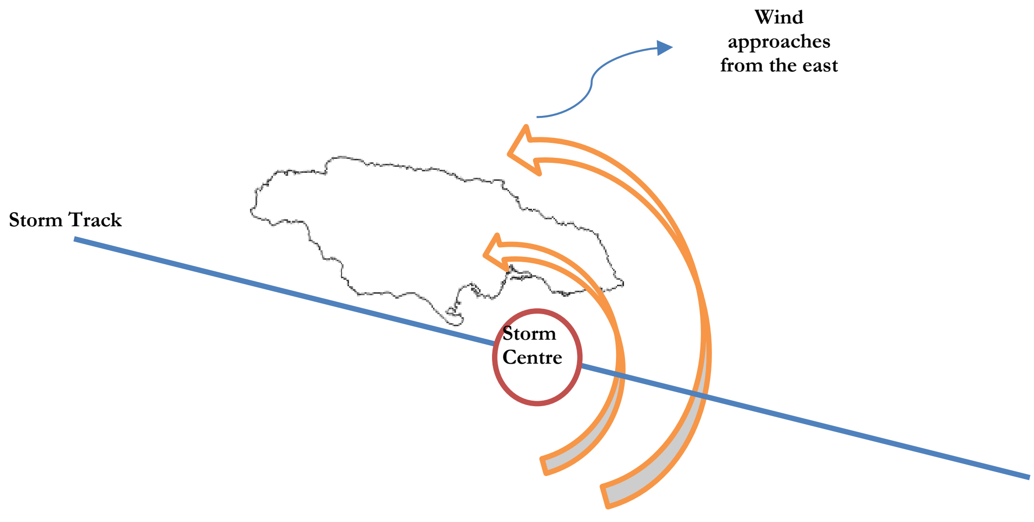


Figure 5.9 Schematic diagram wind & wave directions from hurricane conditions

5.6.2 Hindcasting Hurricane Waves and Surge Levels

Hurricanes have two immediate coastal hazards: (1) stronger waves and (2) higher water levels. These extreme conditions can be calculated using the MIKE21 Spectral Wave (SW)/Hydrodynamic (HD) models. The models can be forced with the highest deep-water wave and water level conditions and will simulate the transformation of waves from deep-water to the shallow water location of the site. It is important that the worst-case wave and water level conditions be used to simulate these shallow water conditions. For worst-case wave conditions, the values are selected through the process of hindcasting where conditions are calculated for a past event at a given time and location. Water levels are obtained by assessing the possible extreme tides and sea level rise conditions under hurricane conditions. The process is described briefly in the following paragraphs.

Deep water wave parameters were calculated for each selected tropical cyclone using parametric hurricane models (Cooper 1988; Young and Burchell 1996). The resulting wave conditions were divided into directional sectors and each set was fit to a statistical function (Weibell) describing their exceedance probability. The wave parameter values for the 50 and 100-year return periods were determined from the best-fit statistical distribution. The deep-water wave parameters corresponding to the 50 and 100-year return periods were computed for five directional sectors of incidence. These return periods were calculated using the probability of exceedance. The 1-in-25, 1-in-50 and 1-in-100-year events have, respectively, a 4%, 2% and 1% chance of occurring (Figure 5.10). The event with the 1% chance of occurrence is typically the design condition used for protecting residential buildings. Table 5-1 shows the wave heights, wind speeds, and periods for the directional sectors investigated. These wave parameters will be used in MIKE21 SW with the inclusion of the static storm surge levels to obtain design wave heights in the nearshore of the selected areas.

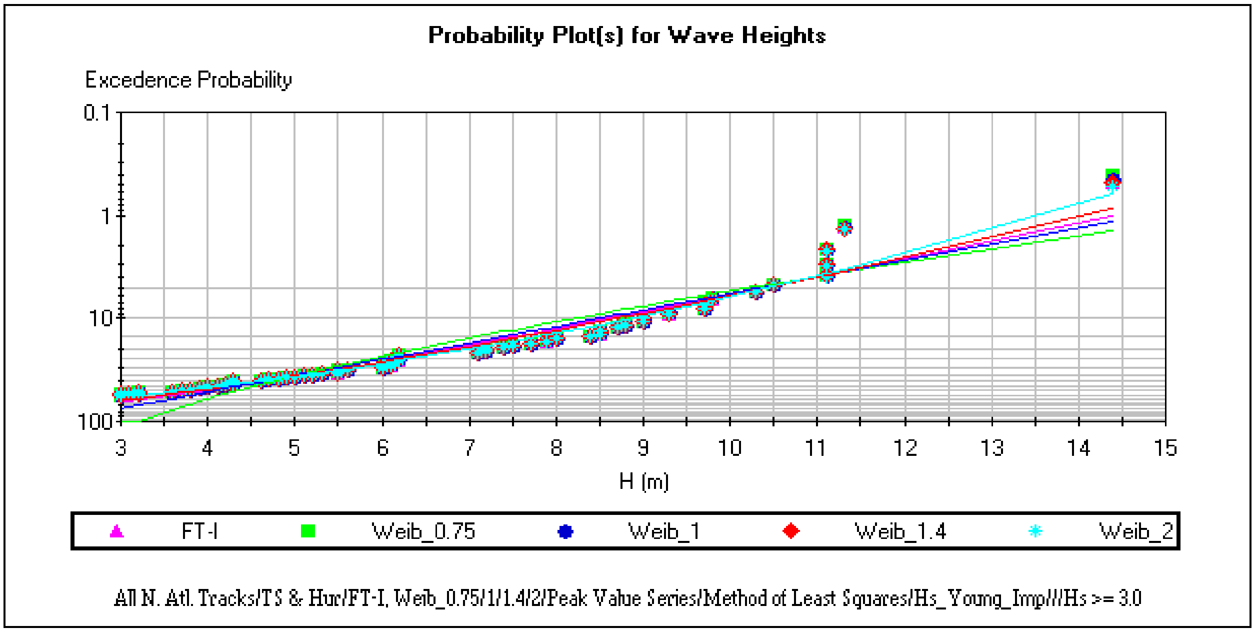


Figure 5.10 Probability of exceedance plots for significant wave heights

Table 5-1 Deep water hurricane wave parameters (significant wave height (Hs), peak period (Tp) and wind speed (Vm) resulting from the 25, 50 and 100-year return periods

Directional Sector	*Parameters*	*Return Period*		
		25	**50**	**100**
North	**Hs (m)**	6.11	7.77	9.28
	Tp (s)	10.33	12.0	13.43
	Vm (m/s)	25.68	29.99	33.94
North-east	**Hs (m)**	8.70	10.79	12.69
	Tp (s)	12.90	14.77	16.36
	Vm (m/s)	34.05	38.44	42.46
East	**Hs (m)**	10.00	11.78	13.41
	Tp (s)	14.08	15.61	16.94
	Vm (m/s)	31.54	35.48	39.09
West	**Hs (m)**	5.78	7.09	8.28
	Tp (s)	9.97	11.33	12.50
	Vm (m/s)	23.18	27.40	31.25
North-west	**Hs (m)**	5.83	7.31	8.66
	Tp (s)	10.02	11.55	12.86
	Vm (m/s)	20.54	24.60	28.88

The elevated water levels that accompany hurricanes and can create flooding and cause damage to coastal infrastructure is known as storm surge. Storm surge is the rise in water surface elevation of the sea above its mean level. Storm surge is made up of two major components:

i. Static surge, which includes:

- Highest Astronomical Tide (HAT)
- Inverse Barometric Rise (IBR) (caused by low pressure under hurricanes)
- Global Sea Level Rise (GSLR)

ii. Dynamic surge, which includes:
- Wind Set-up (when winds push water up onto the land),
- Wave Set-up (caused by wave breaking)

To compute the total static storm surge level in deep water, global sea level rise (GSLR) for the projected year and the highest astronomical tide were added to the IBR values. The results for the 50, 100 and 200-year surface level values are listed in Table 5-2. Results were further used as input boundary conditions to the MIKE21 Spectral Wave (SW) model.

The MIKE21 SW/HD can only calculate waves and static water levels. Therefore, the assessment at the site was done in two steps:

1. Deep water conditions were transitioned to the site using the MIKE21 model suites (see section 5.6.3 Results of Hurricane Simulations)
2. Dynamic surge levels were then calculated using sBEACH (see section 5.6.4 Inundation Levels)

Table 5-2 Calculation of water levels for 25, 50 and 20 – year hurricane return periods

Parameter	Return Period (years)			Notes
	25	**50**	**100**	
IBR (m)	0.26	0.32	0.37	Determined through statistical hind-casting analysis
Highest Astronomical Tide (m)		*0.25*		Determined through historical analysis
Rate of Sea Level Rise (mm/year)		*7.5*		RCP8.6 Scenario value from IPCC 2014 report (Edenhofer et al. 2015)
Design Time Horizon (years)	50	100	200	Design life of the structure
Design Deep Water Surface Level **without Climate Change (m)**	**0.51**	**0.56**	**0.62**	**Sum of IBR, Highest Astronomical Tide**
Design Deep Water Surface Level **with Climate Change (m)**	**0.70**	**0.94**	**1.37**	**Sum of IBR, Highest Astronomical Tide, and Sea Level for 50, 100 and 200 years.**

5.6.3 Results of Hurricane Simulations

The computed and deep-water wave heights and water levels were used as input boundary conditions to the MIKE21 Spectral Wave (SW) module, which was coupled with the MIKE21 Hydrodynamic (HD) module. The model incorporates constant wind speeds from each directional sector in both the wave and hydrodynamic modules. The results were determined for each direction and return period. The computed values at each location were then combined to determine the worst-case (or highest) values for all directions for each of the different return periods.

Hurricanes have the potential to cause flooding from storm surge as well as damage due to the high energy wave impacts. Storm surge levels as shown are related to the increase in sea level due to the low-pressure system caused by a hurricane. Under existing conditions, the site would flood by more than **1.5m** of water in the 50-year hurricane event (Figure 5.11). The rocky shoreline of Hotels 2 and 3 is at the greatest risk to wave damage during a hurricane. The wave heights at this section of the development are greater than 2.5m (Figure 5.12). This has implications for the design of the revetment and groynes proposed for the area, as stronger waves will require larger structures to reduce the associated risks.

The reef system at Hotel 4 reduces the wave heights reaching the shoreline at this section of the property. Reinforcing the reef further could reduce the waves reaching the proposed sandy beach area. Similarly, at Hotel 1, wave heights are reduced by the reefs to between 1-1.5m. While the bay is somewhat sheltered, the stronger waves are towards its western end and hence the area for the Sea Rooms will have some protection.

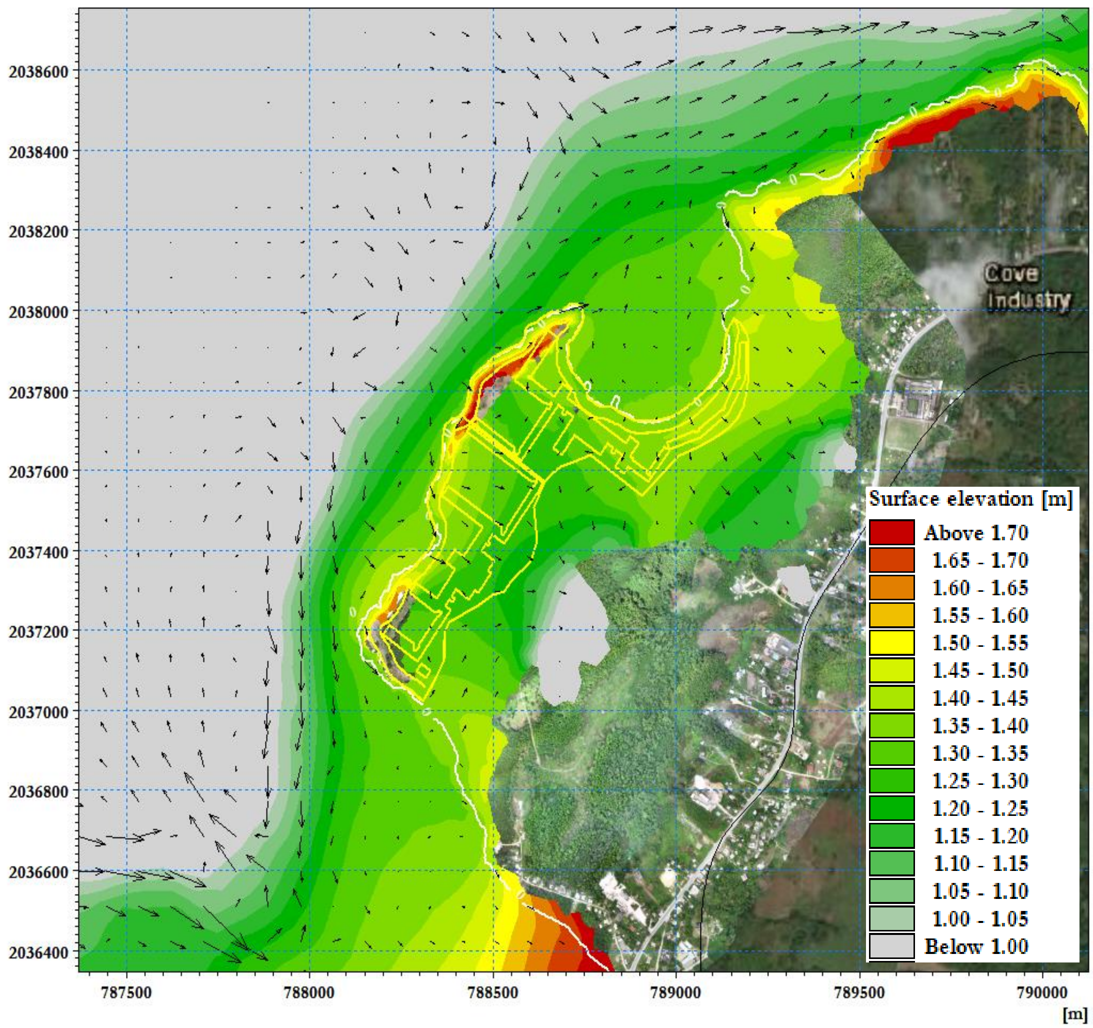


Figure 5.11 Static water level above MSL under the 1 in 50yr hurricane condition

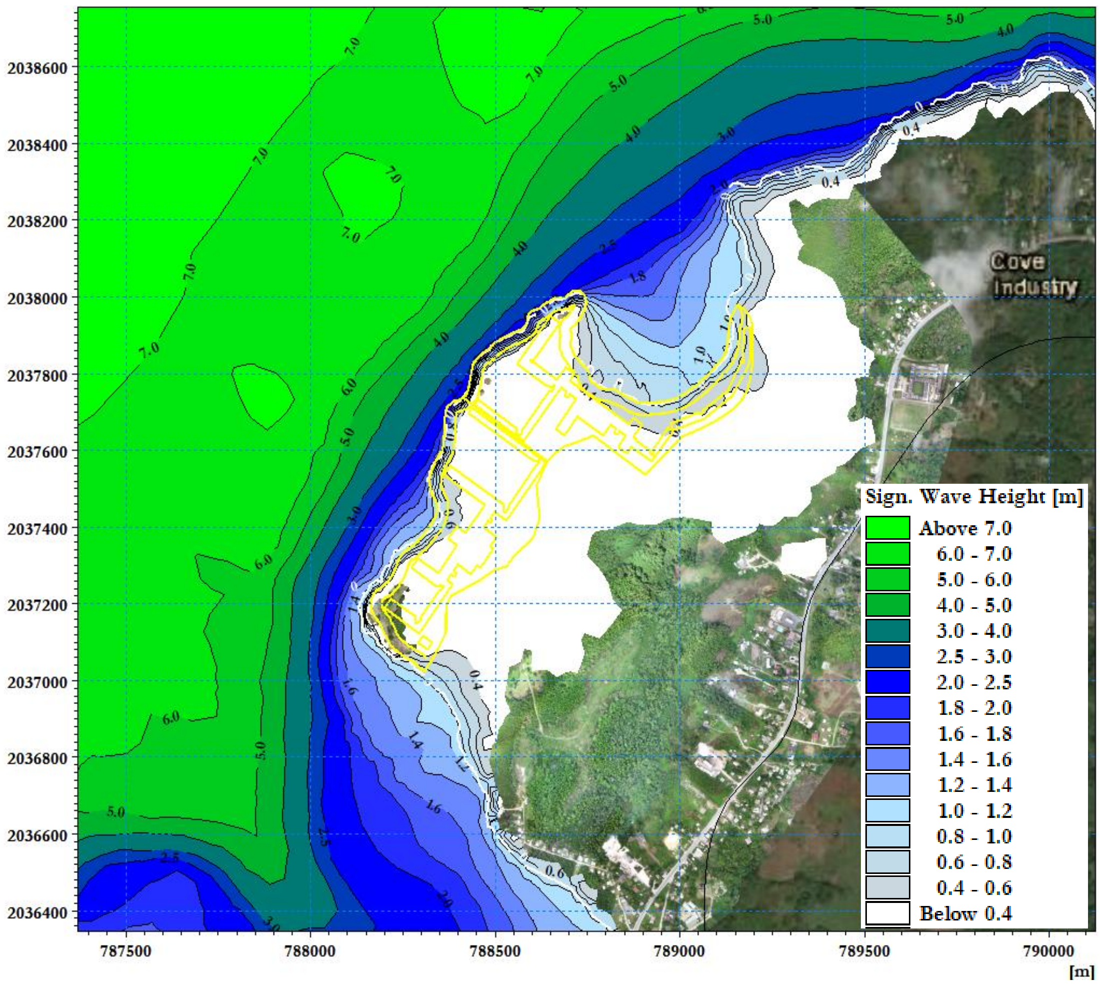


Figure 5.12 Significant wave heights under the 1 in 50yr hurricane condition

5.6.4 Inundation Levels

The numerical modelling presented in the following section presents wave run-up conditions for the existing shoreline. The storm surge calculated previously represents the static water level **(+1.3 – 1.7m MSL)** that will occur *close to* the shoreline. At the shoreline, however, waves run up onto the beach, which further increases the surge level. This component (wave run-up) is the dynamic component of storm surge that, when added to the static surge, gives the total inundation level.

The foregoing analyses provided the necessary design guidelines for the establishment of structure crest elevations and toe protection. These assessments were done in a two-step sequence (described below) by using the results from the sBEACH model as input to the online CRESS application.

sBEACH The 1D sediment transport model and wave transformation model was used to model the cross-shore movement of sediments and expected shoreline changes (areas of high erosion or accretion potential) due to wave impact. The objective of using this model was to predict wave overtopping over any existing shoreline features along the project shoreline.

CRESS The final inundation levels were computed by combining sBEACH results and the wave run-up/ overtopping over the existing shoreline features calculated from

CRESS. CRESS is an online user interface that uses empirical coastal engineering equations. The application provides an approach to calculate wave run-up on either smooth-sloped linear beaches or rough sloped natural beaches, as well as wave run-up and overtopping on rough and smooth sloped structures that are assumed to be impermeable.

Three representative cross-shore profiles were used as input to the sBEACH model. These profiles were extended perpendicularly from the shoreline to the 50m depth contour up to the project site. The wave heights and periods as well as the wind speeds and water level set-up from the 50-year storm event were extracted in 50m depths from the MIKE21 results, and input to the model with a direction perpendicular to the shore (representative of the worst-case scenario). Results were plotted for the 50-year return period after a typical 8-hour storm.

Table 5-3 gives the maximum values at the shoreline for wave heights and water levels. These values represent the boundary conditions and the guides for the design of the coastal structures. The values are indicative of the surging nature of waves in the area. Figure 5.13 to Figure 5.15 show surge levels at each of the hotel shorelines.

Table 5-3 Significant wave height, max water level and depth of scour at the shoreline for Profiles 1 – 3

Profiles	***Results from the 1 in 100yr storm***	
	Max Wave Height before Breaking	**Max Water Level**
Hotel 1	**1.6m**	**2.4m MSL**
Hotel 2 and 3	2.5m	2.7m MSL
Hotel 4	1.4m	2.4m MSL

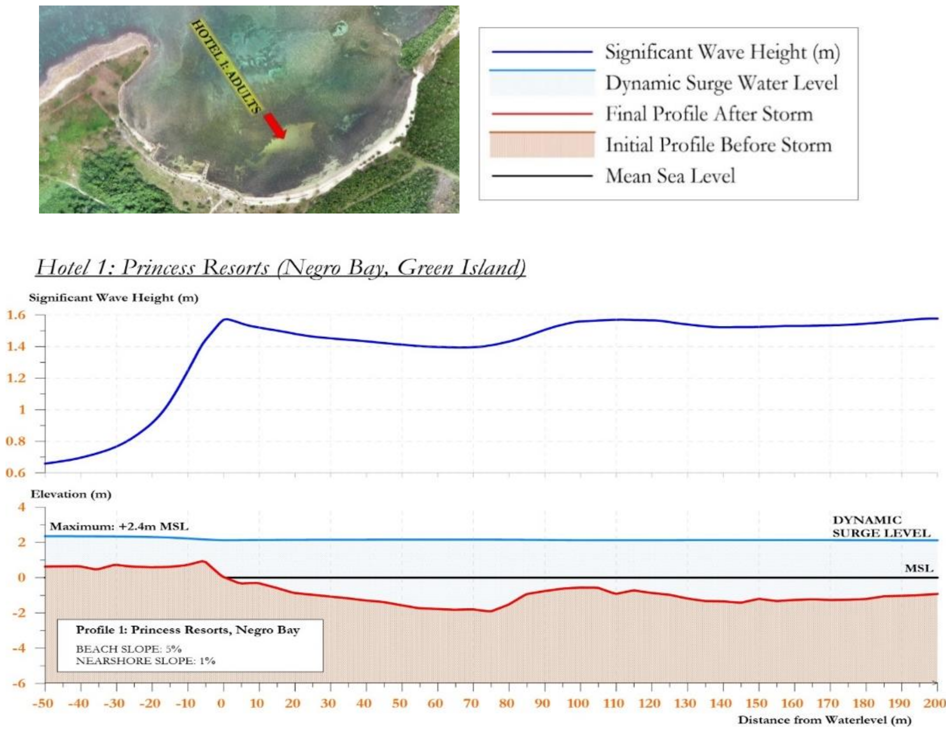


Figure 5.13 Surge levels for Hotel 1

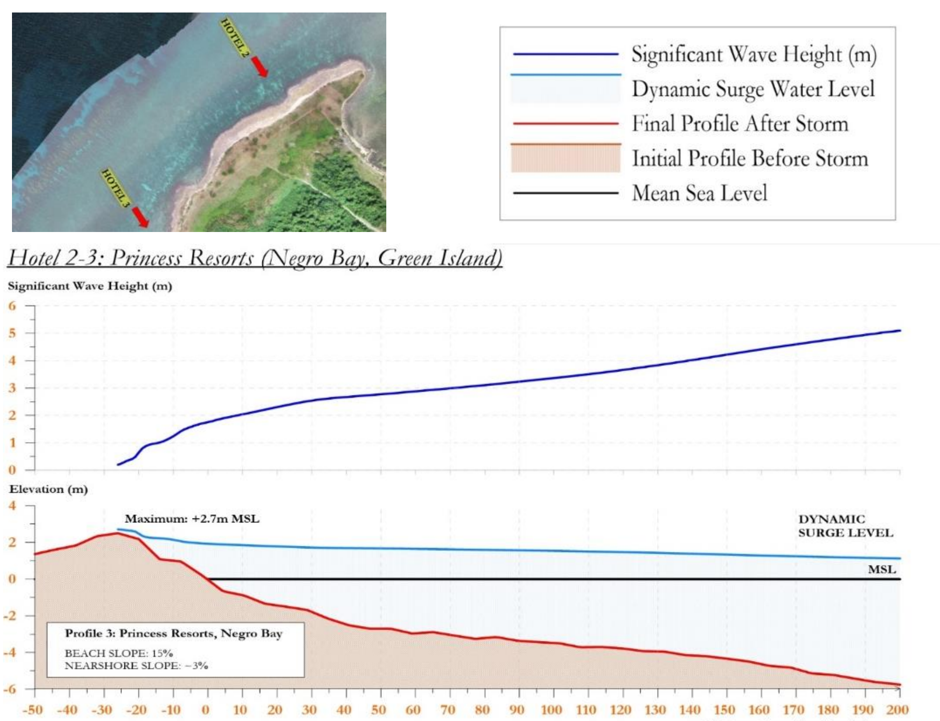


Figure 5.14 Surge levels for Hotel 2 and 3

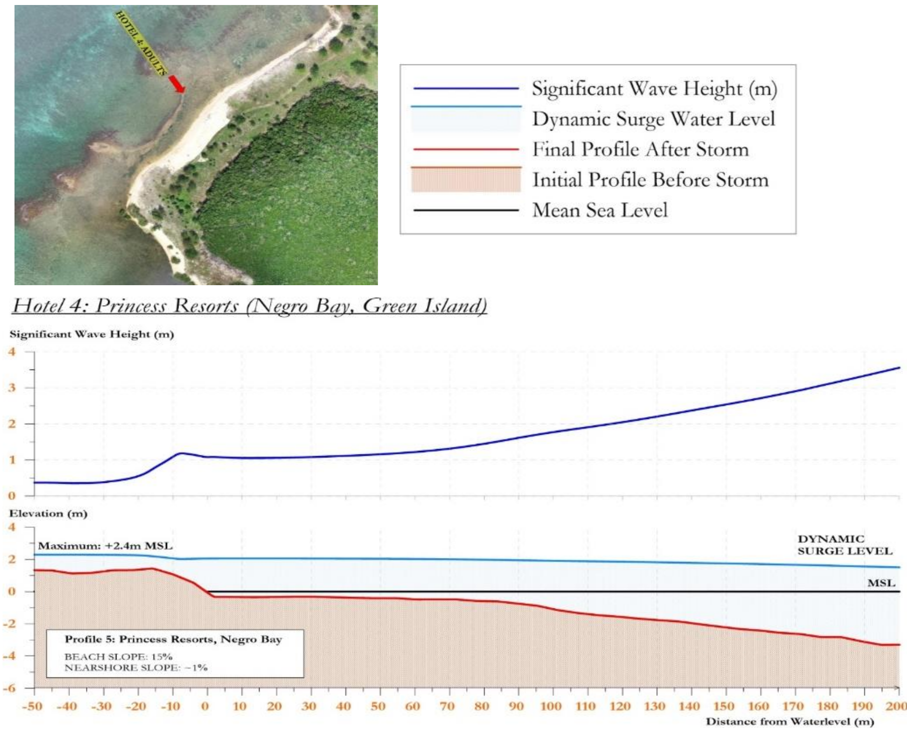


Figure 5.15 Surge levels for Hotel 4

5.7 Shoreline Morphology

This section of the report presents shoreline trends along the project site. Historical movement of the shoreline and results of the numerical modelling were used to predict trends for the area.

5.7.1 Historical Shoreline Analysis

The assessment of erosion and accretion trends in an area starts with an inspection of aerial images of the site over time. A total of eight images were obtained spanning a 16-year period (2003 to 2017, Figure 5.16). The images were georeferenced, and the shorelines were digitized.

From the initial inspection it was found that significant erosion occurred within the bay of Hotel 1 and the beach at Hotel 4. The rocky shoreline at Hotels 2 and 3 did not change over the 16 years and suggests that it stable (it is ironshore). Figure 5.17 shows that at Hotel 1 and Hotel 4 the beach width decreased by 35m and 28m respectively. Six reference points at the back of the shoreline were used to measure the distance to shoreline. This provides an idea of the amount of fluctuation in beach width over the years. The shoreline had the most fluctuation between 2003 and 2009 (Figure 5.18). Between 2014 and 2019 the rate of erosion within the bay of Hotel 1 slowed, suggesting a tendency to equilibrium or stability. For the beach at Hotel 4, there is some level of dynamism (erosion and accretion), but the overall trend is one of erosion.

Based on aerial images in 2003 and 2004 the shoreline was predominantly mangrove coast. Mangrove coasts are more resistant to coastal erosion. Sometime before 2009, the mangroves at Hotels 1 and 4 were removed. Since then, coastal erosion in the area increased. This explains the notable reduction in beach width as seen between 2004 and 2013.

Figure 5.16 Aerial photos from 2003-2017

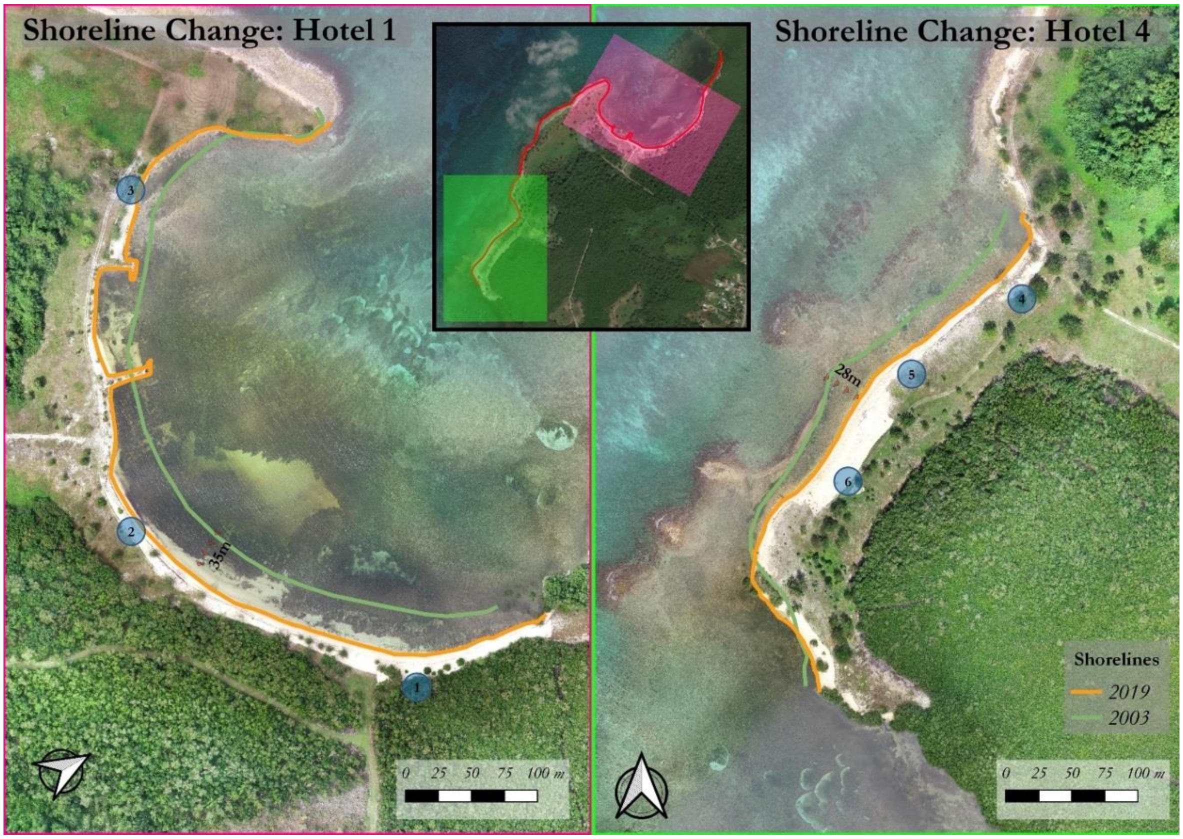


Figure 5.17 Aerial images of the sandy beach sections showing most seaward and landward shorelines

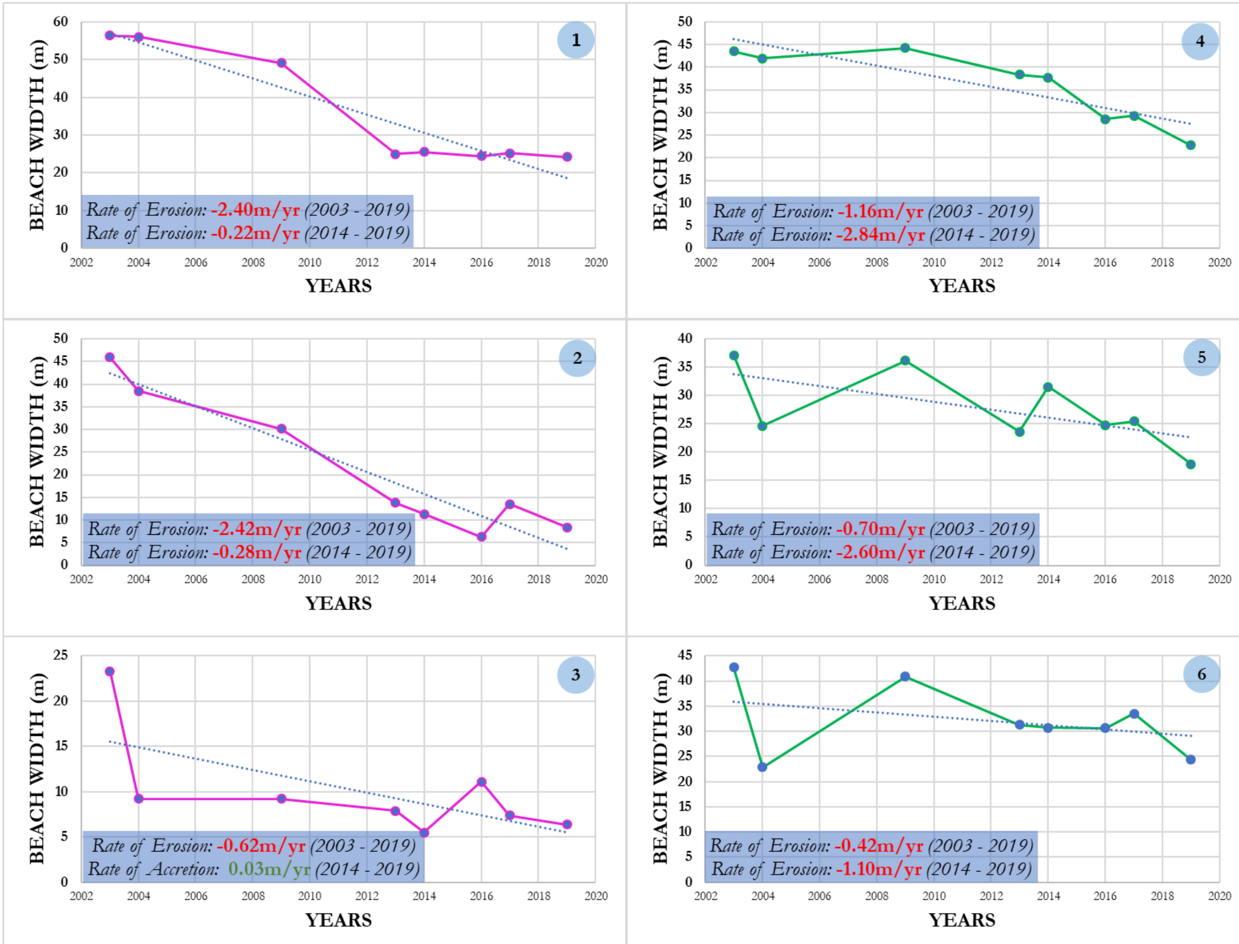


Figure 5.18 Fluctuation in beach widths along the property

5.7.2 Alongshore Sediment Transport

Using the mean annual wave climate, potential alongshore sediment transport was estimated for three profiles. The resulting distribution of cross-shore sediment transport in the nearshore is given in Figure 5.19 and Figure 5.20.

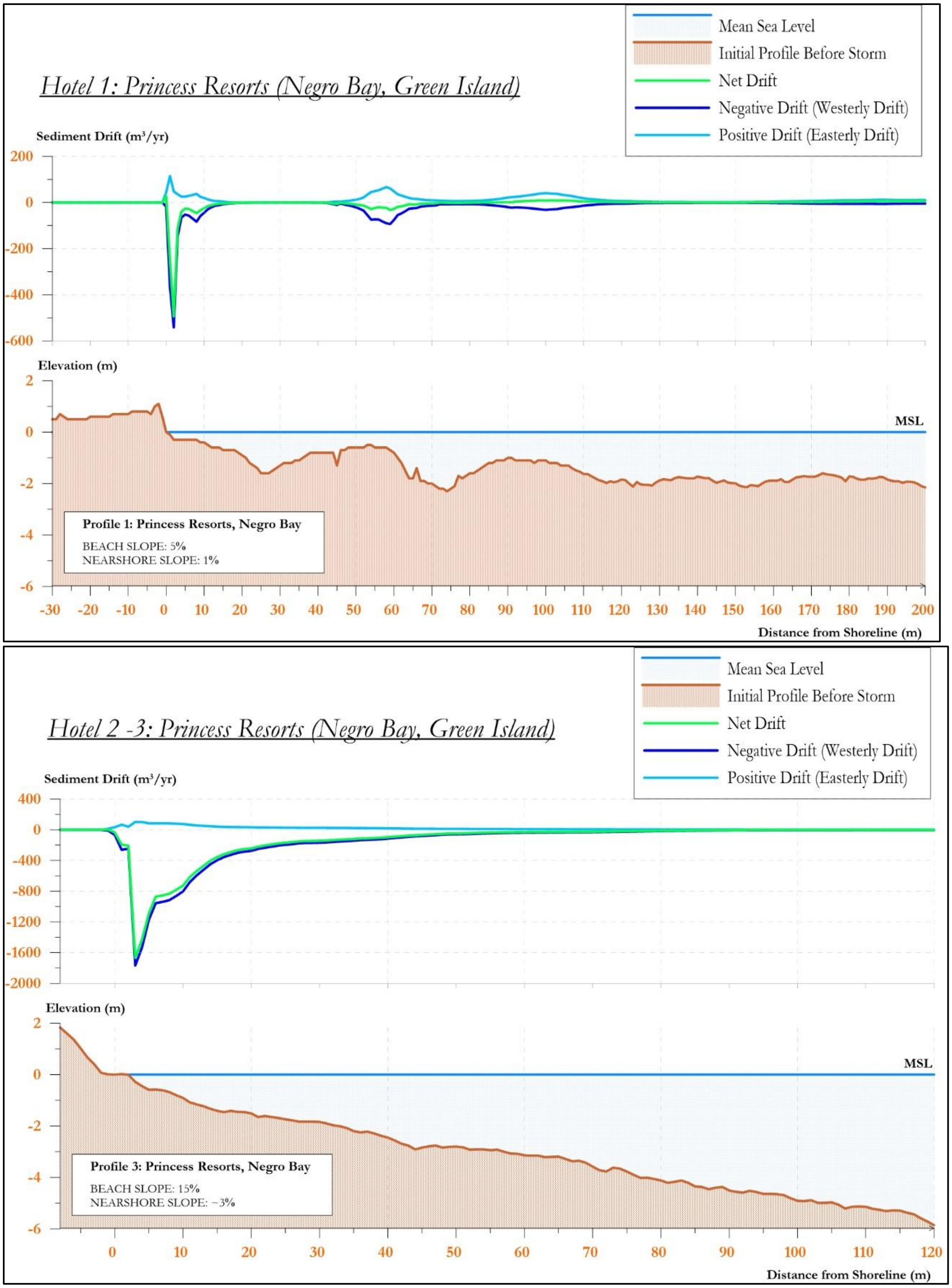


Figure 5.19 Alongshore transport distributed along the cross-shore of Hotels 1, 2 and 3

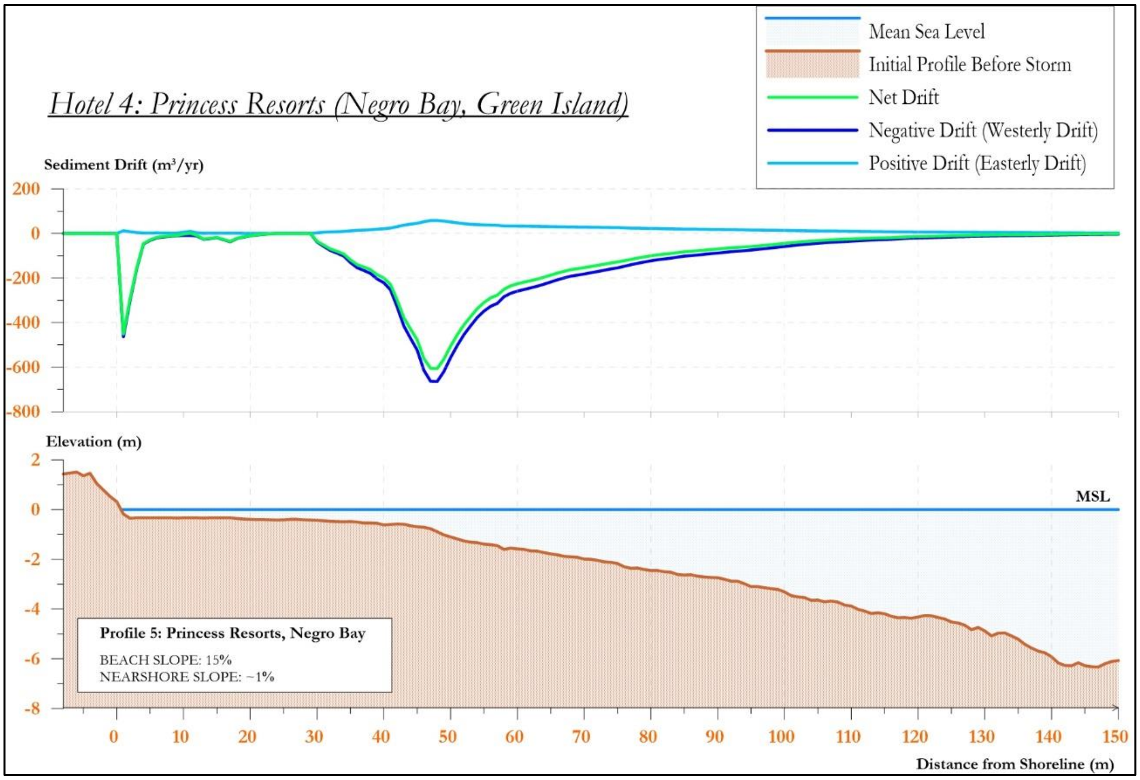


Figure 5.20 Alongshore transport distributed along the cross-shore of Hotel 4

Key points to note are:

- The main direction of sediment drift is toward the west.
- The bulk of sediment transport at Hotel 1 occurs within the first 20m offshore while for Hotels 2 and 3 the bulk of transport occurs approximately 70m offshore.
- The impact of the reef system is clearly shown at Hotels 1 and 4. The wave-breaking process at Hotel 4 occurs at the reef and there is a notable increase in the amount of sediment transport on the reef.
- The amount of potential sediment movement along the rocky shore (Hotels 2 and 3) is almost three times that of Hotels 1 and 4. This indicates a limitation for placing sand along the shoreline in this area. Any sand placed here would have to be protected from the high potential for sediment movement.
- The LITDRIFT model is somewhat limited because of its dependence on alongshore transport, whereas the shape of the beaches in this area means they are swash-aligned. Swash-aligned beaches have a predominant cross-shore drift (i.e. sand moves toward and away from the shoreline). The model is not able to account of this. This is evident as the westerly drift predicted is not supported by the aerial images of the site. If the sediment movement was mainly westerly, a build-up of sediment at the groyne would be seen. For these reasons, MIKE21 ST model was used to incorporate the coupling of alongshore and cross-shore sediment movements.

5.7.3 Beach Response to a Swell Event

A swell event represents a period during the operational wave climate when the amount of wave energy reaching the shoreline is significantly increased. This wave energy can cause notable overtopping and wave-driven erosion. This section of the report describes the morphological response of the shoreline under such extreme operational wave conditions.

To assess the effects of the swell wave conditions at the site, it was necessary to evaluate the 18 years of offshore wave data. This was done by filtering the swell events from the wave dataset. A total of 76 swell events lasting more than two days were found in the wave database. The swell selected from the filtering was the one that had the highest wave heights and came from the NW and occurred from 1-10 Mar 2009. At the peak of this swell, wave heights were greater than 3m and had wave periods longer than 8s.

The results (Figure 5.21) indicate that most of the sediment movement occurs along the rocky shore. The shallow depths within the bay of Hotel 1 do not support the amount of mixing needed to increase the movement of sediments in the area. Similarly, the reef system at Hotel 4 does not support a lot of sediment movement.

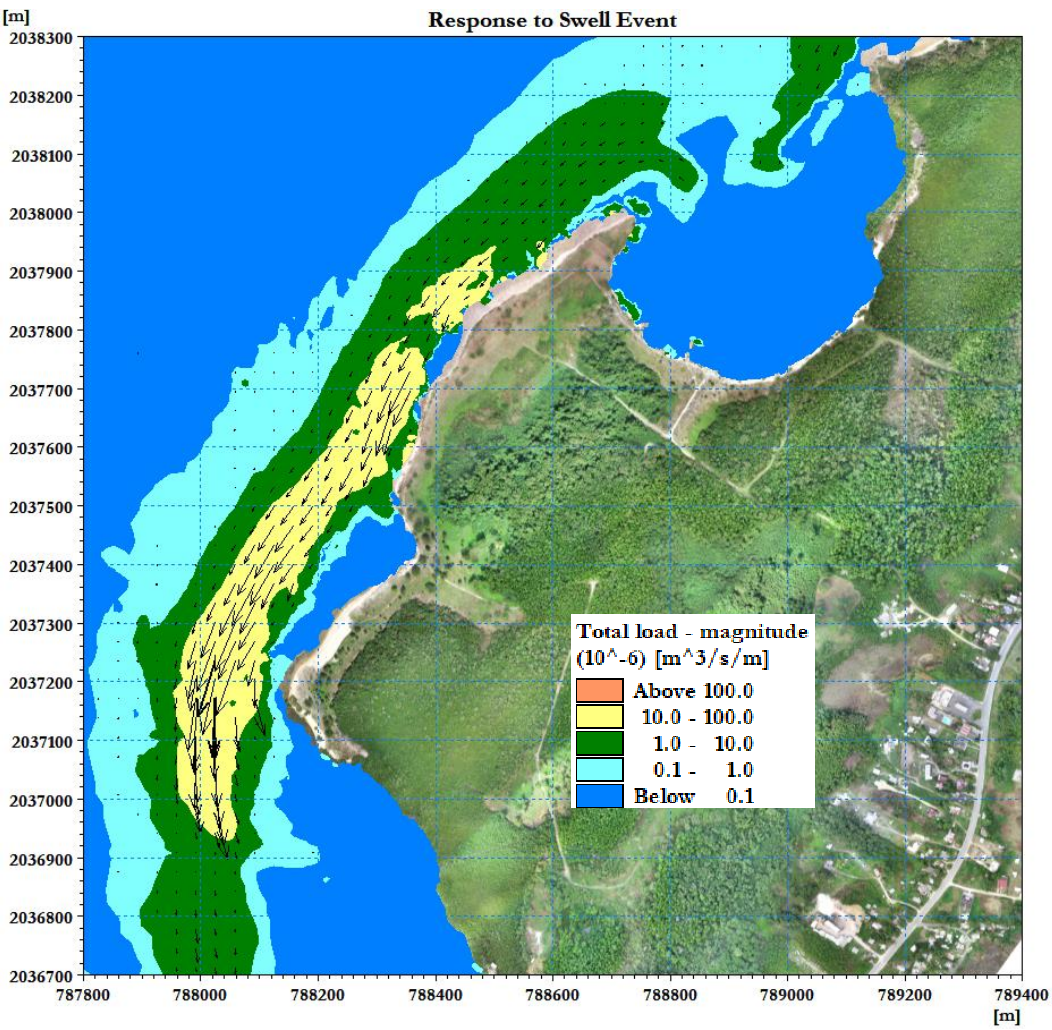


Figure 5.21 Total amount of suspended sediments and the direction of flow at the peak of the swell event

5.8 Summary and Implications

The following points can be made about the findings of the study so far:

- The shoreline positions of Hotels 1 and 4 have retreated significantly since 2003. However, the findings suggest that the retreat may be due to the removal of mangroves in the area and not wave/current actions in the area.
- The water within the bay of Hotel 1 is very shallow and this causes slow currents and circulation issues. The circulation issues must be addressed to increase the beach's viability for resort use. At a minimum, the groynes currently in place must be removed.
- Hotels 2 and 3 are along the rocky shore and bombarded by wave heights greater than 2m. This suggests that the area will not naturally support the formation of a beach.
- Along the rocky shore the currents are very fast and have potential to move large amounts of sediment.
- The site is located within the Negril Marine Park/Protected Area and the environmental sensitivity of the area must be recognized.

6 Concept Development

The project area currently has two man-made coastal structures, however it is still a very natural looking coast. Engineering solutions are needed to develop a resort-grade beach and effectively protect the Princess Resorts property shoreline under extreme events.

Solutions for coastal enhancement generally take one of the following forms:

(1) soft or ecosystem-based adaptation/engineering,

(2) hard engineering, or

(3) a combination of these.

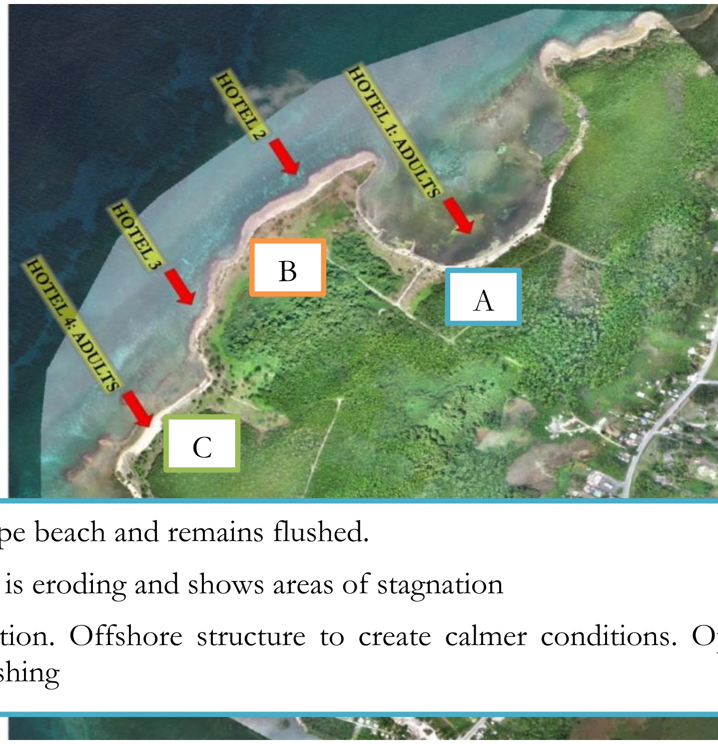
In coastal engineering, "soft engineering", or ecosystem-based adaptation solutions, involve the use of ecological principles and practices to reduce erosion and achieve the stabilization and safety of shorelines and the areas around rivers, while enhancing habitat, improving aesthetics, and saving money. Soft engineering is achieved by using vegetation and other materials to soften the land-water interface, thereby improving ecological features without compromising the engineered integrity of the shoreline or river edges. These defences can be both marine (developing or creating artificial reefs, which are natural wave energy attenuators) or landward (i.e. mangrove planting to defend a shoreline, or vegetation of a dune to trap sediment within it).

Beach nourishment is a common soft engineering solution, and it involves placing (or replacing) suitable beach material either along the shoreline to advance the shoreline seaward, or along the back of beach either through the creation of dunes to act as protection or through dry beach creation to create usable sandy beach in a more protected area. Soft engineering solutions tend to have far less environmental impact than hard solutions and, depending on the source of sand, can be the cheaper option as well. However soft solutions, when used alone, tend to require more maintenance as the beach material can be easily washed away and would have to be continuously replaced.

"Hard engineering" refers to the use of structures for coastal protection and enhancement. As simple protection, structures can be seawalls or revetments, however these options do not foster or aid in beach creation. For coastal enhancement, **groynes** (shore-perpendicular structures) or **breakwaters** (shore-parallel structures) are typically used. These structures can be enhanced through the addition of gazebos to create functional space or vegetation to create habitat. Depending on the source of boulders being used, these structures can be costly to construct, and structures generally have a higher environmental impact than softer solutions such as nourishment.

6.1 Summary of Constraints and Objectives

The site has varying features and was divided, for the purposes of this assessment, into three sections as shown in Figure 6.1. The resort's plans for the area as well as possible options are described briefly below.



A

Aim: To create resort type beach and remains flushed.

Constraints: The beach is eroding and shows areas of stagnation

Options: Land reclamation. Offshore structure to create calmer conditions. Open channel across headland to improve flushing

B

Aim: To create sunbathing area with a pocket beach for swimming.

Constraint: Steep offshore slope with an environmentally sensitive rocky shore with no swimming area.

Options: Move further inland. Excavate to create pocket beaches. Groyne and nourishment to create beaches

C

Aim: To create a stable sandy beach for the resort with some amount of area for wading.

Constraint: This is a natural sandy beach, however, nearshore is shallow pavement.

Options: Nourish to advance shoreline. Seagrass relocation to increase wading area. Move further inland. Excavate to create pocket beaches. Groyne and nourishment to create beaches.

Figure 6.1 Summary of objectives and constraints for different sections of the site

6.2 Discussion of Coastal Concept

Two variations of the concept were presented to the client for review. The major change from was the removal of one of the pockets beaches. The area is environmentally sensitive and the pocket beach would be costly to implement. Further investigations lead to the removal of the breakwaters at Hotel 1. These investigations are discussed in the following sections. Table 6-1 presents the main components of the design group according to the Hotel. The two concept variations are presented in Figure 6.2.

Table 6-1 Components of Final Coastal Design

Hotel 1	Hotels 2 and 3	Hotel 4
Concept includes: • 1.5m deep flushing channel through western headland • 2 groynes • 500m long nourished sandy beach • 79m of nourishment • Fishing/sport jetty • *40 Sea Rooms*	Concept includes: • Perched beach • 130m long pocket beach • Revetment • Two spur groynes • Submerged breakwater	Concept includes: • submerged breakwaters • 450m long nourished sandy beach • groynes • Nearshore dredging

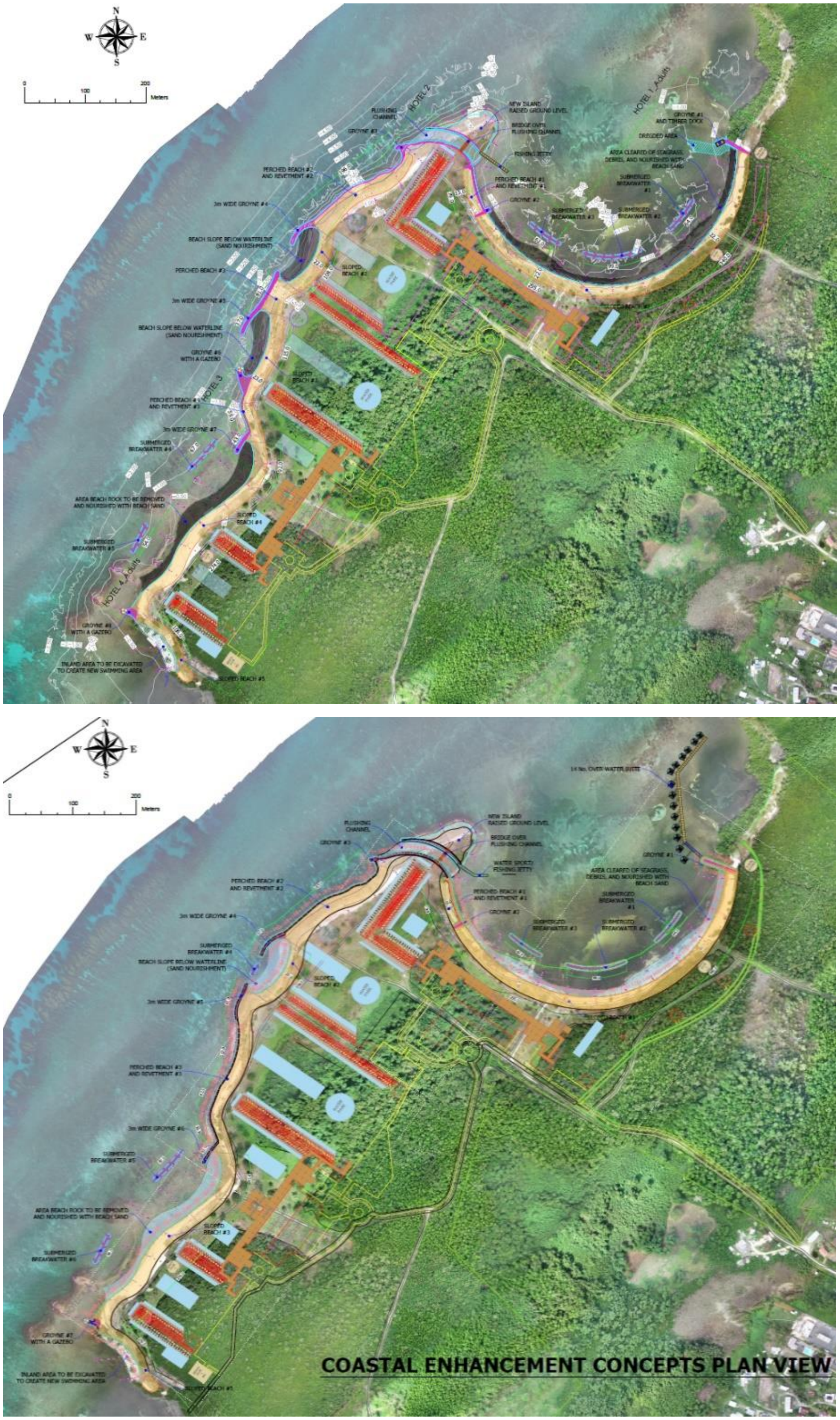


Figure 6.2 Previous concepts presented to Clients

6.3 Discussion of Drainage Options

The low-lying, flat site slopes gently from elevations ranging from 2.5-0.5m above mean sea level (MSL) down towards the mangrove forest, which lies around 0-0.2m MSL. This wetland is also located downstream of a wider catchment area that drains into it. Hence, the mangrove forest acts as a natural retention system during low flows, storing runoff to nourish the flora and fauna within it. The drainage plan therefore has two main sections:

1. The mangrove forest will be separated from the developed area via a dyke road.
2. The develop area will drain to the mangroves via outfall points along the road.

Mangrove Usage and Improvements

The plan allows the proposed site to drain freely into the mangrove forest and maintain its natural drainage pattern as flood levels are increased by 40mm for the 100-year storm event. This was deemed negligible as it has zero impact on the surrounding communities and infrastructure. Enhancement to the mangrove system will be done by introducing several culvert openings throughout the existing road network within the mangrove. This would promote more free movement of water through the entire mangrove forest, which will improve the storage capacity and provide water to areas currently deprived of water. Further improvements are also made to two areas that were observed to not have the full characteristics of the surrounding mangrove – dried out areas. These areas are proposed to be converted to ponds that would function as wetlands by planting similar types of flora (primarily mangroves) that currently grow in the forest. This would also improve the water storage capacity of the area.

Site Drainage Concept

The proposed drainage plan allows rainfall runoff to drain freely into the mangrove via ten outfall points. All the outfall points were set at an elevation higher than the projected flood elevation for the 1 in 50-year storm, with consideration for climate change. The use of multiple outfall points proved advantageous as it works better for a flat site and reduces the amount of grading required, while resulting in smaller drain sizes. These drainage outfalls will be controlled by hydraulic structures consisting of outfall pipes with flap gates to prevent back flow of water into the site when water levels in the mangrove exceeds the 50-year design flood level. These outfall pipes will be encased within a catch pit that contains a 500mm deep sump strategically located to trap sediments prior to discharging into the mangrove. All other internal drains will be primarily buried pipes and covered box drain with catch pits to keep in accordance with the architect's finish concept. All such drains and outfall pipes were designed for a 1 in 25-year storm frequency.

Plan and sections of the final coastal enhancement and drainage plans are included in Appendix C.

7 Coastal Design

This section of the report describes the performance of the chosen coastal design concept. It shows the response of the structure to extreme operational waves (i.e. swell waves), hurricanes, the changes in waves and currents, as well as the flushing potential in the area.

7.1 Flushing Response

The flushing of the area was assessed by introducing a theoretical pollutant into the bay. This pollutant could be an oil spill, sewage, high nutrients from run off, etc. In high concentration, these contaminants can make the beach experience uncomfortable. Our investigation found a high percentage (over 40%) of a pollutant would remain in the area after 24 hours. International guidelines for an area such as a marina suggest that pollutants should be below 5% in less than one day. It would take almost 3.5 days for this bay to get below the 5% concentration.

Results show the contaminant would tend to gather at the east end of the bay and would create unfavorable bathing conditions (Figure 7.1). The pollutant also accumulates beside the existing groyne and the groyne was therefore removed in the proposed concept. This confirms our initial observations of an accumulation of debris in this area.

We modeled the flushing with the proposed concept in place. Results showed that strong currents on the west help to draw out the pollutant and reduce the concentration. The use of the flushing channel is **critical** in improving the flushing of the area. Additionally, the flushing assessment showed that the removal of existing groynes is very important. With the flushing channel in place the pollutant concentration is reduced to less than 5% in just under 1.5 days (Figure 7.2). This is a significant improvement (more than two times more efficient) to the current situation. To reduce the flushing time even further, the breakwaters were removed from the system. The results below show that the breakwater removal does not affect the morphology of the system.

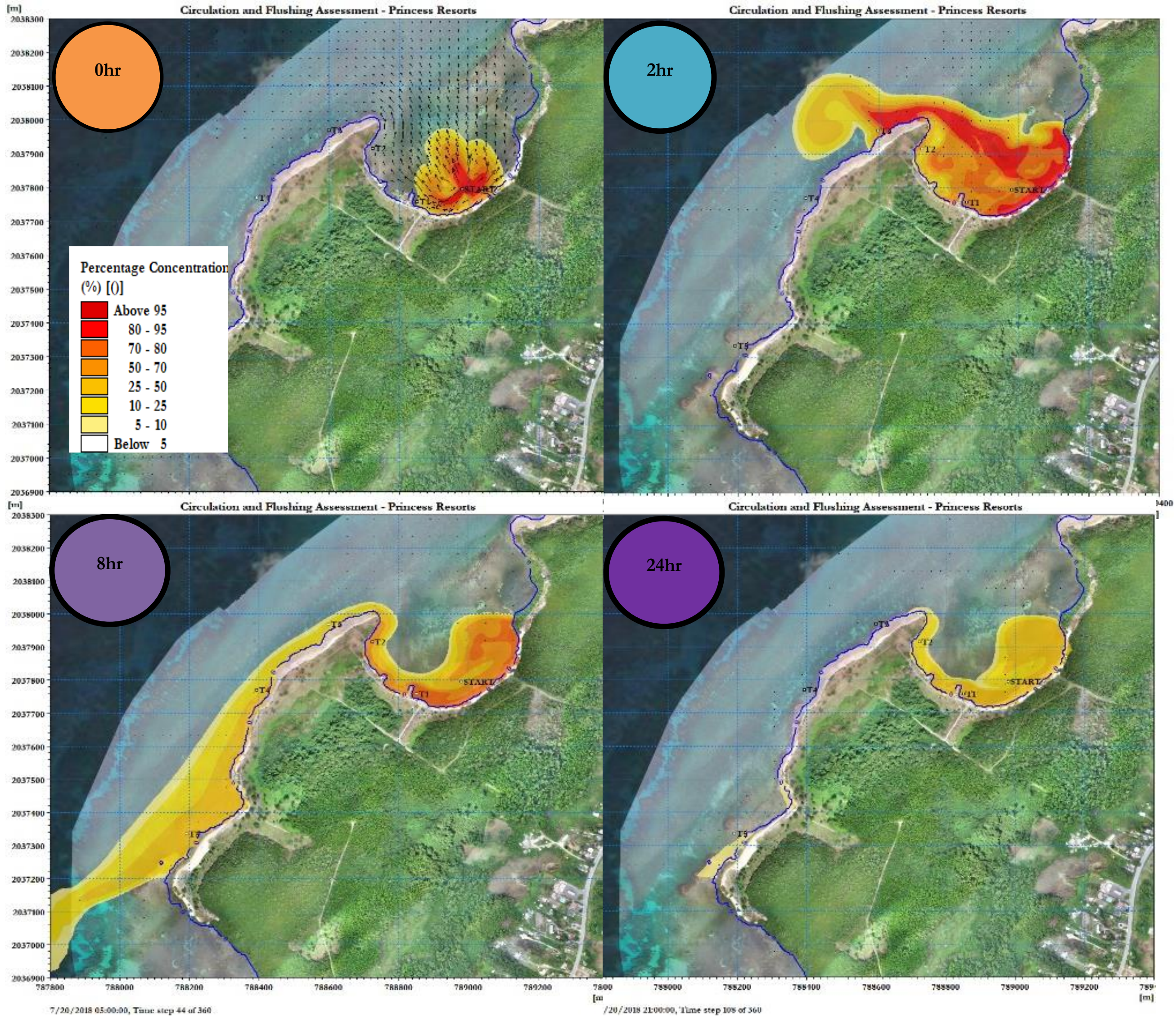


Figure 7.1 Flushing conditions after 24 hours with the introduction of a pollutant in Existing Conditions

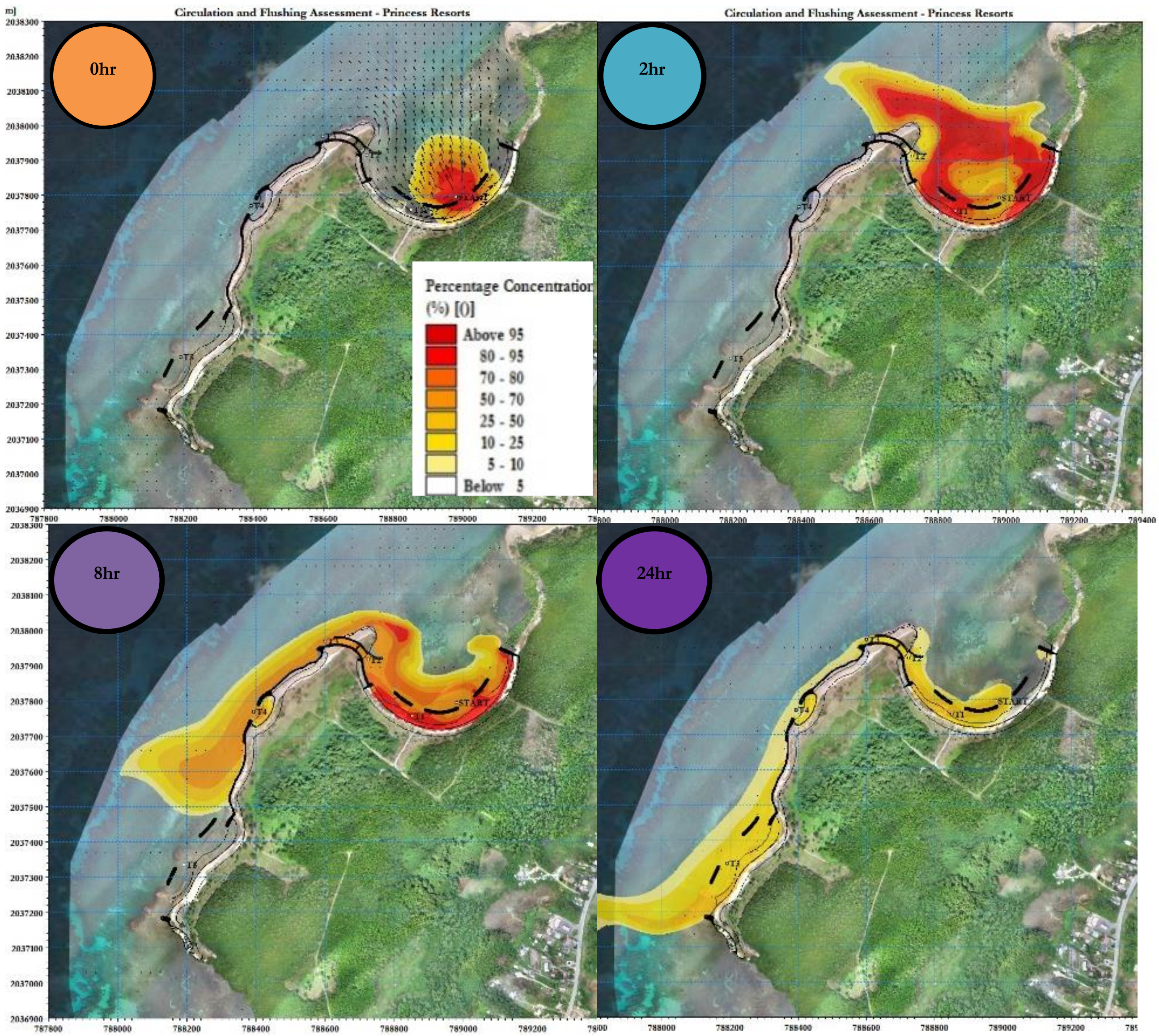


Figure 7.2 Flushing conditions after 24 hours with the introduction of a pollutant in Proposed Conditions

7.2 Response to Swell Wave Climate

The same swell event (1-10 Mar 2009) used previously was used to assess the structures' performance under swell conditions. The resulting waves (Figure 7.3 and Figure 7.4), currents (Figure 7.5 and Figure 7.6) and sediment movement (Figure 7.7) with the structures in place were assessed.

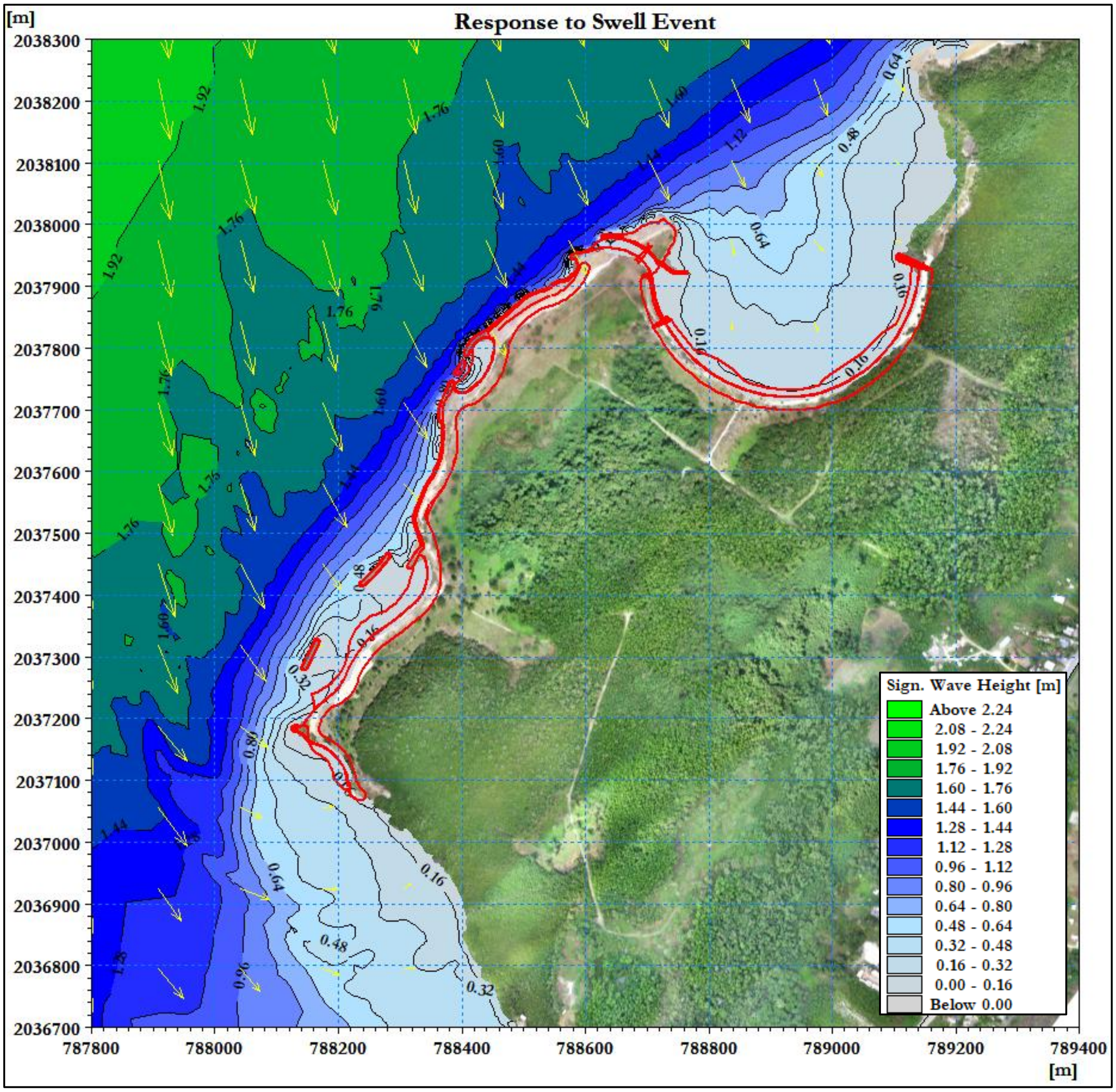


Figure 7.3 Concept Response to Swell Waves

The plot above shows wave conditions at the site at the peak of a swell event. The depth-limited bay of Hotel 1 had wave heights ranging from only 0.16-0.32m under a swell. In the location of the Sea Rooms the wave height is approximately 0.32m. Along the rocky shoreline the waves are still very large and get up to 1.44m. The addition of the breakwaters at Hotel 4 reduces the wave energy from 0.8m to maximum of approximately 0.32m.

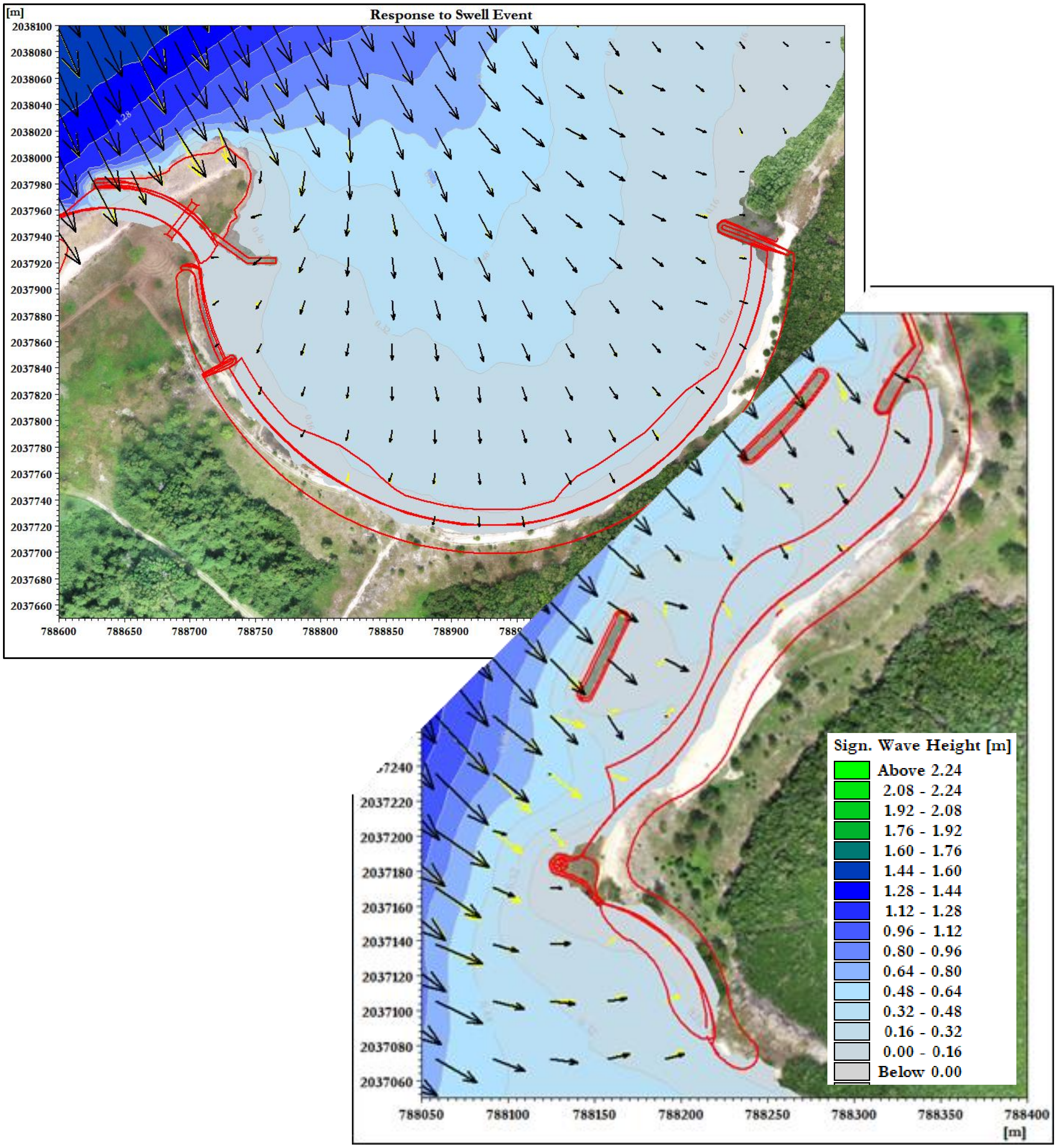


Figure 7.4 Swell wave response at the Hotel 1 and 4 (Yellow Arrows Proposed Conditions – Black Arrows Existing Conditions)

The plot above shows a close-up of wave conditions at Hotel 1 (top) and Hotel 4 (bottom) at the site at the peak of a swell event. The yellow arrows show the magnitude (wave height) and the direction under proposed conditions, while the black represents existing conditions. For Hotel 1, these arrows are similar and represent no change in ambient conditions. For Hotel 4 the waves are smaller (shorter arrows). The change in arrow size only occurs in the vicinity of the structures.

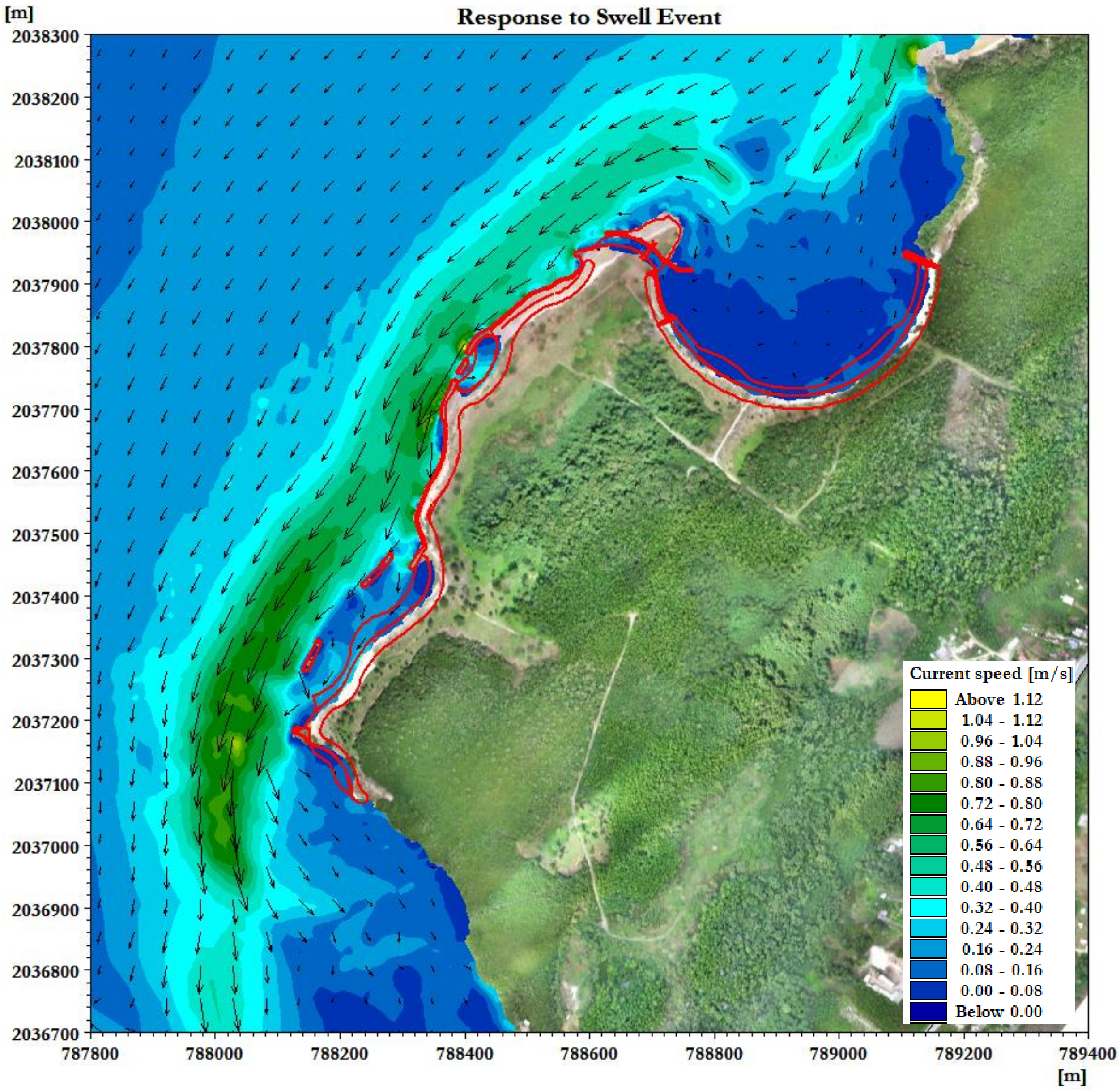


Figure 7.5 Current speeds at the peak of the swell event

Current conditions at the peak of a swell event in the plot above show a dominant westerly movement. In a swell event the wave-driven currents increase to above 0.88m/s in some sections of the rocky shoreline. Currents along the bay of Hotel 1 are still very small (i.e. below 0.16m/s). The introduction of the flushing channel does not significantly change the currents.

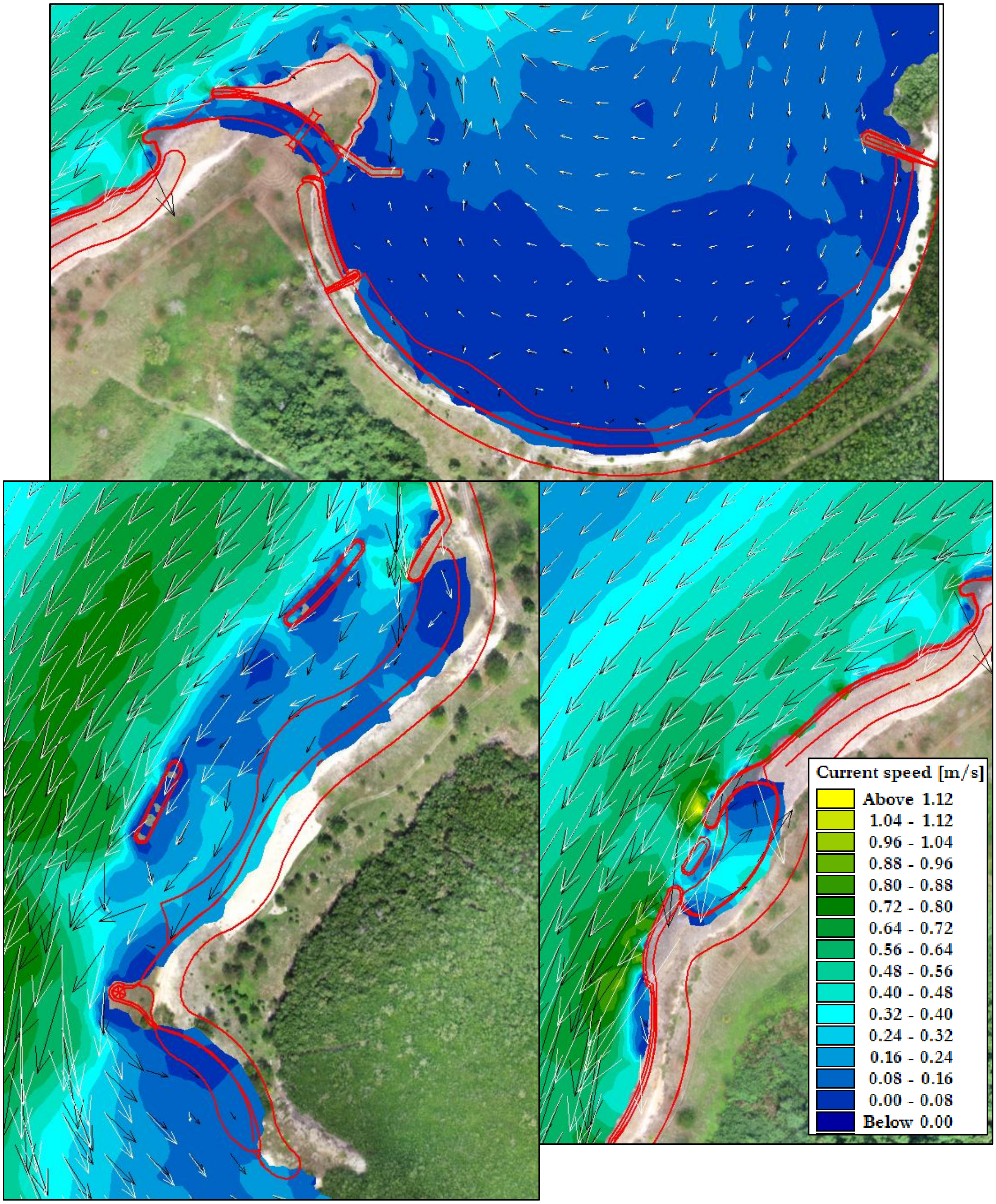


Figure 7.6 Current speeds at the peak of the swell event at Hotels 1, 2 -3, and 4 (black arrows proposed conditions - white arrows existing conditions)

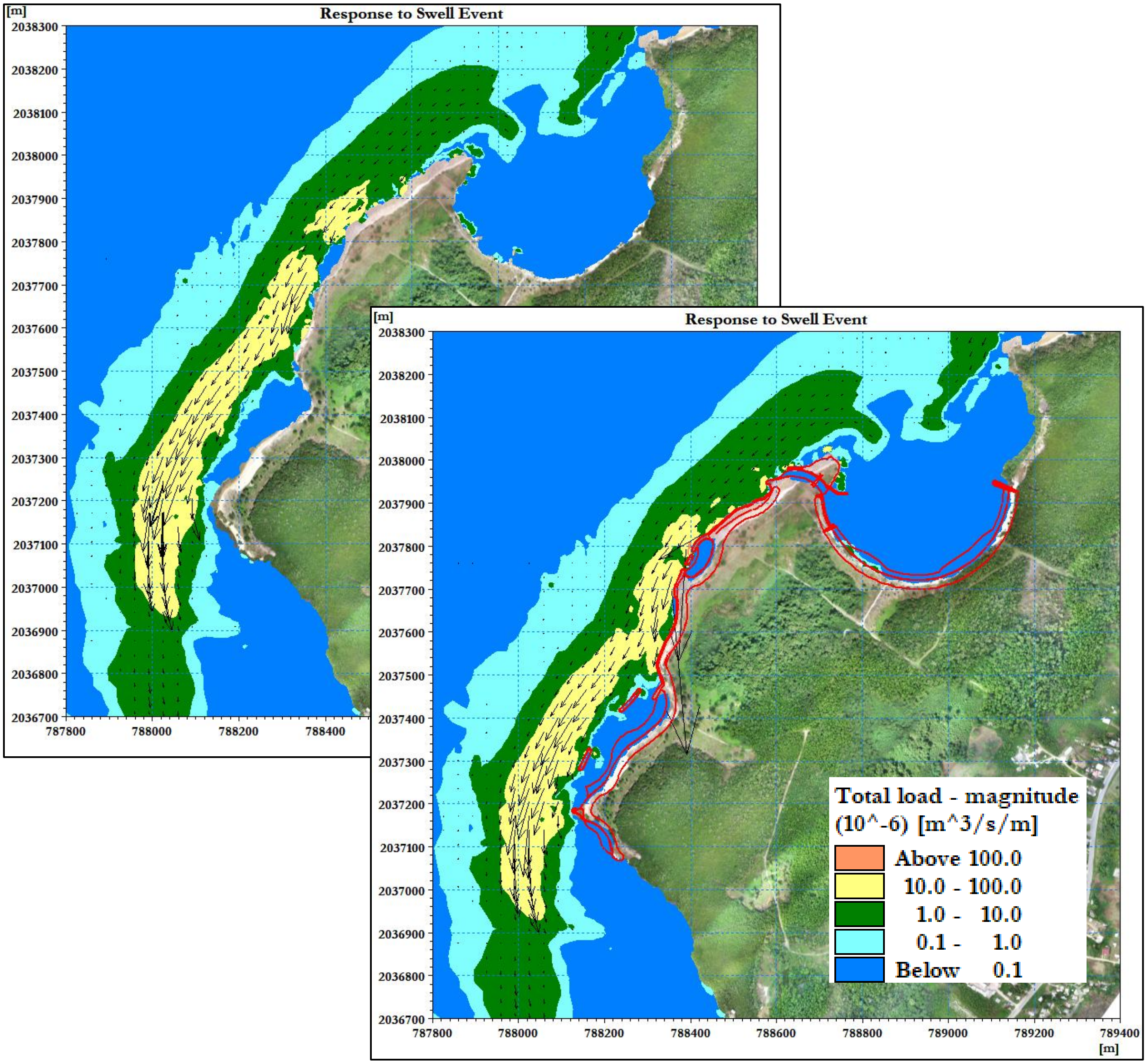


Figure 7.7 Potential total suspended sediment in existing and proposed conditions at the peak of swell

Sediment movement at the peak of a swell event is shown above. The flow of sediment follows the flow of currents in the area. The plot shows the total load, which refers to the volume of sediment moving at a given time. For instance, $100m^3/s/m$ represents a total of $8.64m^3$of sediment moving in a day. This can be visualized as approximately half a load in a standard truck.

The following points can be made with regards to the modeling results:

- At Hotel 1 without the breakwater in place, the wave climate remains relatively unchanged.
- Along Hotels 2 and 3, the swimming area created has wave heights less than 0.8m during the swell event, while the ambient wave conditions are greater than 1.7m. This represents an almost 50% reduction in wave energy.
- Along Hotel 4, the two breakwaters added to reinforce the reef reduce wave heights to 0.3m. These are small waves during swell conditions.
- Figure 7.4 shows wave directions in existing and proposed situations. The length of the arrows is relative to the height of the wave. For Hotel 4 the yellow arrows of the proposed conditions change direction and reduce in length. This shows that waves are reducing in height, and the arrows indicate potential for the formation of a salient. With this impact the nourishment shape is as shown to allow for the spreading out of sediment.
- Currents along the site did not change significantly. Slow currents are created within the channel.
- As the current direction and magnitude did not change significantly, the sediment transport remained almost unchanged.
- It can be concluded that the changes are only within the vicinity of the structures.

7.3 Response to Hurricane Waves and Storm Surge

Hurricanes have the potential to cause flooding from storm surge as well as damage due to high energy waves. Storm surge levels are related to the increase in sea level due to the low-pressure system caused by a hurricane. Under existing conditions, the site would be under more than 1.5m of water in the 50-year hurricane event. However, by increasing the ground level as recommended to facilitate drainage, the development is protected from hurricane-related flooding.

The site is a natural flood plain so unless the protection is extended, the overflow seen at the east and west ends of the property will continue. Such extensive protection is not necessary, however, as the proposed dyke road will keep the developed area safe and an elevated roadway from the resort to the North Coast Highway will manage excess during a storm.

The rocky shoreline of Hotels 2 and 3 is at the greatest risk to wave damage during a hurricane. The wave heights at this section of the development are greater than 2.5m. This has implications for the design of the revetment and groynes proposed for the area, as stronger waves will require larger structures to reduce the associated risks. The reef system at Hotel 4 reduces the wave heights that reach the shoreline at this section of the property. Reinforcing the reef with two breakwaters further reduces the height of waves reaching the proposed sandy beach area. Similarly, at Hotel 1, wave heights are reduced by the reefs to 1-1.5m. The lower wave energy in this area also promotes the use of honeycomb structures for the three proposed breakwaters, which promote marine habitat development. While the bay is somewhat sheltered, the stronger waves are toward its eastern end where the proposed Sea Rooms are to be located. Care must be given to the placement of the footprint of these buildings.

Figure 7.8 and Figure 7.9 show the surge levels and wave heights

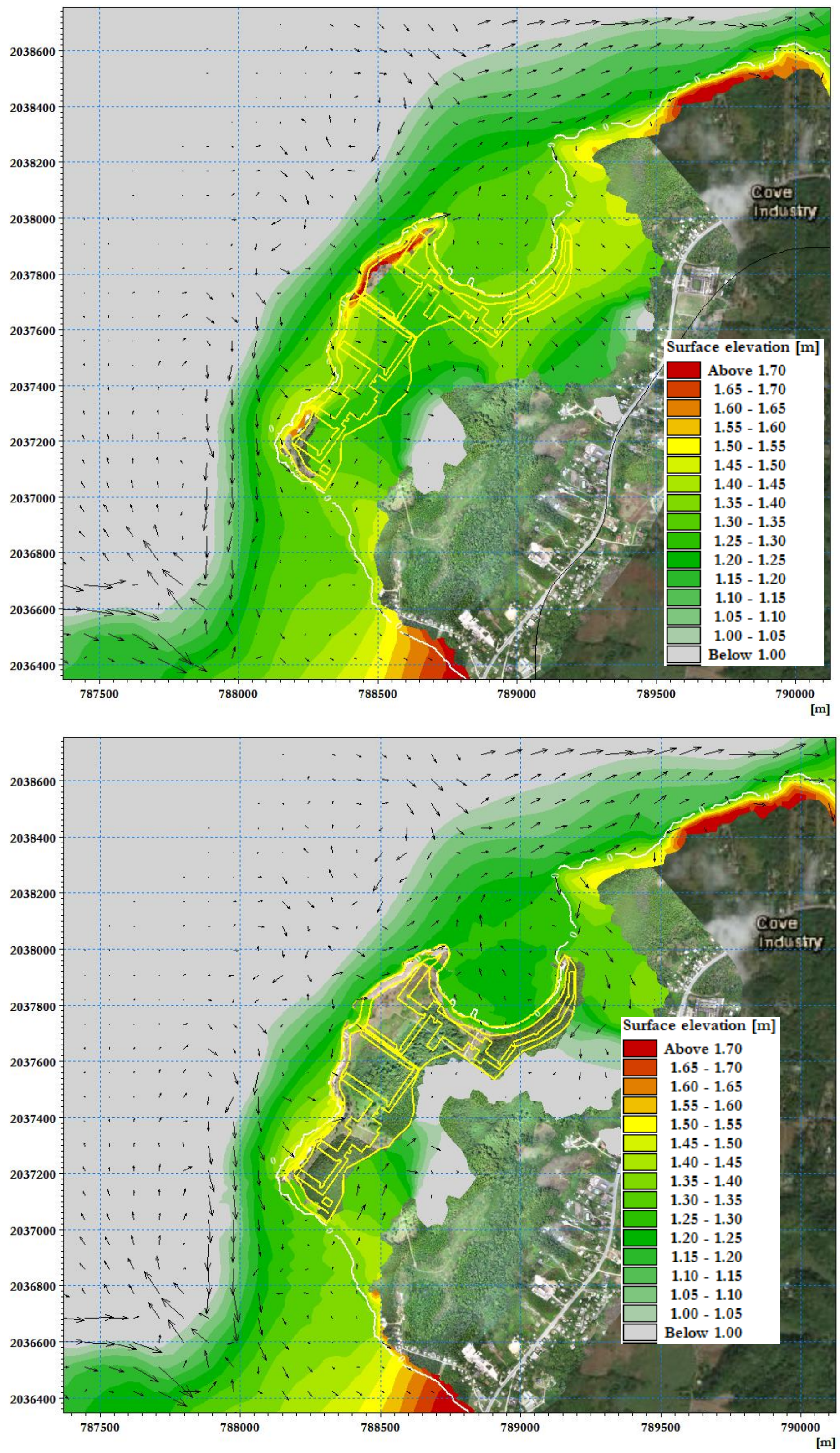


Figure 7.8 Static storm surge levels (50-year event) with no structures in place (top) and with structures in place (bottom)

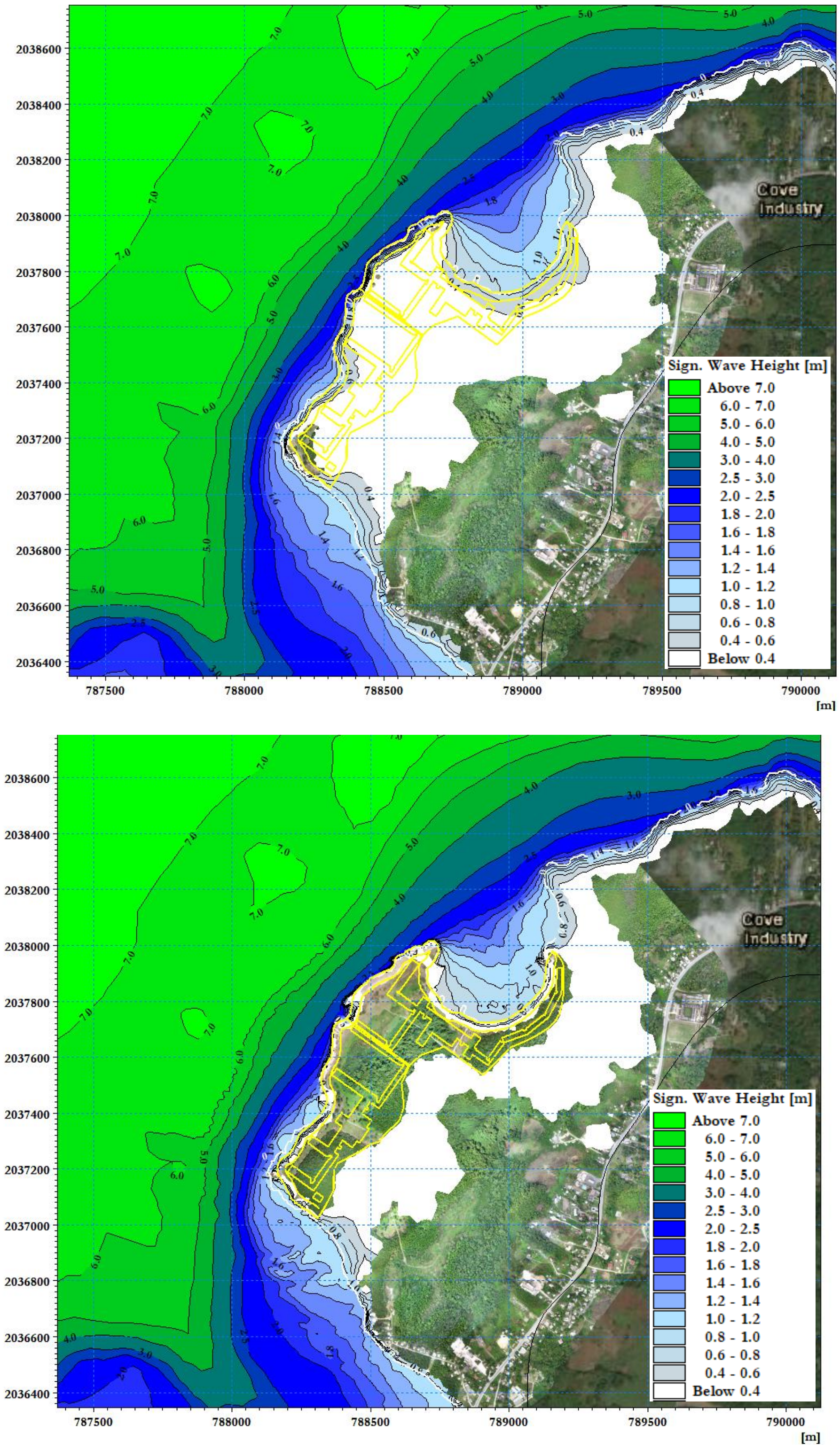


Figure 7.9 Significant wave heights under the 1 in 50-year event with no structures in place (top) and with structures in place (bottom)

7.4 Downdrift Changes

Downdrift changes to adjacent shorelines are not expected to occur with the implementation of the works. Sediment flow in the area is predominantly to the west and the proposed groyne and breakwaters are well outside of the major transport zones. As seen in section 5.7, the bulk of sediment transport at Hotel 1 occurs within the first 20m offshore while for Hotels 2 and 3 the bulk of transport occurs approximately 70m offshore. The proposed groynes will end 15m and 35m from the shoreline which is before the peak of the transport occurs. Additionally, potential sediment transport caused by a swell event remained relatively unchanged with the structures in place compared to the existing conditions. Therefore, there is no significant impact expected to occur for downdrift shorelines (i.e. to the east).

7.5 Climate Change Impacts

The design calculations for the proposed concepts included projections for sea level rise to the year 2100. Even with the anticipated increase in sea levels the site would not be flooded in 50-year storm event.

8 Structural Engineering Design

This section of the report presents considerations and calculations in the design of the structures. Boundary conditions used for the calculations are presented first, after which the sizing of the armour stones is described.

8.1 Design Parameters

For the design of the structures, the maximum wave heights incident on each structure for a different wave forcing were extracted from the MIKE21 model. For this design, we must consider the wave conditions on the structures under:

1. Daily wave conditions;
2. Swell events; and
3. Hurricane conditions.

The use of a return period or design event such as the 1 in 50-year or 1 in 100-year essentially defines the kind of design conditions that will, on average, occur or be exceeded once every 50 years or every 100 years. It is important to understand risk and consider the chance of occurrence of a particular storm condition during the lifetime of a structure so that the associated risk of damage can be understood.

Table 8-1 gives the exposure risk (probability) over a project lifespan for different return period events. For example, a project lifespan of 50 years (Design Life =50) has a 99% chance of a 1:10-year event occurring and a 39% chance of a 1:100-year storm event occurring in 50 years.

Table 8-1 Probability of occurrence for various return periods and design life

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

For submerged structures, the wave forces become less as the water level increases so designing for a 1 in 50-year storm versus a 1 in 100-year storm does not have much impact on the resulting stone sizes. However, for emergent structures that become exposed to wave breaking, cost savings can be made by adopting a lower return period as the design criteria. Note that the plan and sections presented in Appendix C were designed using the 1 in 50-year storm event, which is usually the recommended design return period for the design of coastal engineering structures.

Wave conditions for use in the design were extracted at points where structures are proposed as shown in Figure 8.1. Table 8-2 summarizes the design wave conditions for each of the structures. The rows are colour coded to correlate with the coloured points on the structures in Figure 8.1.

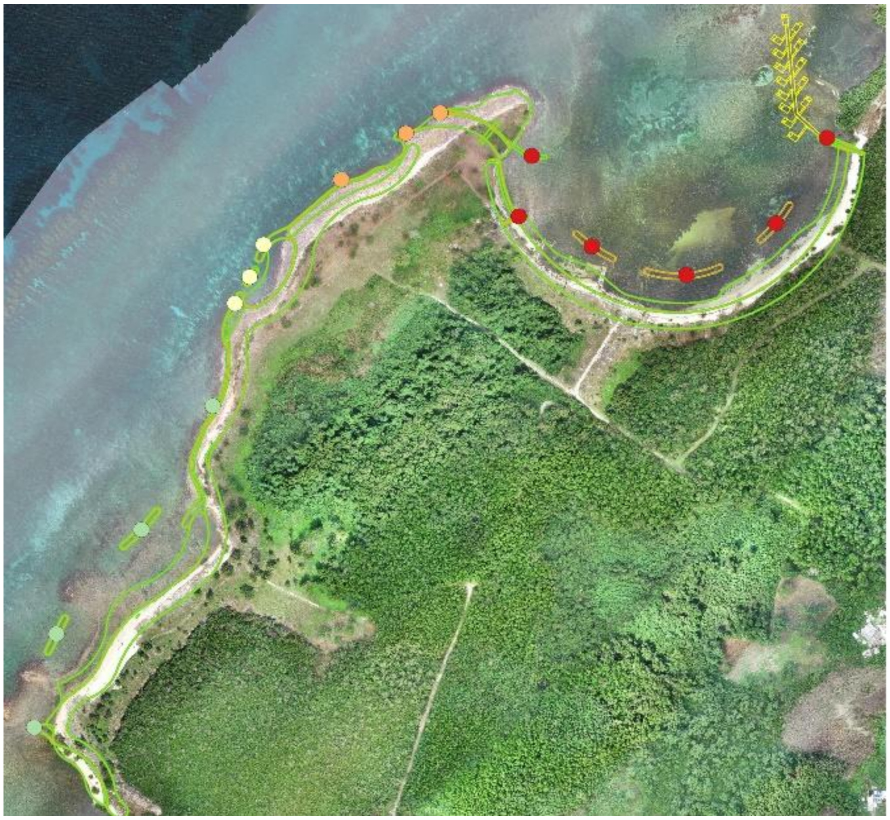


Figure 8.1 Points used for wave conditions extraction

Table 8-2 Wave conditions (1 in 50-year return period) used in the coastal engineering designs

Significant Wave Heights, Hs(m)	*Peak Wave Period, Tp (s)*	*Wave Direction*
1.0	11.46	*NNW - NNE*
1.5	11.46	*NNW*
2.5	11.44	*NW*
1.5	11.46	*WNW*

8.2 Engineering Design of Coastal Protection

This section of the report presents the structural design for the recommended coastal enhancement measures including (i) calculations to specify suitable revetment elevations to limit overtopping during storm conditions and (ii) calculations for the stone sizes that would be stable during a 1 in 50-year hurricane event.

8.2.1 Structural Elevation for Revetment and Groyne

Overtopping occurs when waves run-up or crash into structures along the coast. A major consequence of this overflow is flooding at the site. Overtopping is therefore a major concern where wave heights are not reduced before reaching the site. This occurs along the rocky shoreline of Hotels 2 and 3. Structures along this stretch of shoreline therefore include:

1. Revetment for the perched beach;
2. Cove beach using groynes.

Overtopping rates are determined using swell and hurricane conditions as previously described. Results shown in Table 8-3 are for a structure with a crest at 2.5m above MSL and a 1:2 slope. This ensures the specified structure heights are sufficient to reduce overtopping rates and avoid structural damage under operational and extreme events. Wave overtopping along the shoreline of Hotels 2 and 3 during hurricanes should be less than 10-20l/s for people to stay safe (Figure 8.2). To prevent damage to the structure behind the revetment, the overtopping should be further reduced to less than 5-10l/s (Figure 8.3).

Table 8-3 Overtopping rates for swells and hurricanes

	Overtopping Rate	
	Swell Event	Hurricane Wave
Hotels 2 -3: Revetment and Groynes *Slope of 1:2* *Crest Height: +3.0m MSL*	**0.417 l/s/m**	**1 l/s/m**
Hotel 2 – 3: Cove Beach Groyne Slope of 1:2 Berm of 4m at +1m MSL Crest Height: +2.5m MSL	**0.565 l/s/m**	**8 l/s/m**

Please note that the groynes would have to have almost 17m wide to achieve the limited overtopping as calculated above. The design was therefore optimized by the following means:

1. The groyne was stopped on land and set to the elevation of the revetment (ie. +3 MSL)
2. The submerged breakwater was widen to create more breaking and reduce the run up potential of the waves.

Plan and sections of the final coastal enhancement and drainage plans are included in Appendix C.

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 2000 2000 2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Figure 8.2 Limits for overtopping rates for people and cars behind revetment (EurOTop 2016)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; H_{m0} > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; H_{m0} > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Figure 8.3 Limits for overtopping rates structural damage behind revetment

8.2.2 Armour Stone Sizes for Revetment, Breakwaters and Groyne

In recent years, shoreline armour structures have been designed to be built with boulders or, where boulders are unavailable or less cost effective, concrete armour units. Based on our research, boulders of suitable size and strength are available in Jamaica. Sizing rocks for armour structures depends primarily on the stability of the rocks under wave forcing. The revetments, groynes and breakwaters were designed using an in-house spreadsheet for determining armour stone sizes from the design wave conditions based on work by van der Meer (2003) and Van Gent (2004). These two design methods were selected because they specify formulations for structures in shallow water where the water depth is less than three times the wave heights. This shallow water condition would exist at the base of the revetment when the static water level is increased under the influence of a hurricane.

The following assumptions are also made for the calculations:

- Stones will have a minimum density of 2500kg/m^3;
- A total of two stones can be displaced per meter run after the passage of a 1 in 50-yr hurricane;
- The slope of the structure revetment is 1(H):1.5(V);
- The revetment will have a permeability of 0.3 and will be constructed with a stone filter layer;
- Storms are assumed to have a duration of 8 hours.

With these assumptions and the wave conditions previously calculated, stone sizes were calculated. From the output of these calculations we found that the van der Meer (2003) formulation gave larger stone sizes than the van Gent (2004) formulations. We selected the van der Meer results, which are somewhat more conservative, and the specifications for the calculations are shown in Figure 8.4.

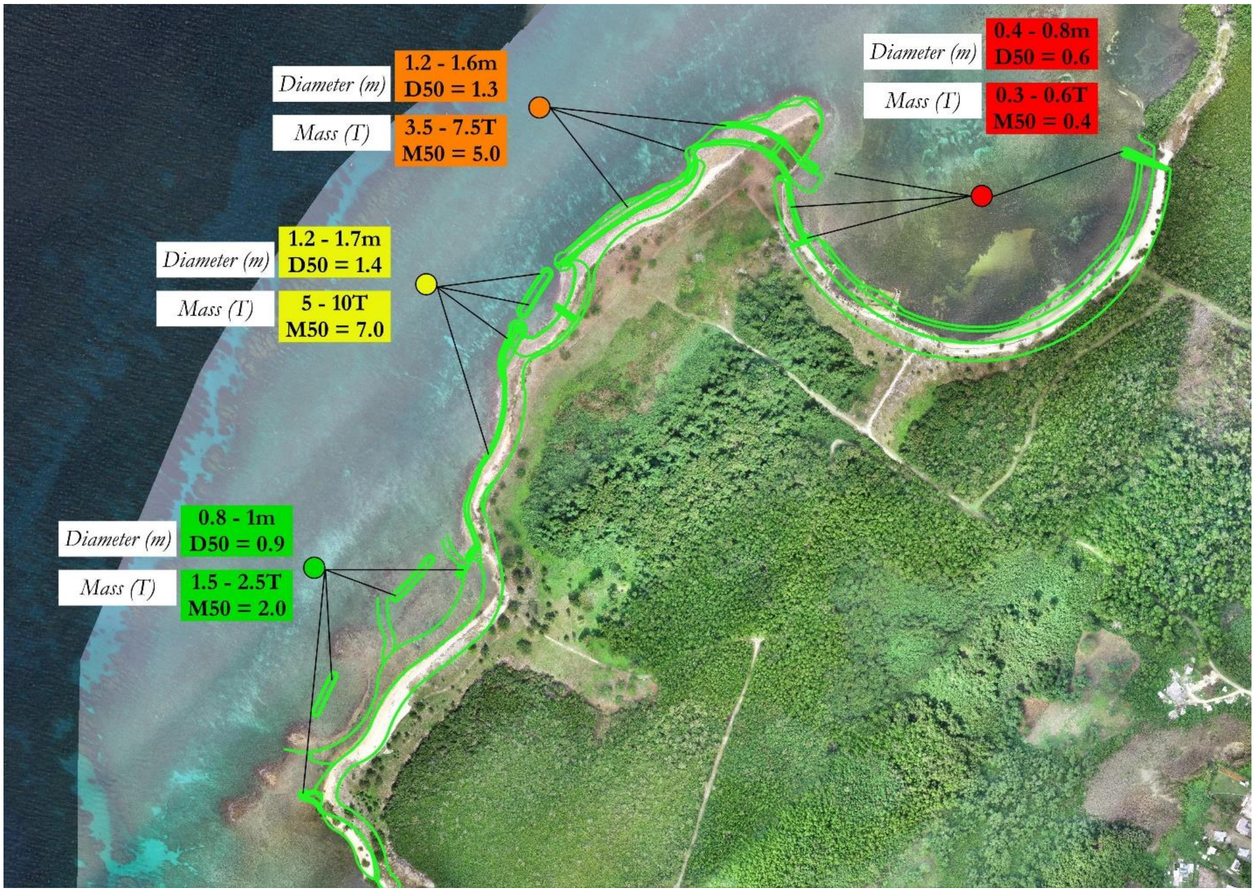


Figure 8.4 Stone sizes at the sections of the site

8.2.3 Structural Elevation for Sea Rooms

The proposed sea rooms should be set at an elevation that limits the potential for flood under extreme events such as hurricanes. The protection is provided by the "airgap" according to the guidelines for piers and jetties (McConnell, Allsop, and Cruickshank 2004). The floor elevation should be set based on:

1. **Static Water Level:** This refers to the High Astronomical Tide (HAT), Sea Level Rise (SLR), and Inverse Barometric Pressure (IBR))
2. **Height of the Wave Crest** (η_{MAX}) – in deep water the water elevation can be determined using an equation as follows:

$$\eta_{MAX} = \frac{H_{MAX}}{2} \exp\left(\frac{2\pi}{L_m}\frac{H_{MAX}}{2}\right)$$

   However, the depth of water at the Sea Rooms is shallow so MIKE21 and sBEACH models were used to obtain to chest elevation of the waves.

3. **Allowance for Service Entities** (i.e. utilities, sewage and drainage, etc.).

Floor levels were developed for hurricane and swell wave conditions. Hurricane conditions would cause the greatest increase in the static water level and, combined with the large waves, there is potential for extreme flooding to occur. However, swell waves can also cause a notable increase in the wave heights. Therefore, we took a two-level approach to setting the Sea Room floor elevations: the lower level is closer to sea level and could be a platform for entering and exiting the sea. The building elements placed on this lower platform should be mobile as this level would be flooded during a hurricane (Figure 8.5). The upper level would not be flooded during hurricane conditions and could house more permanent items (Figure 8.6). Figure 8.7 is a sketch showing the two-floor approach described above.

Table 8-4 Recommended floor levels

Hazard	*Components*	
Hurricane	• Static water level	• 0.94m
	• Height of wave crest	• Variable: 1.13-1.26m
	• Allowance for services	• 0.6m (~2ft)
Hurricane Flooding Protection: +2.75m MSL		
Swell	• Highest astronomical tide	• 0.25m
	• Height of wave crest	• Variable: 0.3-0.4m
	• Allowance for services	• 0.6m (~2ft)
Swell Wave Flooding Protection: +1.25m MSL		

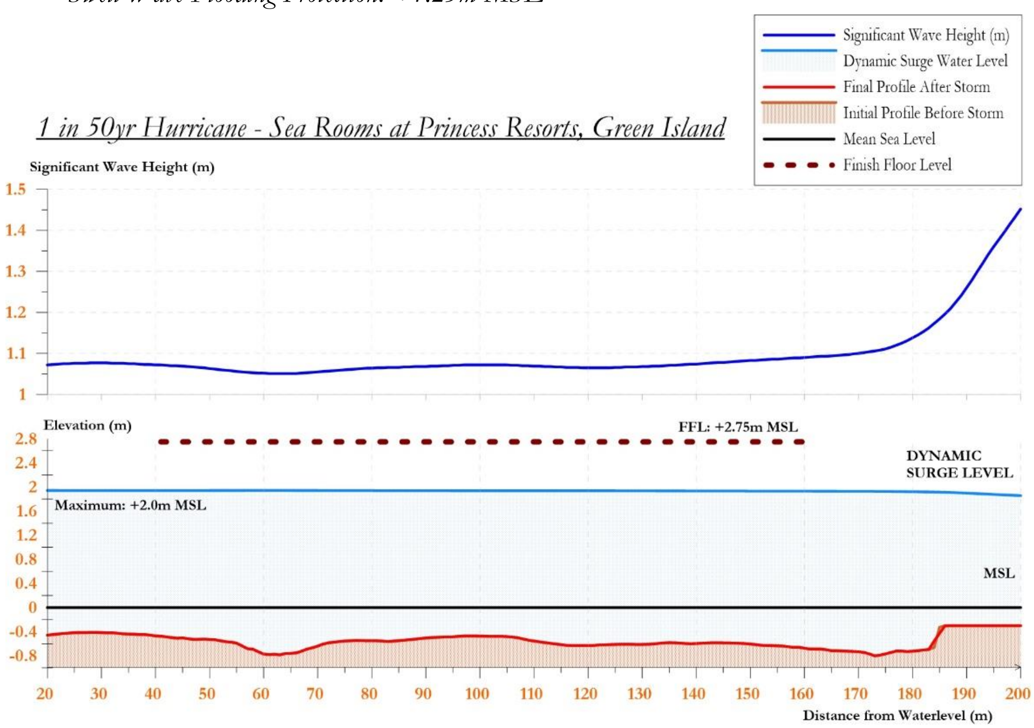


Figure 8.5 Water elevation along the footprint of the Sea Rooms under hurricane conditions

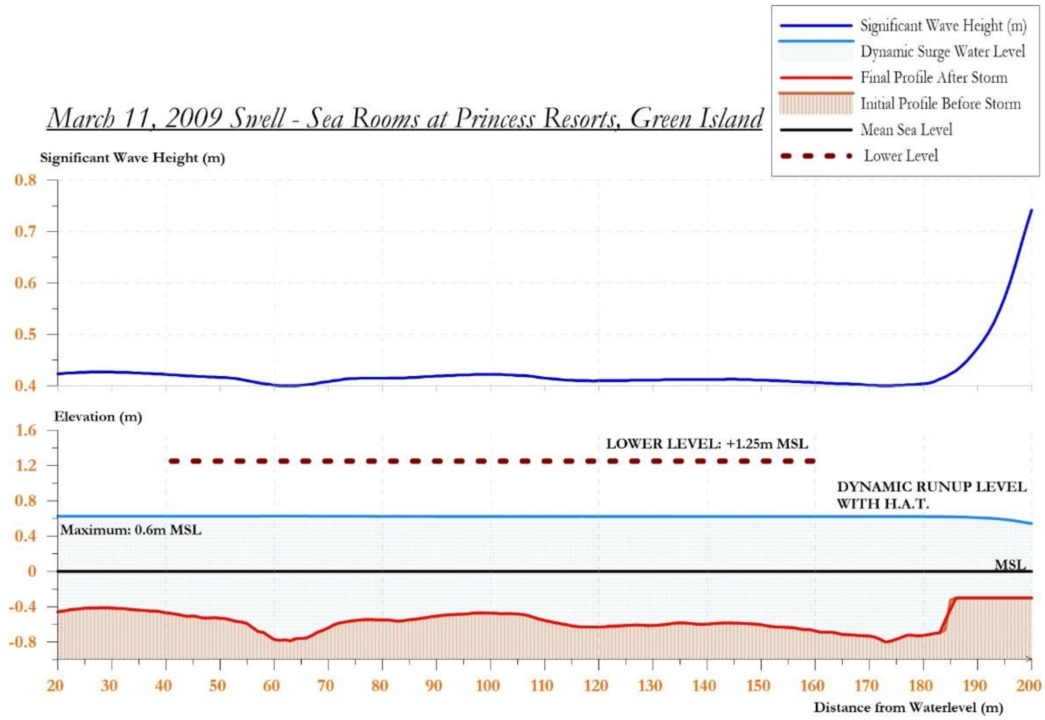


Figure 8.6 Water elevation along the footprint of the Sea Rooms under swell conditions

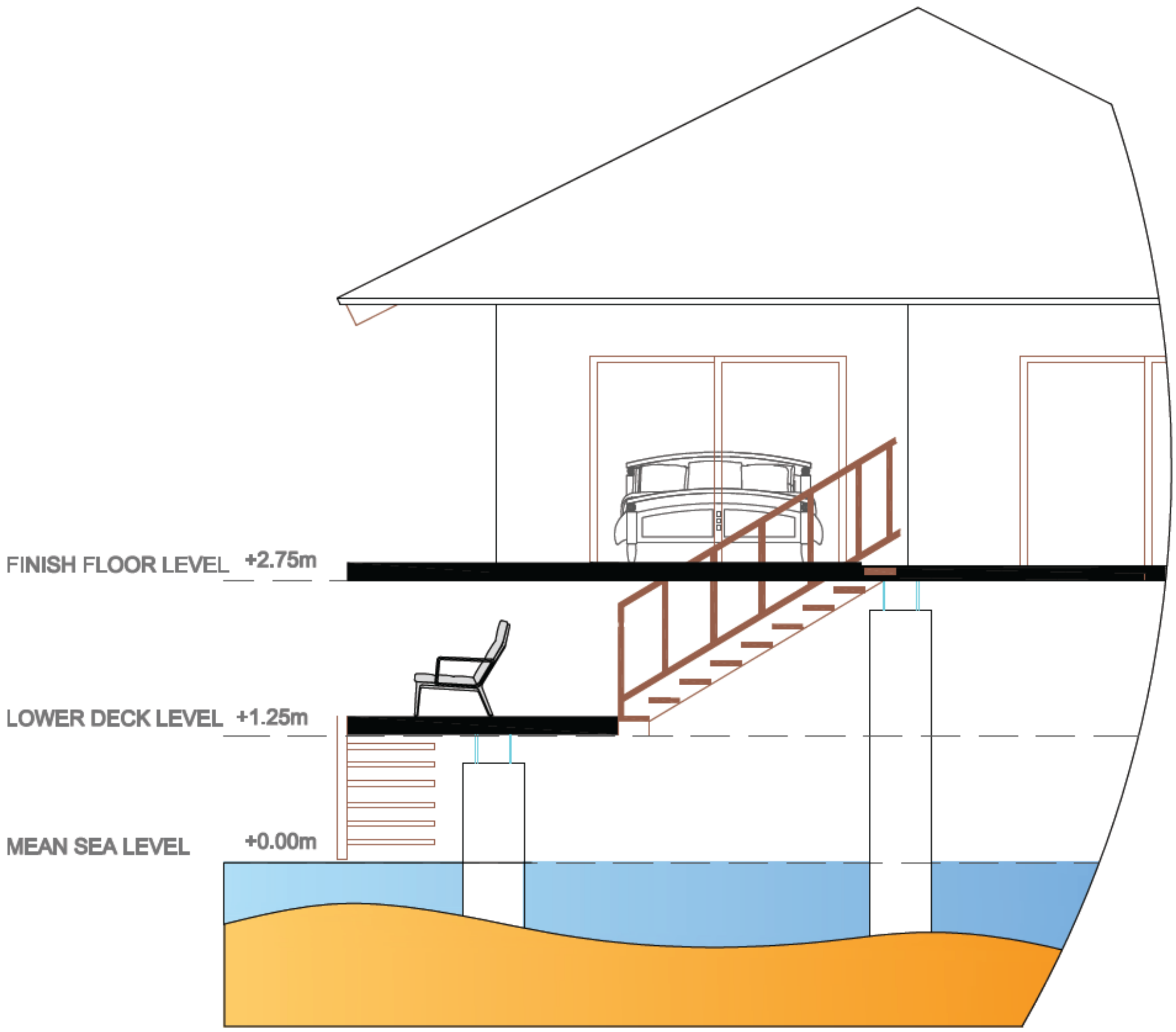


Figure 8.7 Recommendation floor levels for Sea Rooms

8.3 Preliminary Quantities

Preliminary quantities are provided below.

Figure 8.8 Preliminary quantities

9 Impacts and Mitigation

This section addresses impacts from the construction and operational phases of the project. It presents the overall scope of the project as well as mitigation measures proposed to offset the risks.

9.1 Scope of Shoreline Works

The scope of the works for the project as shown below.

Activities	*Area of footprint below sea level*	
Hotel 1 • -1.5m MSL flushing channel through western headland • Two groynes at +2m MSL • 500m long nourished sandy beach at a 1:14 slope • 79m of shoreline nourishment with a revetment at +2.8m MSL • +2m MSL groyne to be used as a sport jetty	2049 m^2	*0.5 ac*
Sea Rooms	3300 m^2	*0.81 ac*
Hotels 2 and 3 • Perched beach at +2.8m MSL retained by a revetment at +2.8m MSL • 130m long pocket beach nourished at a 1:14 slope. • Two spur groynes at +3.0m MSL • One submerged breakwater at MSL	2789 m^2	*0.69 ac*
Hotel 4 • Two submerged breakwaters at -0.3m MSL • 450m long nourished sandy beach at a 1:14 slope • One groyne at +2m MSL. • Nearshore dredging of up to 6500m^2	1789 m^2	*0.44 ac*

9.2 Construction Methodology

To complete the proposed works, the following equipment and materials will be needed.

Main Equipment:

- Medium sized excavators
- Front end loaders
- Small site boat

Main Materials:

- Boulders
- Sand sourced from either manufactured, dredged, or imported sand.

- Fill material
- Filter fabric

9.2.1 Coastal Enhancement Works

The expected sequence for the construction of the proposed works is as follows:

(1) Seagrass removal;
(2) Placement of marine blocks and fish havens;
(3) Coral removal and relocation;
(4) Creation of access route for equipment;
(5) Construction of temporary access road with core stone;
(6) Placement of boulders along the footprint of proposed groyne (revetment, groyne, breakwaters);
(7) Clearing of rocks from seabed;
(8) Placement of sand nourishment to required grade.

All work will be done from land using *land-based equipment*.

9.2.2 Sea Rooms

The construction of the 40 Sea Rooms requires the driving of piles offshore. This can be done in several ways, but the major limitation is the depth of the nearshore. The water depth in this location is only 0.6m on average and using a barge would not be feasible. We propose, instead, that a construction pad be used for the duration of the construction. The pad will then be removed after construction is complete.

The construction will span several months and if a solid construction pad is used, some stagnation of the water behind the pad is likely. Culverts would therefore be required to allow exchange of water (Figure 9.1). Figure 9.2 shows that a contaminant introduced behind a construction pad would reduce in concentration to below 10% in less than 21 hours. This contaminant could be all the suspended spoils of construction including silt and oil. Figure 9.3 shows the flushing that two 15m wide culverts would provide.

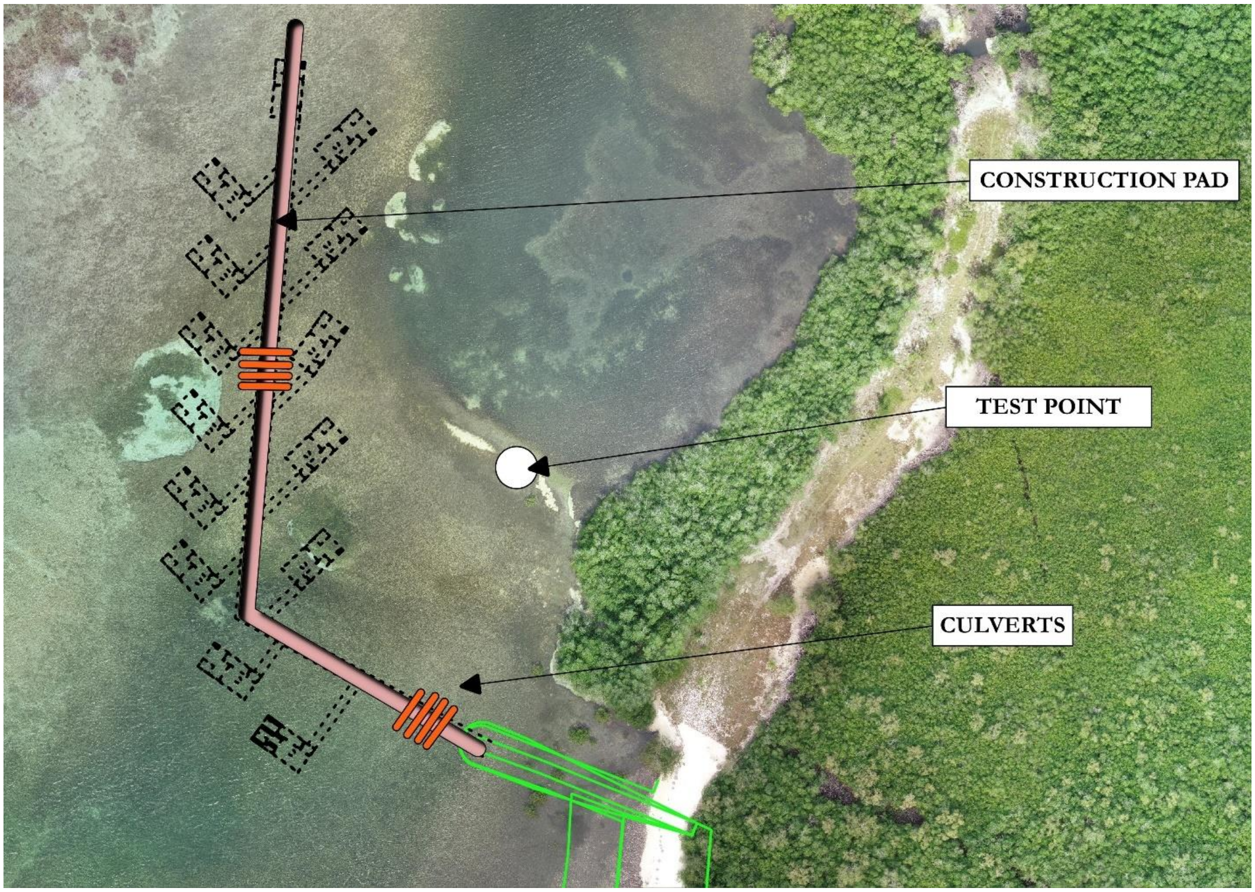


Figure 9.1 Schematic showing the layout of construction pad and culvert

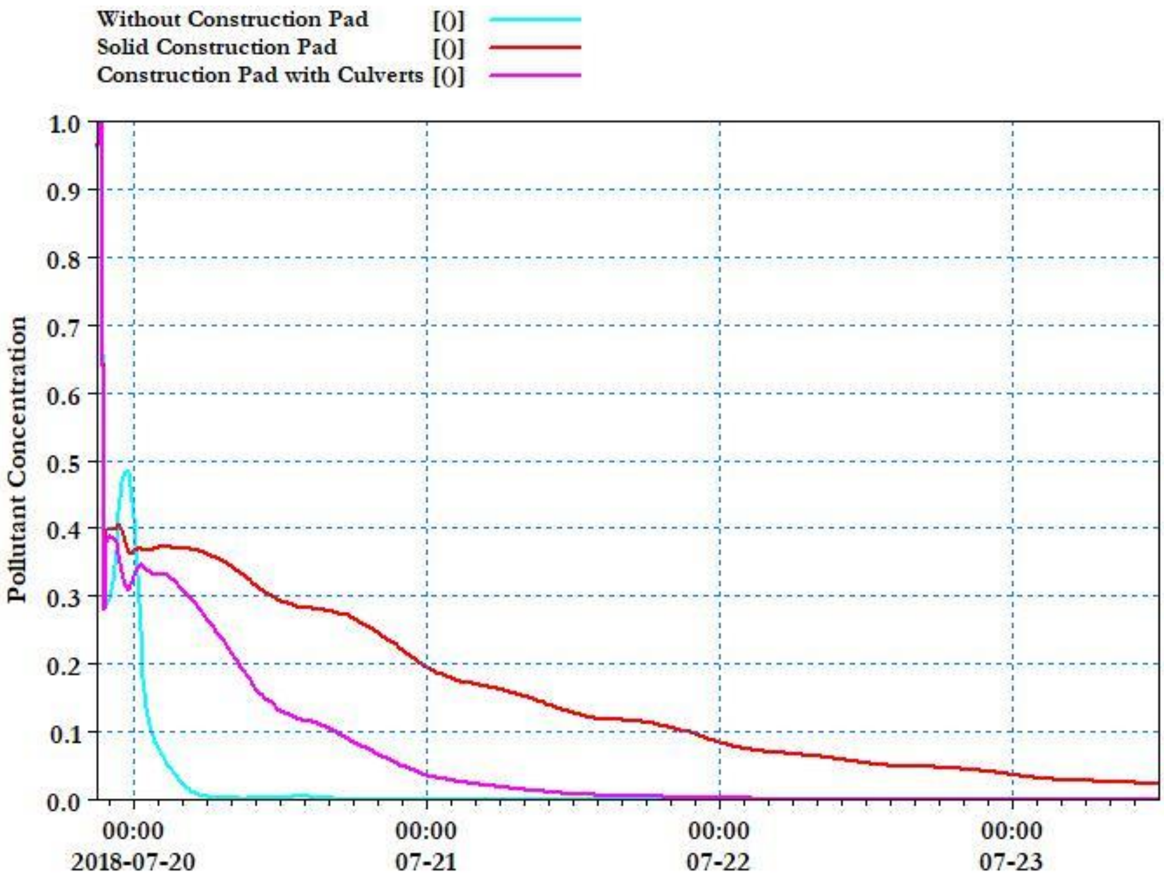


Figure 9.2 Flushing potential with and without the construction pad at the test point shown above

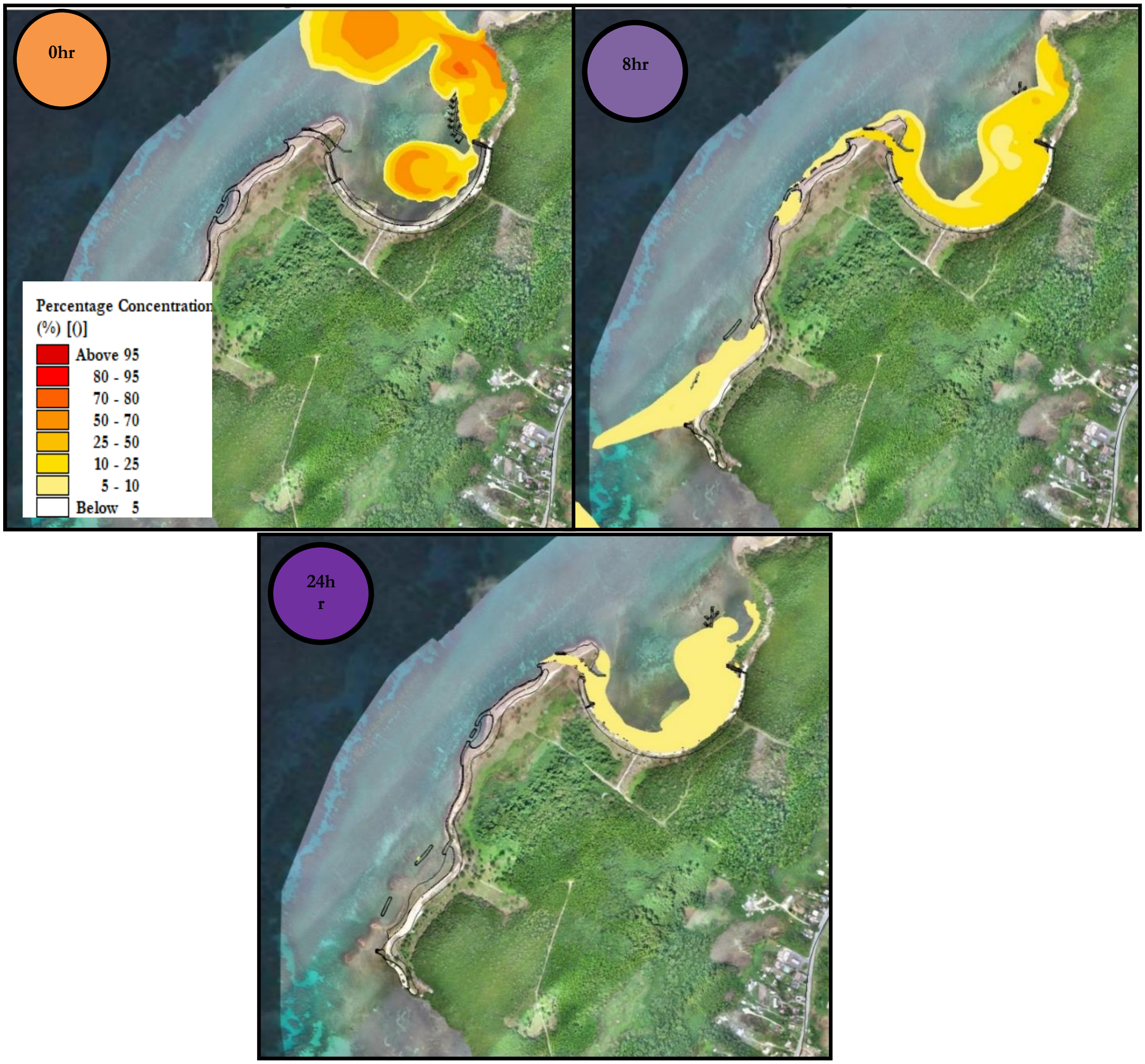


Figure 9.3 Flushing provided by two 15m wide culverts

9.3 Construction Impact Assessment and Mitigation

Table 9-1 outlines the impacts of the construction activities on the physical environment.

Table 9-1 Proposed construction activities and the impacts and mitigations

Construction Activities	Potential Impact	Proposed Mitigation
Construction of temporary access road with core stone.	-Construction debris -Increased turbidity	- Turbidity barriers - Removal of debris - Boulders should be washed before placement.
Placement of marine blocks and fish havens	-Loss of habitat -Increased Turbidity -Destruction of habitat within the footprint of the development	- Coral recruits should be relocated before construction. - Placement of blocks with crane to limit impact to only the footprint of individual marine blocks - Marine block structures (holes and rugosity) increase the available area for habitat - Fish havens will provide additional diverse habitat - Turbidity barriers
Placement of boulders along the footprint of proposed groynes and breakwaters.	-Loss of habitat -Increased turbidity - Destruction of slow moving and sedentary intertidal and subtidal flora and fauna within the footprint of the development	- Coral recruits should be relocated before construction. - Relocation of slow moving and sedentary fauna from footprint before the start of construction each day. -Rock structures provide habitat. -Boulders should be washed before placement. -Turbidity barriers
Packing of groyne, revetment and breakwaters boulders to the design grade and elevation.	-Increased turbidity	-Boulders should be washed before placement. -Turbidity barriers
Clearing of rocks from seabed.	- Destruction and removal of pavement habitat to create swimming area	-As much as possible, rubble should be removed manually -Rock structures provide habitat
Placement of sand nourishment to required grade.	-Turbidity -Sedimentation of the area	-Turbidity barriers -Using sand with low silt content.

9.3.1 Smothering

Any area of seafloor that will be disturbed may experience smothering of small sedentary or slow-moving flora and fauna. All benthic resources in the footprint of the coastal structures and the beach area will be impacted negatively by the physical disturbance resulting from the deployment of boulders and blocks that make up the structures. To mitigate the effects, the benthic resources within the footprint of the structures will have to be relocated prior to construction. For the marine life outside of the footprint, turbidity barriers will be used during construction to prevent fine material from going offshore.

9.3.2 Turbidity

The dominant component of the sediment in the project area is sand. The deployment of boulders for the structure, the excavation of iron shore, 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.

Turbidity barriers will be used to lessen the spread of fines. A turbidity meter will be used to measure the turbidity outside of the construction area to ensure that turbidity readings are within the acceptable range as specified in the licenses.

9.3.3 Oil Pollution

There is the potential for fuel leaks or spills from equipment used for the construction, excavation and/or sand nourishment during refuelling or operation. Refuelling of the boat and sea-based equipment should only be done at anchor out at sea if the sea conditions are calm, otherwise, all refuelling should be done when docked at land. Appropriate refuelling equipment (such as funnels) and techniques should always be used.

There should be appropriate minor spill response equipment (for containment and clean- up) kept on site, including oil absorbent pads and disposal bags.

9.3.4 Post-Construction Debris

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

9.4 Operational Mitigation Measures

Potential negative impacts to benthic resources were examined in relation to the operational phase of the development and are described in the following sections. The impacts of the structures are localized. Localized impacts mean that the effects are only felt within the area of the project site.

9.4.1 Habitat Loss

Seagrass and corals will be lost in the footprint of the Sea Rooms as well as along sections of the shoreline of the bay. Corals within the footprint of structures to be constructed along the ironshore will have to be relocated to avoid negative impacts. A coral relocation study is to be conducted to quantify the area of impact and document the species present. It is proposed that a suite of measures be employed as mitigation along with compensation strategies. The aim of these measures is to provide enough habitat to ensure no net loss of habitat due to the development of the project. These measures are detailed in Table 9-2.

Table 9-2 Summary of impacts and mitigation and compensation measures

Impact	*Mitigation and Compensation Measures*		
Habitat Loss *Seafloor Area Affected: ~2.43acres*	**Electrified Coral Gardens**	20 Electrified Coral Nursery 50 Electrified Coral Pods	Spans approx. 1 acre
	Fish Havens	20 Fish Havens with High Rugosity	Spans approx. 1.5 acre
	Rubble coastal structures as habitat		

Fish Havens

It is recommended that 20 fish havens be constructed within the main bay at a site to referred to as "The Grand Reef" (Figure 9.4). Fish havens have been used successfully in Jamaica and other countries as habitat for juvenile fish and lobster in sheltered areas.

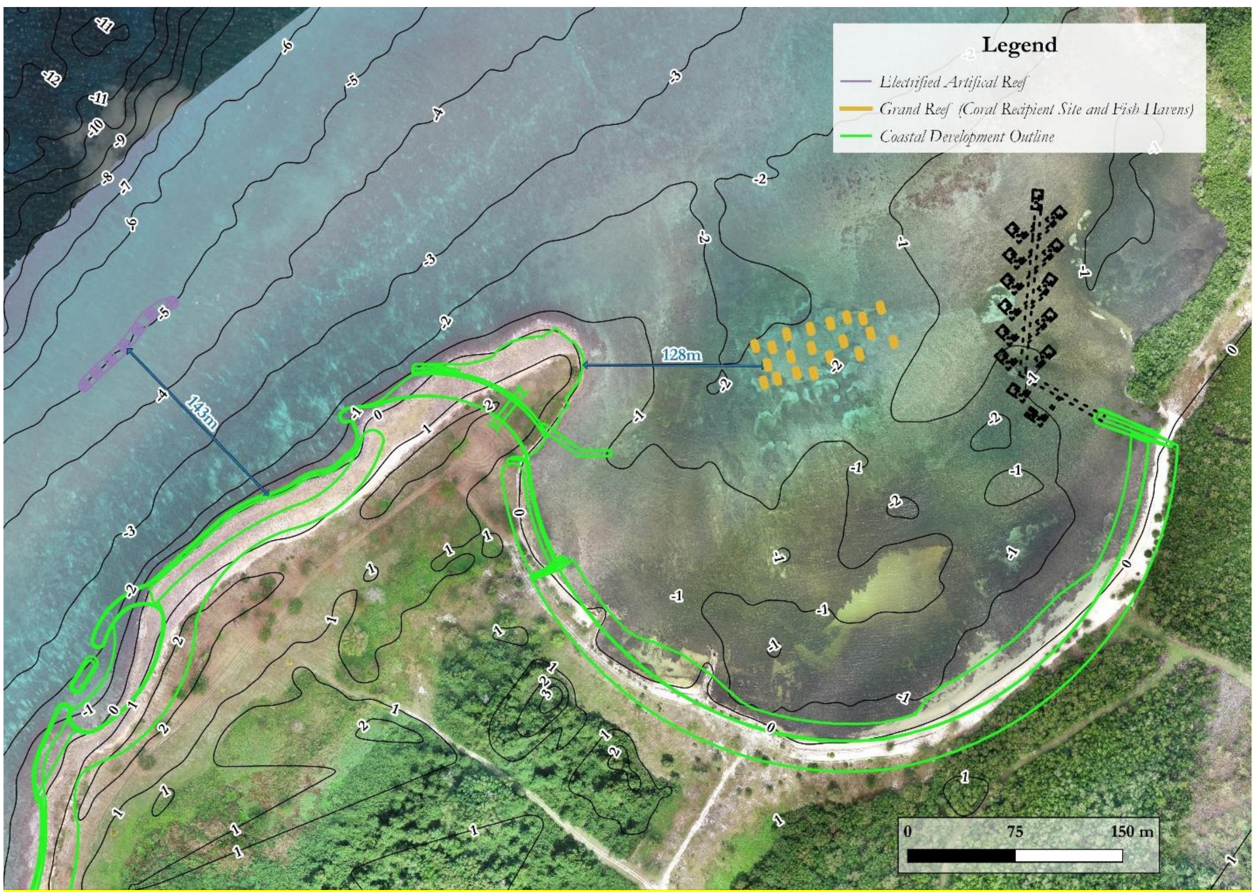


Figure 9.4 Mitigation measures for habitat loss

The photos in Figure 9.5 show examples of the successful application of this simple but effective technique. At this site, adjustments will be made to the design of the fish havens beyond the usage of concrete blocks to create more rugosity and habitat complexity (Figure 9.6).

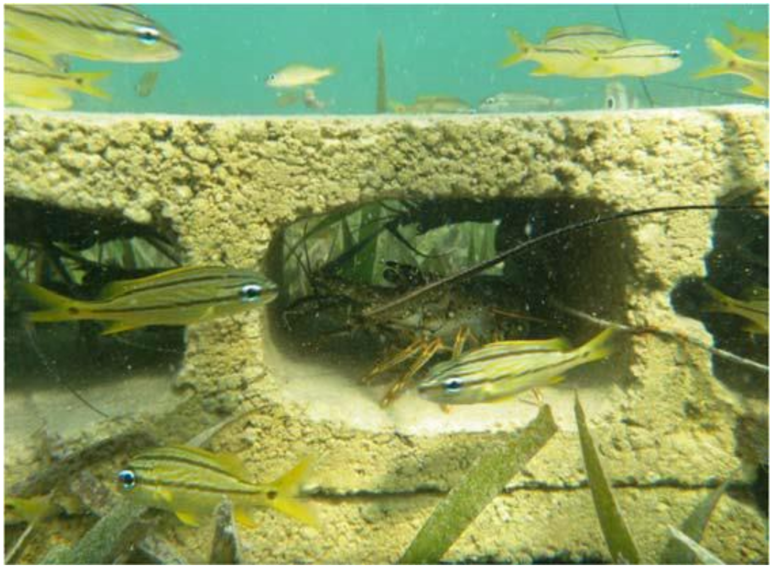


Figure 9.5 Existing fish haven structures in Jamaica

Figure 9.6 Examples of reef enhancement structures (source: Reef Design Lab)

Coral Relocation

Healthy coral reefs are among the most biologically diverse and economically valuable ecosystems on earth, providing valuable and vital ecosystem services. Coral reef contributions include, but are not limited to biodiversity, shore protection, fisheries, medicine, tourism and recreation. Coral reefs in Jamaica are in serious deterioration, suffering massive, long-term declines in abundance, diversity and habitat structure due to overfishing, natural disasters such as hurricanes, pollution, disease, and other anthropogenic and natural causes.

Generally, in the area surrounding the site, the coral cover has been recorded at **~ 20%.** There are numerous soft and hard coral colonies within the footprint of the survey area that must be relocated. For this site, it is proposed that corals will be relocated to the proposed Electrified Reef as shown in Figure 9.4.

Coastal defence structures as habitat

The artificial structures proposed will also serve as habitat in and of themselves. Coastal defence structures such as seawalls, groynes and breakwaters are primarily intended to protect the shorelines in their lee. One aspect of breakwater design and service that has not been considered to any great extent is the potential for these structures to provide habitat for coral organisms and for marine life in general, even though it has been observed that these structures can serve as important habitat for reef fish communities. These structures can be rapidly colonized after construction, and often enhance the recruitment and biomass of species that serve as food, settlement habitat, and shelter for a variety of other organisms. Figure 9.7 shows examples of the ecosystem service provided by such structures.

Figure 9.7 Flora & fauna colonising artificial structures - Sandals Negril (left); Accra Beach, Barbados (right)

Observations recorded in the field demonstrate how, within just a few years of implementation, armour stone breakwaters can develop in fish abundance, richness, and structure aging characteristics that are comparable to that of natural coral reefs. Fish and invertebrates of all demographic stages use these coastal defence structures as habitat, and coral and other species such as gorgonians and sponges are using these structures for recruitment.

9.4.2 Coral Reef Loss and Restoration

While it is being proposed that hard corals in the area be relocated, it is also understood that the relocated corals are at a heightened risk. However, combining the coral relocation process with other proven coral restoration methods like electrification can increase the survival rate.

Coral restoration projects can be assisted by two methods (i) In-situ coral nurseries assisted by mineral accretion technology, and (ii) Lab-based micro-fragmentation techniques.

One of the main benefits in employing both strategies is that the systems combined will support the propagation of both branching (mineral accretion) and massive (micro-fragmentation) species. This has the potential of increasing biodiversity on the reef. The coral propagation techniques are discussed below.

Mineral Accretion and Electrification

A mineral accretion process will be set up for the artificial reef system (Figure 9.8). This process will facilitate the metal structures receiving a coating of limestone. This will act as a booster for increased speed of growth and strength of coral fragments that will be attached onto any part of the metal (cathode). This technique has been deployed at the Royalton Resort Negril site (Figure 9.9).

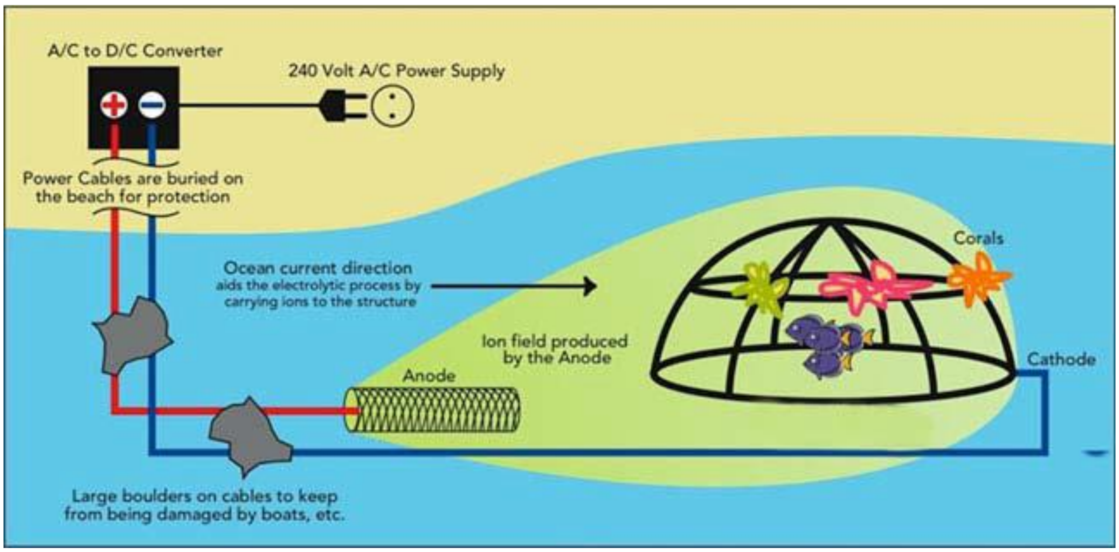


Figure 9.8 Schematic of coral reef electrification process

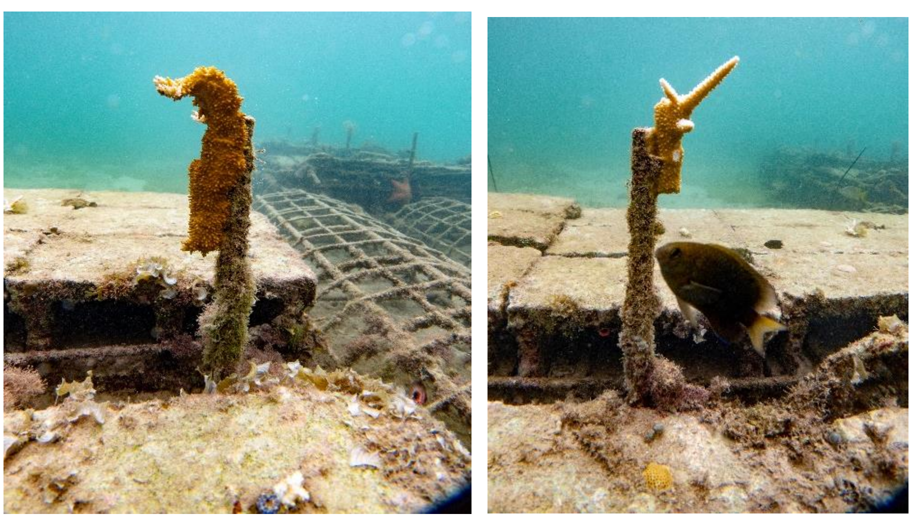


Figure 9.9 Fragments of *Acropora palmata* (left) and *Acropora cervicornis* (right) attached to mineral accretion system at Royalton Negril. The fragments attached to the metal are faring better during the current heatwave than those outplanted directly onto blocks

Further examples of the success of this type of system can be seen in results presented below (Figure 9.10) from a project by the Coral Alive organisation at North Malé Atoll, Maldives. Even though average temperatures were above 30°C, a survival rate of over 90% was recorded. Observations have shown that corals that undergo bleaching have added resilience while placed on the system (Figure 9.11) as do broken/damaged corals (Figure 9.12).

Figure 9.10 Fragments on electrified system 4 months apart

Bleached corals show massive recovery to a healthy growing state whilst on electricity

Figure 9.11 Response of bleached corals on the system

Figure 9.12 Broken damaged corals within 4 months on system

Microfragentation

The fragmentation technique consists of breaking the corals into smaller pieces of 1 to 5 polyps, using a specialised saw. This stimulates the coral tissue to grow, allowing them to grow into clones at 25 to 50 times the normal growth rate. The fragments are then placed in their shallow water tanks. Clone fragments recognise each other so instead of fighting each other for resources fuse together to form larger colonies. After 4-12 months the fully-grown corals are ready to be planted back into the ocean or fragmented to restart the process. The process is driven by light that stimulates reactions in the zooxanthellae. There are very specific areas of the visible spectrum necessary for the zooxanthellae to photosynthesise (Figure 9.13). The light must emit light in the parts of the spectrum necessary for this to occur.

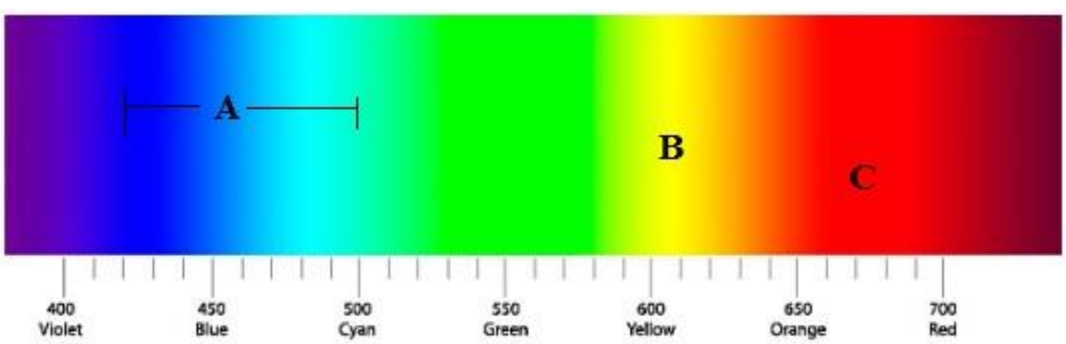


Figure 9.13 Visual light spectrum

A – Phototrophic response – active movement of the algae towards light – the beginnings of photosynthesis

B – Photosynthetic response – energy from light absorbed by photosynthetic reaction centres, which contain chlorophyll

C – Chlorophyll synthesis – chemical reactions of plant hormone cytokinin, which results in the formation of chlorophyll, allowing the ongoing photosynthesis and subsequent thriving of the zooxanthellae and coral.

Thanks to this technique, labs can fragment, grow and recombine corals in under two years to a size that would normally take 100 years. Outplants will fuse together and form a sexually reproductive coral head in two or three years. That is about 50 times faster than natural production (Forsman et al 2015).

Out planting and Receptor Sites

The receptor sites will be located 145m offshore as shown in Figure 9.4 and will house the mineral accretion nursery and outplant pods. This will be in approximately 5m of water. Figure 9.14 shows the outplant pods in use.

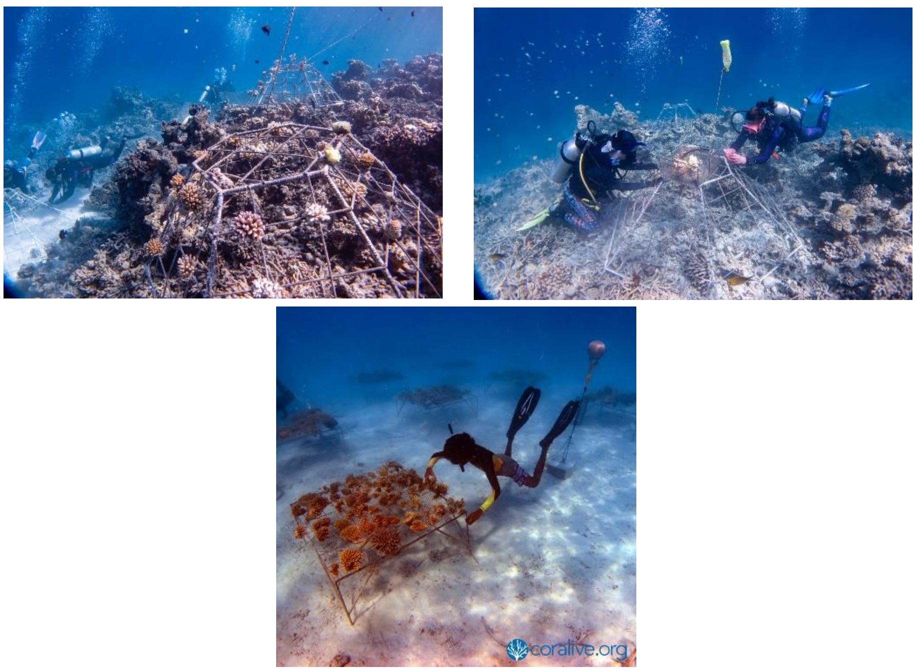


Figure 9.14 Examples of outplant pods

9.4.3 Seagrass Loss

It is not feasible to relocate the seagrass as there are no suitable recipient sites within an acceptable range from the donor site. This is partially due to the surrounding areas supporting healthy populations of seagrass. Given that a relocation exercise would be unsuccessful, it is proposed that other compensation measures be implemented. It is expected that the offsetting plan to be implemented prior to construction of the project will provide sufficient habitat to ensure no net loss of habitat due to the development of the project. The compensation strategy involves the creation/increase of productive capacity in different ecological units and ecosystem services. Seagrass provides several ecosystem services such as habitat and food source. The goal of the proposed measures will be to compensate lost habitat to maintain the fish communities within, and the functionality of, the existing habitat. These measures will include coral propagation as well as the installation of fish havens.

9.4.4 Summary of Mitigation Measures

Mitigation measures to reduce impacts to the marine environment are summarized in the points below and in Figure 9.15.

- To avoid damage to marine benthos a compromise was made by the client to reduce the amount of swimming area. Along the length of Hotels 2 and 3 several perched beaches were proposed to avoid the impact to the benthos in the area.
- In concept development stages, a pocket beach that affected 6000m^2 of seafloor was removed. Additionally, all the beaches created are being proposed through the excavation of land so the shoreline retreats landward.
- On average where the foreshore must be cleared of seagrass only about 20m width is cleared.
- Throughout the design process wherever possible structures like breakwaters and groynes were removed to reduce damage to seagrass and eliminate the need to relocate corals.
- All hard corals that can practically be relocated are to be moved to an electrified reef that support and enhance their growth.
- It is impractical to relocate any seagrass in area; therefore several additional measures must be put in place to compensate for the loss of seagrass.
- Fish havens will be added to the system to increase fish stock in the area. These increase the amount of habitat in the area. Additionally, the 50 coral pods to be added to the system will increase the amount of habitat provided by the corals.
- Recognizing that some corals will be too small to relocate, we will collect these fragments and grow them in the lab and in situ nursery. Additionally, if the area lacks a species of coral the lab growing process can be employed to increase the biodiversity of the area.

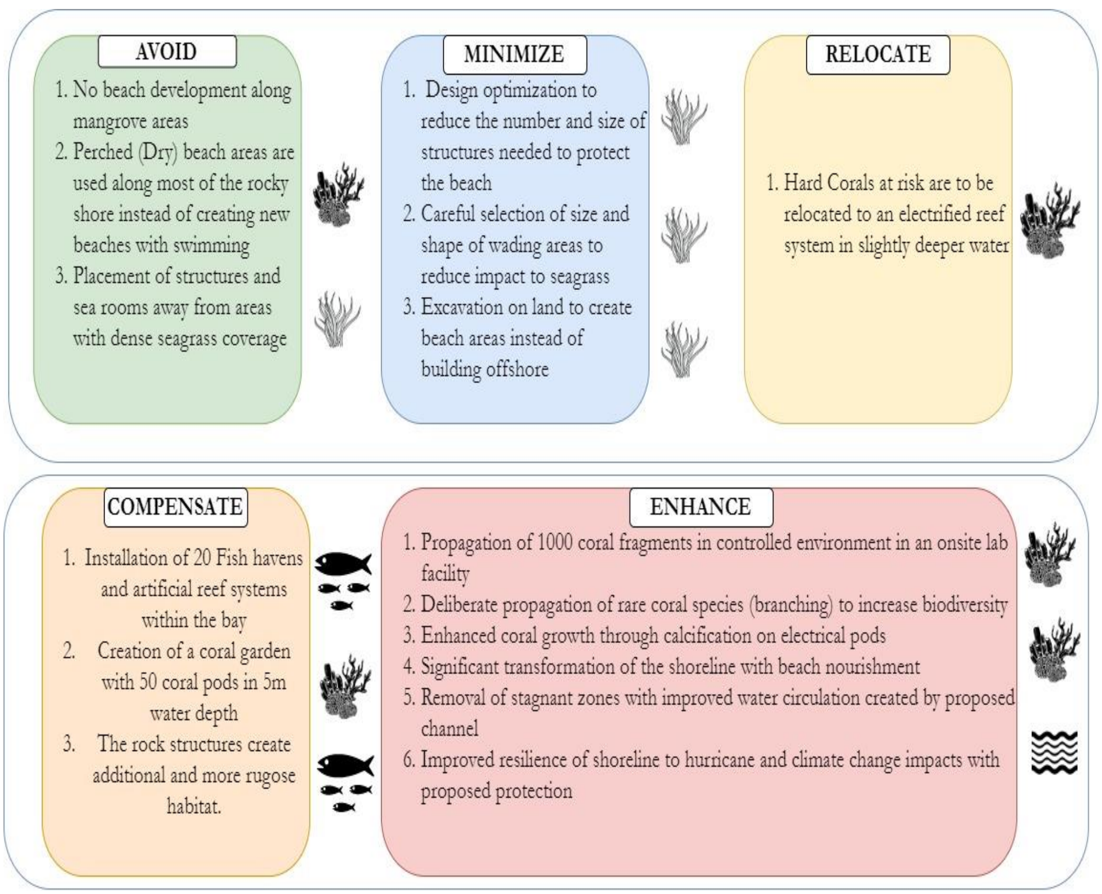


Figure 9.15 Marine impact mitigation strategies

10 Conclusions

This report presents the findings from baseline and impact assessment studies carried out by Smith Warner International Limited for the Princess Resorts Group. The report describes existing coastal conditions at the site and presents concepts for improving the resort areas as well as the anticipated impacts.

The following points were concluded from the ***baseline assessment*** of the site:

- The shoreline positions of Hotels 1 and 4 have retreated significantly from 2003. However, the findings suggest that the retreat was due to the removal of the mangroves in the area and not daily wave and current actions in the area.
- The water depths within the bay of Hotel 1 is very shallow and this causes slow currents (less than 0.1 m/s) and circulation issues. The circulation issues must be addressed to increase the beach's viability for resort use. At a minimum, the groynes currently in place must be removed.
- Hotels 2 and 3 are along the rocky shore which is bombarded by wave heights up to 2m under hurricane conditions and up to 0.9m under day-to-day waves.
- Along the rocky shore the currents are relatively fast (up to 0.5m/s) and therefore it has a potential to move large sediments and does not support the formation of a beach.
- The Hotel 4 shoreline is protected by a shallow reef system. The reef has gaps and therefore wave energy can get to shoreline (up to 0.5m under day-to-day conditions).
- The sediment transport modelling shows the potential sediment movement in the bay of Hotel 1 is very low in comparison to the other sections. At the shoreline of Hotels 2-4 sediment movement is mainly in a westerly direction. However, most of the sediment movement occurs within 20m of the shoreline and between 70 and 80m offshore.
- Under hurricane conditions, the site can be flooded by up to 2.6m MSL. At some sections of the site this represents over 2m of water. The buildings and critical infrastructure must be raised above this flood level. We recommend that the building flood levels be at least +3.0m MSL and ground level be at 2.6m MSL.
- The drainage assessment shows that the site naturally drains to wetland. Whenever the water level excites 0.4m MSL, there is overflow to the sea.

To enhance the coast for the Princess Resorts, we propose construction of the following key elements:

- For Hotel 1, we propose the construction of a 1.5m deep flushing channel to improve circulation in the area. We also propose the construction of a groyne at both ends of the beach as well as nourishment and clearing a wading area.
- For Hotels 2 and 3, we are proposing the addition of a cove beach, with two groynes and submerged breakwaters. Along the rocky shoreline, a perched beach will be introduced via the construction of a revetment on land.
- For Hotel 4, we propose the addition of two submerged breakwaters to enhance the reef system already at the area.

We also identified several positive and negative impacts of the proposed coastal enhancement. The following points were concluded from the ***impact assessment*** of the site.

- With the flushing channel in place the pollutant concentration is reduced to less than 5% in under 1.5 days. This is a significant improvement (more than two times more efficient) to the current situation.
- At Hotel 1 without the breakwater in place, the wave climate remains relatively unchanged. Under day-to-day conditions, wave heights are less than 0.3m which is comfortable for most swimmers and other recreational use.
- Along Hotels 2 and 3, the cove swimming area has wave heights less than 0.8m during swell events, while the wave conditions outside of the cove are greater than 1.7m. We recommend that beach users be warned not to venture outside the cove beach area during rough seas/swells. The wave and current conditions will be uncomfortable and dangerous for swimming.
- Along Hotel 4, the two breakwaters added to reinforce the reef reduce the wave conditions to 0.3m under swell conditions.
- Changes caused by the coastal enhancements occur only within the vicinity of the structures. With the structures being placed well within the 70-80m active transport zone, it is not expected there will be any downdrift impact from the structures.
- The footprint of the structures is expected to affect a total area of 2.43ac. Within the total footprint, corals and seagrass will be affected. For seagrass, it is impractical to relocate as suitable recipient sites are out of reach. However, there will be a deliberate effort to relocate all corals affected as well as propagate coral fragments to improve habitat.
- Additionally, a total of 2.5ac of artificial reef and fish haven structures will be added to improve the fish habitat to increase the fish population in the area.
- The proposed site drainage follows the natural flow path and drains freely into the mangrove, which increases the projected flood elevation in the wetland up to 0.040m.

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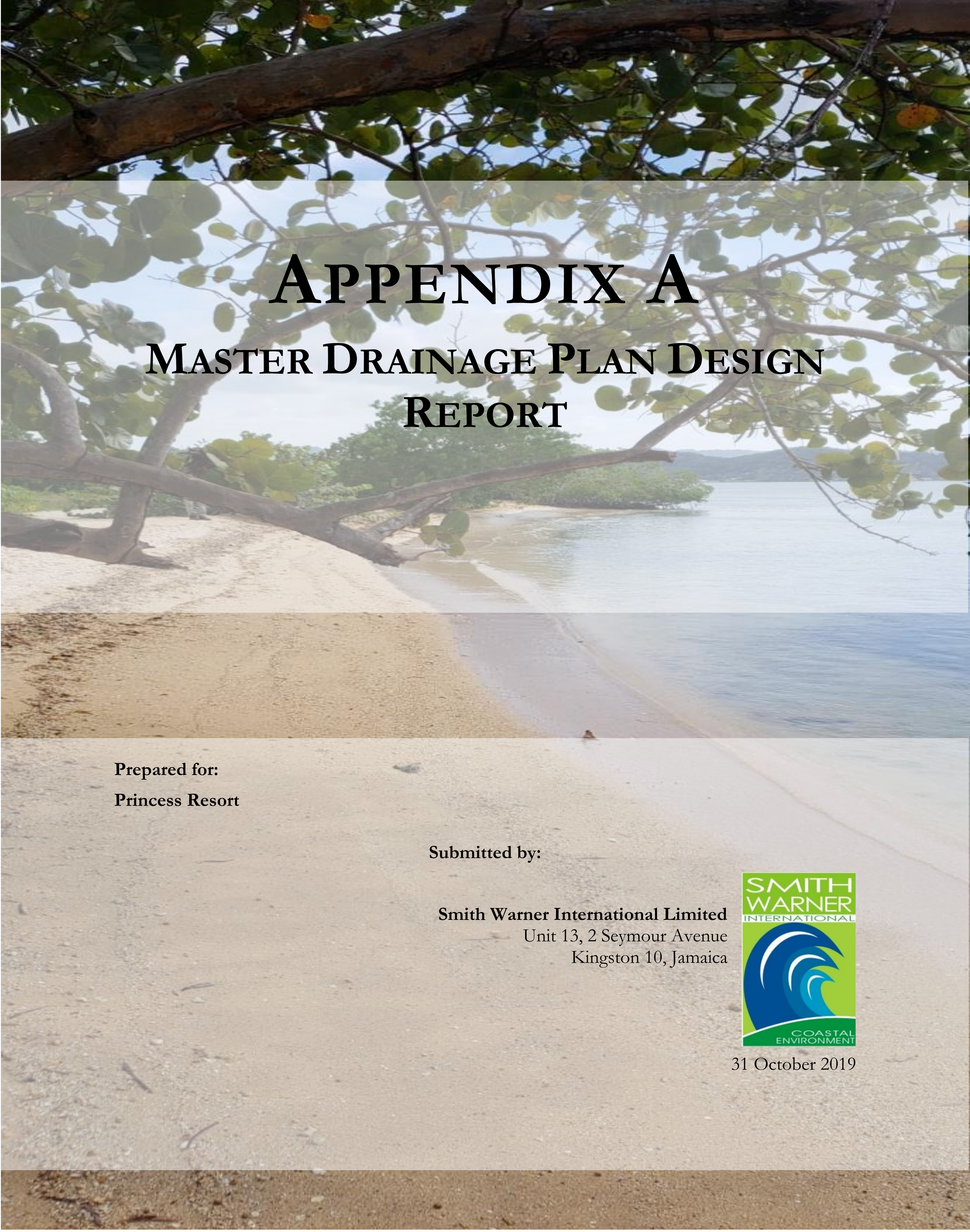

APPENDIX A

MASTER DRAINAGE PLAN DESIGN REPORT

Prepared for:

Princess Resort

Submitted by:

Smith Warner International Limited
Unit 13, 2 Seymour Avenue
Kingston 10, Jamaica

31 October 2019

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Executive Summary

The intention of the Master Drainage Plan is to develop a drainage design concept for the overall site to a sufficient level of detail to make allowance of all hydrologic and hydraulic considerations that can facilitate the detail drainage designs. The Green Island site displayed various important ecological traits, which is expected of a large wetland system. These traits included but are not limited to:

- Habitat for birds, reptiles, crustaceans, molluscs etc;
- Nutrient filtration and water absorption/retention;
- Spawning/breeding and nesting grounds for birds and fishes;
- Habitat for numerous juvenile creatures, especially reef fishes of commercial importance;
- Buffer to coastal energy (storms, high wave energy)

The drainage concept will aim to follow the existing site conditions. The relatively low-lying flat site gently slopes from elevations ranging from 2.5m to 0.5m above mean sea level (MSL) down towards the mangrove forest, which lies around 0 to 0.2m MSL. This wetland is also located downstream of a wider catchment area that drains into it. Hence, the mangrove forest acts as a natural retention system during low flows, storing runoff to nourish the flora and fauna within it.

Therefore, the plan can be visualized in two sections:

1. Wetland mangrove forest will be separated from the developed area via a dyke road.
2. Site will be drained towards the mangrove wetland via several outfalls in the dyke road.

Mangrove Usage and Improvements:

The concept allows the proposed site to drain freely into the mangrove forest and maintain its natural drainage pattern as the increase in flood levels are increased by 40mm for the 100-year storm event. This was deemed negligible as it has no impact on the surrounding communities and infrastructure. Enhancement to the mangrove system is proposed by introducing several culvert openings throughout the existing road network within the mangrove. This would promote more free movement of water through the entire mangrove forest, which will improve the storage capacity and provide water to areas currently deprived of water. Further improvements are also made to two areas that were observed to not contain the full characteristics of the surrounding mangrove – dried out areas. These areas are proposed to be converted to ponds to function as wetlands by planting with similar type flora (primarily mangroves) that currently grow in the forest.

The proposed site drainage follows the natural flow path and drains freely into the mangrove, which increases the projected flood elevation in the mangrove as follows:

Projected Water Level Within Mangrove (17.87% Increase in Rainfall Due to Climate Change)			
Storm Event	**Pre-developed**	**Post-Developed**	**Difference**
2yr	0.350m MSL	0.375m MSL	0.025m
5yr	0.490m MSL	0.532m MSL	0.042m
10yr	0.573m MSL	0.607m MSL	0.034m
25yr	0.620m MSL	0.656m MSL	0.036m

50yr	0.747m MSL	0.795m MSL	0.048m
100yr	0.836m MSL	0.876m MSL	0.040m

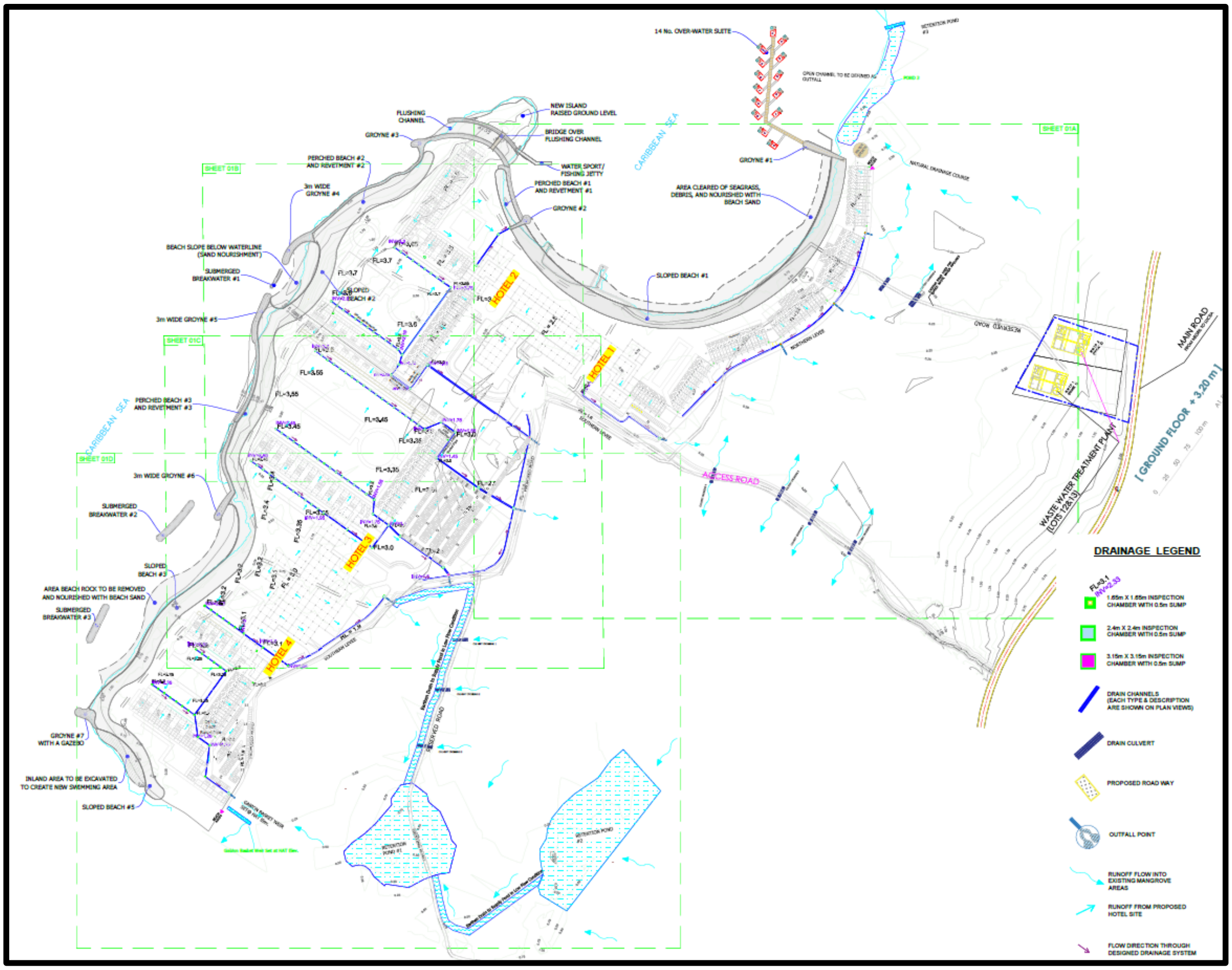
Three ponds would be introduced within the mangrove in areas that require rehabilitation. These ponds add volume capacity to the storage capabilities of the mangrove, and increase the square area of effective mangrove as follows:

Pond	*Increase in Storage Capacity (m^3)*	*Increase in Mangrove Area (m^2)*
Pond 1	7,066	12,602
Pond 2	7,978	16,933
Pond 3	1,149	3,413
Total	**16,193**	**32,948**

Site Drainage Concept:

The proposed drainage concept will allow rainfall runoff to drain freely into the mangrove via ten outfall points. All the outfall points were set at an elevation higher than the projected flood elevation for the 1 in 50-yr storm frequency, with consideration for climate change. The use of multiple outfall points proved advantageous as it works better for a flat site and reduces the amount of grading required while resulting in smaller drain sizes. These drainage outfalls will be controlled by hydraulic structures consisting of outfall pipes with flap gates to prevent back flow of water into the site when water levels in the mangrove exceeds the 50-year design flood level within the forest. These outfall pipes will be encased within a catch pit that contains a 500mm deep sump strategically located to trap sediments prior to discharging into the mangrove.

All other internal drains will be primarily buried pipes and covered box drain with catch pits to keep in accordance with the architect's finish concept. All such drains and outfall pipes were designed for a 1 in 25-year storm frequency.



Master Drainage Plan for Princess Resorts

1 Introduction

This report presents the Master Drainage Plan to be implemented at the proposed site for Princess Resorts. The intention of this Master Drainage Plan is to develop a drainage design concept for the overall site to a sufficient level of detail to make allowance of all hydrologic and hydraulic considerations that can facilitate the detailed drainage designs. In doing so, the natural drainage condition of the site is assessed, and a proposed drainage plan is develop using internationally recognized best storm water management practices to ensure minimal to no impacts are made to the surrounding environment and communities.

1.1 Site Location

The site is located off the north coast of Green Island within the Parish of Hanover, Jamaica. It is bounded by the Caribbean Sea to the north and the A1 Highway to the south (Figure 1.1).

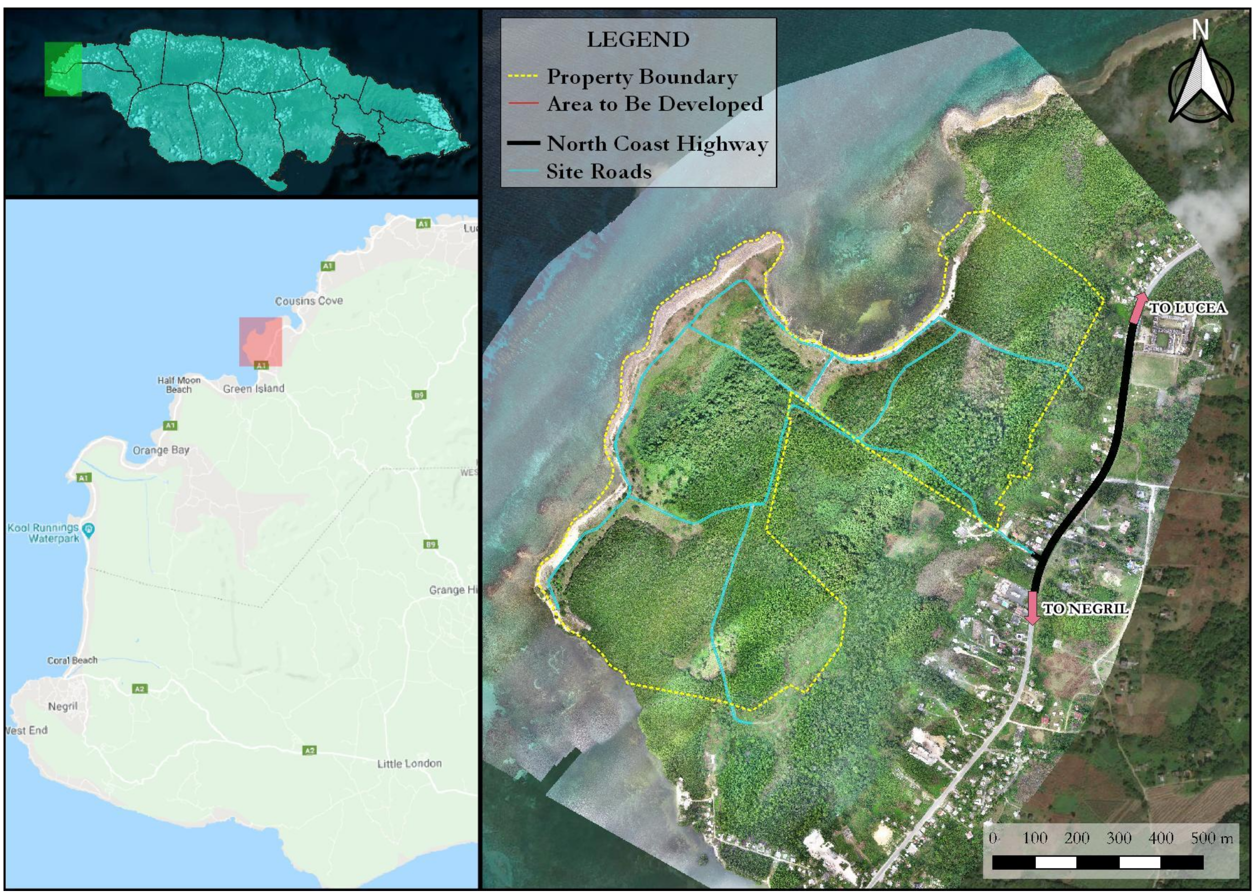


Figure 1.1: Map of Jamaica showing location of Green Island within Hanover, Jamaica

Figure 1.2: Map showing location of study area within Green Island, Jamaica

1.2 Wetland Ecology

The Princess Hotel proposed site may be described as a wetland floodplain and drainage collection area for major storm drains along the Green Island to Negril Highway. This very mature mangrove forest system has a traditional and expected Caribbean mangrove forest tree zonation and a high presence of mangrove/golden ferns (*Acrostichum aureum*; regarded as mangrove plants worldwide). The forest is interspersed with other emergent wetland vegetation (*Typha sp., Dalbergia sp., Spartina sp.*), with these species replacing true mangroves (Red, Black and White mangroves) to the north and south primarily. Logwood (*Haematoxylum campechianum*) trees dominate most of the degraded/reclaimed sections of the forest. The main flora species observed are summarized in Table 1-1.

Mangrove trees are known to exist and survive in a range of salinity, from 0ppt to over 40ppt and from low to high nutrient loads. However, it is documented that these flowering trees perform best in waters with brackish influence (Doyle 2003; World Bank-Jamaica 2019) and a moderate amount of nutrients to absorb and assimilate into their biomass, as they are nutrient limited (Reef et al. 2010).

The Green Island impact site displayed various important ecological traits, which is expected of a large wetland system. These traits included but are not limited to:

- Habitat for birds, reptiles, crustaceans, molluscs etc;
- Nutrient filtration and water absorption/retention;
- Spawning/breeding and nesting grounds for birds and fishes;
- Habitat for numerous juvenile creatures, especially reef fishes of commercial importance; and
- Buffer to coastal energy (storms, high wave energy)

Despite the impressive occurrence of large undisturbed area of mangrove forest, the site shows strong evidence of human disturbance and fragmentation. Based on satellite images, this work occurred over 10 years prior to the site visits, and does not seriously hinder the wetland's ecological functions. The area shows minimal habitat fragmentation as the disturbances were of a small scale and localized in some sections.

Table 1-1: Flora observed in drainage areas surveys-including ranking in DAFOR index

Flora Observed	*DAFOR Index*	*Status*
Acacia sp.	D	Least concern
Baceda /Bastard cedar (*Guazuma ulmifolia*)	R	Least concern
Black mangrove (*Avicennia germinans*)	F	Least concern
Bromeliads	F	Least concern
Buttonwood (*Conocarpus erectus*)	R	Least concern
Coin vine (*Dalbergia sp.)*	A	Least concern
Guinea grass *(Megathyrsus sp.)*	A	Least concern
Logwood (Haematoxylum campechianum)		Least concern
Lead tree (*Leucaena sp.)*	A	Least concern
Mahoe *(Hibiscus elatus)*	R	Least concern
Mangrove Fern *(Acrostichum aureum)*	A	Least concern
Noni *(Morinda citrifolia)*	F	Least concern
Orchids	R	*
Red mangrove (*Rhizophora mangle*)	O	Least concern
Shame ole lady *(Mimosa pudica)*	A	Least concern
Spartina sp.	O	Least concern
Typha sp	F	Least concern
White mangrove (*Laguncularia racemosa*)	F	Least concern

1.3 Existing Drainage Conditions

1.3.1 Observed Flow Paths

The extent of the wetland displays a complex hydrological regime with the eastern and western sections strongly influenced by tides, southern areas being primarily riverine and some mid-sections having mixed/estuarine properties. Water originating from the highway to the south travels in a northern direction to the northern wetland extent, diverting to two main eastern and western tidal exchange points, and mixing with saltwater in some eastern and western sections of the forest.

Five culverts were identified along the highway draining north towards the wetland (Figure 1.3). A very large and actively flowing culvert (#5, as seen in Figure 1.4) was observed at the eastern extent of the road. This is the culvert which likely supplies the eastern area with fresh water and results in a heavy outflow through the service road culvert, towards the eastern mangrove.

The physical barriers created by the roadways in the wetland area are likely preventing more widespread mixing of fresh and saltwater in some segregated forest sections. Brackish waters were recorded in areas closer to the eastern and western tidal influence, but minimal saltwater influence was detected/recorded in the mid and southern sections of the forest, despite being only a few meters away from tidal occurrence areas, being separated by limestone and/or earthen roadways. Figure 1.5 summarizes the observed hydrology flows.

Figure 1.3: Culvert locations (along highway) and important drainage features on the property

Figure 1.4: Culvert #5 - main supply of fresh water to the Eastern end of the property

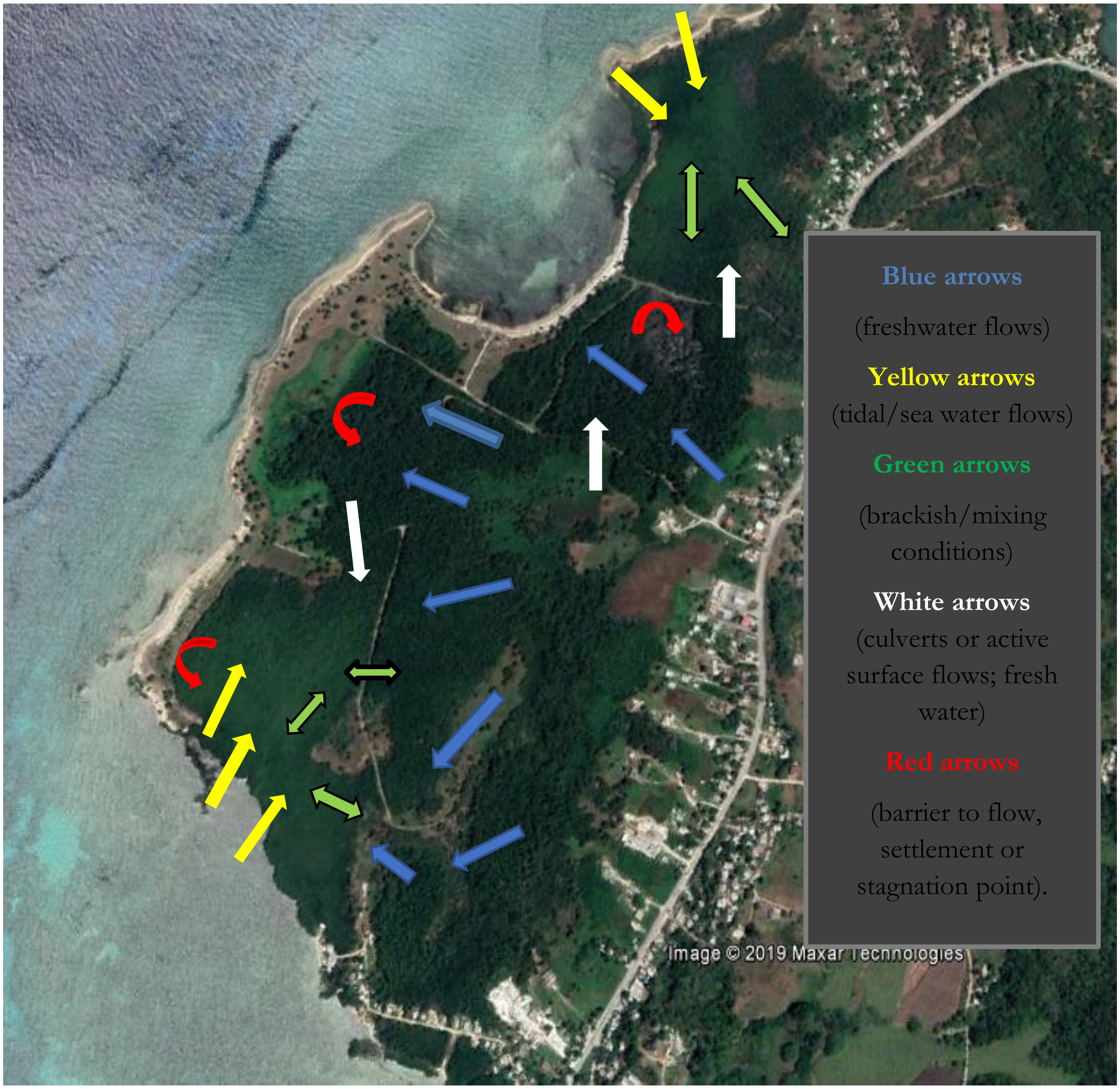


Figure 1.5: Observed wetland surface hydrology regime

1.3.2 Site Topography and Land Use Patterns

The topographic surveys (Figure 1.6 and Figure 1.7) provide supporting evidence for the theory that roadways act as physical impediments for tidal influence of the whole forest. Figure 1.6 shows that the NW sections of the forest have an identical elevation (+0.25m MSL) to the mangrove forest areas surveyed south of the access roadway, as close to the freshwater transition zone.

A similar occurrence is found in the NE forest. A 0.20m elevation is found at the eastern canal area, throughout most of the forest and all the way south close to the community. However, no saltwater was recorded in the mid-south area of the forest (yellow circles).

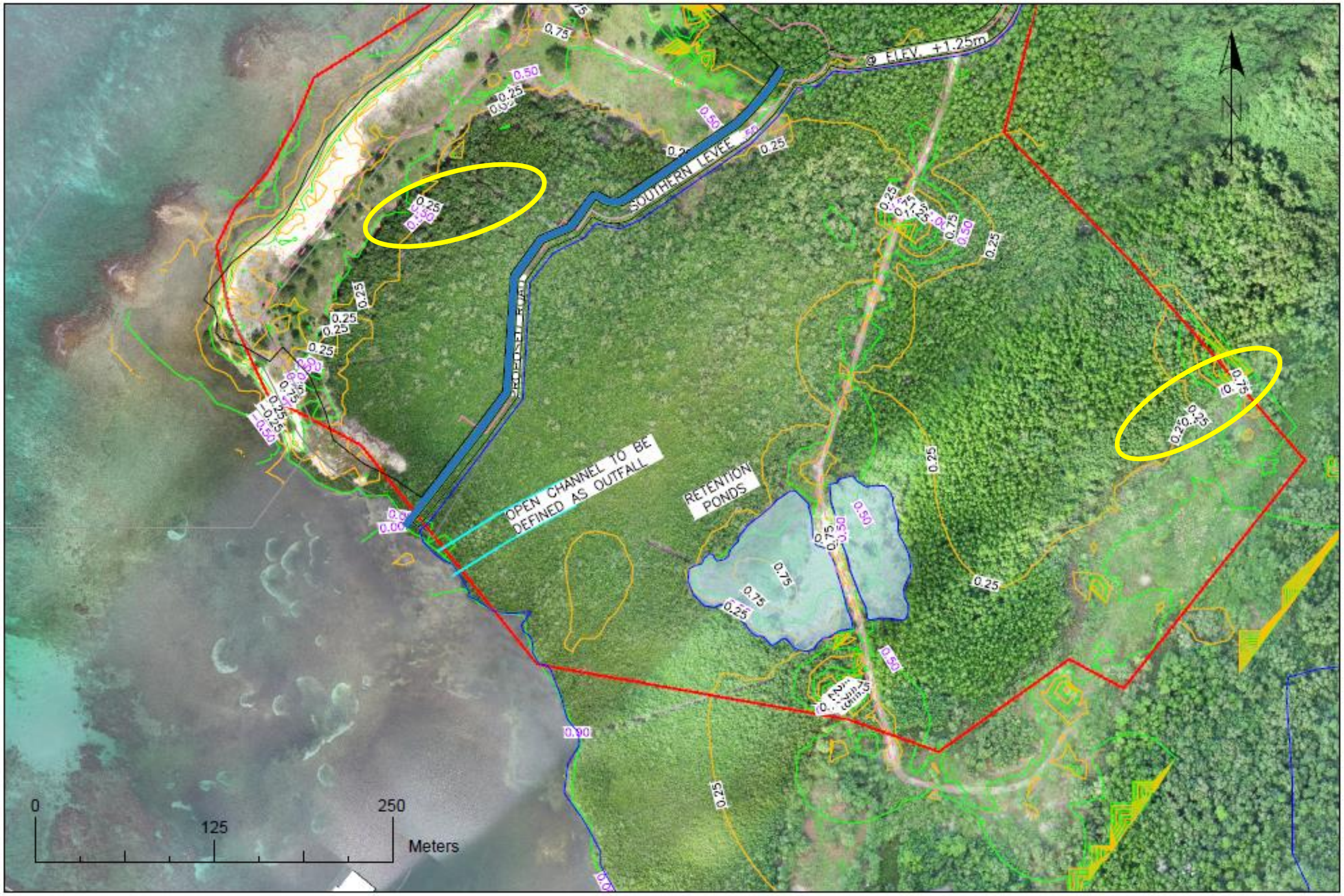


Figure 1.6: Western mangrove topography with proposed drainage ponds

Figure 1.7: Topographic results for eastern end of property, highlighting identical elevations in forest (yellow circles) and degraded mangrove area (Rehab#1)

The total catchment area was delineated using the 1:2500 topographic map for the area and was found to be approximately 2.32km^2. The terrain of the site is generally flat from the shoreline to the A1 highway after which it rises with slopes varying between 10% and 18%. Using Google Earth, the approximate developed area within this site was estimated to be 0.23km^2 or 9.92% of the total catchment area as depicted in Figure 1.8 below. Such development was noted to be along the A1 Road primarily.

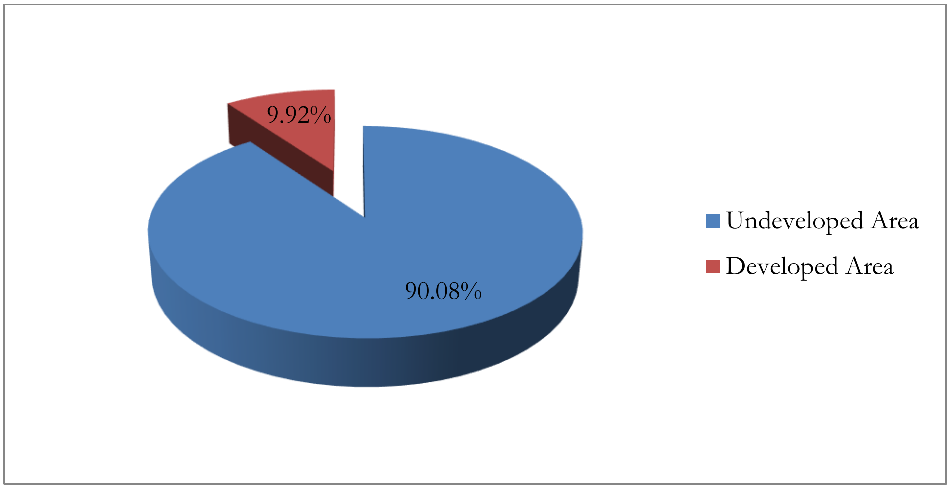


Figure 1.8: Land use distribution of existing catchment area

1.3.3 Water Logger Results

A strong tidal movement was observed at the eastern "service road" canal (Figure 1.5) area during site visits and confirmed with Hobo U-20 water level logger data collected from that location. Table 1-2 below provides water level logger results.

Table 1-2: Water level logger results: PSI converted to cm depths

	Drainage 1	*Drainage 2*	*Drainage 3*
Max (PSI)	14.92	14.72	14.72
Mean (PSI)	14.67	14.64	14.63
Difference	0.25	0.08	0.1
cm depth	**17.65**	**5.62**	**7.03**

The northern edge of the proposed drainage pond received a mean water level of 17.65cm of water during a 2-week monitoring period (Figure 1.9).

The degraded mangrove west of the service road is an end point for fresh water travelling north (from Culvert 5). A water level logger on the edge of the area showed an average recorded water depth of 5.86 cm over a 2-week period. More importantly the area showed a clear peak in water level on 4 October 2019 likely linked to a rainwater event (Figure 1.10). The daily water level fluctuation was not very drastic, indicating a weak or negligible tidal influence. A salinity of 0ppt recorded during all four field visits confirms that this area is a freshwater end point with no significant tidal exchange.

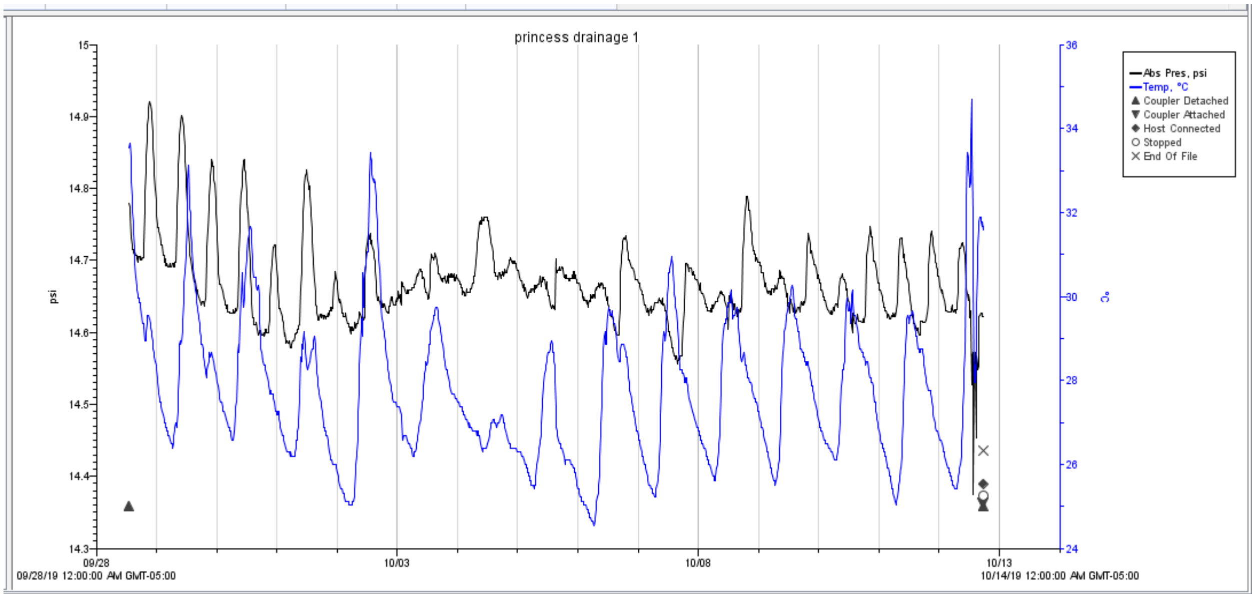


Figure 1.9: Water level logger results for western drainage pond area periphery

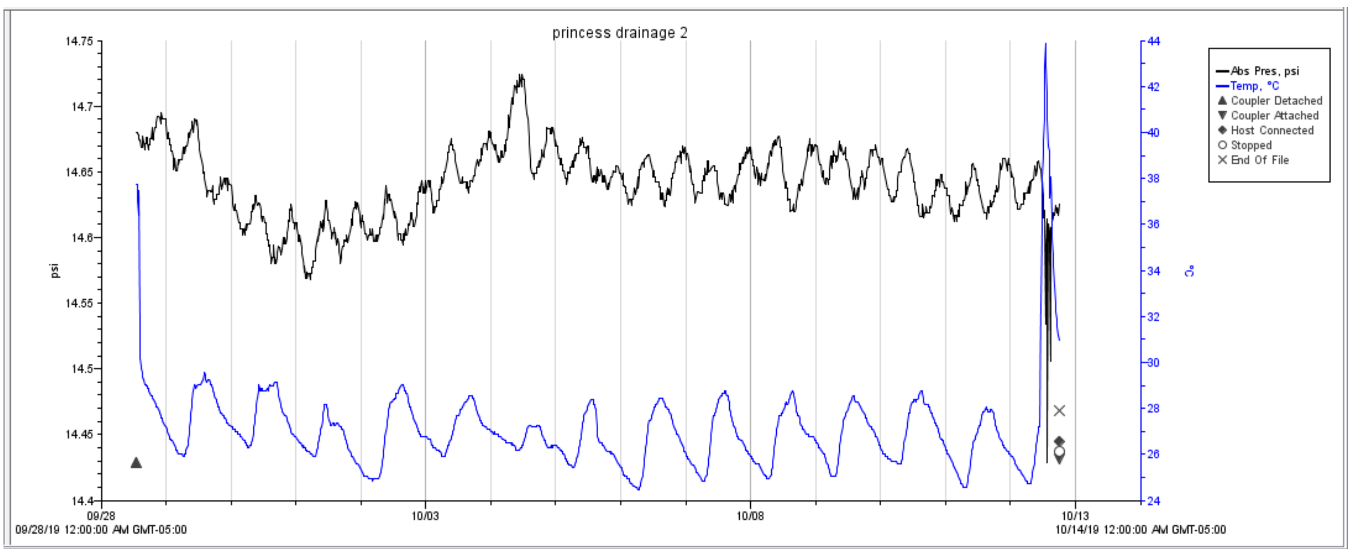


Figure 1.10: Water level logger results for degraded mangrove section along service road

Figure 1.11 shows that the area receives regular tidal fluctuations. The mean tidal height of this area was calculated at 6.87cm. This low tidal height is expected as this location is over 300m away from the coastline. All loggers reflected a matching peak in water level on 4 October 2019 despite being physically separated, hundreds of meters apart. This gives evidence that the mangrove forest receives pulses of fresh water during rainfall events.

Most roadways have no culverts to allow forest connectivity and drainage. An ideal example of the lack of forest connectivity may be found at the western-most access road. This road, which is of marl and rubble construction, showed a segregation of fresh and tidal salty waters, separated by a 2m wide road. However, previous records of salinity show that the areas have some amounts of water exchange through or below the roadway.

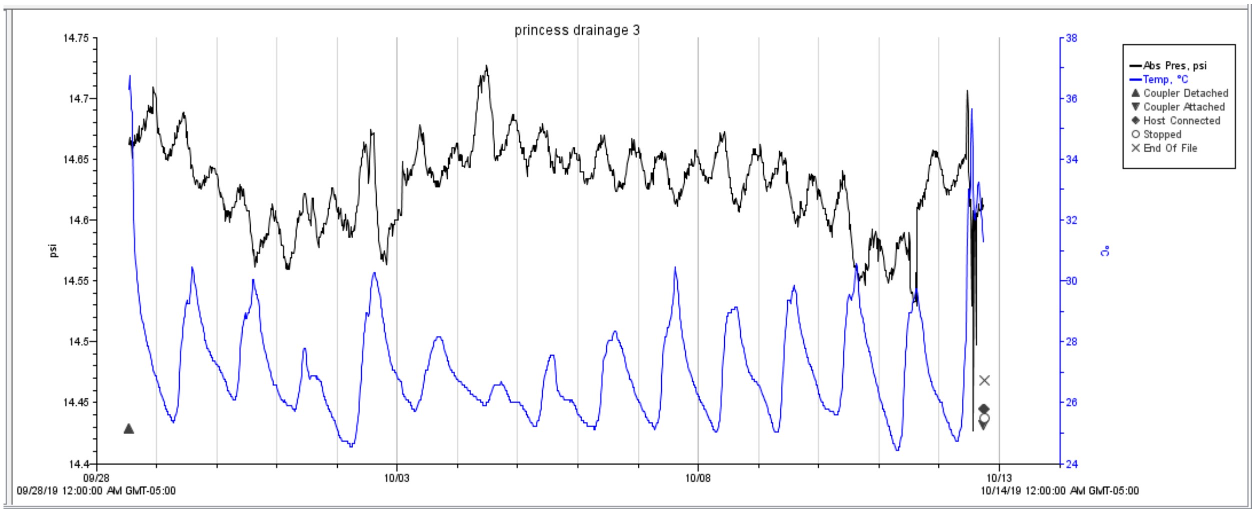


Figure 1.11: Water level logger results for eastern degraded mangrove area (East of "service road")

A very clear northern movement of fresh water beneath the roadway was observed on the field visit on 12 October 2019. This section of roadway had a wooden log perpendicular beneath the roadway (Figure 1.12), which is seemingly rotting and provided a passage for the fresh water to enter the tidal mangrove section. A salinity of 0ppt was measured here on both sides of the roadway.

Figure 1.12: Western access road with log beneath roadway provides visible route for freshwater movement below the road

1.4 Proposed Drainage Design Concept

1.4.1 Mangrove Usage and Improvements

The concept allows the proposed site to drain freely into the mangrove forest and maintain its natural drainage pattern as the flood levels are increased by 40mm for the 100-year storm event. This was deemed negligible as it has no impact on the surrounding communities and infrastructure as detailed later in this report. The mangrove is also enhanced by introducing several culvert openings throughout the existing road network within the mangrove. This would promote more free movement of water through the entire mangrove forest, which will improve the storage capacity and provide water to areas deprived of water. Further improvements would also made to two areas that were observed to not contain the full characteristics of the surrounding mangrove – dried out areas. These areas would be converted to ponds to function as wetlands by planting with similar type flora (primarily mangroves) that currently grow in the forest. This would also improve the water storage capacity of the area. Later sections of this report present the hydrologic and hydraulic assessments of these proposed usage and improvements to the mangrove forest.

1.4.2 Site Drainage Concept

The proposed drainage concept will allow rainfall runoff to drain freely into the mangrove via ten outfall points. All the outfall points were set at an elevation higher than the projected flood elevation for the 1 in 50-year storm frequency, with consideration for climate change. The use of multiple outfall points proved advantageous as it works better for a flat site and reduces the amount of grading required while resulting in smaller drain sizes. These drainage outfalls will be controlled by hydraulic structures consisting of outfall pipes with flap gates to prevent back flow of water into the site when water levels in the mangrove exceeds the 50-year design flood level within the forest These outfall pipes will be encased within a catch pit that contains a 500mm deep sump strategically located to trap sediments prior to discharging into the mangrove.

All other internal drains will be primarily buried pipes and covered box drain with catch pits to keep in accordance with the architect's finish concept. All such drains and outfall pipes were designed for a 1 in 25-year storm frequency.

1.5 Design Approach

The hydrologic and hydraulic designs will be approached in two parts as follows:

1.5.1.1 Part 1: Catchment Wide Analysis

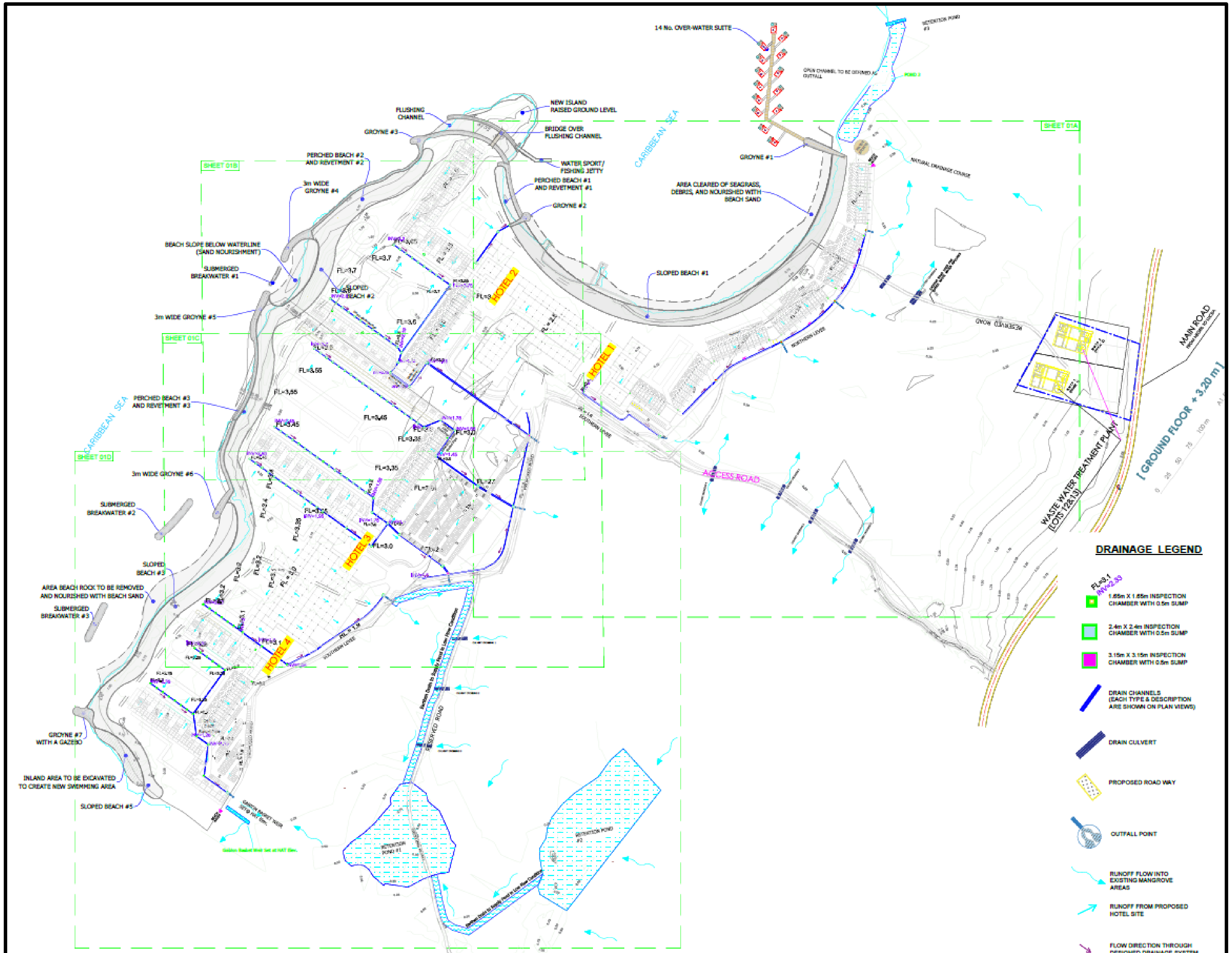
This will include the hydrologic assessment of the mangrove area, which quantifies the expected flow rate and volume within for various storm frequencies using the Soil Conservation Service (SCS) method analysed in HEC-HMS. This information will then be used in combination with a stage-storage curve for the mangrove to determine the projected increase in flood levels within the mangrove due to the proposed development.

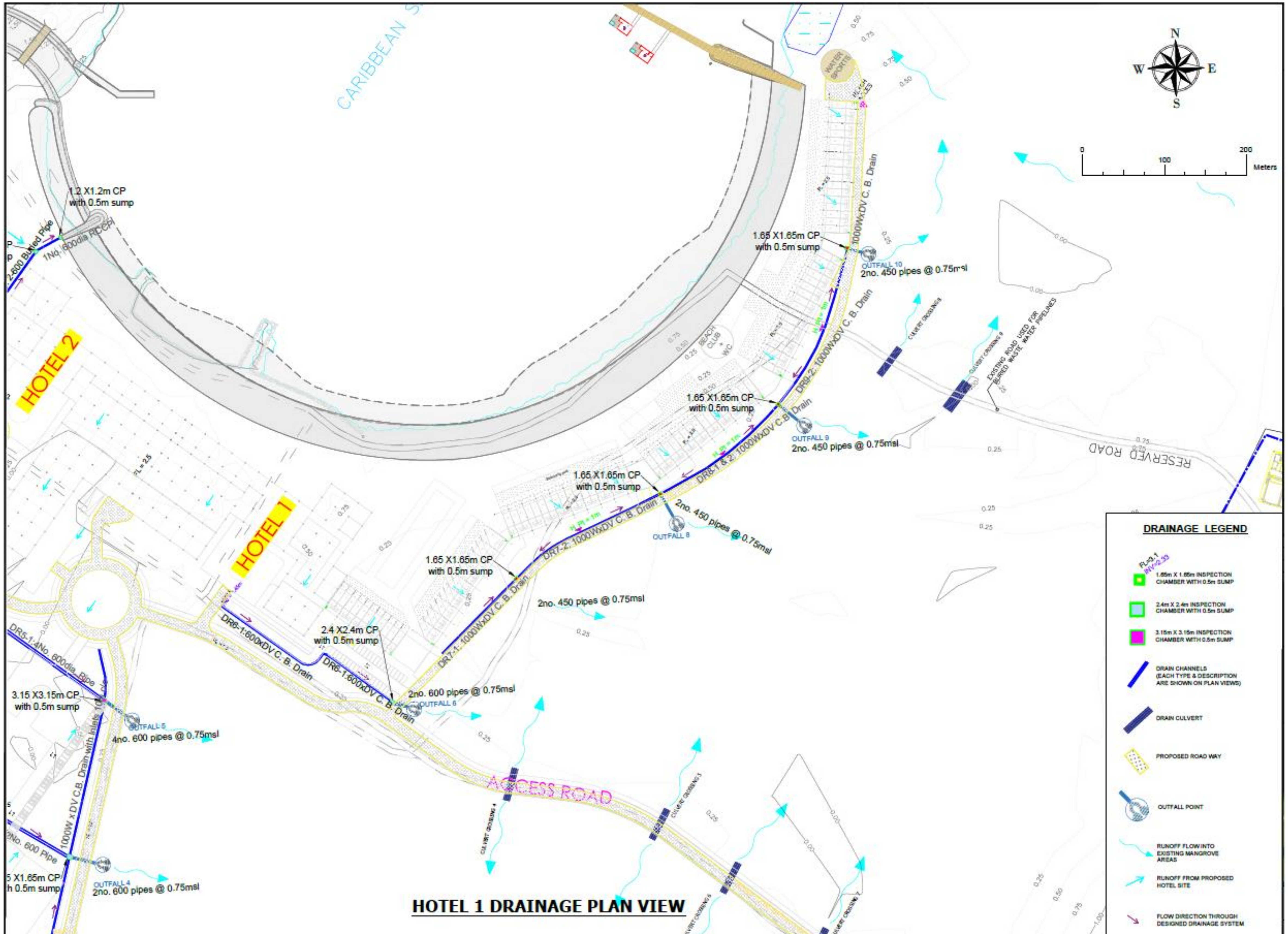
1.5.1.2 Part 2: Site Drainage

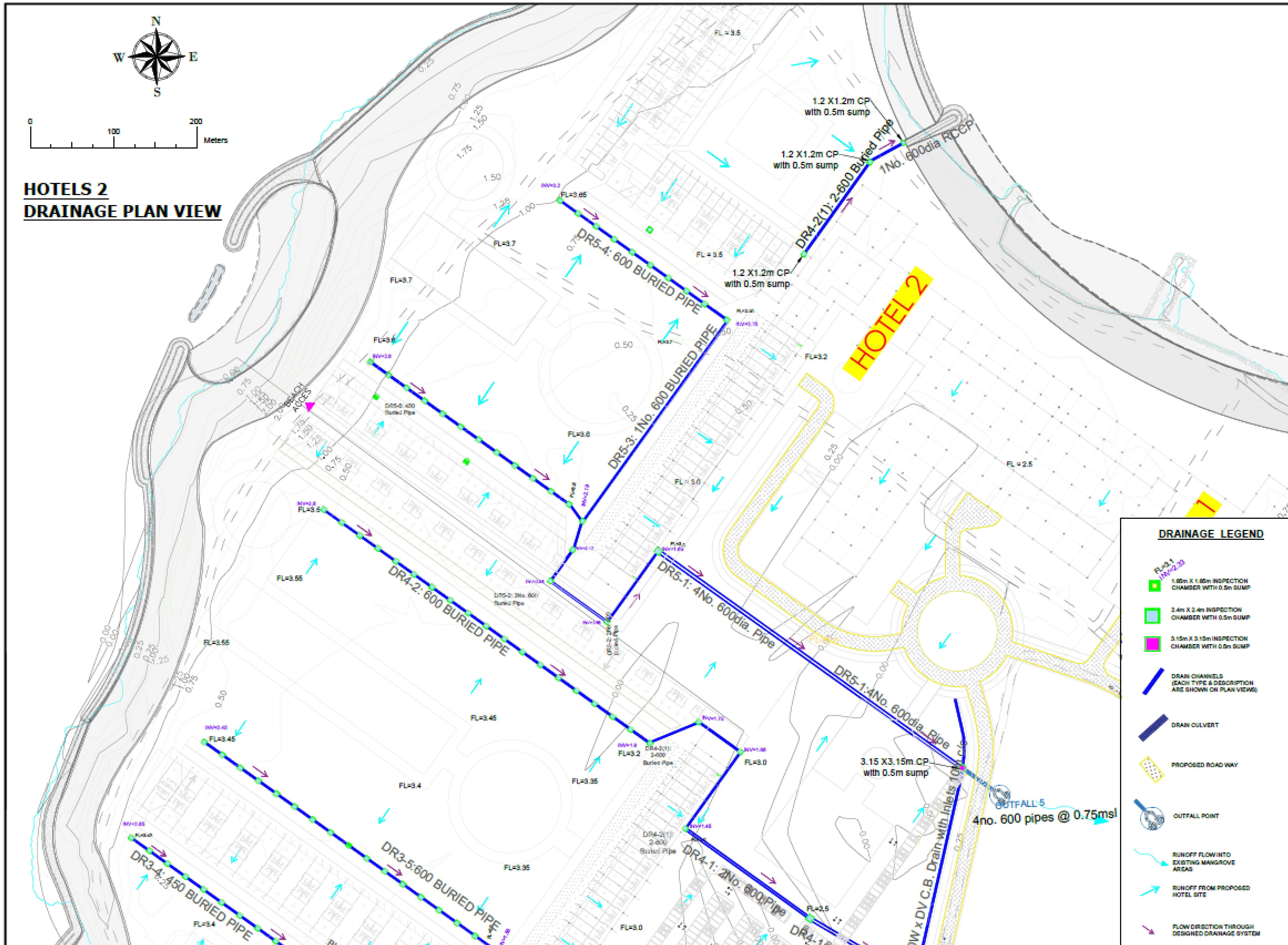
The proposed drainage network is limited to the main drains within the site to allow coordination and ensure hydrologic and hydraulic feasibility of a system that can be detailed further in the final design stage.

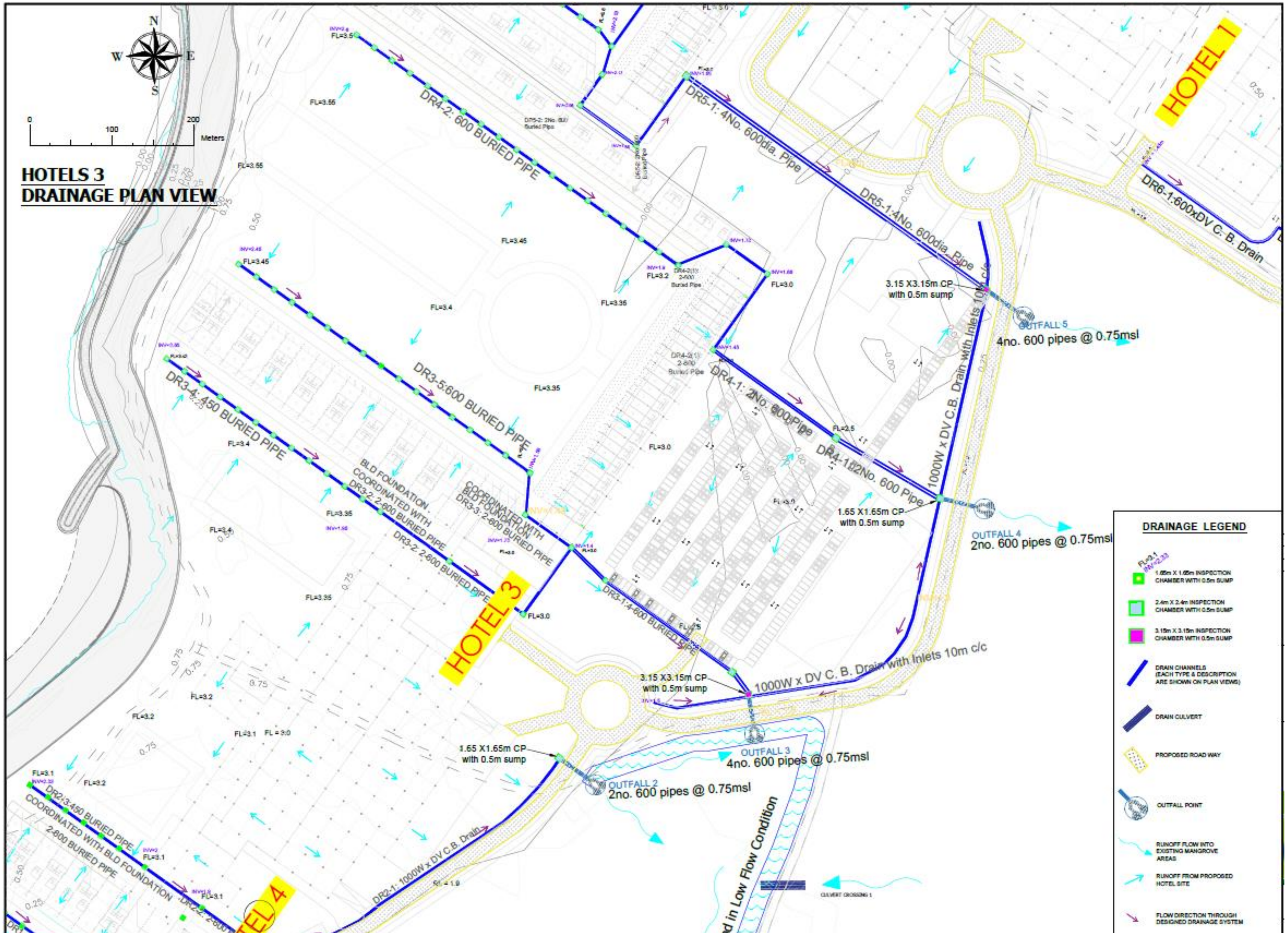
1.6 Master Plan

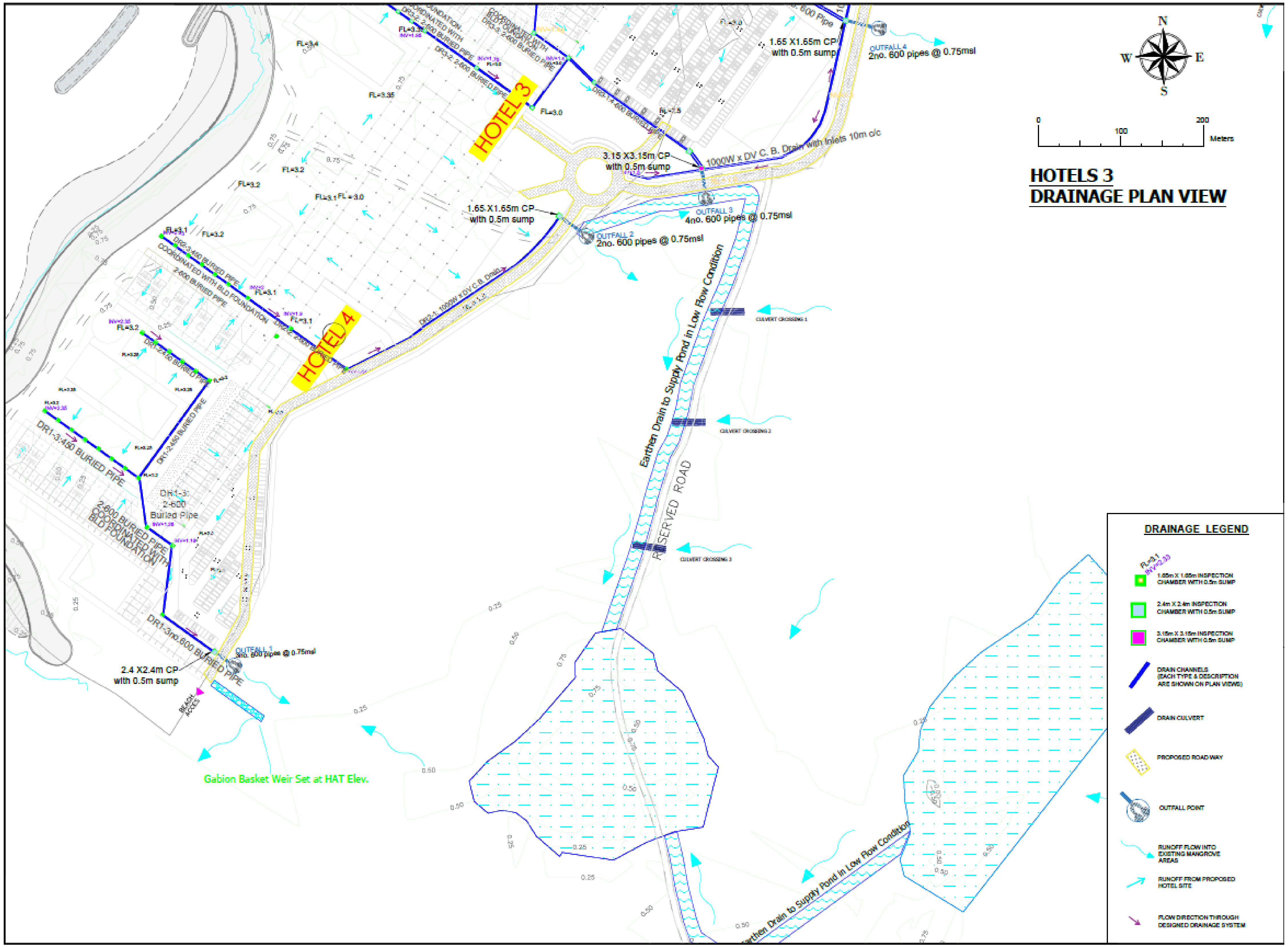
The drainage master plan drawings are presented in the following pages.


DRAINAGE LEGEND
1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
DRAIN CULVERT
PROPOSED ROAD WAY
OUTFALL POINT
RUNOFF FLOW INTO EXISTING MANGROVE AREAS
RUNOFF FROM PROPOSED HOTEL SITE
FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM
HOTEL 1
HOTEL 2
HOTEL 3
HOTEL 4
CARIBBEAN SEA
ACCESS ROAD
MAIN ROAD
WASTE WATER TREATMENT PLANT
[GROUND FLOOR + 3.20 m]
14 No. OVER-WATER SUITE
NEW ISLAND RAISED GROUND LEVEL
BRIDGE OVER FLUSHING CHANNEL
FLUSHING CHANNEL
GROYNE #3
PERCHED BEACH #2 AND REVETMENT #2
WATER SPORT/ FISHING JETTY
PERCHED BEACH #1 AND REVETMENT #1
GROYNE #2
GROYNE #1
AREA CLEARED OF SEAGRASS, DEBRIS, AND NOURISHED WITH BEACH SAND
SLOPED BEACH #1
3m WIDE GROYNE #4
BEACH SLOPE BELOW WATERLINE (SAND NOURISHMENT)
SUBMERGED BREAKWATER #1
SLOPED BEACH #2
3m WIDE GROYNE #5
PERCHED BEACH #3 AND REVETMENT #3
3m WIDE GROYNE #6
SUBMERGED BREAKWATER #2
SLOPED BEACH #3
AREA BEACH ROCK TO BE REMOVED AND NOURISHED WITH BEACH SAND
SUBMERGED BREAKWATER #3
GROYNE #7 WITH A GAZEBO
INLAND AREA TO BE EXCAVATED TO CREATE NEW SWIMMING AREA
SLOPED BEACH #5
SHEET 01A
SHEET 01B
SHEET 01C
SHEET 01D


CARIBBEAN S
N
W
E
S
0
100
200
Meters
1.2 X1.2m CP
with 0.5m sump
1No. 600dia RCCP
HOTEL 2
HOTEL 1
1.65 X1.65m CP
with 0.5m sump
OUTFALL 10
2no. 450 pipes @ 0.75msl
1000WxDV C.B. Drain
DR9-2:1000WxDV C.B. Drain
OUTFALL 9
2no. 450 pipes @ 0.75msl
DR8-1 & 2: 1000WxDV C.B Drain
RESERVED ROAD
OUTFALL 8
2no. 450 pipes @ 0.75msl
DR7-2: 1000WxDV C.B. Drain
DR7-1: 1000WxDV C.B. Drain
2no. 450 pipes @ 0.75msl
2.4 X2.4m CP
with 0.5m sump
DR6-1:600xDV C.B. Drain
DR6-1:600xDV C.B. Drain
2no. 600 pipes @ 0.75msl
OUTFALL 6
3.15 X3.15m CP
with 0.5m sump
DR5-1:4No. 600dia. Pipe
OUTFALL 5
4no. 600 pipes @ 0.75msl
1000W x DV C.B. Drain with Inlets
2No. 600 Pipe
OUTFALL 4
2no. 600 pipes @ 0.75msl
ACCESS ROAD
DRAINAGE LEGEND
1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
DRAIN CULVERT
PROPOSED ROAD WAY
OUTFALL POINT
RUNOFF FLOW INTO EXISTING MANGROVE AREAS
RUNOFF FROM PROPOSED HOTEL SITE
FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM
HOTEL 1 DRAINAGE PLAN VIEW


HOTELS 2
DRAINAGE PLAN VIEW
N
S
E
W
0
100
200
Meters
HOTEL 2
DR5-4: 600 BURIED PIPE
DR5-3: 1No. 600 BURIED PIPE
DR4-2(1): 2-600 Buried Pipe
1No. 600dia RCCP
1.2 X1.2m CP with 0.5m sump
DR5-1: 4No. 600dia. Pipe
DR5-1:4No. 600dia. Pipe
DR4-2: 600 BURIED PIPE
DR3-5:600 BURIED PIPE
DR3-4: 450 BURIED PIPE
DR4-1: 2No. 600 Pipe
3.15 X3.15m CP with 0.5m sump
OUTFALL 5
4no. 600 pipes @ 0.75msl
DRAINAGE LEGEND
DRAIN CULVERT
PROPOSED ROAD WAY
OUTFALL POINT


HOTELS 3
DRAINAGE PLAN VIEW
HOTEL 1
HOTEL 3
DR4-2: 600 BURIED PIPE
DR5-1: 4No. 600dia. Pipe
DR5-1:4No. 600dia. Pipe
DR6-1:600xDV C. B. Drain
DR3-5:600 BURIED PIPE
DR3-4: 450 BURIED PIPE
DR4-1: 2No. 600 Pipe
DR4-1:2No. 600 Pipe
1000W x DV C.B. Drain with Inlets 10m c/c
3.15 X3.15m CP with 0.5m sump
OUTFALL 5
4no. 600 pipes @ 0.75msl
1.65 X1.65m CP with 0.5m sump
OUTFALL 4
2no. 600 pipes @ 0.75msl
OUTFALL 3
4no. 600 pipes @ 0.75msl
OUTFALL 2
2no. 600 pipes @ 0.75msl
DR2-1: 1000W x DV C.B. Drain
DR2-3:450 BURIED PIPE
COORDINATED WITH BLD FOUNDATION
2-600 BURIED PIPE
Low Flow Condition
DRAINAGE LEGEND
1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
DRAIN CULVERT
PROPOSED ROAD WAY
OUTFALL POINT
RUNOFF FLOW INTO EXISTING MANGROVE AREAS
RUNOFF FROM PROPOSED HOTEL SITE
FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM


HOTELS 3
DRAINAGE PLAN VIEW
N
E
S
W
0
100
200
Meters
HOTEL 3
HOTEL 4
1.65 X1.65m CP with 0.5m sump
3.15 X3.15m CP with 0.5m sump
1000W x DV C. B. Drain with Inlets 10m c/c
OUTFALL 4
2no. 600 pipes @ 0.75msl
OUTFALL 3
4no. 600 pipes @ 0.75msl
OUTFALL 2
2no. 600 pipes @ 0.75msl
1.65 X1.65m CP with 0.5m sump
Earthen Drain to Supply Pond in Low Flow Condition
RESERVED ROAD
DR1-3: 2-600 Buried Pipe
DR1-3:450 BURIED PIPE
DR1-3no.600 BURIED PIPE
2.4 X2.4m CP with 0.5m sump
Gabion Basket Weir Set at HAT Elev.
Earthen Drain to Supply Pond in Low Flow Condition
DRAINAGE LEGEND
1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
DRAIN CULVERT
PROPOSED ROAD WAY
OUTFALL POINT
RUNOFF FLOW INTO EXISTING MANGROVE AREAS
RUNOFF FROM PROPOSED HOTEL SITE
FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

2 Catchment Wide Hydrologic Analysis

2.1 Design Storm Frequencies

With reference to Section 3.3 of the National Works Agency – *Guidelines for Preparing Hydrologic and Hydraulic Design Reports for Drainage Systems of Proposed Developments (NWA – 2015)*, design of minor drainage systems should be designed for Storm Frequencies up to 10 years and major drainage systems should be designed for Storm Frequencies up to 100 years.

In addition, with reference to Section 4.2 of the NWA – 2015, detention basins should be designed for a 1 in 10-year storm frequency if the catchment area is less than 250ha and a 1 in 25-year storm frequency if the catchment area is greater than 250ha.

Further break down and guidelines on design storm frequencies are presented in The Government of Jamaica (GOJ) *Development and Investment Manual,* Volume 3 Section 1, Chapter 12, article 12.1, part (ix) as follows:

- Minor drainage systems designed to accommodate 1 in 5-year flood event.
- Major drainage system to be designed to accommodate 1 in 25-year flood event.
- Bridges designed for 1 in 50-year flood event.

Considering the above, the Drainage Master Plan for the proposed Princess Resorts development will be done to manage storm water within the site for a 1:25-year flood event. However, the impacts on said development and surrounding areas will be assessed for the 50-year and 100-year flood events.

2.2 Method of Hydrologic Assessment

The hydrologic assessment for the mangrove catchment will be guided by the SCS Runoff Curve Number Method developed by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS). This method will be executed using HEC-HMS. HEC-HMS is a numerical hydrologic modeling system developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. It is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing.

For this catchment, using the technique and software mentioned above, the following hydrologic models and assessments were performed for various storm frequencies/events:

i. Pre-developed hydrologic models representing the existing condition of the watershed. This model includes the undeveloped footprint of the proposed site.
ii. Post-developed hydrologic model of the watershed which represents the watershed post construction of the proposed site.

2.3 Design Rainfall and Climate Change

Rainfall data were taken from the gauge closest to the Princess Resorts' site on the Jamaica 24-hr. Extreme Rainfall (mm) Isohyetal Maps for the various return periods as shown in Table 2-1.

Table 2-1: 24hr extreme rainfall for Hanover, Jamaica.

Storm Frequency	Peak 24hr Rainfall(mm)
2	76.8
5	112.3
10	134.7
25	163.4
50	185.6
100	208.5

The impact of climate change was accounted for by using the projected changes in rainfall throughout Jamaica guided by the Climate Studies Group of Mona University of the West Indies in their 2017 publication of "*The State of the Jamaican Climate 2015*." Table 2-2 presents such changes for the western side (zone3) of Jamaica where the site is located:

Table 2-2: Projected %Changes in Rainfall by Season for the Western Side of Jamaica (Table 49. Climate Study Group Mona U.W.I..2017)

SEASON (months)	TIMELINE							
	2020's		2030's		2050's		2080's	
NDJ	3.27	16.13	2.15	26.56	1.63	29.71	7.1	35.1
FMA	1.12	28.36	-5.89	28.23	16.12	39.86	-1.09	36.23
MJJ	4.21	17.09	-11.84	12.77	-8.54	17.59	-29.46	4.98
ASO	-12.9	7.01	-25.13	3.17	-20.92	4.13	-26.92	0.29

As noted in the table above, the February-March-April ("FMA") season displayed the most significant increases in rainfall. Considering the design life of the drainage structures is typically 50 years as per ASCE 2005 engineering guidelines, the change within the entire timeline presented above should be considered. Hence, the mean value for the FMA season was calculated for the full timeline to be **17.87% increase** in peak rainfall. This percentage increase was used to project the peak rainfall used in the hydrologic analysis and drainage designs as follows:

Table 2-3: Projected 24hr Extreme Rainfall for Hanover, Jamaica.

Storm Frequency	*Peak 24hr Rainfall(mm)*	*24hr Rainfall Increased by 17.87% due to Climate Change (mm)*
2	76.8	**90.5**
5	112.3	**132.4**
10	134.7	**158.8**
25	163.4	**192.6**
50	185.6	**218.8**
100	208.5	**245.8**

The 24hr extreme rainfall data was then distributed over the 24-hour period (Figure 2.1) by using synthetic rainfall distributions developed by the United States, Natural Resource Conservation Service (NRCS) as described in the NCRS – TR 55 document. These rainfall distributions were developed for various parts of the United States including Florida such as Type I, IA, II and III.

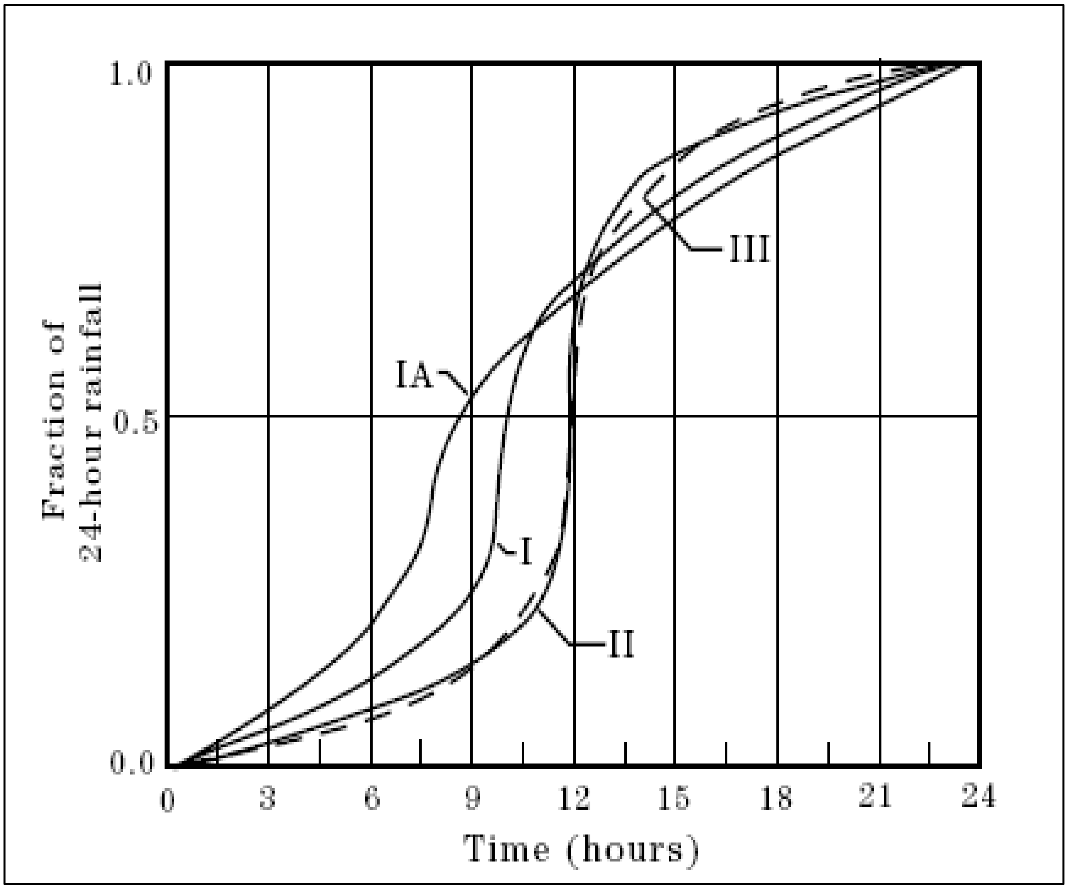


Figure 2.1: SCS 24hr Rainfall Distribution

Type IA represents least intense and Type II represents the most intense rainfall events. Considering Jamaica is just south of Florida, which is affected by tropical storms, the rainfall data for the site was distributed to represent a Type II temporal rainfall distribution.

2.4 Design Hyetographs

The rainfall data was distributed using the SCS Type II rainfall distribution. The following presents the resulting hyetographs for each design storm frequency using the rainfall data presented above. Each hyetograph was used as input meteorological data input into the HEC-HMS Models.

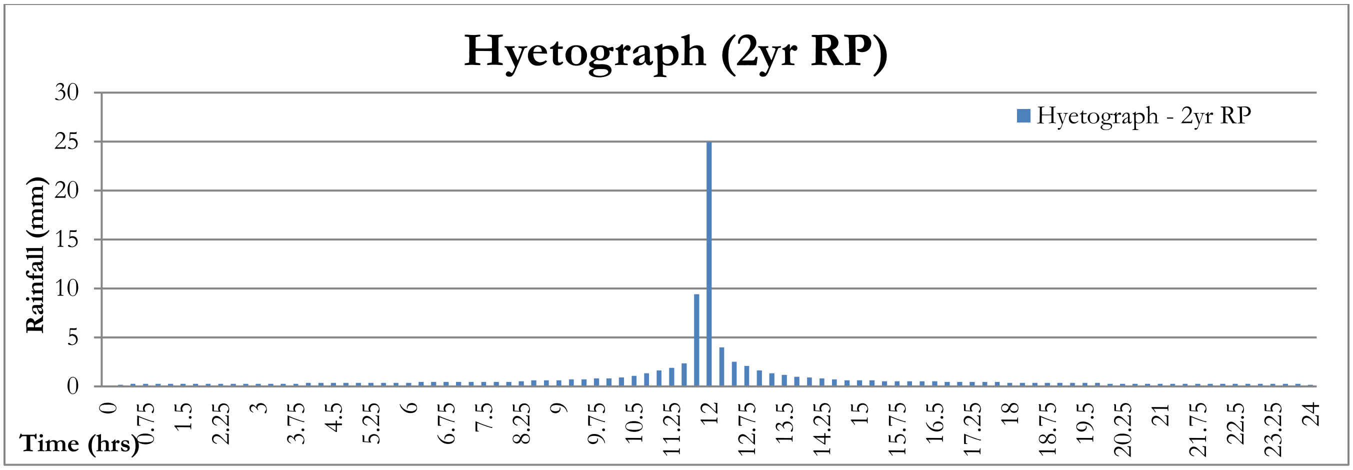


Figure 2.2: Rainfall Hyetograph for 2-year RP

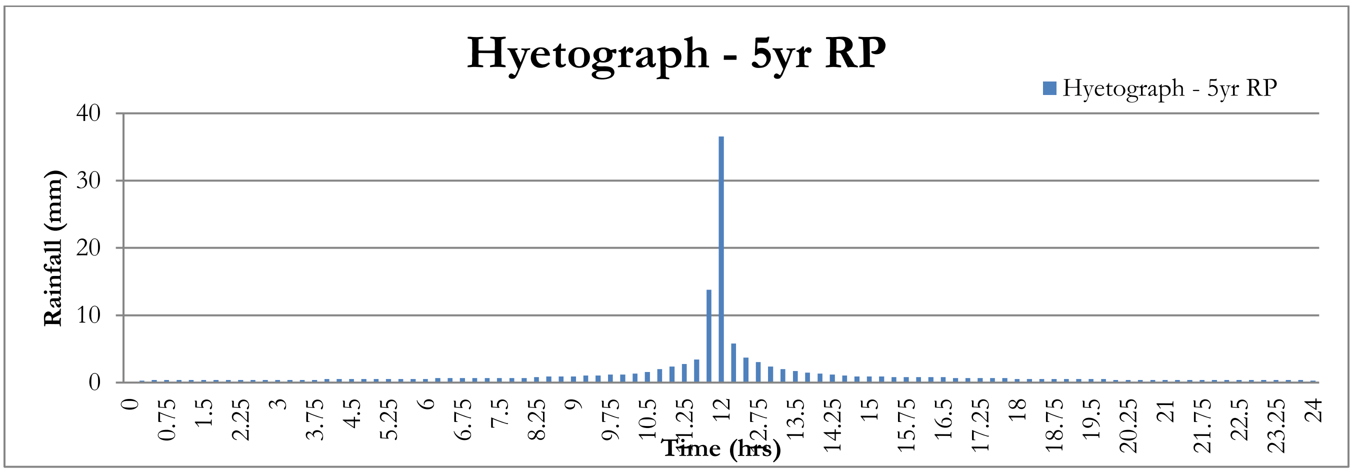


Figure 2.3: Rainfall Hyetograph for 5-year RP

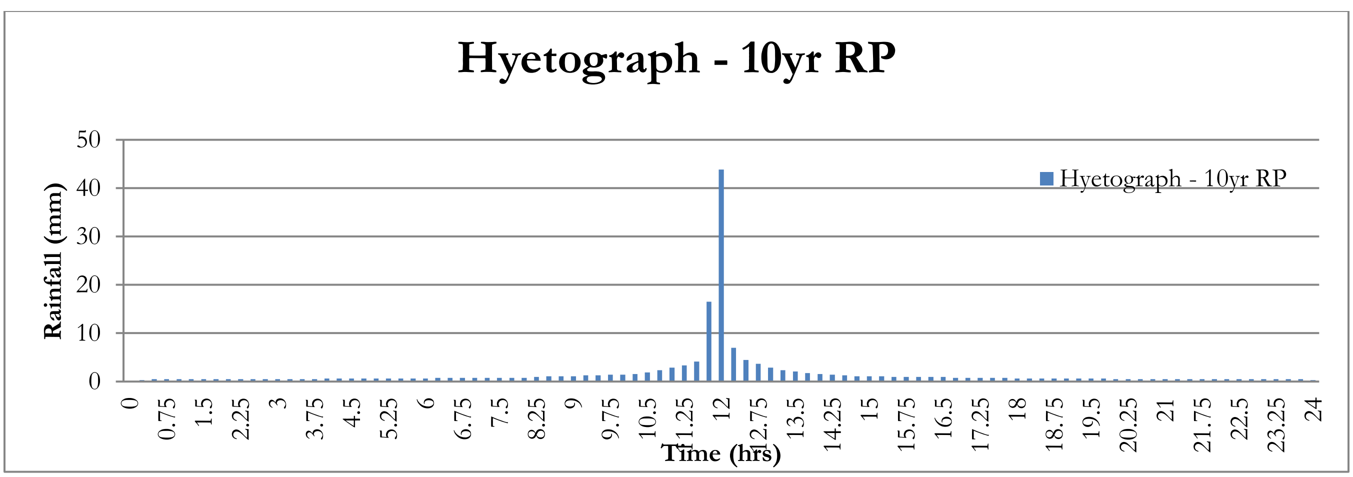


Figure 2.4: Rainfall Hyetograph for 10-year RP

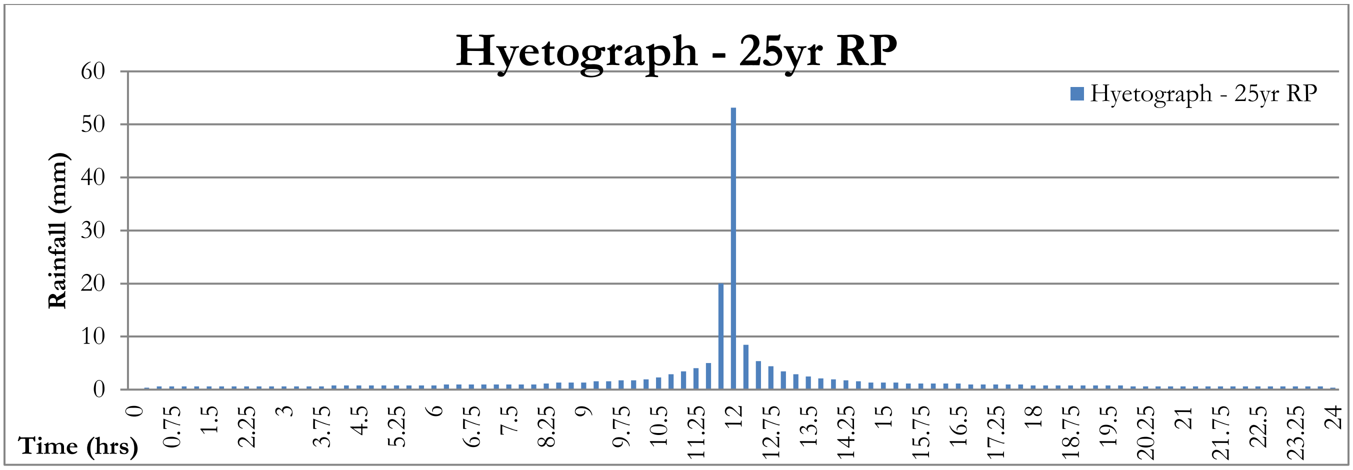


Figure 2.5: Rainfall Hyetograph for 25-year RP

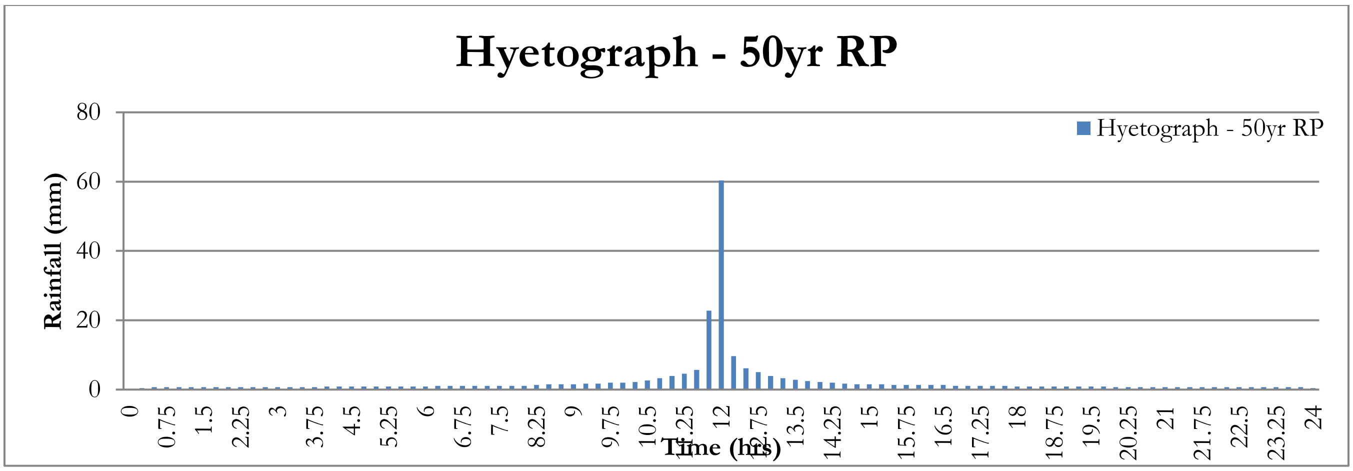


Figure 2.6: Rainfall Hyetograph for 50-year RP

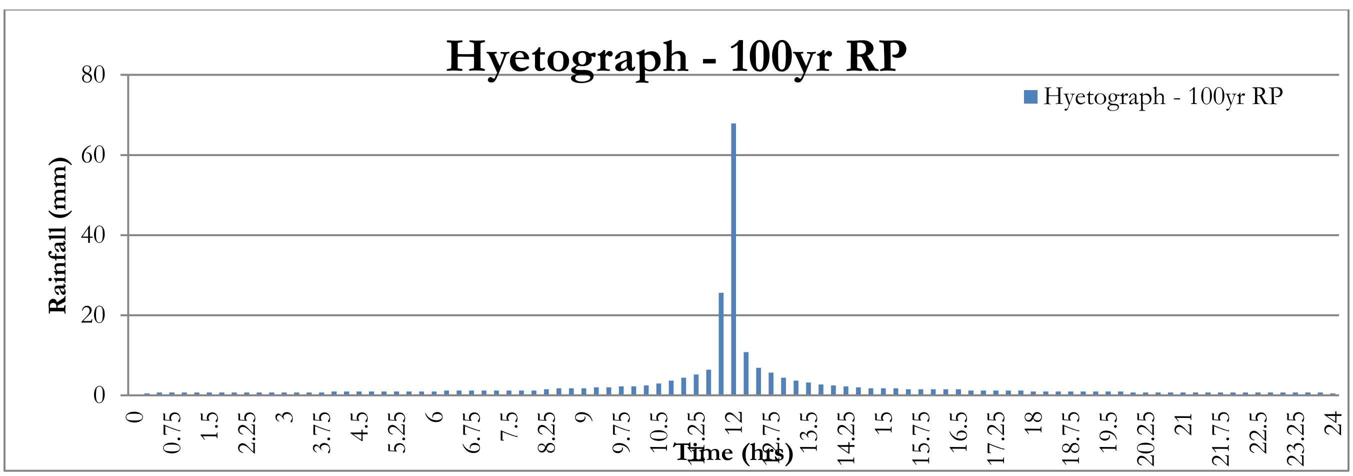


Figure 2.7: Rainfall Hyetograph for 100-year RP

2.5 Hydrologic Characteristics of Watershed and Catchment

The following presents the calculation of various hydrologic parameters that represent the hydrologic characteristics of the watershed or catchment under study and are used as input parameters into the hydrologic model. These are as follows:

- **Catchment Areas**: Catchment areas can be defined as the drainage area or area from which all rainfall runoff is collected upon flowing to the point of investigation.
- **Antecedent Moisture Content (AMC):** This was developed as part of the SCS Curve Number Method to describe the saturation of the soil within the catchment. There are three categories of AMC and they are as follows:
  AMC I: Low Moisture
  AMC II: Average moisture condition, usually used for annual flood estimates
  AMC III: high moisture, heavy rainfall over preceding days
- **Soil Type:** The SCS Curve Number Method also considers the various soil types as the rate of infiltration an initial loss is heavily depend on such. The four soil groups used in this method are as follows:
  Group A: Deep sand, deep Loess and aggregated silts
  Group B: Shallow Loess and sandy loam
  Group C: Clay loams, shallow sandy loams, soils low in organic content and soils high in clay
- **Curve Number (CN):** CN values are an empirical parameter used in hydrology for predicting direct runoff from rainfall excess. Refer to Exhibit 1: Runoff CN Values
- **Initial Abstractions (I_a):** This accounts for all losses before runoff begins. It includes water being retained by depressions, intercepted by vegetation, infiltration and evaporation.
- **Potential Maximum Retention (S):** this is related to the soil and cover condition of the watershed through the CN value. It represents the potential maximum retention of rainfall volume after runoff begins.
- **Time of Concentration (T_c):** The time a particle of water takes to travel from the furthest point of a watershed to the point under investigation.
- **Lag Time (T_L):** the time from the centre of mass of the rainfall excess to the peak discharge.

Table 2-4 presents the varying hydrologic characteristics under which the catchments were hydrologically assessed.

Table 2-4 Varying hydrologic characteristics under which the catchments were hydrologically assessed

WATERSHED UNDER PRE-DEVELOPED CONDITION:

O **Calculating Watershed Areas:**

Watershed Areas

Catchment Area, A =	2323894.50	sq. m	2.32	sq.km	100%
Developed Area, A_{dev}=	230576.80	sq. m	0.23	sq.km	9.92%
Undeveloped Area, A_{und}=	2093317.70	sq. m	2.09	sq.km	90.08 %

O **Defining Land Use and Calculating Curve Numbers (CN):**

Land Use	Area %	Soil Type	AMC	Curve Number , CN
Non-cultivated range with terrain	0.90	B	II	67
Residential with 1/2 lot sizes	0.10	B	II	75
Weighted CN =				**68**

O **Maximum Potential Retention, S** = (1000/CN) -10 = **4.75** **in**

O **Initial Abstraction, Ia** = 0.2S = **0.95** **in**

O **Lag Time (T_{lag})=** $\dfrac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$ = **1.19** **hr**

L=	2870	m	9416.01	ft
Upstream Elev.=	150	m		
Downstream Elev.=	0	m		
Y=	0.0522648	=	5.2	%

WATERSHED UNDER POST-DEVELOPED CONDITION:

O **Calculating Watershed Areas:**

	Watershed Areas				
Catchment Area, A =	2323894.50	sq.m	2.32	sq.km	100%
Developed Area, A_{dev}=	467702.46	sq.m	0.47	sq.km	20.13 %
Undeveloped Area, A_{und}=	1856192.04	sq.m	1.86	sq.km	79.87 %

O **Defining Land Use and Calculating Curve Numbers (CN):**

Land Use	Area %	Soil Type	AMC	Curve Number , CN
Non-cultivated range with terrain	0.80	B	II	67
Residential with 1/2 lot sizes	0.20	B	II	75
Weighted CN =				**69**

O **Maximum Potential Retention, S** = (1000/CN) - 10 = **4.58** **in**

O **Initial Abstraction, Ia** = 0.2S = **0.9150242** **in**

O **Lag Time (T_{lag})=** $\frac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$ = **1.16** **hr**

L=	2870	m	9416	ft
Upstream Elev.=	150	m		
Downstream Elev.=	0	m		
Y=	0.0522648	=	5.2	%

2.6 Hydrologic Response of The Catchment

Table 2-5 presents the hydrologic response of the catchments for various design storm frequencies from the HEC-HMS numerical models.

Table 2-5: Summary of discharge generated from watershed

Mangrove Watershed (Including Site) 17.87%					
Storm Event	Flow Rate (m3/s)		Volume (m3)		Detention Volume Req'd (m3)
	Pre-developed	Post-developed	Pre-developed	Post-developed	
2 year	4.5	5.8	58100	73200	15100
5 year	11.6	13.2	135300	155500	20200
10year	15.8	17.7	182000	204200	22200
25year	18.7	20.7	213,200	236600	23400
50year	26.2	28.6	296400	322200	25800
100year	31.0	33.6	350500	377500	27000

Refer to Appendix for Hydrograph Output from HEC-HMS.

3 Hydraulic Assessment and Improvement to Mangrove

3.1 Method of Hydraulic Assessment

The hydraulic assessment involves estimating the flood depths within the mangrove under pre-developed condition. The projected increase in flood depths expected after the proposed development has been established are also estimated. These were carried out for various storm frequencies and impacts were assessed.

The flood depths in the mangrove were estimated using a stage storage curve developed from the terrain within the mangrove assuming zero out flow.

3.2 Storage Capacity of Mangrove

From the topographic survey and the 1:2500 contour maps for the area, it was observed that the mangrove is a flat area with natural depressions as ponds at various locations with ground elevations close to and sometimes lower than 0m MSL. Using topographic surveys for the area, it was noted that the mangrove can potentially retain water at 0.4 to 0.5m MSL before discharging excess runoff into the sea from these natural low points located to the east and the west ends of the mangrove. Hence, the mangrove can be described as the natural retention system for the watershed where it stores water at an elevation of 0.3 to 0.4m and releases water into the sea when water levels are above that. To assess the potential storage capacity of the mangrove, a stage-storage curve was developed using AutoCAD Civil 3D and is presented in Figure 3.1.

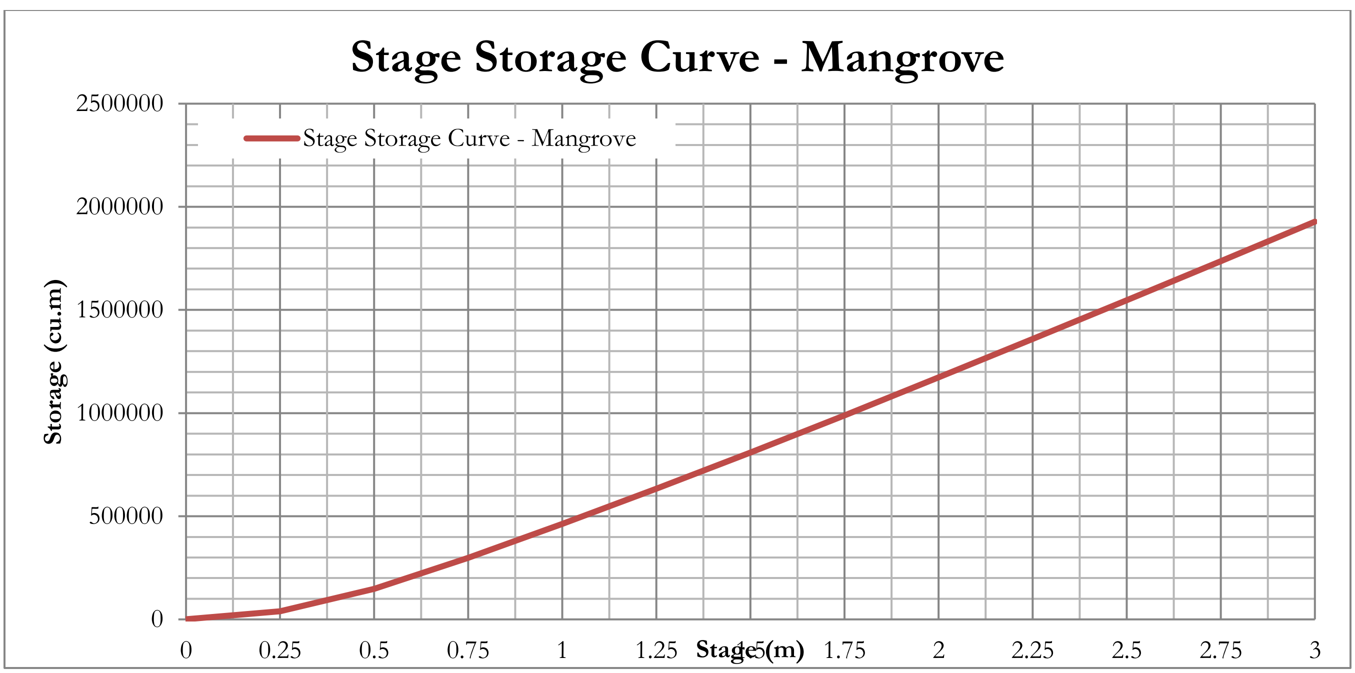


Figure 3.1: Stage-storage curve of mangrove

From the above stage-storage curve the following water levels were derived within the swamp for various storm events (Table 3-1).

Table 3-1: Flood level within mangrove

Projected Water Level Within Mangrove (m MSL) (17.87% Increase in Rainfall Due To Climate Change)			
Storm Event	**Pre-developed**	**Post-Developed**	**Difference**
2yr	0.35	0.375	0.025
5yr	0.49	0.532	0.042
10yr	0.573	0.607	0.034
25yr	0.62	0.656	0.036
50yr	0.747	0.795	0.048
100yr	0.836	0.876	0.040

From the table above, it can be seen that the flood elevation will be increased by 40mm for a 100-year storm event if all storm water runoff from the proposed site is discharged into the mangrove. The 100-year floodplain was delineated along the 0.9m MSL contour line within the mangrove and a 30m buffer was established as per *Section 1.6.4 of the NWA-2015* document. From this, it can be deemed that the increased flood elevation of 40mm for a 100-year event has negligible impacts to adjacent communities.

3.3 Improving the Storage Capacity and Flood Rehabilitating Areas within the Mangrove

Previous sections indicated that allowing the proposed development to drain into the mangrove provides negligible impacts on the mangrove and its surrounding communities. Nevertheless, several ponds are proposed within the barren elevated areas in the midst of the mangrove. Such elevated areas would be excavated to 0m MSL to increase the storage capacity of the mangrove and encourage expansion of the mangrove's flora. The Master Plan shows the location and extents of these ponds while Table 3-2 below shows the additional volume provided and the respective decrease in projected flood elevations.

Table 3-2: Increase in Storage Capacity and Mangrove Area from Proposed Ponds

Pond	Increase in Storage Capacity (m^3)	Increase in Mangrove Area (m^2)
Pond 1	7,066	12,602
Pond 2	7,978	16,933
Pond 3	1,149	3,413
Total =	**16,193**	**32,948**

3.4 Improving Flow throughout the Mangrove

There exists a small road network within the mangrove that has effectively reduced the free flow of water throughout the mangrove resulting some areas being deprived of sufficient water. This Master Drainage Plan presents the introduction of several culverts opening under these roads at frequent and strategic locations to improve the flow of water through the mangrove. This will inherently improve the effective use of the storage capacity the mangrove possesses as well as promote the transportation of water and nutrients through the mangrove, promoting a healthier ecosystem. Refer to the Master Drainage Design drawing for the locations of these proposed culverts.

The western area of the forest has a previously reclaimed and raised area. The two sites are separated by an access road with no mangrove species within the middle of the area, but small recruiting mangroves on the periphery. Mangroves have not naturally re-established on the site as the elevation is too high for tidal wetting. Instead, the area has several plants associated with disturbed and freshwater wetlands, which have outcompeted the surrounding mangrove vegetation.

The sites are dominated by Logwood trees (*Haematoxylum campechianum*), *Spartina sp.* (cordgrass) and mangrove ferns (*Acrostichum aureum*). The outline of the footprint is found in Figure 3.2 below.

Figure 3.2: Current footprint of degraded mangroves-raised areas suitable for mitigation and created drainage features

These degraded mangrove areas present an opportunity for rehabilitation. These areas may be used to recreate a mangrove habitat with specialized drainage collection functions. The location is ideal for mitigation purposes as the area is physically separated from the proposed hotel by over 100m of dense mangrove forest and surrounded by mangrove conservation zones to the south and east.

The areas could be connected by culverts, excavated to a substrate level sufficiently deep to prevent natural mangrove seedling establishment (1-1.5m). A natural drainage creek was observed in the northern section which displayed a northern flow of water to the forest, then west to exit the mangrove forest. The pond creation should use the existing drainage point.

This will serve multiple functions including:

- Water retention;
- Sediment and nutrient trapping;
- Bird wading and feeding habitat;
- Improved aesthetic appearance;
- Creation of an eco-tourism feature.

Isolated mounds of substrate can be retained within the created pond footprint, which will enhance the drainage feature aesthetically, in addition to providing optimal nesting and perching areas for birds. An example of a similar concept is seen in Figure 3.3 below.

Figure 3.3: Drainage pond concept showing raised areas with mangrove islets Photo courtesy of: https://www.hindustantimes.com/mumbai-news/in-4-years-94-000-mangrove-saplings-died-in-mumbai/story-OMug9kwCpizjv71x2iNfdI.html

4 Hydrologic Assessment of The Proposed Site

4.1 Drainage Design Concept for Proposed Site

The purpose of the Drainage Master Plan is to ensure that the main drains and outfall points are coordinated with the proposed development to ensure feasibility at the detail design stage. Hence, the hydrologic analysis of the site is limited to the sizing of all outfall points and the main drains used to convey rainfall runoff to these outfall points for each catchment.

4.1.1 Hydrologic Concept

The drainage system was designed for a 1:25 year recurrence interval as presented previously in this report. All flows within the site were calculated using the Rational Method since the catchment areas of the site are less than 100 hectares (≈ 1 km^2).

4.1.2 Hydraulic Network

The drainage system includes a variety of underground piping to facilitate the Client's design concept of the finished environment. These are supplemented by catch pits, manholes and catch basin-manholes at various locations to facilitate both the hydraulic and maintenance requirements. The Manning's Equation was primarily used to estimate the required capacities.

4.2 Hydrologic Design Criteria

4.2.1 Hydrologic Design Criteria

The analysis was governed by the Rational Method since the areas of the catchments are less than 100 hectares. The rational equation is as follows:

$Q = FCiA$

Where: Q = Peak Discharge (m^3/s)
c = runoff coefficient
i = Rainfall Intensity (mm/hr)
A = Catchment Area (km^2)
F= unit constant = 0.278

4.2.2 Catchment Areas (A)

This represents the overland area that contributes flow to the channel/point under investigation.

Impervious Factor C for the Site:

- c=0.2:- Used for pre-developed area with grass cover over primarily sand
- c=0.9:- Used to represent the building roofs and paved areas

4.2.3 Rainfall and Climate Change

The rainfall was estimated for Green Island by using the Jamaica 24hr Extreme Rainfall Isohyetal Maps and increasing these values by 17.87% as presented previously in this report.

When using the rational method, the 24-hour rainfall values were adjusted based on the design rainfall duration for each storm frequency and catchment as per Appendix III of the NWA-2015 Guidelines. Table 4-1 shows the ratios and constants used for such adjustments.

Table 4-1: Ratios and constants for storm frequency analysis

Duration (D)	Cn	Ratios for Return Period (Tyr) and Duration (D)				
		5yr	10yr	25yr	50yr	100yr
10 min		0.4	0.4	0.4	0.4	0.4
15 min		0.52	0.52	0.52	0.52	0.52
20min (extrapolated)		0.64	0.64	0.64	0.64	0.64
30 min		0.76	0.76	0.76	0.76	0.76
1 hour		0.41	0.4	0.38	0.37	0.35
2 hours	0.25					
6 hours	0.5					
12 hours	0.69					
24 hours		1	1	1	1	1

4.2.4 Time of Concentration (t_c)

This is defined as the time taken for a drop of water to travel from the most remote point to the point under investigation. The kirpich equation was used and can be defined as follows:

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where: t_c = time of concentration (mins)
L = longest flow path (m)
S = slope along the flow path

4.3 Hydrologic Flow Calculations for Site Catchment Areas and Outfalls

The hydrologic flow calculations for each site catchment were done using the design criteria outlined above. The rainfall intensity of each site catchment is presented below.

Member Ref.	CALCULATIONS	CODE

CATCHMENT 1 - OUTFALL 1

RATIONAL METHOD:

Area:

Total Catchment=	0.028784	km2
Paved	0.024466	km2
Grass/Sand	0.004318	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation:

Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	314	m
Upstream Elev=	3	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0073248	

Hence;

t_c = 10.83 mins

There, Peak Time, Tp = (2/3) tc= 7.22 mins

Duration or Base Time, Tb = (8/3)tp= 19.26 mins

Chow Et Al

Duration Used = 20.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact:

Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	40.65	46.84	51.80	55.05

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	121.94	140.52	155.41	165.15

Runoff (Q):

Chow Et Al

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.776	0.894	0.989	1.051

Member Ref.	CALCULATIONS	CODE

CATCHMENT 2 - OUTFALL 2

RATIONAL METHOD:

Area:

Total Catchment=	0.027306	km2
Paved	0.023210	km2
Grass/Sand	0.004096	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: — Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	314	m
Upstream Elev=	3	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0073248	

Hence;

t_c = 10.83 mins

There, Peak Time, Tp = (2/3) tc= 7.22 mins

Duration or Base Time, Tb = (8/3)tp= 19.26 mins — Chow Et Al

Duration Used = 20.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: — Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	40.65	46.84	51.80	55.05

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	121.94	140.52	155.41	165.15

Runoff (Q): — Chow Et Al

Where: F= 0.278

Q=FciA =

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.736	0.848	0.938	0.997

Member Ref.	CALCULATIONS	CODE

CATCHMENT 3 - OUTFALL 3

RATIONAL METHOD:

Area:

Total Catchment=	0.056192	km2
Paved	0.036525	km2
Grass/Sand	0.019667	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: — Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	363	m
Upstream Elev=	3	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0063361	

Hence;

t_c = 12.81 mins

There, Peak Time, Tp = (2/3) tc= 8.54 mins

Duration or Base Time, Tb = (8/3)tp= 22.77 mins — Chow Et Al

Duration Used = 20.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: — Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	40.65	46.84	51.80	55.05

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	121.94	140.52	155.41	165.15

Runoff (Q): — Chow Et Al

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	1.248	1.438	1.590	1.690

Member Ref.	CALCULATIONS	CODE

CATCHMENT 4 - OUTFALL 4

RATIONAL METHOD :

Area:

Total Catchment=	0.034316	km2
Paved	0.020590	km2
Grass/Sand	0.013726	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: — CODE: Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	447	m
Upstream Elev=	3	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0051454	

Hence;

t_c = 16.29 mins

There, Peak Time, Tp = (2/3) tc= 10.86 mins

Duration or Base Time, Tb = (8/3)tp= 28.96 mins — CODE: Chow Et Al

Duration Used = 30.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: — CODE: Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	48.27	55.62	61.52	65.37

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	96.53	111.25	123.03	130.74

Runoff (Q): — CODE: Chow Et Al

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.571	0.658	0.728	0.773

Member Ref.	CALCULATIONS	CODE

CATCHMENT 5 - OUTFALL 5

RATIONAL METHOD :

Area:

Total Catchment=	0.077811	km2
Paved	0.046687	km2
Grass/Sand	0.031124	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: (CODE: Chow Et Al)

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	506	m
Upstream Elev=	3.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0055336	

Hence;

t_c = 17.42 mins

There, Peak Time, Tp = (2/3) tc= 11.62 mins

Duration or Base Time, Tb = (8/3)tp= 30.98 mins (CODE: Chow Et Al)

Duration Used = 30.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: (CODE: Appendix III NWA)

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	48.27	55.62	61.52	65.37

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	96.53	111.25	123.03	130.74

Runoff (Q): (CODE: Chow Et Al)

Where: F= 0.278

Q=FciA =

	10yr	25yr	50yr	100yr
Q (m3/s)=	1.295	1.492	1.650	1.753

Member Ref.	CALCULATIONS	CODE

CATCHMENT 6 - OUTFALL 6

RATIONAL METHOD:

Area:

Total Catchment=	0.018379	km2
Paved	0.015622	km2
Grass/Sand	0.002757	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: (Code: Chow Et Al)

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	200	m
Upstream Elev=	2.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.009	

Hence;

t_c = 7.07 mins

There, Peak Time, Tp = (2/3) tc= 4.71 mins

Duration or Base Time, Tb = (8/3)tp= 12.57 mins (Code: Chow Et Al)

Duration Used = 15.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: (Code: Appendix III NWA)

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	33.02	38.06	42.09	44.73

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	132.10	152.23	168.36	178.91

Runoff (Q): (Code: Chow Et Al)

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.537	0.618	0.684	0.727

Member Ref.	CALCULATIONS	CODE

CATCHMENT 7 - OUTFALL 7

RATIONAL METHOD:

Area:

Total Catchment=	0.007029	km2
Paved	0.006326	km2
Grass/Sand	0.000703	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: — Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	98	m
Upstream Elev=	2.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0183673	

Hence;

t_c = 3.10 mins

There, Peak Time, Tp = (2/3) tc= 2.07 mins

Duration or Base Time, Tb = (8/3)tp= 5.51 mins — Chow Et Al

Duration Used = 10.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: — Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	25.40	29.28	32.38	34.41

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	152.42	175.65	194.26	206.44

Runoff (Q): — Chow Et Al

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.247	0.285	0.315	0.335

Member Ref.	CALCULATIONS	CODE

CATCHMENT 8 - OUTFALL 8

RATIONAL METHOD:

Area:

Total Catchment=	0.005492	km2
Paved	0.004668	km2
Grass/Sand	0.000824	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	115	m
Upstream Elev=	2.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0156522	

Hence;

t_c = 3.73 mins

There, Peak Time, Tp = (2/3) tc= 2.49 mins

Duration or Base Time, Tb = (8/3)tp= 6.63 mins — Chow Et Al

Duration Used = 5.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	25.40	29.28	32.38	34.41

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	304.84	351.30	388.53	412.88

Runoff (Q): Chow Et Al

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.370	0.426	0.472	0.501

Member Ref.	CALCULATIONS	CODE

CATCHMENT 9 - OUTFALL 9

RATIONAL METHOD:

Area:

Total Catchment=	0.004724	km2
Paved	0.004015	km2
Grass/Sand	0.000709	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand
Undeveloped Area=	0.20	Grassed with 2% to 7% slope
Paved: c=	0.90	Interlocking Pavers

Time of Concentration (Tc) using Kirpich Equation: Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	118	m
Upstream Elev=	2.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.0152542	

Hence;

t_c = 3.84 mins

There, Peak Time, Tp = (2/3) tc= 2.56 mins

Duration or Base Time, Tb = (8/3)tp= 6.83 mins — Chow Et Al

Duration Used = 5.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: Appendix III NWA

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	25.40	29.28	32.38	34.41

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	304.84	351.30	388.53	412.88

Runoff (Q):

Q=FciA = Where: F= 0.278 — Chow Et Al

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.318	0.367	0.406	0.431

Member Ref.	CALCULATIONS	CODE

CATCHMENT 10 - OUTFALL 10

RATIONAL METHOD:

Area:

Total Catchment=	0.005095	km2
Paved	0.004331	km2
Grass/Sand	0.000764	km2

Land Cover/Runoff Coefficient:

Soil Group =	D	Silty Sand	
Undeveloped Area=	0.20	Grassed with 2% to 7% slope	
Paved: c=	0.90	Interlocking Pavers	

Time of Concentration (Tc) using Kirpich Equation: Chow Et Al

$$t_c = 0.0195 \frac{L^{0.77}}{S^{0.385}}$$

Where :

L= Longest Flow Path=	120	m
Upstream Elev=	2.5	m msl
Downstream Elev.=	0.7	m msl
Slope=	0.015	

Hence;

t_c = 3.92 mins

There, Peak Time, Tp = (2/3) tc= 2.61 mins

Duration or Base Time, Tb = (8/3)tp= 6.97 mins (Chow Et Al)

Duration Used = 5.00 mins

Rainfall using Jamaica 24hr Extreme Rainfall Isohyetal Maps and Increased by 7.5% for Climate Change Impact: (Appendix III NWA)

	10yr	25yr	50yr	100yr
24 hr Rainfall (mm)	134.70	163.40	185.6	208.5
24 hr Rainfall Increased by 17.87% (mm)	158.77	192.60	218.77	245.76
Rainfall (mm) for Storm Duration	25.40	29.28	32.38	34.41

Intensity (i) :

i = Rainfall (mm)/Tc (hr) =

	10yr	25yr	50yr	100yr
Rainfall Intensity (mm/hr)	304.84	351.30	388.53	412.88

Runoff (Q): (Chow Et Al)

Q=FciA = Where: F= 0.278

	10yr	25yr	50yr	100yr
Q (m3/s)=	0.343	0.396	0.438	0.465

4.4 Hydrologic Flow Calculations for Internal Site Drainage

The hydrologic flow calculations for each of the main drains proposed in the drainage layout are presented below. The design rainfall intensity for each drain was taken as the design rainfall intensity for the catchment within which they are located and calculated as shown previously.

Table 4-2: Hydrologic Design Flow for Internal Drains for 1 in 25yr Storm Frequency

Hydrologic Flows within Internal Drains Incorporating Increase in Rainfall By 17.87% Due To Climate Change							
Drains	**Total Catchment (Km2)**	**Paved Area (Km2)**	**Grassed/ Sand Area (Km2)**	**Runoff Coeff. For Paved Area (C)**	**Runoff Coeff. For Grass/ Sand Area (C)**	**25 Yr Rainfall Intensity (Mm/Hr)**	**25 Yr Design Discharge (M3/S)**
Catchment 1:							**0.89**
Drain 1-2	0.006427	0.004774	0.001653	0.9	0.2	140.52	**0.18**
Drain 1-3	0.006374	0.003035	0.003339	0.9	0.2	140.52	**0.13**
Catchment 2:							**0.85**
Drain 2-1	0.027306	0.0232101	0.0040959	0.9	0.2	140.52	**0.85**
Drain 2-2	0.012402	0.009643	0.002759	0.9	0.2	140.52	**0.36**
Drain 2-3	0.00388	0.002497	0.001383	0.9	0.2	140.52	**0.10**
Catchment 3:							**1.44**
Drain 3-1	0.056192	0.0365248	0.0196672	0.9	0.2	140.52	**1.44**
Drain 3-2	0.01716	0.011651	0.005509	0.9	0.2	140.52	**0.45**
Drain 3-3	0.016058	0.007831	0.008227	0.9	0.2	140.52	**0.34**
Drain 3-4	0.007507	0.005022	0.002485	0.9	0.2	140.52	**0.20**
Catchment 4:							**0.66**
Drain 4-1	0.034316	0.0205896	0.0137264	0.9	0.2	111.25	**0.66**
Drain 4-2	0.021258	0.013762	0.007496	0.9	0.2	111.25	**0.43**
Catchment 5:							**1.49**
Drain 5-1	0.077811	0.0466866	0.0311244	0.9	0.2	111.25	**1.49**
Drain 5-2	0.035322	0.027933	0.007389	0.9	0.2	111.25	**0.82**
Drain 5-3	0.023856	0.011928	0.011928	0.9	0.2	111.25	**0.41**
Drain 5-4	0.021192	0.010034	0.011158	0.9	0.2	111.25	**0.35**
Drain 5-5	0.011466	0.005971	0.005495	0.9	0.2	111.25	**0.20**
Catchment 6:							**0.62**
Drain 6-1	0.018379	0.01562215	0.00275685	0.9	0.2	152.23	**0.62**
Catchment 7:							**0.28**
Drain 7-1	0.003943	0.0035487	0.0003943	0.9	0.2	175.65	**0.16**
Drain 7-2	0.003086	0.0027774	0.0003086	0.9	0.2	175.65	**0.13**
Catchment 8:							**0.43**
Drain 8-1	0.0021968	0.00186728	0.00032952	0.9	0.2	351.30	**0.17**
Drain 8-2	0.0032952	0.00280092	0.00049428	0.9	0.2	351.30	**0.26**
Catchment 9:							**0.37**
Drain 9-1	0.0018896	0.00160616	0.00028344	0.9	0.2	351.30	**0.15**
Drain 9-2	0.0028344	0.00240924	0.00042516	0.9	0.2	351.30	**0.22**
Catchment 10:							**0.40**
Drain 10-1	0.005095	0.00433075	0.00076425	0.9	0.2	351.30	**0.40**

5 Hydraulic Analysis of Site Drainage System

5.1 Hydraulic Design Criteria

5.1.1 Discharge Capacity:

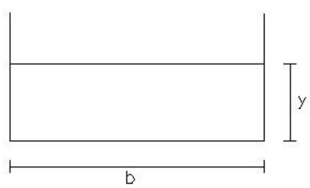
The Manning's Equation was used as the basis for the drainage designs in assessing the discharge capacity of the various drainage elements. This is as follows:

$$\boxed{Q = \frac{1}{n} A M^{\frac{2}{3}} i^{\frac{1}{2}}}$$

Where:
Q = Discharge Capacity (m3/s)
n = Manning's Coefficient
A = Cross sectional Area of Drain (m2)
M = Hydraulic Radius (m)
i = Slope of Channel (m/m) (Chadwick 2004, 31)

5.1.2 Geometrical Properties for Sections Utilized:

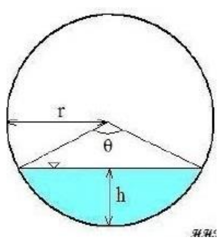
Box Sections:



A= b*y, P= (2*y) +b, M= (b*y) / [(2*y) +b], D_m = y

Circular Sections:

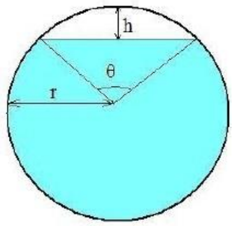
Partially full pipes less than half full:



A= [r^2 (θ-sine θ)]/2, P=r*θ, M= A/P, h=y,

θ=2arccos(r-h/r), D_m = 0.125D * [(θ-sinθ)/sin0.5θ)]

Partially full pipes more than half full:

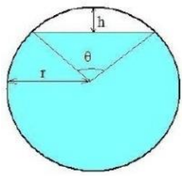


$A = \pi r^2 - [r^2 (\Theta\text{-sine } \Theta)]/2$, $P = 2\pi r - r\Theta$, $M = A/P$, $h = 2r - y$

$\Theta = 2\arccos(r-h/r)$, $D_m = 0.125D * [(\Theta - \sin\Theta)/\sin 0.5\Theta)]$

Where:
A = Cross sectional area of flow, P = Wetted Perimeter, M = Hydraulic Radius, D_m = Hydraulic Mean Depth

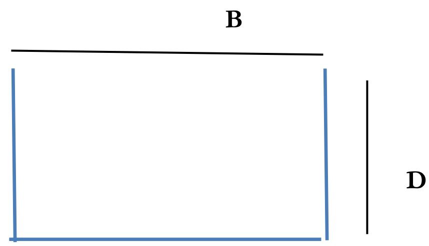
5.2 Sample Hydraulic Design Calculations for Circular Section

Member Ref.	CALCULATIONS	OUTPUT
	OUTFALL PIPE 2:	
	- 	
	Designed for 25 year Reccurence Interval:	
	Qdes = 0.850 m^3/s	
	General Design Data for Pipe More than Half Full:	
	D = 0.60 m	
	r = 0.30 m	
	h = 0.11 m	
	n = 0.011	
	i = 0.005	
	φ = 2.48 rads	
	Design Drain Capacity using Equations for More than Half Full:	
	Using Manning's Equation = $1/n * A * M^{2/3} * I^{1/2}$	
	Where:	
	Area, A = 0.198692965 m2	
	Wetted Perimeter, P = 1.140593398 m	
	Hydraulic Radius, M = (A/P)= 0.174201398 m	
	Hence, the Design Drain Capacity, Q_{cap} = **0.398 m3/s**	
	No. of Pipes= 3	
	Total Discharge = 1.195172209	**OK!**
	Velocity Check:	
	Velocity = Q/A = 2.01 m/s	
	Velocity < 3.5m/s (for concrete surface in accordance with NWA-2015)	**OK**
	Velocity >1 m/s (for concrete surface in accordance with NWA-2015)	**OK**

5.3 Sample Hydraulic Design Calculations for Box Section

Member Ref.	CALCULATIONS	OUTPUT

<u>DRAIN 2-1</u>



Design flow, Q_{des} = 0.850 m^3/s

<u>General Design Data:</u>

B	=	1.00	
D	=	0.45	m
n	=	0.013	
i	=	0.005	

<u>Design Drain Capacity</u>

Using Manning's Equation = $1/n * A * M^{2/3} * I^{1/2}$

where :

Manning's coefficient, n

Area, A = B * D = 0.4500 m^2

Wetted Perimeter, P = 2D + B = 1.9000 m

Hydraulic Radius, M = $(A/P)^{2/3}$ = 0.2368 0.3828006

Hence, the Design Drain Capacity, Q_{cap} = **0.937** m^3/s

Since the design drain capactiy Q_{cap} > Design Flow Q_{des} **O.K**

<u>Velocity Check:</u>

Solving for Actual Flow Depth by equating Manning's eq'n to Qdes and making "d"

(flow depth) the variable. From trial and error:

Flow Depth = 0.42 m

Hence, Flow Area = width x flow depth = **0.42** m2

Therefore, Flow Velocity, V= Qdes/Flow Area = 2.0238095 m/s

Velocity < 3.5m/s (for concrete surface in accordance with NWA-2015) **OK**

Velocity >1 m/s (for concrete surface in accordance with NWA-2015) **OK**

5.4 Hydraulic Design Calculations

The hydraulic sizing of the internal drains was done to ensure that the sizes being proposed would be sufficient to convey the required flow during detail designs. The following table presents the proposed sizes, slopes and discharge capacities.

Table 5-1: Hydraulic Sizing of Internal Drains

HYDRAULIC SIZING OF INTERNAL DRAINS FOR FLOW DERIVED UTILIZING A 17.87% INCREASE IN RAINFALL DUE TO CLIMATE CHANGE										
Drain	**25 Yr Design Discharge (m3/s)**	**Type**	**No. of Drains**	**Dia. (m)**	**Width (m)**	**Heigth (m)**	**Manning's Roughness Coeff. (n)**	**Slope**	**Design Capacity (m3/s)**	**Design Capacity Check (Q_{des}<Q_{cap})**
Outfall 1:	**0.89**	PIPE	3	0.60			0.011	0.005	1.195	OK!
Drain 1-2	**0.18**	PIPE	1	0.45			0.011	0.005	0.202	OK!
Drain 1-3	**0.13**	PIPE	1	0.45			0.011	0.005	0.202	OK!
Outfall 2:	**0.85**	PIPE	3	0.60			0.011	0.005	1.195	OK!
Drain 2-1	**0.85**	COVERED BOX	1		1.00	0.45	0.013	0.005	0.937	OK!
Drain 2-2	**0.36**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 2-3	**0.10**	PIPE	1	0.45			0.011	0.005	0.202	OK!
Outfall 3:	**1.44**	PIPE	4	0.60			0.011	0.006	1.746	OK!
Drain 3-1	**1.44**	PIPE	4	0.60			0.011	0.006	1.746	OK!
Drain 3-2	**0.45**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 3-3	**0.34**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 3-4	**0.20**	PIPE	1	0.45			0.011	0.005	0.202	OK!
Outfall 4:	**0.66**	PIPE	2	0.60			0.011	0.006	0.873	OK!
Drain 4-1	**0.66**	PIPE	2	0.60			0.011	0.006	0.873	OK!
Drain 4-2	**0.43**	PIPE	1	0.60			0.011	0.006	0.436	OK!
Outfall 5:	**1.49**	PIPE	4	0.60			0.01	0.005	1.594	OK!
Drain 5-1	**1.49**	PIPE	4	0.60			0.011	0.005	1.594	OK!
Drain 5-2	**0.82**	PIPE	3	0.60			0.011	0.005	1.195	OK!
Drain 5-3	**0.41**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 5-4	**0.35**	PIPE	1	0.60			0.011	0.005	0.398	OK!
Drain 5-5	**0.20**	PIPE	1	0.45			0.011	0.005	0.202	OK!
Outfall 6:	**0.62**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 6-1	**0.62**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Outfall 7:	**0.28**	PIPE	2	0.45			0.011	0.005	0.403	OK!
Drain 7-1	**0.16**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Drain 7-2	**0.13**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Outfall 8:	**0.43**	PIPE	2	0.60			0.011	0.005	0.797	OK!
Drain 8-1	**0.17**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Drain 8-2	**0.26**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Outfall 9:	**0.37**	PIPE	2	0.45			0.011	0.005	0.403	OK!
Drain 9-1	**0.15**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Drain 9-2	**0.22**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!
Outfall 10:	**0.40**	PIPE	2	0.45			0.011	0.005	0.403	OK!
Drain 10-1	**0.40**	COVERED BOX			1.00	0.45	0.013	0.005	0.937	OK!

6 Conclusion

6.1 Summary of Design Conditions for the Proposed Site

The key outcomes of the Master Drainage Plan design are as follows:

- The proposed site drainage follows the natural flow path and drains freely into the mangrove which increases the projected flood elevation in the mangrove as follows:

Projected water level within mangrove (m MSL) (17.87% increase in rainfall due to climate change)			
Storm Event	**Pre-developed**	**Post-Developed**	**Difference**
2yr	0.35	0.375	0.025
5yr	0.49	0.532	0.042
10yr	0.573	0.607	0.034
25yr	0.62	0.656	0.036
50yr	0.747	0.795	0.048
100yr	0.836	0.876	0.040

- To account for climate change, the peak 24hr rainfall was increased by 17.87% as per the projections. Additionally, recognizing that climate change will affect other variables that influence rainfall (i.e. intensity and duration), the site was designed to withstand return periods that are greater than industry standards.
- The increases in flood elevation within the mangrove were deemed negligible and do not impact the surrounding communities and infrastructure.
- The proposed drainage for the site was designed for a 1 in 25-year storm frequency using ten outfall points into the mangrove therefore maintaining smaller catchment areas, shorter flow paths and smaller drain sizes.
- These outfall points were set above the 1 in 50-year projected flood elevation within the mangrove and therefore provided the level of protection to the proposed development. Further, an economical protection is added against the projected 1 in 100-year flood elevation within the mangrove by implementing flap gates to the outfall pipes.
- Several culverts were added along the existing roadways that pass through the mangrove to improve the flow of water throughout the entire mangrove. This improves the storage capacity of the mangrove as well as provides transportation of nutrients throughout the mangrove.
- Three ponds were introduced within the mangrove in areas that required rehabilitation. These ponds add volume capacity to the storage capabilities of the mangrove as well and increase the square area of effective mangrove as follows:

Pond	***Increase in Storage Capacity (m^3)***	***Increase in Mangrove Area (m^2)***
Pond 1	7,066	12,602
Pond 2	7,978	16,933
Pond 3	1,149	3,413
Total	**16,193**	**32,948**

6.2 Summary of Hydraulic Performance of Proposed Drainage Plan

As per best storm water management practices, all drainage systems are design for a particular storm frequency, striking a balance between flood risks and project cost. All drainage systems will flood as it is uneconomical and against industry practice to design for the "worst case" scenario. As such and as summarized in the preceding section, the proposed drainage plan for this site was designed using a 1 in 25-year storm frequency and the site was set above the expected 1 in 50-year flood elevation. Table 6-1 below summarizes the hydraulic performance of the drainage system under various design storm frequencies.

Table 6-1: Summary of hydraulic Performance of the Proposed Drainage System

STORM FREQUENCY	ANNUAL EXCEEDANCE PROBABILITY	HYDRAULIC PERFORMANCE	
		Site Drainage	**Mangrove**
1 in 2 year	50%	Site drains will perform good allowing rainfall runoff to drain freely into the mangrove.	Max. water level is expected to be 0.375m MSL which is below design outfall points. This is approx. 75mm above natural outfall elevations of the mangrove
1 in 5 year	20%	Site drains will perform good allowing rainfall runoff to drain freely into the mangrove.	Max. water level is expected to be 0.532m MSL which is below design elevations of the outfall points
1 in 10 year	10%	Site drains will perform good allowing rainfall runoff to drain freely into the mangrove.	Max. water level is expected to be 0.607m MSL which is below the design elevations of the outfall points
1 in 25 year	4%	Site drains will perform good allowing rainfall runoff to drain freely into the mangrove.	Max. water level is expected to be 0.656m MSL which is below the design elevations of the outfall points
1 in 50 year	2%	Site drainage is expected to experience "flash flooding" as the internal drains were designed for a 1:25yr hydraulic capacity. The freeboard in each drain will absorb some of the excess flow. Since the outfall points are set higher than the projected water elevation in the mangrove for this storm frequency, the site water will be able to drain freely into the mangrove. Hence, the flood time within the site will be minimal.	Max. water level is expected to be 0.795m MSL which is below the design elevations of the outfall points
1 in 100 year	1%	Site is expected to flood as the rainfall intensities exceed that which the drains were designed for (1:25yr intensity). It is expected that there will be zero outflow from the site for some time when the projected water levels in the mangrove goes beyond the 0.8m MSL design outfall elevation. At these elevations the flap gates connected to the outfall pipes will close. Water from the site will then outfall through said outfalls once the flood elevations in the mangrove drops below 0.8m MSL	Max. water level is expected to be 0.876m MSL which is above the design elevations of the outfall points. This will result in the flap gates on the outfall pipes to be closed for some time, preventing flood waters from the mangrove to backflow onto the site.

7 References

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8 Appendices

8.1 Runoff C_N Values

US Department of Agriculture Soil Conservation Service-1986

Table 8.7.3 Runoff Curve Numbers (Average Watershed Condition, $I_a = 0.2S$)

Land use description		Average % impervious[b]	Curve numbers for hydrologic soil group A	B	C	D
Fully developed urban areas[a] (vegetation established)						
Lawns, open spaces, parks, golf courses, cemeteries, etc.						
Good condition; grass cover on 75% or more of the area			39	61	74	80
Fair condition; grass cover on 50% to 75% of the area			49	69	79	84
Poor condition; grass cover on 50% or less of the area			68	79	86	89
Paved parking lots, roofs, driveways, etc.			98	98	98	98
Streets and roads						
Paved with curbs and storm sewers			98	98	98	98
Gravel			76	85	89	91
Dirt			72	82	87	89
Paved with open ditches			83	89	92	93
Commercial and business areas		85	89	92	94	95
Industrial districts		72	81	88	91	93
Row houses, town houses, and residential with lot sizes 1/8 acre or less		65	77	85	90	92
Residential: average lot size						
1/4 acre		38	61	75	83	87
1/3 acre		30	57	72	81	86
1/2 acre		25	54	70	80	85
1 acre		20	51	68	79	84
2 acre		12	46	65	77	82
Developing urban areas[c] (no vegetation established)						
Newly graded area			77	86	91	94
Cover						
Land use	Treatment of practice	Hydrologic condition[d]				
Cultivated agricultural land						
Fallow	Straight row		77	86	91	94
	Conservation tillage	Poor	76	85	90	93
	Conservation tillage	Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90

Table 8.7.3 (Continued)

Cover: Land use	Cover: Treatment of practice	Hydrologic condition[a]	Curve numbers for hydrologic soil group: A	B	C	D
	Conservation tillage	Good	64	75	82	85
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and conservation	Poor	69	78	83	87
	tillage	Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
	Contoured and terraces	Good	62	71	78	81
	Contoured and terraces	Poor	65	73	79	81
	and conservation tillage	Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
	Conservation tillage	Good	60	72	80	84
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and conservation	Poor	62	73	81	84
	tillage	Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
	Contoured and terraces	Good	59	70	78	81
	Contoured and terraces	Poor	60	71	78	81
	and conservation tillage	Good	58	69	77	80
Close-seeded legumes or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
	Contoured and terraces	Good	51	67	76	80
Noncultivated agricultural land, pasture or range	No mechanical treatment	Poor	68	79	86	89
	No mechanical treatment	Fair	49	69	79	84
	No mechanical treatment	Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		—	30	58	71	78
Forested—grass or orchards—evergreen or deciduous		Poor	55	73	82	86
		Fair	44	65	76	82
		Good	32	58	72	79
Brush		Poor	48	67	77	83
		Good	20	48	65	73
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		—	59	74	82	86
Forest-range Herbaceous		Poor		79	86	92
		Fair		71	80	89
		Good		61	74	84

Table 8.7.3 (*Continued*)

Cover		Hydrologic condition[a]	Curve numbers for hydrologic soil group			
Land use	Treatment of practice		A	B	C	D
Oak–aspen		Poor		65	74	
		Fair		47	57	
		Good		30	41	
Juniper–grass		Poor		72	83	
		Fair		58	73	
		Good		41	61	
Sage–grass		Poor		67	80	
		Fair		50	63	
		Good		35	48	

[a]For land uses with impervious areas, curve numbers are computed assuming that 100% of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a *CN* of 98.

[b]Includes paved streets.

[c]Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area *CN* and Figure 8.7.1*a* or *b*, the composite *CN* can be computed for any degree of development.

[d]For conservation tillage in poor hydrologic condition, 5 percent to 20 percent of the surface is covered with residue (less than 750-lb/acre row crops or 300-lb/acre small grain).

For conservation tillage in good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 750-lb/acre row crops or 300-lb/acre small grain).

[e]Close-drilled or broadcast.

For noncultivated agricultural land:

Poor hydrologic condition has less than 25 percent ground cover density.

Fair hydrologic condition has between 25 percent and 50 percent ground cover density.

Good hydrologic condition has more than 50 percent ground cover density.

For forest–range:

Poor hydrologic condition has less than 30 percent ground cover density.

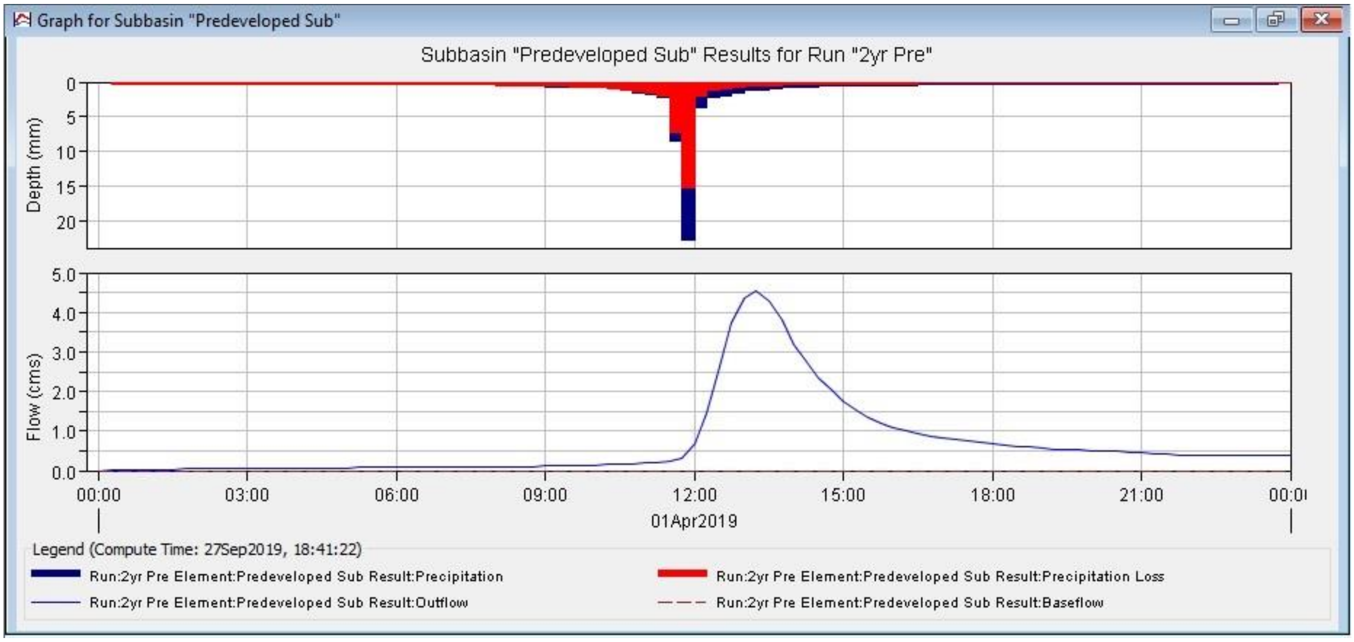
Fair hydrologic condition has between 30 percent and 70 percent ground cover density.

Good hydrologic condition has more than 70 percent ground cover density.

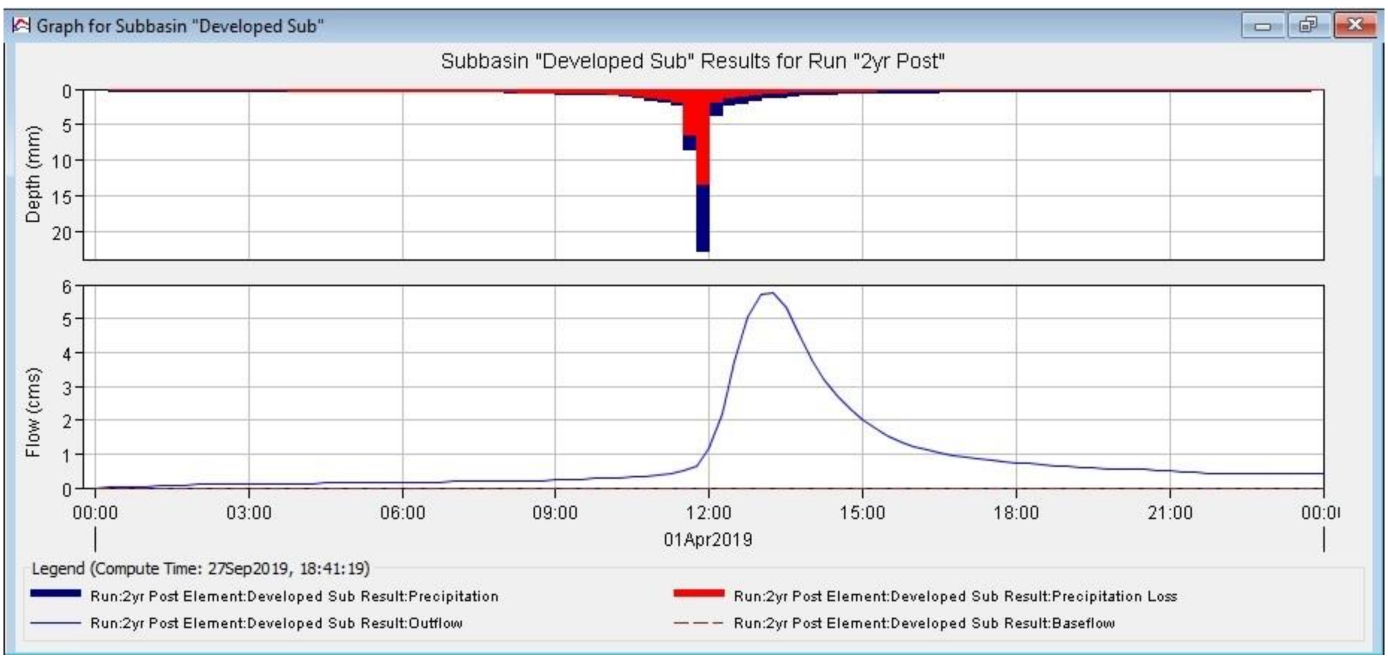
Source: U.S. Department of Agriculture Soil Conservation Service (1986).

8.2 Hydrograph Output from HEC-RMS

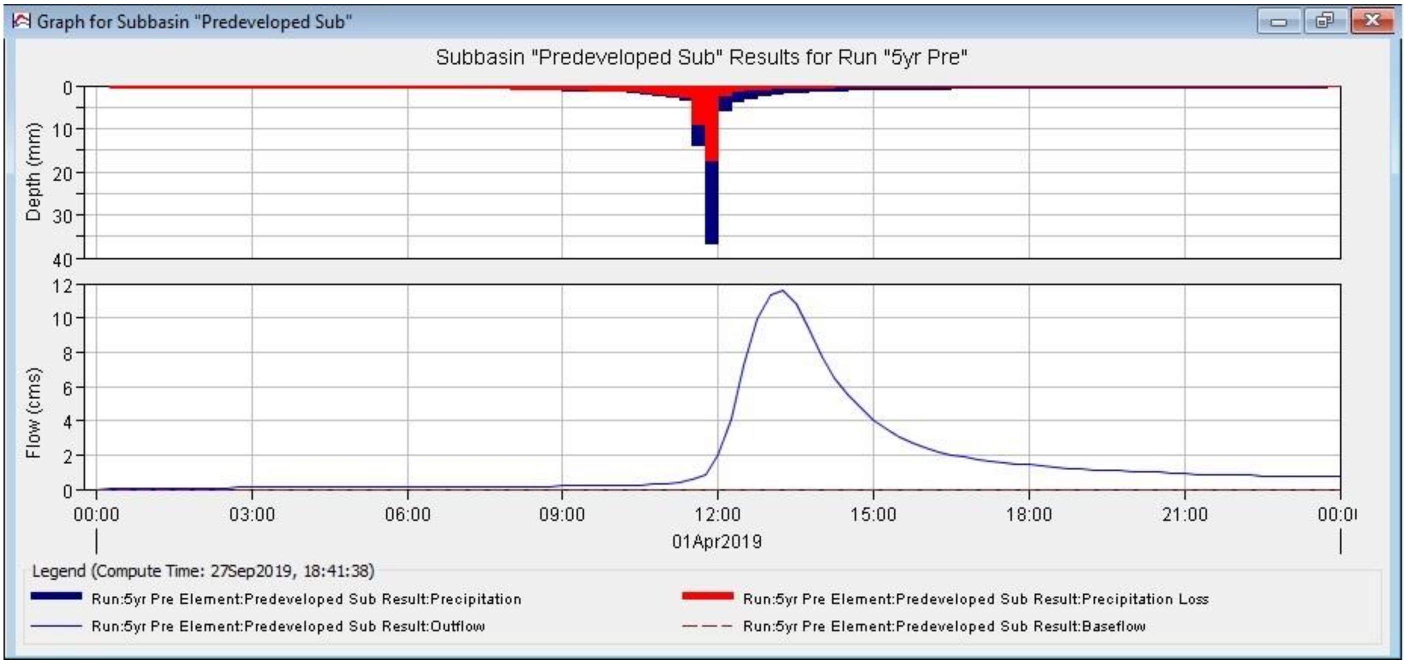
2year – Pre-developed Hydrograph



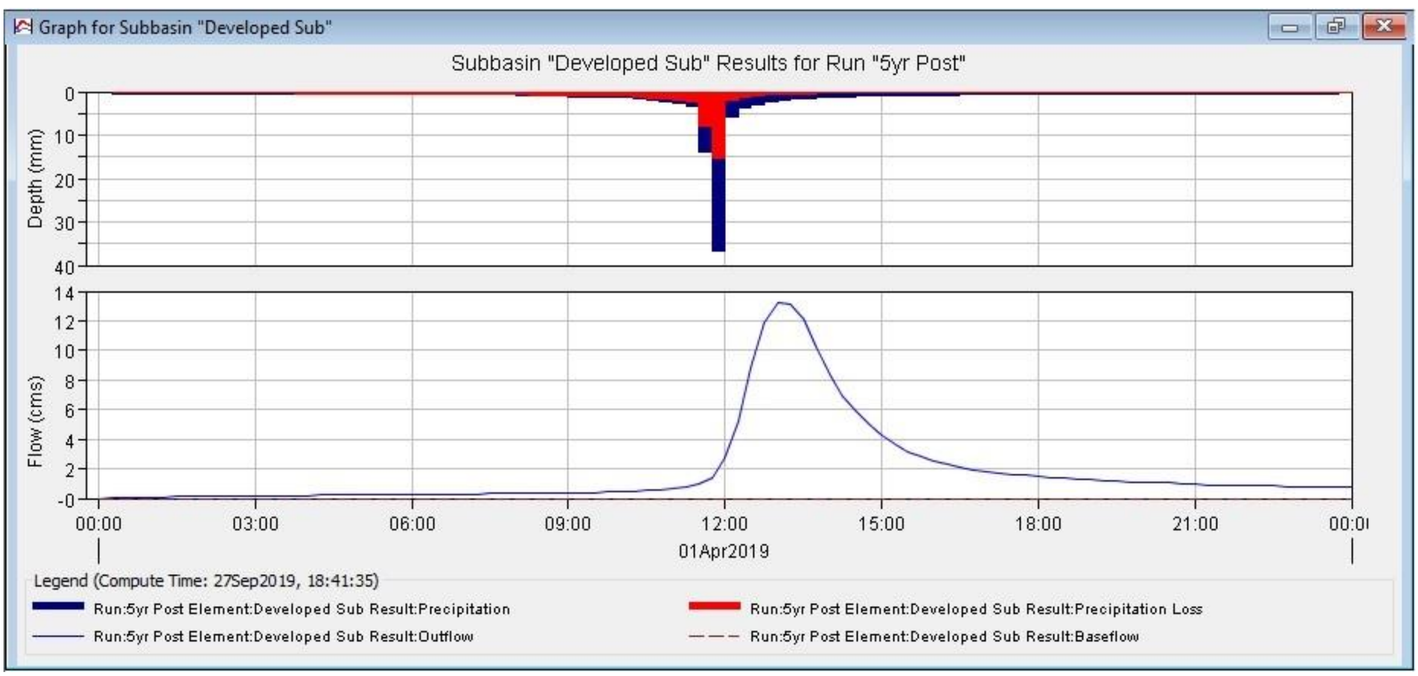
2year – Post-developed Hydrograph



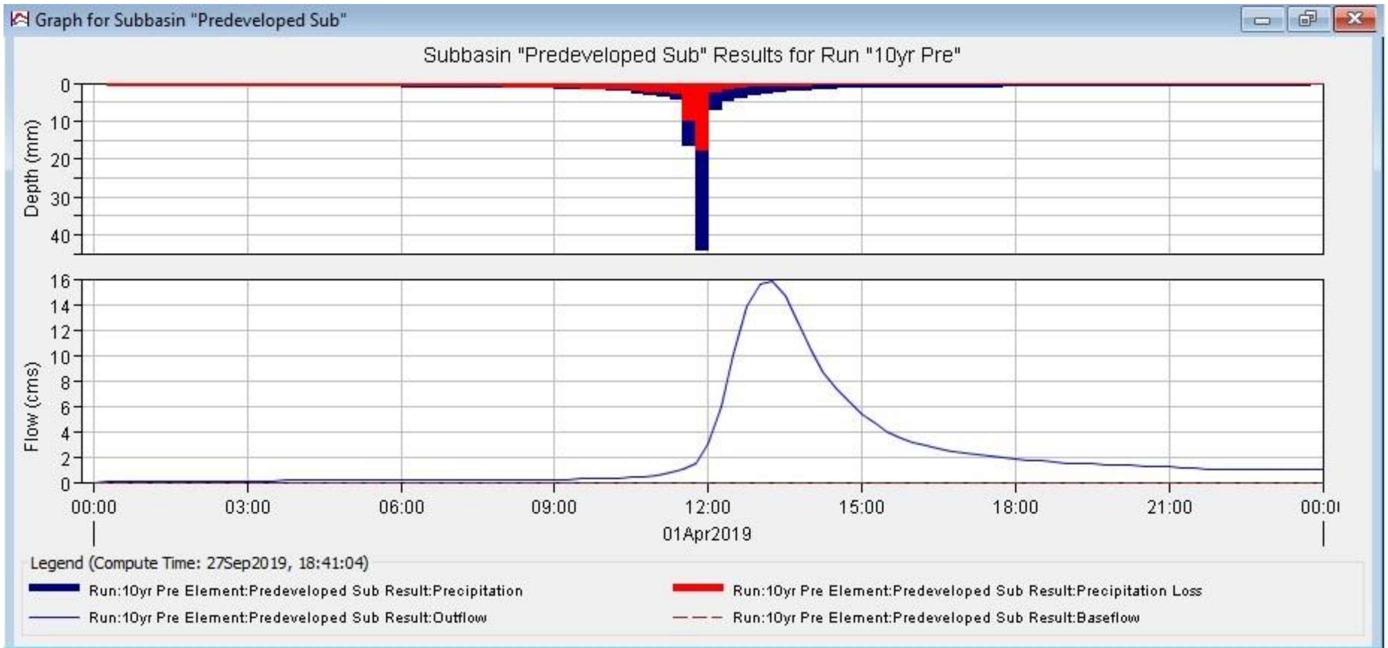
5year – Pre-developed Hydrograph



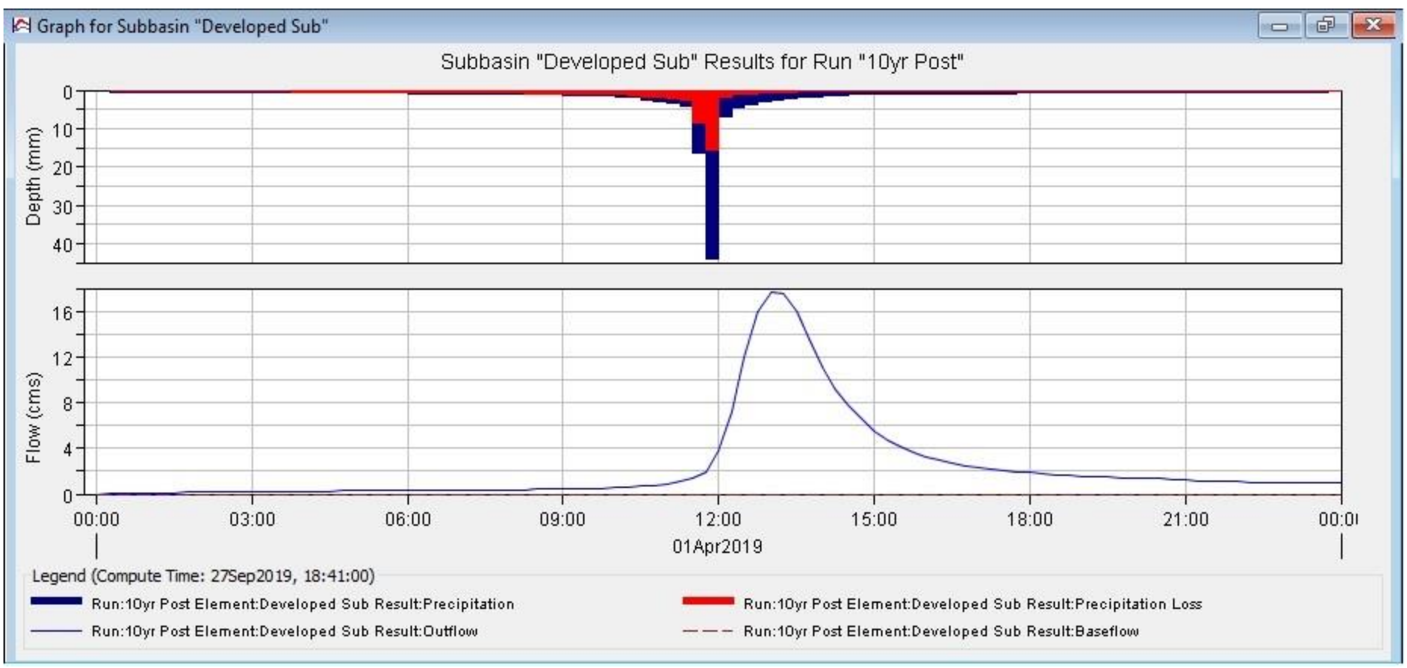
5year – Post-developed Hydrograph



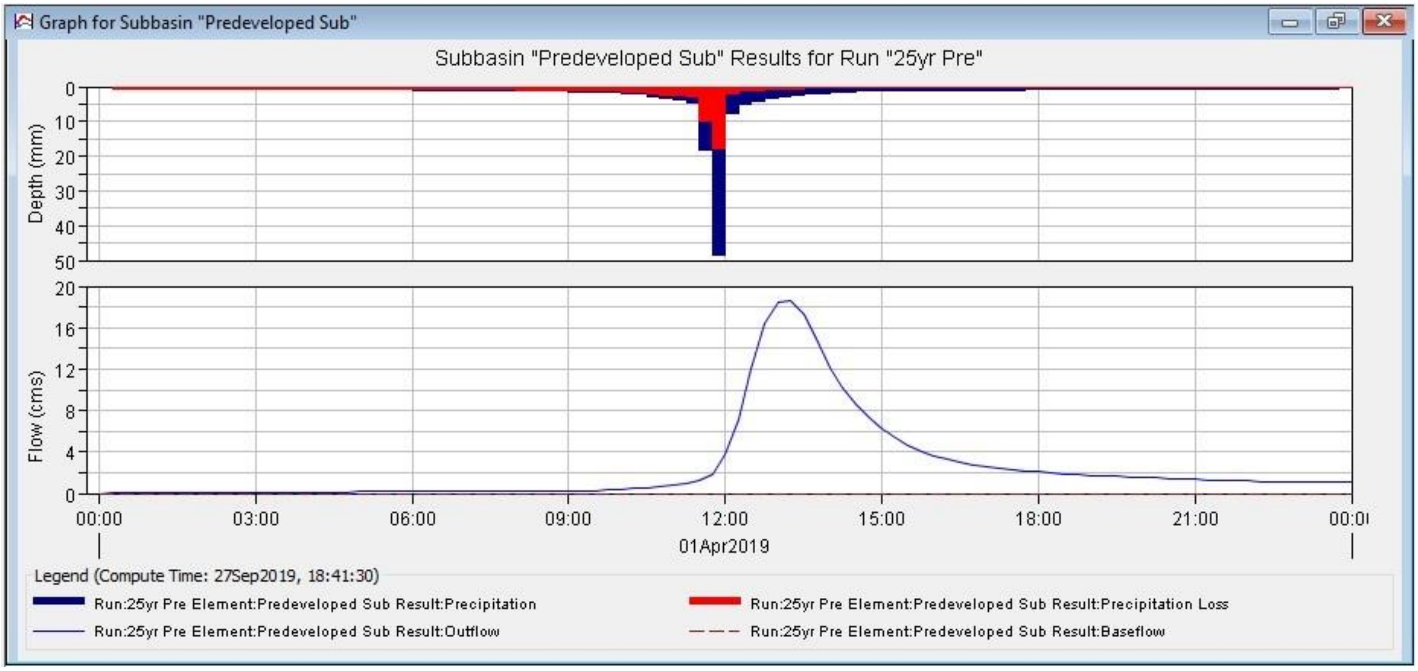
10year – Pre-developed Hydrograph



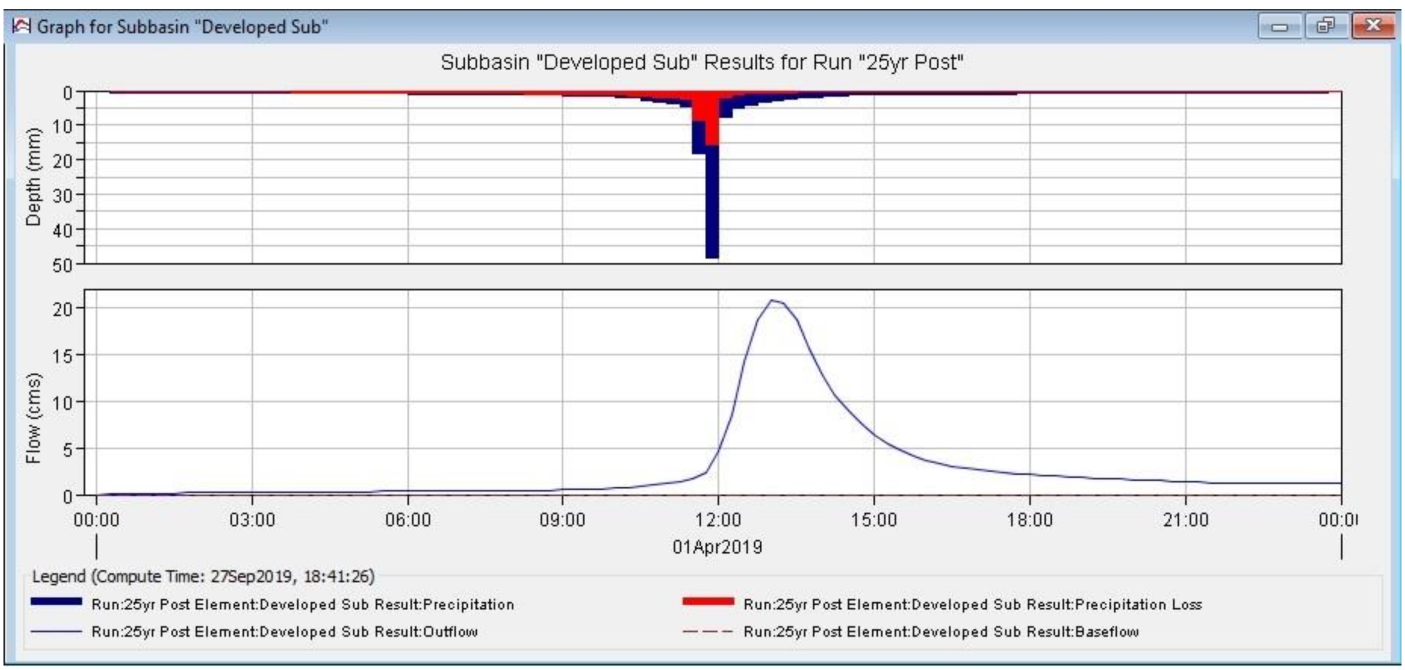
10year – Post-developed Hydrograph



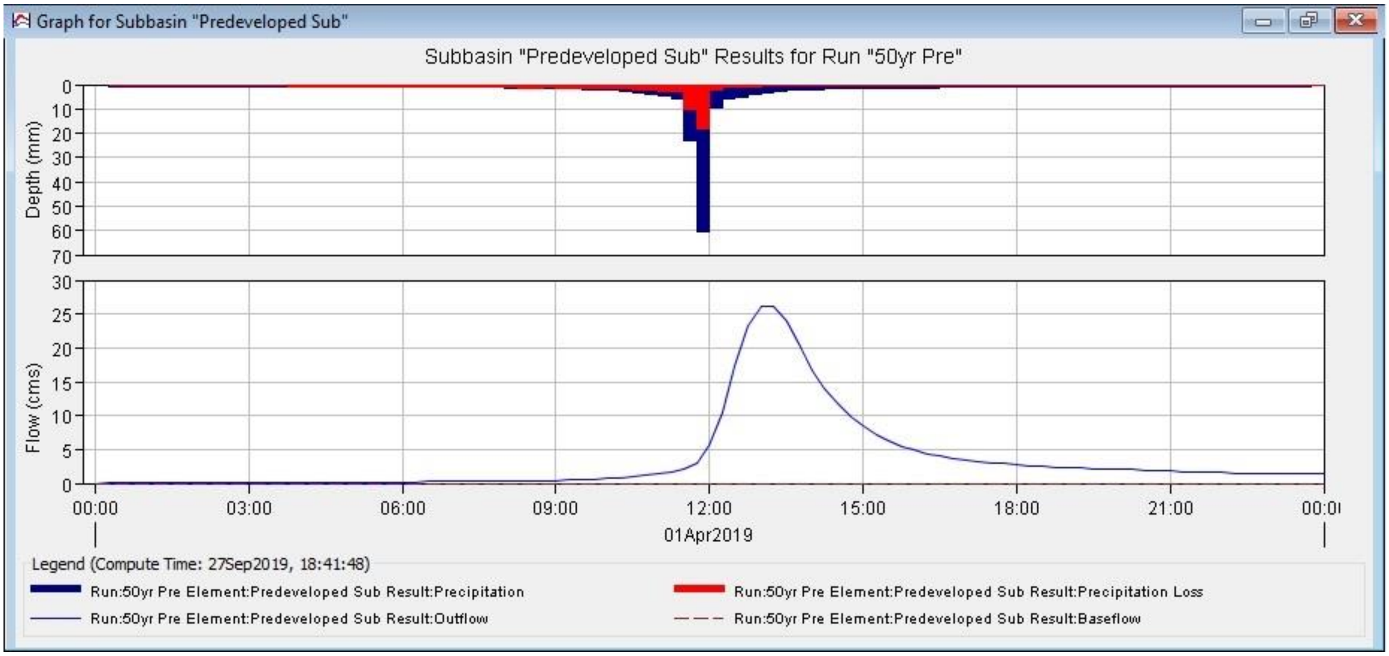
25year – Pre-developed Hydrograph



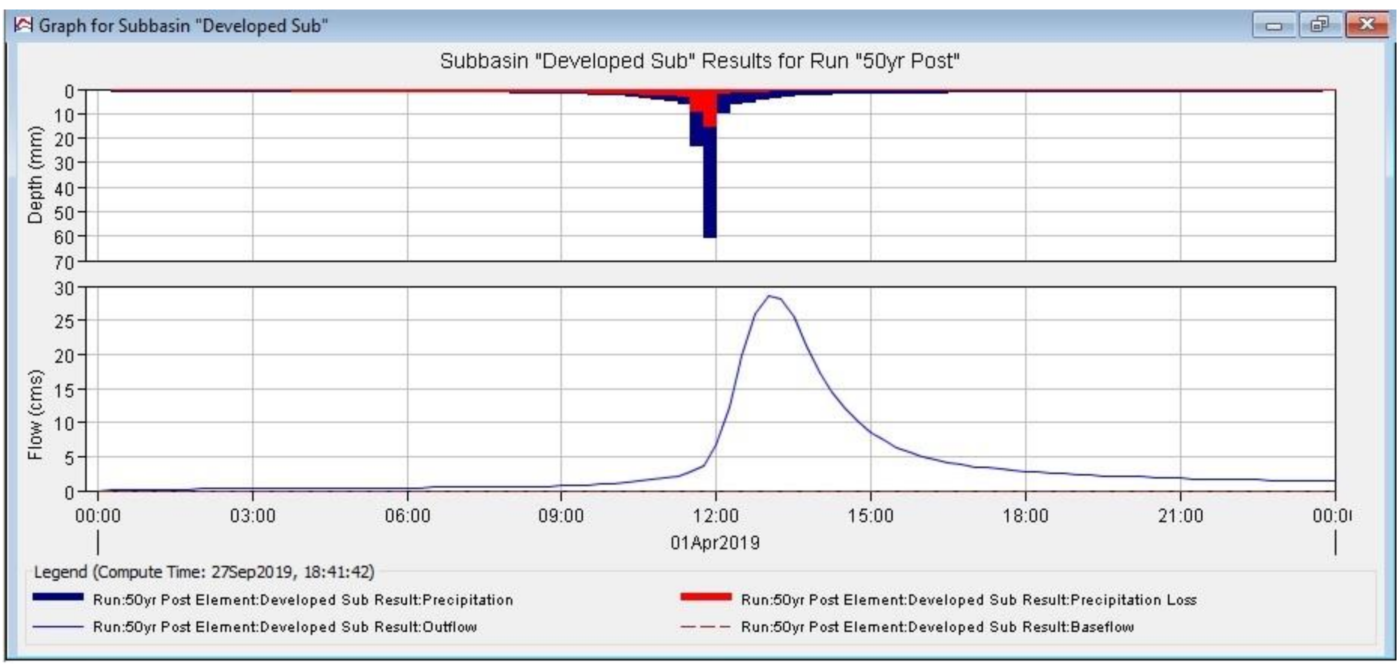
25year – Post-developed Hydrograph



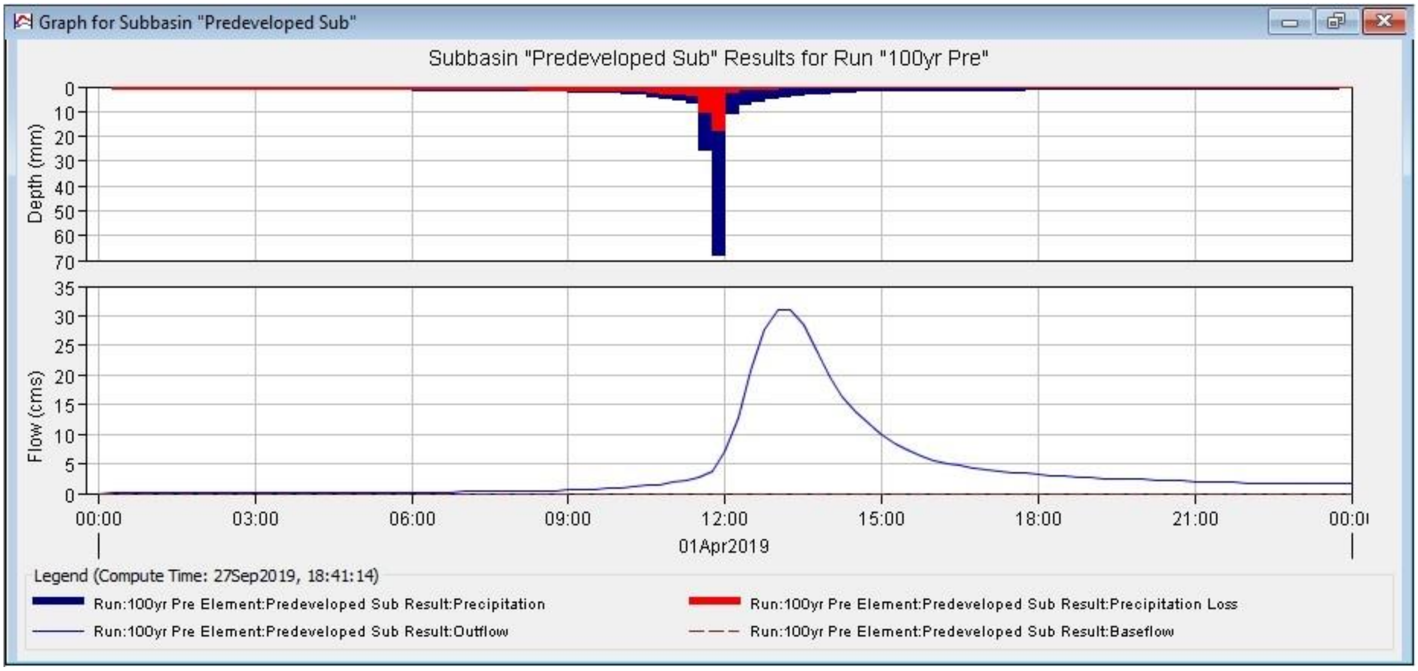
50year – Pre-developed Hydrograph



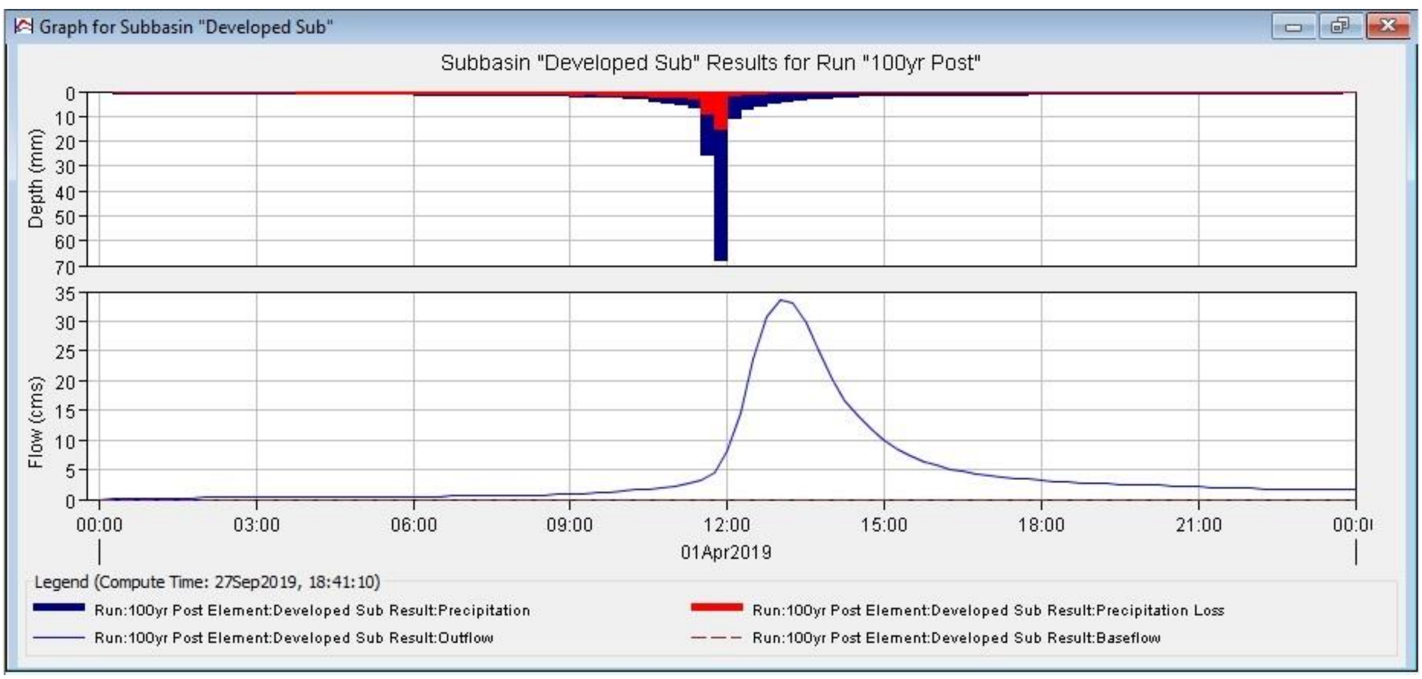
50year – Post-developed Hydrograph



100year – Pre-developed Hydrograph



100year – Post-developed Hydrograph

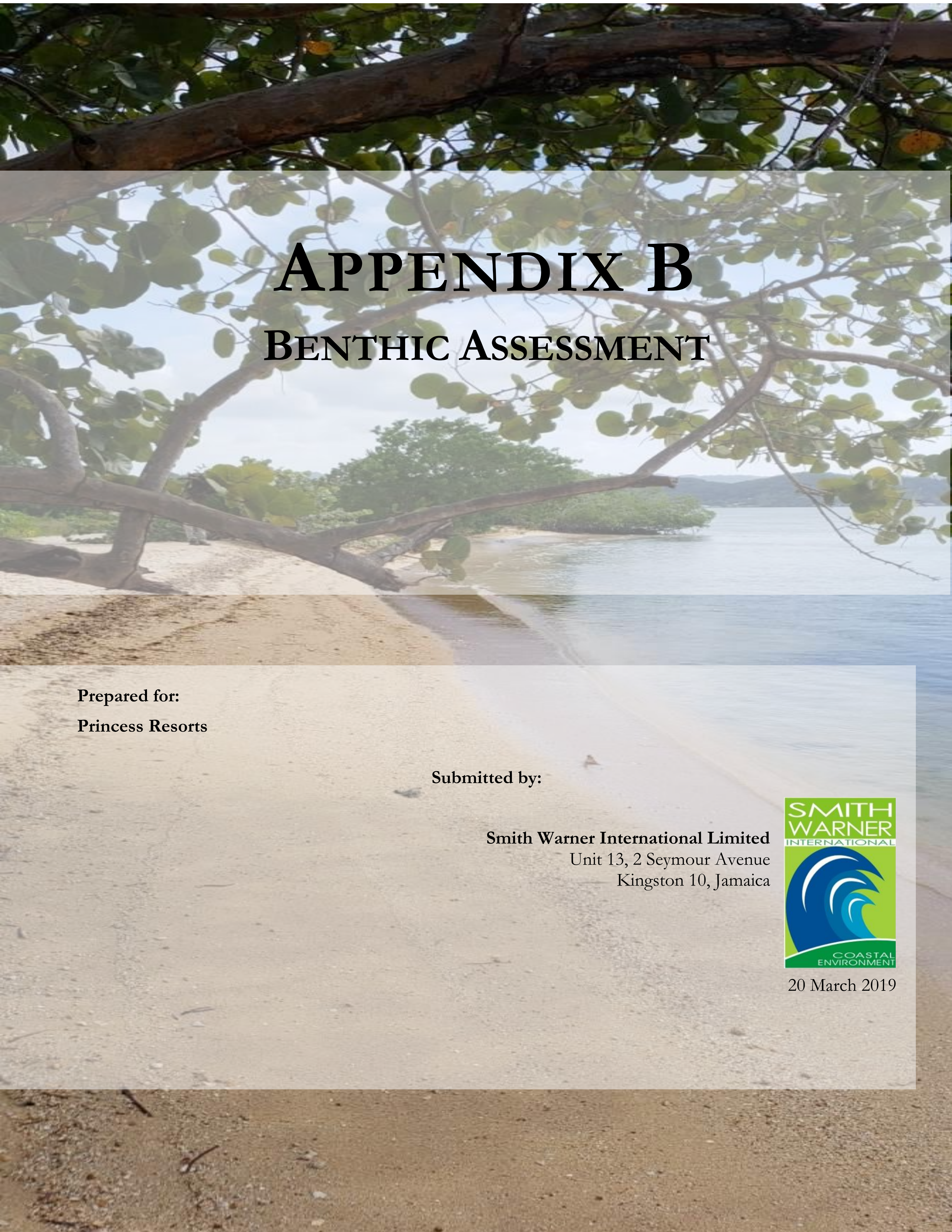


8.3 Percentage Change in Rainfall by Season Over Time

Table 49-52: Projected percentage changes in rainfall by season and for annual average (°C) for the 2020's, 2030's, 2050's and 2080's relative to the 1961-1990 baseline. Data presented for the mean value of a six-member ensemble. Range shown is over all the grid boxes in the zone (see Table 2). *Source: PRECIS RCM perturbed physics ensemble run for A1B scenario.*

TABLE 49: WEST (ZONE 3)				
	2020's	2030's	2050's	2080's
NDJ	3.27 – 16.13	2.15 – 26.56	1.63 – 29.71	7.10 – 35.10
FMA	1.12 – 28.36	-5.89 – 28.23	16.12 – 39.86	-1.09 – 36.23
MJJ	4.21 – 17.09	-11.84 – 12.77	-8.54 – 17.59	-29.46 – 4.98
ASO	-12.90 – 7.01	-25.13 – -3.17	-20.92 – 4.13	-26.92 – -0.29
ANNUAL	2.44 – 4.50	-10.11 – 34.37	-5.70 – 9.95	-13.23 – 6.09

Figure 33. Taylor et. al.. 2017



APPENDIX B

BENTHIC ASSESSMENT

Prepared for:

Princess Resorts

Submitted by:

Smith Warner International Limited
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Kingston 10, Jamaica

20 March 2019

Contents

1 Introduction

The main aim of this assessment is a determination of the potential environmental impacts of the proposed construction works on the surrounding marine environment. The work also aims to provide an assessment of the long-term impacts of the proposed development on the surrounding marine environment.

This environmental impact study was designed with the following objectives:

- to determine the existing conditions at the site;
- to provide assessments of the biological and physical environments;
- to assess the impact of works on the surrounding marine environment; and
- to propose mitigation measures to minimise impact from the proposed work.

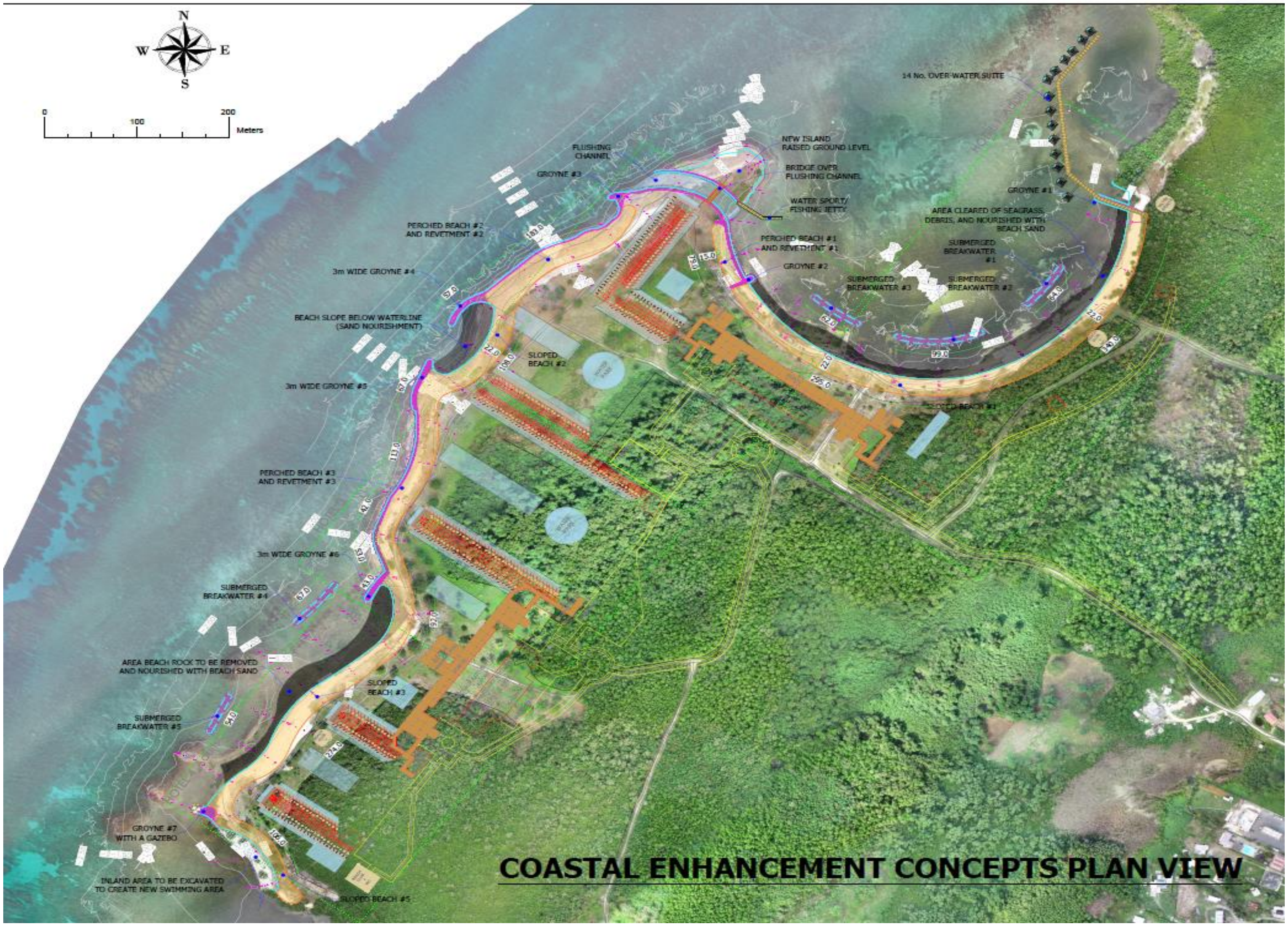

Figure 1.1 Site plan of proposed coastal enhancement works at Princess Resort, Hanover

2 Methodology

A review was conducted using available satellite and aerial imagery of the project site, the project brief, and the proposed construction positioning. An initial benthic survey of the site for the proposed development was conducted on 5 March 2019. The site was accessed directly from shore and the survey area demarcated using a hand-held Global Positioning System (GPS) instrument.

2.1 Benthic Survey

A modified Reef Check® Method was used to assess the substrate in the survey area. Modifications to this method were made and these included the following:

- Only the Substrate transect was executed. No Fish or Invertebrate transects were conducted.
- The category Sponge (SP) was replaced by Seagrass (SG). Seagrass is usually recorded under other (OT) but was disaggregated due to sizeable presence at this site.
- **Error! Reference source not found.** shows the transect lines used within survey area Section A . Substrate type was identified and recorded at 0.5m intervals along each transect line. Substrate types were categorized as Hard Coral (HC), Soft Coral (SC), Recently Killed Coral (RKC), Macroalgae (MA – including Nutrient Indicating Algae), Seagrass (SG), Rock (RC), Rubble (RB), Sand (SD), Silt/clay (SI) and Other (OT). Definitions for each substrate category are outlined in the appendix. For further details please see www.reefcheck.org.

Figure 2.1 Survey area and transects

2.2 Fish Survey

A modified Visual Fish Census of **A**tlantic and **G**ulf **R**apid **R**eef **A**ssessment (AGRRA) Protocol was employed to capture fish data (presence/absence and frequency). All fish observed were identified and given a frequency rating (based on occurrence) of Single (S = single individual), Few (F = 2-10 individuals), Many (M = 11-100 individuals), or Abundant (A = >100 individuals). For further details on the AGRRA Methodology please see www.agrra.org/method/methodhome.html.

3 Observations & Results

Ten transect line surveys were conducted across the site; the results along with general observations are presented below.

3.1 Section A- The Eastern Bay

In general, the substrate in this bay area (Figure 3.1) is dominated by seagrass (*Thalassia testudinium)* cover. Significant stands of healthy seagrass are present. There is a section in the southwest corner of the bay in which silt, dark organic matter and nutrient indicator algae are present. This may suggest that there are some circulation issues. Four transects were conducted in this section; the results are presented below.

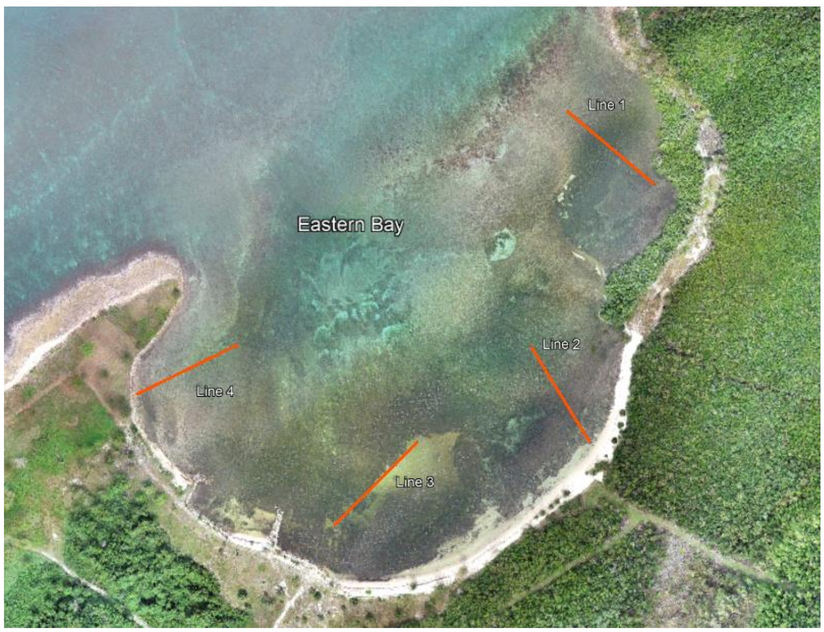

Figure 3.1 The Eastern Bay Section

3.1.1 Line 1

The substrate along Line 1(Figure 3.2) was characterized by a mean percent cover of 98% seagrass. This area is adjacent to mangrove and numerous juvenile fish were observed.

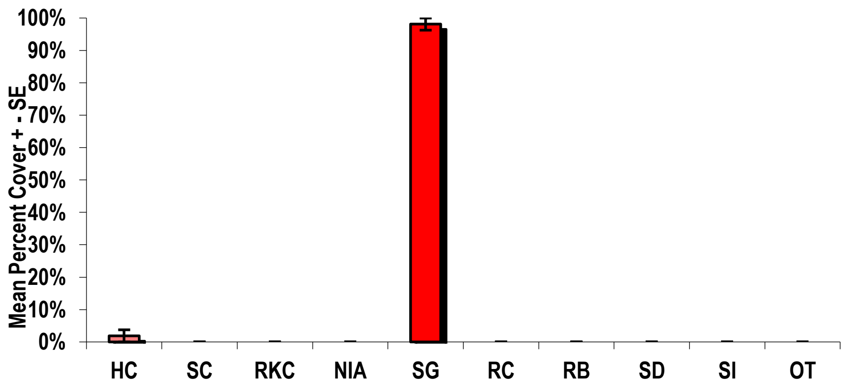

Figure 3.2 Substrate at Line 1

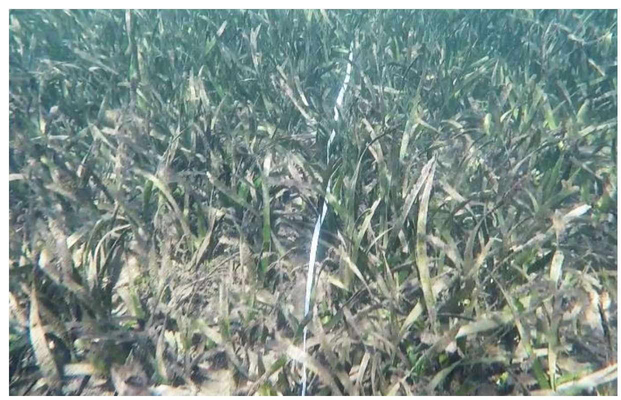

Figure 3.3 Example of substrate observed along Line 1

3.1.2 Line 2

The substrate along Line 2 (Figure 3.4), was dominated by seagrass with 88% mean coverage. Sand coverage followed with 11% mean coverage.

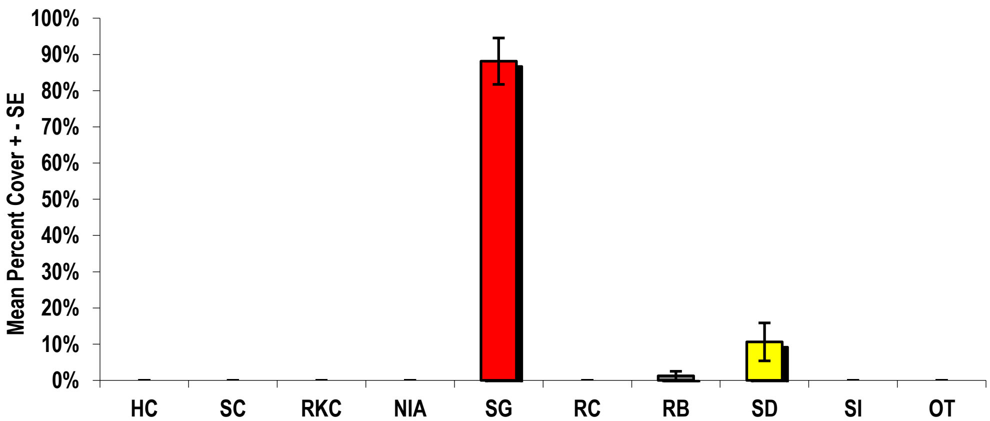

Figure 3.4 Substrate at Line 2

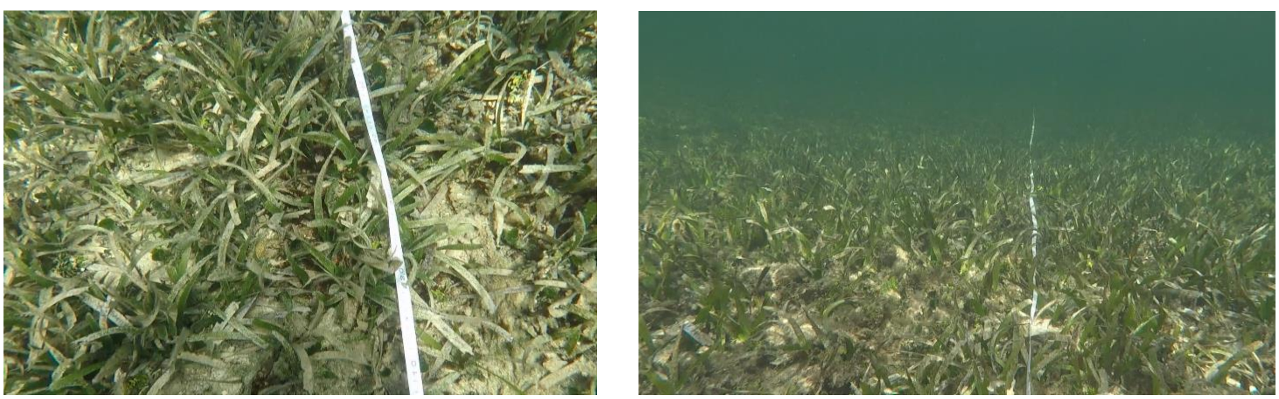

Figure 3.5 Example of substrate observed along Line 2

3.1.3 Line 3

The substrate along Line 3 (Figure 3.6) was characterized by sand coverage of 38%, followed by seagrass with a mean percent cover of 35%. The area also had a notable siltation close to the shore with silt at 23% along this line. Nutrient Indicating algae was also observed in this section, which suggests some issues with water quality/circulation.

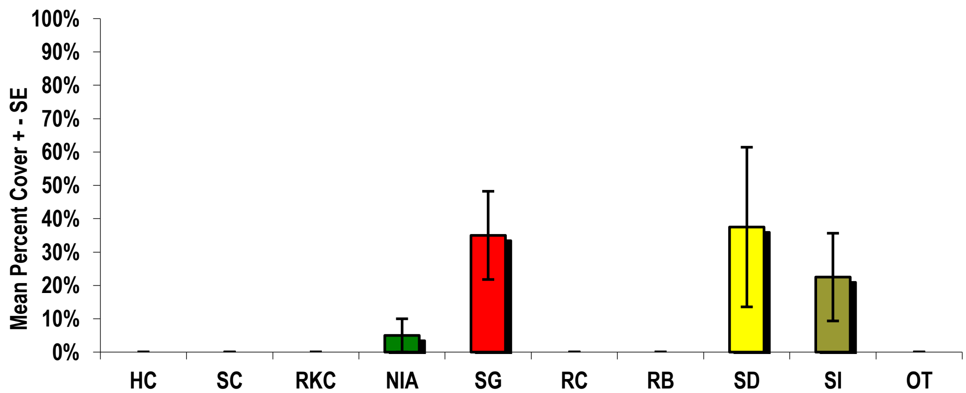

Figure 3.6 Substrate at Line 3

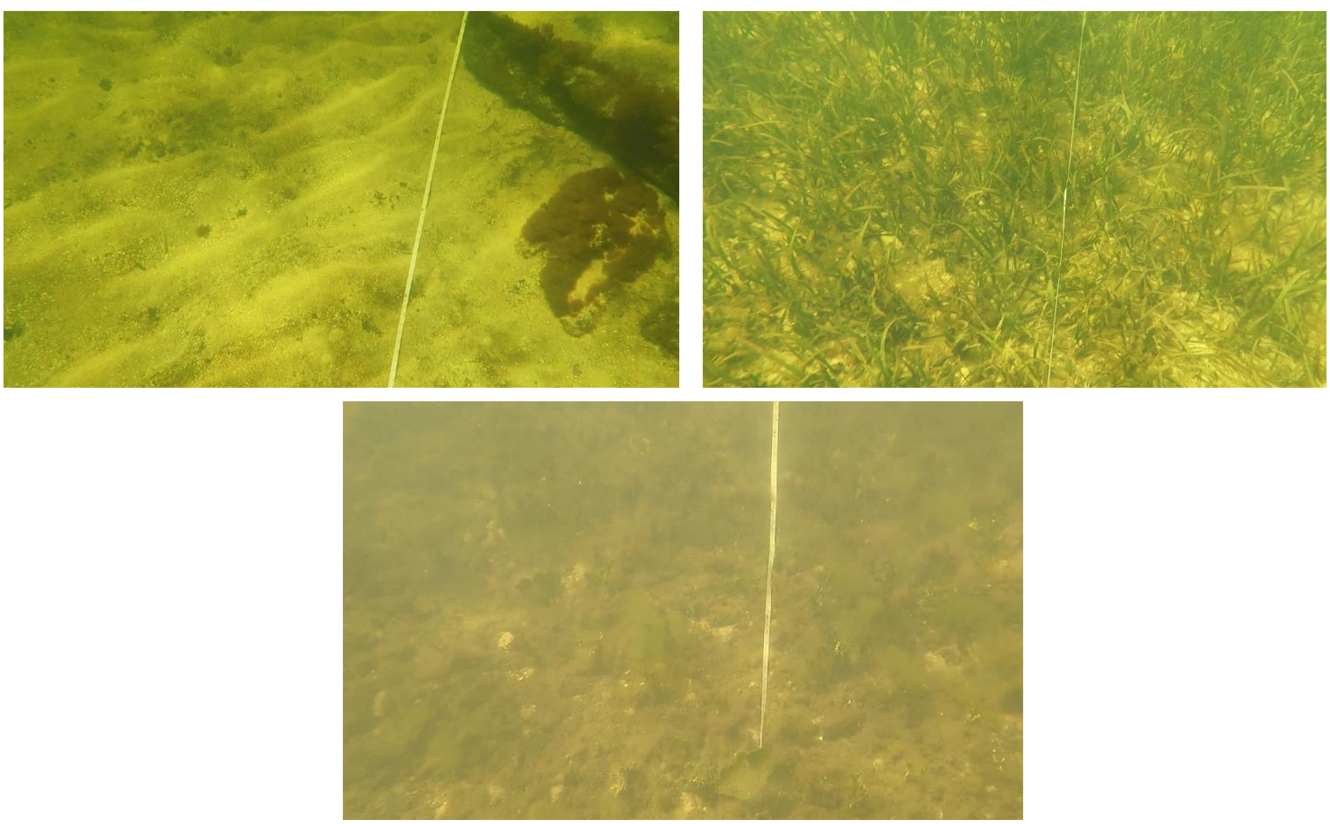

Figure 3.7 Example of substrate observed along Line 3

3.1.4 Line 4

The substrate along Line 4 (Figure 3.8) was characterised by seagrass with 71% coverage, followed by sand and hard pavement/rock at 14% and 9% coverage respectively. There was also some hard coral coverage at 5%.

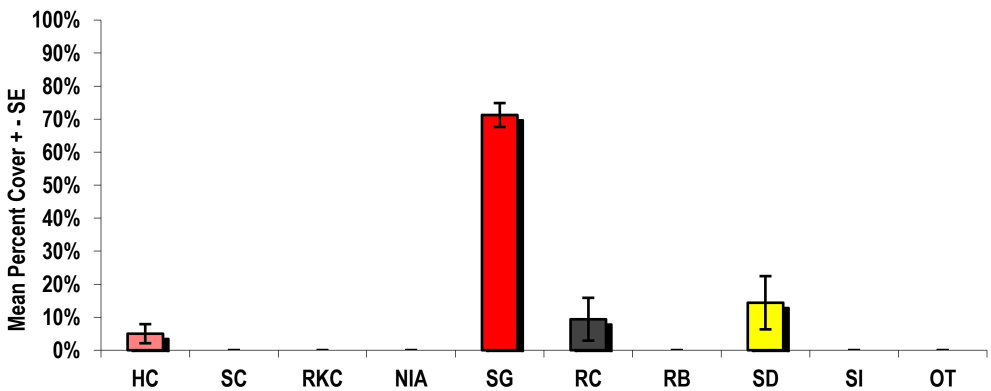

Figure 3.8 Substrate at Line 4

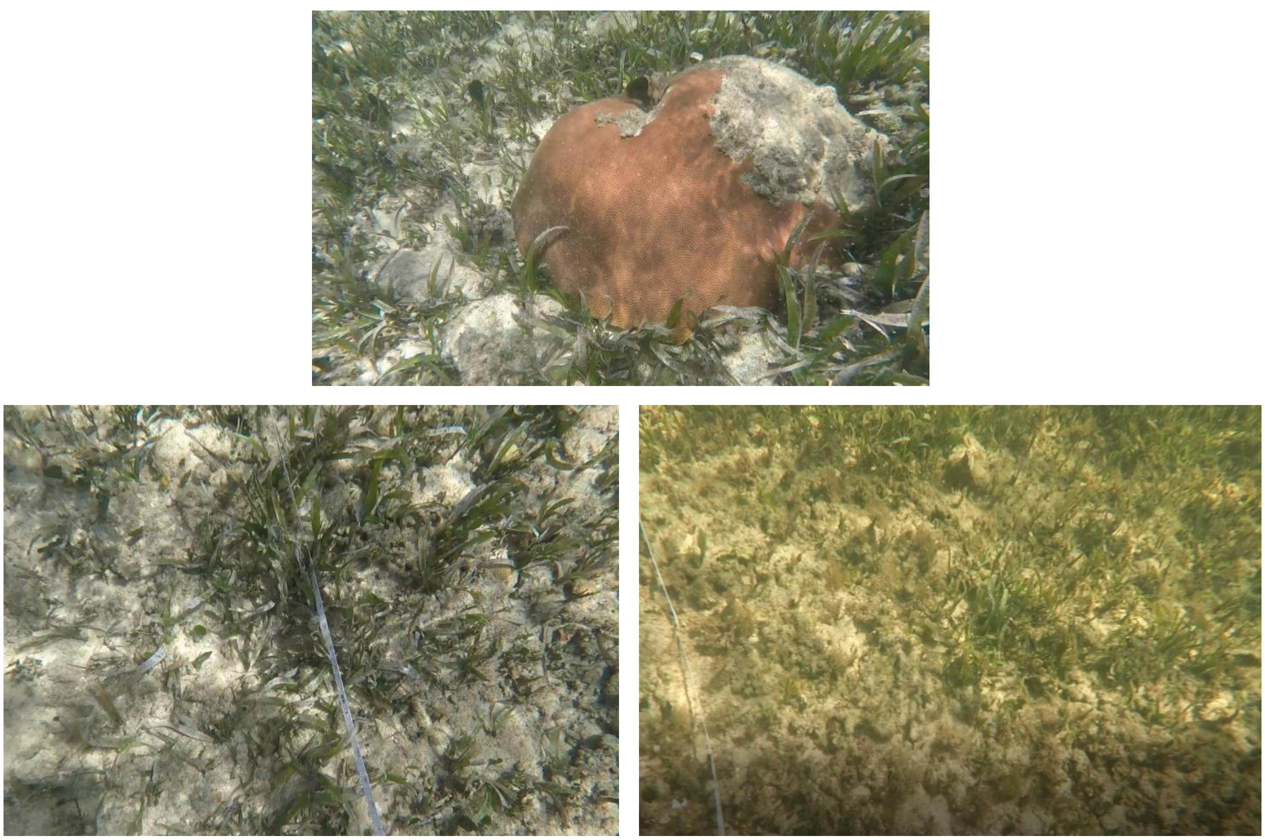

Figure 3.9 Example of substrate observed along Line 4

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3.2 Section B - The Ironshore

The area encompasses the central section of the site (Figure 3.10). Along much of this shoreline is a limestone ironshore/elevated limestone platform. In the nearshore, numerous hard corals, gorgonians and hydrozoans were observed. There is a thriving reef community present. Beyond the edge of the ironshore the substrate is mainly comprised of hard pavement populated by macroalgae and hard corals (many >20cm in size).

Figure 3.10 The Ironshore section

Figure 3.11 Example of reef structure just offshore

3.2.1 Line 5

The substrate alone line 5 (Figure 3.12) was characterised by rock/hard pavement (80%) and the hard corals (20%) populating the seafloor.

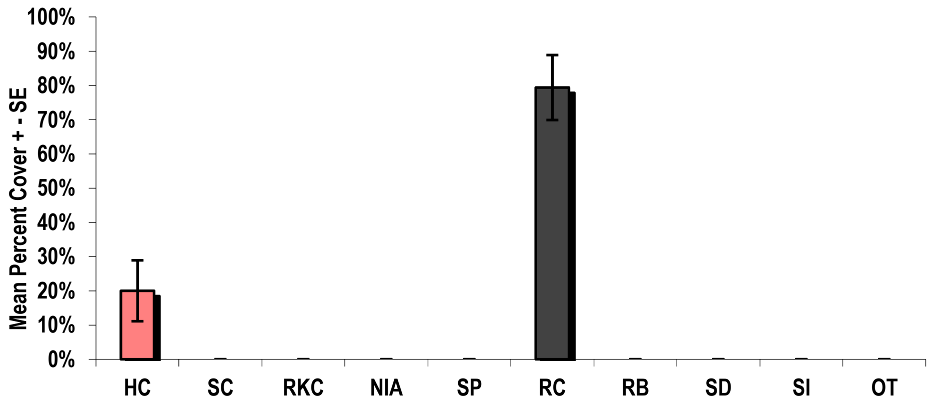


Figure 3.12 Substrate at Line 5

Figure 3.13 Example of substrate at Line 5

3.2.2 Line 6

Similar to Line 5, the substrate alone line 6 (Figure 3.14) was characterised by rock/hard pavement (79%) and the hard corals (21%) populating the seafloor.

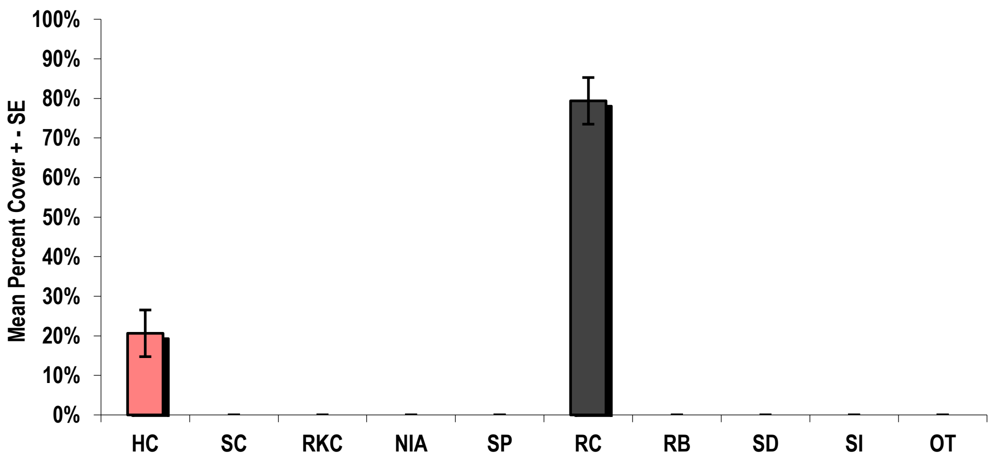


Figure 3.14 Substrate at Line 6

Figure 3.15 Example of substrate at Line 6

3.2.3 Line 7

Similar to Line 5, the substrate alone line 7 (Figure 3.17) was characterised by rock/hard pavement (80%) and the hard corals (20%) populating the seafloor.

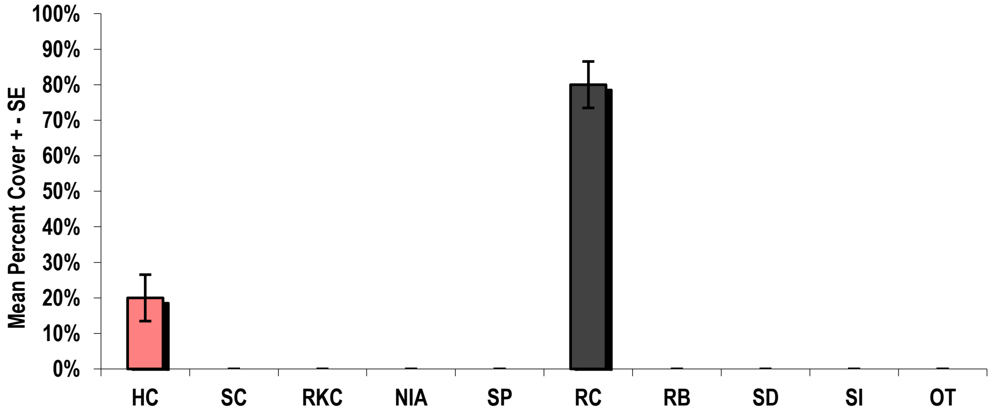


Figure 3.16 Substrate along Line 7

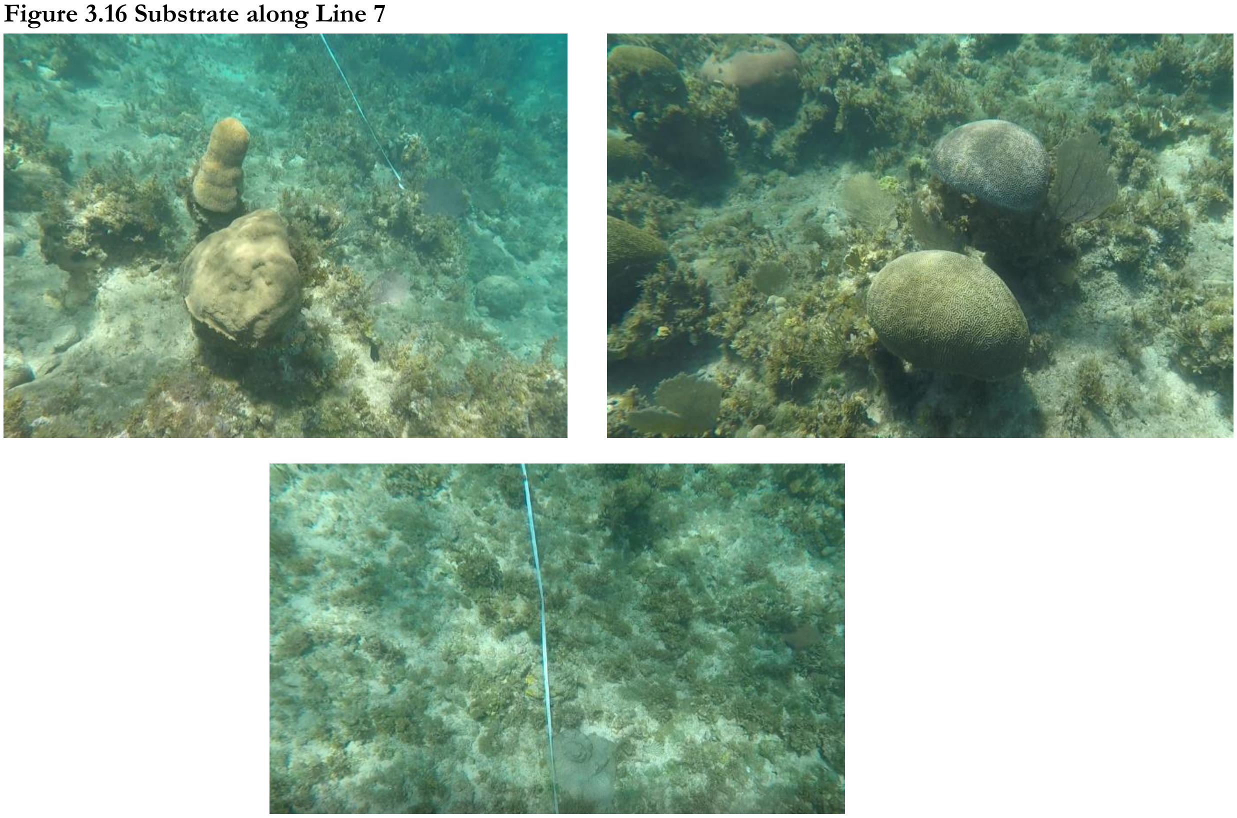


Figure 3.17 Example of substrate at Line 7

3.3 Western Section

Further west along the property, the shoreline composition changes (Figure 3.18). There is a narrow beach along the shoreline in this area. The nearshore is very shallow and the substrate is dominated by limestone pavement, seagrass and scattered corals.

Figure 3.18 Western Section

3.3.1 Line 8

The substrate at Line 8 (Figure 3.19) is characterised by mainly limestone pavement (38%) with seagrass (29%). Numerous corals (including small branching corals) are scattered across the seafloor.

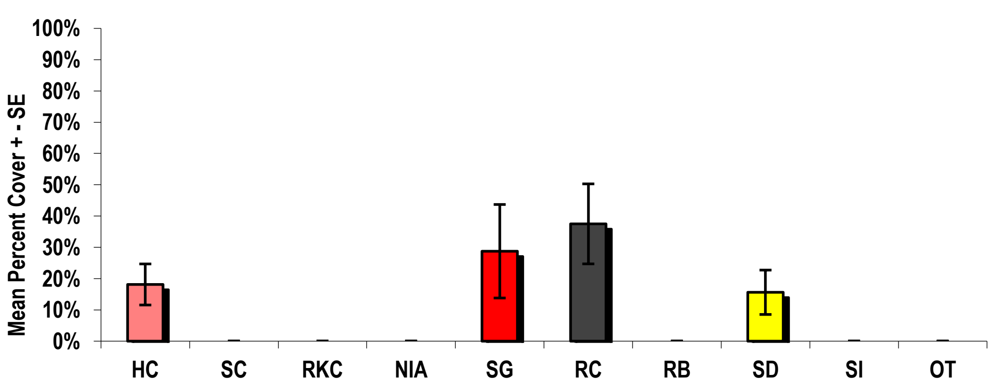


Figure 3.19 Substrate along Line 8

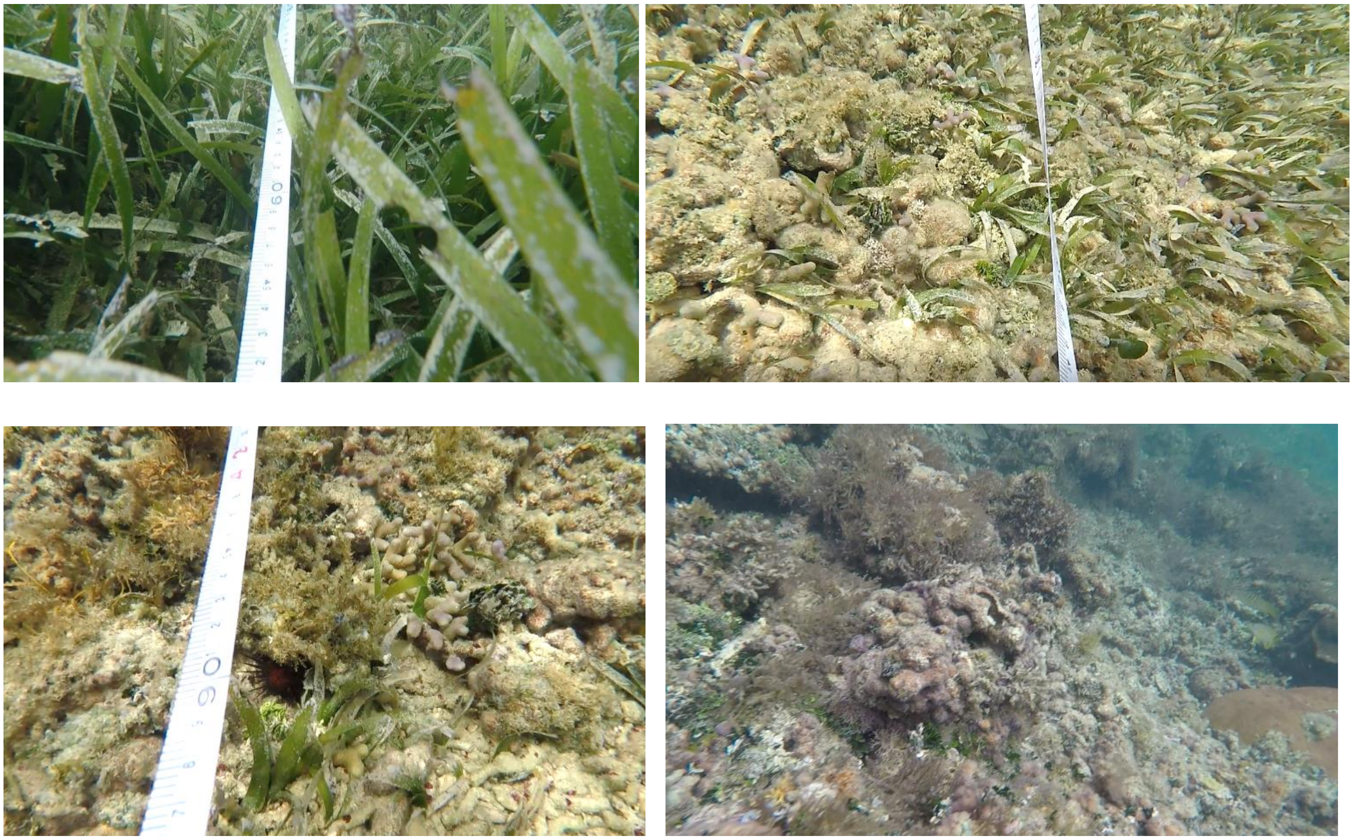


Figure 3.20 Example of substrate along Line 8

3.3.2 Line 9

The nearshore along Line 9 was very shallow (<0.5m). The substrate was characterised by seagrass, sand, pavement and macroalgae such as *halimeda (*Figure 3.21*)*.

Figure 3.21 Example of substrate observed along Line 9

3.3.3 Line 10

The substrate along Line 10, was dominated by almost 100% seagrass (Figure 3.22) with occasional coral colonies.

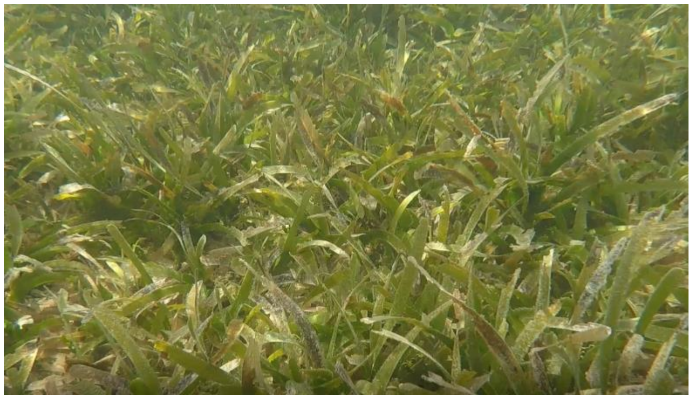


Figure 3.22 Example of substrate along Line 10

3.4 Fish – Rove Survey

The fish population observed throughout the entire area was relatively low both in total number of fish and number of fish per species. The reef fish observed were typical and small in size (5-10cm size class). Twenty one species of fish were observed. Fish species and other invertebrates observed are categorized and listed in the appendix. These are indicative lists and not exhaustive due to the nature (rapid review) of the survey.

4 Discussion and Recommendations

The composition of the substrate along this site varied along the starch of coastline. The eastern bay was dominated by seagrass cover. Along the central section of the site, the substrate was characterised by limestone rock/pavement and corals. The western section of the site was characterised by a mix of limestone pavement and seagrass. Within each section, the typical marine flora and fauna associated with each type of substrate and wave energy were observed.

The broader ecosystem represents relatively well-developed and ecologically significant marine resources (reefs and seagrass meadows). Seagrass meadows are essential coastal ecosystems that provide many ecosystem services such as improved water quality and light availability, increases in biodiversity and habitat, and sediment stabilization. Seagrass beds are also highly productive habitats that provide important ecosystem services in the coastal zone, including carbon and nutrient sequestration therefore acting as a carbon sink.

Impacts on marine resources could include excessive sedimentation during the nourishment/ construction works and settling of spoil, physical damage from heavy equipment on site, and loss/disruption of habitat. Corals, seagrasses and other valued ecosystem components are present within the footprint of the proposed works, and therefore it is important to mitigate potential impacts.

The positioning of transect Line 1 is aligned with the proposed footprint of a groyne/jetty structure and overwater suites. The footprint of this structure is dominated by seagrass. Few hard corals were observed within the footprint. The presence of juvenile fish indicates that the lagoon functions as a nursery. Disruption of this thriving community (through the removal of seagrass for example), could have negative ecological effects

The positioning of transect Lines 2 & 3 are aligned with the proposed footprint of two submerged breakwater structures. The footprints of these structures are dominated by seagrass. Few hard corals were observed within the footprint. There are also plans that the seafloor (seagrass, debris etc) along the shoreline of this area will be excavated and the shoreline nourished with sand. There is potential that seagrass could be smothered/disturbed/ destroyed by these works.

In the western section of the bay (which includes Line 4), the substrate is dominated by seagrass with some pavement and occasional coral colonies. There are plans for a groyne, perched beach and revetment, which will require some alteration of the shoreline and seafloor there. A jetty will also be installed in this area.

Along the central section of the site, which encompasses Lines 5, 6 and 7, there are plans for several groynes. The sensitive resources that this structure would potentially impact directly would include corals. Many thriving corals were observed along the entire stretch of ironshore and within the footprint of the proposed structures. Coral > 5cm would need to be relocated to a suitable site.

At the western section of the site, there are plans for two offshore breakwaters and the excavation of hard pavement at the shoreline to create a new beach. In the nearshore the substrate is characterised by hard pavement, seagrass and numerous corals. There are some areas in which the seagrass growth pattern (in tufts in spaces in the pavement) is such that relocation would be unfeasible. At the westernmost section of the site, some beach improvement works are planned. While these works will mainly be confined landward, there is potential that seagrass could be smothered/disturbed by this exercise.

5 Impacts and Mitigation

There are a number of potential negative impacts related to the excavation and sand nourishment for the proposed beach development. Where the effect of an impact is negative, consideration should be given to implementing mitigation measures. It is important to carefully design a development including potential mitigation measures so that potential negative impacts are minimized as much as possible, thereby reducing any damage to the environment. 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.

Smothering: The areas of sea floor to be excavated/nourished, the groynes/jetties and the breakwaters are all to be constructed using land-based heavy machinery. The benthic resources in the footprint of the coastal structures, the beach area and the construction pads will be impacted negatively by the physical disturbance.

> *To mitigate the effects, the benthic resources within the footprint of the structure should be relocated prior to construction. For the marine life outside of the footprint, a turbidity barrier should be used during construction to prevent fine material from spreading.*

Turbidity: The deployment of boulders for the breakwaters and the groyne, 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.

Turbidity barriers will be utilised to lessen the spread of fines. A turbidity meter will be used to measure the turbidity outside of the construction area to ensure that the turbidity readings are within the acceptable range as specified in the licenses.

Loss of habitat/ biodiversity: A direct impact from the works proposed at the site is the loss of the animals and plants living on and in the seagrass, limestone pavement and sand as well as loss of ecological function when the pavement/sediment/seagrass/corals are removed/smothered. The loss of these animals and plants will include a related disruption of the trophic web, which has as its base the organisms found in and on the sand, seagrass and reef organisms.

Where possible, sensitive resources (e.g. seagrass and corals) should be relocated to appropriate sites. For those resources that cannot be suitably relocated, an appropriate compensation mechanism should be devised.

Observations recorded in the field demonstrate how, within just a few years from implementation, breakwaters can develop in fish abundance, richness, and structure aging characteristics that are comparable to that of natural coral reefs. Fish and invertebrates of all demographic stages use these coastal defense structures as habitat, and coral and other species such as gorgonians and sponges are using these structures for recruitment. The structures also encourage coral recruitment where the availability of hard-bottom habitat is a limiting factor.

Invertebrates (particularly Echinoderms - sea urchins, sea cucumbers and star fish) must be relocated immediately before (morning of) excavation/construction work begins to prevent them from re-occupying the space during construction

Oil Pollution: There is the potential for fuel leaks or spills from equipment used for groyne and breakwater construction, excavation and or sand nourishment during refuelling or operation.

Appropriate refuelling equipment (such as funnels) and techniques should be used at all times. There should be appropriate minor spill response equipment (for containment and clean up).

The assessments have concluded that the project could have moderate to significant marine environmental effects. Works are likely to cause some short-term increases in suspended sediment concentrations, although turbidity barriers are to be erected around the area of works to control this. Some ecological habitats and species may be at risk of increased turbidity, increased pollution during the works, and habitat damage due to the placement of the permanent structures. With adherence to mitigation measures and monitoring procedures, potentially adverse impacts can be minimised, avoided or compensated for.

Appendix A

Reef Check – Substrate type categories and respective description

Substrate type	Code	Category	Description
Hard Coral	HC	Living	All living coral including bleached coral; includes fire, blue and organ pipe corals
Soft Coral	SC	Living	Include zooanthids but not anemones (OT)
Recently Killed Coral	RKC	Non-Living	Coral that has died within the past year; appears fresh and white or with corallite structures still recognizable
Nutrient Indicating Algae (or Marco Algae)	NIA	Living	All macro-algae except coralline, calcareous and turf (record the substrate beneath for these); Halimeda is recorded as OT; turf is shorter than 3cm
Sponge	SP	Living	All erect and encrusting sponges (but no tunicates)
Rock (or Pavement)	RC	Non-Living	Any hard substrate; includes dead coral more than 1 yr old and may be covered by turf or encrusting coralline algae, barnacles, etc.
Rubble	RB	Non-Living	Reef rocks between 0.5 and 15cm in diameter
Sand	SD	Non-Living	Sediment less than 0.5cm in diameter; in water, falls quickly to the bottom when dropped
Silt/Clay	SI	Non-Living	Sediment that remains in suspension if disturbed; recorded if color of the underlying surface is obscured by silt
Other	OT	Living	Any other sessile organism including sea anemones, tunicates, gorgonians or non-living substrate

For details on the Reef Check Method visit http://reefcheck.org/rcca/monitoring_protocol.php for details.

AGRRA Protocol – Fish Census Frequency

A – Abundant (>100 individuals)

M – Many (11 – 100 individuals)

F – Few (2 – 10 individuals)

S- Single

Corals and Invertebrates observed in Section A

Common Name	Scientific Name	Economic/ Ecological Value/Comments	Frequency
Finger Coral	*Porites porites*	Reef building coral	F
Mustard Hill coral	*Porites asteroides*	Reef building coral	A
Brain Coral	*Colpophyllia natans*		F
Brain Coral	*Diploria strigosa*	Reef building coral	A
Brain Coral	*Diploria labyrinthiformis*		F
Brain Coral	*Diploria clivosa*	Reef building coral	A
Brain Coral	*Meandrina meandrites*		F
	Dichocoenia stokesii		F
Lesser Starlet Coral	*Siderastrea radians*		A
Star Coral	*Montastrea annularis*	Reef building coral	A
Star Coral	*Montastrea faveolata*	Reef building coral	F
	Montastrea cavernosa	Reef building coral	F
Rose Coral	*Manicina areolata*		F
Tube Sponge	*Psuedoceratina crassa*		F

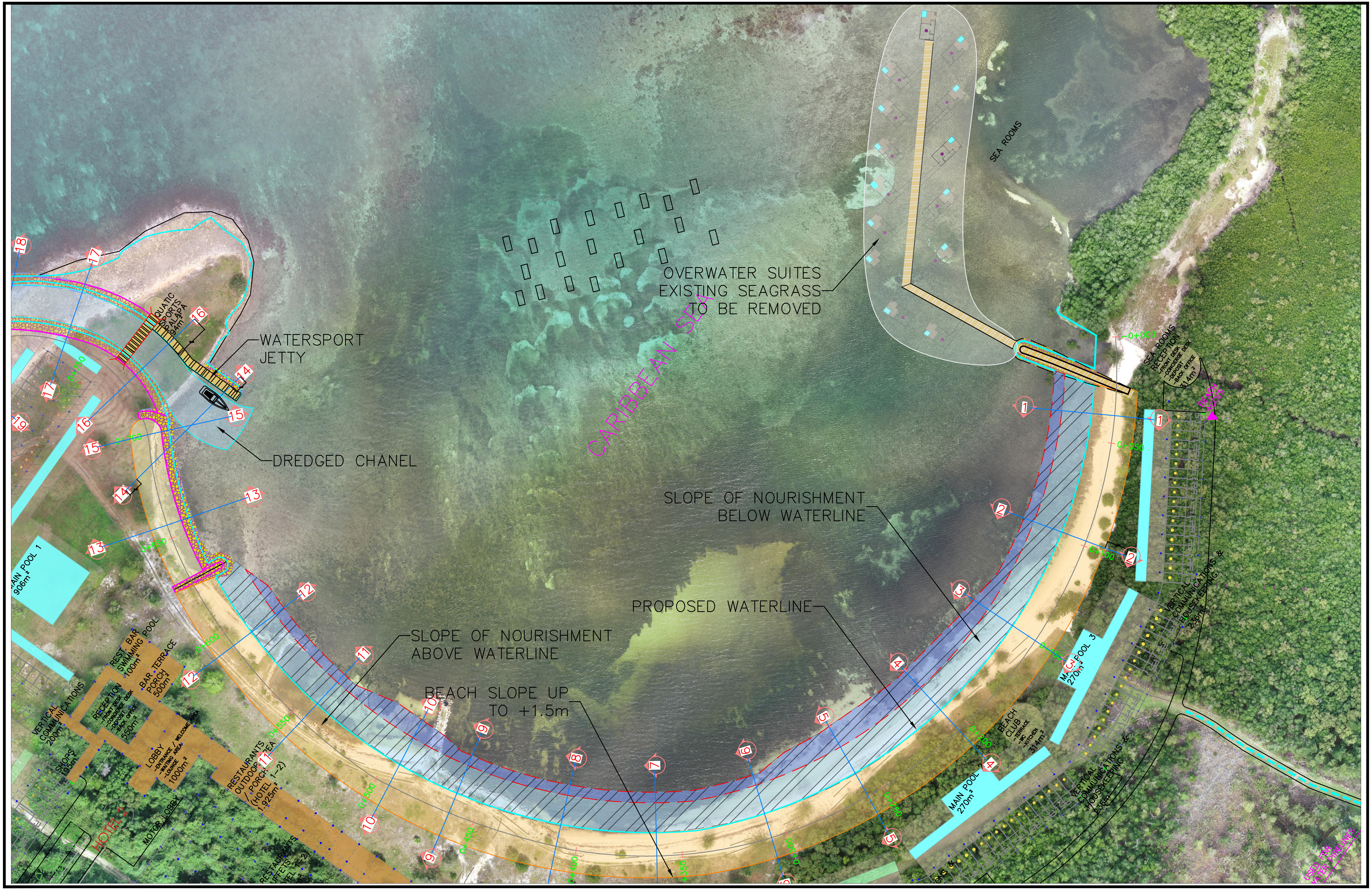
A – Abundant (>100) ; M – Many (11 – 100); F – Few (2 – 10); S – Single (1)

Fish species observed during survey

Common Name	Scientific Name	Frequency
Diodon holocanthus	Balloonfish	S
Hemiramphus brasiliensis	Ballyhoo	F
Stegastes leucostictus	Beaugregory	F
Stegastes partitus	Bicolor Damsel	F
Thalassoma bifasciatum	Blue Head Wrasse	M
Haemulon sciurus	Blue Striped Grunt	M
Acanthurus coeruleus	Blue Tang	F
Acanthurus chirurgus	Doctorfish	F
Dactylopterus volitans	Flying Gurnard	S
Stegastes dorsopunicans	Dusky Squirrelfish	M
Myrichthys ocellatus	Gold Spot Eel	S
Lutjanus synagris	Lane Snapper	F
Echeneidae sp	Remora	F
Sparisoma chrysopterum	Redfin Parrot	F
Lutjanus apodus	Schoolmaster	M
Abudefduf saxatilis	Sergeant Major	M
Canthigaster rostrata	Sharpnose puffer	S
Halichoeres bivittatus	Slippery Dick	M
Aulostomus maculatus	Trumpetfish	S

Appendix C Drawings


N
S
W
E
WATERSPORT JETTY
OVERWATER SUITES EXISTING SEAGRASS TO BE REMOVED
CARIBBEAN SEA
REVETMENT ⊚ +3.0m
PERCHED BEACH ⊚ +2.5m
DREDGED CHANEL
SLOPE OF NOURISHMENT BELOW WATERLINE
GAZEBO
PROPOSED WATERLINE
BREAKWATER
BURIED GROYNE
SLOPE OF NOURISHMENT ABOVE WATERLINE
GAZEBO
CUT STONE WALL
BEACH SLOPE UP TO +1.5m
REVETMENT ⊚ +3.0m
DREDGED CHANEL
CARIBBEAN SEA
BREAKWATER ⊚ −0.3m
BEACH SLOPE UP TO +1.3m
DREDGED CHANEL
DREDGED SWIMMING AREA
BREAKWATER ⊚ −0.3m
BEACH SLOPE UP TO +1.3m
DREDGED CHANEL
GAZEBO
GROYNE
EXCAVATED BEACH
0
50
100
150
200
Meters

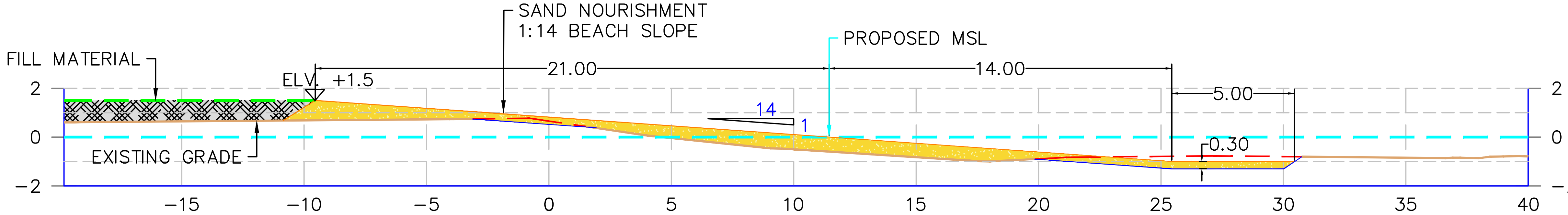


OVERWATER SUITES
EXISTING SEAGRASS
TO BE REMOVED
SEA ROOMS
WATERSPORT
JETTY
DREDGED CHANEL
CARIBBEAN SEA
SLOPE OF NOURISHMENT
BELOW WATERLINE
PROPOSED WATERLINE
SLOPE OF NOURISHMENT
ABOVE WATERLINE
BEACH SLOPE UP
TO +1.5m
MAIN POOL 1
906m²
MAIN POOL
270m²
BEACH CLUB

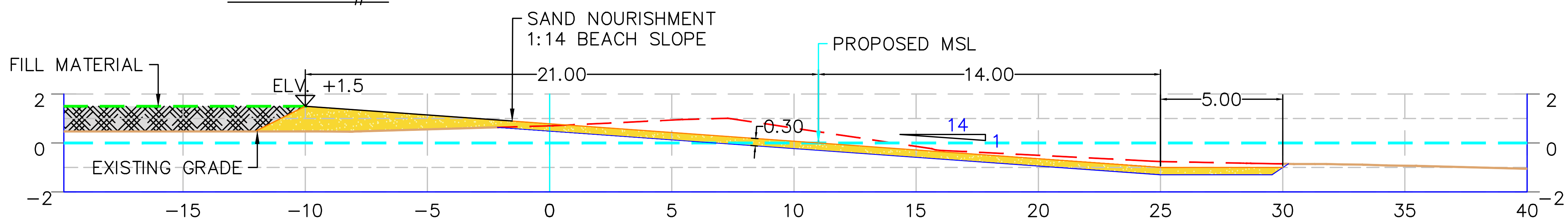

N
W
E
S
REVETMENT @ +3.0m
PERCHED BEACH @ +2.5m
GAZEBO
BREAKWATER
BURIED GROYNE
CUT STONE WALL
GAZEBO
REVETMENT @ +3.0m
VERTICAL COMMUNICATIONS & HOUSEKEEPING 132m²
MAIN POOL 1 906m²
BEACH CLUB
MAIN POOL 2 1723m²
WATER PARK 1862m²
VERTICAL COMMUNICATIONS 200m²
SHOPS
HOTEL 2
BEACH CLUB
VERTICAL COMMUNICATIONS & HOUSEKEEPING 375m²
COMMISSARY SERVICE AREA KITCHEN
MAIN POOL 1 2985m²
WATER PARK 1862m²
VERTICAL COMMUNICATIONS & HOUSEKEEPING
SERVICE YARD (CASINO)
CASINO
MAIN POOL 1 (HOTEL 3-4) 1671m²
LOBBY
RECEPTION 373m²
VERTICAL COMMUNICATIONS 207m²
MOTOR LOBBY
HOTEL 3
0
25
50
75
100
Meters


N
W
E
S
CARIBBEAN SEA
DREDGED CHANEL
BREAKWATER @ −0.3m
BEACH SLOPE UP TO +1.3m
DREDGED CHANEL
DREDGED SWIMMING AREA
BREAKWATER @ −0.3m
BEACH SLOPE UP TO +1.3m
DREDGED CHANEL
GAZEBO
GROYNE
EXCAVATED BEACH
MAIN POOL 1 (HOTEL 3-4) 1671m²
BAR TERRACE/PORCH 1010m²
REST. BAR SWIMMING POOL (HOTEL 3-4) 200m²
RESTAURANTS OUTDOOR AREA / PORCH (HOTEL 3-4) 2,050m²
MAIN POOL 2 (HOTEL 3-4) 906m²
LOBBY
MOTOR LOBBY
BEACH CLUB 298m²
MAIN POOL 1 1671m²
MAIN POOL 2 1037 m²
VERTICAL COMMUNICATIONS SHOPS
VERTICAL COMMUNICATIONS & HOUSEKEEPING
SURFACE PARKING (HOTEL 4) 146 PLACES
28
29
29
30
30
31
31
32
32
33
33
34
34
35
35
36
36
37
37
38
38
39
39
40
40
41
41
0 25 50 75 100
Meters

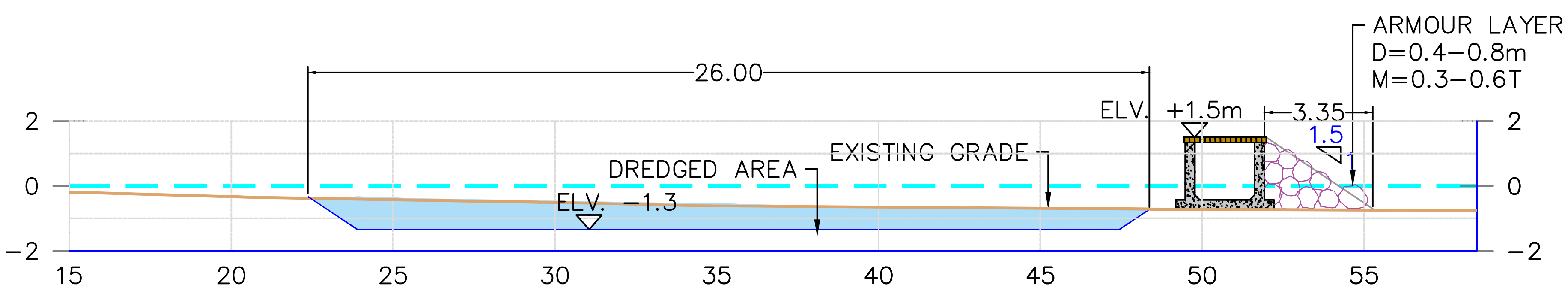
PROFILE #2



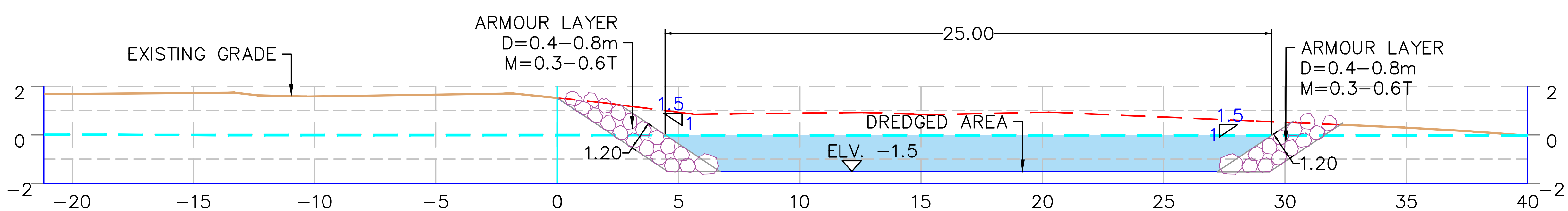
PROFILE #6

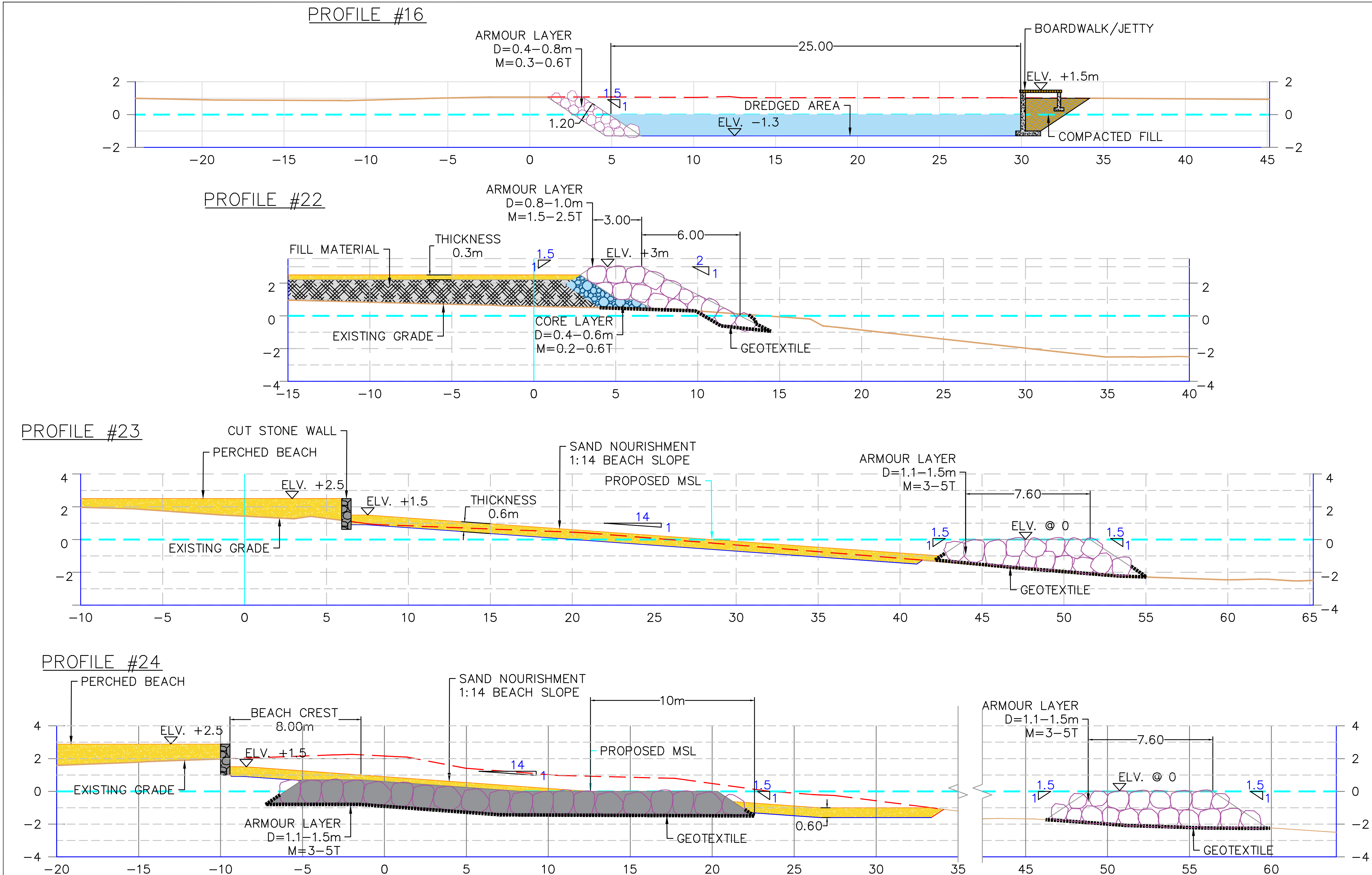


PROFILE #14



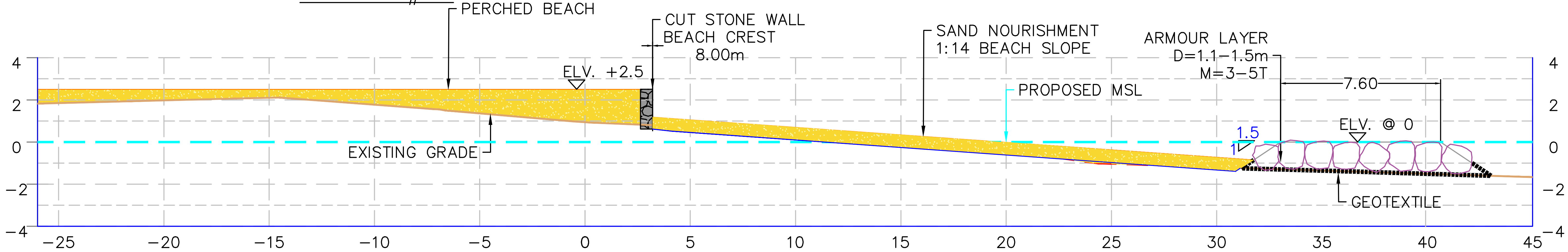
PROFILE #17



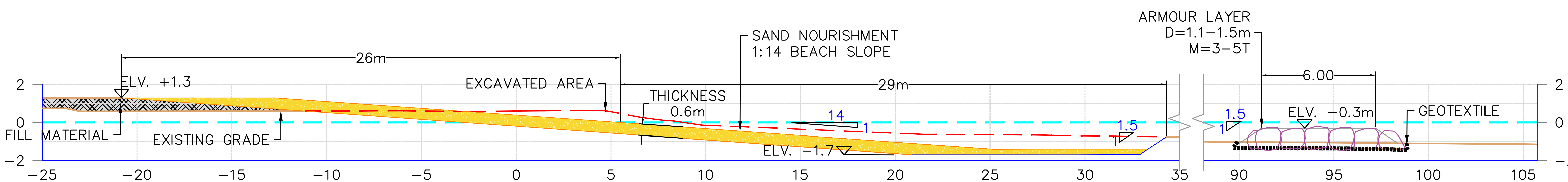


PROFILE #16
ARMOUR LAYER
D=0.4–0.8m
M=0.3–0.6T
BOARDWALK/JETTY
25.00
ELV. +1.5m
1.5
1
DREDGED AREA
ELV. −1.3
1.20
COMPACTED FILL
PROFILE #22
ARMOUR LAYER
D=0.8–1.0m
M=1.5–2.5T
3.00
6.00
THICKNESS
0.3m
FILL MATERIAL
ELV. +3m
2
1
EXISTING GRADE
CORE LAYER
D=0.4–0.6m
M=0.2–0.6T
GEOTEXTILE
PROFILE #23
CUT STONE WALL
PERCHED BEACH
SAND NOURISHMENT
1:14 BEACH SLOPE
ARMOUR LAYER
D=1.1–1.5m
M=3–5T
PROPOSED MSL
ELV. +2.5
ELV. +1.5
THICKNESS
0.6m
14
7.60
ELV. @ 0
EXISTING GRADE
GEOTEXTILE
PROFILE #24
PERCHED BEACH
SAND NOURISHMENT
1:14 BEACH SLOPE
BEACH CREST
8.00m
10m
ELV. +2.5
ELV. +1.5
PROPOSED MSL
EXISTING GRADE
ARMOUR LAYER
D=1.1–1.5m
M=3–5T
GEOTEXTILE
0.60
7.60
ELV. @ 0

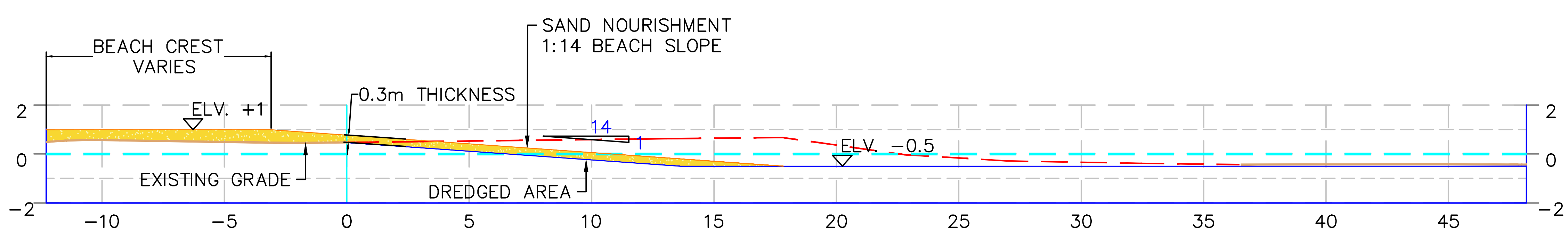
PROFILE #25



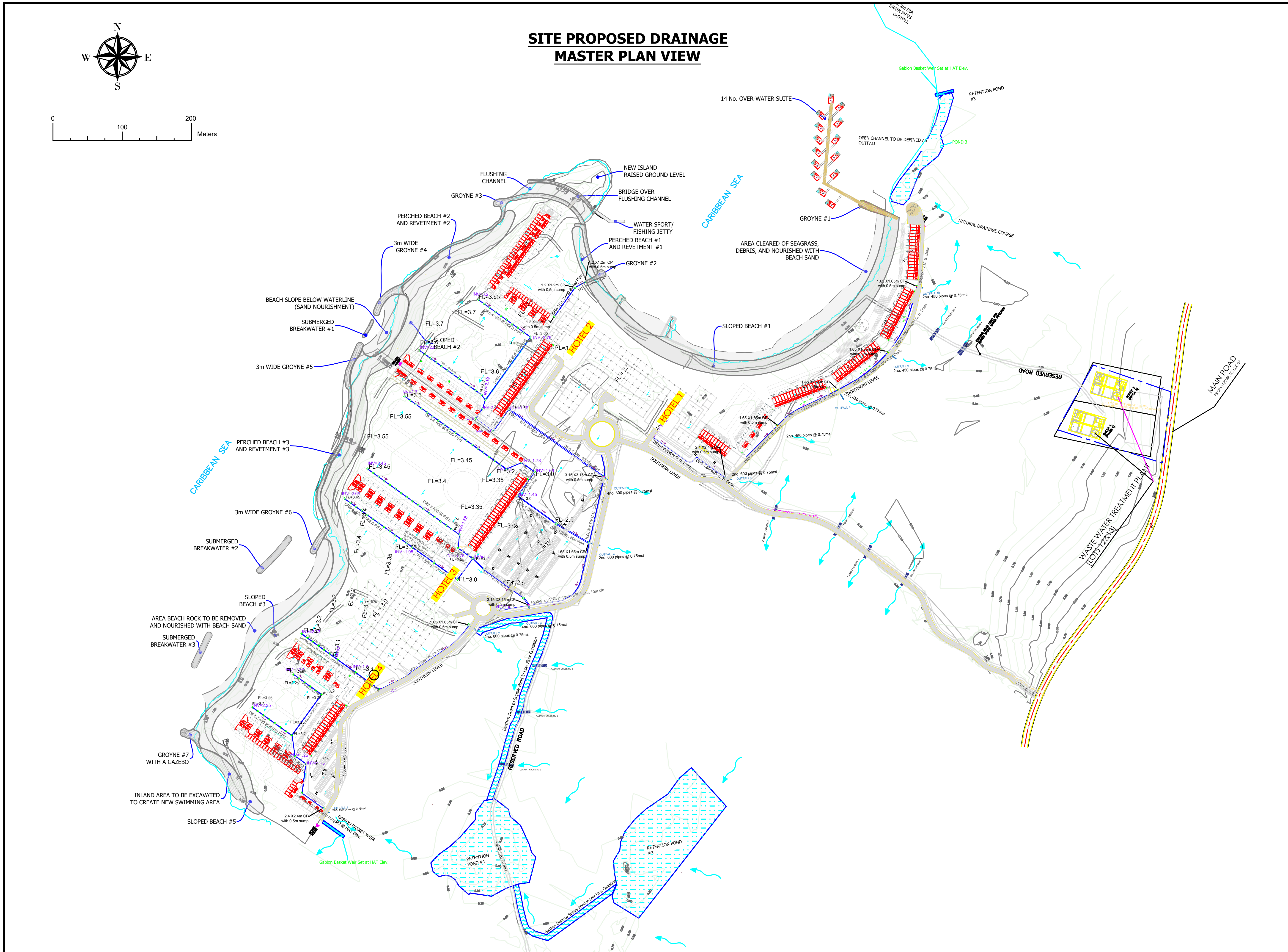
PROFILE #33



PROFILE #40



SITE PROPOSED DRAINAGE MASTER PLAN VIEW



1. All dimensions and elevations are in meters unless otherwise shown.

2. Survey to be verified on site prior to start of works.

DRAINAGE LEGEND

FL=3.1
INV=2.33

- 1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
- 2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
- 3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
- DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
- DRAIN CULVERT
- PROPOSED ROAD WAY
- OUTFALL POINT
- RUNOFF FLOW INTO EXISTING MANGROVE AREAS
- RUNOFF FROM PROPOSED HOTEL SITE
- FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

No.	description	date
01	Client Review	Sept-30-19

project title

PRINCESS HOTEL & RESORTS, SHORELINE ENHANCEMENT

client

PRINCESS HOTELS & RESORTS, GREEN ISALND JAMAICA

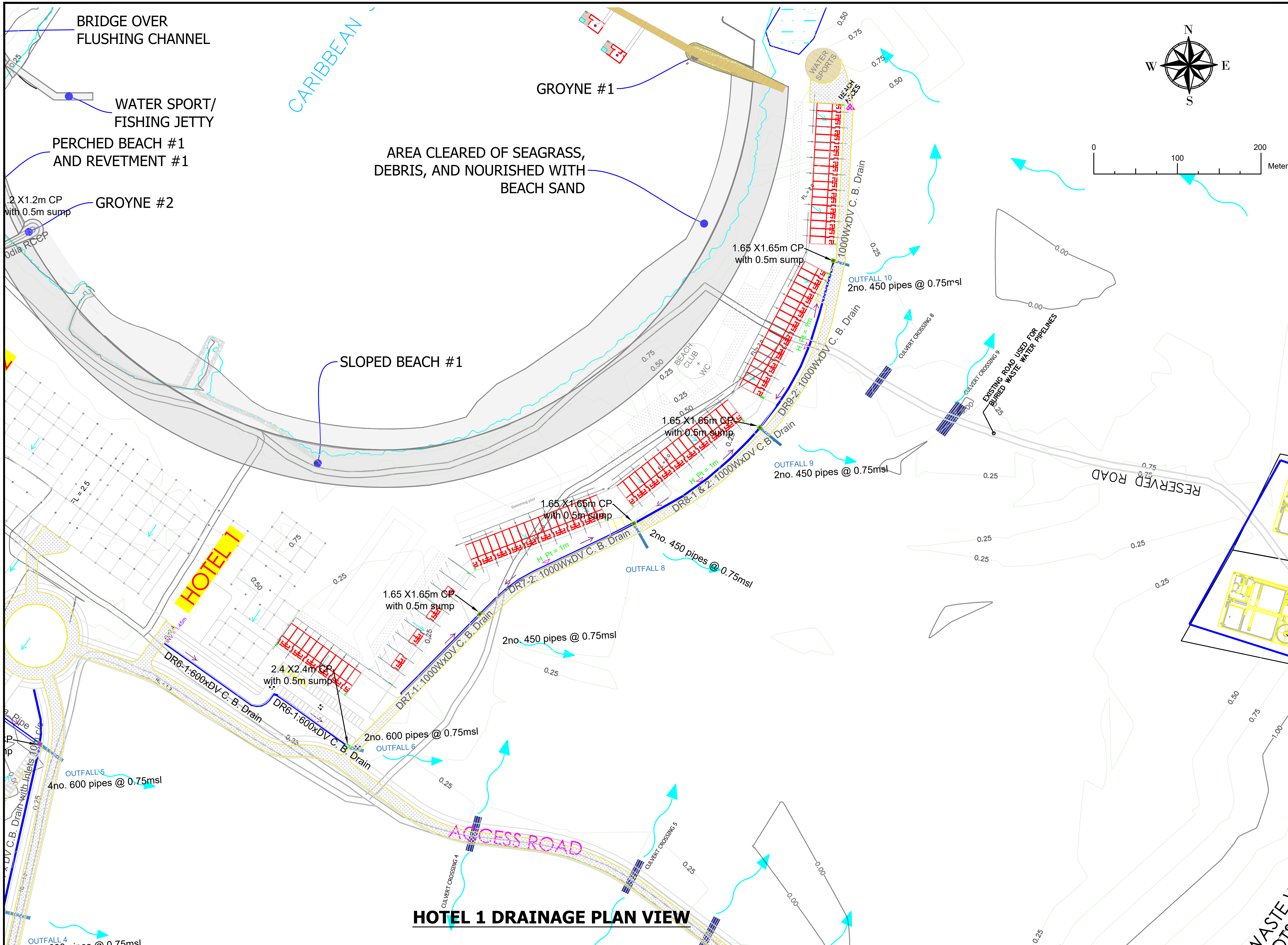
drawing title

SITE PROPOSED DRAINAGE MASTER PLAN

SMITH WARNER INTERNATIONAL
COASTAL ENVIRONMENT

Unit 13, Seymour Park,
2 Seymour Avenue
Kingston 10, Jamaica, W.I.
Tel: (876)978-8950 / (876)978-7415
Fax: (876)978-0685

Designed by:	M. Harris	Aug '19	sheet no. 1 of 5	rev. 01
Checked by:	J. Banton	Aug '19		
Drawn by:	R. Lyn	Aug '19		
scale AS SHOWN	drawing no. SWIL-PHRD01			



HOTEL 1 DRAINAGE PLAN VIEW

1. All dimensions and elevations are in meters unless otherwise shown.

2. Survey to be verified on site prior to start of works.

DRAINAGE LEGEND

FL=3.1 INV=2.33

1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP

2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP

3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP

DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)

DRAIN CULVERT

PROPOSED ROAD WAY

OUTFALL POINT

RUNOFF FLOW INTO EXISTING MANGROVE AREAS

RUNOFF FROM PROPOSED HOTEL SITE

FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

No.	description	date
01	Client Review	Sept-30-19

project title

PRINCESS HOTEL & RESORTS, SHORELINE ENHANCEMENT

client

PRINCESS HOTELS & RESORTS, GREEN ISALND JAMAICA

drawing title

HOTEL 1 DRAINAGE PLAN VIEW- SHEET 01A

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Drawn by:	R. Lyn	Aug' 19		
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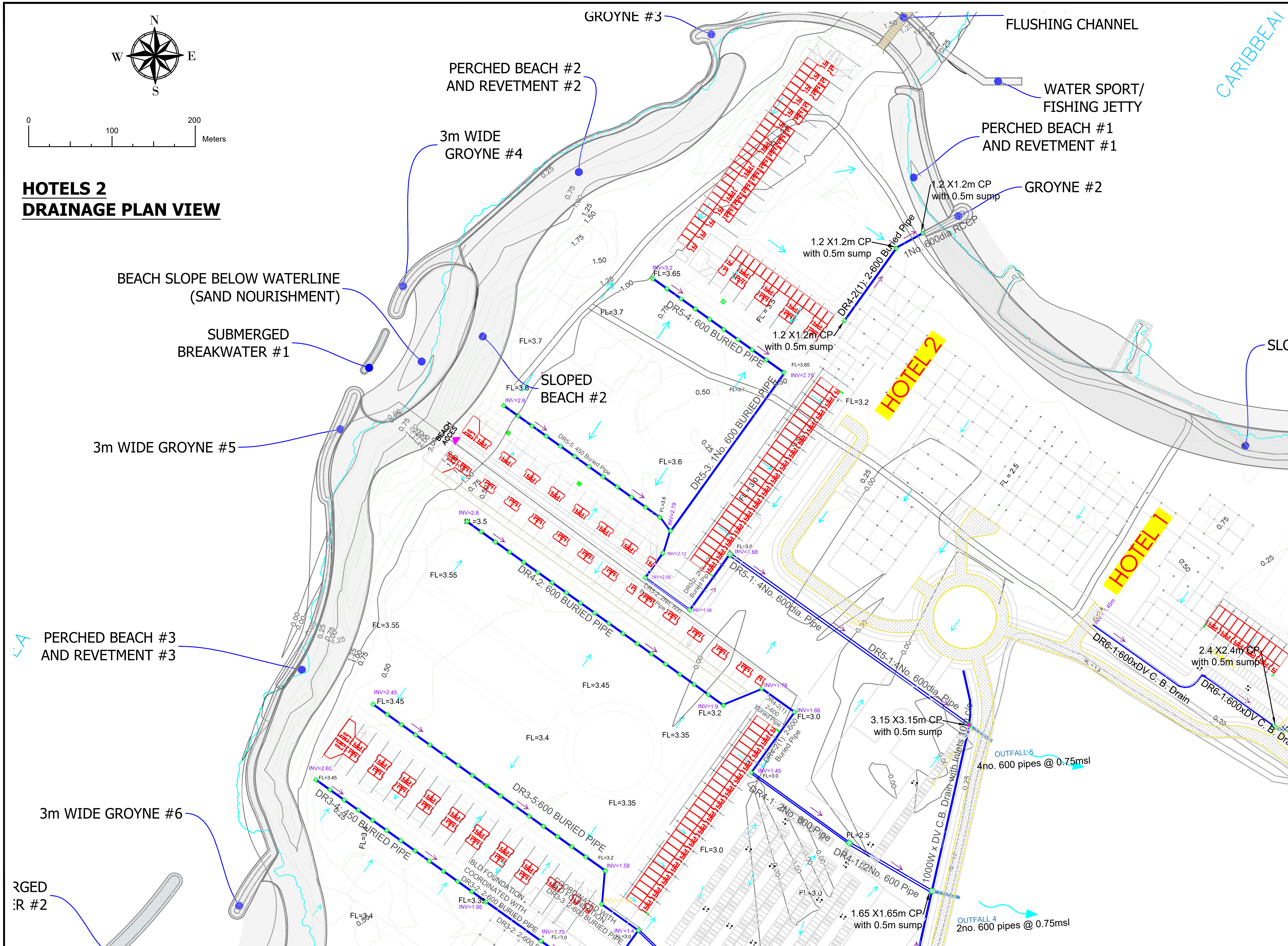

1. All dimensions and elevations are in meters unless otherwise shown.

2. Survey to be verified on site prior to start of works.

DRAINAGE LEGEND

- FL=3.1 INV=2.33 — 1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
- 2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
- 3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
- DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
- DRAIN CULVERT
- PROPOSED ROAD WAY
- OUTFALL POINT
- RUNOFF FLOW INTO EXISTING MANGROVE AREAS
- RUNOFF FROM PROPOSED HOTEL SITE
- FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

No.	description	date
01	Client Review	Sept-30-19

project title

PRINCESS HOTEL & RESORTS, SHORELINE ENHANCEMENT

client

PRINCESS HOTELS & RESORTS, GREEN ISALND JAMAICA

drawing title

HOTEL 2 DRAINAGE PLAN VIEW-SHEET 01B

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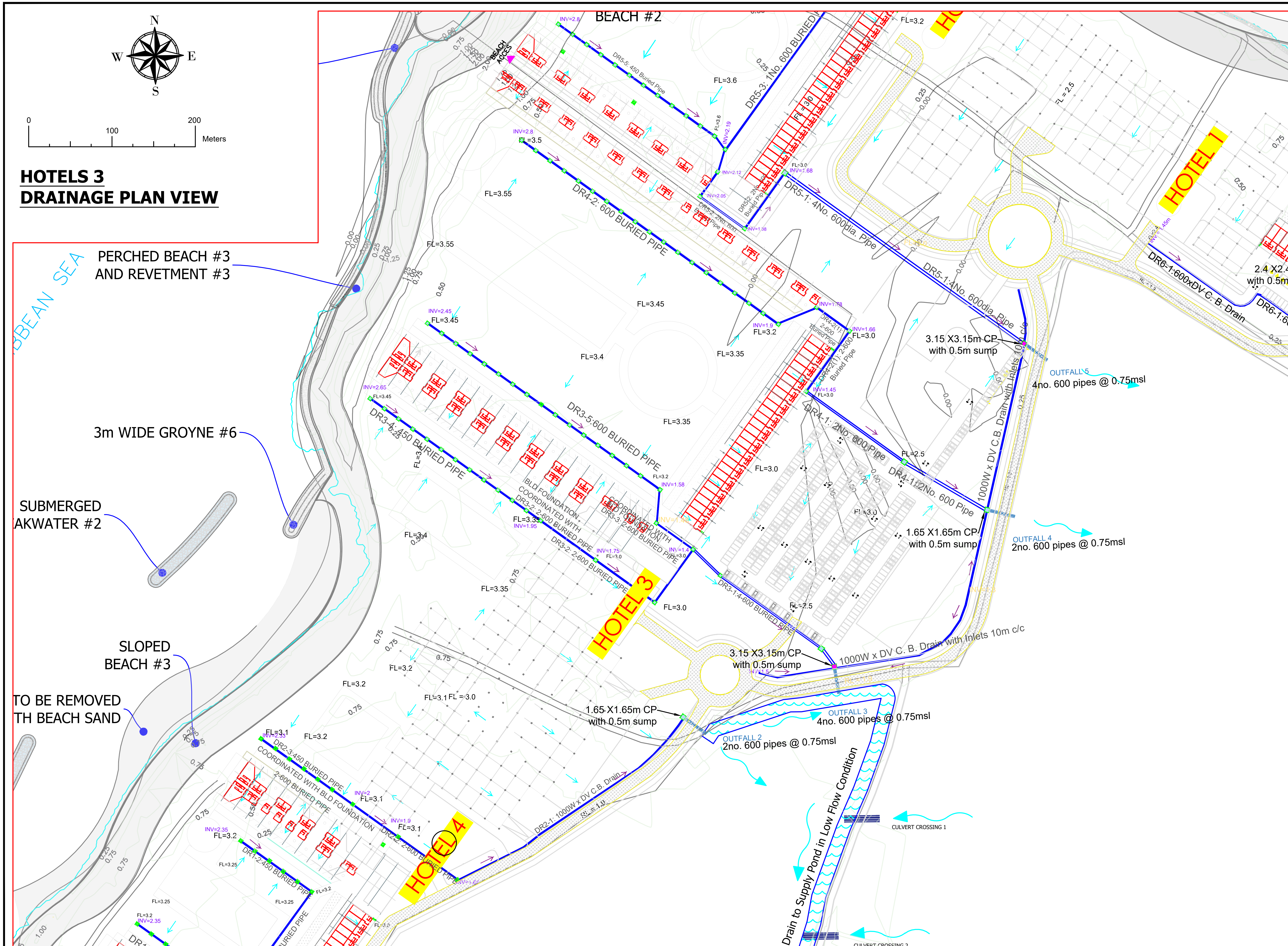
Designed by:	M. Harris	Aug '19	sheet no. 3 of 5	rev. 01
Checked by:	J. Banton	Aug '19		
Drawn by:	R. Lyn	Aug '19		
scale AS SHOWN	drawing no. SWIL-PHRD03			



1. All dimensions and elevations are in meters unless otherwise shown.

2. Survey to be verified on site prior to start of works.

DRAINAGE LEGEND

- FL=3.1 INV=2.33 — 1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
- 2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
- 3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
- DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
- DRAIN CULVERT
- PROPOSED ROAD WAY
- OUTFALL POINT
- RUNOFF FLOW INTO EXISTING MANGROVE AREAS
- RUNOFF FROM PROPOSED HOTEL SITE
- FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

No.	description	date
01	Client Review	Sept-30-19

project title

PRINCESS HOTEL & RESORTS, SHORELINE ENHANCEMENT

client

PRINCESS HOTELS & RESORTS, GREEN ISALND JAMAICA

drawing title

HOTEL 3 DRAINAGE PLAN VIEW-SHEET 01C

SMITH WARNER INTERNATIONAL
COASTAL ENVIRONMENT

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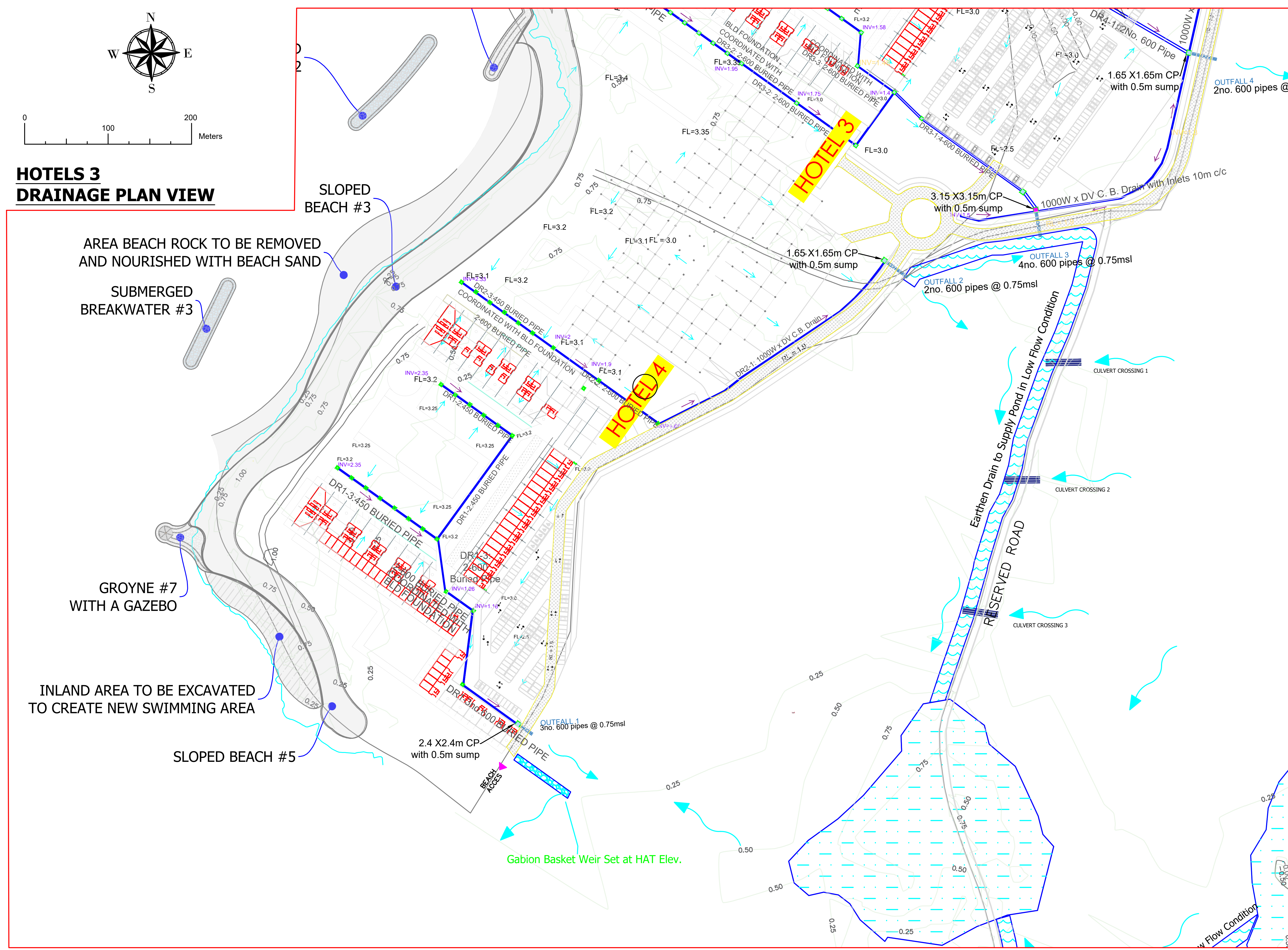
Checked by:	J. Banton	Aug' 19	
Drawn by:	R. Lyn	Aug' 19	rev. 01
scale: AS SHOWN	drawing no. SWIL-PHRD04		



1. All dimensions and elevations are in meters unless otherwise shown.

2. Survey to be verified on site prior to start of works.

DRAINAGE LEGEND

FL=3.1
INV=2.33

- 1.65m X 1.65m INSPECTION CHAMBER WITH 0.5m SUMP
- 2.4m X 2.4m INSPECTION CHAMBER WITH 0.5m SUMP
- 3.15m X 3.15m INSPECTION CHAMBER WITH 0.5m SUMP
- DRAIN CHANNELS (EACH TYPE & DESCRIPTION ARE SHOWN ON PLAN VIEWS)
- DRAIN CULVERT
- PROPOSED ROAD WAY
- OUTFALL POINT
- RUNOFF FLOW INTO EXISTING MANGROVE AREAS
- RUNOFF FROM PROPOSED HOTEL SITE
- FLOW DIRECTION THROUGH DESIGNED DRAINAGE SYSTEM

No.	description	date
01	Client Review	Sept-30-19

project title

PRINCESS HOTEL & RESORTS, SHORELINE ENHANCEMENT

client

PRINCESS HOTELS & RESORTS, GREEN ISALND JAMAICA

drawing title

HOTEL 4 DRAINAGE PLAN VIEW, SHEET 01D

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COASTAL ENVIRONMENT

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rev. 01

scale: AS SHOWN

drawing no.: SWIL-PHRD05