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ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

PRELIMINARY DESIGN REPORT

REPORT NO. : 1097/02/04 REV C

AUGUST 2013

PRESTEDGE RETIEF DRESNER WIJNBERG (PTY) LTD
CONSULTING PORT AND COASTAL ENGINEERS

Rupert's Bay Permanent Wharf – Phase 2: Preliminary Design Report Report No. 1097/02/04 Rev C					
Revision	**Date**	**Author**	**Checked**	**Status**	**Approved**
A	14 May 2013	SRP	AHH		
B	21 May 2013	SRP	AHH		
C	16 Aug 2013	SRP	AHH	For Use	
Keywords: St Helena Island, Rupert's Bay, Permanent Wharf,					

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RUPERT'S BAY PERMANENT WHARF - PHASE 2

PRELIMINARY DESIGN

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1. INTRODUCTION

The construction of a new airport on the island of St Helena will require the existing port facilities on the island to be upgraded to allow both the landing of contractor's equipment and supplies during construction, and the provision of permanent facilities for handling bulk cargo, petroleum products, general cargoes and containers in the medium to long-term. The site selected for this facility is Rupert's Bay on the North West coast of the island. The location of the site is shown in Drawing No. PRDW-900-MN-0002-01.

This document presents the preliminary design for the provision of a permanent wharf structure at Rupert’s Bay, St Helena Island. The latest revision of the document updates design parameters that were changed in order to meet a target project budget. The main changes relate to a reduced design vessel, exclusion of the Ro-Ro ramp and exclusion of a Lighter Berth facility in favour of incorporation of the facility into the main berth structure.

1.1 Project Phase Definition

The design component of the Rupert’s Bay Permanent Wharf Project comprises three phases defined below:

- Phase 1 : Scoping and Optimisation
- Phase 2 : Preliminary Engineering Design
- Phase 3 : Detailed Design

This report deals with Phase 2: Preliminary Engineering Design.

1.2 Report Structure

This report consists of six sections including the current section. Section 2 describes the updates made to the design basis as a result of the design development process, whilst Section 3 provides a high level summary of the site conditions in Rupert’s Bay. Section 4 provides a description of the general wharf layout including navigational and operational issues and the relocation of the Bulk Fuel Offloading Facility. Section 5 describes the marine structures (breakwaters and all quay structures) which were considered as part of the preliminary design process. The report is concluded in Section 6 with recommendations for the next design stage.

1.3 Conventions and Terminology

The following conventions and terminology are used in this report:

- Wave direction is the direction from which the wave is coming, measured clockwise from true north.
- Wind direction is the direction from which the wind is coming, measured clockwise from true north.
- Current direction is the direction towards which the current is flowing, measured clockwise from true north.
- Hm0 is the significant wave height, determined from the zeroth moment of the wave energy spectrum. It is approximately equal to the average of the highest one-third of the waves in a given sea state.
- Tp is the peak wave period, defined as the wave period with maximum wave energy density in the wave energy spectrum.
- Mean wave direction (Dir) is defined as the mean direction calculated from the full two-dimensional wave spectrum by weighting the energy at each frequency
- Seabed and water levels are measured relative to Chart Datum. Chart Datum (CD) is 0.50 m below Mean Sea Level.

2. UPDATES TO DESIGN BASIS

The basis of design document has guided the development of the preliminary design of the permanent wharf facilities in Rupert's Bay. A copy of the latest basis of design document (PRDW, 2013a) is included in Appendix A for ease of reference.

The basis of design provides a detailed description of the Guidelines and Codes of Practice used in the development of the design as well as the functional requirements of the proposed facility.

The following changes to the design basis as a result of the preliminary design process are noted:

1. The apron width of main berth is 13m which extends from the cope to the road kerb barrier. A 0.7 m wide servitude has been allowed for in the design.
2. A passenger landing facility is provided at the root of the main quay for lighters.
3. A Ro-Ro ramp is no longer included in the facility. . The contractor's existing temporary Ro-Ro facility will be retained.
4. The design vessel draft has been reduced to 5.5m and the beam width to 17m.
5. The desktop vessel manoeuvring assessment is based on a 5,500 DWT multipurpose container vessel, which corresponds to existing vessels currently operating in the South Atlantic off the coast of South and West Africa.
6. The design water level has been reviewed and updated based on an extreme water level analysis. The design water level excluding and including an allowance for climate change (sea level rise) is +1.22 m CD and +1.92 m CD, respectively.
7. A combined bathymetric data-set based on a single-beam (2006) and multi-beam (2012) bathymetric survey was used as a basis of the preliminary design.
8. Operational and extreme wave conditions for the design of marine structures are defined as the 1:1 yr (1.6m), 1:30 yr (2.8m) and 1:1000 yr (4.6m) return period wave height, respectively.

3. SITE CONDITIONS

This section provides a summary of the site conditions at Rupert's Bay. A more detailed description of the met ocean site conditions in Rupert's Bay is provided in Appendix B - Coastal Processes Report (PRDW, 2013b).

3.1 Design Water Levels

The design water level is summarised in Table 3-1. The design water level was calculated as the 1:100 year residual water level superimposed on the Mean High Water Spring (MHWS) tide. The effects of sea-level rise and increases in storm surge were included while taking into account the 70 year design life of the structure.

Table 3-1: Extreme water levels

Parameter	Water level excluding Climate Change	Water level including Climate Change
Tide Level (MHWS)	+0.94 m CD	+0.94 m CD
Residual (1:100)	0.28 m	0.33 m
Sea-level rise	0 m	0.65 m
Total Water Level	+1.22 m CD	+1.92 m CD

3.2 Bathymetry

The results of a single-beam bathymetric survey performed in 2006 (Tritan, 2006) and a multi-beam survey performed in 2012 (Tritan, 2012) were used in this study. The combined bathymetry is presented in Figure 3-1. A bay wide multi-beam survey will be required as part of the detail design.

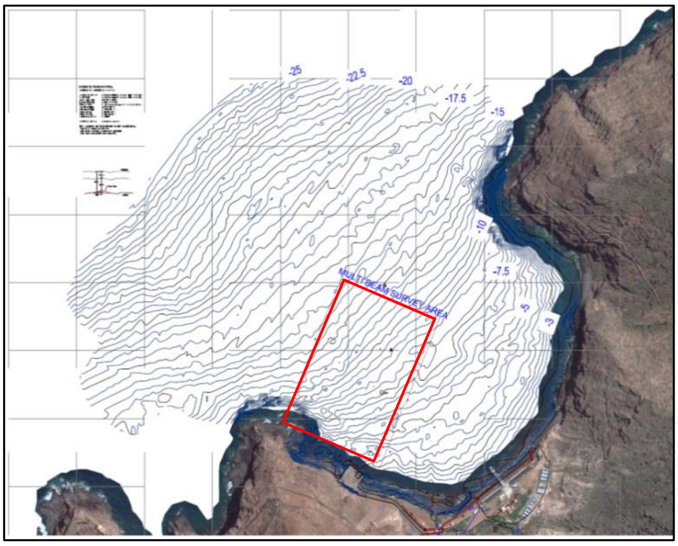


Figure 3-1: Rupert's Bay bathymetry plan. Consolidated from the 2006 and 2012 surveys.

3.3 Seabed and Geotechnical Conditions

The seabed in Rupert's Bay is characterised by a layer of fine to medium grained sediments overlying the igneous bedrock, expected to comprise hard to extremely hard rock. The sediment thickness ranges from nothing to 3.0 m thick within the bay. The sediments may be mobile under the seasonal storm wave events (Tritan, 2006). The sub-sea geology plan is included in Figure 3-2.

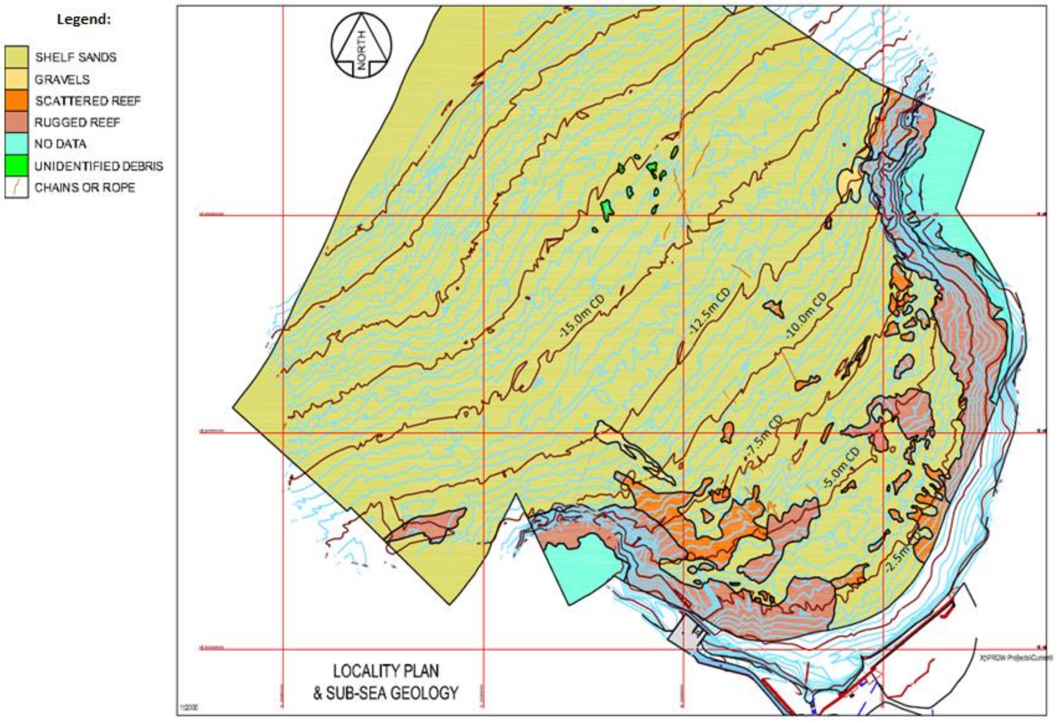


Figure 3-2: Rupert's Bay sub-sea geology (Tritan, 2006).

A coastal sediment sampling survey revealed that the coastline of Rupert’s Bay is characterised by medium to very coarse sediment, with the median particle diameter ranging from 0.27 mm at the swimming beach to 32 mm at the southern headland.

A bay-wide sediment survey covering the extent of Rupert’s Bay was also undertaken in which sediment samples were collected from the sea bottom and observations were made regarding the positions of exposed reefs. The data indicates a fining of the sediment with increasing distance offshore into deeper water. The results of the survey are presented in Figure 3-3 as a spatial distribution of the size of the median particle diameter (D_{50}).

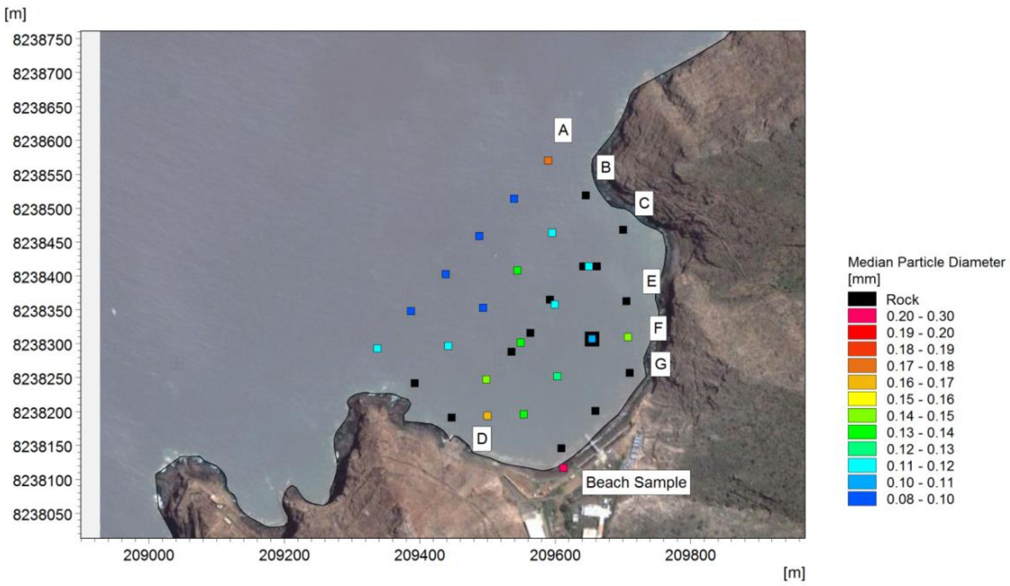


Figure 3-3: Spatial distribution of median particle diameters (D_{50}) as determined in the bay-wide sediment assessment.

Based on the available information and interpretations of the seabed geology, the mobilisation of a marine jack-up platform to carry out a detailed marine geotechnical investigation (boreholes, SPT's etc) seems excessive. However, there will always be an inherent risk about the founding conditions which relate to:

1. Material properties of the overlying sediments;
2. Integrity of the bedrock which relates to rock cavities below the seabed.

It is recommended that SPTu's from a barge as well as vibrocores be carried out along the axis of the breakwater and main quay structure once marine kit is established in Rupert's Bay. A geotechnical desktop study and some landside core drilling at the toe of the breakwater will highlight if the bedrock integrity is of concern.

3.4 Design Wave Conditions

Regional wave modelling has been performed using offshore hindcast wave data to determine the nearshore wave climate within Rupert's Bay. Calibration has been performed by comparing the simulated waves to measured waves. An Extreme Value Analysis of the modelled nearshore conditions was subsequently performed on the modelled conditions, to determine the design wave conditions (Table 3-2) for the marine infrastructure.

It is anticipated that the mobilization of repair plant will be significantly higher for this structure due to the remote location of the island, and that a 7% risk of occurrence was more appropriate rather than the recommended 10% (AS 4997 -2005). This relates to a return period of 1000 years.

Table 3-2: Design waves (PRDW, 2013b)

Parameter	Extreme*	Drainage**	Operational***
Return period [yr]	1000	30	1
H_{mo} [m]	4.6	2.8	1.6
Tp [s]	16	16	16

Notes:

*slope stability and crown wall design (no damage criteria)

**design of drainage system

***maximum allowable overtopping rates and criteria for crest elevation

3.5 Currents

The current speeds in Rupert's Bay are very low, with the highest current speed measured between December 2006 and September 2012 being 0.25 m/s. An investigation into the mechanism forcing the currents revealed that the currents in Rupert's Bay include a tidal forcing, as observed in the oscillation of current direction with the tide.

3.6 Wind

Winds on St. Helena Island blow almost constantly from the SE with an hourly average wind speed of 6.5 m/s. Due to the topography of the valley leading down to Rupert's Bay, winds are expected to follow the path of the valley, which roughly runs in a SE-NW orientation. A slight seasonality was observed in the wind data with winds in the months of spring and winter being slightly stronger than those in autumn and summer. The extreme wind conditions (hourly average) are presented in Table 3-3.

Table 3-3: Extreme wind conditions (hourly average)

Return Period [years]	Wind Speed [m/s]
1	14.9
10	17.7
50	19.8
100	20.7

3.7 Sedimentation

Coupled two-dimensional sediment transport modelling has been performed to investigate the direct impact of the proposed development on the wave, current and sediment transport characteristics.

Results of this analysis indicate that the development will result in significant wave sheltering in the southern region of Rupert's Bay. This sheltering results in a changed current pattern in Rupert's Bay, with the rip current that occurs during large wave events in the status quo not being present following the implementation of the proposed development.

The changed wave and current patterns resulting from the implementation of the proposed permanent wharf facilities result in changes to the sediment transport regime. A status quo scenario of the change in sediment regime within Rupert's Bay based on a 1:100 year storm event is shown in Figure 3-4 below. Figure 3-5 shows a similar scenario with the inclusion of the proposed permanent wharf.

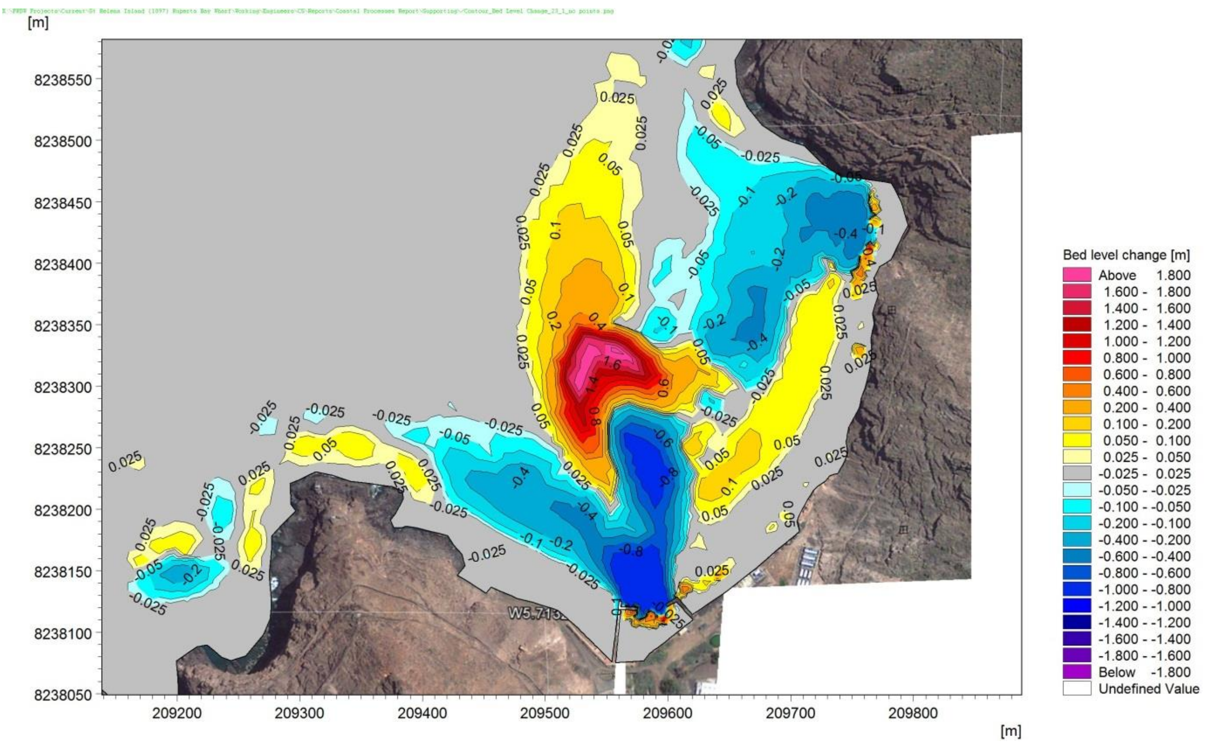


Figure 3-4: Bed level change - status quo (100-year return period event) – North upwards.

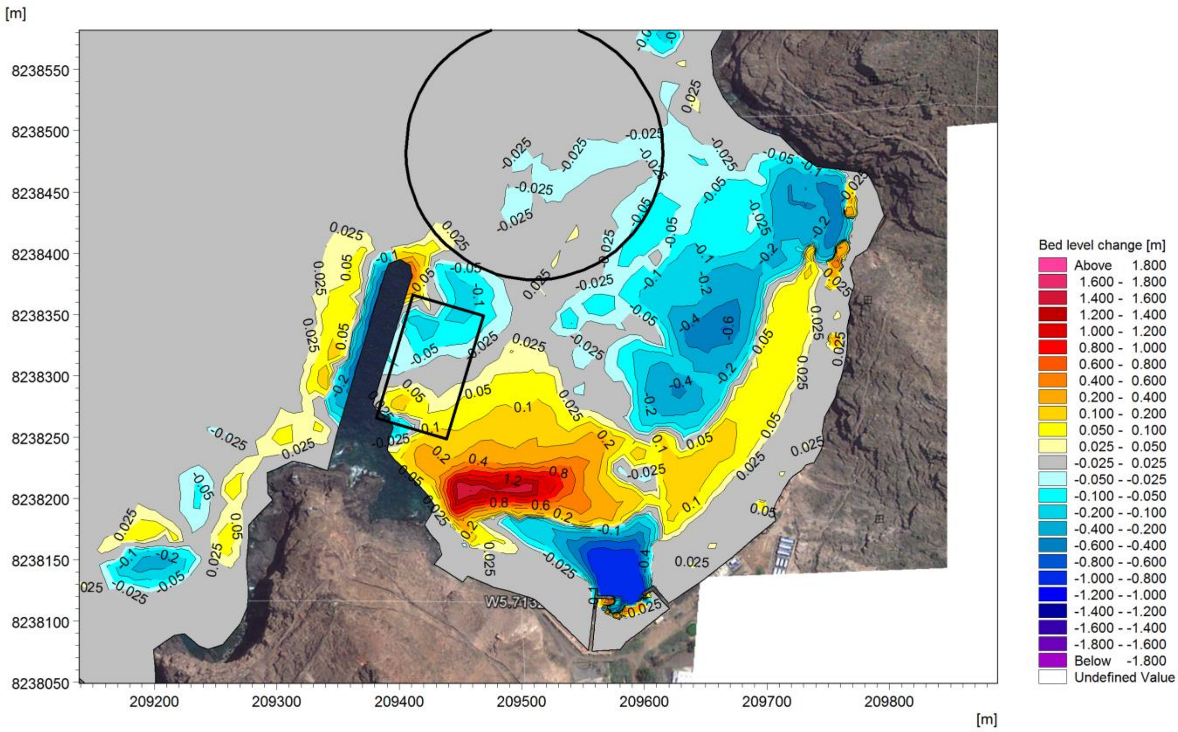


Figure 3-5: Bed level change - Including permanent wharf (100-year return period event) - North upwards.

These changes in the sediment regime within Rupert's Bay can be summarized as follows:

- Sedimentation of the facility's navigational area is predicted to occur during storm conditions only. Minimal accretion is expected during operational conditions, with approximately 0.1 m accretion occurring in the south-eastern corner of the berth pocket during the 100-year storm event. Nevertheless, it is recommended that small dredging equipment be included in the development, to facilitate intermittent dredging as and when required.

- 0.5 m to 1.5 m of sedimentation is expected to occur along the south-western edge of Rupert's Bay. Currently, this region is a rocky reef, which, if covered by sand, may change the marine ecology.

- The implementation of the permanent wharf facility does not significantly change the waves, currents or sediment transport conditions at the south-eastern swimming beach. It has however been shown that the stability of the beach is critically linked to the presence of the concrete pipeline and offshore breakwater. Failure of maintaining these structures will result in the rapid erosion of the swimming beach, irrespective of the proposed development.

3.8 Quarry Rock

Three varying grades of quarry rock were identified from a nearby site in Rupert's Bay valley. The quality of this rock was assessed by carrying out rock density and water absorption tests from representative samples of the three grades of rock.

Rock samples collected at the quarry face after blasting indicated varying rock quality. Unfortunately, reliable rock yields for each of the rock grades was not possible as samples were only taken from the quarry face.

The results of laboratory tests carried out in St Helena Island (dated 15 April 2013) are shown Table 3-4.

Table 3-4: Rock density and water absorption capacity of quarry from Rupert's Bay Valley

Visual description of rock sample	Density [kg/m3] $\bar{x} \pm \sigma$ (*)	Good quality Range [kg/m3]	Water absorption [%] $\bar{x} \pm \sigma$ (*)	Good quality range [%]
Highly dense	2 709 ± 54	2 500 to 2 700	1.23 ± 0.59	0.5 to 2.0
Moderately dense	2 536 ± 113		2.62 ± 1.17	
Porous	2 269 ± 83		4.8 ± 1.65	

Notes: (*): average ± one standard deviation.

Marine grade rock that will be required for the permanent wharf will be governed by a generic rock specification which is largely based on international guidelines and codes. A summary of the average density and water absorption requirements is provided below:

- Average density of rock used for armour or core must be at least 2 600 kg/m3 with 90% of the rocks having a density of at least 2 500 kg/m3. Good quality rock will have a density in the range 2500 to 2700 kg/m3.
- The average water absorption capacity of rock must be less than 2% and the water absorption capacity of nine of the individual rocks less than 2.5%. Good quality rock will have water absorption capacity in the range 0.5% to 2%.

Based on the above requirements and laboratory results:

- Highly dense rocks fulfil density and water absorption requirements, and can be regarded to be of good quality for marine construction.

- The average density of moderately dense rocks is acceptable. However, there is an additional requirement that 90% of all samples should have a density greater than 2500 kg/m3. Of the moderately dense rocks tested, only 80% fulfilled this requirement. In addition, these rocks had an average water absorption capacity of greater than 2%, classifying the rocks as of marginal quality.
- Porous rocks do not meet density or water absorption requirements and its quality is regarded as poor.

Good quality rocks must be used for armour stone, under layers and core. Further assessment on using quarry run as core material for the breakwater construction has been carried out and discussed in Icebreak (2013) (Appendix E). The study shows that quarry run can be used and in terms of rock quality no water absorption limit is required. Marginal quality rocks may therefore be used for the core. Poor quality material shall not be used for the construction of the breakwater.

4. PERMANENT WHARF LAYOUT

4.1 General Layout

The design of the general layout has aimed to provide the most cost effective permanent wharf solution while keeping safety and efficiency of navigation and ship operations paramount. The layout design has as far as reasonably practical aimed to meet the following requirements:

- Sympathetically reflect the coastal landscape
- Avoid any land uptake
- Avoid adverse impacts on Rupert’s beach and amenity area
- Avoid disturbance of the Boer prisoner of war desalination chimney
- Minimise direct effects on Rupert’s lines (the fortification wall)
- Minimise adverse effects on the marine and coastal ecology

The key factors that governed the location and configuration of the permanent wharf layout included:

- Capital and maintenance cost implications
- Degree of shelter and annual down time during adverse wave conditions
- Safety and efficiency of navigation, ship manoeuvring, berthing and unberthing manoeuvres

The general arrangement of the proposed permanent wharf facility is shown on Drawing No. PRDW-900-MN-0003-01 and Drawing No. PRDW-900-MN-0004-01. The main components of the permanent wharf facility include the following marine elements:

- Breakwater
- Main Berth
- Existing temporary Ro-Ro Ramp Facility
- Passenger Landing Facility
- Aids to Navigation
- Relocation of Bulk Fuel Offloading Facility
- Fixed Concrete Boat Ramp for launching Sea Rescue Inflatable Boats (RIBs)

4.2 Permanent Wharf Layout Updates

The port layout developed as part of the previous project stage is presented in the design basis document (Appendix A). This layout has been updated as part of the preliminary design process. The major layout changes are described below:

- Counter-clockwise rotation of the head of breakwater. This change is required to ensure that navigation onto the main quay is not obstructed by underwater revetment slopes.
- A new Ro-Ro facility has been removed from the root of the main quay. The contractor's existing temporary Ro-Ro facility will be retained.
- The lighter berth has been replaced with a passenger landing facility located at the root of the main quay.
- Nominal changes due the size of the pre-cast concrete blocks and the overall block layout plan.

In principle the functional dimensions of the quay structures have been maintained.

4.3 Navigation and Manoeuvring Areas

4.3.1 Design Vessels

The design vessel parameters considered include a vessel with a maximum length overall of 105 m, a maximum beam of 17 m and a maximum draft of 5.5 m. These parameters are shown in Table 4-1.

Table 4-1: Maximum design vessel parameters

Parameters	Value
Dead weight*	6 400 t
Displacement*	7 500 t
Length overall	105 m
Length Between Perpendiculars*	100 m
Beam	17.0 m
Laden Draft	5.5 m

* Inferred

For the purposes of navigation and manoeuvring, a 5 500 DWT multipurpose geared vessel (Dinkeldiep) has been considered based on a review of available shipping currently operating in the South Atlantic off the coast of South and West Africa (PRDW, 2012). The characteristics of this vessel are shown in Table 4-2.

Table 4-2: 5 500 DWT Design vessel parameters

Parameters	Value
Dead weight	5 500 t
Displacement	6 300 t
Length overall (Loa)	106.86 m
Length Between Perpendiculars	100.62 m
Beam	15.2 m
Fully Laden Draft	5.25 m
Depth to main deck	6.6 m
Block Coefficient	0.78
Main Engine Thrust	1 980 kW
Bow Thruster	300 kW
Lateral Windage	600 m^2

4.3.2 Channel Widths

The minimum bottom width of navigation channels will primarily depend on the manoeuvrability of the design vessel and how protected the environment is from the effects of wind, waves and currents. The channel width requirements for the design vessel are based on the guidelines and recommendations from PIANC (1997). The channel width requirements for both the breakwater protected channel width section and the exposed channel seaward of the breakwater are based on the maximum design vessel. The channel width requirements are summarised below in Table 4-3.

Table 4-3: Channel width requirements: protected and exposed channels

Channel Parameter	Protected Channel	Exposed Channel
Design Vessel Beam (m)	17.0	17.0
Channel Width Factor	3.0	3.9
Channel Width (PIANC, 1997) (m)	**51**	**+/- 68**

The channel width requirement for the protected manoeuvring area is 51 m while the channel width requirement for the exposed channel to seaward of the breakwater is approximately 68 m.

4.3.3 Channel Depth Requirements

The channel depth requirements for the maximum design vessel have been calculated using a deterministic method and are summarised in Table 4-4. The protected channel considers the channel adjacent to the berth and within the lee of the breakwater. The unprotected channel considers the outer approach channel where the design vessel will be exposed to wave response motions.

Table 4-4: Channel depth requirements

Zone	Channel Depth Related Factors	Protected Channel Section	Unprotected Channel Section
Nominal Depth Zone (Vessel-related Factors)	Design Draft	5.5	5.5
	Tide (LAT)	+0.1	+0.1
	Vertical Vessel Motion:		
	Swell (Wave Response Motion)	0.5	1.0
	Dynamic List	0.35	0.85
	Squat (Dynamic Trim)	0.25	0.45
	Net Under-keel Clearance	0.6	0.6
	Nominal Depth (Advertised Depth)	**7.3 m***	**8.5 m***
Maintenance Zone (Seabed-related Factors)	Allowance for Sounding Accuracy	0.1	0.1
	Allowance for Siltation	0.1	0.0
	Allowance for Dredging Accuracy	0.0	0.0
	Scour Protection Clearance	0.0	0.0
	Total Channel Depth Requirement	**7.5 m***	**8.6 m***

* Depths are indicated as below Chart Datum

4.3.4 Proposed Turning Circle

The turning circle requirements are based on the guidelines and recommendations from Thoresen (2010). The layout illustrates the proposed position of the turning circle based on the maximum design vessel and a manoeuvring area which should be more protected. The turning circle requirements for a vessel in protected water conditions are defined as 1.8 to 2.0 times the vessel length overall (Loa). The turning circle requirements for the maximum design vessel are summarised in Table 4-5 below.

Table 4-5: Proposed turning circle requirements

Channel Parameter	Maximum Design Vessel
Maximum Design Vessel Loa (m)	105
Turning Circle Factor (Thoresen, 2010)	2.0
Turning Circle Required (m)	**210**

4.3.5 Berth Depth Requirements

The berth depth requirements are based on the guidelines and recommendations from PIANC (1997) and PIANC (1985). The berth depth related factors for the maximum design vessel are calculated using a deterministic method and shown in Table 4-6 below.

Table 4-6: Berth depth requirements

Zone	Berth Depth Related Factors	(m)
Nominal Depth Zone (Vessel-related Factors)	Design Draft (Ro-Ro Vessel)	5.5
	Tide (LAT)	0.1
	Vertical Vessel Motion:	
	Swell (Wave Response Motion)	0.3
	Dynamic List	0.15
	Squat (Dynamic Trim)	0.0
	Out of Trim Allowance	0.25
	Net Under-keel Clearance	0.6
	Nominal Berth Depth (Advertised Berth depth)	**6.9 m***
Maintenance Zone (Seabed-related Factors)	Allowance for Sounding Accuracy	0.1
	Allowance for Siltation	0.0
	Allowance for Dredging Accuracy	0.0
	Scour Protection Clearance	0.0
	Total Berth Depth Requirement	**7.0 m***

* Depths are indicated as below CD.

As shown in Table 4-6, the berth depth required for the design vessel is -7.0 m CD which considers +0.1 m for the allowance of the lowest astronomical tide (LAT). Some allowance for list and trim has been made in this report revision for a more robust underkeel clearance value.

4.3.6 Manoeuvring Operations – Permanent Wharf Structure

Manoeuvring operations for the Permanent Wharf Structure consider both the arrival and sailing manoeuvres in Rupert's Bay with the 5 500 DWT design vessel. The vessel arrival and sailing manoeuvres will consider manoeuvring to and from the proposed berth port side and starboard side alongside. Considering both options provides flexibility to the overall port operations within Rupert's Bay, however, the port side berthing manoeuvre is considered to be more limiting of the two arrival manoeuvres and has been considered in more detail in this desktop study.

It is assumed that vessel operations will continue similar to the existing operations in Rupert's Bay in that a pilot will not board the vessel, but an experienced vessel's master will carry out the vessel manoeuvring operations. It is further assumed that all vessels using the proposed wharf will be required to self-berth without the aid of tugs. It should be noted that a bow thruster is considered essential when navigating without tug assistance. The 5 500 DWT design vessel has a bow thruster capacity of 300 kW or 4 t bollard pull.

4.3.6.1 Vessel Arrival Manoeuvre – 5 500 DWT Design Vessel

The arriving vessel, in most cases, will either be fully laden or in a nearly fully laden condition. The manoeuvre will consist of transiting the approach channel, turning to port within the turning circle and backing into the berth port side alongside.

Initially the vessel will be heading on a course of approximately 135° (TN) towards Rupert's Bay at a speed of approximately five to six knots directly towards the centre of the turning circle. The vessel will be reducing speed continuously until it reaches the turning circle. The vessel will use a combination of bow thruster, hard port rudder and vessel's engine thrust to complete the turn within the turning circle. The vessel will be affectively turning short round which implies short bursts of both forward and astern engine thrust in order to manoeuvre the vessel round within a limited manoeuvring area. Engine power is reduced before the vessel's longitudinal inertia is overcome and the vessel begins to accelerate.

Although typically vessels are turned short round to starboard, this manoeuvre will be more efficiently carried out turning the bow to port as the wind blowing offshore will assist the manoeuvre towards the berth. The engine movement astern will produce a movement of the bow to starboard owing to transverse thrust but it will assist in manoeuvring the stern towards the berth. As the vessel gathers sternway and the pivot point moves further aft, with the momentum of the vessel, the bow thruster can be used for steerage and to counteract excessive transverse thrust. As the vessel approaches the berth at an angle of less than 10 degrees, the speed will be less than 1 knot and the lateral speed will be increasing as the longitudinal speed decreases.

The vessels engine, rudder and thrusters will be controlled in order to manoeuvre the vessel into the berth. These resources should be controlled in order to ensure that the vessel remains parallel to the berth and with minimal longitudinal and lateral speed. This is necessary to avoid potential damage to the fenders and/or the quay structure. When the vessel is close enough to the quay, the mooring lines can be sent ashore in order to further assist manoeuvring into the berth. Once the vessel is in position and alongside, the mooring lines can be secured and the vessels gangway can be landed.

The vessel arrival manoeuvre on to the permanent wharf structure is illustrated in Figure 4-1.

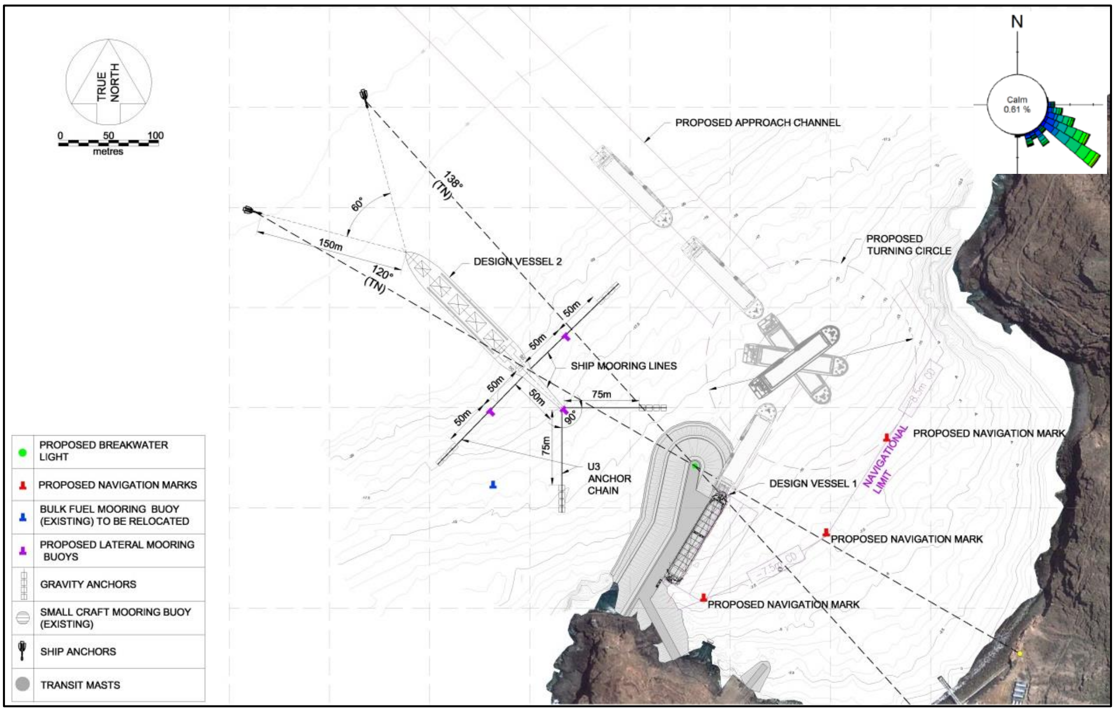


Figure 4-1: Typical vessel arrival manoeuvre with wind rose.

4.3.6.2 *Vessel Sailing Manoeuvre - 5 500 DWT Design Vessel*

A typical sailing manoeuvre for a vessel berthed port side alongside would commence with the letting go of the vessel's mooring lines from the shore. In all likelihood the vessel will spring-off the berth. This is described as manoeuvring ahead with the vessel's engine against the forward spring in order to kick the stern of the vessel out. This may be accelerated by using the vessels engines moving ahead briefly against a port rudder.

Once the vessels stern is clear of the berth, the vessel can manoeuvre using the engine moving ahead against the starboard rudder and making use of the vessels bow thrusters to counter the swing of the stern towards the quay wall structure. The vessel will slowly manoeuvre towards the centre of the turning circle. This will ensure that it sufficiently and safely clears both the quay and the breakwater structures during the manoeuvre.

As the vessel gets closer to the centre of the turning circle, the vessel will utilise its engine and port rudder in order to commence a swing to port to a heading of approximately 315°(TN). Once the vessel has completed the turn to port, the vessel can start increasing engine speed in order to clear the bay.

4.3.7 Berthing and Manoeuvring Assistance – Permanent Wharf Structure

Presently only a mooring launch provides berthing assistance in Rupert’s Bay for tankers that call at the existing bulk fuel installation. It is assumed that no additional berthing or manoeuvring assistance vessels will be required for the manoeuvring operation at the permanent wharf structure. This assumption is considered reasonable given the navigation geometry selected and mild wave and weather conditions. This will however be verified during the next stage when a ship manoeuvring study will be undertaken.

4.3.8 Navigation Aids - Permanent Wharf Structure

Navigation aids are in general required to reduce marine risk. This is done by permanently demarcating specific navigation areas such as navigation channels as well as locations that may cause an obstruction to navigation within the port and approaches. A preliminary assessment of the required aids to navigation for the permanent wharf structure has been completed. The required navigation marks for the permanent wharf structure include three lateral marks and a breakwater light. The navigation requirements are illustrated in Drawing No. PRDW-900-MN-0004-01.

4.3.9 Limiting Conditions for Vessel Manoeuvring Operations

The limiting operational conditions typically adopted for vessel navigation and manoeuvring (stopping and turning) are based on factors such as the available manoeuvring assistance in the form of tugs, the quay fender system and the manoeuvrability of the vessel itself.

ROM (2003) defines the limiting wind condition for vessel manoeuvring as between 10 m/s and 17 m/s (one-minute average at 10 m above sea level), for transverse and longitudinal directions, Based on the orientation of the main quay structure and the dominant wind direction (SE'ly), the dominant wind will be acting on the vessel in a direction 14° abaft the beam. Abaft the beam refers to a relative bearing greater than 90° from the bow. A one-minute averaged 11 m/s (resultant vector) limiting wind condition has therefore been considered. This is equivalent to a 8.8 m/s one-hourly average mean wind velocity at a height of 10 m above sea level with an annual exceedance of 10%.

The maximum wind force exerted on the vessels beam at any stage during a berthing or sailing manoeuvre will need to be overcome by a combination of the vessel's bow thruster, engine and rudder. This maximum wind force is calculated by using an industry standard formula:

$$\mathbf{K_{\mathit{wind}} = k \times A \times V^2}$$

K_{wind} = Maximum wind force (t)

k = 0.52×10^{-4}

A = Vessel wind area (m^2)

V = Wind speed (m/s)

The maximum wind force calculated based on the design vessel wind area (600 m^2) and the limiting wind speed (11 m/s) is 3.7 t. The 5 500 DWT design vessel has a bow thruster capacity of 300 kW or 4 t bollard pull, which is sufficient to overcome the maximum wind force. This does not consider the additional assistance from the combined effect of using the bow thruster with the vessel's engine and rudder.

The centre of the turning circle is aligned with a line running near the centre of the vessel manoeuvring area. Although limited, this would indicate that there is sufficient manoeuvring area available for either a vessel backing into the berth (port side alongside) or for a vessel manoeuvring bow-first into the berth (starboard side alongside).

The limiting wave condition being considered is a significant wave height of 1 m. As presented in the Coastal Processes Report (PRDW, 2013b), this wave height has an annual exceedance of 6%. The influence of the wave period on the design vessel will be considered in the ship manoeuvring study as this may affect vessel manoeuvrability significantly.

4.3.10 Manoeuvring Downtime

The downtime calculations presented here are based on overlapping wind and wave data only. Table 4-7 below provides information on the monthly data coverage and percentage coverage where both wind and wave data are available over a period of approximately six (6)years (December 2006 to September 2012). The figure indicates low data coverage for this dataset, which should be taken into account when interpreting the results of the downtime assessment.

Downtime is calculated by counting the number of occurrences where either the wind or wave limit is exceeded. For the proposed layout, this corresponds to a significant wave height (H_{m0}) of 1 m and an hourly average wind speed of 8.8 m/s.

Table 4-7: Monthly data coverage of overlapping wind and wave data (December 2006 and September 2012)

% DATA COVERAGE								
Months	Year							Monthly Average
	2006	2007	2008	2009	2010	2011	2012	
January	0%	11%	100%	60%	0%	99%	0%	45%
February	0%	85%	99%	44%	0%	97%	0%	54%
March	0%	76%	97%	28%	0%	31%	0%	39%
April	0%	99%	99%	0%	0%	0%	0%	33%
May	0%	32%	15%	0%	0%	17%	0%	11%
June	0%	0%	0%	0%	0%	95%	0%	16%
July	0%	65%	0%	0%	0%	94%	0%	27%
August	0%	97%	80%	0%	0%	11%	28%	36%
September	0%	31%	94%	0%	0%	0%	99%	33%
October	0%	0%	57%	0%	0%	77%	0%	27%
November	0%	71%	31%	0%	0%	31%	0%	27%
December	100%	98%	44%	0%	59%	0%	0%	49%
Annual Average	8%	55%	60%	11%	5%	46%	11%	33%

Table 4-8 shows wind, wave, a combined wind and wave and the overall average of the combined downtime. Assuming that the above combined data coverage is representative, an average of 17% downtime can be expected in a typical year.

It is assumed reasonable for a vessel to wait up to 3 hours before entering Rupert's Bay. Excluding downtime events of this duration reduces average downtime from 17% to 13%. The maximum downtime for November reduces from 35% to 26%. Of the remaining downtime events 81% will have a duration of less than 12 hours. If it is assumed that downtime events of less than 6 hours is an acceptable delay for berthing, the average downtime reduces to 10% and the maximum downtime for November reduces to 22%.

Table 4-8: Monthly ship manoeuvring downtime estimates based on wind and wave data

% COMBINED WIND AND WAVE DOWNTIME								
	Year							Monthly
Months	2006	2007	2008	2009	2010	2011	2012	Average
January		10%	8%	4%		26%		14%
February		30%	8%	1%		28%		19%
March		9%	16%	0%		33%		14%
April		15%	2%					8%
May		12%	0%			9%		9%
June						4%		4%
July		34%				6%		17%
August		35%	20%			5%	28%	27%
September		58%	20%				20%	27%
October			27%			12%		19%
November		34%	8%			62%		35%
December	6%	9%	4%		17%			9%
Min	6%	9%	0%	0%	17%	4%	20%	4%
Mean	6%	24%	12%	2%	17%	18%	23%	17%
Max	6%	58%	27%	4%	17%	62%	28%	35%

4.4 Moored Vessel Motions

This section provides a summary of the availability of the permanent wharf facilities in regards to typical vessel loading and unloading. Wave penetration modelling was performed in order to inform a high level downtime assessment based on guideline limiting wave height criterion. For more detail, the reader is referred to the Vessel Motions Report (PRDW, 2013c) provided in Appendix C.

4.4.1 Wave Penetration Modelling

Wave conditions at the berth were determined by forcing the boundary of a Boussinesq model with discrete events linked to a yearly occurrence. Regional wave modelling as detailed in the Coastal Processes Report (PRDW, 2013b) provided the operational time series of wave parameters at the boundary of the Boussinesq model. Table 4-9 shows the percentage occurrences of the operational wave climate discretized at the boundary of the Boussinesq model.

The percentage boxes highlighted in blue represent the 15 discrete model cases that were run; these were defined as the bins with the highest percentage occurrence. The bin sizes for the discrete cases are +/-0.25 m, +/-10˚ and +/-2 s about the centre of the bin for the significant wave height (H_{m0}), peak wave direction (PWD) and peak period (T_p) bins respectively. In order to decrease the number of model runs, in certain cases wave occurrences were binned conservatively, for instance any incident waves propagating from less than 270˚ were added to the 270˚ bin.

Table 4-9: Discrete event selection and percentage occurrence

H_{m0} (m)	T_p (s)	PWD (˚)					Percentage Occurence in wave bin (%)
		270	290	310	330	350	
0.5	9	5.6	0.0	0.0	0.0	13.0	93.5
	13	18.8	0.0	5.3	47.7	0.0	
	17	0.2	0.0	0.0	2.9	0.0	
1	9	0.0	0.0	0.0	0.0	0.0	6.2
	13	0.3	0.0	1.0	3.6	0.0	
	17	0.0	0.0	0.4	0.9	0.0	
1.5	9	0.0	0.0	0.0	0.0	0.0	0.3
	13	0.0	0.0	0.3	0.0	0.0	
	17	0.0	0.0	0.0	0.0	0.0	
					Total percentage		100

Figure 4-2 to Figure 4-3 show disturbance coefficients for typical wave conditions. The series of figures show the effect that a change in PWD (270° to 310° to 330°) on the boundary has on wave agitation in the bay, and by keeping all other model parameters similar. The expected trend is evident showing the wave agitation coefficient adjacent to the main berth increasing from approximately 0.5 to 0.6 to 0.8 as the wave train has a more direct path into the bay and behind the breakwater.

Figure 4-4 and Figure 4-5 show the effect that an increase in wave period has on the wave agitation coefficient in the bay. Generally the bay experiences slightly higher agitation particularly on the eastern side due to increased refraction, characteristic of a higher period wave.

In general the figures show strong shoaling in the centre of the bay and nodal patterns created by reflected waves forming a standing wave pattern across the basin.

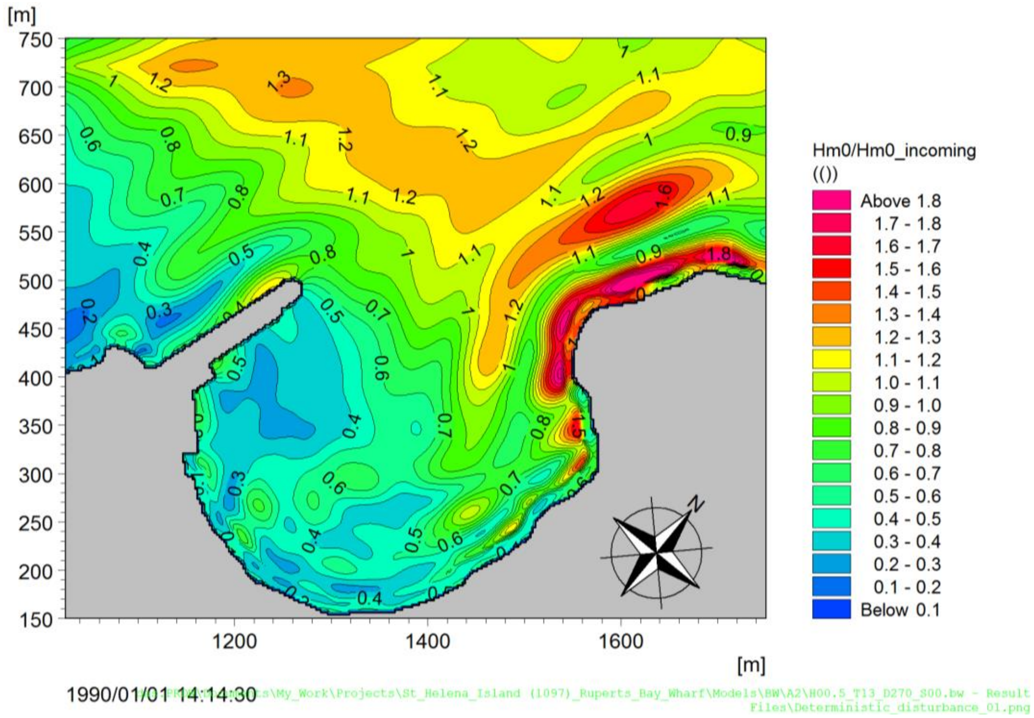


Figure 4-2: Disturbance coefficients for typical wave conditions, PWD = 270˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 18.8 %.

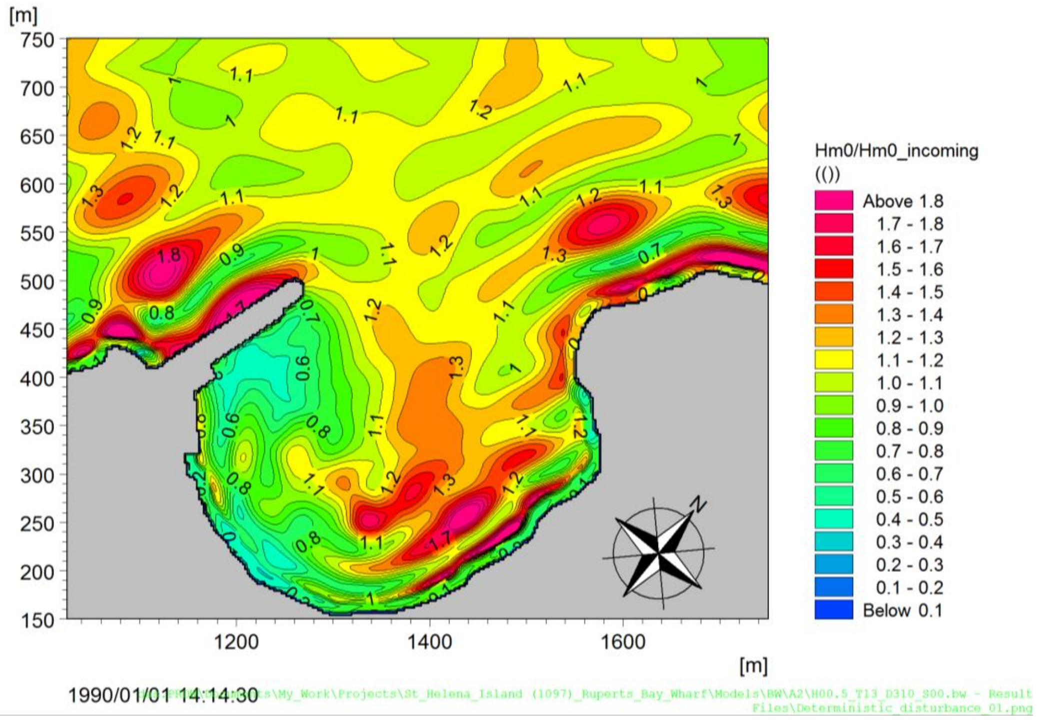


Figure 4-3: Disturbance coefficients for typical wave conditions, PWD = 310˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 5.3 %.

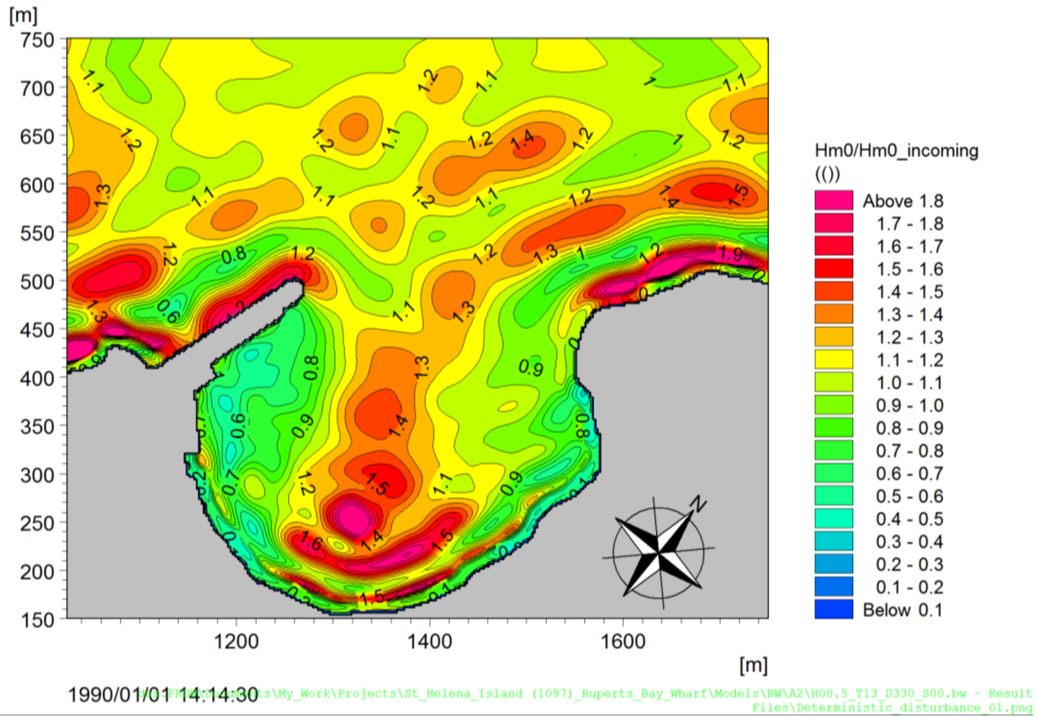


Figure 4-4: Disturbance coefficients for typical wave conditions, PWD = 330˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 47.7 %.

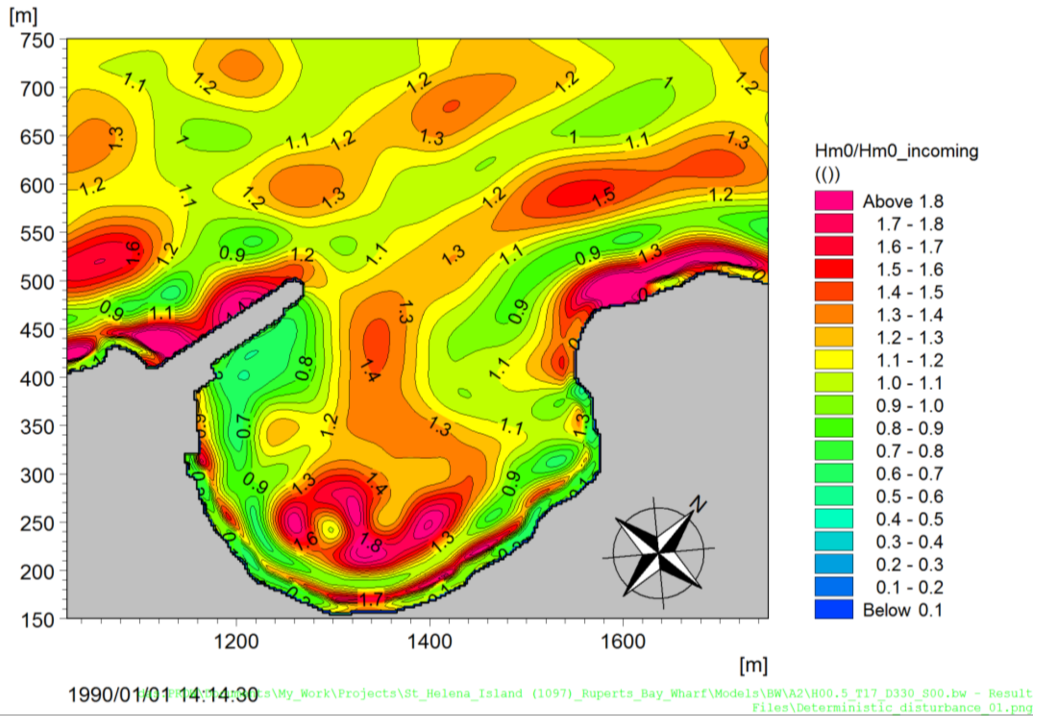


Figure 4-5: Disturbance coefficients for typical wave conditions, PWD = 330˚, H_{m0} = 0.5 m, T_p = 17 s. Yearly occurrence = 2.9 %.

Results from the BW model have been used in determining the H_{m0} at the berth for the assessment of downtime.

4.4.2 High Level Downtime Assessment

Preliminary investigations for the assessment of downtime have focused on the exceedance of limiting significant wave heights at the berth. This high level approach considers only the significant wave height at the berth and a general description of the direction of wave attack on the vessel. Limiting wave criteria are obtained from the handbook on port design (Thoresen, 2010). The limiting criteria are provided in Table 4-10 for a general cargo vessel and a Ro/Ro vessel.

Table 4-10: Limiting criteria on significant wave weights

Vessel Type	Limiting wave height H_{m0} in meters	
	0° (head-on or stern-on)	45 to 90°
General Cargo	1.0	0.8
Ro/Ro	0.5	0.3

Availability of the berth has been determined based on the calculated wave heights (6 hourly averages) shown in Table 4-11 below.

Table 4-11: Wave conditions (6 hourly averages) and percentage occurrence

Offshore conditions			Conditions at berth			Percentage occurrence [%]
H_{m0} [m]	T_p [s]	PWD [° TN]	H_{m0} Total (m)	H_{m0} Incident (m)	H_{m0} Reflected (m)	
0.5	9	350	0.34	0.24	0.24	13.0
0.5	9	270	0.17	0.12	0.12	5.6
0.5	13	330	0.33	0.24	0.24	47.7
0.5	13	310	0.29	0.21	0.21	5.3
0.5	13	270	0.24	0.17	0.17	18.8
0.5	17	330	0.34	0.24	0.24	2.9
0.5	17	310	0.32	0.23	0.23	0.0
0.5	17	270	0.20	0.14	0.14	0.2
1.0	13	330	0.71	0.50	0.50	3.6
1.0	13	310	0.67	0.47	0.47	1.0
1.0	13	270	0.55	0.39	0.39	0.3
1.0	17	330	0.76	0.54	0.54	0.9
1.0	17	310	0.70	0.49	0.49	0.4
1.5	13	310	1.17	0.83	0.83	0.3
1.5	17	330	1.19	0.84	0.84	0.0
1.5	17	310	1.07	0.75	0.75	0.0

Percentage downtime has been calculated using the significant wave heights and related percentage occurrence provided in Table 4-11 and comparing these values to the limiting criteria for a general cargo vessel and a Ro/Ro vessel (refer to Table 4-10). The total availability is then calculated simply as the accumulation of the percentage occurrence where the reflected significant wave height is less than the limiting criteria for beam (45 to 90°) waves, as this is the critical condition. This is tabulated below for the general cargo vessel and the Ro/Ro vessel.

Table 4-12: High level downtime assessment for moored vessel motions

Vessel Type	Availability [%] Based on 6 Hourly Average Wave Heights
General Cargo	100%
Ro/Ro	94%

As analysed in the Coastal Processes Report (PRDW, 2013b) there is evidence of highly variable wave groups at the site which can cause the significant wave height to vary significantly from one hour to the next.

The implication of short term variability needs to be considered in assessing downtime estimates. Wave heights at the berth can be expected to vary from the average 6 hourly heights used in assessing downtime.

4.4.3 Berth Availability

Availability of the berth has been calculated based on the above calculated wave heights. These preliminary calculations show a high availability for both the general cargo (100% availability) and the Ro/Ro (94% availability) vessels. Short term variability in wave heights may lead to short periods over which loading would be difficult and inefficient. While it is not envisaged that vessels will have to leave the berth due to this short term variability, the loading inefficiency could be interpreted as downtime which would reduce the estimated availability slightly.

Concerns have also been raised that the limiting criteria for wave heights may be too high based on the predominant wave period for the site, and thus the availability may be lower than given above. It is recommended that vessel motion modelling studies be undertaken in order to quantify the availability in terms of critical limiting motions. Such a study will also enable an evaluation of the combined effect of head on and beam on conditions which was not possible with the present approach.

4.5 Bulk Fuel Offloading Facility Re-location

The bulk fuel offloading facility is proposed to be relocated from its existing location. This will ensure maximum availability of the berthing facilities at the wharf while a tanker is discharging fuel. The Contractor has not included this full solution in his pricing but supplied 'rate only' items to cater for such. A final solution between Contractor and Employer will need to be agreed.

4.5.1 Design Vessel

The bulk fuel offloading facility in Rupert's Bay comprises a mooring buoy, a fuel line buoy and a floating hose suspended from a shore hose gantry system. The design vessel for the facility considers Handy Size tankers which provide fuel supply to the Island. The characteristics of the design vessel are shown in Table 4-1.

Table 4-13: Design vessel characteristics – Bulk Fuel Offloading Facility

Parameters	Value
Length overall	170 m
Beam	25.6 m
Laden Draft	11.0 m

4.5.2 Re-location of Offloading Facility

It has been considered as part of the design of the permanent wharf structure to relocate the mooring buoy system. It is proposed that the buoy mooring system be relocated seaward of the proposed breakwater structure.

The mooring buoy which comprises of three anchor legs can be positioned approximately 75 m from the breakwater head. The mooring legs can be consolidated into two mooring legs anchored to the sea-bed by gravity anchors. In order to ensure that sufficient lateral support is provided to the mooring buoy system, two additional mooring buoys will be installed perpendicular to the stern of the tanker in order to moor the vessel's breast lines.

As is the case with the existing installation, the floating hose connection will be made on the port side manifold of the tanker. The arrangement of the mooring for the relocated mooring buoy system is illustrated in Drawing No. PRDW-900-MN-0004-01. The relocation of the fuel offloading facility will allow both the bulk fuel terminal and the proposed wharf structure to operate independently and will reduce the navigation risk to both terminals.

The preliminary design includes all marine aspects related to the mooring system but excludes the system hydraulics from the vessel to the quay and then to the landward storage facility as required for a fully functioning bulk fuel installation.

4.5.3 Manoeuvring Operation – Offloading Facility

The manoeuvre of the tanker on to the buoy mooring system will be similar to that of the existing operation with the exception that it will now be relocated to a position with a greater water depth.

The arriving tanker will approach Rupert's Bay on a heading of approximately 225°(TN). When the breakwater light is bearing approximately 138°(TN) the starboard anchor can be let-go. It is recommended that a lit mast in transit with the breakwater light be installed. This will assist as a reference point should there be no conspicuous landmarks suitable for a transit. The vessel will continue on the same track of 225°(TN) until the breakwater is bearing 120°(TN) at a range of approximately 450m. In this position the vessel will let-go the port anchor. Again a mast installed in transit with the breakwater will assist vessels with the position to drop the vessels anchor. Once the anchor has been deployed the vessel will then swing around to a north westerly position in order to moor the stern to the mooring buoy. The stern of the tanker will be moored approximately 50 m from the mooring buoy using four stern lines from the vessel. The vessel will then moor breast lines to the breasting mooring buoys in order to provide lateral positioning within the mooring. The vessel arrival manoeuvre on to the bulk fuel offloading facility is illustrated in Figure 4-6.

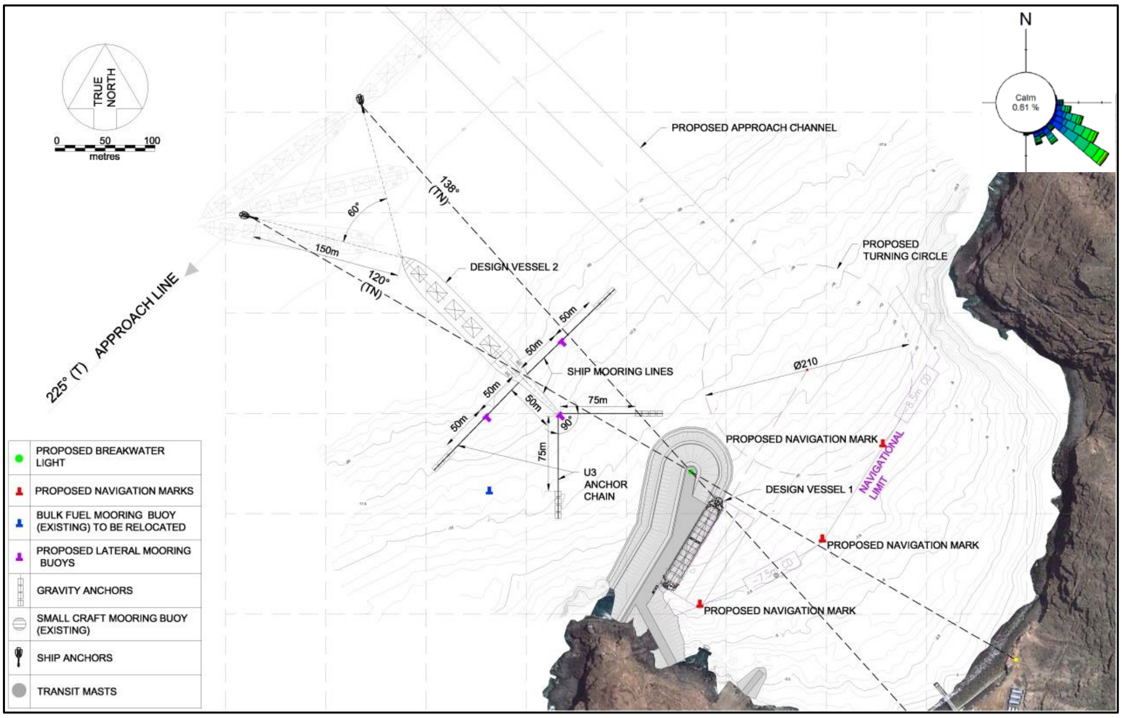


Figure 4-6: Typical bulk fuel vessel arrival manoeuvre with wind rose.

4.5.4 Berthing and Manoeuvring Assistance – Bulk Fuel Tanker

Presently a mooring launch provides berthing assistance to tankers mooring to the offloading facility in Rupert’s Bay. It is assumed that this operation will continue and that no additional berthing or manoeuvring assistance craft will be required for the operation of the offloading facility.

4.5.5 Navigation Aids – Bulk Fuel Offloading Facility

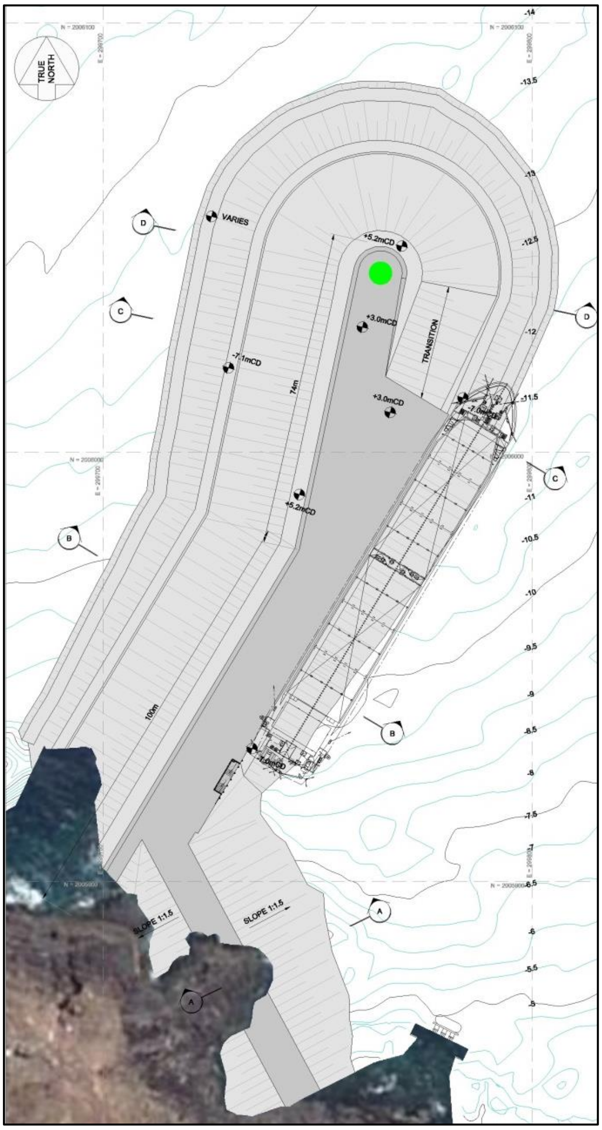
A preliminary assessment of the required aids to navigation for the offloading facility has been completed. The required navigation marks for the facility are a recommendation only as described in Section 4.3.8. It is recommended that two masts in transit with the breakwater light be installed so as to assist tankers with reference marks to drop the vessel’s anchors for manoeuvring on to the mooring buoy system. These masts are illustrated in Drawing No. PRDW-900-MN-0004-01.

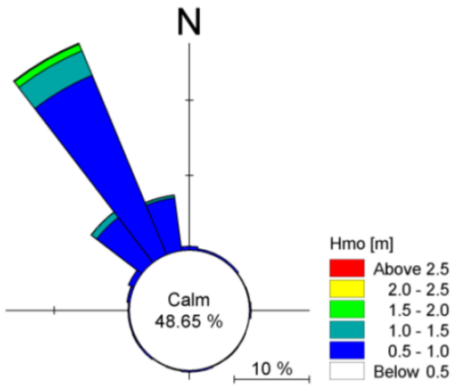
5. MARINE STRUCTURES

5.1 Breakwater Structures

5.1.1 Layout

The breakwater layout is shown in Figure 5-1 together with the wave rose measured by an ADCP instrument. A more easterly orientation would have provided better shelter from the dominant wave direction. This was not possible given the spatial requirements for safe navigation. The kink at the head of the structure is required to avoid the lee slope from extending into the navigation area.





Representative inshore wave rose (ADCP measurements). The most dominant wave direction (30^{o} from N) forms angles of 28^{o} and 47^{o} respectively with the head and root sections of the breakwater.

Figure 5-1: Breakwater layout and wave rose.

5.1.2 Sections

The breakwater section at the head of the structure is shown in Drawing No. PRDW-900-MN-0005-01-A. The seaward slope, shown in more detail in Figure 5-2, has the same detail over the full length of the breakwater.

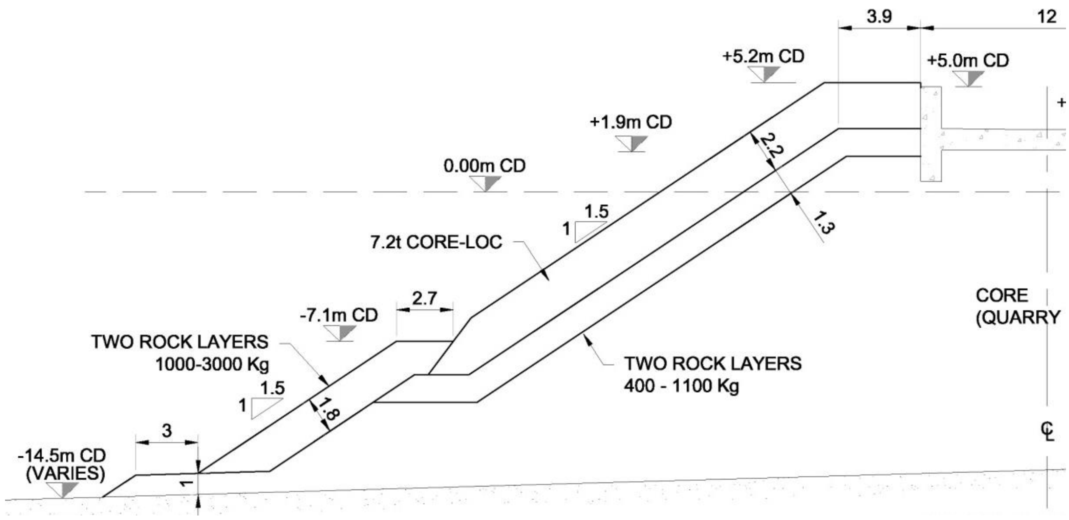


Figure 5-2: Breakwater seaward slope.

The breakwater trunk section has been tested in a two dimensional flume model using 5t core-loc armour units (CSIR, 2013) – Appendix D. At the completion of 2D model study a decision to increase the size of armour unit from 5t to 7.2t was taken as it would decrease the overall number of units that would need to be manufactured and placed leading, to overall cost savings and risk reduction.

The leeward slope details are shown in Figure 5-3. This leeward Core-loc section extends over approximately only 20 m.

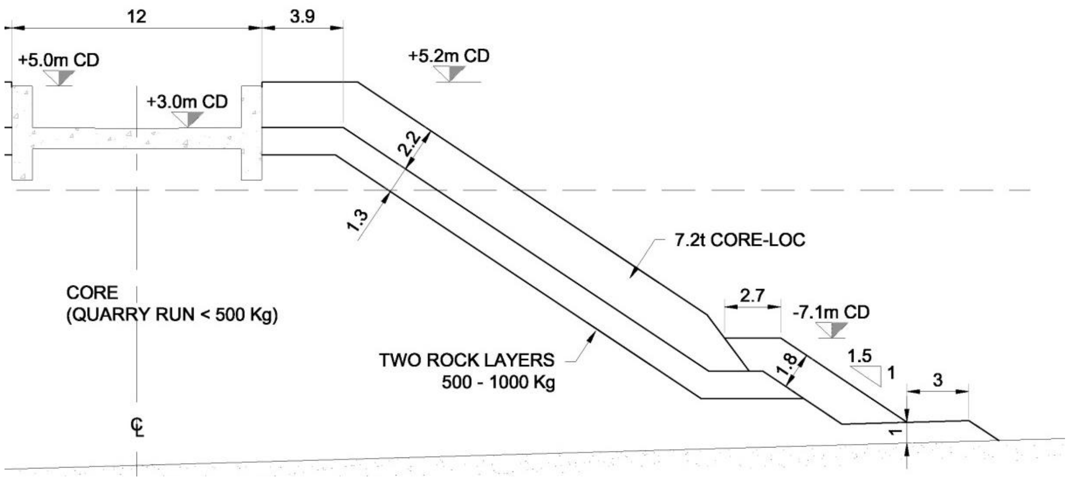


Figure 5-3: Breakwater leeward slope.

A transition from a slope of 1:1.5 to 3:4 takes place on the lee of the head as shown in Figure 5-4.

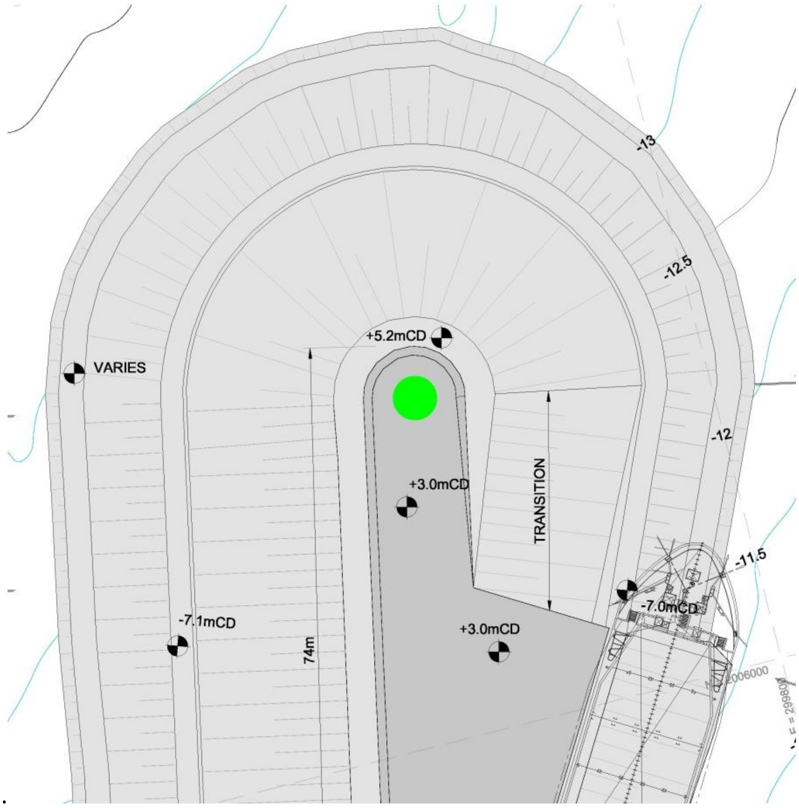


Figure 5-4: Breakwater head layout.

5.1.3 Breakwater Trunk Design

The results of the two dimensional flume model study which tested the breakwater trunk section is described in this section.

5.1.3.1 Seaward Slope Stability

Stability of the seaward slope was confirmed for significant wave heights up to 4.8 m which exceeds the design wave height of 4.6 m (PRDW, 2013b). During the tests some damage occurred on the crest of the Core-loc slope indicating the need for careful placement of units at the intersection of the 1:1.5 slope and the horizontal crest.

5.1.3.2 Overtopping

Significant overtopping was measured for larger wave heights as shown in Figure 5-5 (PRDW, 2013d).

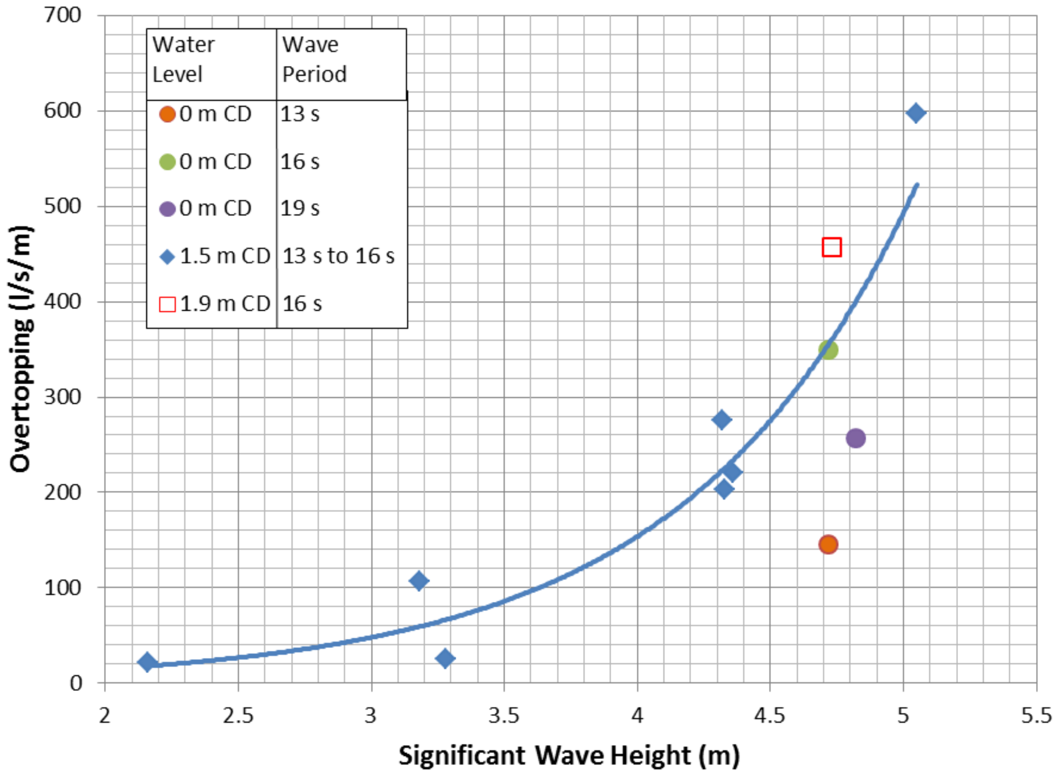


Figure 5-5: Overtopping versus significant wave height.

Drainage of all overtopped water is not considered practical. For significant wave heights around 3 m, only about 10 per cent of waves actually overtop the structure. The overtopping of individual waves lasts only a few seconds which means that the instantaneous overtopping rate can easily exceed 20 times the average overtopping rate. Any system aimed at capturing and draining the resulting instantaneous flow rates of 600 l/s/m or more will require a significant leeward wave wall which is not considered practical. A more practical approach to avoiding pollution from the breakwater cap is to ensure that it is cleaned regularly and to allow overtopping water to spill over the lee edge of the cap.

Overtopping has an implication for loading and offloading, especially if cargo has to remain completely dry. To assess the percentage of time that overtopping will occur an exceedance graph of measured ADCP wave heights has been produced and is shown in Figure 5-6. It is estimated that a significant wave height of 1.6 m at MHWS would not result in overtopping with a noticeable effect on loading or offloading. This is exceeded only 1 per cent of the time so that the number of days during which loading or offloading may be affected by wave overtopping is estimated at no more than 4 days per year.

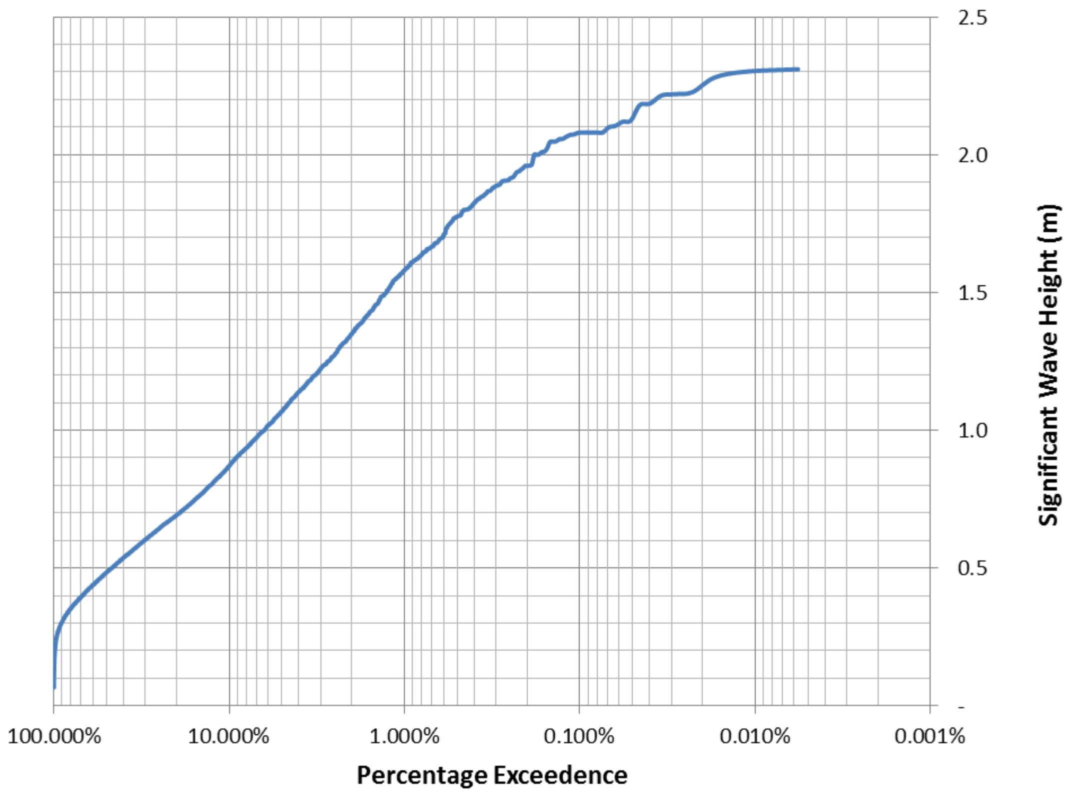


Figure 5-6: Exceedance of measured ADCP wave data.

5.1.3.3 Leeward Slope Stability

It should be noted that damage due to overtopping would be overestimated in the flume model where the actual oblique wave attack cannot be simulated. In addition, the modelled cap width was 6.5 m compared to the final cap width of 12.4 m (the width increase is required for roundhead stability which was established after the model tests had been completed). Results should therefore be interpreted with care and final tests in a three dimensional model will be required for the detail design.

For the tests with high water levels wave overtopping projected approximately 10 m leeward of the deck, damaging the toe berm located at -4.3 m CD (original toe level). This level was therefore dropped to -7 m CD. During the low water level conditions, wave overtopping projected onto the deck and flowed over, washing out the top row of Core-loc units from the slope. This resulted in progressive damage of Core-locs as consecutive rows were exposed to direct impact from overtopping water.

A design change is required to avoid the risk of lee damage. Core-loc units in the top row should either be sheltered from direct wave overtopping flows or anchored to the deck.

Splitter blocks combined with an inclined leeward wave wall can also be used to project overtopping waves further leeward which will reduce wave impact on rear Core-loc units.

For the present design, anchoring the top row of Core-locs to the deck was considered. This can be achieved by casting the units into the concrete deck as shown in Figure 5-7.

Three dimensional tests in which oblique wave attack is modelled accurately are required for the final design. These tests may prove that lee stability is sufficient without the recommended stabilization of top row units; however, it is recommended that provision for such stabilization be included in the preliminary design cost estimates.

Figure 5-7: Breakwater cap with Core-loc units cast into the deck (Podoski, 2012).

5.1.4 Roundhead Stability

Preliminary guidelines for Core-loc stability indicate Hudson stability factors (K_d) of 16 and 13 for the trunk and head, respectively. This implies a 23% increase in required mass on the roundhead. Without test results from a 3D model, a preliminary way of estimating whether the 5 tonne Core-locs will be stable on the head is to test these units on the trunk section with an increased wave height. A 23% increase in K_d is equivalent to a 7% increase in wave height ($1.23^{1/3}$). If units on the trunk are therefore stable for a wave height of 4.9 m, the same units should be stable for a 4.6 m wave on the roundhead.

Test B8 (PRDW, 2013d) was close to a wave height of 4.9 m (4.82 m) without any extractions or movements in excess of 0.5 C. In Test A3 (PRDW, 2013d) the wave height was increased to 5.05 m. One extraction on the crest was experienced for this wave condition. Typically damage is concentrated around the still water level and damage to the crest is associated with overtopping and potentially poor control over placement. Based on these test results the proposed Core-loc mass of 5 tonne is considered reasonable.

For the detailed design three dimensional model tests will be required to confirm stability of aspects not covered in two dimensional tests.

5.1.5 Breakwater Root

Detail bathymetry is required of the cliffs at the root of the breakwater to define a practical tie-in of the rock toe and Core-loc slope into the land. A provisional amount is to be provided for this tie-in in the preliminary cost estimates.

5.1.6 Construction Materials

5.1.6.1 Rock

Required rock quality parameters are described in the General Rock Specifications for Rubble Mound Structures (PRDW, 2013e)

5.1.6.2 Concrete

The following will apply with regard to concrete material:

Armour units:

- Concrete armour units will not be reinforced.
- Quality will be based on the Maritime Structures Code BS 6349.

Crown wall:

- If steel-reinforced concrete is used for the crown wall, it will be designed for maximum durability under "very severe" exposure conditions as defined in BS8110.
- The minimum 28-day strength of all reinforced concrete shall be 40Mpa, and the minimum cover to all steel reinforcement shall be 75 mm.
- In areas of high abrasion, the cover to the reinforcement shall be 100 mm. All reinforced concrete shall be wet-cured with fresh water for a minimum period of 10 days after casting.

5.2 Quay Structures

5.2.1 Description

The proposed Rupert's Bay permanent wharf requires the construction of the following quay structures:

- Main Berth 97.5 m long, 7.0 m minimum berth depth
- Passenger landing facility, 3.0 m minimum berth depth
- RIB Boat Ramp adjacent to the existing fisherman's wharf

A general arrangement of the above quay structures is provided in Drawing No. PRDW-900-MN-0006-01.

Based on the previous concept stage the preferred quay structure for the main berth was a precast reinforced concrete rectangular hollow block design. The preliminary design work undertaken as part of this study has optimised and refined this concept solution.

5.2.2 Main Berth Design

The structural details of the main berth is described below and shown in Drawing No. PRDW-900-MN-0007-01 and PRDW-900-MN-0010-01.

The typical section of the quay structure consists of a base block plus 9 precast hollow blocks placed in a vertical stack. The base block is placed on a 1.0 m thick stone foundation bed screeded to a level of -8.0 m CD and the precast concrete blocks are stacked on top each other to a level of +1.6 m CD. Stacking the blocks to +1.6 m CD and the use of steel crane mats will allow the construction plant and equipment to operate along the top of the units with 1.5 m of freeboard above Highest Astronomical Tide (HAT). The +3 m CD capping level as shown on the original reference design drawing has been maintained and will be reviewed during the detailed design stage.

The base unit has been optimised with voids in the base slab to reduce unit weight and facilitate placing at maximum reach of the crane. In addition the base unit also utilises vertical interlock nibs that improve the placing efficiency and accuracy of the first (bottom) block. The blocks have been detailed with lateral nibs that interlock and perform a dual function of ensuring transverse block connection as well as acting as guides for placing of adjacent blocks.

The addition of grout bags is proposed as an additional stability measure in the construction condition. This will significantly reduce movement and help to stabilize the wall until the capping is installed. The vertical block stacks are tied together longitudinally with an insitu cast reinforced concrete capping. The insitu capping extends into the voids of the top blocks of each stack ensuring the transfer and distribution of shear stresses into the adjacent stacks. The cope face utilises a precast fender panel that acts as a permanent shutter for the insitu capping cast. At fender and safety ladder locations special fender and ladder panels are used, pre-fitted with the required fixings to accommodate the fender and ladder units.

Fender and bollard spacing has been governed by the block width of 3.4 m c/c so that fenders can always be mounted centrally on a precast fender panel and bollard forces are taken symmetrically down into the block stacks of the quay wall.

5.2.3 Passenger Landing Facility

The structural details of the passenger landing facility is described below and shown in Drawing No. PRDW-900-MN-0016-01.

The facility is located at the root of the main quay structure and has been designed to accommodate lighter boats with an overall length of 12m and a draft of 3m. The facility consists of an in-situ cast concrete staircase extending from the capping level (+3m CD) down to an upper landing at +1.7 mCD and a lower landing at +0.8 mCD. The staircase is 2 m wide and the facility is equipped with handrails for the safe embarkation/ disembarkation of passengers from lighters. The passenger facility is equipped with Trelleborg 300 DC fenders at 1.7 m c/c with two fenders mounted per fender panel. Two special fender panels will be required for the upper and bottom landings.

5.2.4 RIB Boat Ramp

The structural details of the RIB boat ramp is described below and shown in Drawing No. PRDW-900-MN-0014-01.

The proposed ramp for the RIB Sea Rescue vessel requires a 1:8 slope and is designed. Any steeper and the towing vehicle will struggle to pull the loaded boat trailer up the ramp. Any flatter and the vehicle's back axle will be immersed during launching, causing serious damage to the vehicle over time.

The proposed ramp structure will consist of a core rock slope extending from the existing boat ramp to a level of -2.0 m CD. A 300 mm thick blinding consisting of 53mm crushed stone will be screeded over the core rock prior to the placing of the precast concrete ramp panels and supports. All concrete ramp surfaces will have a roughened finish to ensure tyre traction.

No provisions for a jetty structure adjacent to the ramp have been made. All loading/unloading of personnel or equipment, not launched with the boat, will take place at the existing wharf structure or at the new passenger landing facility.

5.2.5 Berth Structure Analysis

All structural analysis undertaken as part of this preliminary design stage has been governed by the Design Basis document included in Appendix A.

The following analysis models have been used in the blockwall verification:

Slope/W - Slope Stability Package from GeoStudio
Geo5 Prefab - Retaining Wall Package from Fine Civil Engineering Software

The structures have been verified to the following Ultimate Limit States:

Overall Slip Failure	-	Slope/W
Toppling Failure	-	Geo5
Sliding Failure	-	Geo5
Bearing Failure	-	Geo5 (and Spreadsheet Calculation)
Structural Failure – Interblock sliding	-	Geo5
Structural Failure – Interblock overturning	-	Geo5

The blockwall slip failure was analysed using the traditional unfactored approach and the Morgenstern Price methodology ensuring both moment and force equilibrium. The use of high phi (Ø) core rock as backfill results in satisfactory factors of safety of above the 1.4 targeted for all structures. The use of high phi (Ø) backfill (core rock) and wider block units (8.8 m) both have positive effects on the slip failure results.

The worst case situation is represented by the Main Berth with the rock bed placed on a 2.5 m thick fine to medium sand layer overlaying the bedrock. The slip failure propagates from the splash wall, below the blockwall heel, down through the sand layer, along the bedrock sand interface (planar failure), before rising up and exiting at the surface, achieving a FOS of 1.56.

The toppling, sliding, bearing and interblock verifications were analysed to BS 6349 and Eurocode requirements. The Geo5 package takes account of Eurocode partial and combination factors and assigns favourable and unfavourable actions to determine a worst case.

The interblock overturning verifications have been conducted taking into account the friction developed on the inside faces of the blocks due to the silo pressures exerted by the crushed stone infill.

The interblock friction factor has been set as 0.4 consistent with the lower end for preformed concrete on gravel recommendation from NAVFAC supplied as a Geo5 programme input.

No design work has been undertaken on the insitu capping beam in this preliminary design phase. The capping has been sized based on previous project experience and will be properly analysed in the detailed design stage.

5.2.6 Pre-cast Block Placement

The proposed layout of the precast block units is shown on Drawing No. PRDW-900-MN-0015-01. The block arrangement has been driven by constructability issues as the placing crane is required to work along the top of the block stacks, ahead of the rubble mound breakwater. This requires that in certain cornering points extra blocks are placed to facilitates crane moments and block delivery.

5.3 Quay Furniture and Services

5.3.1 Quay Furniture

The quay furniture detail for the Main Berth and Passenger Landing Facility is described below and shown in Drawing No. PRDW-900-MN-00012-01 and PRDW-900-MN-0016-01.

The proposed fendering system for the Main Berth consists of 1450 Cell Fenders spaced at 13.6 m c/c. The fenders have been sized based on the specified design vessel, using difficult berthing in sheltered conditions in accordance with PIANC (2002) guidelines - case B. The fender spacing has been has been selected based on the size and spacing of the fender panels and will be checked in detail and confirmed during the detailed design stage.

D fenders installed vertically are proposed for the passenger landing facility. The fender spacing of 1.7 m c/c will allow up to two fenders per panel.

Tee Bollards with a capacity of 30 tonnes are proposed for the Main Berth. A bollard spacing of 10.2 m c/c has been used to tie in with the 3.4 m wide block dimension. Bollards for the vessel fore lines are located on the cap of the breakwater round head. The mooring points for the stern lines are located alongside the Main Berth and against the splashwall. Stern lines running to the rear bollards pose an access problem for the Main Berth and are only intended for use by vessels requiring additional restraint during periods of bad weather. Under these conditions quayside access to the vessel will be restricted.

Recessed stainless steel safety ladders with accompanying mooring rings spaced at less than 30 m centres are proposed for the Main Berth and Passenger Landing Facility.

5.3.2 Quayside Services

The quayside services details for the Main Berth, is described below and shown in Drawing No. PRDW-900-MN-0013-01.

The proposed splash wall will accommodate the three 110 mm ducts required for power, lighting and communications with draw boxes spaced every 60 m. The water line will be mounted to the lower section of wall with stainless steel brackets for easy access for maintenance.

The 6" fuel line will be accommodated on concrete plinths located at the base of the splash wall, allowing easy access for inspection, repair and maintenance. A vehicle barrier consisting of concrete posts at 1.0 m centres is proposed to prevent vehicle contact with the pipelines.

The top of the splash wall has been detailed with an overhang to protect all exposed lines from overtopping. The splash wall extends all the way around the breakwater head up to the corner of the main berth.

The back of quay drainage system has been preliminarily designed as drainage channels located against the splash wall. As specified all drainage flow is away from the cope edge draining to a collection point fitted with an oil separator. Drainage of all overtopped water is not considered practical as the over topping volumes could exceed flow capacity of the oil separator. It is not feasible to providing a drainage system to channel and treat over topping water. A more practical approach to avoiding pollution from the breakwater cap is to ensure that it is cleaned regularly as well as providing designated wash down area for contaminated equipment draining to an oil separator. Excess overtopping water can then drain over the cope edge as per standard breakwater design.

6. CONCLUSIONS AND RECOMMENDATIONS

Site Conditions

- A thorough assessment of the met-ocean conditions within Rupert's Bay has been carried out. This has allowed for more certainty in the design of the marine structures specifically with regards to the complex design wave conditions in the bay. The design wave condition (H_{mo} = 4.6 m and Tp = 16 s) corresponds to a 1:1000 year return period with a risk of occurrence equivalent to 7%.
- It is recommended that a comprehensive bay wide multi-beam survey be carried to confirm the latest seadbed geometry in Rupert's Bay. In addition, beach survey profiles should be carried out along the bay coastline to confirm the average slope of the shoreline.
- It is recommended that SPT's from a barge as well as vibrocores be carried out along the axis of the breakwater and main quay structure once marine kit is established in Rupert's Bay. A geotechnical desktop study and some landside core drilling at the toe of the breakwater will highlight if the bedrock integrity is of concern.
- Two dimensional sediment modelling has been carried and the results indicate that the development will result in significant wave sheltering in the southern region of Rupert's Bay. This sheltering results in a change of hydrodynamics in the bay summarised as follows:
  - Sedimentation of the facility's navigational area is predicted to occur during storm conditions only and with 0.1 m accretion in the south-eastern corner of the berth pocket during the 100-year storm event. It is recommended that small dredging equipment be included in the development, to facilitate intermittent dredging as and when required.
  - 0.5 m to 1.5 m of sedimentation is expected to occur along the south-western edge of Rupert's Bay. Currently, this region is a rocky reef, which, if covered by sand, may change the marine ecology.
  - The implementation of the permanent wharf facility does not significantly change the waves, currents or sediment transport conditions at the south-eastern swimming beach. The beach stability is critically linked to the presence of the concrete pipeline and offshore breakwater. Failure of maintaining these structures will result in the rapid erosion of the swimming beach, irrespective of the proposed development.
- High density (Armour, core and underlayers) and moderate density (Core only) quarry rock identified and tested in Rupert's Valley is considered adequate for marine construction purposes based on the tests and independent core rock assessments (Icebreak, 2013) carried out to date. It is recommended that all rock tests as specified in PRDW (2013e) be carried out and cores drilled to confirm quarry yields.

Navigation

- To avoid a potential navigation hazard, the breakwater layout has been modified to prevent the breakwater slopes extending into the berth pocket.
- The ship manoeuvring to be carried out in the next design stage will verify if any additional berthing or manoeuvring assistance vessels are required during operations. However, the assumption that no assistance is required is considered reasonable given the navigation geometry selected and the mild wave and weather conditions.

Berth Availability/Operability

- Wave penetration modelling was performed in order to inform a high level downtime assessment for moored vessel motions based on guideline limiting wave height criterion.
    - These preliminary calculations show a high availability for both the general cargo (100% availability) and the Ro/Ro (94% availability) vessels.
    - Short term variability in wave heights may lead to short periods over which loading would be difficult and inefficient. While it is not envisaged that vessels will have to leave the berth due to this short term variability, the loading inefficiency could be interpreted as downtime which would reduce the estimated availability slightly.
    - The available limiting criteria relate to wave periods not exceeding 10 s. Typical wave periods for Rupert's Bay are longer and could mean that limiting criteria should be lowered. This would lead to increased downtime.
- It is recommended that vessel motion modelling studies be undertaken in order to quantify the availability in terms of critical limiting motions. Such a study will also enable an evaluation of the combined effect of head on and beam on conditions which was not possible with the present approach.
- The limiting conditions for manoeuvring operations has been reviewed and the downtime related to ship berthing/unberthing is described below:
    - The limiting wind condition for manoeuvring operations is a one-minute averaged 11 m/s (8.8 m/s one-hourly average) wind velocity at a height of 10 m above sea level. This wind condition has an annual exceedance of 10%.
    - A significant wave height of 1.0 m has an annual exceedance of 6% per year.
- The combined average monthly downtime percentage excluding short duration events (< 3 hours) is 13% and the maximum downtime for the month of November is 26%. Of these downtime events 81% will have a duration of less than 12 hours. If it is assumed that downtime events of less than 6 hours is an acceptable delay for berthing, the average downtime reduces to 10% and the maximum downtime for November reduces to 22%.

Bulk Fuel Offloading Facility Relocation

- The preliminary design has included the relocation of the mooring buoy system. The relocation of the fuel offloading facility will allow both the bulk fuel terminal and the proposed wharf structure to operate independently and will reduce the navigation risk to both terminals.
- The preliminary design includes all marine aspects related to the mooring system but excludes the system hydraulics from the vessel to the quay and then to the landward storage facility as required for a fully functioning bulk fuel installation.
- In the next design stage a more detailed review of the anchor details for the mooring system will be required.

Breakwater

- The breakwater layout was tested in a two dimensional flume model. Stability of the seaward slope was confirmed for the design wave conditions; however the tests demonstrated the need for careful placement of units at the intersection of the 1:1.5 slope and horizontal crest.
- Significant wave overtopping volumes were measured during the flume model tests which lead to failure of the lee slope after extended low water storm wave action. To avoid this damage it is recommended that the top row of units be anchored by casting the units into the concrete deck.
- Three dimensional tests in which oblique wave attack is modelled accurately are required for the final design. These tests may prove that lee stability is sufficient without the recommended stabilization of top row units; however, it is recommended that provision for such stabilization be included in the preliminary design cost estimates.
- It is estimated that loading and offloading may be affected by wave overtopping for significant wave heights exceeding 1.6 m. This is estimated to be exceeded no more than 4 days per year.
- Detail bathymetry is required of the cliffs at the root of the breakwater to define a practical tie-in of the rock toe and Core-loc slope into the land.

Quay Structures

- The quay structure has been designed using precast reinforced concrete rectangular hollow blocks. The preliminary design work undertaken as part of this study has optimised and refined the previous conceptual engineering solution.

- It is not feasible to providing a drainage system to channel and treat over topping water. A more practical approach to avoiding pollution from the breakwater cap is to ensure that it is cleaned regularly as well as providing designated wash down area for contaminated equipment draining to an oil separator. Excess overtopping water can then drain over the cope edge as per standard breakwater design.

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PRELIMINARY DESIGN DRAWINGS

APPENDIX A

DESIGN BASIS

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

DESIGN BASIS

REPORT NO. : 1097/02/01 REV D

FEBRUARY 2013

PRESTEDGE RETIEF DRESNER WIJNBERG (PTY) LTD
CONSULTING PORT AND COASTAL ENGINEERS

Rupert's Bay Permanent Wharf – Phase 2: Design Basis Report No. 1097/02/01 Rev C					
Revision	**Date**	**Author**	**Checked**	**Status**	**Approved**
A	23 January 2013	SRP/WGD/CS/FMG	SRP	For Comment	
B	31 January 2013	SRP	AHH	For Issue	
C	28 February 2013	SRP	AHH	For Use	
D	08 March 2013	SRP	AHH	For Use	

Keywords: St Helena Island, Rupert's Bay, Permanent Wharf, Design Basis

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

DESIGN BASIS

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1. INTRODUCTION

The construction of a new airport on the island of St Helena will require the existing port facilities on the island to be upgraded. These upgrades will include the provision of permanent wharf facilities for handling bulk cargo, petroleum products, general cargoes and containers in medium to long-term. The site selected for this facility is Rupert's Bay on the North West coast of the island. The location of the site is shown in Figure 1.

This document provides the design basis that will be used for the preliminary engineering design for the provision of a permanent wharf structure.

1.1 Project Phase Definition

The design component of the Rupert’s Bay Permanent Wharf Project comprises three phases defined below:

- Phase 1 : Scoping and Optimisation
- Phase 2 : Preliminary Engineering Design
- Phase 3 : Detailed Design

This document is the design basis for Phase 2.

1.2 Scope of Work

The scope of work for this design stage is the preliminary engineering design of the following marine elements:

- Breakwater
- Main Quay Wall
- Ro-Ro Berth
- Lighter Berth
- Navigational Aids
- Fixed Concrete Boat Ramp for launching Sea Rescue Boats (RIBs)

The study will also include a detailed investigation into the local coastal processes, including waves, currents and sediment transport.

1.4 Report Structure

In Section 2 the guidelines, codes of practice and other external references that will be utilised during the design are listed. Section 3 of this report summarises the design criteria. The marine environmental conditions, as far as they are known, are described in Section 4. Section 5 discusses the marine structure design options and methodology. The berth services, quay furniture and navigational aid requirements are presented in Section 6.

1.5 Conventions and terminology

The following conventions and terminology are used in this report:

- Wave direction is the direction from which the wave is coming, measured clockwise from true north.
- Wind direction is the direction from which the wind is coming, measured clockwise from true north.
- Current direction is the direction towards which the current is flowing, measured clockwise from true north.
- H_{m0} is the significant wave height, determined from the zeroth moment of the wave energy spectrum. It is approximately equal to the average of the highest one-third of the waves in a given sea state.
- T_p is the peak wave period, defined as the wave period with maximum wave energy density in the wave energy spectrum.
- Mean wave direction (Dir) is defined as the mean direction calculated from the full two-dimensional wave spectrum by weighting the energy at each frequency
- Seabed and water levels are measured relative to Chart Datum. Chart Datum (CD) is 0.50 m below Mean Sea Level.

2. GUIDELINES AND CODES OF PRACTICE

The Rupert's Bay permanent wharf preliminary engineering design will be executed within the framework of a number of complementary and interrelated design guidelines and codes of practices.

2.1 ISO 9000 Series

All design work will be undertaken within the Prestedge Retief Dresner Wijnberg (Pty) Ltd Quality Management System that has been set out in terms of ISO 9001.

Design work will be done by or under the direction and supervision of a professional engineer with the relevant experience. All design work will be reviewed internally before it is issued to the Employer.

2.2 Guidelines

A broad spectrum of design guidelines will be used for the design of the wharf structure. The following publications may be referenced:

- Coastal Engineering Manual (CERC, 2003)
- The Rock Manual (CIRIA, 2007)
- Recommendations of the Committee for Waterfront Structures Harbours and Waterways (EAU, 1996)
- Supplement for Bulletin No. 51, 1985. Underkeel Clearance for Large Ships in Maritime Fairways with Hard Bottom (PIANC, 1985):
- Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles in Marine Environment (PIANC, 1992)
- Guidelines for the design of Armoured Slopes under quay walls PIANC Working Group 22 (PIANC, 1997a):
- Approach Channels, A Guide for Design. Final Report Supplement to Bulletin 95, June 1997 (PIANC ,1997b):
- Guidelines for the Design of Fender Systems PIANC (2002)
- Wave Overtopping of Sea Defences and Related Structures: Assessment Manual (EurOtop, 2007)
- Port Engineer's Handbook: Recommendations and Guidelines Second Edition (Thoresen, 2010)
- ROM 0.2-90 Actions in the Design of Maritime Harbour Works (ROM, 0.2-90)

2.3 Codes of Practice

The design shall be based on internationally approved codes of practices. The latest version of the following codes at the time of commencing the design shall apply where necessary:

Primary Codes:

- Eurocode 0 – Basis of structural design
  - BS EN 1990 Basis of structural design
- Eurocode 1 - Actions on structures
  - BS EN 1991-1-1 General actions – Densities, self-weight and imposed loads
  - BS EN 1991-1-4 General actions – Wind actions
  - BS EN 1991-1-5 General actions – Thermal actions
  - BS EN 1991-1-6 General actions – Actions during execution
  - BS EN 1991-1-7 General actions – Accidental actions
  - BS EN 1991-2 Traffic loads on bridges
  - BS EN 1991-3 Actions induced by cranes and machinery
- Eurocode 2 – Design of concrete structures
  - BS EN 1992-1-1 General – Common rules for building and civil engineering structures
  - BS EN 1992-2 Bridges
- Eurocode 3 – Design of steel structures
  - BS EN 1993-1-1 General rules and rules for buildings
  - BS EN 1993-1-5 Strength and stability of planar plated structures without transverse loading
  - BS EN 1993-1-8 Design of joints
  - BS EN 1993-1-9 Fatigue strength
  - BS EN 1993-2 Bridges
  - BS EN 1993-5 Piling
  - BS EN 1993-6 Crane supporting structures
- Eurocode 7 – Geotechnical design
  - BS EN 1997-1 General rules and rules for buildings
  - BS EN 1997-2 Ground investigation and testing
- Eurocode 8 – Design of structures for earthquake resistance
  - BS EN 1998-1 General rules seismic actions and rules for buildings
  - BS EN 1998-2 Bridges
  - BS EN 1998-5 Foundations, retaining structures and geotechnical aspects

Supplementary Codes:

- BS 6349: Maritime Structures - Part 1 through 7.
- NORSOK M-501: Surface Preparation and Protective Coating.
- ISO 6812: Roll on/Roll off ship-to-shore connection – Interface between terminals and ships with straight stern/bow ramps

3. DESIGN CRITERIA

The design criteria and functional requirements defined in this section have been extracted from SHA (2011) as well as variation No.3 received from Basil Read on 08 January 2013.

3.1 Owner's Project Requirements

The design aims to provide the most cost effective permanent wharf solution while keeping safety and efficiency of navigation and ship operations paramount.

As per 16.3.2 (SHA, 2011a) the design shall as far as reasonably practical:

- Sympathetically reflect the coastal landscape
- Avoid any land uptake
- Avoid adverse impacts on Rupert's beach and amenity area
- Avoid disturbance of the Boer prisoner of war desalination chimney
- Minimise direct effects on Rupert's lines (the fortification wall)
- Minimise adverse effects on the marine and coastal ecology

The locations of these site constraints are shown in Figure 2. Additional site photographs are presented as Figures 3 & 4.

3.2 Environmental Requirements

It is the Contractor's understanding that a limited Environmental Impact Assessment (EIA) has been carried out which concentrated on the onshore impacts. The location of the wharf has changed from that in the reference design implying additional scoping or an EIA will be required. The requirements and principles from the Aecom, St Helena Airport and Supporting Infrastructure Environmental Management Plan (February 2011) will be taken into consideration during the preliminary design (Phase 2). The main requirements in respect to the wharf are summarised below with a detailed list provided in section 4.2.5 of the EMP (February, 2011):

- Mitigation for the loss of littoral benthic habitats by the provision of attachment of substrates and cavities for marine wildlife.
- The wharf shall be designed to sympathetically reflect the coastal landscape and minimise the effects on water quality and the marine and coastal ecology.
- The wharf shall be designed to avoid impeding the natural flow of water and sediment around the bay.

- Rock armour shall be used in preference to concrete armour units provided that the structural integrity of the marine structures is not compromised.

3.3 Design Life

Primary marine structures shall have a design life of 70 years (SHA, 2011b).

3.4 Port Layout

Key factors that have governed the configuration of the port that best meets the needs of all parties both during the airport construction stage and future port and shipping requirements include:

- Capital and maintenance cost implications
- Degree of shelter and annual down time during adverse wave conditions
- Safety and efficiency of navigation, ship manoeuvring, berthing and unberthing manoeuvres.

The preliminary design is to be based on Option Layout 1 (Figure 5) which consists of a 17m wide quay and a 95m long quay which can accommodate a design vessel with a maximum length of 105m. This layout is shown in Figure 5. The existing fishermen's wharf will be retained and continue to be used by local fishermen.

3.5 Design Vessels

The berthing facility will be designed for the following design vessels:

TABLE 1

DESIGN VESSELS

Vessel Type	Deadweight (tonne)	Length Overall (m)	Beam (m)	Design Draft (m)
Dry bulk carrier	5 000	90.0	16.0	5.7
General Cargo/Ro-Ro	2 600	105.0	20.0	6.0

3.6 Navigation and Ship Manoeuvring

Navigation and ship manoeuvring design requirements with regard to vessel turning circles and approach channel dimensions will be defined using PIANC guidelines and recommendations from the Port Designer's Handbook (Thoresen, 2010). All vessels using the proposed wharf will be required to self-berth without the aid of tugs. The design will take account of the navigation and manoeuvring requirements for this berthing procedure. A ship manoeuvring study will be carried out during the detail design stage to verify the layout.

3.7 Cargo Handling Operations

Cargo handling shall include dry bulk, containerised and general cargoes.

Vessel loading and unloading operations will be conducted through the use of ship's gear and mobile cranes operating from the wharf apron. The size and lifting capacity of the mobile cranes is defined in Section 3.10 below.

3.8 Berth Facilities

The design parameters for the berthing facility are defined in Section 16.2 (SHA, 2011a), however the length and back of quay width of the main quay has been shortened from 120m to 95m and 25m to 17m, respectively.

The shortening of the quay structure is based on a shipping review assessment (PRDW, 2012) requested by the Employer and carried out in 2012. PRDW (2012a) recommended that the overall length (Loa) of vessels presently being used along adjacent shipping routes to St Helena Island range between 63m and 115m. This implied that a shorter quay structure would suffice, for the vessel fleet that is likely to service the Island in the future.

The back of quay width reduction is largely a cost optimisation of the structure, with a 17m quay width considered adequate for limited storage and traffic movement on the quay.

The physical dimensions of the berth facilities are summarised below:

- Main Quay 95m long, 7.0m minimum berth depth, with 17m back of cope.
- Ro-Ro berth Ramp designed to ISO 6812.
- Lighter Berths 40m long, 3.0m minimum berth depth.
- Slipway Ramp Ramp design to accommodate sea rescue boats

3.9 Operability

The berth should be operable for general cargo handling by ship's gear and mobile crane. A 96% operability preference has been indicated by the Employer. The present study will assess operability for the layout shown in Figure 5 which may differ from the preferred percentage workability.

3.10 Berth Loading

Quay loading to accommodate wave, berthing and mooring forces, together with vertical live loading of not less than 20 kN/m^2. The quay will be designed to accommodate crawler cranes up to 200t.

4. MARINE ENVIRONMENTAL CONDITIONS

4.1 Tidal Levels

The astronomical tidal levels shown in Table 2 have been extracted from Admiralty Chart 1771 which was last updated on 20 October 2005. The tidal regime in St Helena is characterised as semidiurnal with a range of 1.0m and a mean sea level of 0.5m.

TABLE 2:

PREDICTED TIDE LEVELS

Tide	(m, CD)
Highest Astronomical Tide (HAT)	+ 1.06
Mean high water springs (MHWS)	+ 0.94
Mean high water neaps (MHWN)	+ 0.72
Mean sea level (MSL)	+ 0.50
Mean low water neaps (MLWN)	+ 0.28
Mean low water springs (MLWS)	+ 0.07
Lowest Astronomical Tide (LAT)	- 0.06

The tidal range for Rupert's Bay is relatively small, with the present design water level set to 1.005 m CD as defined in previous studies. During the preliminary design the design water level will be reviewed where an extreme water level analysis will be carried out including an assessment of local storm surges and sea level rise.

4.2 Waves

4.2.1 Offshore Wave Data

A hind-cast wave data-set as used in PRDW (2012b) will be re-used during this study. The data set is computed based on TOPEX/POSEIDON altimetry and calibrated with measured buoy data. The data was extracted from a model grid-point (15^{o}S, 6^{o}W), located approximately 100 km north-west of St Helena Island (Figure 6).

The wave data set covers a duration of 15 years and corresponds to the period from January 1993 to December 2003 and from December 2006 and November 2011. The wave data set consists of wave spectral and parametric information extracted at 6 hourly intervals.

Figure 7 includes a wave rose, time-series and a non-exceedance graph for the offshore wave data set. The offshore wave climate is characterised by waves being generated from the SE'ly, SW'ly and the NW'ly direction sectors. These direction sectors account for 25.6%, 45.7% and 6.9% of the total wave data set, respectively.

Waves being generated from the SE'ly sector can be characterised as wind seas with wave periods (Tp) ranging between 6 and 10 s. Waves generated from the SW'ly and NW'ly direction sectors are predominantly characterised as swells, with wave periods (Tp) ranging between 12 and 16 s and significant wave heights (H_{mo}) ranging from 0.5 to 5.0 m. The average wave height (H_{mo}) for the complete data set is 1.9m with 90% of the data set containing wave heights below 2.7m.

Considering the location of Rupert's Bay and the offshore model grid point, wave conditions most relevant for the design originate from the NW'ly direction sector. The highest wave height (H_{mo}) recorded from this sector in the 15 year data set was 2.0 m.

4.2.2 Nearshore Wave Data

The wave climate in Rupert's Bay is dominated by the refracted south-easterly trades through most of the year. However during the months of January to March (Cartwright, 1971) heavy continuous swells, or 'rollers' set in from the NNW causing a heavy break on the north-western coast of the island.

Local wave data has been collected at the site using an Acoustic Doppler Current Profiler (ADCP) deployed in approximately 10 metre water depth within Rupert's Bay (5°42.737 W, 15° 54.991 S) between December 2006 and March 2009. The location of the ADCP is shown in Figure 6. Pressure and velocity readings were taken at an interval of 3 hours, with readings every hour between February 2007 and March 2007.

Figure 8 shows the data coverage over this period which shows intermittent data collection (36 % complete). The data set does however cover the critical periods of January to March over three years. The highest wave height (H_{m0}) recorded in this period was 2.3 m.

4.2.3 Comparison of Measured and Modelled Wave Climates

A wave propagation study using the MIKE 21 Spectral Waves Flexible Mesh (MIKE 21SW) model was carried out in the previous project stage in order to transform the offshore wave data set to nearshore locations in Rupert's Bay.

The 15 year OCEANOR spectral data set was transformed (modelled) to the ADCP location to provide a nearshore wave climate. A comparison between the modelled and measured data sets was carried out which is presented in Figure 9. The trend observed was that the measurements are generally higher than the modelled data, with the highest wave height recorded of 2.3 m.

Figure 10 illustrates an under prediction of the measured data along with an erratic time series record of measurements for a storm event. In this example the wave height measurements change from 2.3m to 1.4m in 4 hours and then up to 2.2 m within the next hour, raising questions and concerns about the quality of the measured data.

4.2.4 Design Wave Conditions

In light of the above the reliability of wave measurements in Rupert's Bay is of concern. Further analysis will therefore be carried out during this project stage to confirm the design wave conditions. This will include further coastal modelling and more detailed assessments of the measured data.

Operational and extreme wave conditions for the design of marine structures are defined as the 1:10yr and 1:100yr return period wave height respectively. The preliminary design wave conditions shown in Table 3 will be verified as part of this study.

TABLE 3:

ASSUMED DESIGN WAVE CONDITIONS FOR MARINE STRUCTURES

Design wave	Hs [m]	Tp [s]	Dir
Operational (1:10yr)	TBC	TBC	TBC
Extreme (1:100yr)	4.0	16	NW

4.3 Currents

Figure 11 shows a cross-current rose for currents recorded at the ADCP location in Rupert's Bay. The current directions are orientated predominantly in the SW'ly and NE'ly directions. The currents are very low with a maximum measured current of 0.25m/s flowing towards the south-west. These low current magnitudes do not influence the design of the structures involved in the project, however it should be considered from a navigation point of view.

4.4 Wind

Wind data has been sourced from the Met Office at Horse Point, St Helena Island at an elevation of approximately 300 m. The duration of the data set is from June 2004 to December 2012, the data coverage over this period shows intermittent data collection (50% complete). Unfortunately, this wind data set is not considered representative of the local conditions in Rupert's Bay.

A 15 year offshore wind time-series data set which coincides with the offshore wave data set was provided by OCEANOR. Due to the distance from the offshore location to Rupert's Bay and local topography this data set would not be representative of all directions. The effect of the island topography on this data will be assessed based on engineering judgement and previous experience in order to estimate the wind climate at the berth.

4.5 Seabed

The seabed in Rupert's Bay is characterised by a layer of fine to medium grained sediments overlying the igneous bedrock. The bedrock is exposed around the headlands, and up to 200 metres offshore, and may be expected to comprise hard to extremely hard rock. The sediments may be mobile under the seasonal storm wave events (Tritan, 2006). The sub-sea geology plan is included in Figure 12.

4.6 Bathymetry

Bathymetric data from Tritan's 2006 survey (Tritan, 2006) will be used for this study in addition to surveys undertaken in 2011. The results of the Tritan (2006) survey are shown in Figure 12.

The bathymetry of Rupert's Bay was determined from 210 580 individual soundings and contoured at a 0.5 m interval, referenced to LAT. The uniformly seaward-dipping seafloor is characterised by generally smooth isobath contours (Tritan, 2006).

5. MARINE STRUCTURES

The type of marine structures that will best fulfil the Employer's requirements for the wharf consist of a rubble mound breakwater with concrete armour units and a gravity quay structure (Figure 13). The following aspects were considered during Phase 1 of the project in the selection of these marine structures.

- Site conditions (geophysics, bathymetric)
- Methods of construction (available expertise and plant)
- Serviceability
- Low maintenance requirements
- Durability
- Available construction materials, plant and equipment
- Construction schedule requirements, including phasing of construction
- Available budget for construction

5.1 Breakwater

5.1.1 Functional Requirements

The main function of a breakwater will be to protect and shelter the berths from waves entering the bay. The inner face of the breakwater will be designed as a quay wall to provide a safe berthing environment for the design vessel.

5.1.2 Geometric Requirements

The quay wall requires 17 m of back of cope area for the loading and unloading of vessels. In addition, the breakwater must provide adequate protection for a 95 m long quay.

5.1.3 Design Criteria

The inner side of the breakwater will be supported by a quay wall, while the outer face will be protected by a rubble mound rock revetment with 5t concrete armour units.

The breakwater crest elevation and crown wall will be designed according to the EurOtop Manual, considering the following functional requirements:

- Under operational conditions, the wave overtopping shall be safe for pedestrians and vehicles ($q < 0.1$ l/s/m). Equipment may be required to be stationed a safe distance from the cope edge (set back 5m from the crown wall, $q < 0.4$ l/s/m).
- For extreme conditions, the operation of the quay may be restricted, and any equipment may need to be removed from the breakwater. Wave overtopping shall not damage the crown wall and pavements ($q < 200$ l/s/m).

The stability of the breakwater slope will be optimised in a physical model test (2D wave flume). Toe stability and mean overtopping discharges will be considered.

5.2 Quay Walls

The quay wall structure will be designed as a gravity wall concept as the site's subsea conditions are suitable for gravity type wall structures supported on a sufficiently thick stone bed. During construction of the permanent wharf, boreholes over the edge of the block wall structure will be required to confirm founding conditions. The back of quay width will be set as 17m.

The preferred block shape is the prefabricated rectangular reinforced concrete box. The prefabricated rectangular concrete blocks are placed on top of each other from -8m CD up to +2m CD with the voids filled with rock material thereby minimising block weight but maximising utilisation of local materials.

A constant shape and size for all blocks is proposed to streamline the fabrication, transport and placing processes.

The quay walls and associated structures will be designed for maximum durability in the marine environment. Durability will be ensured by providing sufficient concrete cover for all reinforced concrete elements and detailing all exposed steel elements as stainless steel or hot tip galvanised.

5.3 Construction Materials

All construction materials used shall be specified for durability and low maintenance. Material durability would be specified in the material specification. Materials will be sourced from the island, South Africa and other international markets as required. The Employer's brief is aimed at cost effectiveness favouring the use local material as the logistics are simpler than importing from South Africa/Namibia etc.

Suitable marine grade concrete as well as suitable grades of stainless steel will be specified to ensure that the structures reach their economic life with minimum maintenance requirements.

It is envisaged that all the rock required for the construction of the breakwater shall be supplied from a local quarry. The following rock tests will be carried out to assess the quality of the rock to be used in the wharf construction:

- Gradings
- Shape
- Density
- Water Absorption
- Drop Test Breakage Index

The density and water absorption tests will be carried by Basil Read out as part of the preliminary design stage to confirm the quality of the rock being considered.

6. QUAY FURNITURE, NAVIGATION AIDS AND SERVICES

6.1 Quay Furniture

Mooring bollards, fendering, mooring hooks and ladders will be designed to accommodate the berthing and mooring loads from the largest design vessel in accordance with international standards and procedures.

6.2 Navigation Aids

All moorings, berths and approach channels shall have lights and marker buoys as defined in 16.3.11 (SHA, 2011a).

6.3 Services

Containment for power, water, lighting and communications will be provided and surface water runoff will be handled in accordance with the requirements of 16.3.13 (SHA, 2011a).

Provisions for a fuel services corridor on the permanent wharf will be provided. The corridor will accommodate a fixed fuel pipeline extending from the end of the permanent wharf to the emergency shutdown valve on the shoreline. The end of the permanent wharf will accommodate the hose gantry as well the storage of deployment and recovery equipment required for the floating hose equipment.

The permanent wharf will make provisions for the export of waste/oil disposal from the Island. Waste oil will be transported to the quay, as is presently being done in Jamestown, and pumped into a ships waste oil sludge tank for onward disposal in accordance with the requirements of 16.3.13 (SHA, 2011a).

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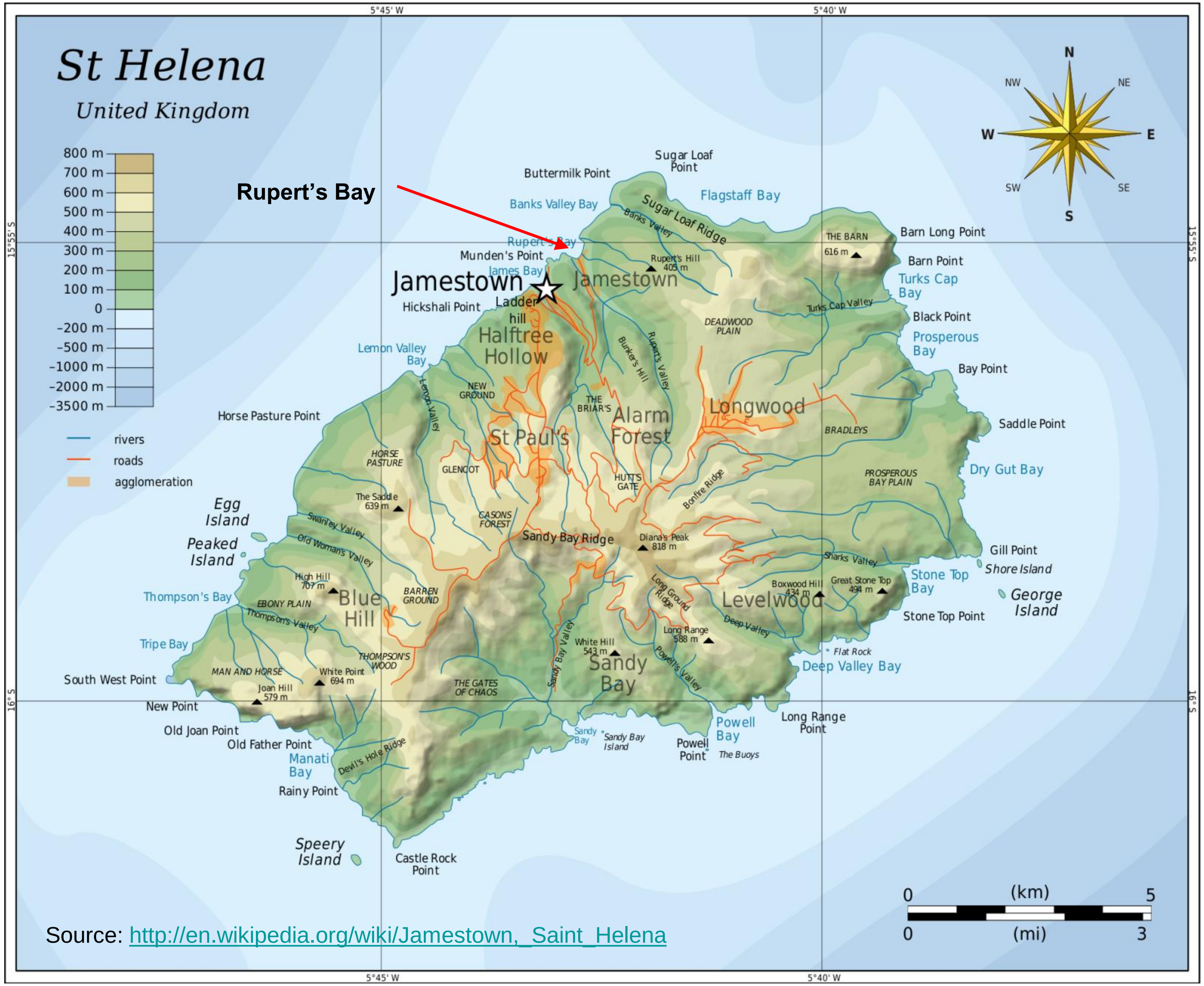
FIGURES

Saint Helena Island

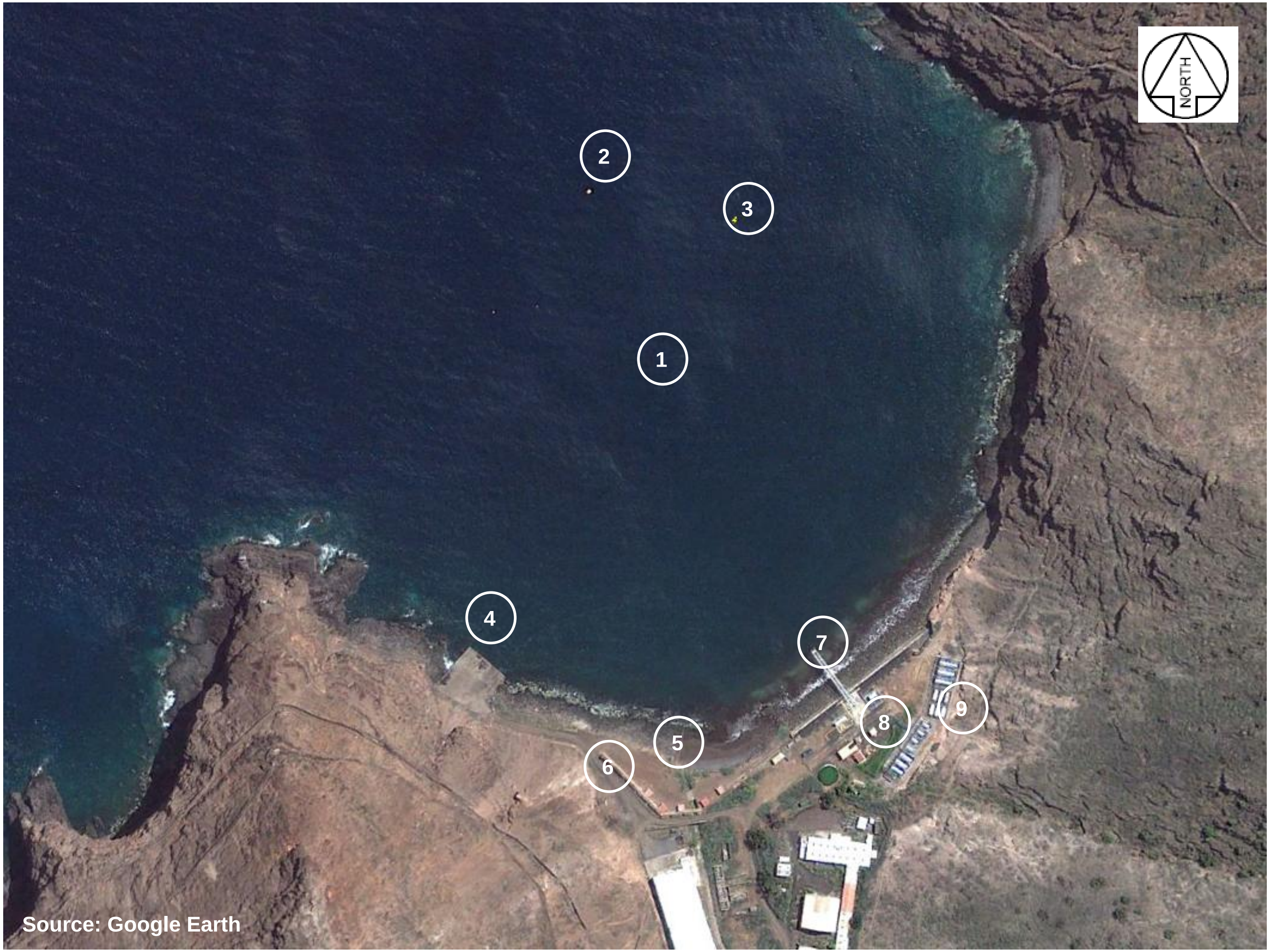
Geographic coordinates: 15° 56’ S, 5° 42’ W



Title:	**Rupert’s Bay Permanent Wharf – Phase 2** **Locality Map St Helena Island & Rupert’s Bay**	**Figure No.** **1**

Legend:

1. Rupert's Bay
2. Fuel buoy
3. Position of ADCP
4. Shallow wharf (Sheet pile with concrete capping)
5. Beach & amenity area
6. Rupert's lines (fortification wall)
7. Fuel hose gantry
8. Boer War desalination chimney
9. Tank farm



Title:	Rupert's Bay Permanent Wharf – Phase 2 **Rupert's Bay Site Constraints**	Figure No. 2

Legend:

1. Shallow wharf (Sheet pile with concrete capping)
2. Beach & amenity area
3. Rupert's lines (fortification wall)
4. Fuel gantry
5. Boer War desalination chimney
6. Tank farm

Title:	Rupert's Bay Permanent Wharf – Phase 2 **Rupert's Bay Site Photograph No. 1**	Figure No. 3

Legend:

1. Shallow wharf (Sheet pile with concrete capping)
2. Beach & amenity area
3. Rupert's lines (fortification wall)

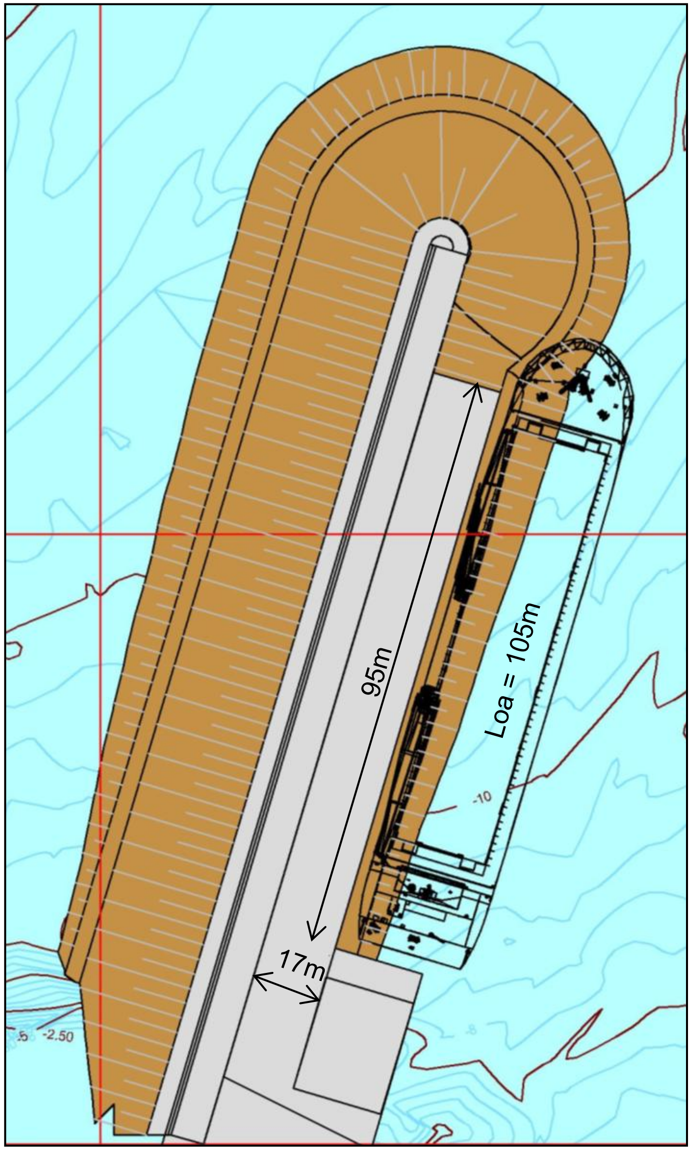
Title:

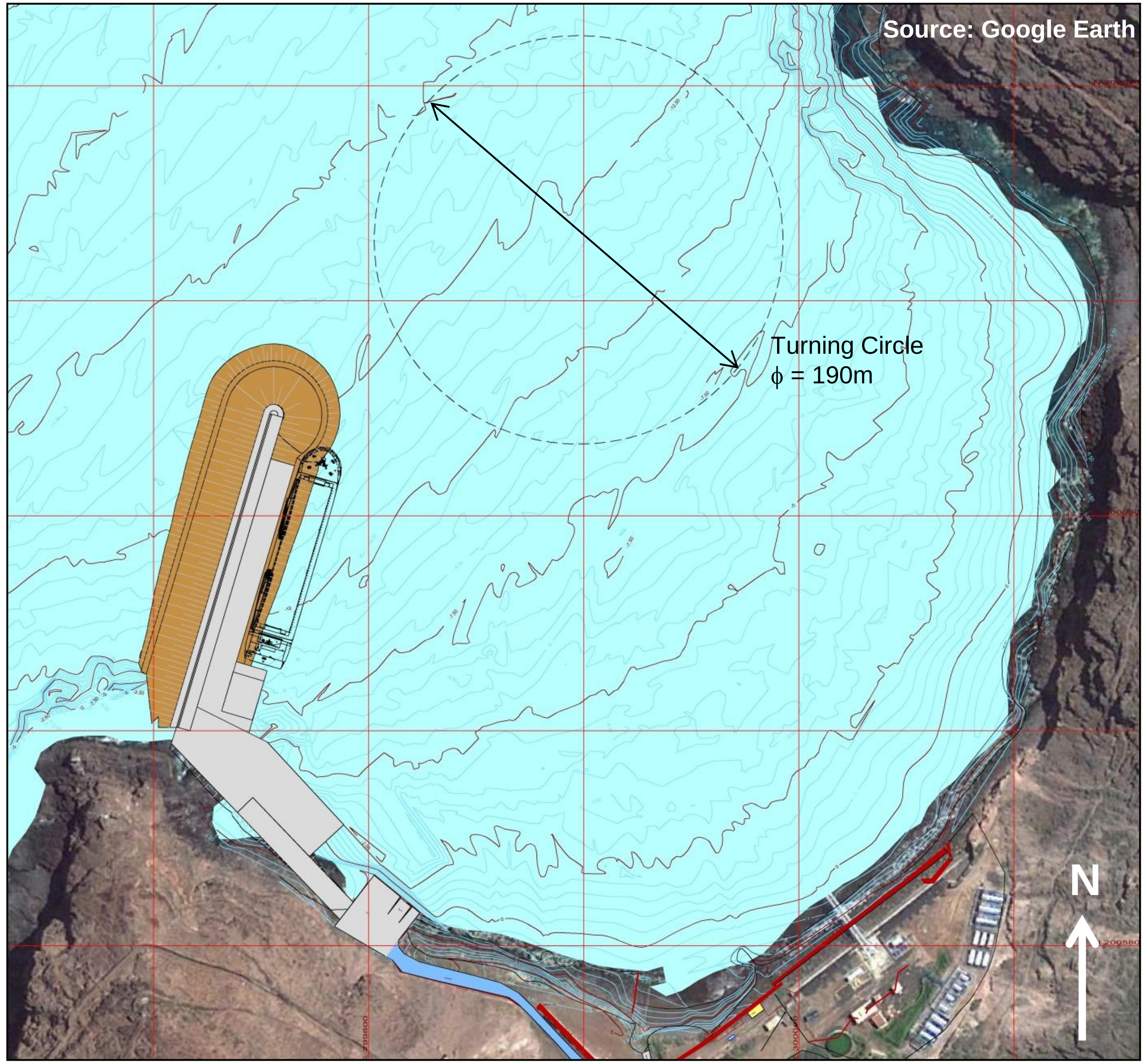
Rupert's Bay Permanent Wharf – Phase 2

Rupert's Bay Site Photograph No. 2

Figure No.

4





Title:

Rupert's Bay Permanent Wharf – Phase 2

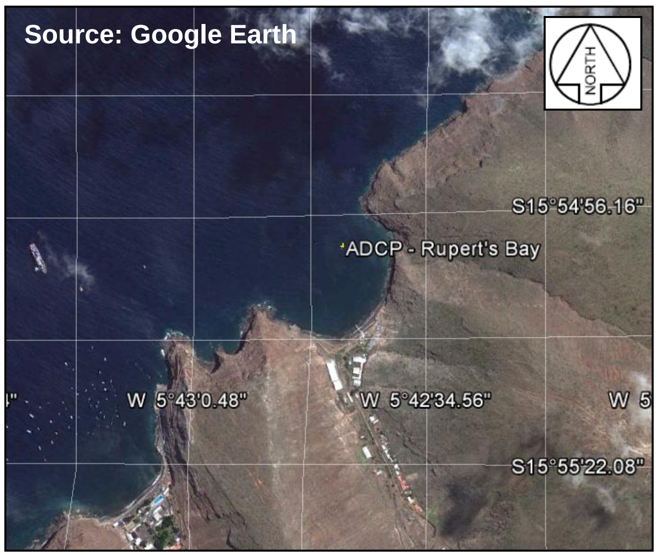
Layout Option 1 | Quay = 17m wide and 95m long | Design Vessel Loa = 105m

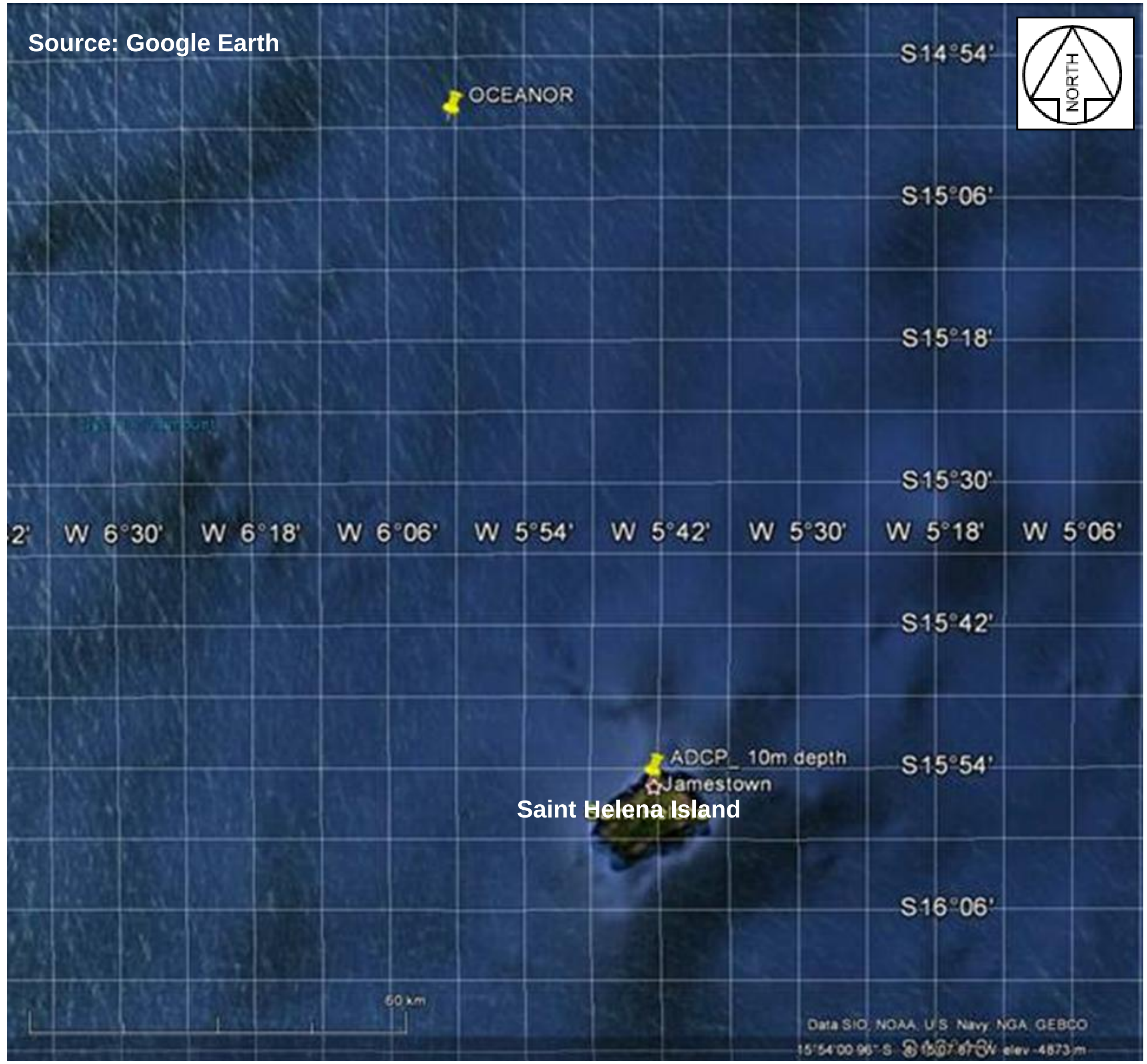
Figure No.

5

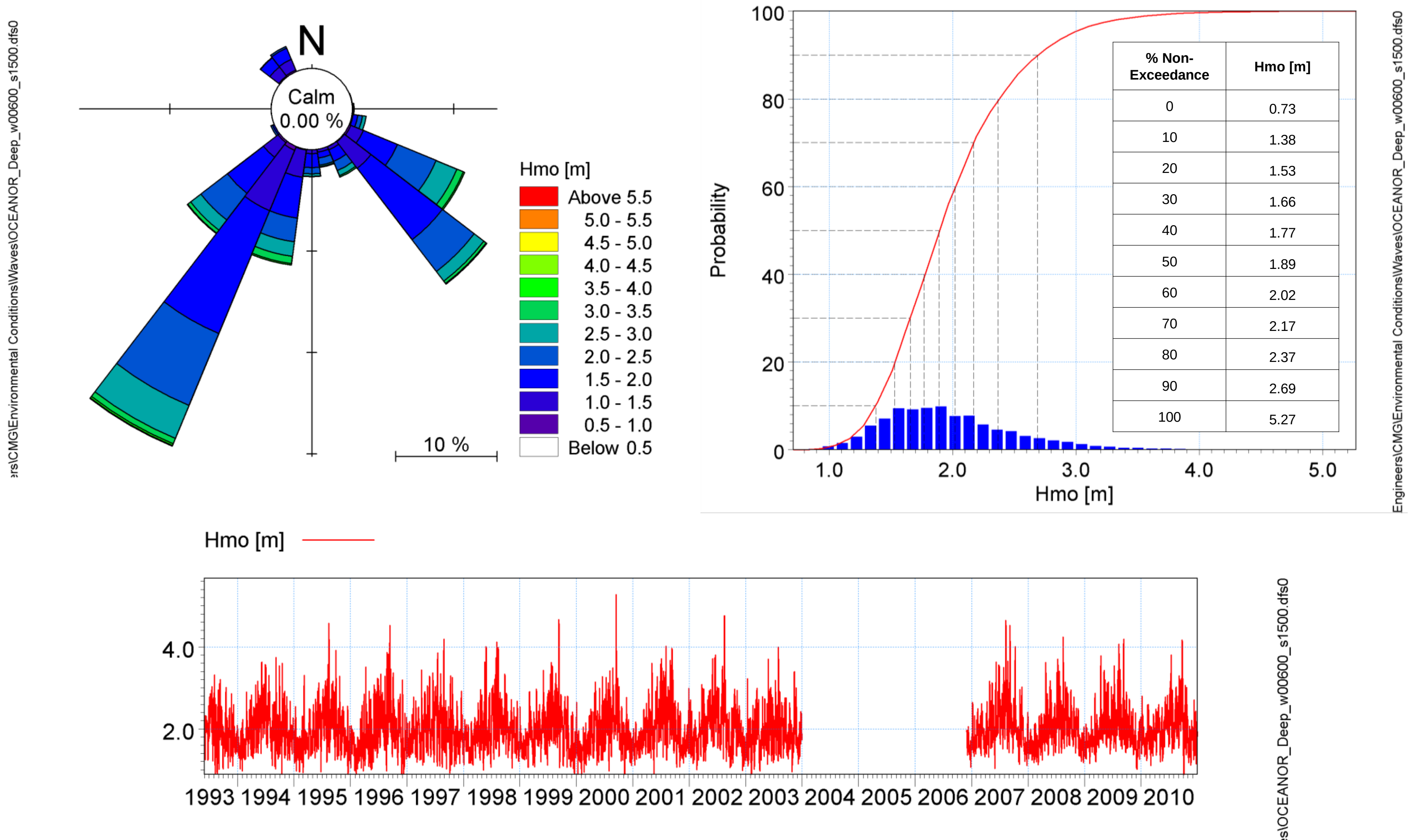
Geographic Coordinates

Rupert’s Bay ADCP: (Nearshore)	15° 54’59.46” S 5° 42’44.22” W
OCEANOR Point (Offshore)	15° 00’0.0” S 6° 0.0’0.0” W





Title:	Rupert’s Bay Permanent Wharf – Phase 2 **Location of Nearshore and Offshore Wave Data Points**	Figure No. 6



% Non-Exceedance	Hmo [m]
0	0.73
10	1.38
20	1.53
30	1.66
40	1.77
50	1.89
60	2.02
70	2.17
80	2.37
90	2.69
100	5.27

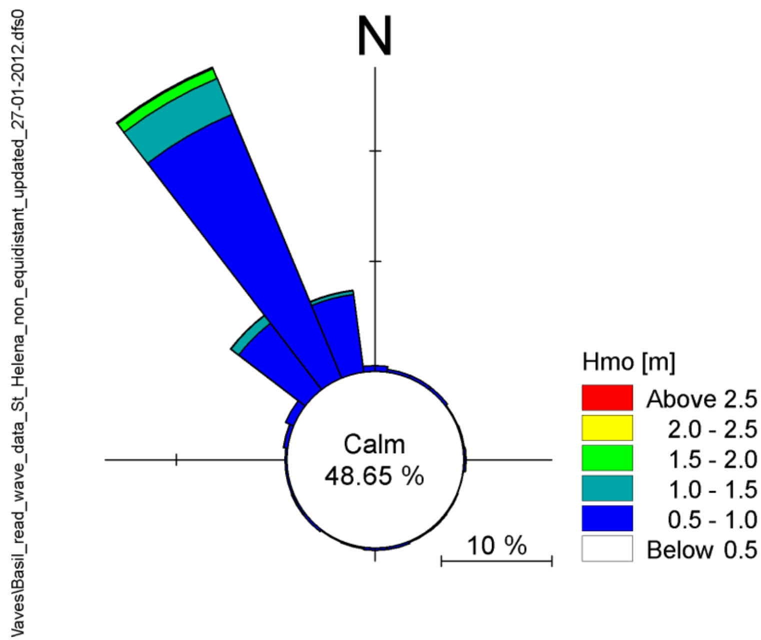
Title:

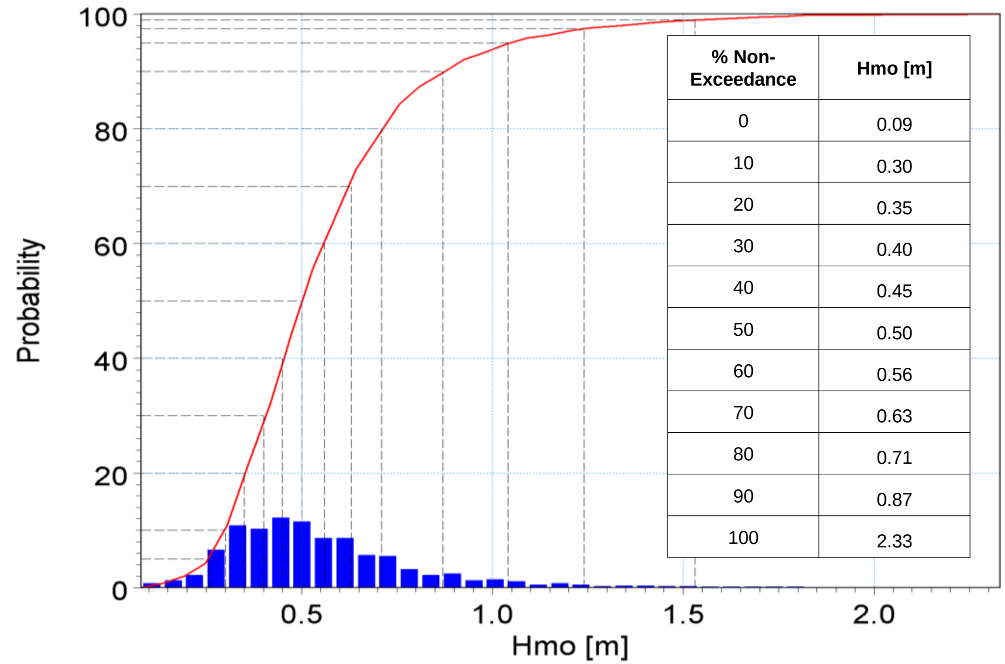
Rupert's Bay Permanent Wharf – Phase 2

OCEANOR Hindcast Data (Wave rose, Exceedance Curve and Time Series)

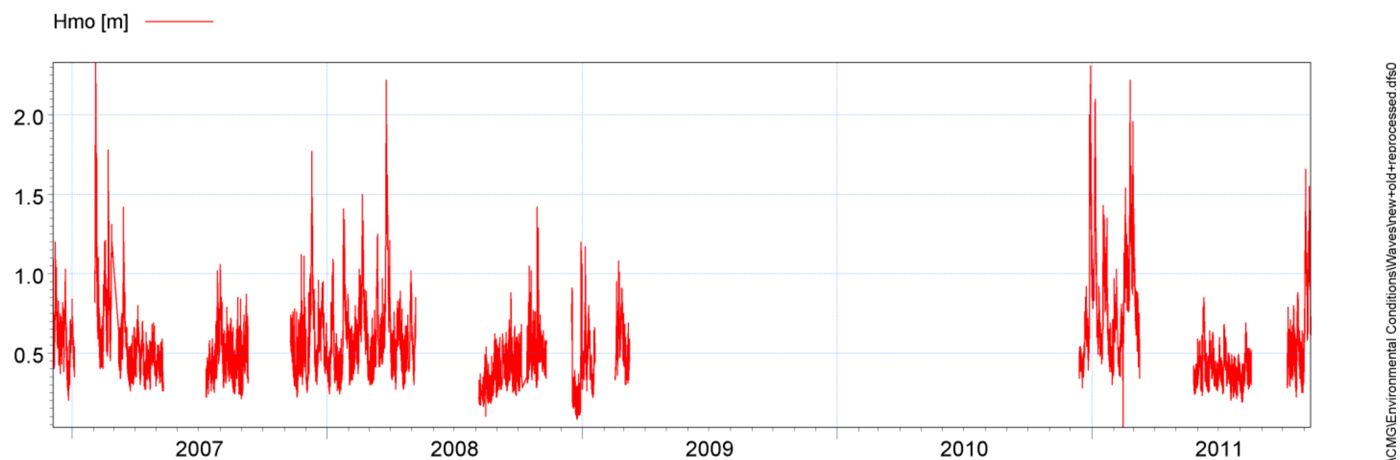
Figure No.

7





% Non-Exceedance	Hmo [m]
0	0.09
10	0.30
20	0.35
30	0.40
40	0.45
50	0.50
60	0.56
70	0.63
80	0.71
90	0.87
100	2.33



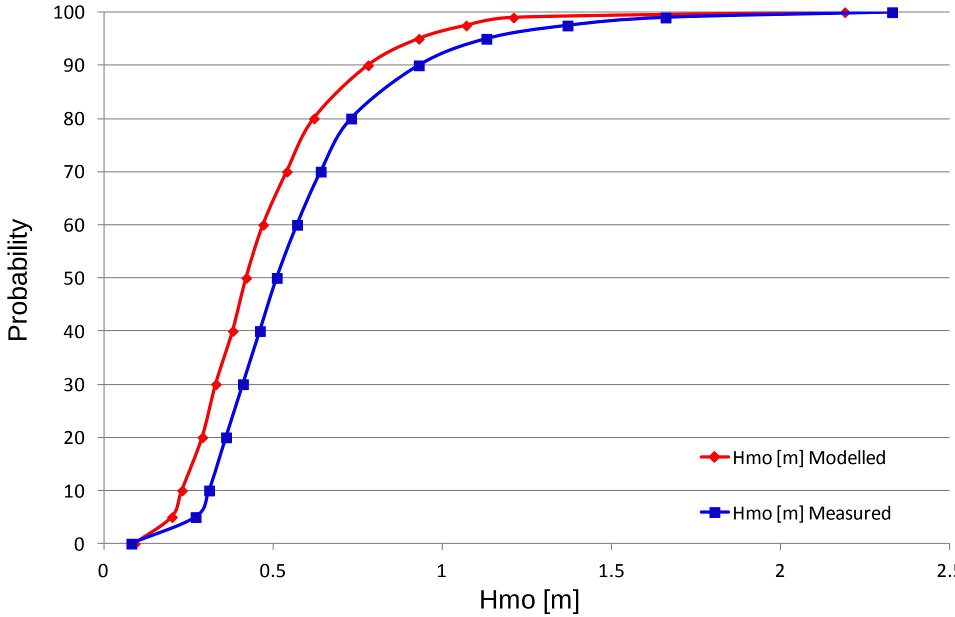
Title:

Rupert's Bay Permanent Wharf – Phase 2

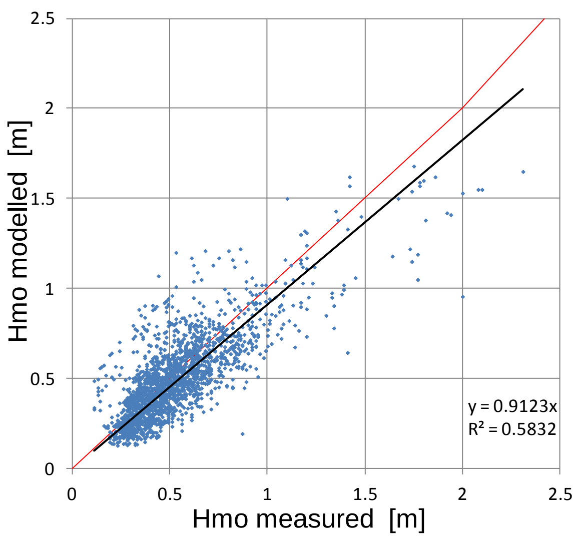
Measured Wave Climate at the ADCP Location

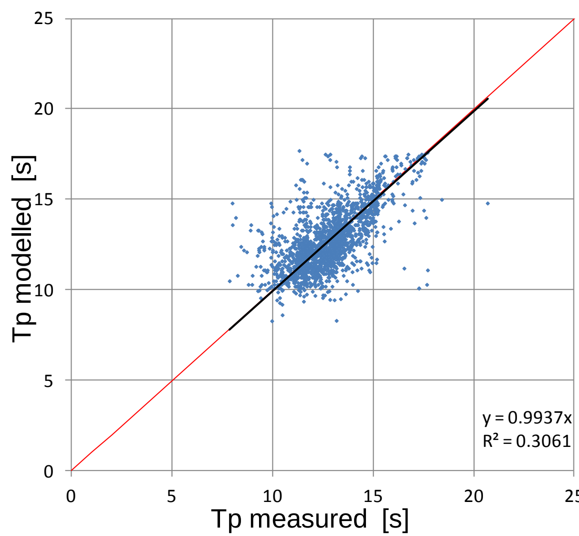
Figure No.

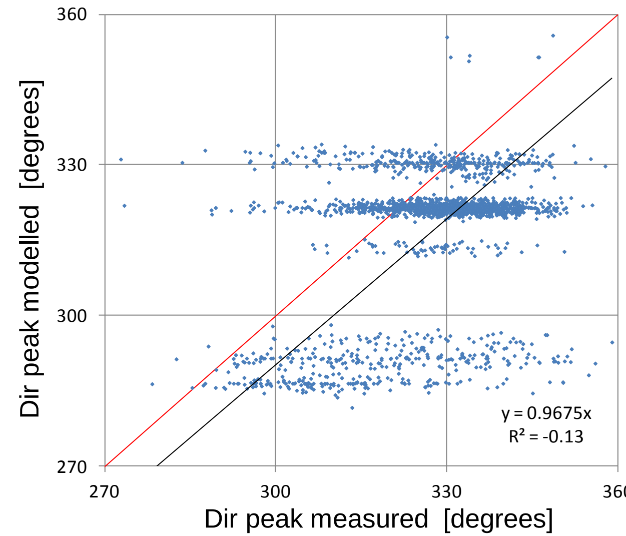
8



% Non-Exceedance	Hmo [m] Modelled	Hmo [m] Measured
0	0.09	0.08
10	0.23	0.31
20	0.29	0.36
30	0.33	0.41
40	0.38	0.46
50	0.42	0.51
60	0.47	0.57
70	0.54	0.64
80	0.62	0.73
90	0.78	0.93
100	2.19	2.33





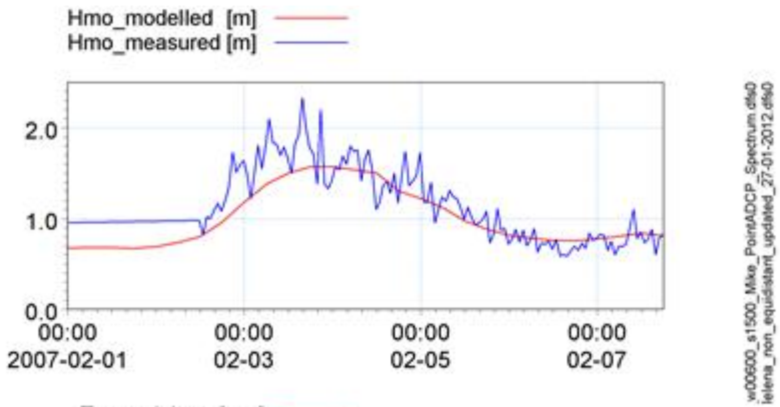


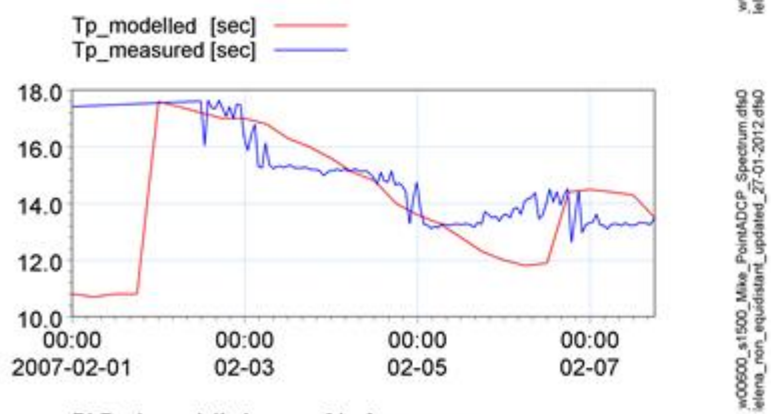
Title:

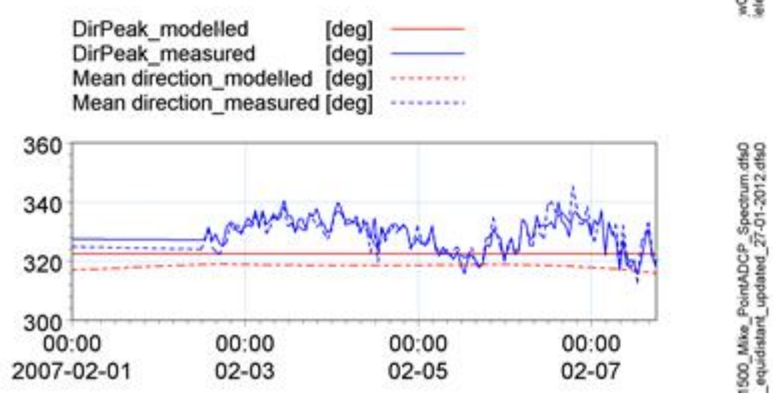
Rupert's Bay Permanent Wharf – Phase 2

Comparison of Measured versus Modelled Data at the ADCP location

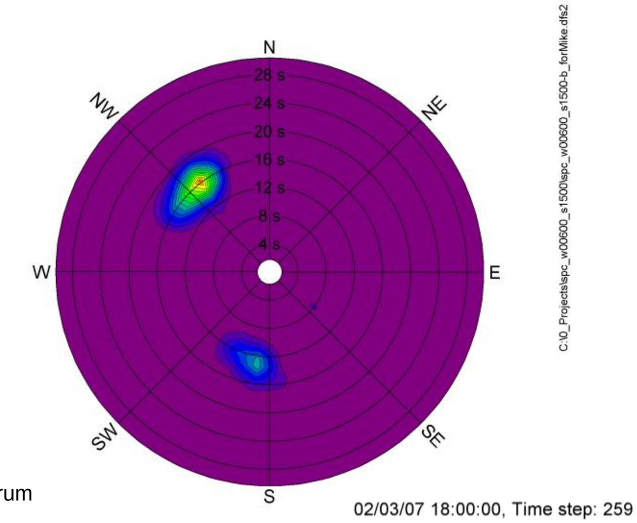
Figure No. **9**



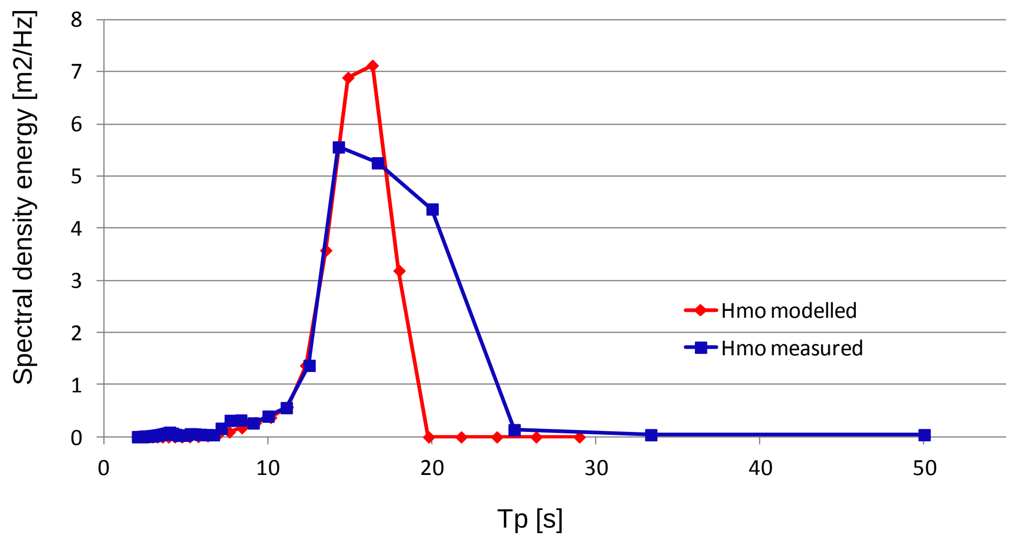




Deep water spectra from OCEANOR for February 3rd storm



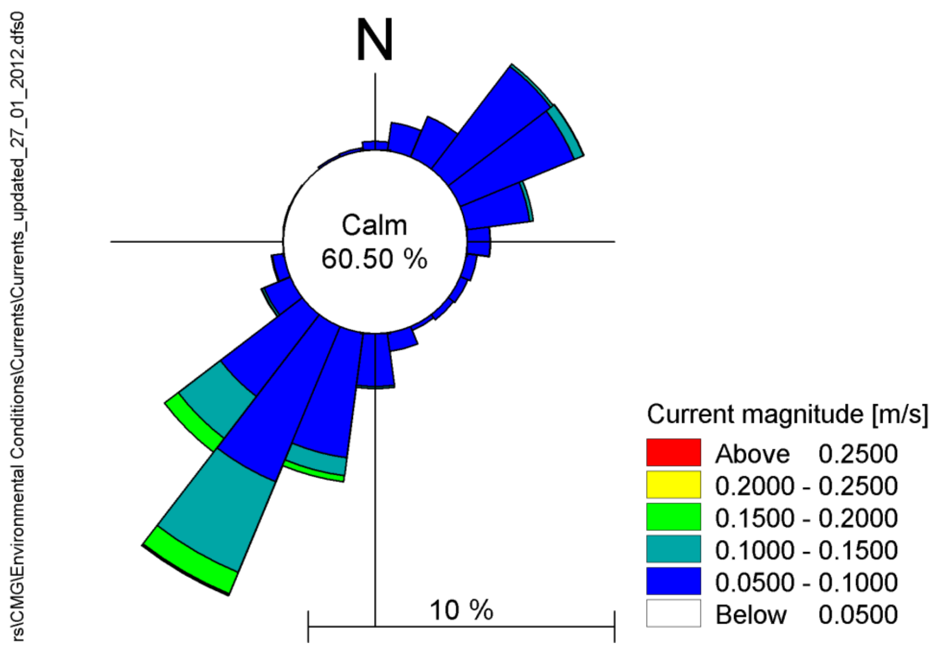
Storm 1D Frequency Spectrum Modelled v/s Measured for February 3rd 18.00hrs

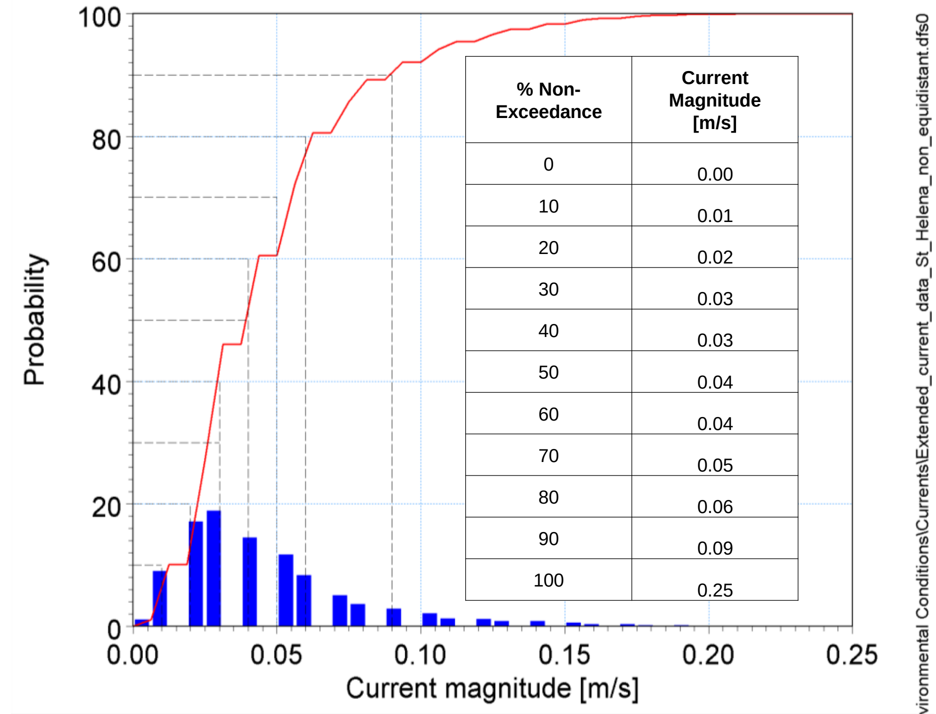


Title: **Rupert's Bay Permanent Wharf – Phase 2**

Comparison of Measured vs Modelled Results for a Storm Event between 2nd and 7th February 07

Figure No. **10**





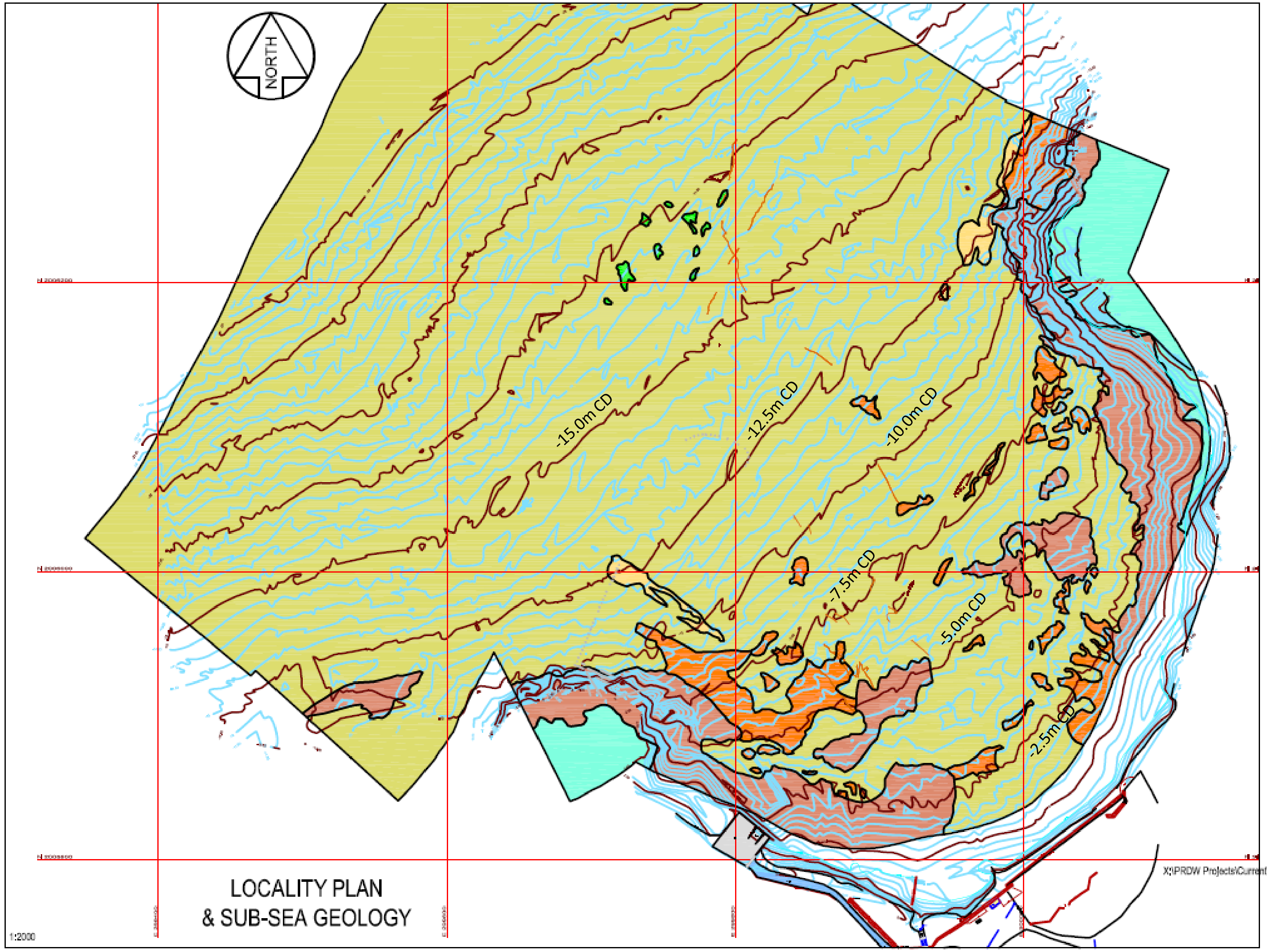
% Non-Exceedance	Current Magnitude [m/s]
0	0.00
10	0.01
20	0.02
30	0.03
40	0.03
50	0.04
60	0.04
70	0.05
80	0.06
90	0.09
100	0.25

Title:	Rupert's Bay Permanent Wharf – Phase 2 **Currents in Rupert's Bay as recorded at the ADCP location**	Figure No. **11**

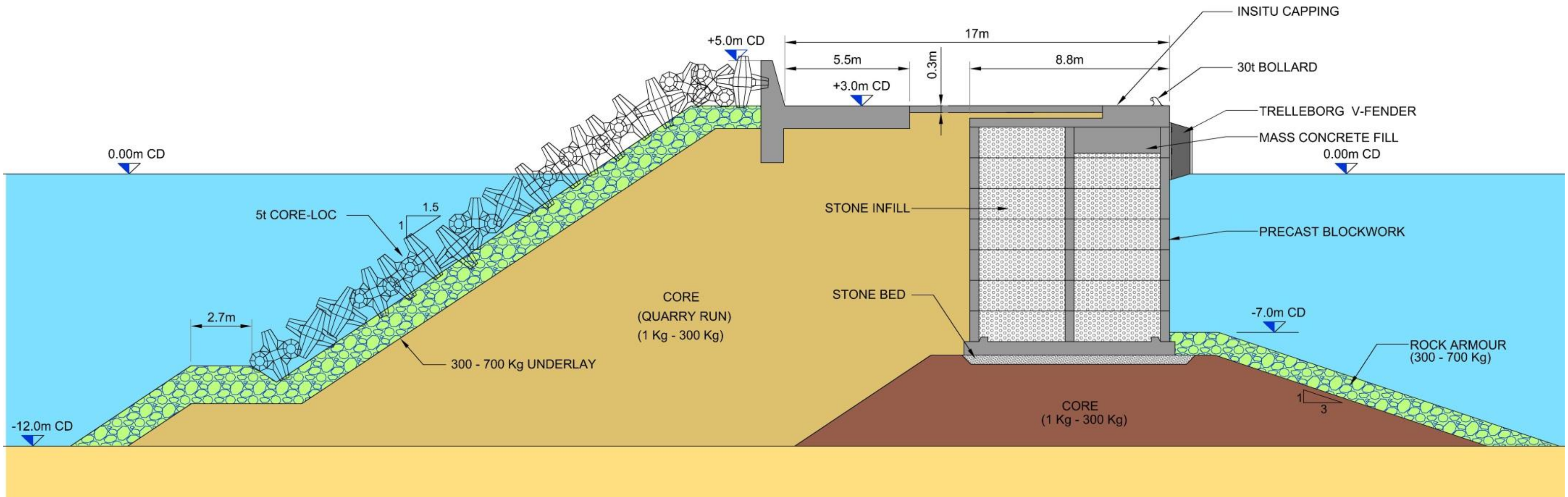
Legend:

- SHELF SANDS
- GRAVELS
- SCATTERED REEF
- RUGGED REEF
- NO DATA
- UNIDENTIFIED DEBRIS
- CHAINS OR ROPE

Reference: “Geophysical Survey and Data Quality Report of the Rupert's Bay Bathymetric, Side-Scan Sonar and ‘Pinger’ Seismic Survey, St Helena” Tritan Survey cc, Report 2006-, Project No. FR-2007-5633, 2006



Title:	Rupert’s Bay Permanent Wharf – Phase 2 **Rupert’s Bay Bathymetric and Sub-Sea Geology Plan**	Figure No. **12**



Title:

Rupert's Bay Permanent Wharf – Phase 2

Optimised Breakwater Concept with 5tonne Concrete Armour Units and Gravity Quay Wall

Figure No.

13

APPENDIX B

COASTAL PROCESSES REPORT

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

COASTAL PROCESSES REPORT (WAVES AND SEDIMENT)

REPORT NO. : 1097/02/02 REV B

APRIL 2013

PRESTEDGE RETIEF DRESNER WIJNBERG (PTY) LTD
CONSULTING PORT AND COASTAL ENGINEERS

Rupert's Bay Permanent Wharf – Phase 2: Coastal Processes Report (Waves and Sediment) Report No. 1097/02/02 Rev A					
Revision	**Date**	**Author**	**Checked**	**Status**	**Approved**
A	03 May 2013	CS/PMH	SAL	For Comment	
B	17 May 2013	CS/PMH	SAL	For Comment	a Hofl

Keywords: St Helena Island, Rupert's Bay, Permanent Wharf,

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

COASTAL PROCESSES REPORT (WAVES AND SEDIMENT)

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1. INTRODUCTION

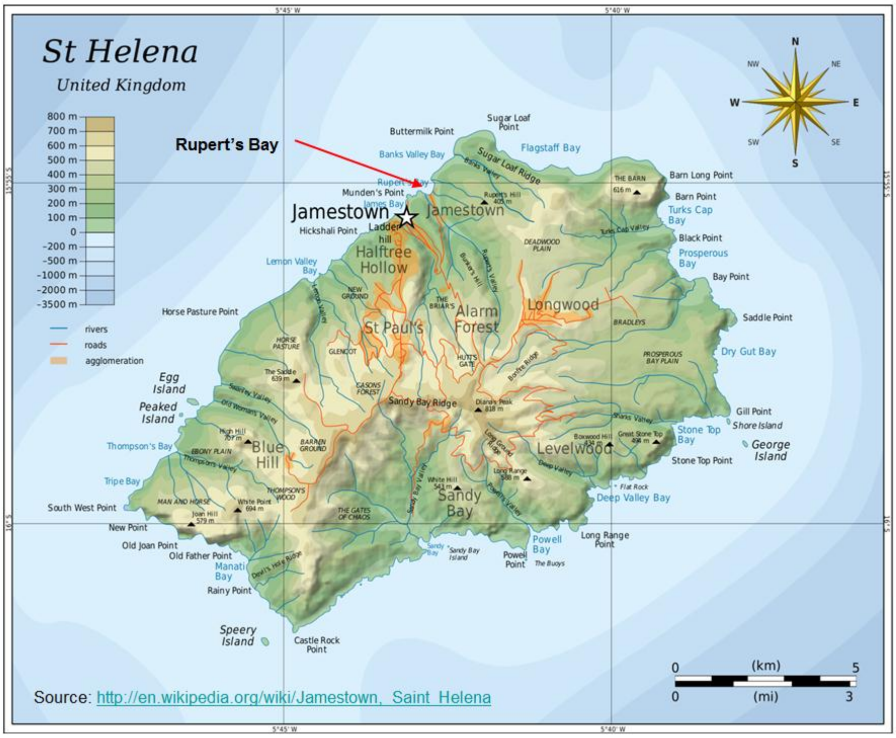
1.1 Background

The construction of a new airport on the island of St. Helena will require the existing port facilities on the island to be upgraded. These upgrades will include the provision of permanent wharf facilities for handling bulk cargo, petroleum products, general cargoes and, in the medium to long-term, containers. The site selected for this facility is Rupert's Bay on the North West coast of the island. The location of the site is shown in Figure 1-1.

Figure 1-1: St. Helena and Rupert's Bay locality map

Saint Helena Island

Geographic coordinates:	15° 56' S 5° 42' W



This document provides an analysis of the coastal processes in Rupert's Bay, which include water levels, wind, waves, and the sediment transport regime. The report focusses on the evaluation of the impact of the proposed development on these processes, and vice versa.

1.2 Scope of Work

The scope of work covered in this document includes the analysis of the following processes:

- Extreme water levels
- Local wind conditions
- Regional wave conditions

- Nearshore wave conditions
- Sediment transport regime

1.3 Report Structure

This report is composed of seven sections, including the current section. Section 2 introduces the measured and hindcast data that was used during the current investigation, whilst Section 3 and 4 present the extreme water level and wind analysis for Rupert's Bay respectively.

The regional wave modelling is discussed in Section 5, with the sediment transport assessment being introduced in Section 6. The report is concluded with a summary in Section 7.

1.4 Conventions and Terminology

The following conventions and terminology are used in this report:

- Wave direction is the direction from which the wave is coming, measured clockwise from true north.
- Wind direction is the direction from which the wind is coming, measured clockwise from true north.
- Current direction is the direction towards which the current is flowing, measured clockwise from true north.
- H_{m0} is the significant wave height, determined from the zeroth moment of the wave energy spectrum. It is approximately equal to the average of the highest one-third of the waves in a given sea state.
- T_p is the peak wave period, defined as the wave period with maximum wave energy density in the wave energy spectrum.
- Mean wave direction (Dir) is defined as the mean direction calculated from the full two-dimensional wave spectrum by weighting the energy at each frequency.
- Seabed and water levels are measured relative to Chart Datum. Chart Datum (CD) is 0.50 m below Mean Sea Level.
- All figures are orientated such that north is at the top of the figure.

Seasons for St. Helena Island are defined as follows:

Summer	1st December – 28th (29th) February
Autumn	1st March – 31st May
Winter	1st June – 31st August
Spring	1st September – 30th November

2. DATA

2.1 Tide Levels

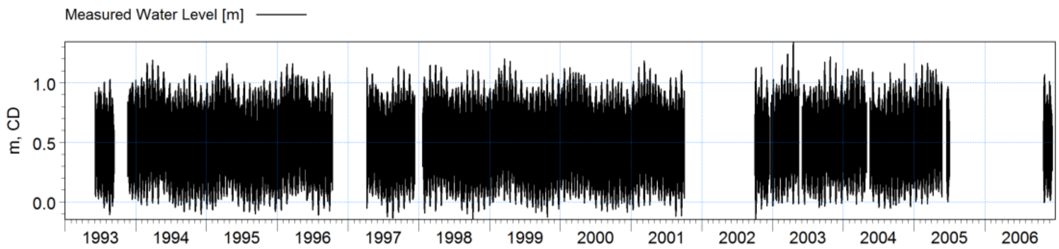
The astronomical tidal levels shown in Table 2-1 have been extracted from Admiralty Chart 1771 which was last updated on 20 October 2005. The tidal regime in St. Helena is characterised as semidiurnal with a maximum range (HAT to LAT) of 1.12 m and a mean sea level of 0.5 m above Chart Datum (CD). The tidal levels given in the table below have been confirmed by a tidal analysis of predicted tides performed as part of the current study.

Table 2-1: Predicted tide levels

Tide	(m, CD)
Highest Astronomical Tide (HAT)	+ 1.06
Mean high water springs (MHWS)	+ 0.94
Mean high water neaps (MHWN)	+ 0.72
Mean sea level (MSL)	+ 0.50
Mean low water neaps (MLWN)	+ 0.28
Mean low water springs (MLWS)	+ 0.07
Lowest Astronomical Tide (LAT)	- 0.06

Tidal measurements taken at Jamestown Harbour were obtained from the University of Hawaii Sea Level Centre (UHSLC, n.d.). The available data-set of hourly average water levels spans the period of 5 June 1993 to 14 December 2006. The hourly water levels represent the average of fifteen-minute values taken at 7.5 minutes before and after the hour. Accounting for gaps, the effective data-set length is 10.2 years. The total data-set is presented in Figure 2-1.

Figure 2-1: Measured water level at Jamestown, St. Helena

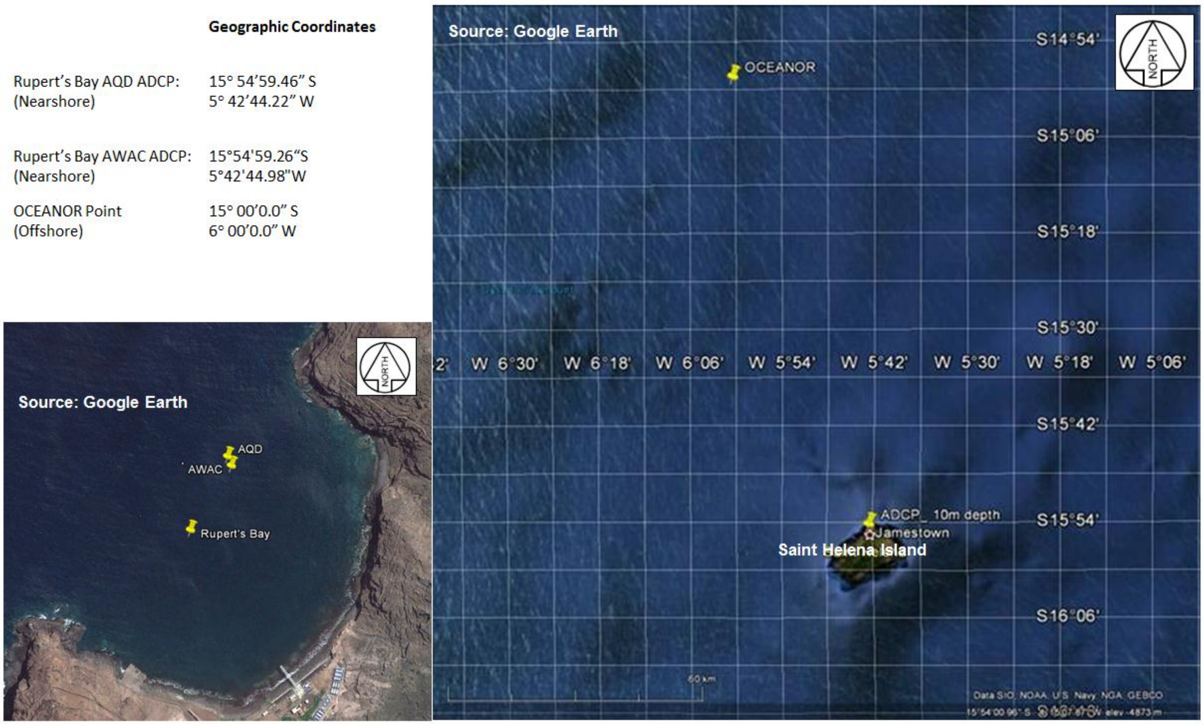


2.2 Waves

2.2.1 Offshore Wave Data

A calibrated hind-cast wave data-set as used in the previous Scoping Study (PRDW, 2012) was re-used during this study. The data was extracted from a model grid-point (15°S, 6°W), located approximately 100 km NNW of St. Helena Island, as indicated in Figure 2-2.

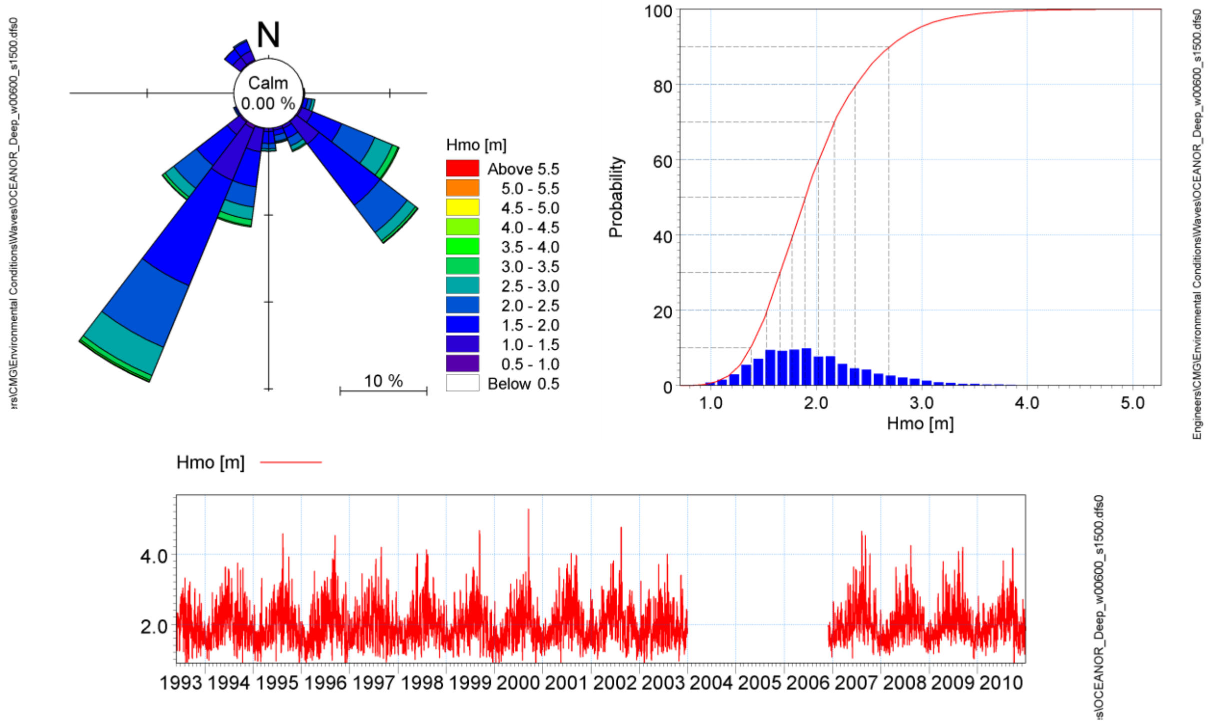
Figure 2-2: Locations of offshore and nearshore wave data



The wave data-set covers a duration of 15 years and corresponds to the period from January 1993 to December 2003 and from December 2006 to November 2011. The wave data-set consists of spectral and parametric wave information extracted at 6-hourly intervals.

Figure 2-3 includes a wave rose, time-series and a non-exceedance plot for the offshore wave data-set. The offshore wave climate is characterised by waves being generated from the SE'ly, SW'ly and the NW'ly direction sectors. These direction sectors account for 25.6%, 45.7% and 6.9% of the total wave data-set respectively.

Figure 2-3: OCEANOR offshore hindcast wave data at 15°S, 6°W



Waves being generated from the SE'ly sector can be characterised as wind seas with wave periods (T_p) ranging between 6 s and 10 s. Waves generated from the SW'ly and NW'ly direction sectors are predominantly characterised as swells, with wave periods (T_p) ranging between 12 s and 16 s and significant wave heights (H_{m0}) ranging from 0.5 m to 5.0 m. The median wave height (H_{m0}) for the complete data-set is 1.9 m, with 90% of the data-set containing wave heights below 2.7 m.

Considering the location of Rupert's Bay and the offshore model grid point, wave conditions most relevant for the design originate from the NW'ly direction sector. The highest wave height (H_{m0}) recorded from this sector in the 15 year data-set was 2.0 m.

2.2.2 Nearshore Wave Data

The wave climate in Rupert's Bay is dominated by the refracted south-easterly waves through most of the year. However, during the months of January to March heavy continuous swells or 'rollers' set in from the NNW causing a heavy break on the north-western coast of the island (Cartwright, 1971).

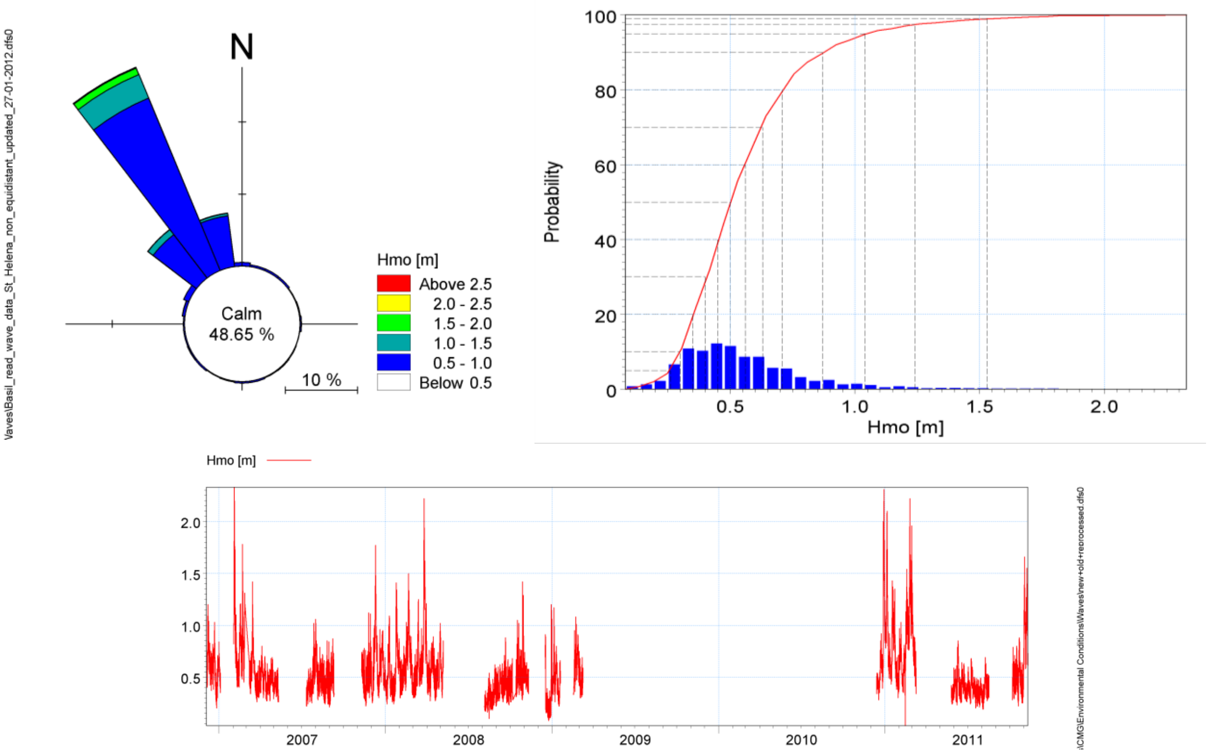
Local wave data has been collected at the site using an Aquadopp (AQD) Acoustic Doppler Current Profiler (ADCP) deployed in approximately 11 m water depth within Rupert's Bay (5°42.737" W, 15° 54.991" S). The AQD ADCP was set up using the following settings:

- Bin/Cell size: 1 m / 2 m
- Burst duration: 17 minutes
- Sampling rate: 1 Hz
- Sampling interval: 1 hour / 3 hours

Measurements were taken between December 2006 and November 2011, a duration of 3 years and 11 months. The location of the AQD ADCP is shown in Figure 2-2. Pressure and velocity readings were taken at an interval of 3 hours, with readings every hour between February 2007 and March 2007.

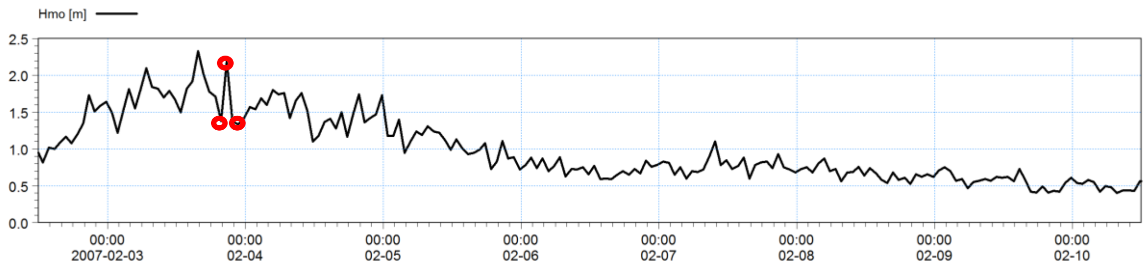
Figure 2-4 shows the data coverage over this period which shows intermittent data collection (36% complete). The data-set does however cover the critical periods of January to March over three years. The highest wave height (H_{m0}) recorded in this period was 2.3 m. Figure 2-4 also presents the wave rose and non-exceedance plot for the data-set.

Figure 2-4: Measured wave climate at the AQD ADCP location



Referring to Figure 2-5, significant short-term variability in the significant measured wave height in the nearshore measurements can be observed. During a storm on 3rd February 2007, the significant wave height (H_{m0}) increased from 1.38 m to 2.20 m within a one-hour period, reducing back down to 1.38 m within the next hour. These measurements are circled red in Figure 2-5 below.

Figure 2-5: Short-term variation of significant wave height (3rd & 4th February 2007)



This short-term variability in the significant wave height raised concerns regarding the quality of the measurements, prompting a second measurement campaign to be performed from the 22nd August to the 18th September 2012. During this one-month period, an AQD ADCP was re-deployed, with an additional Acoustic Wave And Current Meter (AWAC) ADCP being deployed approximately 15 m from the AQD in approximately the same water depth. The objective of this second measuring campaign was to test the measurements made by the AQD ADCP, by comparing these to the measurements of the AWAC ADCP.

The measurement periods of the two instruments are summarized in Table 2-2 below. Wave measurements were taken at one-hourly intervals at both instruments; using a 17 minute burst duration and a 1 Hz sampling rate. The measured wave data of the second measuring campaign is presented in Figure 2-6.

Table 2-2: Deployment of ADCP's in the second measuring campaign

Instrument	Start Date	End Date
AQD	2012/08/22 15:00:00	2012/09/18 08:20:00
AWAC	2012/08/13 16:00:00	2012/09/18 08:30:00

Lwandle Technologies undertook a detailed review of the measurements taken by both instruments, and concluded that wave heights showed general agreement, however the AQD ADCP recorded wave heights approximately 4 cm to 5 cm greater than those of the AWAC ADCP in most data records. Comparisons of individual spectra showed good agreement between the measured spectral energy for periods of increased wave height during the measurement program. Peak periods also showed good agreement between instruments, with minor differences noted between the data records (Lwandle, 2012).

In addition to the review by Lwandle Technologies, PRDW undertook a detailed review of the measurements during the above-mentioned storm on the 3rd February 2007. During this review, the measured water level variations were analysed in an attempt to explain the significant short-term variation in the measured wave heights.

Figure 2-6: Time series of H_{m0}, T_p and direction during the second measuring campaign

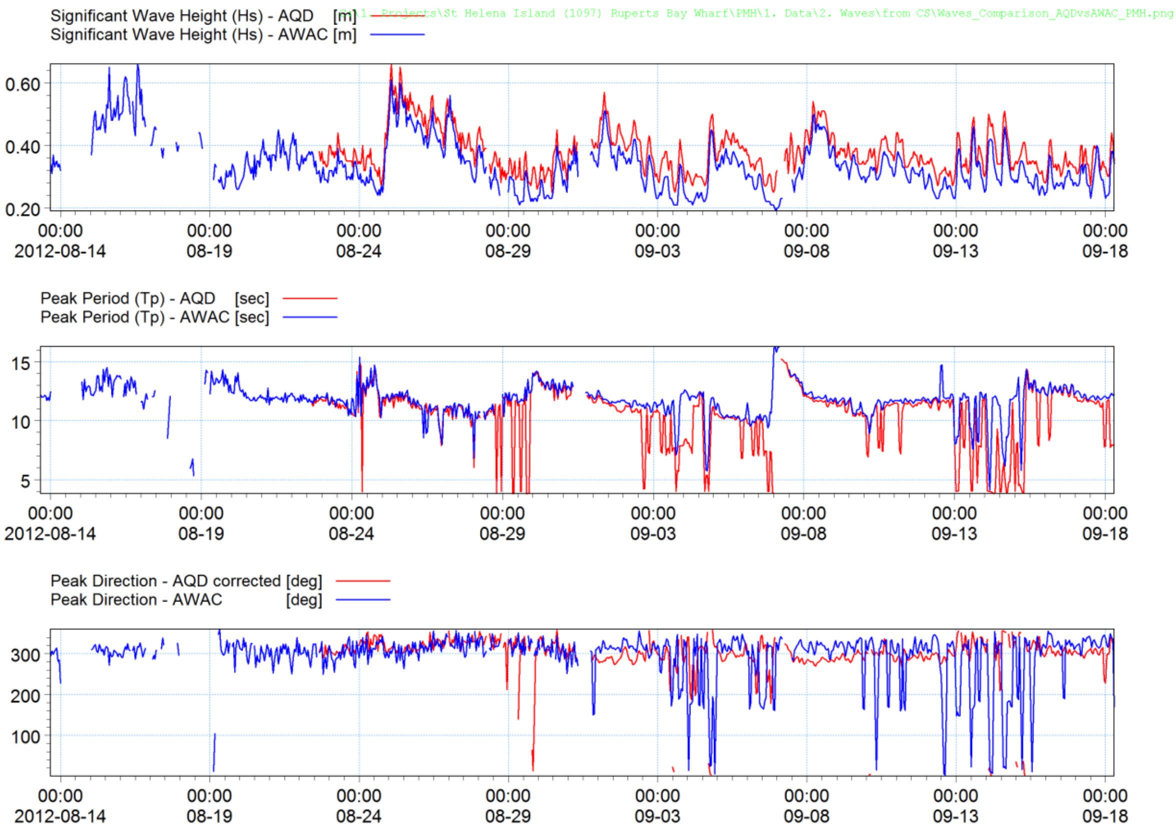


Figure 2-7 shows the water level variations for the storm on 3rd February 2007. Each of the three records is 17 minutes long, with the water levels sampled at 1 Hz. A total of 1 024 water level measurements are therefore included in each of the three records.

These water levels were used to determine three wave parameters, namely H_s, H_{m0} and $H_{1/3}$. H_s is calculated by multiplying the standard deviation of the surface elevation by four. H_{m0} is equal to four times the square root of the zeroth-order moment of the wave spectrum, whilst $H_{1/3}$ is determined by taking the average of the highest one-third of all waves, as determined by the zero-up crossing method. During this method, each wave is identified when the rising water level passes the zero datum.

Generally, the difference between these three parameters is small in deep water, and as such, H_s, $H_{1/3}$ and H_{m0} are often used as proxy for each other in coastal engineering designs. By determining each of the three parameters for the storm on 3rd February 2007, any spurious observations can be identified through a notable difference in H_s, $H_{1/3}$ and H_{m0}.

Figure 2-7: Water level measurements at 19h47, 20h47 and 21h47 on 3rd February 2007

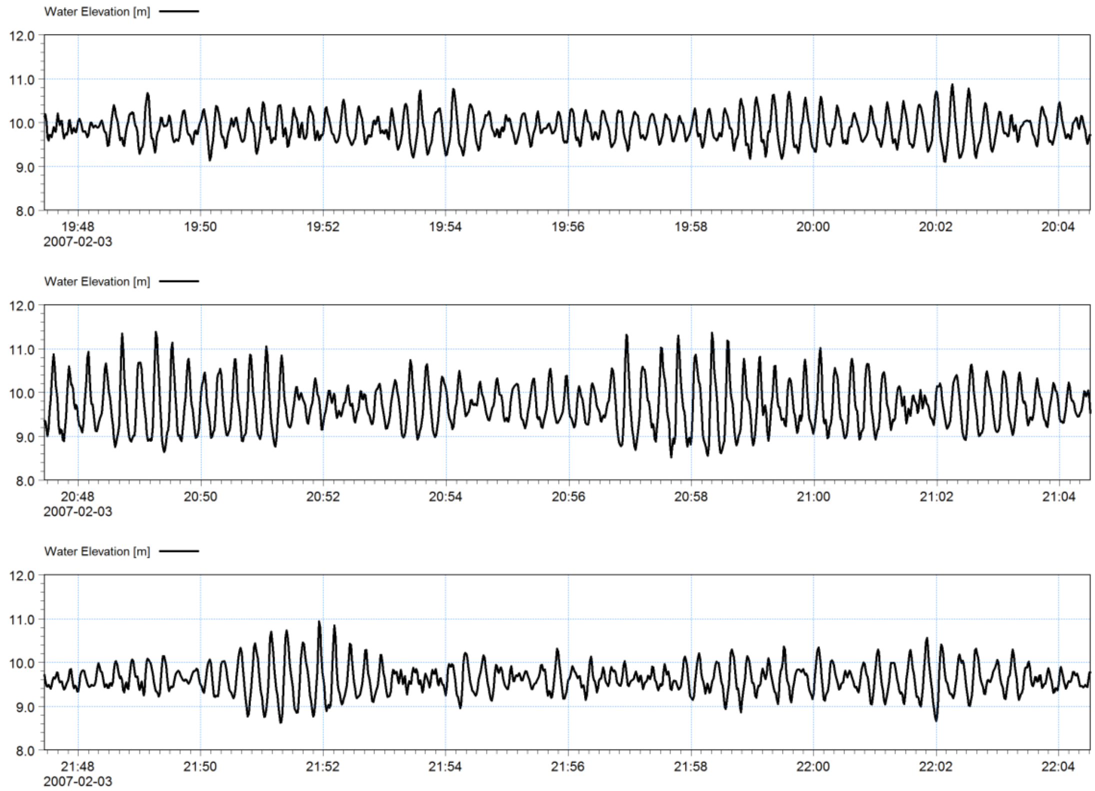


Table 2-3 summarizes the results of the analysis, compared to the wave height provided by the AQD ADCP. These results are graphically shown in

Figure 2-8.

From Table 2-3 and

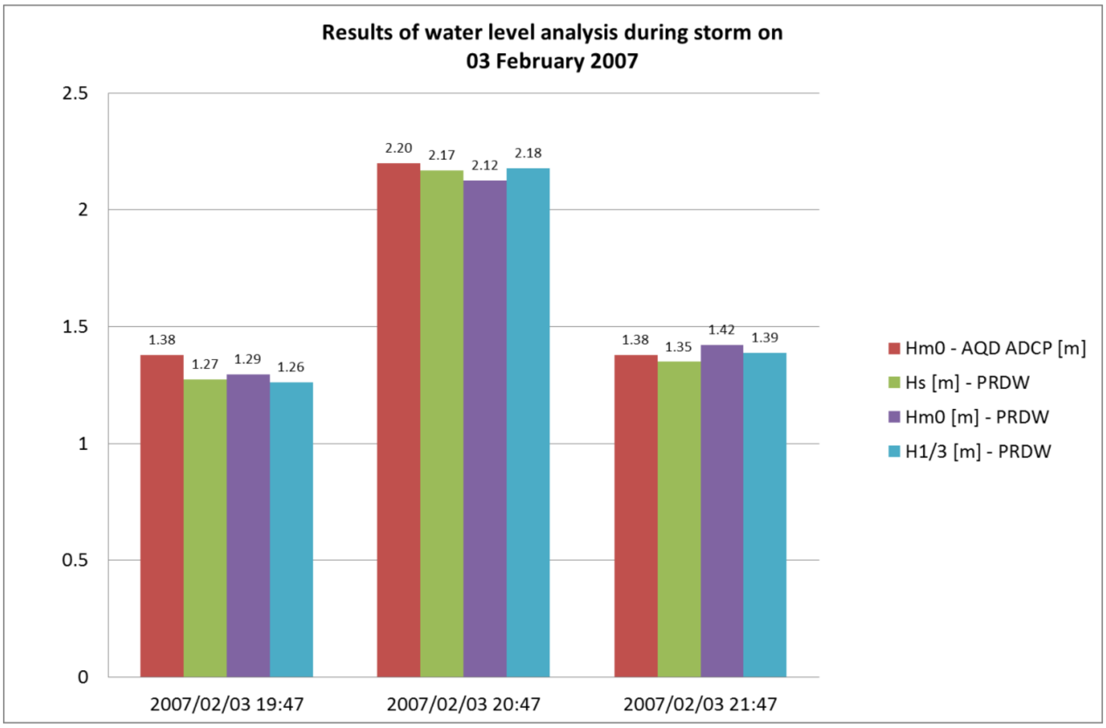
Figure 2-8 it is clear that the wave heights provided by the AQD ADCP agree well with those determined by the current analysis. In all cases, all wave heights are within 5% of each other.

From this, it is concluded that the nearshore wave measurements of the AQD are accurate, and are suitable for design purposes. The short-term variability of the significant wave height is likely to be a function of distinct wave groups in the wave train, which are not adequately sampled during the relatively short burst sampling duration of 17 minutes.

Table 2-3: Results of water level analysis during storm on 3rd February 2007

Date, Time	H_{m0} – AQD ADCP [m]	H_s [m]	H_{m0} – PRDW [m]	$H_{1/3}$ [m]
3rd Feb 2007, 19h47	1.38	1.27	1.29	1.26
3rd Feb 2007, 20h47	2.20	2.17	2.12	2.18
3rd Feb 2007, 21h47	1.38	1.35	1.42	1.39

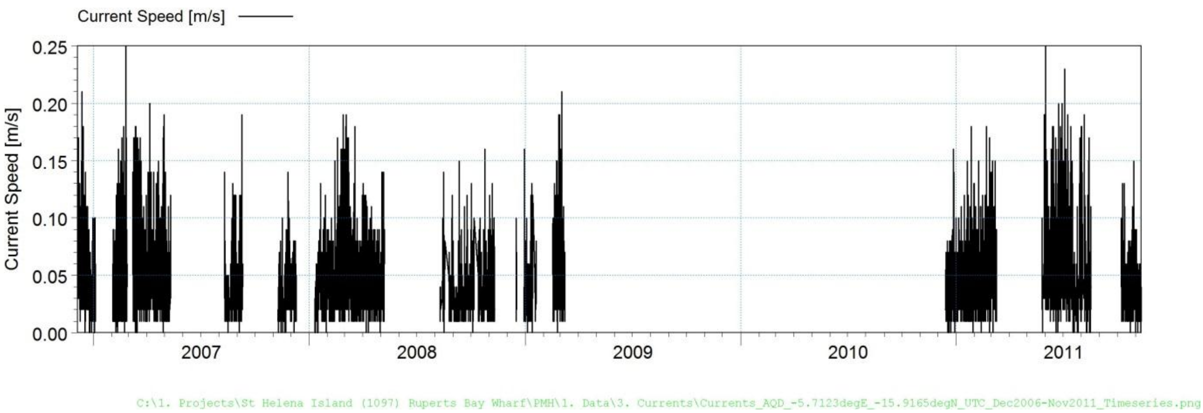
Figure 2-8: Results of water level analysis during storm on 3rd February 2007



2.3 Nearshore Currents

Current speed and direction were measured by the ADCP at the same time as the waves. Current data was therefore available at 3-hourly intervals between December 2006 and November 2011, with readings every hour in February and March 2007. The current data refer to a depth of approximately 7.5 m. The data-set from this measuring campaign is presented in Figure 2-9. The current speeds in Rupert's Bay are very low, with the highest current speed measured during this period being 0.25 m/s.

Figure 2-9: AQD current measurements during first measuring campaign (Dec 2006 to Nov 2011)



Further measurements were taken by both the AQD and AWAC ADCP's in August and September 2012 as part of the additional wave measurement campaign discussed in Section 2.2.2. These measurements were taken at 10-minute intervals throughout the water column, thereby providing more detail on the depth profile of currents in Rupert's Bay. The maximum depth-averaged current speeds for this period are 0.22 m/s for the AQD and 0.24 m/s for the AWAC. In the calculation of the depth-averaged current, the top two and three layers of the AWAC and AQD, respectively, were disregarded due to interference caused by proximity to the water surface.

For comparative purposes, the depth-averaged current speeds and directions of the two ADCP's for the first week of September are presented in Figure 2-10. The measurements from the two instruments were observed to agree very well. Subsequently, the AQD ADCP was chosen for further analysis since a longer set of measurements was available for this instrument than for the AWAC ADCP.

Current roses of the near-surface, near-bottom, mid-level and depth-averaged currents compiled from the AQD ADCP measurements during the 2012 measuring campaign are presented in Figure 2-11. A comparison of the roses indicates that the currents at the AQD location are generally uniform with depth.

An investigation into the mechanism forcing the currents revealed that the currents in Rupert's Bay include a tidal forcing. This is observed in the oscillations of the depth-averaged current direction with the tide as presented in Figure 2-12. However, the variations in the current magnitude are not caused by tides and are expected to be caused by some other forcing mechanism. Plots of the depth-averaged current magnitude against the local wind speed (discussed in Section 2.4.2) and the wave height measured by the AQD (discussed in Section 2.2.2) are presented in Figure 2-13. No correlation of wind or waves to the current speed can be observed from these plots.

Figure 2-10: Comparison of depth-averaged current speed and direction measured by AWAC and AQD

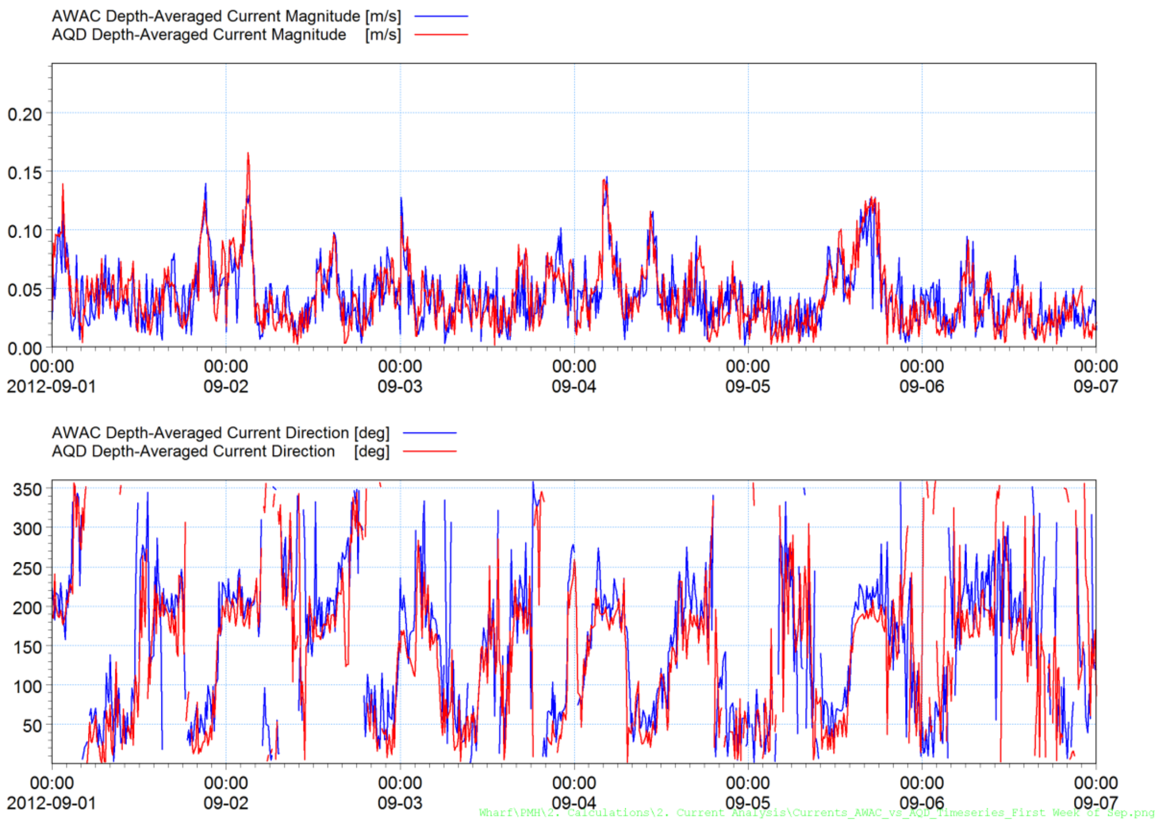


Figure 2-11: Currents measured by the AQD ADCP during the 2012 measuring campaign

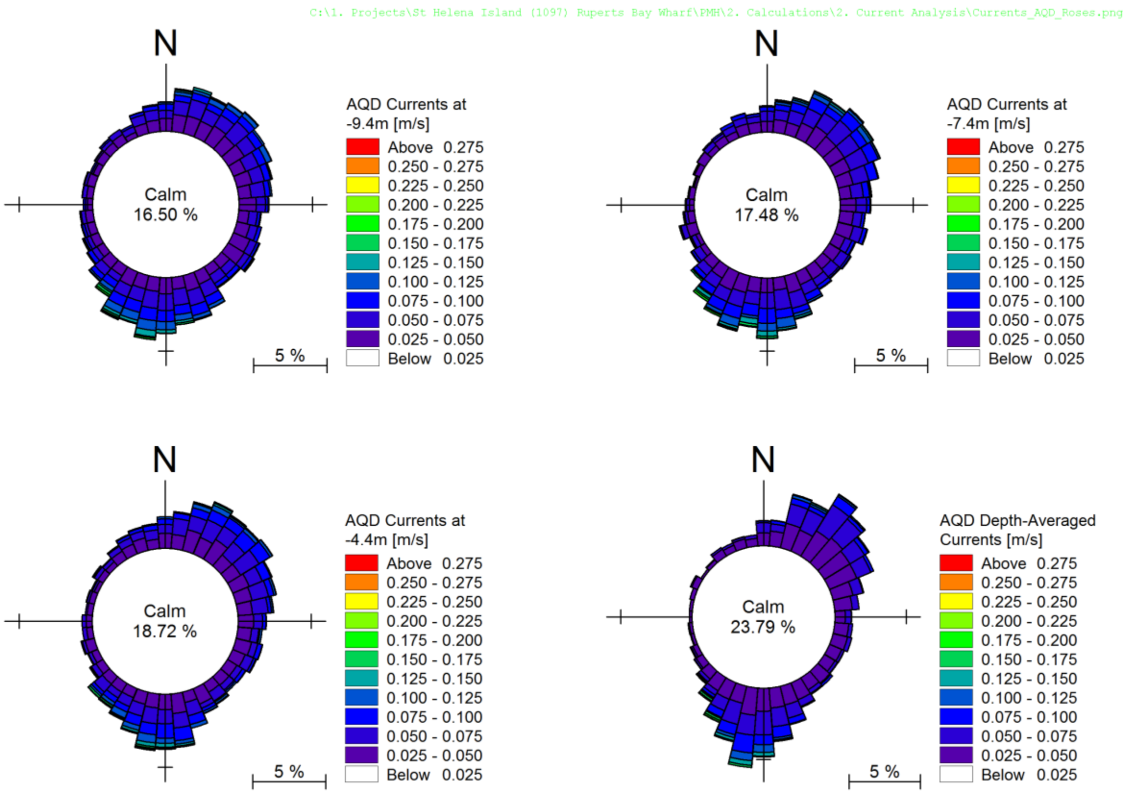


Figure 2-12: AQD depth-averaged current vs. predicted tide

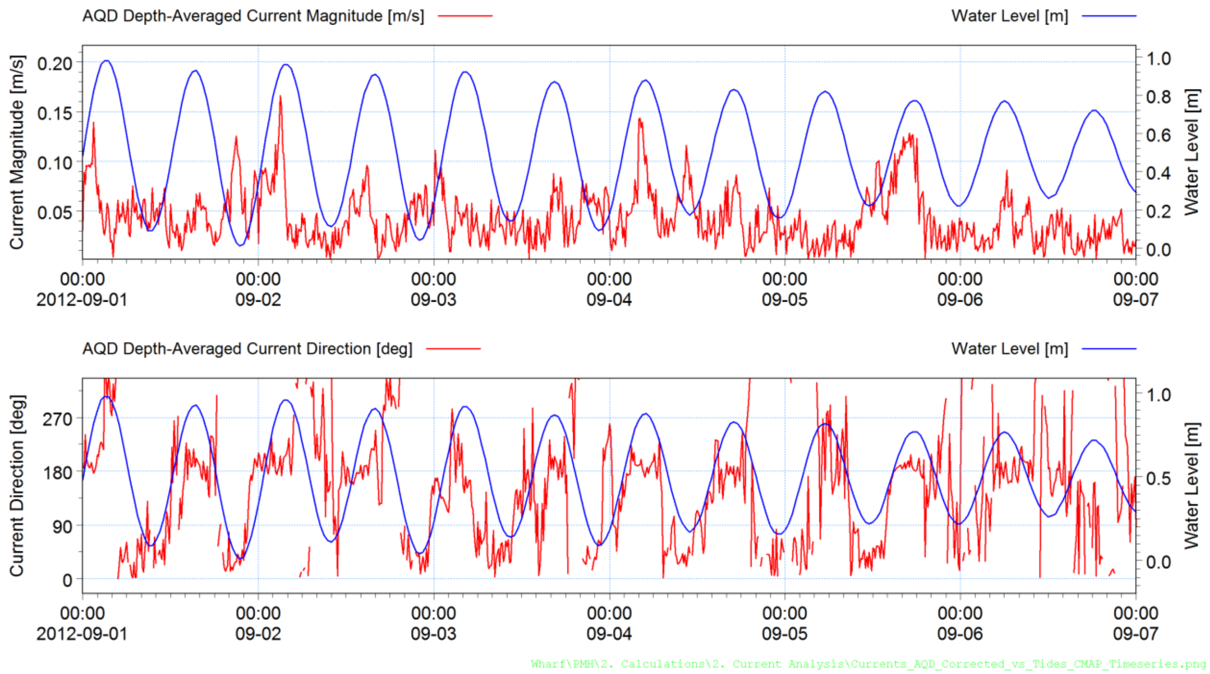
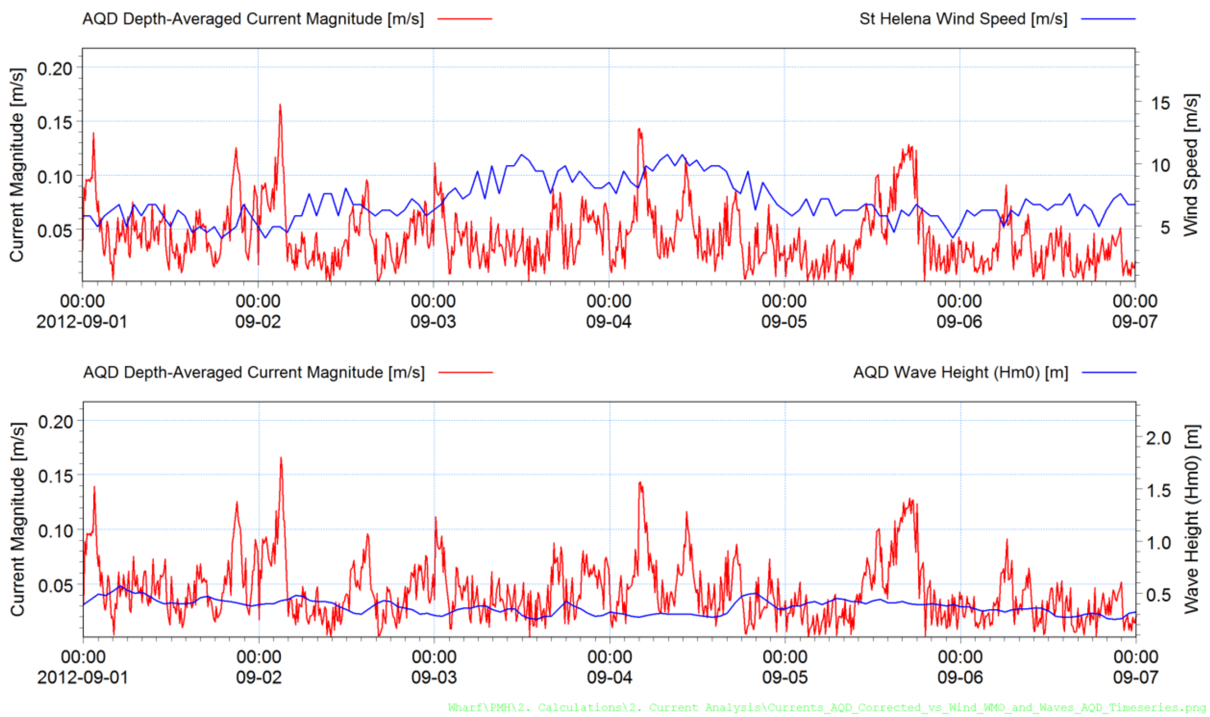


Figure 2-13: AQD depth-averaged current vs local wind speed and AQD wave height (H_{m0})

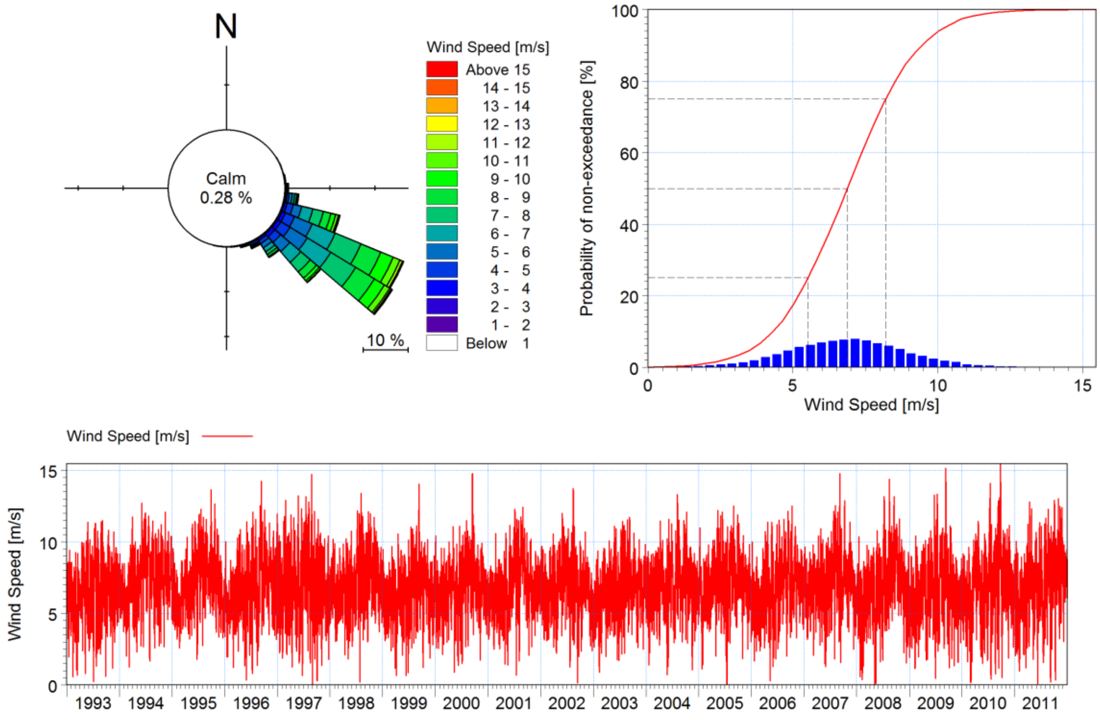


2.4 Wind

2.4.1 Offshore Wind Data

A 15 year offshore wind hindcast data-set was included in the OCEANOR offshore wave data-set. The data-set consists of hourly average wind speeds reported every six hours for the period of January 1993 to December 2011. The mean and maximum wind speeds are 6.9 m/s and 15.5 m/s respectively, with winds blowing almost permanently from the SE. Figure 2-14 provides a rose plot, non-exceedance graph and a time-series plot of the data-set.

Figure 2-14: OCEANOR hindcast wind data



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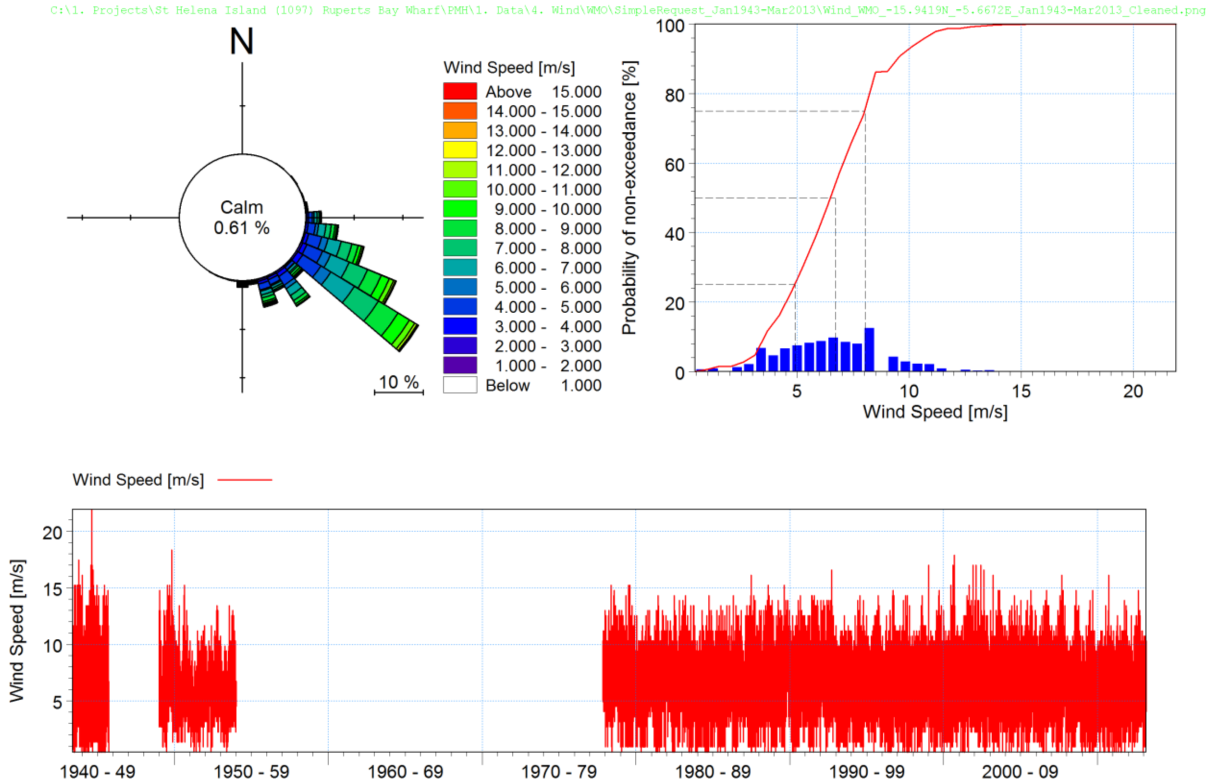
2.4.2 Local Wind Data

Local wind measurements were available at WMO Station Nr 61901, located on St. Helena Island at an elevation of +436 m. The position of the station in relation to Rupert's Bay is shown in Figure 2-15. Taking into account all other missing data, the total record length is 46.3 years. The data-set was cleaned by the manual removal of spikes which were judged to be non-physical and were probably caused by bad measurements. The maximum wind speed measurement in the cleaned data-set is 21.9 m/s with a mean of 6.5 m/s. Figure 2-16 provides a rose plot, non-exceedance graph and a time-series plot of the data-set.

Figure 2-15: Locations of nearshore and offshore wind data points

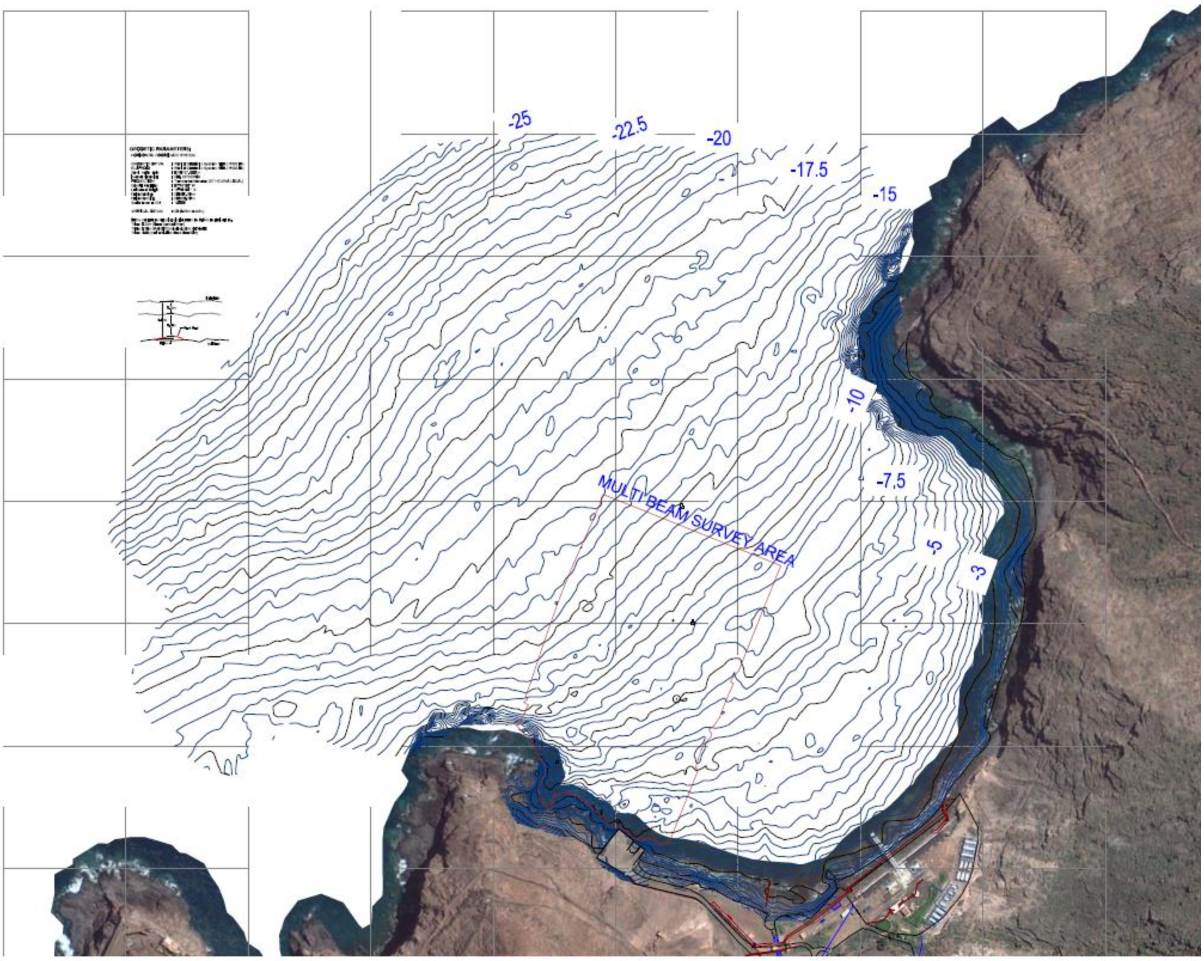
Figure 2-16: St. Helena measured wind data



2.5 Bathymetry

The results of a single-beam bathymetric survey performed in 2006 (Tritan, 2006) and a multi-beam survey performed in 2012 (Tritan, 2012) were used in this investigation. A datum discrepancy between the two surveys was discovered. Closer inspection revealed a change in the local control point which necessitated a downward adjustment of 0.22 m to the 2006 survey. In order to consolidate the full datum discrepancy between the two surveys, the 2006 single beam survey required a further downward adjustment of 0.43 m. The resulting bathymetry is presented in Figure 2-17.

Figure 2-17: Rupert's Bay bathymetry plan. Consolidated from the 2006 and 2012 surveys

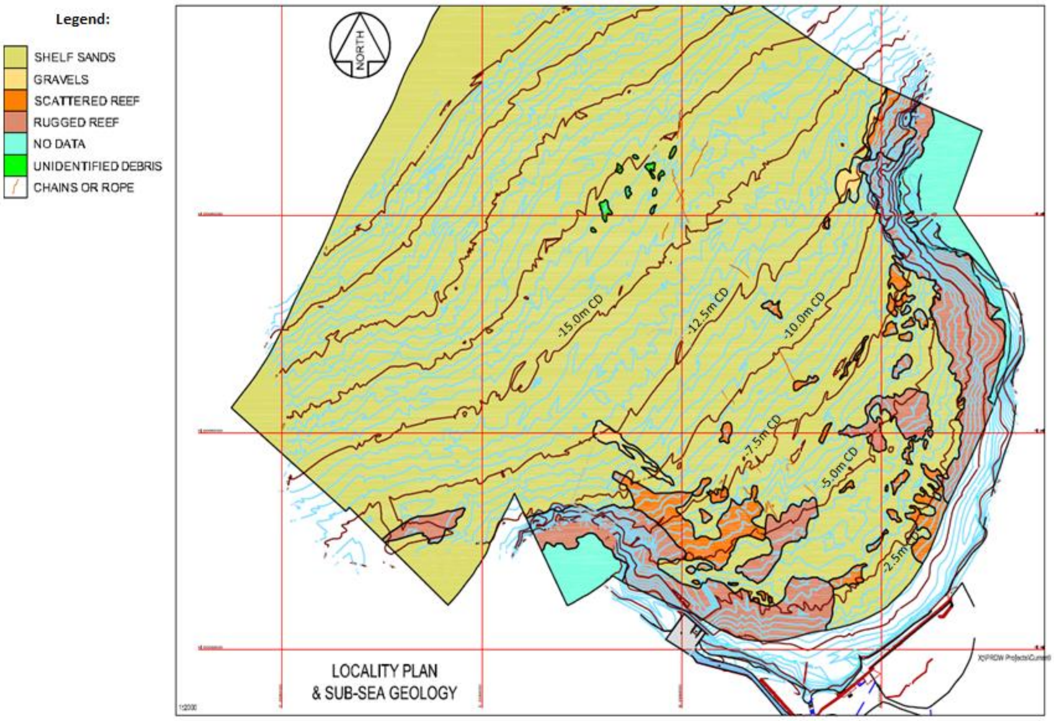


2.6 Seabed Characteristics

The seabed in Rupert's Bay is characterised by a layer of fine to medium grained sediments overlying the igneous bedrock. The bedrock is exposed around the headlands, and up to 200 m offshore, and may be expected to be comprised of hard to extremely hard rock (PRDW, 2012).

The sub-sea geology of Rupert's Bay is shown in Figure 2-18, indicating the presence of a number of scattered and rugged rock reefs. These are located predominantly along the shoreline, with the central area of the bay being covered in shelf sands.

Figure 2-18: Rupert's Bay sub-sea geology (Tritan, 2006)



2.7 Sediment Properties

2.7.1 Coastal Sediment Data

Sediment samples were taken at 12 locations along the periphery of Rupert's Bay (Fonternel, 2013b). The locations of the samples are presented in Figure 2-19 with the median particle diameters (D_{50}) and the geometric spreading of the samples listed in Table 2-4. D_{16} and D_{84} represent the 16^{th} and 84^{th} percentiles of the sample, respectively, i.e. 16% of the particles have smaller diameters than D_{16} and 84% have smaller diameters than D_{84}. The geometric spreading provides information on the grading of the sediment, and is determined as:

$$Geometric\ Spreading = \sqrt{D_{84}/D_{16}}$$

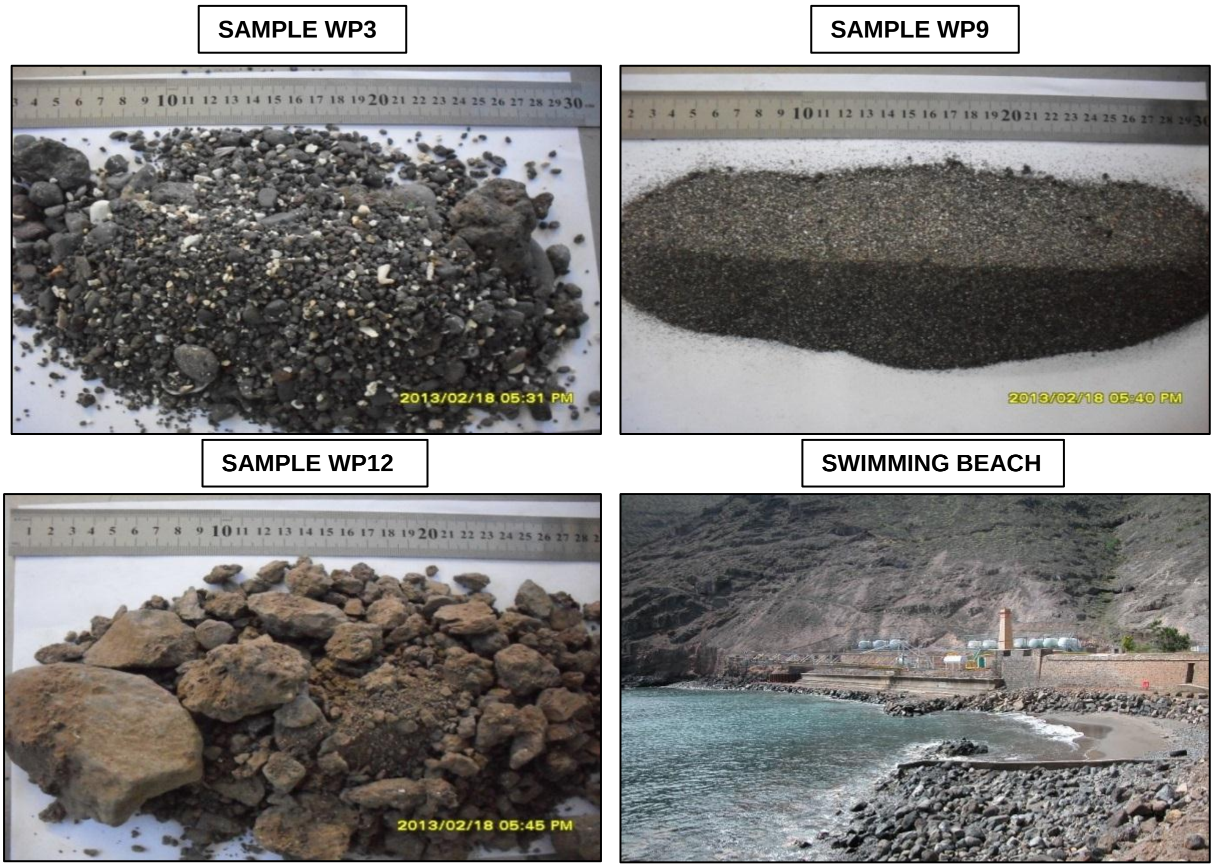
Figure 2-19: Locations of coastal sediment sampling sites

Table 2-4 : Grading analyses for coastal sediment data

Sample Name	Co-ordinates (UTM 30L)		D_{50} [mm]	D_{16} [mm]	D_{84} [mm]	Geometric Spreading
	X [m]	Y [m]				
WP1	209360.7	8238212	13.68	0.43	23.42	7.42
WP2	209363.9	8238190	5.81	1.59	22.75	3.79
WP3	209372.9	8238187	3.74	1.37	10.58	2.78
WP4	209369.8	8238194	2.22	0.86	4.75	2.34
WP5	209361.1	8238175	1.89	0.68	4.30	2.52
WP6	209361.2	8238169	2.00	0.24	39.95	12.95
WP7	209370.1	8238172	31.58	9.44	N/A	Insufficient Information
WP8	209585.2	8238110	1.31	0.52	3.00	2.41
WP9	209611.9	8238117	0.27	0.13	0.40	1.73
WP10	209644.5	8238133	19.34	2.55	39.95	3.96
WP11	209741.4	8238232	5.10	0.33	18.11	7.41
WP12	209783.8	8238408	8.62	1.31	26.50	4.50

From the survey, it is clear that the coastline of Rupert's Bay is characterised by medium to very coarse sediment. The sediment on the so-called swimming beach along the south-eastern extents of Rupert's Bay has a median grain diameter of approximately 0.27 mm. Photographs of the collected samples at locations WP3 (south-western cliff), WP9 (swimming beach) and WP12 (north-eastern beach) are presented in Figure 2-20, together with a photograph of the swimming beach.

Figure 2-20: Coastal sediment samples collected at Rupert's Bay (Fonternel, 2013b)



2.7.2 Bay-wide Sediment data

A bay-wide sediment survey covering the extent of Rupert's Bay was undertaken in which sediment samples were collected from the sea bottom at the sites indicated in Figure 2-21 (Fonternel, 2013b). At eight of the sites, no samples were taken as the bottom consisted of rock. A sieve analysis was conducted on each of the samples from the 17 remaining sites where samples could be collected. The median particle diameter (D_{50}) resulting from the corresponding grading analyses varied between 0.10 mm along the outer part of the bay to 0.18 mm close to the coastline. The results of the grading analyses are presented in Table 2-5.

Figure 2-21: Locations of bay-wide sediment sampling positions

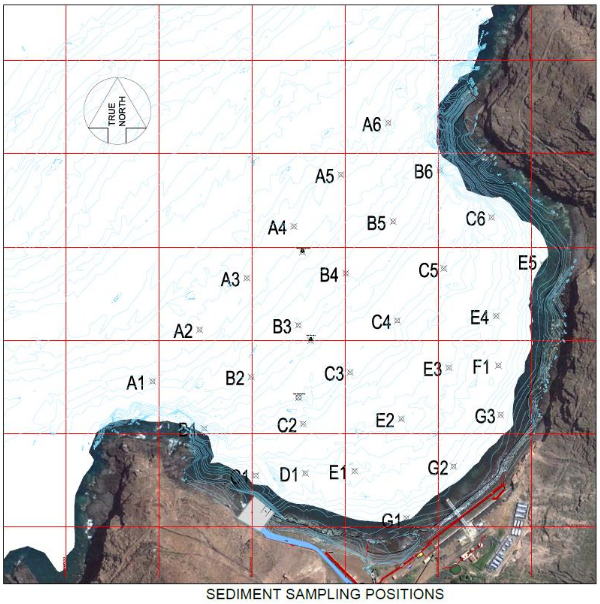


Table 2-5: Grading analysis for bay-wide sediment data

Site Number	Coordinates (UTM 30L)		D_{16} [mm]	D_{50} [mm]	D_{84} [mm]	Geometric Spreading	Mud Content (<0.075mm) [%]
	X [m]	Y [m]					
A1	209337	8238293	0.08	0.12	0.19	1.55	12
A2	209387	8238348		0.10	0.15		28
A3	209438	8238403		0.10	0.14		25
A4	209488	8238459		0.10	0.15		25
A5	209539	8238514		0.10	0.14		31
A6	209589	8238570	0.11	0.18	0.24	1.51	4
B1	209392	8238242	ROCK – No sample taken				
B2	209442	8238297	0.08	0.12	0.20	1.54	10
B3	209493	8238353		0.09	0.14		35
B4	209544	8238408	0.09	0.14	0.23	1.59	5
B5	209595	8238464	0.09	0.12	0.19	1.47	6
B6	209645	8238519	ROCK– No sample taken				
C1	209447	8238191	ROCK– No sample taken				
C2	209498	8238247	0.09	0.15	0.22	1.59	10
C3	209549	8238302	0.08	0.14	0.21	1.62	12
C4	209599	8238358		0.12	0.20		21
C5	209649	8238414	0.08	0.12	0.17	1.46	13
C6	209700	8238468	ROCK– No sample taken				
D1	209500	8238194	0.10	0.17	0.24	1.51	3
E1	209553	8238196	0.09	0.14	0.24	1.60	5
E2	209603	8238252	0.09	0.13	0.21	1.58	8
E3	209654	8238307		0.11	0.18		22
E4	209705	8238363	ROCK– No sample taken				
F1	209707	8238310	0.09	0.15	0.23	1.58	4
G1	209609	8238146	ROCK– No sample taken				
G2	209659	8238201	ROCK– No sample taken				
G3	209710	8238257	ROCK– No sample taken				

Observations regarding the position of reefs were also made by the diver. A summary of the observations is presented in Figure 2-22, which presents the spatial distribution of median particle diameters (D_{50}) determined in the bay-wide sediment analyses. The median diameter of the sediment on the swimming beach is also included in the figure. The data indicates a fining of the sediment with increasing distance offshore into deeper water. The spatial distribution of sediment data is further used in the modelling of sediment transport in Rupert's Bay, presented in Section 6.

Figure 2-22: Spatial distribution of median particle diameters (D_{50}) as determined in the bay-wide sediment assessment

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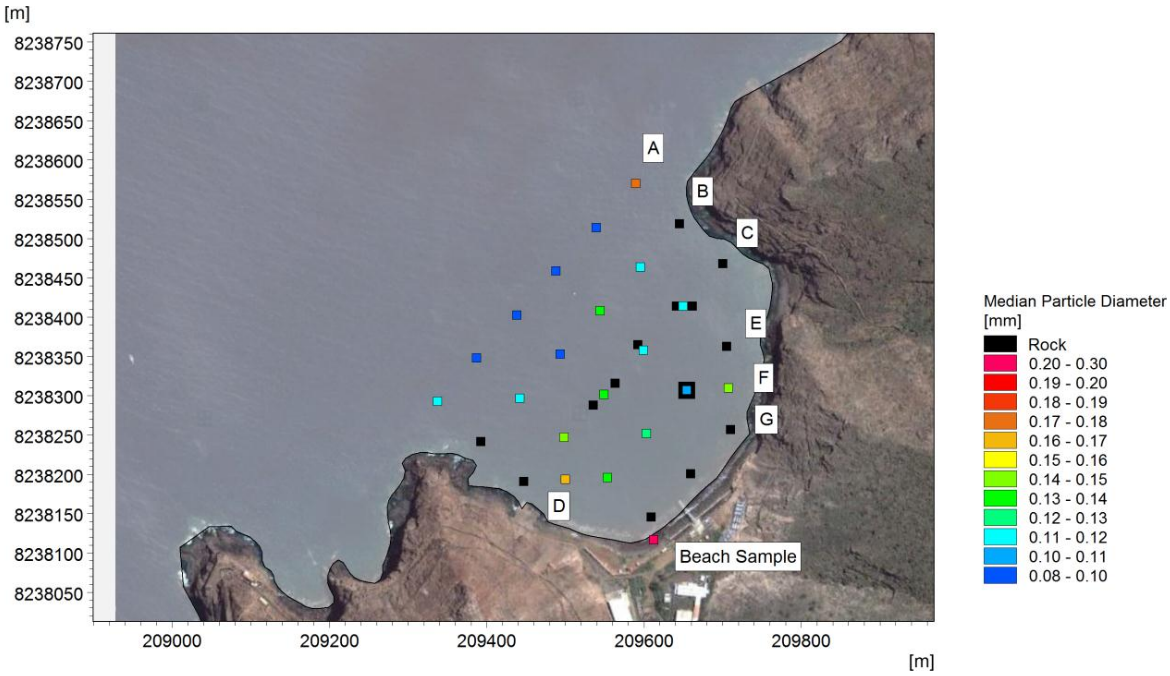


3. EXTREME WATER LEVEL ANALYSIS

3.1 Predicted Tidal Water Levels

The MIKE 21 Tidal Analysis and Prediction software (DHI, 2012a) was used to calculate the tidal constituents from the water level data measured at Jamestown. Predicted water levels were then calculated based on these tidal constituents using the same software.

3.2 Residual Water Levels

For the purpose of this report, storm surge is defined as the influence of meteorological effects such as winds and barometric pressure that result in the actual sea level being above or below the predicted astronomical tide level. The storm surge events have durations of hours to days and can thus be extracted from hourly tidal measurements.

The storm surge was estimated by calculating the residual water levels from the 10.2 years of hourly tidal measurements at Jamestown Harbour and performing an extreme value analysis (EVA) on these residuals. The residual water level was calculated as the predicted tide (described in Section 3.1) subtracted from measured hourly water level. The measured, predicted and residual tides are presented in Figure 3-1. The maximum and minimum residuals determined from the 10.2 year data-set are +0.22 m and -0.12 m, respectively.

An EVA was carried out using the MIKE ZERO EVA software (DHI, 2012b) to estimate the positive storm surge (actual water level higher than predicted tide) for the 1, 10, 20, 50 and 100 year return periods. An extreme value series was extracted from the input time series through the partial duration series approach with an average of six events per year. The extreme value series was subsequently analysed by fitting a three parameter Weibull distribution. The probability plot and extreme water levels resulting from the analysis are presented in Figure 3-2 and Table 3-1, respectively.

It is important to note that, since the tidal data-set only provides hourly water level measurements, higher frequency water level components such as tsunamis, edge waves and surf beat are not resolved in the tidal data. The effects of these phenomena are therefore not represented in the tidal residual.

Figure 3-1: Predicted tide, measured water level and residual water level at Jamestown, St. Helena

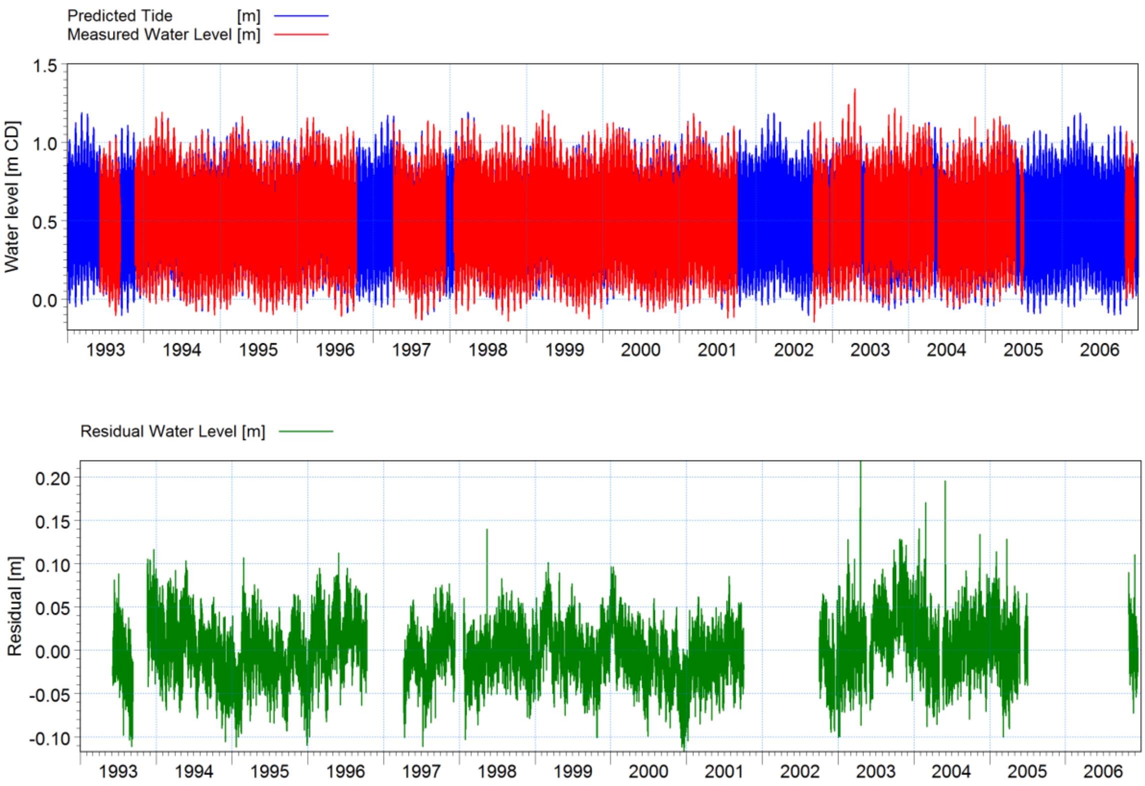


Figure 3-2: Probability plot of residual water level

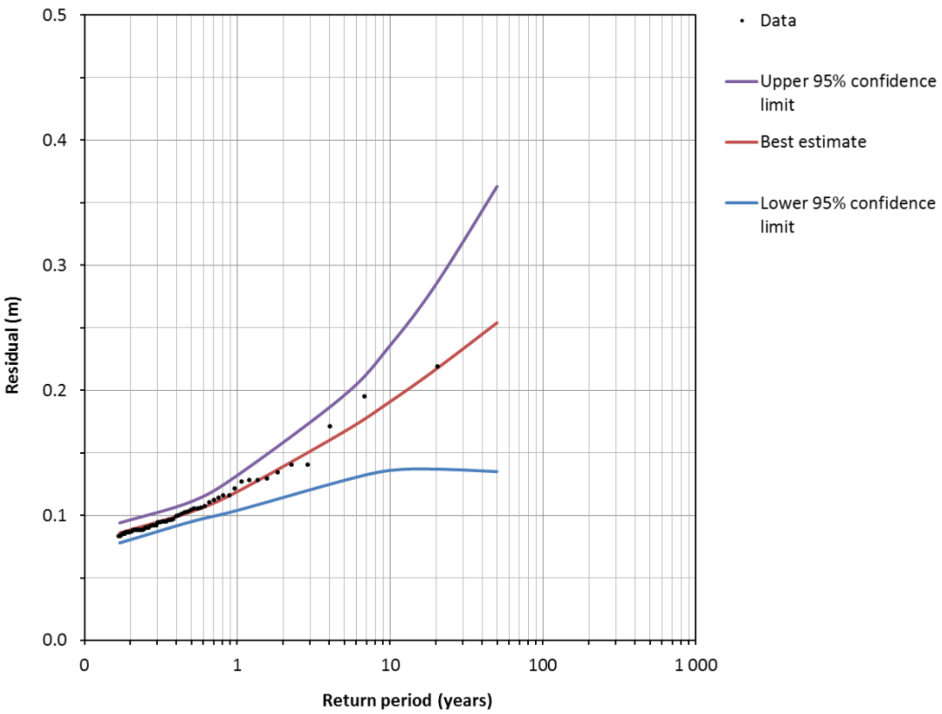


Table 3-1: Extreme residual water level

Return Period [years]	Residual Water Level [m]
1	0.12
10	0.19
20	0.22
50	0.25
100	0.28

3.2.1 Extreme Water Level

The effects of Climate Change were included in the calculation of the extreme water level according to the PRDW Position Paper on Climate Change (PRDW, 2010). For sea-level rise, mid-point projections were used which estimate a global average sea level rise of 0.8 m by 2100. For the 70 year design life, this results in a 0.65 m rise in sea level. Due to increasing wind speeds, storm surge is estimated to increase by 21% by 2100. For the 70 year design life, a 17% increase was imposed on the 1:100 year residual water level.

For the extreme water level, the residual water level corresponding to the 100 year return period was superimposed on the Mean High Water Spring Tide (MHWS). The extreme water level for design is presented in Table 3-2.

Table 3-2: Extreme water level

Parameter	Water level excluding Climate Change	Water level including Climate Change
Tide Level (MHWS)	+0.94 m CD	+0.94 m CD
Residual (1:100)	0.28 m	0.33 m
Sea-level rise	0 m	0.65 m
TOTAL WATER LEVEL [+m CD]	+1.22 m CD	+1.92 m CD

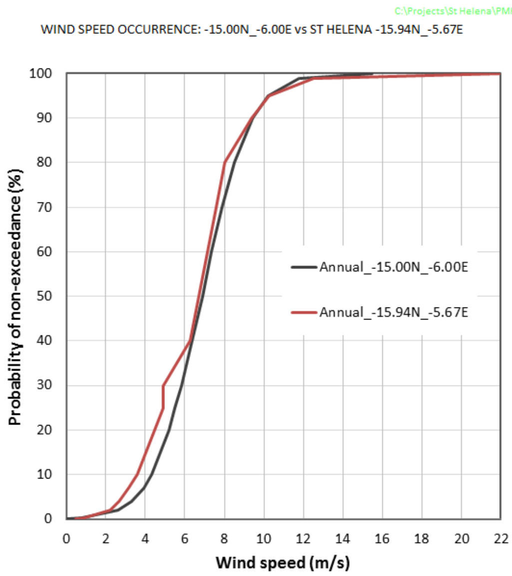
4. WIND ANALYSIS

The two wind data-sets discussed in Section 2.4 were analysed in an effort to determine the wind climate in Rupert's Bay. The offshore wind data (hourly averaged wind speed reported every six hours) set is located roughly 100 km NNW of St. Helena. The local data-set is located on St. Helena Island at an elevation of +436 m.

A comparison of the non-exceedance curves of the two data-sets presented in Figure 4-1 indicates that the wind speeds at the two sites sets are very similar. These speeds also agree well with a published mean wind speed of 6.9 m/s for Jamestown, St. Helena (UK Hydrographic Office, 2002). In Figure 4-2, wind roses of the two data-sets are presented. From this figure, an almost constant SE'ly wind direction is observed in both data-sets. Due to the NW-SE orientation of the valley leading down to Rupert's Bay, the wind direction in the bay is expected to remain more or less SE'ly.

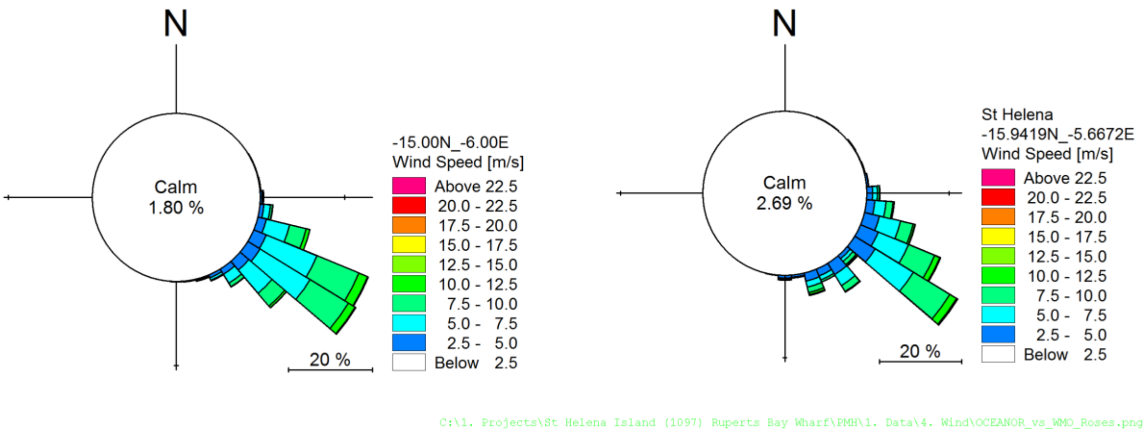
Since the two data-sets agree closely, engineering judgement was used to select a data-set to be used in further analyses. Considering the difference in elevation between the local meteorological station and Rupert's Bay, concerns arose regarding the applicability of the local wind data. However, based on the expert opinion of Professor Hannes Rautenbach (Pers. Comm, 2013), wind speeds in Rupert's Bay are unlikely to be higher than at the meteorological station because of friction, mountain turbulence and considering that the valley runs from the top of mountain downwards to the bay, rather than forming a low point in the island profile that would accelerate the wind. The measurements from local meteorological station were thus considered to be representative of the wind climate in Rupert's Bay.

Figure 4-1: Comparison of local (St. Helena) and offshore (OCEANOR) wind speeds



Probability of non-exceedance (%)	Wind speed (m/s)	
	Annual_-15.00N_-6.00E	Annual_ST HELENA_-15.15N_-5.67E
0	0.0	0.4
0.2	0.8	0.9
2	2.6	2.2
4	3.3	2.7
7	3.9	3.1
10	4.3	3.6
20	5.2	4.5
25	5.5	4.9
30	5.8	4.9
40	6.4	6.3
50	6.9	6.7
60	7.4	7.2
70	7.9	7.6
80	8.5	8.0
90	9.5	9.4
95	10.2	10.3
99	11.8	12.5
100	15.5	21.9

Figure 4-2: Comparison of local (St. Helena) and offshore (OCEANOR) wind directions



Some seasonality was observed in the local wind data, with stronger winds present in the spring and winter months. To indicate this seasonality, seasonal non-exceedance plots are presented in Figure 4-3.

Figure 4-3: Seasonal non-exceedance curves for wind speed measured on St Helena

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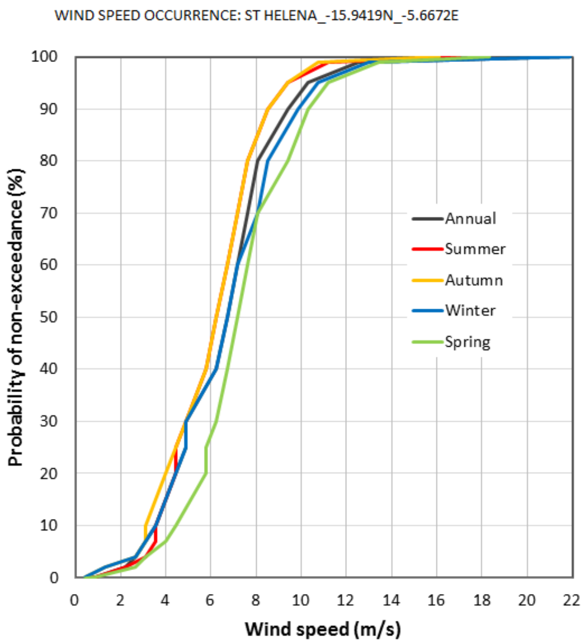


Probability of non-exceedance (%)	Wind speed (m/s)				
	Annual	Summer	Autumn	Winter	Spring
0	0.4	0.4	0.4	0.4	0.4
0.2	0.9	0.9	0.4	0.4	0.9
2	2.2	2.2	1.3	1.3	2.7
4	2.7	3.1	2.7	2.7	3.1
7	3.1	3.6	3.1	3.1	4.0
10	3.6	3.6	3.1	3.6	4.5
20	4.5	4.5	4.0	4.5	5.8
25	4.9	4.5	4.5	4.9	5.8
30	4.9	4.9	4.9	4.9	6.3
40	6.3	5.8	5.8	6.3	6.7
50	6.7	6.3	6.3	6.7	7.2
60	7.2	6.7	6.7	7.2	7.6
70	7.6	7.2	7.2	8.0	8.0
80	8.0	7.6	7.6	8.5	9.4
90	9.4	8.5	8.5	9.8	10.3
95	10.3	9.4	9.4	10.7	11.2
99	12.5	11.2	10.7	13.0	13.4
100	21.9	17.0	16.1	21.9	18.3

An extreme value analysis (EVA) was carried out on the wind measurements from the local wind station using the MIKE EVA Software (DHI, 2012b). An extreme value series was extracted from the data-set following the partial duration series approach using an average of five exceedances per year and an inter-event time of 72 hours. A three-parameter Weibull distribution was fitted to the series and the 1, 10, 50 and 100 year return periods were estimated. Uncertainty calculations were carried out with the Monte Carlo approach. The results of the EVA are presented in Figure 4-4 and Table 4-1.

Figure 4-4: Probability plot of wind speed on St Helena Island

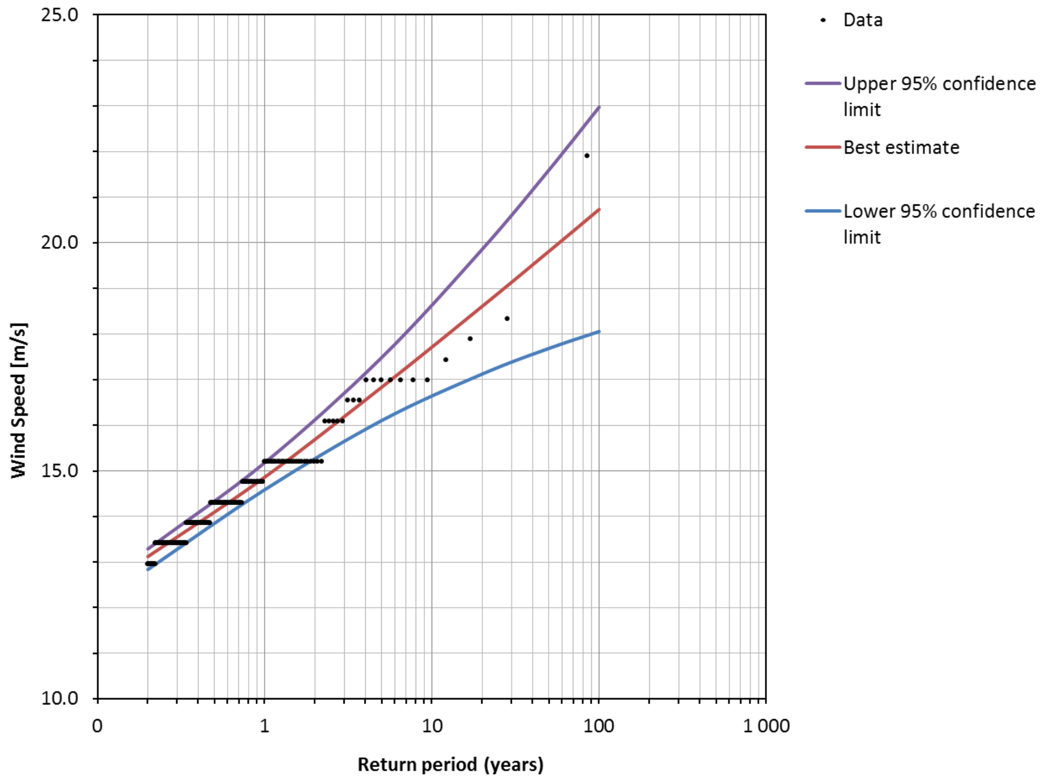


Table 4-1: Extreme wind conditions

Return Period [years]	Wind Speed [m/s]
1	14.9
10	17.7
50	19.8
100	20.7

5. REGIONAL WAVE MODELLING

5.1 Model Description

The *MIKE by DHI Spectral Waves Flexible Mesh* model was used for wave refraction modelling. The application of the model is described in the User Manual (DHI, 2012c), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2012d).

The model simulates the growth, decay and transformation of wind-generated waves and swells in offshore and coastal areas using unstructured meshes. The fully spectral formulation in quasi-stationary mode was used for modelling waves, which is based on the wave action conservation equation, where the directional-frequency wave action spectrum is the dependant variable.

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

In this study the model included the following physical phenomena:

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

5.2 Modelling Strategy

Regional wave modelling has been performed in two distinct modes. The first of these modes is the analysis of storms, to determine the design conditions for the marine structures of the proposed permanent wharf development. The second mode includes the simulation of operational wave conditions within Rupert's Bay. This information is used to analyse vessel motions and potential downtime.

The storm analysis has been performed by simulating only the events resulting in a nearshore wave height of greater than 0.8 m. In this way, a realistic representation of the storms, as well as their frequency is obtained. By excluding the calm conditions between storm events, computational requirements are minimized.

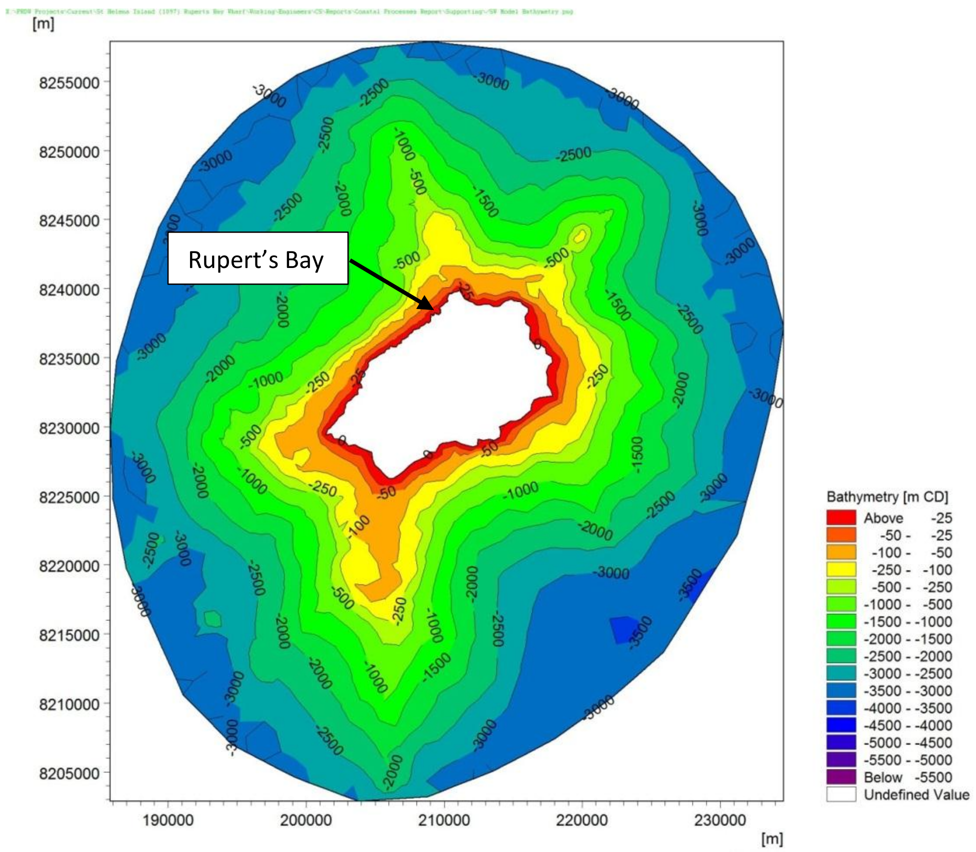
The average annual operational conditions are calculated by simulating one complete year. For the current study, this year has been chosen to be from November 2007 to October 2008. This period was chosen since it includes a high density of measured nearshore wave data, as well as including a number of large storm events during December, February and March.

5.3 Model Setup

5.3.1 Bathymetry

The model bathymetry was defined using the results of the two bathymetric surveys as described in Section 2.5. The model domain for the regional wave modelling includes the entire island and nearshore environment, extending approximately 20 km offshore in all directions. The location of the model boundary has been chosen such that the point at which the offshore OCEANOR wave data has been extracted is positioned on the model boundary, which occurs at depth of approximately -3 000 m CD. To ensure numerical accuracy of the model, the depth along the boundary has been maintained, as can be seen on Figure 5-1.

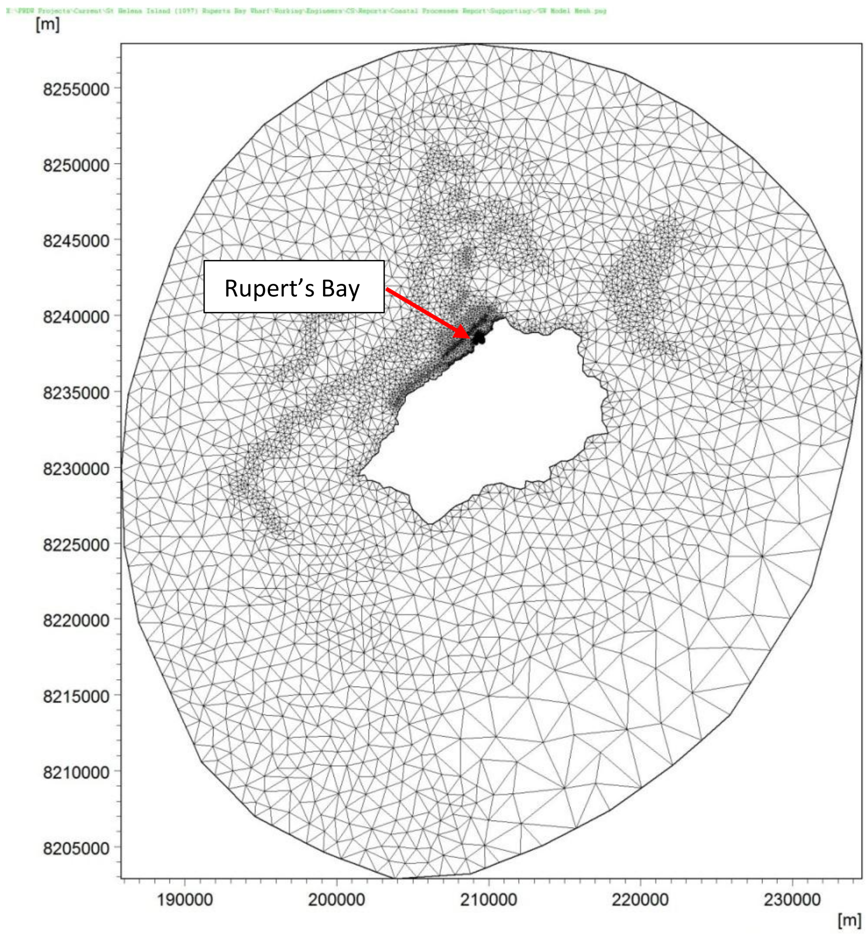
Figure 5-1: Regional wave model bathymetry



It is further clear from the figure that the nearshore bathymetry is characterized by large reefs, which extend in a shore-normal direction from the north-eastern and south-western corners of the island.

The model makes use of a flexible mesh, with the mesh becoming more refined, and thereby more accurate, closer to Rupert's Bay. This is visually presented in Figure 5-2. Additional refinements are included in areas of high bathymetric data coverage, to ensure the correct resolution of the seabed.

Figure 5-2: Regional wave model flexible mesh



5.3.2 Water Levels

Water level variations have been applied differently for each of the two modelling modes for the regional wave modelling. For the analysis of storms, a constant water level of +0.94 m CD has been applied, which is equivalent to Mean High Water Springs. For the analysis of operational wave conditions, the predicted tidal water level variations have been applied for the period of November 2007 to October 2008.

5.3.3 Winds

It is assumed that the offshore wave data, together with the adjustments relating to wind generated waves (see Section 5.3.5) include locally generated wind waves. The effect of wind was therefore not additionally included in the wave transformation model.

5.3.4 Bottom Friction

Bottom friction is usually used as a calibration parameter for regional wave modelling, since it impacts the amount of energy that is dissipated as waves propagate to the nearshore, as well as impacting the extent of rotation of the waves as they travel towards the coast.

Bottom friction is only applied to the wave if the orbital motions of the waves extend to the seabed. This means that if the water is deep, bottom friction does not play a role, whilst if the water is shallow, bottom friction becomes critical.

For the current application, the foreshore slope is very steep, with the water depth reaching 3 000 m at a distance of 20 km offshore. This causes bottom friction to be an ineffective calibration parameter for St. Helena Island. The distance over which bottom friction is applied to the waves is too short to impact the wave characteristics notably.

Nevertheless, bottom friction was included in the regional wave model by using a friction factor of 0.015. This dissipation coefficient is independent of wave and hydrodynamic conditions, i.e. the extent of friction remains constant regardless of wave height.

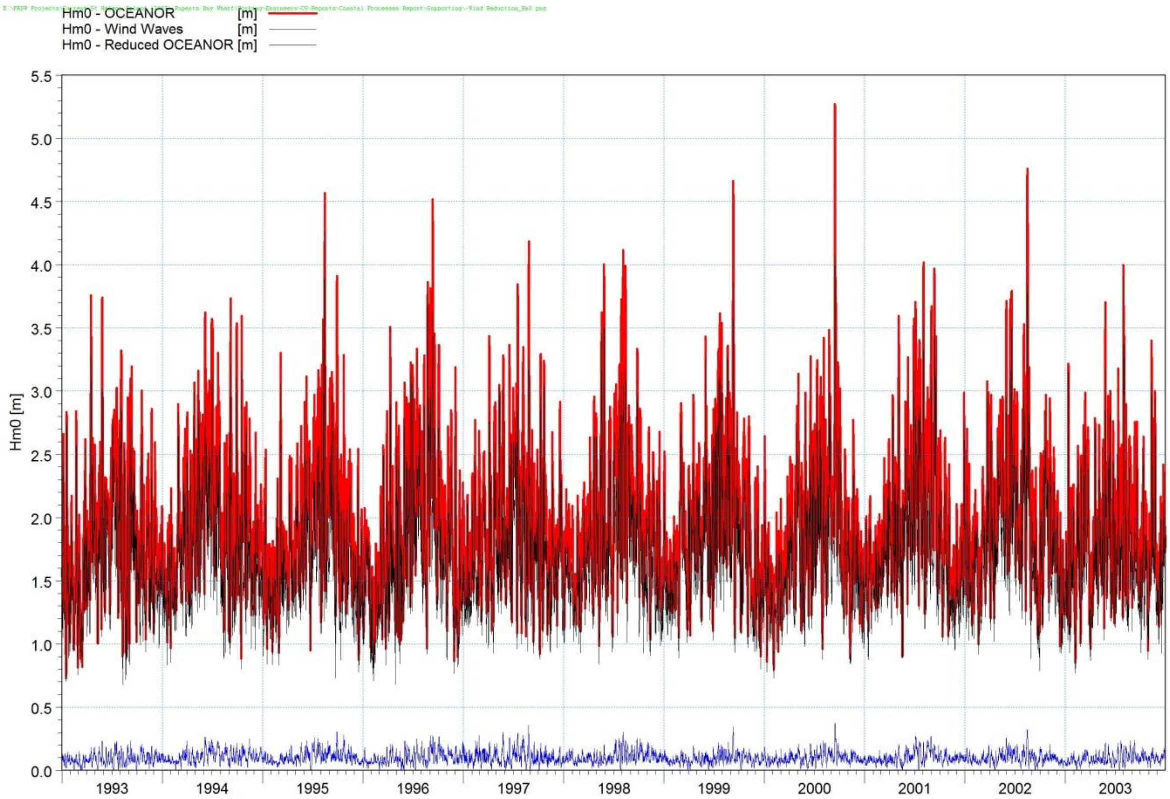
5.3.5 Offshore Boundary Conditions

The OCEANOR spectral wave data, discussed in Section 2.2.1, has been applied at the offshore model boundary. As shown in Figure 2-2, the location at which this data has been extracted is approximately 100 km NNW of St. Helena Island. Furthermore, referring to Section 2.4.1, the wind direction near the island is predominantly from the SE. This means that the wind between the island and the OCEANOR extraction point is adding to the wave energy at the OCEANOR extraction point.

If the OCEANOR offshore wave climate were to be applied directly to the regional wave model, the nearshore wave conditions in Rupert's Bay would be overestimated, since the effect of wind would be double-counted. As such, a reduction was applied to the offshore data, equivalent to the wave energy generated by the local winds over a distance of 100 km.

The extent of the reduction was calculated using a numerical wind flume model, using a constant depth of 1 000 m. The wind characteristics as discussed in Section 2.4.1 were applied to the model as the only model forcing, and the spectral wave energy at a distance of 100 km from the island was extracted. This spectral energy was then subtracted from the original OCEANOR wave climate. The parameterized results of this analysis for the period of 1993 to 2003 are shown in Figure 5-3 below.

Figure 5-3: Time series of OCEANOR, wind-generated and reduced OCEANOR parameterized wave climate



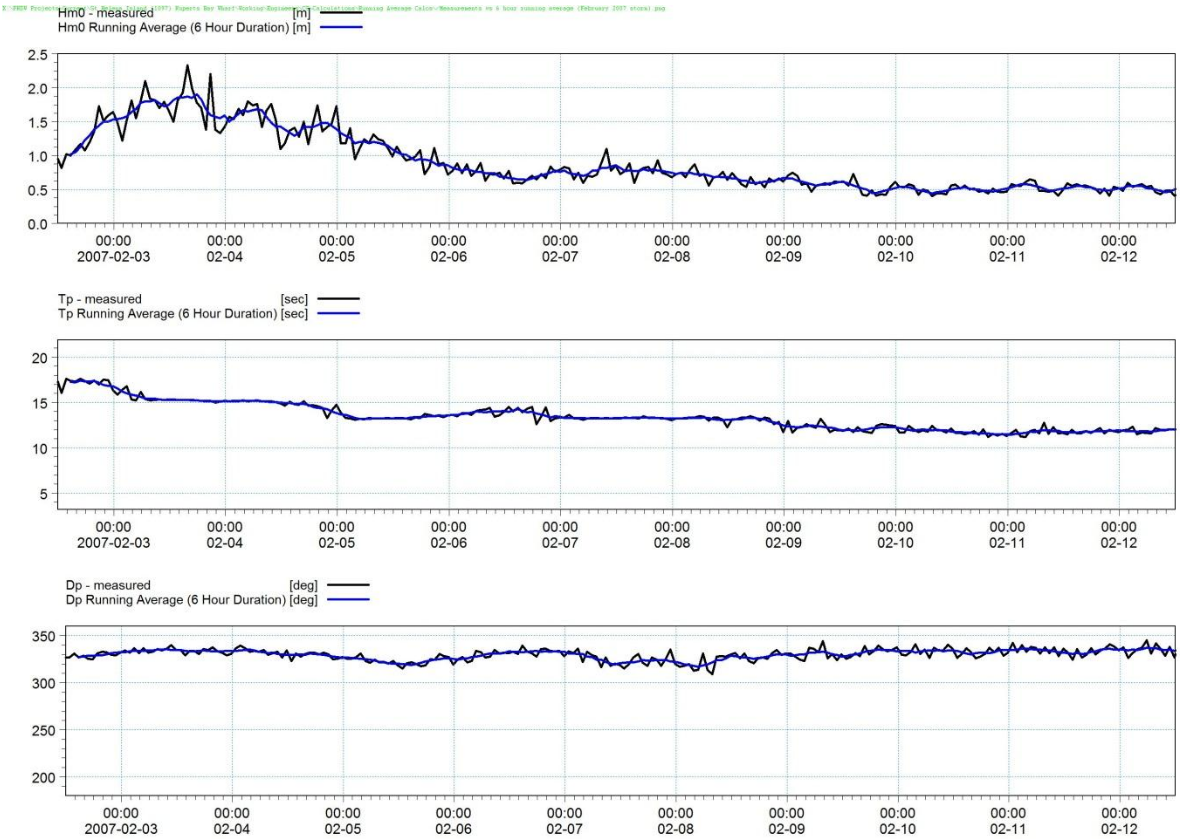
5.4 Model Calibration

5.4.1 Calibration Data

Nearshore wave measurements, discussed in Section 2.2.2 were used to calibrate the regional wave model. The short-term variability of the nearshore wave measurements has been introduced in the same section. Considering that the offshore wave data used as model forcing is made up of six-hourly wave data, the short-term variability of the nearshore data cannot be reproduced using the numerical model.

A running average approach was therefore applied to the nearshore wave measurements to generate a six-hourly data-set, which could be used to calibrate the regional wave model. The running average was calculated as the average of the significant wave heights of three hours prior to and three hours after a specific time step. The original and the averaged nearshore wave measurements (H_{m0}, T_p and direction) for a storm in February 2007 are graphically presented in Figure 5-4 below.

Figure 5-4: Original and averaged nearshore wave measurements – 3rd February 2007

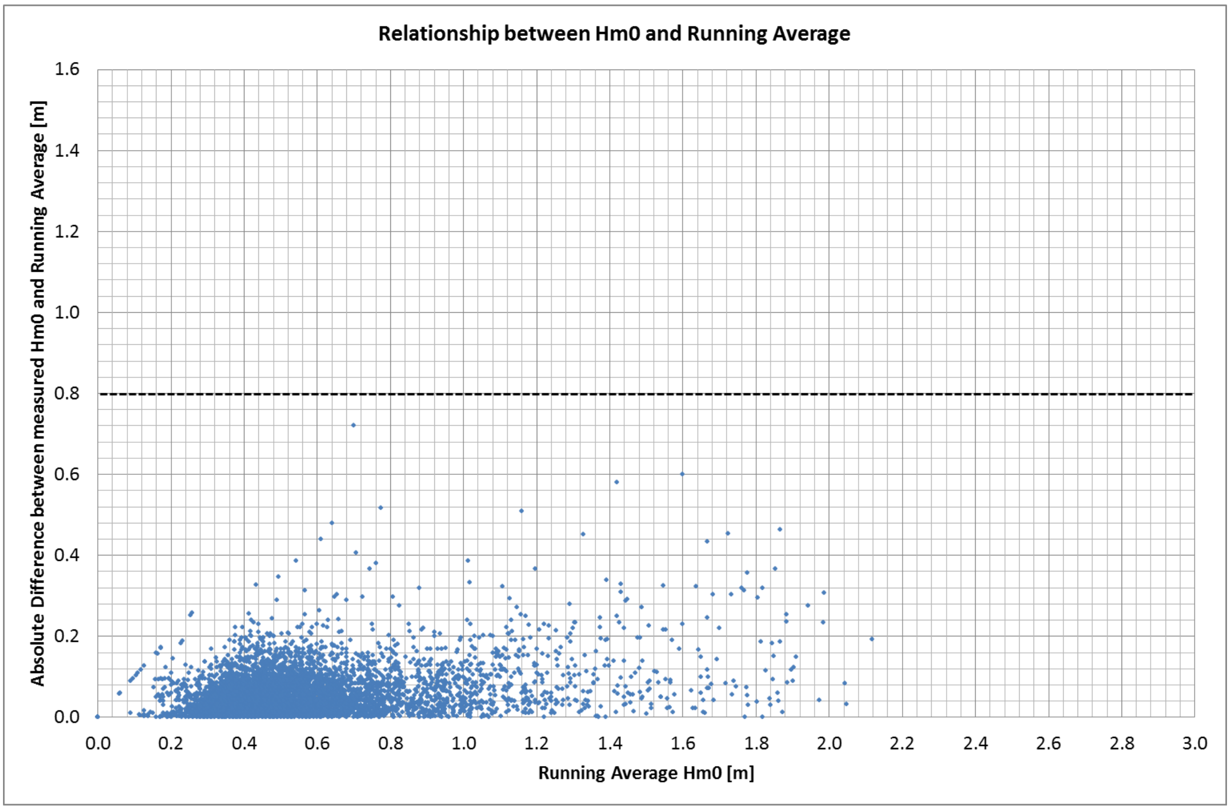


As can be seen from the figure above, the wave heights in the averaged wave climate are significantly smoothed compared to the original measurements. The regional wave model was calibrated against the running average nearshore wave climate.

From Figure 5-4 it is clear that the smoothed wave climate underestimates the wave peaks, and can therefore not be used directly without adjustment to determine the design conditions for the marine infrastructure. A further analysis was therefore performed to determine the absolute difference between the actual and running average nearshore wave climate, the results of which are presented in Figure 5-5 below. It is noted that the absolute difference has been presented here, since the positive and negative differences are approximately evenly distributed.

From this figure, a clear trend with regards to the difference between the actual and running average cannot be identified. The largest variance between the actual and running average has been determined to be 0.76 m, which occurred for a running average H_{m0} of 0.7 m. The incorporation of the difference between the actual and running average wave heights in the determination of nearshore wave climate is discussed in greater detail in Section 5.5 of this report.

Figure 5-5: Difference between actual and running average nearshore wave measurements



5.4.2 Wave Reflection

Due to the shape of Rupert's Bay, internal reflections significantly impact the waves within the bay. After a number of iterations, the following reflection coefficients were used to accurately replicate the nearshore running average wave measurements:

Vertical edges (cliffs, etc.)	0.8
Sloping edges (revetments, etc.)	0.5

It is noted that these reflections are higher than normally applied, which can be explained by the very low steepness of the incident waves, i.e. low H_{m0} and long T_p. The wave reflection coefficients are assessed in more detail as part of the vessel motions study (PRDW, 2013a).

5.4.3 Calibration Results

Calibration of the regional wave model was performed for the period November 2007 to October 2008.

As can be seen from Figure 5-6, the model replicates the trend of the measured nearshore wave conditions relatively well, reproducing the main storm events in December, February and March. Peak wave heights during the storms in December and March are slightly underestimated. Smaller events during the remainder of the year seem to be marginally overestimated.

Figure 5-6: Modelled vs running average measured wave conditions November 2007 to October 2008 at AQD ADCP location

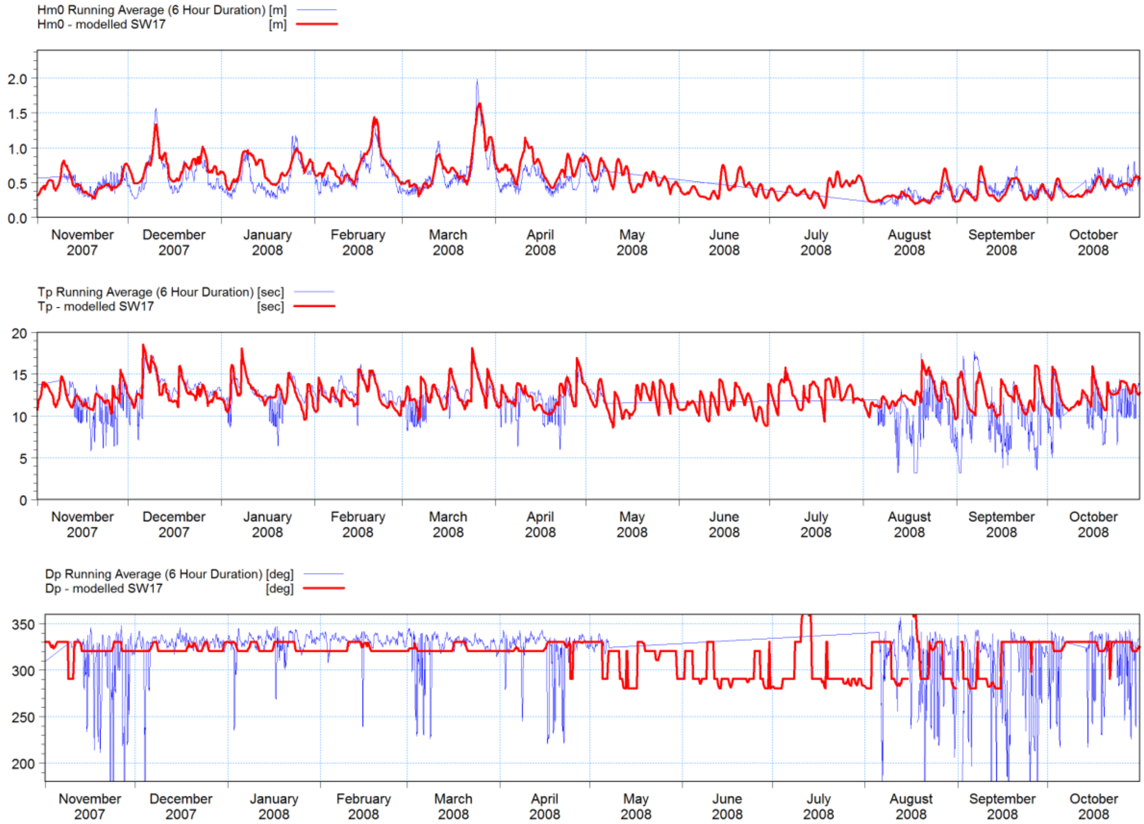
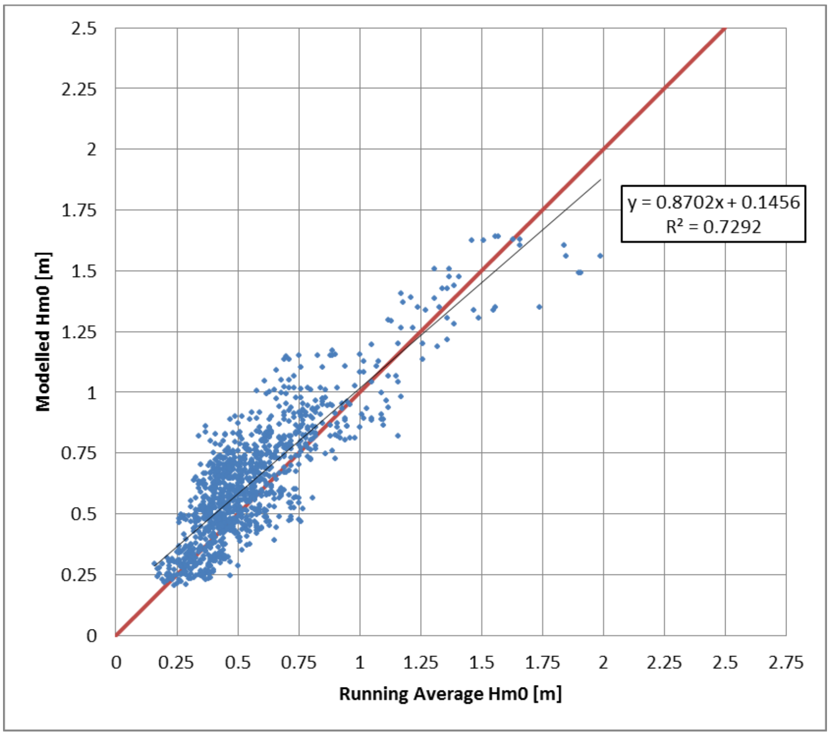


Figure 5-7 shows a scatter plot of the modelled wave heights versus the running average measured wave heights. This figure confirms that smaller wave heights are slightly overestimated, whilst the largest waves tend to be slightly underestimated.

Figure 5-7: Modelled vs running average nearshore wave heights



5.5 Modelling Results

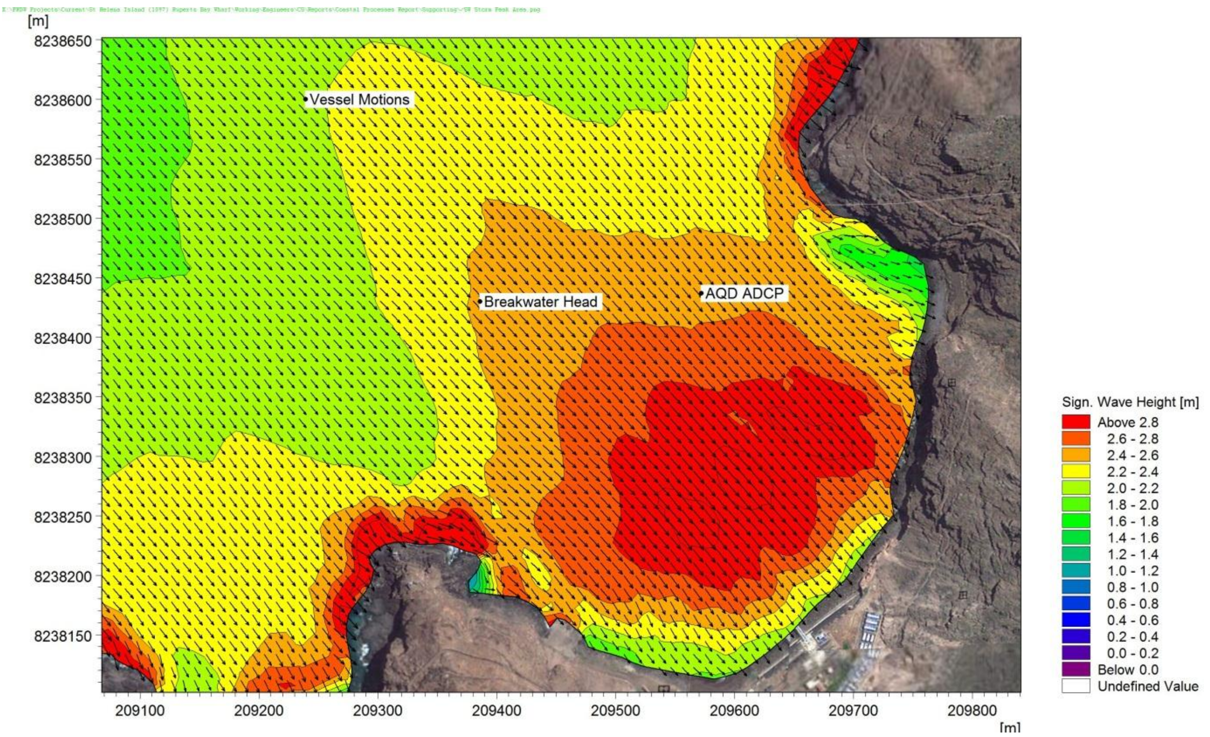
Figure 5-8 shows the wave climate within Rupert's Bay during a storm on 12th February 2010. Wave shoaling and reflection results in an increase in wave height in the central part of the bay, before wave breaking occurs against the coastline. Wave refraction in the bay results in an anti-clockwise and clockwise rotation of the waves along the northern and southern flanks of the bay respectively, whilst also reducing the wave height in these areas. Wave focussing occurs at the headlands on the northern and southern boundaries of the bay, resulting in an increased wave height in these areas.

The wave time series at the locations indicated on Figure 5-8 were extracted to further analyse the Rupert's Bay wave conditions, as well as to determine the design conditions for the marine infrastructure. The coordinates of these locations are given in Table 5-1 below.

Table 5-1: Location of time series extractions

Name	UTM Zone 30S	
	Easting [m]	Northing [m]
AQD ADCP	209 572	8 238 437
Breakwater Head	209 386	8 238 430
Vessel Motions (boundary for Boussinesq wave model)	209 239	8 238 600

Figure 5-8: Significant wave height in Rupert's Bay at 6 AM on 12th February 2010



5.5.1 Extreme Wave Conditions at the AQD ADCP

The storm modelling results of the regional wave model were used to perform an EVA of the wave conditions at the AQD ADCP location. The time series of H_{m0}, T_p and mean wave direction is given in Figure 5-9 below. The results of the EVA are presented graphically in Figure 5-10.

The short-term variability in measured wave height in Rupert's Bay has been introduced in Section 5.4.1. It has been discussed that the maximum variance between the actual measured wave height and the running average measured wave height is approximately 0.75 m, and that the magnitude of this is independent of the actual measured wave height. Considering that the regional wave model is used to simulate the running average measured wave height, an allowance of 0.8 m has been included on top of the extreme wave heights presented Figure 5-10 to account for the short-term variability of the nearshore wave climate.

Increasing wind speeds due to climate change are projected to increase wave heights by approximately 14% over the 70 year design life of the structure (PRDW, 2010). To account for climate change as well as the unorthodox approach followed, an additional 20% allowance has been included to determine the design wave conditions at the AQD ADCP location. The results of this are tabulated in Table 5-2.

Figure 5-9: H_{m0}, T_p and mean wave direction at AQD ADCP for storm events

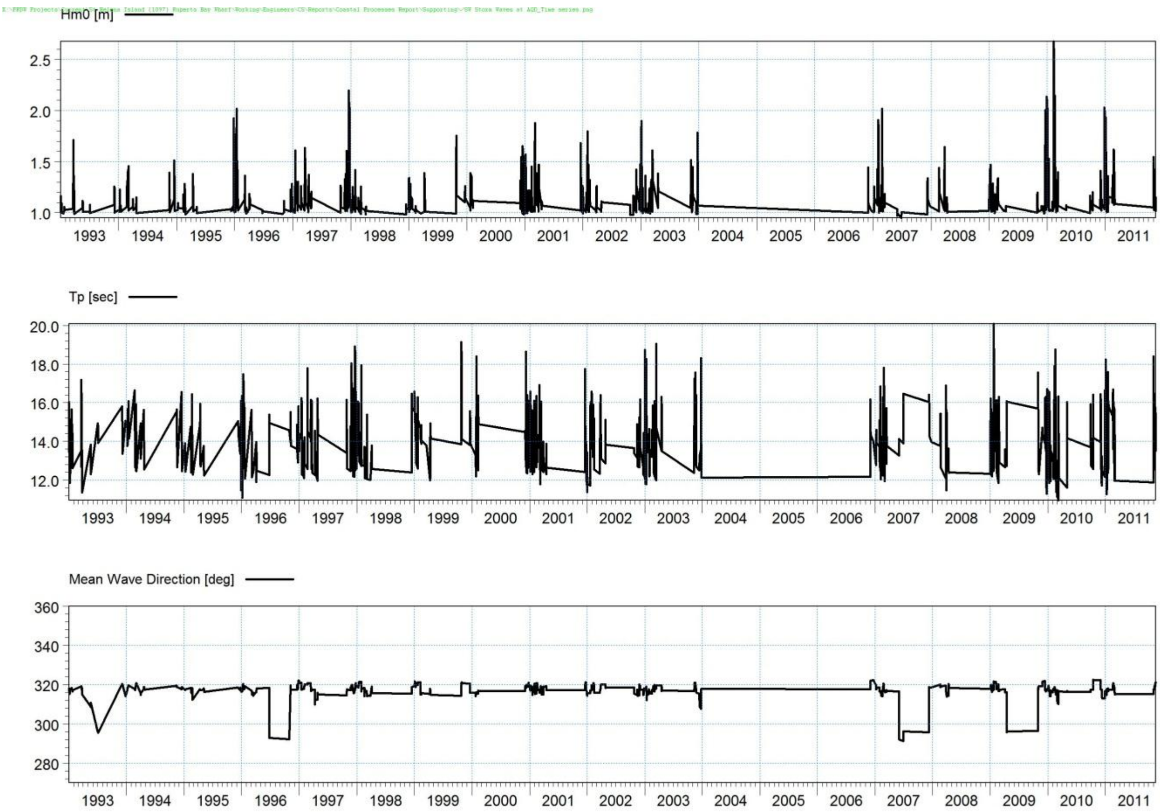


Figure 5-10: Extreme value analysis of wave heights at AQD ADCP

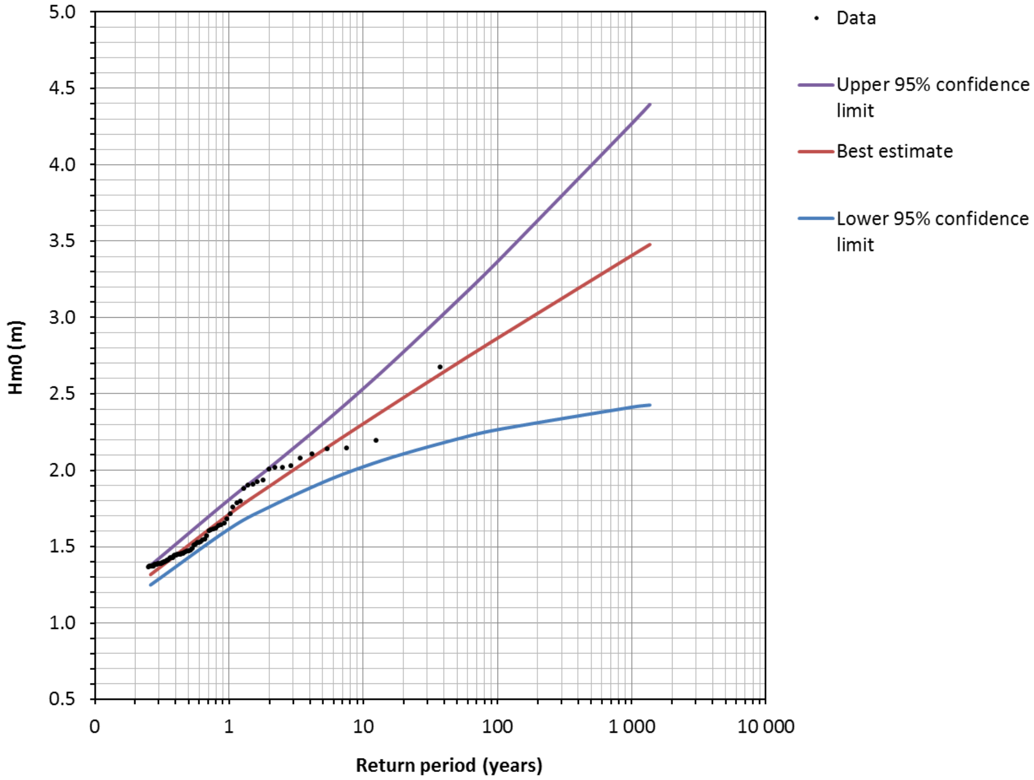


Table 5-2: Extreme wave conditions at AQD ADCP

Return Period [years]	Risk of Exceedance During Design Life [%]	H_{m0} [m]	Allowance for short-term variability [m]	Uncertainty factor [%]	Design H_{m0} [m]
1	100	1.7	0.8	20	3.0
2	100	1.9			3.2
5	100	2.1			3.5
10	100	2.3			3.7
20	97	2.5			3.9
50	76	2.7			4.2
100	51	2.9			4.4
1 000	7	3.4			5.0
1 365	5	3.5			5.1
10 000	1	3.9			5.7

5.5.2 Extreme Waves at Breakwater Head

Storm modelling results of the regional wave model were used to determine the design wave conditions for the breakwater structure. The same methodology as described in Section 5.5.1 was used here. The results of this analysis are presented graphically in Figure 5-11, and are tabulated in Table 5-3. A plot of the relationship between the risk of a specific event being exceeded during the design life as a function of design significant wave height is provided in Figure 5-12.

Comparing the results given in Table 5-2 and Table 5-3, it is clear that the wave conditions at the breakwater head are less severe than those at the AQD ADCP location. The cause for this is the reduced effect of wave shoaling at the breakwater head compared to at the AQD ADCP. The comparative difference in wave conditions obtained from the regional wave model is therefore considered to be accurate and representative of reality.

Figure 5-11: Extreme value analysis of wave heights at breakwater head

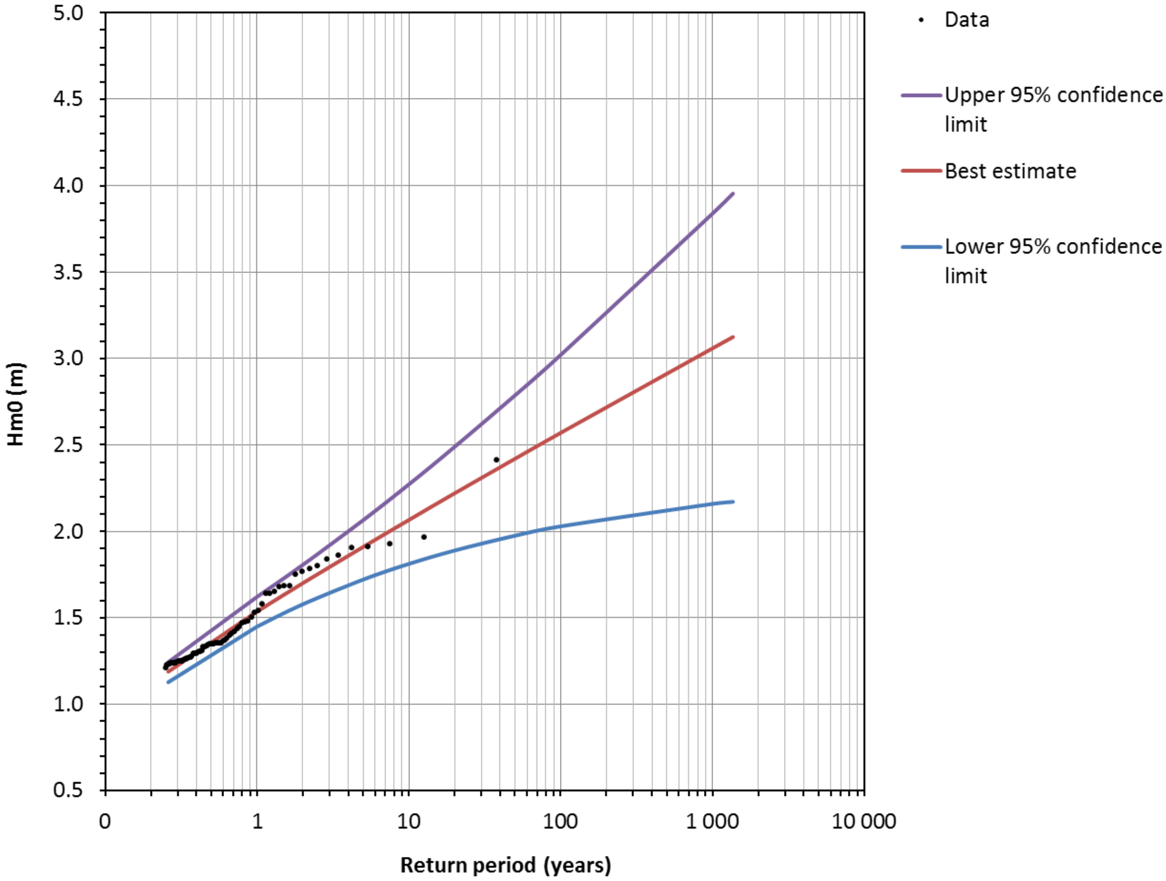
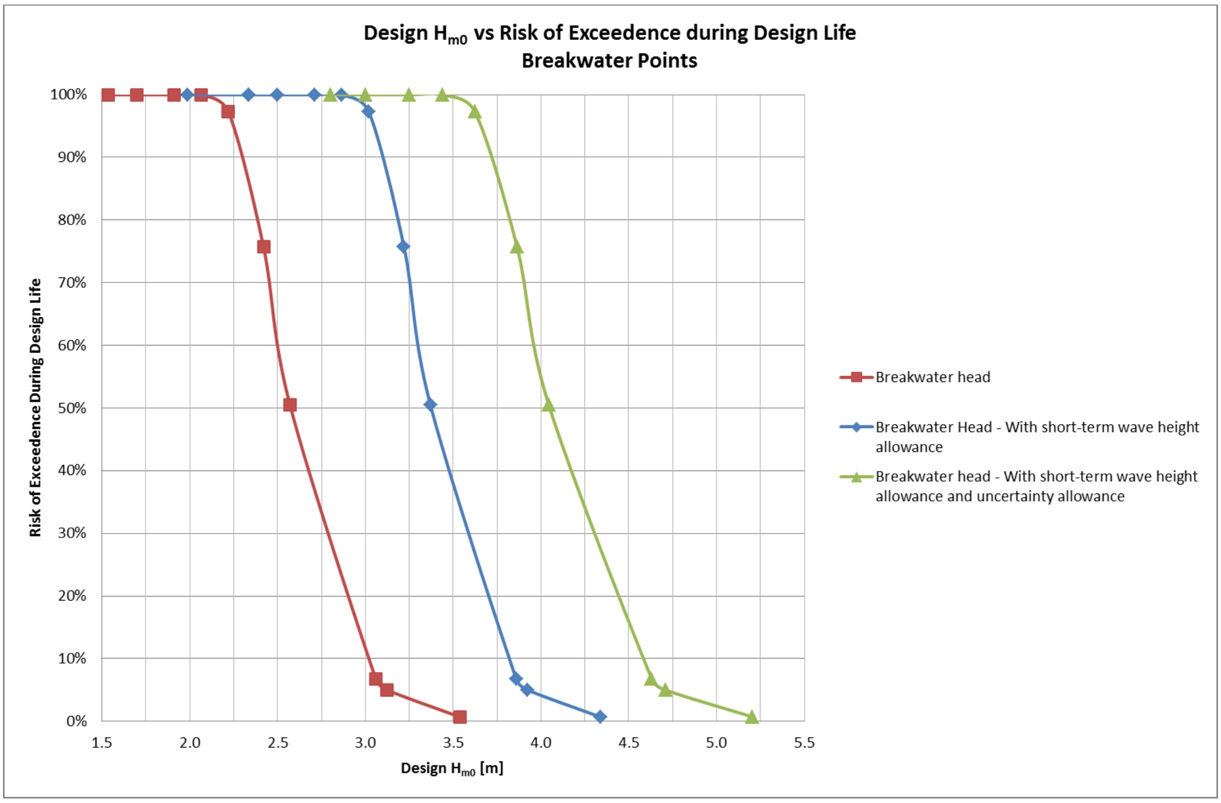


Table 5-3: Extreme wave conditions at breakwater head

Return Period [years]	Risk of Exceedance During Design Life [%]	H_{m0} [m]	Allowance for short-term variability [m]	Uncertainty factor [%]	Design H_{m0} [m]
1	100	1.5	0.8	20	2.8
2	100	1.7			3.0
5	100	1.9			3.3
10	100	2.1			3.4
20	97	2.2			3.6
50	76	2.4			3.9
100	51	2.6			4.0
1 000	7	3.1			4.6
1 365	5	3.1			4.7
10 000	1	3.5			5.2

Figure 5-12: Design H_{m0} vs risk of exceedance during design life



5.5.3 Operational Wave Conditions for Vessel Motions Study

Operational wave conditions between November 2007 and October 2008 have been introduced previously during the discussion of the calibration results of the regional wave model (Section 5.4.3). However, only the direct model results were introduced, excluding a discussion regarding design allowances. These are discussed below.

Referring to the previous discussions regarding the design wave conditions at the AQD ADCP location and breakwater head, a relatively conservative approach was adopted since the wave heights were used for the stability design of critical marine infrastructure, viz. the breakwater. Because of this, a conservative approach of adding an allowance for the short-term wave height variability as well as an inclusion of an uncertainty allowance is warranted. However, considering that average operational conditions are required for the vessel motions study, a less conservative approach should be considered.

Referring to Figure 5-5, the majority of the short-term wave height variability magnitudes are smaller than 0.1 m. A 1 per cent exceedance (3.7 days per year) is equivalent to 0.22 m or more for a period of approximately 20 minutes (within any given 6 hourly period). When calculating downtime this short term variability in wave height needs to be addressed by selecting appropriate exceedance levels and taking account of the duration of these events.

6. SEDIMENT TRANSPORT ASSESSMENT

6.1 Model Description

The numerical model used was the *MIKE by DHI Coupled Flexible Mesh* model. The model comprises a dynamic coupling between the following models:

- Spectral wave model
- Hydrodynamic model
- Non-cohesive sediment transport model

Each of these is described in more detail in the following sections.

6.1.1 Spectral Wave Model

The *MIKE by DHI Spectral Waves Flexible Mesh* model, which was described in Section 5.1, was used for wave refraction modelling. For this application, the quasi-stationary decoupled parametric formulation was used, compared to the fully spectral formulation used during the regional wave modelling. This is based on a parameterization of the wave action conservation equation. The parameterization is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum as dependant variables.

The physical phenomena included in this study are listed in Section 5.1.

6.1.2 Hydrodynamic Model

The MIKE by DHI Flow Flexible Mesh model was used for hydrodynamic modelling. The application of the model is described in the User Manual (DHI, 2012e), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2012f). The model is based on the shallow water equations, i.e. the depth-integrated incompressible Reynolds averaged Navier-Stokes equations.

The time integration of the shallow water equations is performed using an explicit scheme. Horizontal eddy viscosity is modelled with the Smagorinsky formulation. The mesh is the same unstructured flexible mesh used for the Spectral Wave model.

In this study the model includes the following physical phenomena:

- Currents due to wind stress on the water surface

- Currents due to waves: the second order stresses due to breaking of short-period waves are including using the radiation stresses computed in the Spectral Waves Model
- Coriolis forcing
- Bottom friction
- Flooding and drying

6.1.3 Non-Cohesive Sediment Transport Model

The sediment transport model used was the *MIKE by DHI Sand Transport* model. The application of the model is described in the User Manual (DHI, 2012g), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2012h).

The *Sand Transport* model calculates the transport of non-cohesive sediment (grain size > 0.063 mm) based on the combination of flow conditions from the hydrodynamic module and wave conditions from the spectral wave module. For the case of combined wave and currents, sediment transport rates are derived by linear interpolation in a sediment transport lookup table. The values in the table are calculated by the quasi three-dimensional sediment transport model (STPQ3D). STPQ3D calculates the instantaneous and time-averaged hydrodynamics and sediment transport in two horizontal directions. As the model calculates the bed load and the suspended load separately, the values in the sediment transport table are the total load. The model accounts for graded sediment by dividing the grading curve into a number of size classes, calculating the sediment transport for each class and then averaging to obtain the total transport rate.

The temporal and vertical variations of shear stress, turbulence, flow velocity and sediment concentrations are resolved. The time evolution of the boundary layer due to combined wave/current motion is solved by means of an integrated momentum approach. The force balance includes contributions from the near bed wave orbital motion, forces associated with wave breaking (gradients of radiation stresses) and the sloping water surface.

Equilibrium sediment transport conditions are assumed in the model, which implies that the sediment responds instantaneously to hydrodynamic forcing, i.e. without lag effects. In the present study, this may result in the distribution of the sedimentation being concentrated in the centre of Rupert's Bay, as opposed to being spread out more if lag effects were to be included. However, the sedimentation volumes and mechanisms are unlikely to change.

6.2 Modelling Strategy

The objective of the sediment transport assessment is the identification of the impact of the proposed development on the local sediment transport regime. To enable this, the sediment transport characteristics need to be analysed excluding and including the permanent wharf structure.

This assessment has been performed for discrete storm events, with return periods of 1, 5, 10, 20, 50 and 100 years. Although it is expected that sediment transport will continue under operational conditions, this has not been simulated since the transport magnitude are not expected to be significant. This is however discussed in more detail in later sections of this report.

The simulations performed during the sediment transport assessment are summarized in Table 6-1

Table 6-1: Sediment transport assessment simulation scenarios

<table>
<tr><th>Run No</th><th>Breakwater</th><th>Return Period [years]</th><th>Peak Storm H_{m0} [m]</th><th>T_p [s]</th><th>D_{50} [mm]</th><th>Wind Speed [m/s]</th><th>MWD [deg]</th></tr>
<tr><td>1</td><td rowspan="8">No</td><td rowspan="8">100</td><td rowspan="8">2.58</td><td rowspan="5">16.6</td><td rowspan="3">0.15</td><td rowspan="7">20.7</td><td>320</td></tr>
<tr><td>2</td><td>310</td></tr>
<tr><td>3</td><td>330</td></tr>
<tr><td>4</td><td>0.1</td><td rowspan="5">320</td></tr>
<tr><td>5</td><td>0.2</td></tr>
<tr><td>6</td><td>14.6</td><td rowspan="3">0.15</td></tr>
<tr><td>7</td><td>18.6</td></tr>
<tr><td>8</td><td>16.6</td><td>0</td></tr>
<tr><td>9</td><td rowspan="8">Yes</td><td rowspan="8">100</td><td rowspan="8">2.58</td><td rowspan="5">16.6</td><td rowspan="3">0.15</td><td rowspan="7">20.7</td><td>320</td></tr>
<tr><td>10</td><td>310</td></tr>
<tr><td>11</td><td>330</td></tr>
<tr><td>12</td><td>0.1</td><td rowspan="5">320</td></tr>
<tr><td>13</td><td>0.2</td></tr>
<tr><td>14</td><td>14.6</td><td rowspan="3">0.15</td></tr>
<tr><td>15</td><td>18.6</td></tr>
<tr><td>16</td><td>16.6</td><td>0</td></tr>
<tr><td>17</td><td>No</td><td rowspan="2">50</td><td rowspan="2">2.39</td><td rowspan="2">16.3</td><td rowspan="2">0.15</td><td rowspan="2">19.8</td><td rowspan="2">320</td></tr>
<tr><td>18</td><td>Yes</td></tr>
<tr><td>19</td><td>No</td><td rowspan="2">20</td><td rowspan="2">2.15</td><td rowspan="2">15.8</td><td rowspan="2">0.15</td><td rowspan="2">19.3</td><td rowspan="2">320</td></tr>
<tr><td>20</td><td>Yes</td></tr>
<tr><td>21</td><td>No</td><td rowspan="2">10</td><td rowspan="2">1.96</td><td rowspan="2">15.5</td><td rowspan="2">0.15</td><td rowspan="2">17.7</td><td rowspan="2">320</td></tr>
<tr><td>22</td><td>Yes</td></tr>
<tr><td>23</td><td>No</td><td rowspan="2">5</td><td rowspan="2">1.76</td><td rowspan="2">15.2</td><td rowspan="2">0.15</td><td rowspan="2">16.1</td><td rowspan="2">320</td></tr>
<tr><td>24</td><td>Yes</td></tr>
<tr><td>25</td><td>No</td><td rowspan="2">1</td><td rowspan="2">1.30</td><td rowspan="2">14.3</td><td rowspan="2">0.15</td><td rowspan="2">14.9</td><td rowspan="2">320</td></tr>
<tr><td>26</td><td>Yes</td></tr>
</table>

From this table, it can be identified that the sediment transport assessment includes a range of sensitivity analyses. These analyses have been performed by varying a number of parameters around their mean value. The mean value for each of the parameters is discussed in later section of this report. The parameters which were included in the sensitivity analyses are:

- Wave peak period [s]
- Median grain size of Rupert's Bay sediment [mm]
- Effect of wind, i.e. the transport characteristics including and excluding wind
- Mean wave direction (MWD) [deg]

The objective of the sensitivity analysis was to confirm that the model results were not overly sensitive to small variations in the input parameters.

It should further be noted that the impact of the permanent wharf was modelled using the layout given in the Design Basis Document (PRDW, 2013b). Differences in the layout given in that report versus the final layout are however not expected to change the results and conclusions of this report significantly.

6.3 Model Setup

6.3.1 General

Each of the settings discussed in this section are common to each of the three modules of the coupled numerical model. Settings which are unique to each of the three modules are discussed in Sections 6.3.2 to 6.3.4.

6.3.1.1 Bathymetry

Two model bathymetries have been developed, excluding and including the permanent wharf structure. These are shown in Figure 6-1 and Figure 6-2 respectively. The model domain for the sediment transport assessment is significantly reduced compared to the regional wave modelling. This has been done to reduce computational requirements. The offshore boundary (boundary no. 1) of the model is located approximately along the -50 m CD depth contour. The north-eastern (boundary no. 3) and south-western (boundary no. 2) boundaries of the model have been chosen ensuring that boundary inaccuracies do not reach the area of interest in Rupert's Bay.

As can be seen from Figure 6-1 and Figure 6-2, the model meshes become more refined close to the coastline, to ensure the accuracy of the simulation. The mesh along the Rupert's Bay shoreline is refined to 5 m, to ensure that wave breaking and the associated currents are accurately resolved.

The meshes of the models have been generated to be identical, with the only difference being the presence of the permanent wharf structure. Differences in coastal processes between the two models can therefore be compared directly, and are devoid of numerical differences.

Figure 6-1: Coupled model bathymetry and model mesh, excluding permanent wharf structure

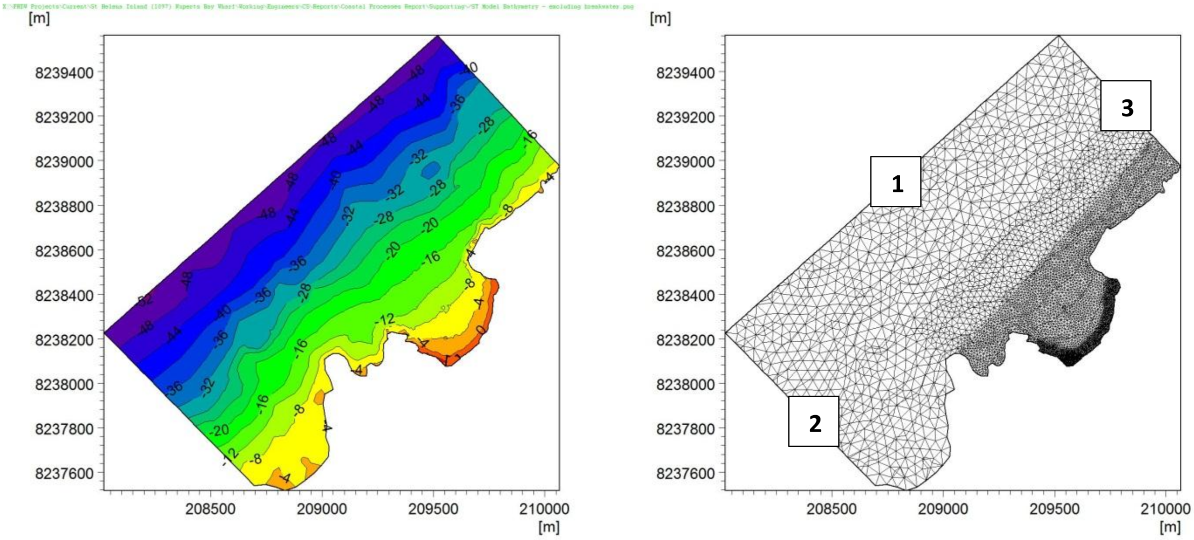
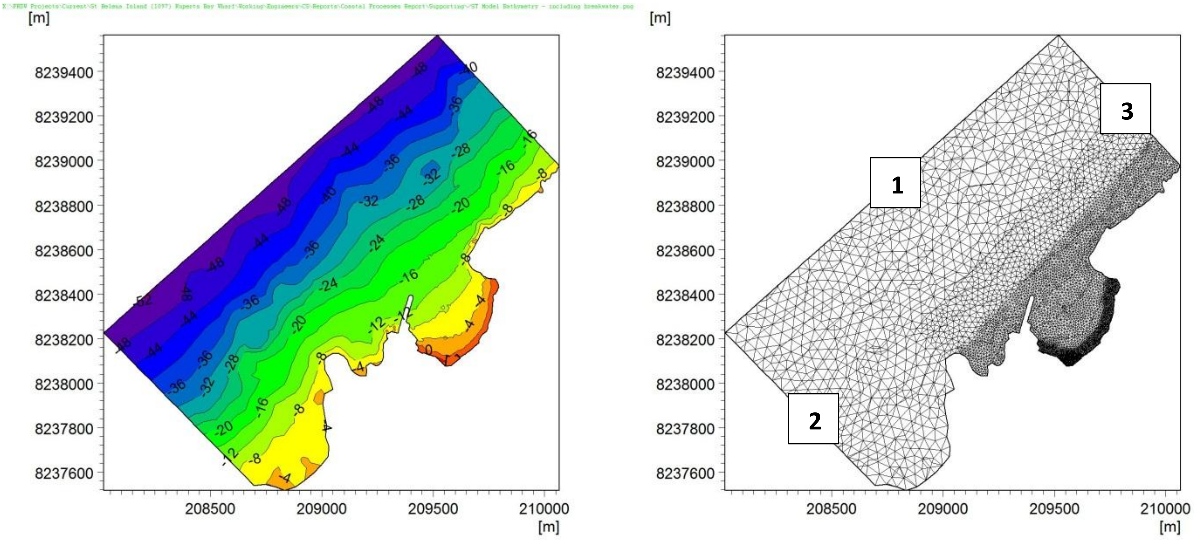


Figure 6-2: Coupled model bathymetry and model mesh, including permanent wharf structure



6.3.1.2 *Water Levels*

As introduced in Section 6.2, only discrete storm events have been simulated during the sediment transport assessment. Because of this, time-varying tidal variations have not been included in the model. A fixed tidal level of +0.94 m CD has been used during the modelling. Local water level variations as a result of wind/wave setup are included, and are calculated by the hydrodynamics module.

6.3.1.3 Wind

The effect of wind has been included in the sediment transport assessment. The results of the EVA of the local wind data has been discussed in Section 4. As a conservative approach, the 100-year return period wind speed has been combined with the 100-year wave event, and similarly for the other return periods, e.g. the 10-year wind with the 10-year wave event.

The wind has been set to a constant direction of 147^{o}. It has been discussed that the wind funnels through Rupert's valley, causing the local wind direction in Rupert's Bay to be relatively constant, regardless of minor variations in the regional wind direction.

A sensitivity analysis has been performed by analysing the sediment transport characteristics including and excluding the wind. Results of this analysis are provided in Section 6.5.1.3.

6.3.2 Spectral Wave Model

6.3.2.1 Bottom Friction

As discussed previously, bottom friction does not play a significant role in the transformation of waves as they propagate towards the coast. The same friction factor of 0.015 was used for the spectral waves module during the sediment transport assessment.

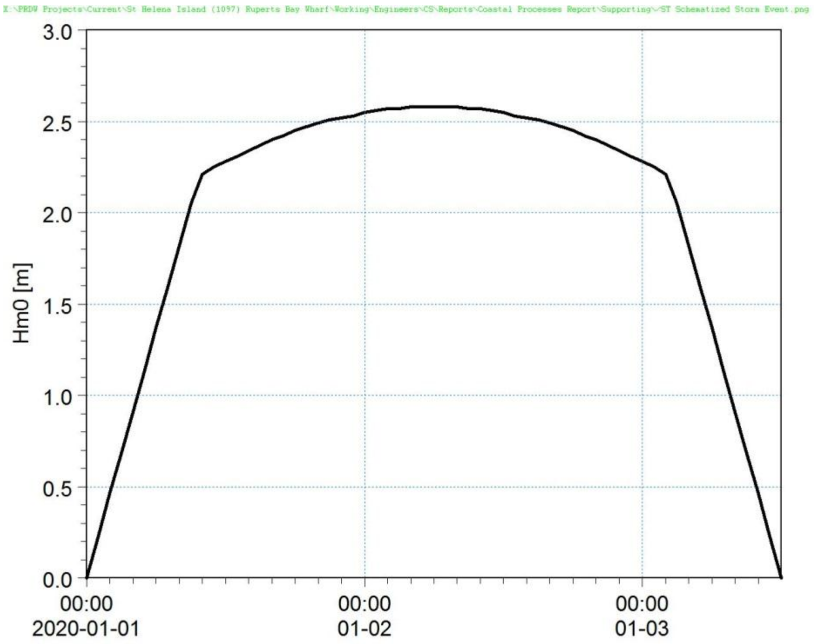
6.3.2.2 Boundary Conditions

The storm wave conditions have been applied to the offshore boundary (boundary no. 1) of the spectral wave module. Boundaries no. 2 and 3 are so-called lateral boundaries. Linear wave refraction is performed along these lines, based on the wave conditions at the offshore node of the boundary. In this way, wave conditions are accurately replicated along all three marine edges of the model.

Schematized storm events have been developed for each of the return periods under consideration. A review of the storm events identified by the nearshore wave measurements has shown that storm durations are approximately constant, irrespective of peak wave height. The typical storm duration is 40 hours. Wave heights were ramped up/down for ten hours before and after the actual storm to avoid any numerical instability.

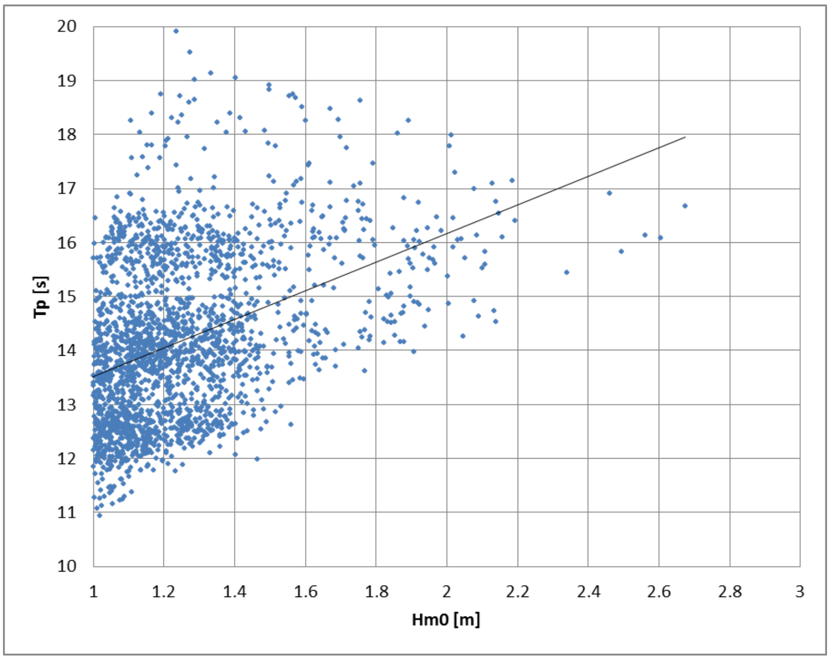
The schematized 100 year return period storm is presented in Figure 6-3, which includes the ramp-up/down of waves before and after the actual storm event. Note that the date of the time series is fictive.

Figure 6-3: Wave height of schematized 100 year return period storm event



A constant wave period has been used for each of the storm events. The period has been determined by analysis of the relationship between wave height and period for the nearshore wave measurements. The result of this is given in Figure 6-4 below.

Figure 6-4: Scatter plot of measured nearshore H_{m0} vs T_p



Based on the regression analysis presented in the figure above, using the peak storm significant wave heights for each of the storm events (see Table 6-1), the representative storm peak periods could be determined. The results of this analysis are summarized in Table 6-1. To determine the sensitivity of the sediment transport regime to the peak period, the base case period was varied by two seconds up and down for the 100 year storm event. Results of this analysis are discussed in Section 6.5.1.1.

The nearshore mean wave direction has been introduced in Section 2.2.2, and varies predominantly between 310^o and 330^o. A base case constant storm wave direction of 320^o was therefore chosen, with a sensitivity analysis being performed by varying this by 10^o up and down. Results of this analysis are provided in Section 6.5.1.4.

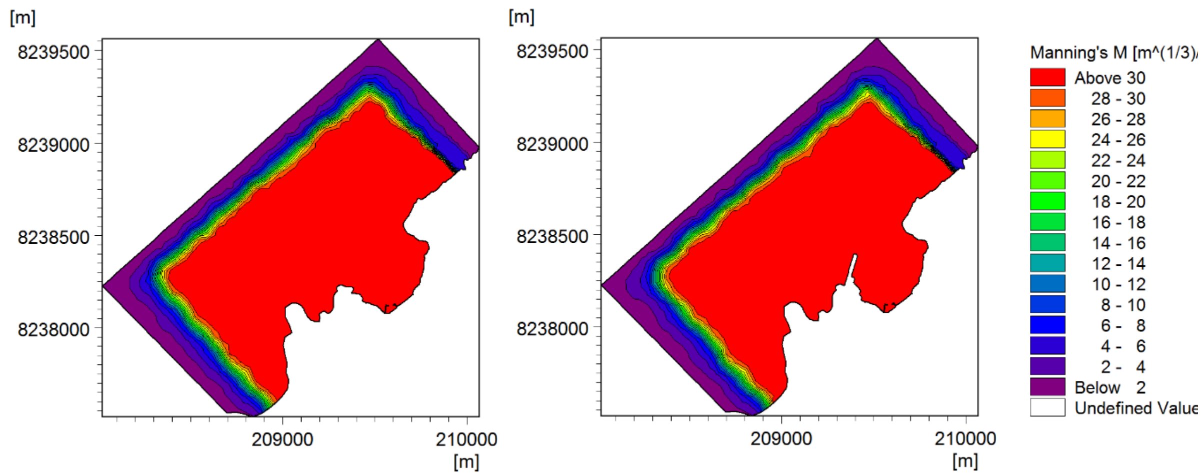
6.3.3 Hydrodynamic Model

6.3.3.1 Bed Roughness

A spatially varying bed roughness formulation has been applied for the hydrodynamic module. A Manning number of 32 $m^{1/3}/s$ has been applied in the inner model domain, reducing to 1 $m^{1/3}/s$ around the model edges. It is noted that a lower Manning number represents higher friction. A Manning number of 1 $m^{1/3}/s$ therefore indicates very high friction, effectively stopping all current flow around the marine edges of the model.

The motivation to use a high bed roughness along the outer marine edges of the hydrodynamic module is to stop spurious current instabilities in the hydrodynamic module. Referring to Figure 6-5, a low current speed along the outer model domain does not impact on the conditions within Rupert's Bay.

Figure 6-5: Bed roughness for hydrodynamic module



6.3.3.2 Boundary Conditions

Boundary conditions along the marine edges for the hydrodynamic model have been set using a fixed water level of +0.94 m CD, equivalent to a tidal level of mean high water springs. No ocean or tidal currents have been applied.

6.3.4 Non-Cohesive Sediment Transport Model

6.3.4.1 Sediment Grain Size

The sediment characteristics of Rupert's Bay have been introduced in Section 2.7. Referring to Figure 2-22, the average of the median grain diameter (D_{50}) as obtained from the bay-wide grab sampling campaign is approximately 0.15 mm. The sediment on the so-called 'swimming beach' is 0.26 mm.

As introduced previously, a sensitivity analysis with regards to grain size has been performed. During this analysis, the bay-wide median grain size was reduced to 0.1 mm and increased to 0.2 mm respectively. The sediment characteristics on the swimming beach, as well as the beach in the north-eastern corner of Rupert's Bay have been kept constant at 0.26 mm for all cases. The results of this sensitivity analysis are provided in Section 6.5.1.2.

The spatially varying mean grain diameter of the base case, as well as the two sensitivity cases, is shown in Figure 6-6 to Figure 6-8.

A constant grading coefficient of 1.5 has been applied for all cases, informed by the bay-wide sediment sampling campaign.

Figure 6-6: Median grain diameter for sediment transport module (Base case – 0.15 mm)

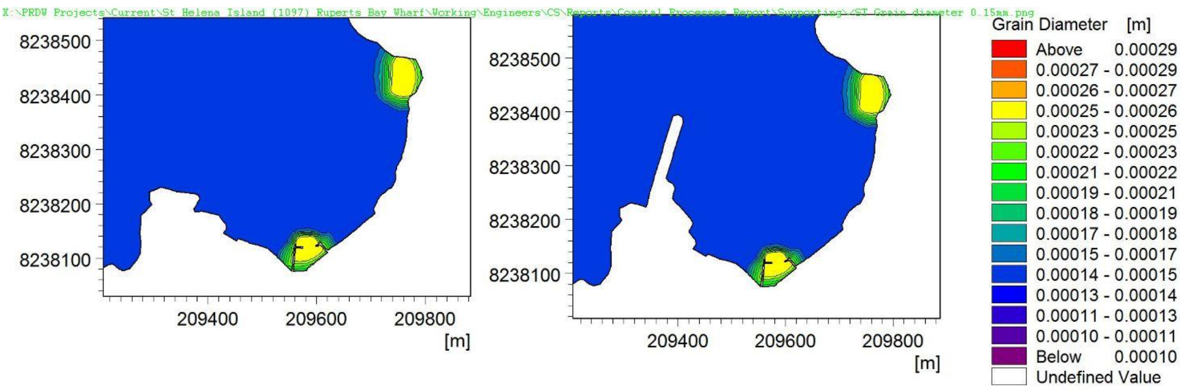


Figure 6-7: Median grain diameter for sediment transport module (Sensitivity case – 0.10 mm)

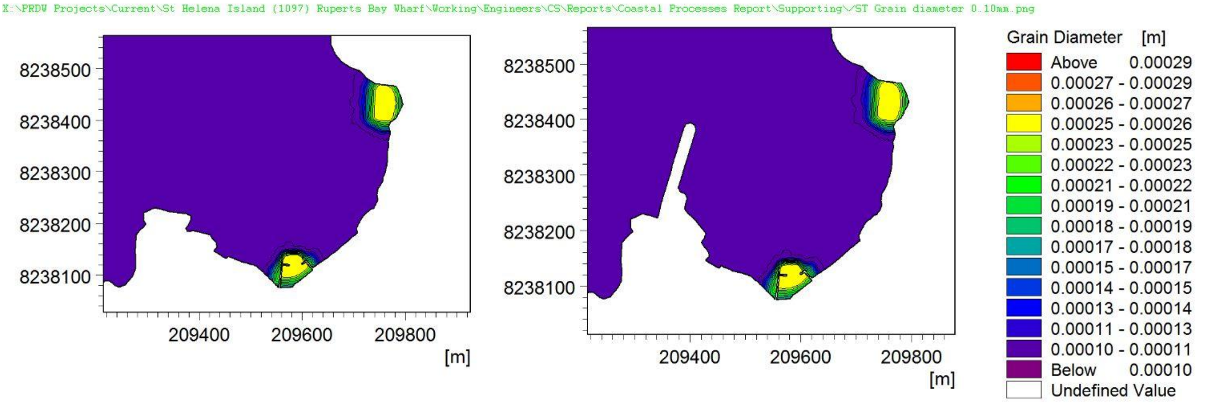
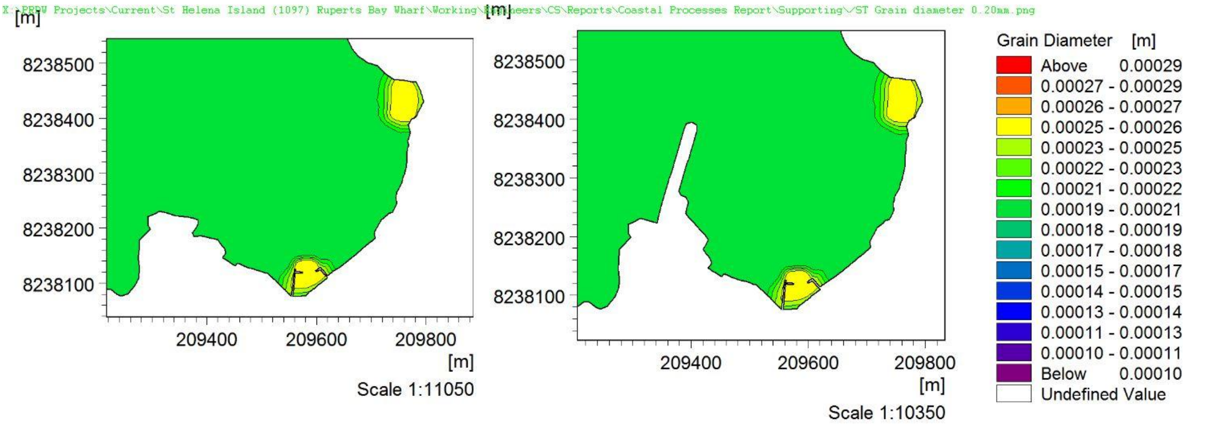


Figure 6-8: Median grain diameter for sediment transport module (Sensitivity case – 0.20 mm)

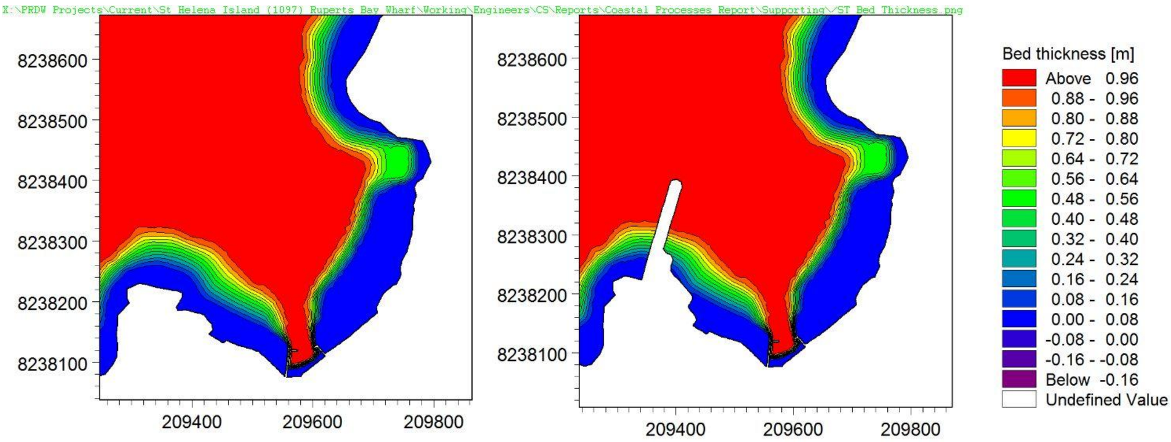


6.3.4.2 *Sediment Thickness*

A spatially varying sediment thickness description (i.e. the thickness of the sand layer overlying the non-erodible rock) has been used for the two-dimensional sediment transport assessment. The development of this has been guided by the results of the bay-wide sediment sampling campaign, as well as the description of the seabed characteristics described in Section 2.6.

As can be seen from the figure, the maximum thickness of the sediment has been assumed to be 1 m, which reduces to zero around the edges of Rupert's Bay. The sediment thickness on the swimming beach has been assumed to be 1 m, whilst it has been assumed that the sand is 0.5 m thick on the beach on the north-eastern corner of Rupert's Bay.

Figure 6-9: Sediment thickness for sediment transport module



6.3.4.3 Boundary Conditions

The boundary conditions of the sediment transport module have been set up as open boundaries, i.e. sediment is able to enter or leave the model domain based on the local wave and current conditions.

6.4 Model Validation

Explicit model calibration as was done for the regional wave model has not been possible for the coupled sediment transport model. This is because measured sediment transport rates or changes in bathymetry before and after specific storm events are not available. Instead, a process of model validation has been performed, by comparing the modelled bathymetry to available site data to judge the correctness of the coastal mechanisms presented by the numerical model.

The site data that has been used is the description of the seabed characteristics (see Section 2.6), as well as the results of the bay-wide sediment grab sampling campaign in Rupert's Bay (see Section 2.7.2). Figure 2-18 indicating the seabed characteristics has been repeated here for readability.

Figure 6-10: Rupert's Bay sub-sea geology (repeated Figure 2-18)

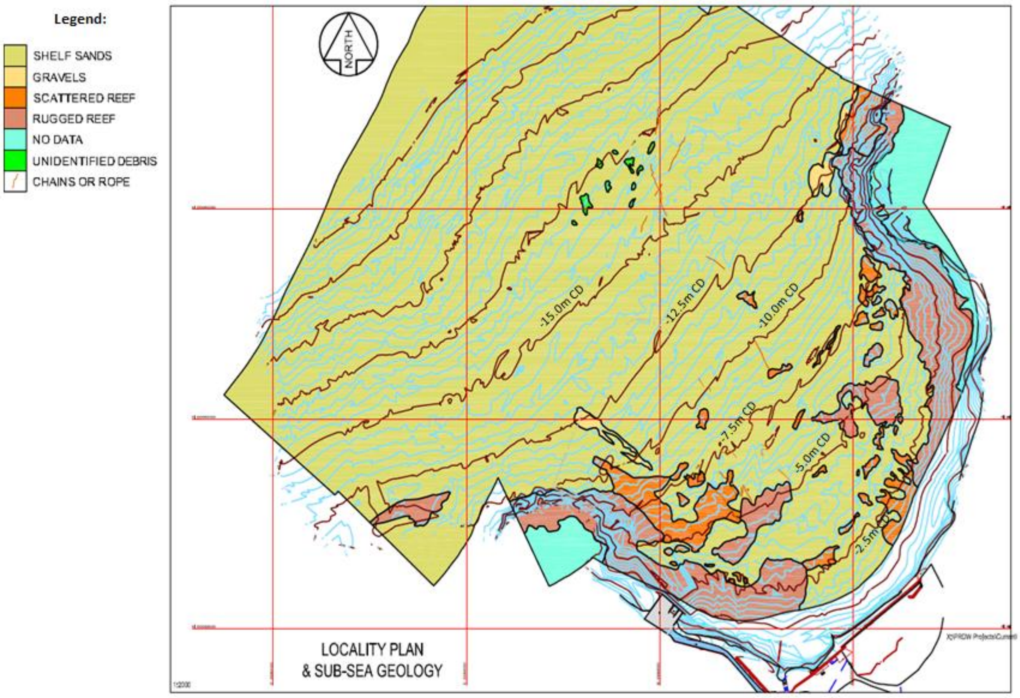


Figure 6-11 shows the changes in bathymetry following a 100 year return period storm event, using a storm direction of 320° and a constant peak period of 16.6 s. Erosion is identified along the south-western area of Rupert's Bay, as well as offshore the swimming beach and along the north-eastern boundary of the bay.

Figure 6-11: Bed level change for base case 100-year return period storm event – base case

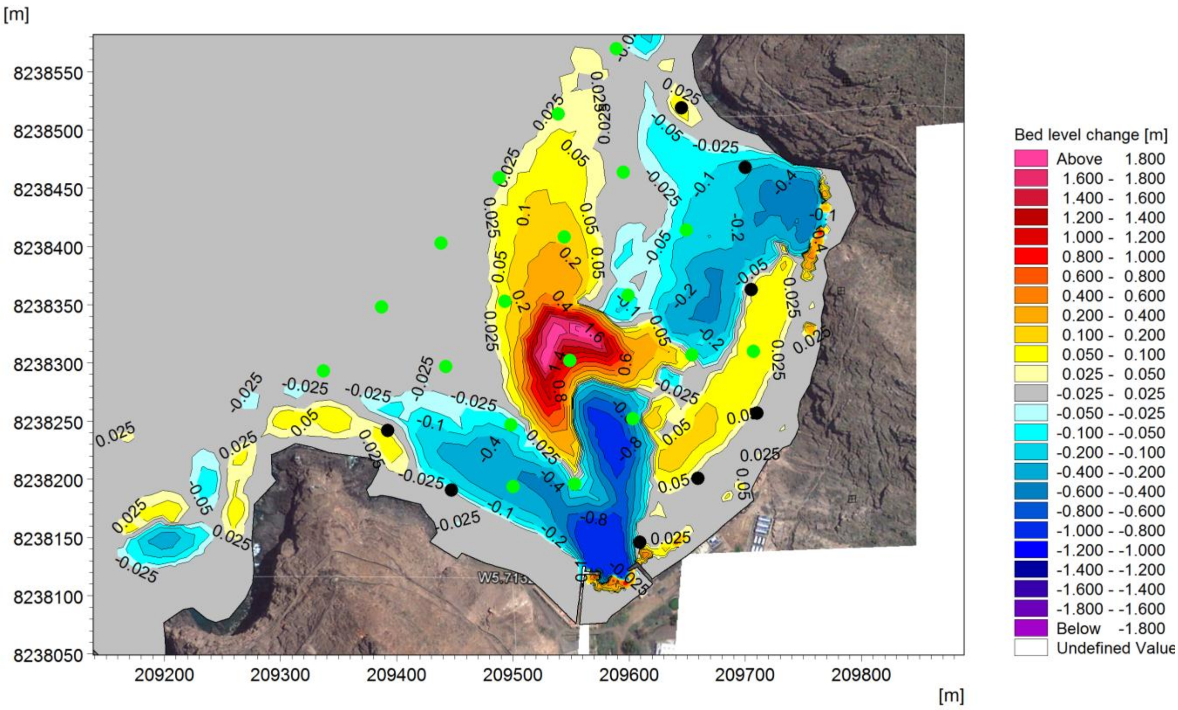


Figure 6-11 also includes the results of the bay-wide sediment sampling campaign, the green dots indicating the presence of sand, whilst the black dots indicate the presence of rock.

An initial model validation was done by comparing the areas of accretion and erosion as identified by the numerical model to the results of the bay-wide sediment sampling campaign, specifically to the areas in which rock was identified. A simplified criterion for model accuracy can be hypothesized as the model needing to predict accretion in those areas in which sediment has been found in the sampling campaign. Generally, this criterion is satisfied across the entire model domain, although a few rock locations are situated along the borders of areas of accretion. The thickness of accretion in these areas is however very thin, between 0 cm and 5 cm. Following this criterion, the model seems to accurately replicate the sediment transport mechanisms in Rupert's Bay.

A further model validation was done by comparing Figure 6-10 to Figure 6-11, referring specifically to the location of the rock reefs shown in Figure 6-10 to the areas of erosion indicated in Figure 6-11. From this, a close resemblance can be identified. It stands to reason that the areas in which rock reefs are located are those areas in which, if rock were absent, erosion would occur.

Although model calibration has not been possible, based on the two model validation comparisons, the coupled sediment transport model seems to accurately simulated the movement of sediment within Rupert's Bay. It is therefore used to investigate the direct impact of the proposed development on the sediment transport regime within the bay.

6.5 Modelling Results

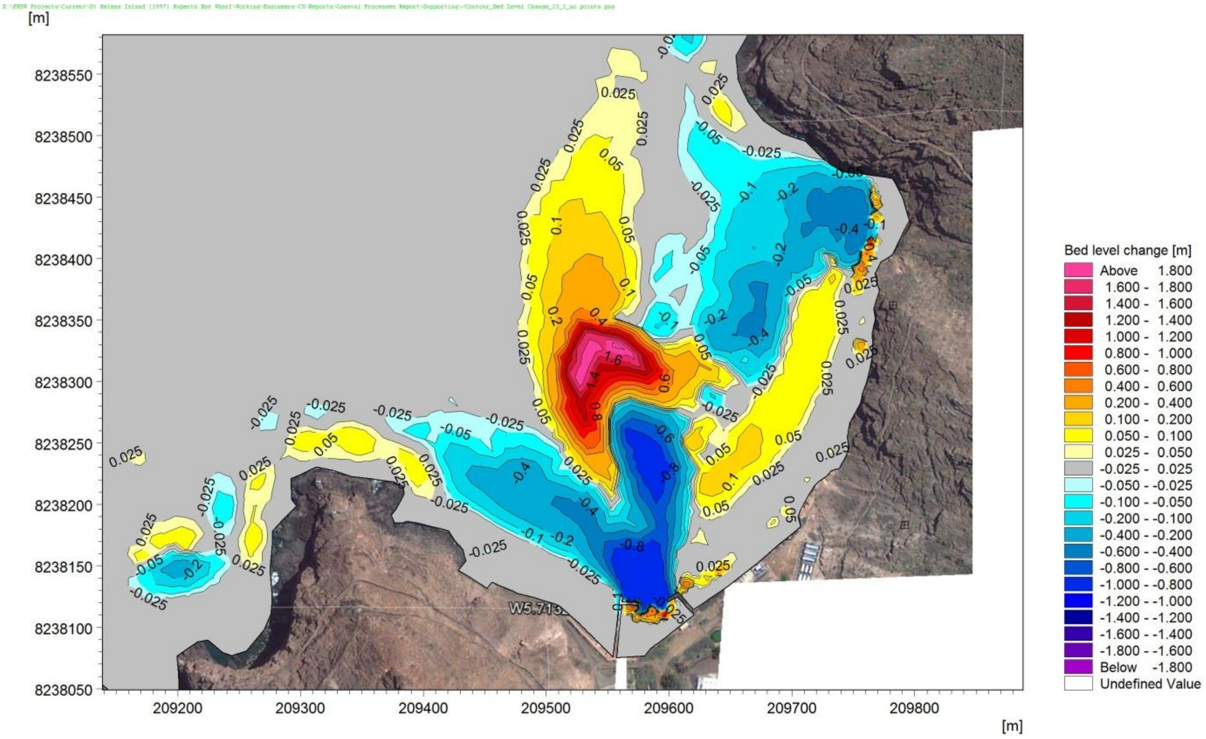
6.5.1 Sensitivity Analyses

As introduced previously, a number of sensitivity analyses were performed to test the influence of a range of parameters on the sediment transport regime in Rupert's Bay. Each of the sensitivity tests were compared against a so-called base-case simulation, the result of which is given in Figure 6-12. The following parameters were used in the base-case simulation, with a return period of 100 years:

- Peak Period T_p 16.6 s
- Bay median grain size D_{50} 0.15 mm
- Wind speed 20.7 m/s
- Mean wave direction 320^o

The impact of varying the above parameters is discussed in the following sections.

Figure 6-12: Base case simulation for sensitivity analysis (100-year return period event)



6.5.1.1 *The Effect of Peak Wave Period*

The impact of the peak wave period on the sediment transport characteristics was tested by performing two simulations reducing and increasing the peak period by two seconds respectively.

Comparing Figure 6-12 to Figure 6-14, a similar accretion and erosion pattern is observed regardless of wave period. A longer wave period does however result in the sediment being deposited further offshore compared to a shorter wave period. This is caused by the increased wave stirring and resulting turbulence of long period waves, which result in sediment remaining in suspension for a longer duration.

Comparing the accretion isopachs presented in Figure 6-13 and Figure 6-14, a shorter wave period results in increased sedimentation compared to a longer wave period. This difference is caused by the reduced distance over which sediment is transported for the shorter wave period, resulting in most sediment being deposited in one central area. For the longer wave period, sediment is transported for a longer duration, and is therefore distributed over a greater area, resulting in a smaller deposition isopach.

It is concluded that, although the wave period does have an impact on the sediment transport regime, specifically on the duration over which sediment remains in suspension, it does not change the overall transport mechanism. Conclusions drawn from analysing the impact of the permanent wharf structure using the base case wave period are therefore considered to be sufficiently accurate for this level of study.

Figure 6-13: Base case simulation with T_p=14.6 seconds

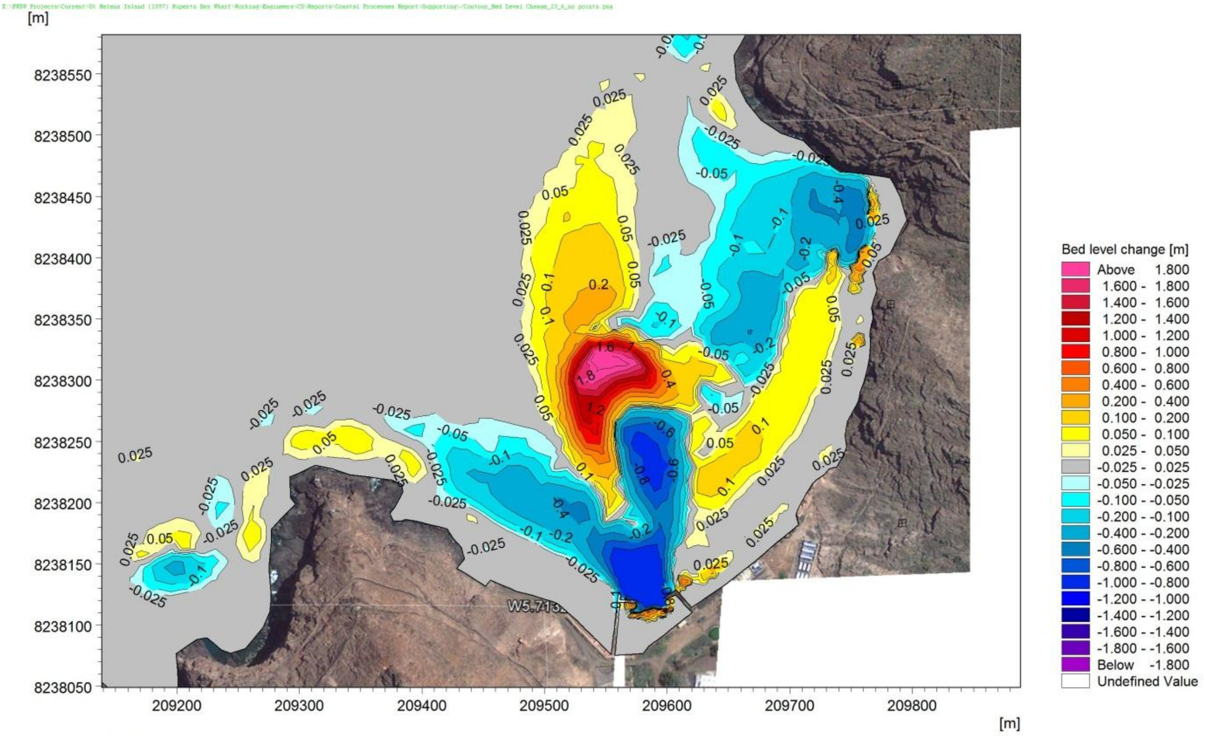
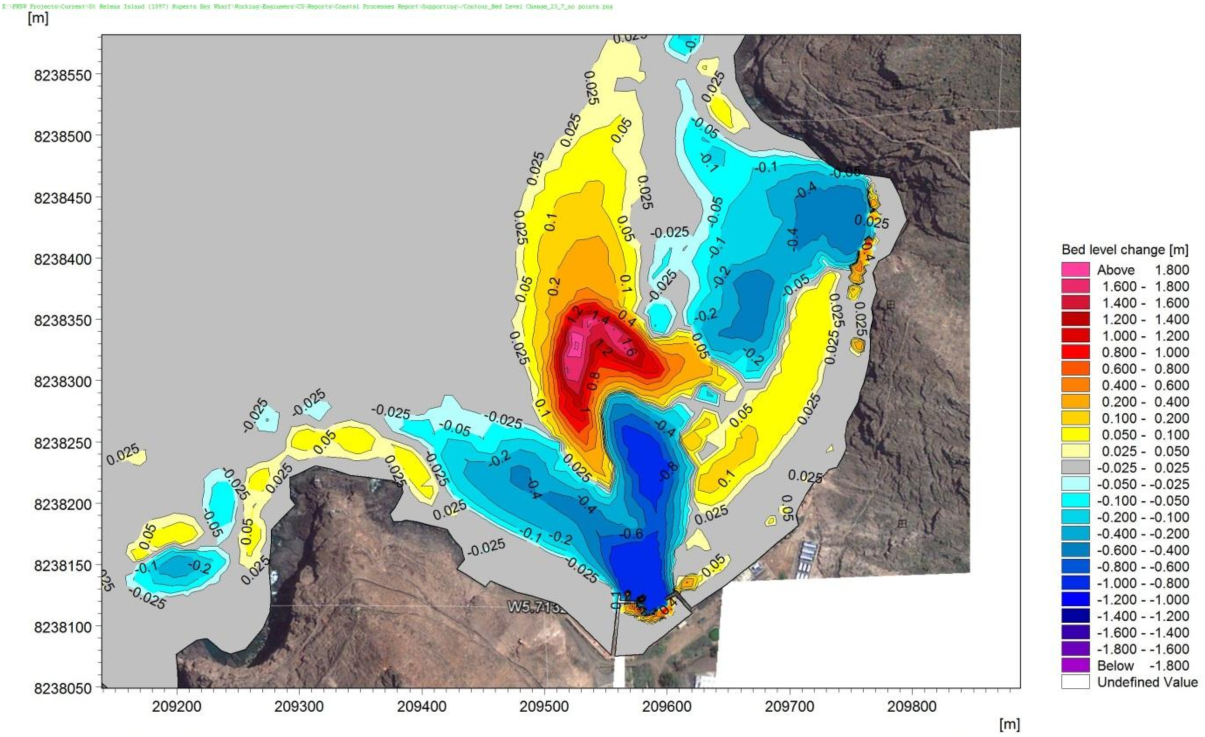


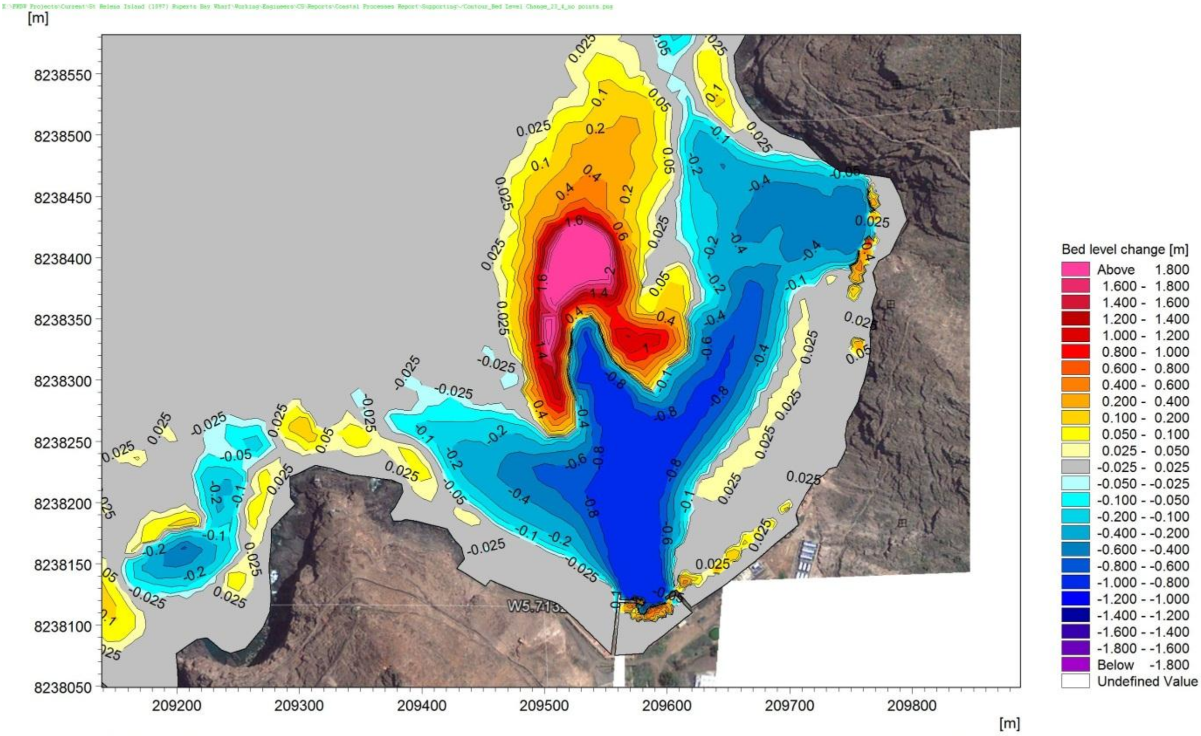
Figure 6-14: Base case simulation with T_p=18.6 seconds



6.5.1.2 The Effect of Median Grain Size of Rupert's Bay

The impact of the median grain size in Rupert's Bay, excluding the areas on the two beaches as discussed in Section 6.3.4.1, has been tested by reducing and increasing the base-case grain size by 0.05 mm respectively. The results of this analysis are provided in Figure 6-15 and Figure 6-16.

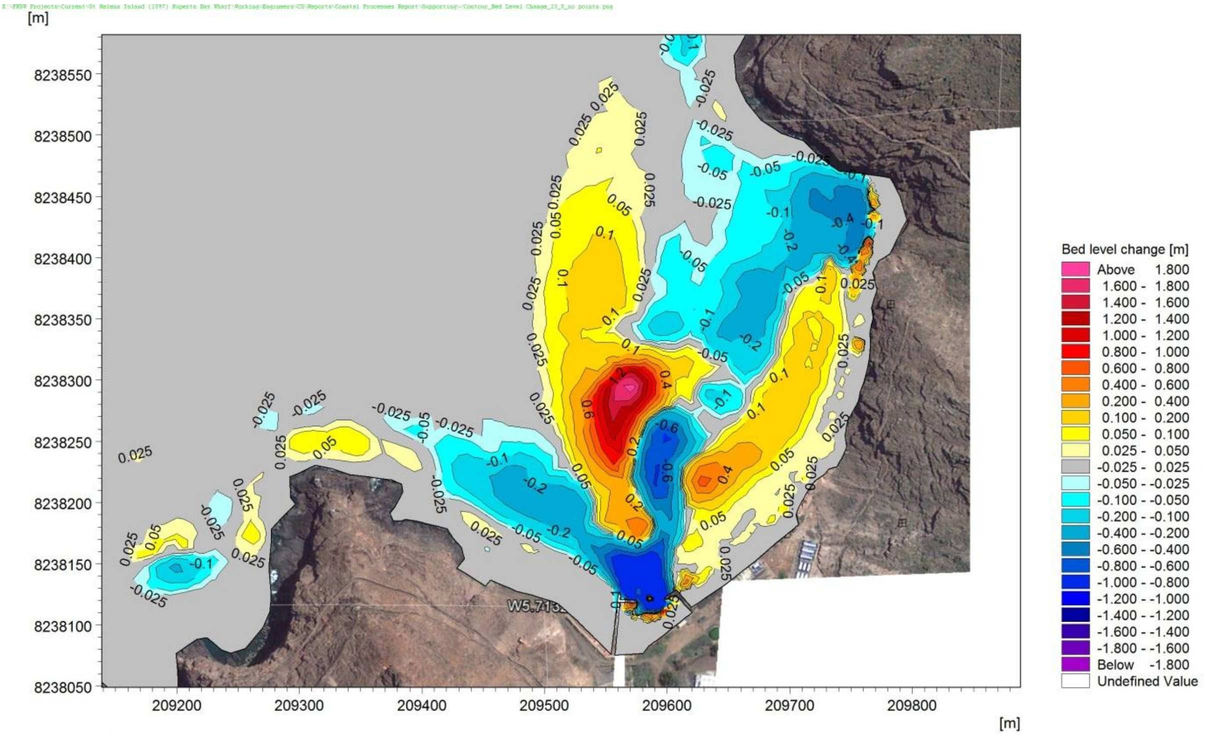
Figure 6-15: Base case simulation with D_{50}=0.01 mm



Based on these figures, it is clear that a change in median grain size has a significant impact on the sediment transport characteristics in Rupert's Bay. A reduction in grain size results in sediment being transported further offshore, since it remains in suspension for a longer period of time. In addition, erosion isopachs are increased for a finer grain size, since the transport rates are increased for finer material, which means that a larger volume of sediment is put into suspension for the same environmental conditions.

Although the median grain size clearly impacts the sediment transport characteristics in Rupert's Bay, the sensitivity analysis shows that the transport patterns remain approximately the same. As such, conclusions drawn from analysing the impact of the permanent wharf structure using the base-case sediment grain size are considered to be sufficiently accurate for this level of study.

Figure 6-16: Base case simulation with D_{50}=0.02 mm



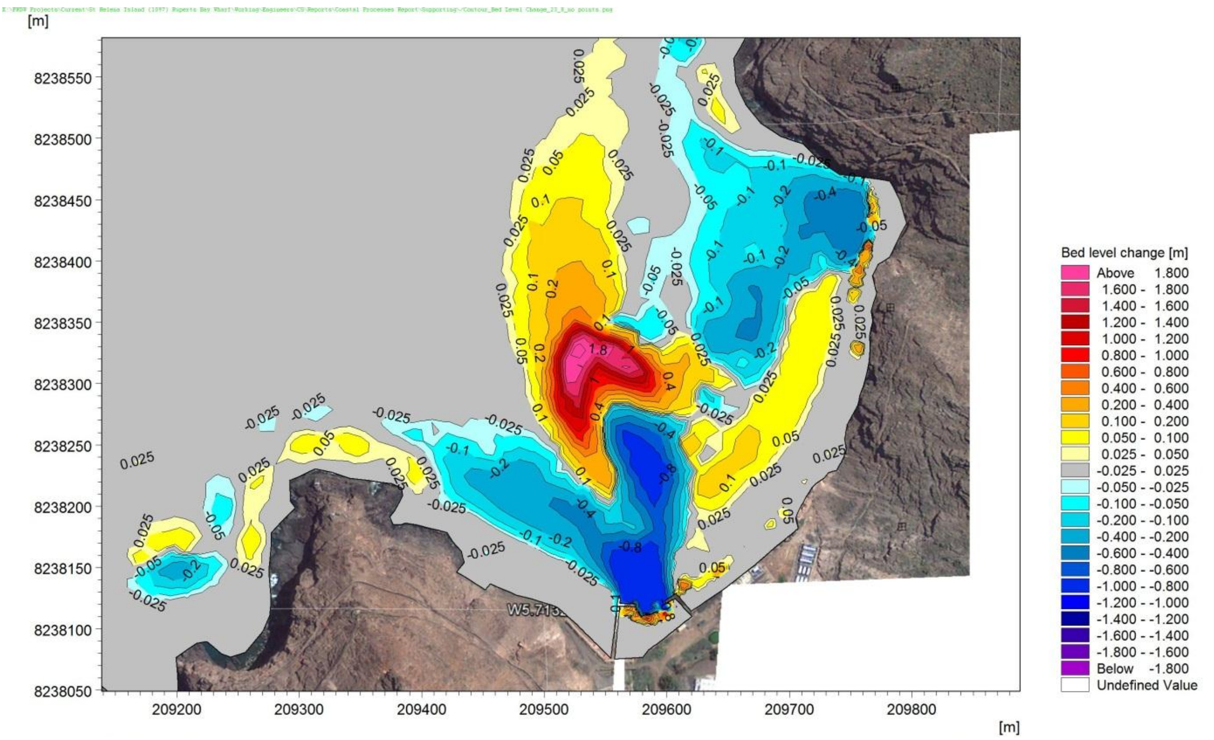
6.5.1.3 The Effect of Wind

The effect of the wind on the Rupert’s Bay sediment transport regime was tested by performing an additional simulation excluding all wind forcing. The result of this simulation is provided in Figure 6-17.

Comparing this figure to the result of the base-case simulation (see Figure 6-12), it is observed that the exclusion of wind does not have a significant impact on the Rupert’s Bay sediment transport regime. A slight rotation of the sedimentation and erosion areas is observed, however, these are not significant, and the accretion and erosion isopachs remains approximately constant.

Conclusions drawn from the base-case simulation are therefore accurate to account for all wind conditions in Rupert’s Bay.

Figure 6-17: Base case simulation excluding wind forcing



6.5.1.4 The Effect of Mean Wave Direction

The effect of the mean wave direction on the Rupert's Bay sediment transport regime was tested by varying the base case wave direction by ten degrees either side. The results of this simulation are shown in Figure 6-18 and Figure 6-19.

From these figures, it is clear that a rotation of the mean wave direction does not significantly affect the sediment transport characteristics of Rupert's Bay. Although a slight rotation of the accretion and erosion areas is identified, the magnitude of the bed level changes remains approximately constant.

As such, conclusions drawn by analysing the results of the base-case simulation are considered to be accurate, and account for rotations in offshore wave climate.

Figure 6-18: Base case simulation with MWD=310^{o}

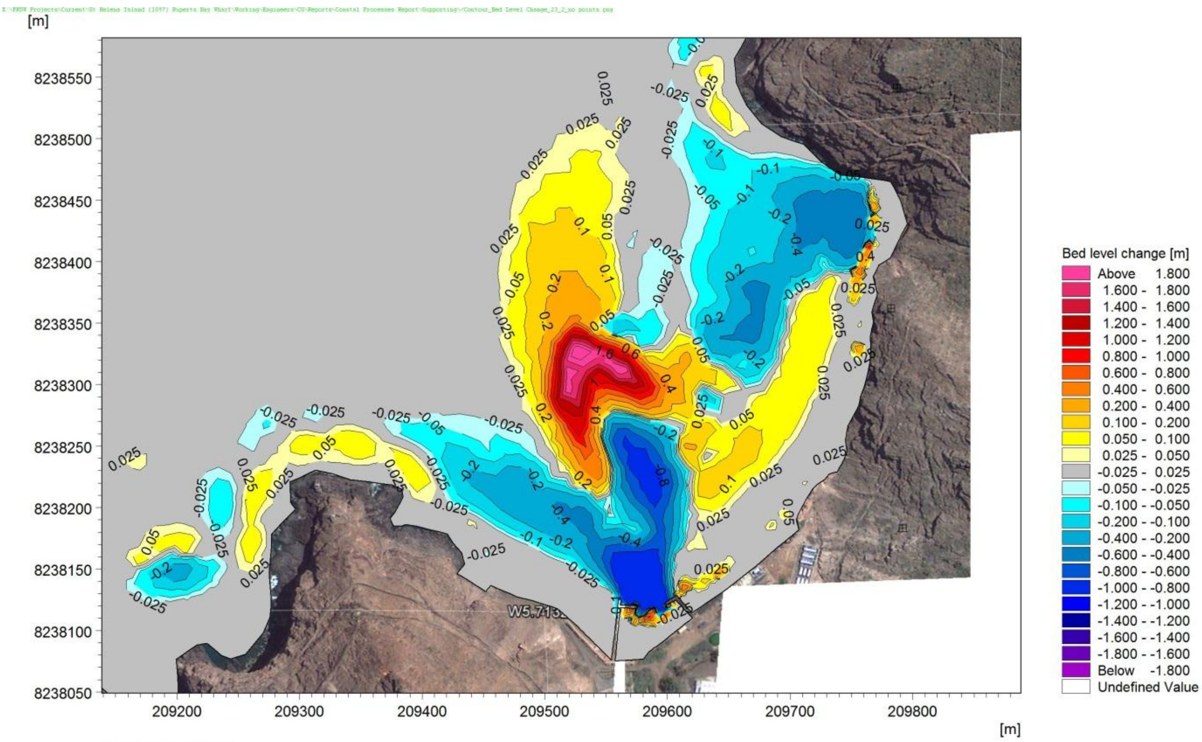
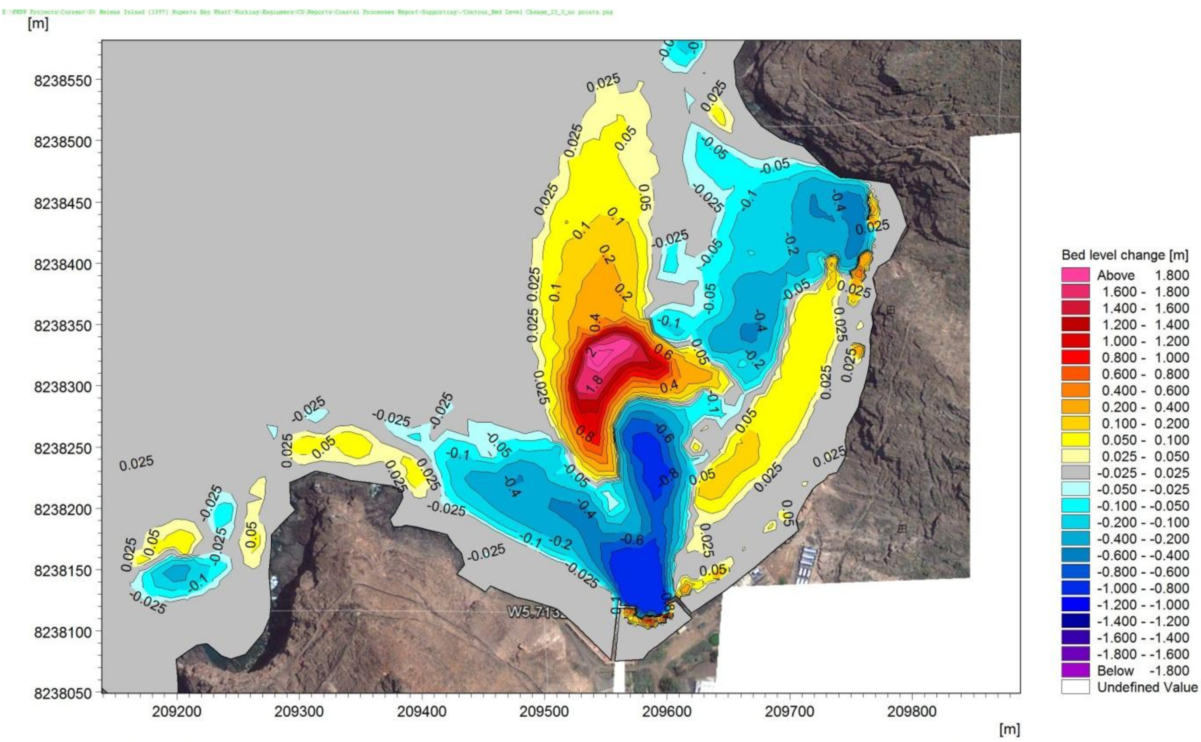


Figure 6-19: Base case simulation with MWD=330^{o}



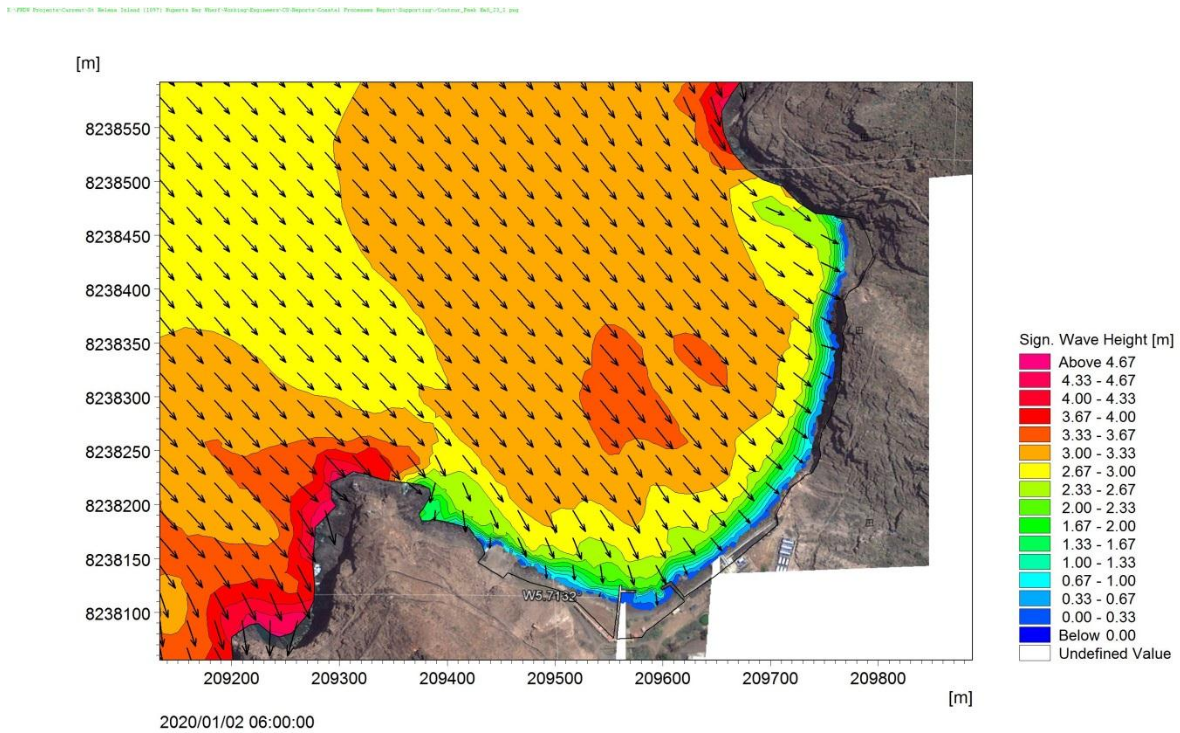
6.5.2 Rupert's Bay Sediment Transport Regime – Status Quo

To be able to analyse the direct impact of the proposed development, a sound understanding of the status quo is required. This understanding, incorporating waves, hydrodynamics and the resulting sediment transport regime, is discussed in the following sections.

6.5.2.1 Waves

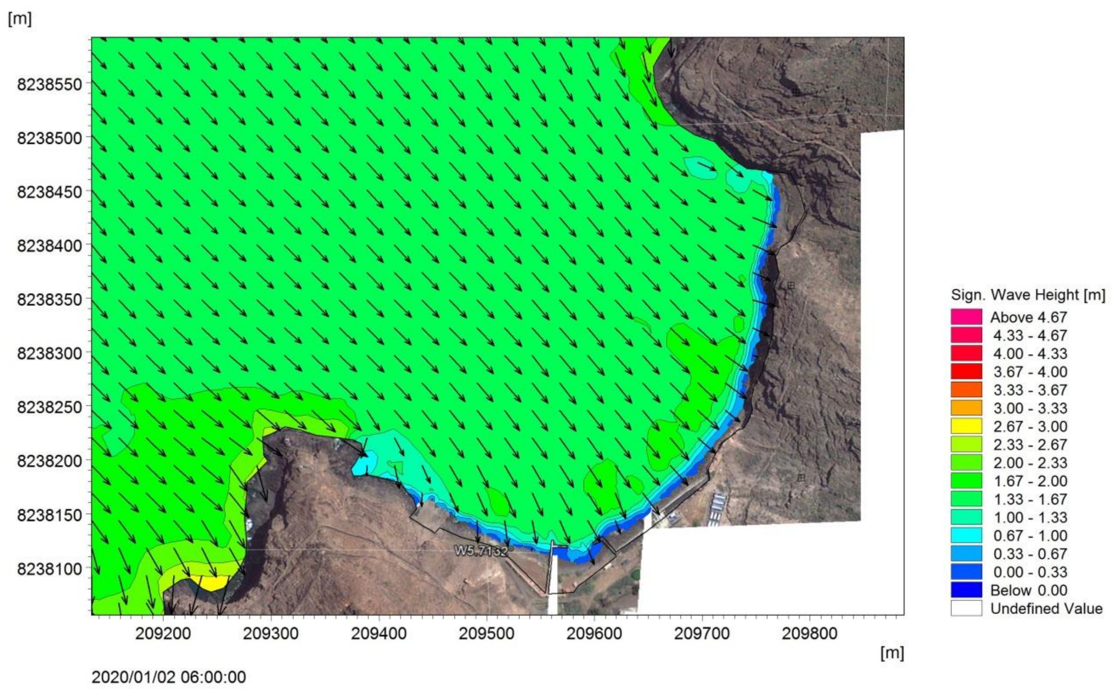
The significant wave heights within Rupert's Bay at the peak of the 100- and 1-year return period events are provided in Figure 6-20 and Figure 6-21 respectively. Wave heights are significantly reduced for the 1 year return period event, with a significant wave height of around 2.0 m in the bay, compared to the between 3.0 m and 4.0 m for the 100 year return period event.

Figure 6-20: Status quo – waves – 100-year return period event



Waves rotate in an anti-clockwise and clockwise direction along the northern and southern boundaries of Rupert's Bay respectively, due to the process of refraction. Wave heights peak in the central area of the bay due to wave shoaling and reflection.

Figure 6-21: Status quo – waves – 1-year return period event



6.5.2.2 Hydrodynamics

The current speed and direction within Rupert's Bay at the peak of the 100- and 1-year storm events are provided in Figure 6-22 and Figure 6-23, respectively. The oblique wave angles along the northern and southern boundaries of Rupert's Bay result in the generation of strong wave-driven currents. These currents flow in a clockwise direction in the northern area of the bay, and in an anti-clockwise direction in the southern area of the bay. The opposing currents collide in the area of the fuel-loading arm, generating a strong rip current.

Due to the increased wave height, current speeds are higher for the 100-year event. The stronger rip current for the 100-year event also extends further offshore, compared to the 1-year event.

The currents for the 100- and 1-year return period events at the swimming beach in the south-eastern corner of Rupert's Bay are provided in Figure 6-26 and Figure 6-27 respectively. Due to the protection of the concrete pipe and the small offshore breakwater, relatively low current speeds of around 0.5 m/s are experienced at the swimming beach. It is further observed that current speeds are only marginally stronger for the 100-year event compared to the 1-year event.

It should be noted that if the concrete pipe and rubble mound rock structures were not present, significantly stronger currents would be experienced at the swimming beach, which may result in dangerous bathing conditions.

Figure 6-22: Status quo – current– 100-year return period event

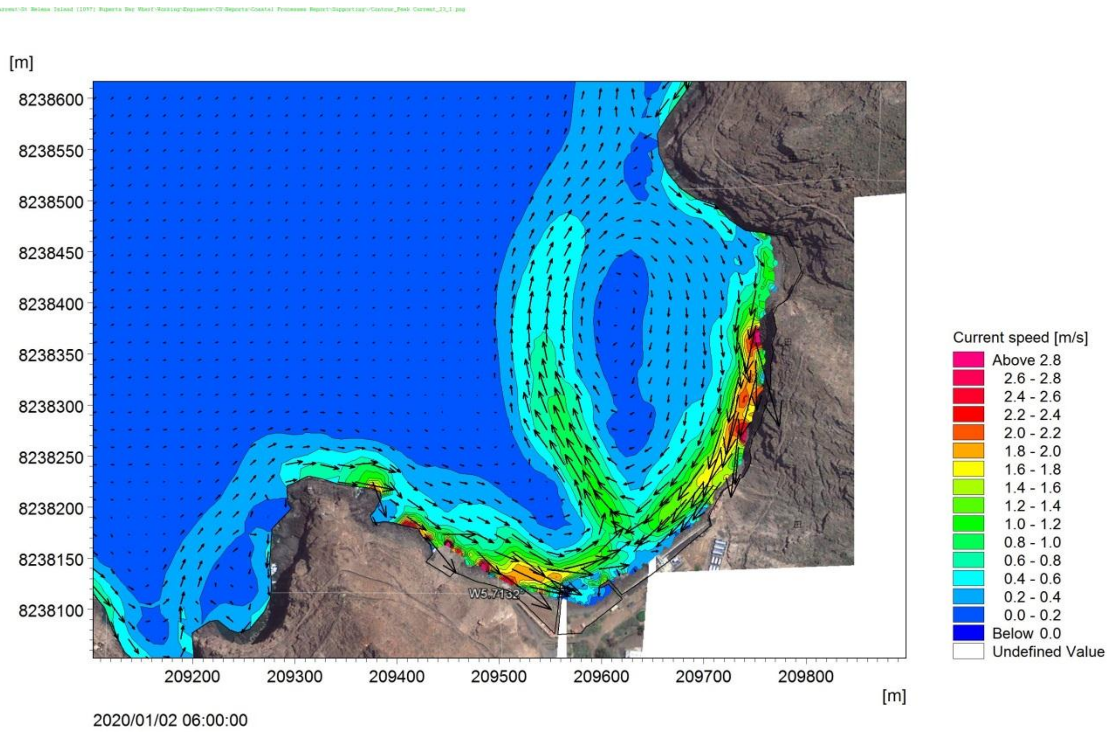


Figure 6-23: Status quo – current– 1-year return period event

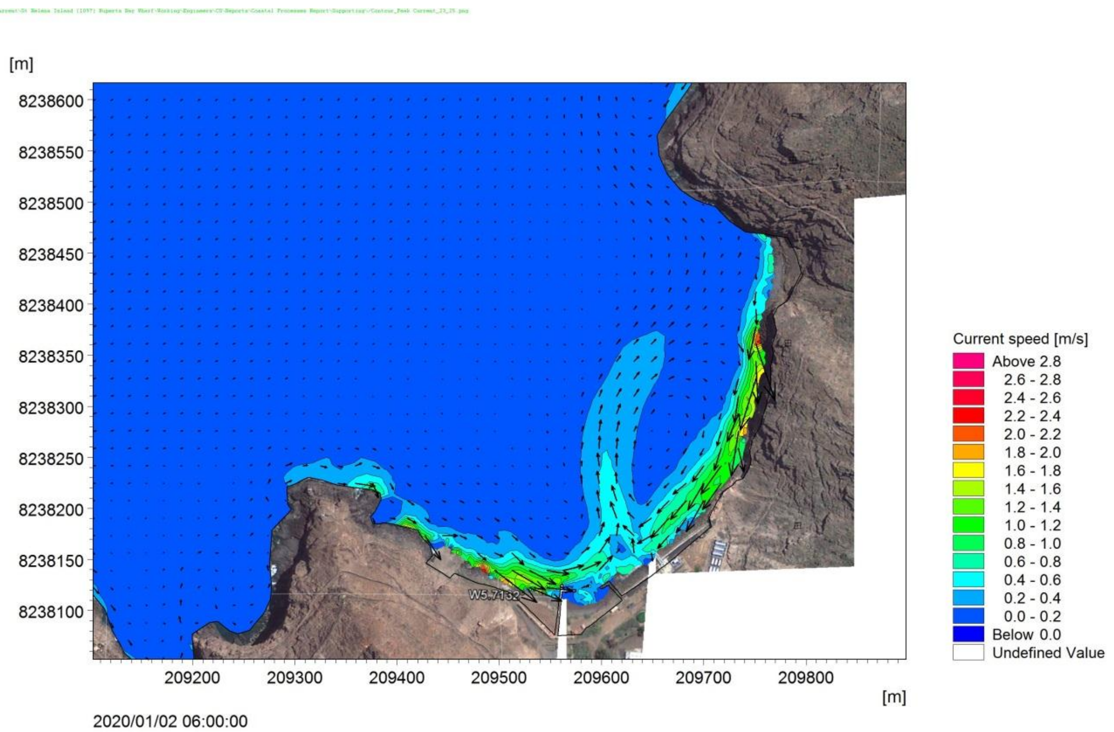


Figure 6-24: Status quo – current at swimming beach – 100-year return period event

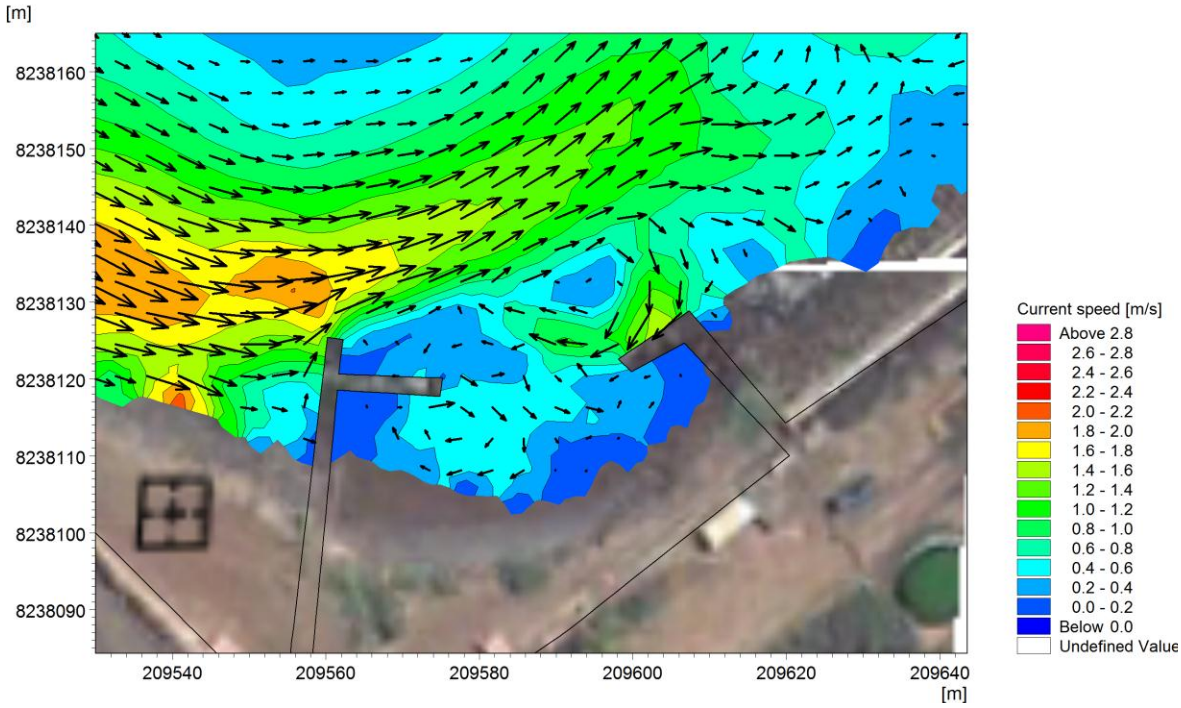
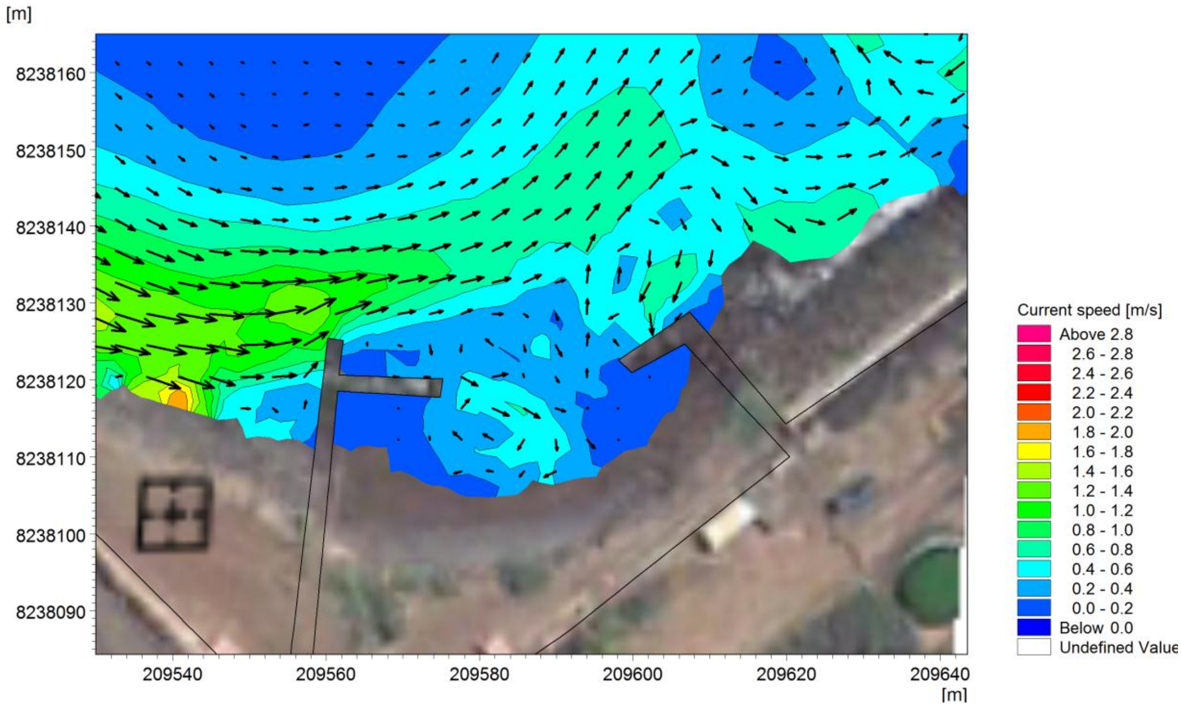


Figure 6-25: Status quo – current at swimming beach – 1-year return period event



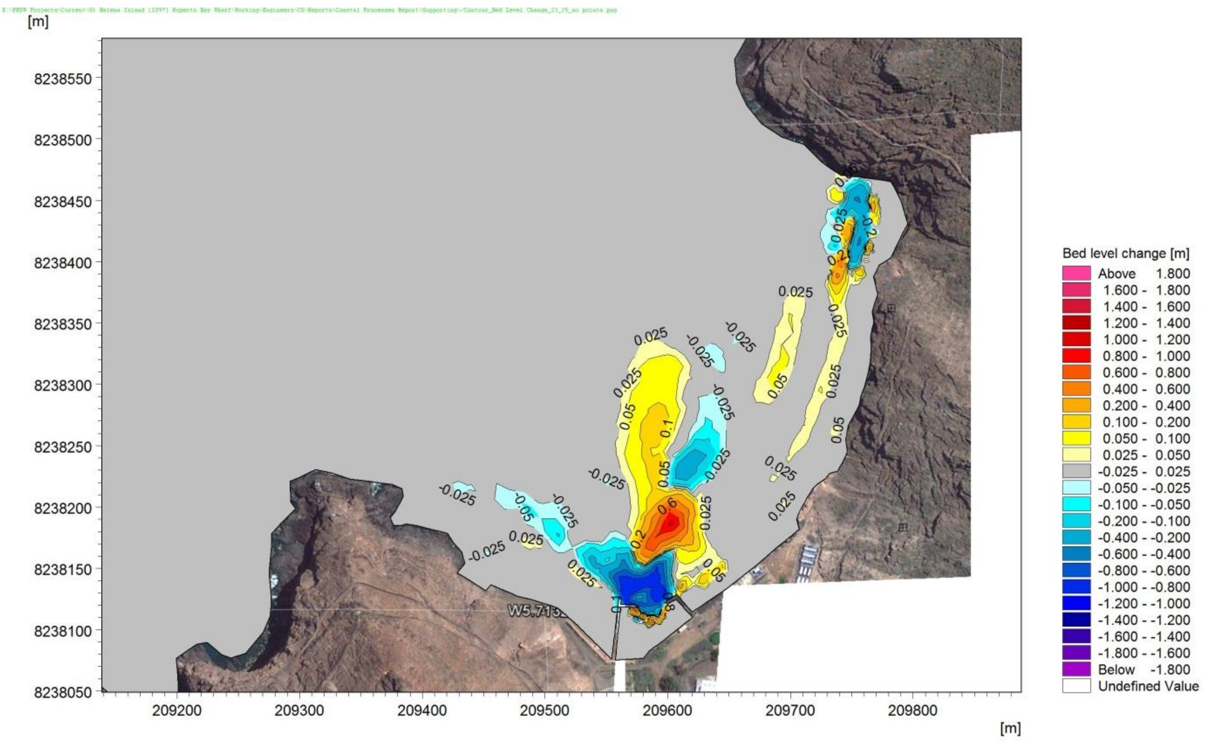
6.5.2.3 Sediment Transport

The change in bathymetry for the 100-year storm event within Rupert's Bay has been presented earlier in Figure 6-12, whilst the accretion and erosion for the 1-year storm event is presented in Figure 6-26 below. For the 100-year return period event, the maximum accretion is approximately

1.8 m, occurring in the central area of Rupert's Bay. Erosion to the extent of 0.8 m is observed immediately offshore of the swimming beach, whilst approximately 0.4 m of erosion is observed immediately offshore of the beach in the north-eastern corner of the bay.

Comparing this to the bed level changes observed during the 1-year return period event, a significant reduction is identified, due to the smaller waves and weaker currents. The maximum accretion in the bay for the 1-year event is approximately 0.6 m, and this is located closer to the shoreline compared to the 100-year return period event. Similarly, erosion in front of both of the beaches is reduced.

Figure 6-26: Status quo – bed level change – 1-year return period event



The modelled bed level changes at the swimming beach in the south-eastern corner of Rupert's Bay during the 100- and 1-year return period event are shown in Figure 6-27 and Figure 6-28 respectively. It is observed that accretion up to approximately 0.5 m is expected in the lee of the offshore breakwaters for the 100-year event, with this reducing to approximately 0.2 m for the 1-year event. The increased sedimentation for the 100-year event is caused by the larger waves and marginally stronger currents during that event compared to the 1-year event.

It is noted that the stability of the swimming beach is critically linked to the presence of the concrete pipe and the small offshore breakwater. The absence of these structures would result in significantly stronger currents being experienced along the beach, which would cause larger volumes of sediment being suspended, resulting in increased erosion.

Figure 6-27: Status quo – bed level change at swimming beach – 100-year return period event

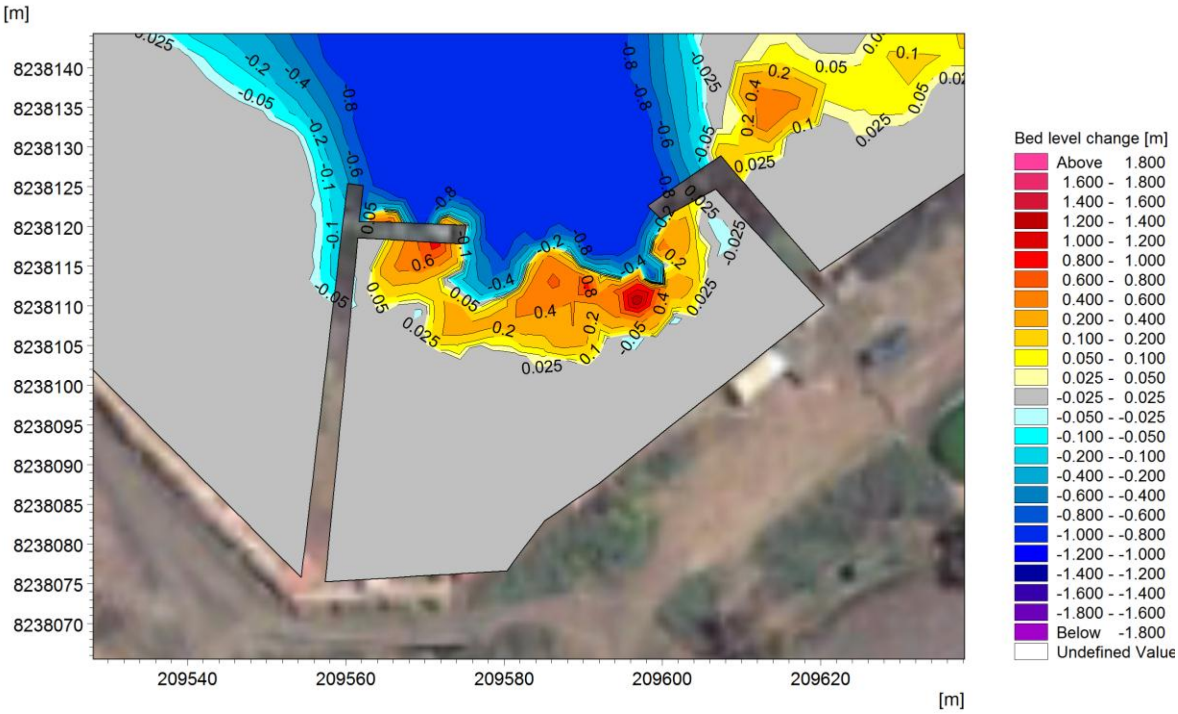
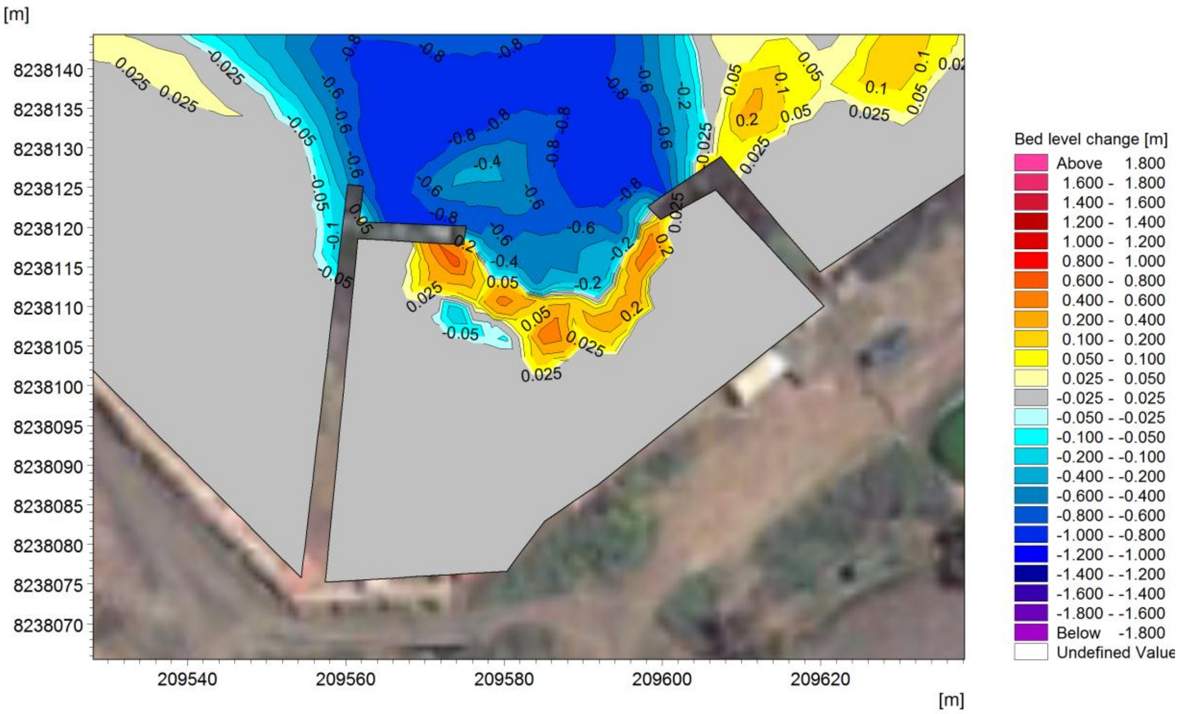


Figure 6-28: Status quo – bed level change at swimming beach – 1-year return period event



6.5.3 Rupert's Bay Sediment Transport Regime – Including Proposed Development

The impact of the proposed permanent wharf development, in terms of waves, hydrodynamics and sediment transport, is discussed in the following paragraphs.

6.5.3.1 Waves

The significant wave height at the peak of the 100- and 1-year return period events are provided in Figure 6-29 and Figure 6-30, respectively. It is clear that the proposed wharf structure results in significant wave sheltering in the southern half of Rupert's Bay, whilst the wave climate in the northern half of the bay remains relatively unaffected. The peak wave height in the central area of the bay is reduced, with the maximum bay-wide wave height being between 3.0 m and 3.5 m.

The wave height at the swimming beach is not affected significantly by the wharf structure. Due to the shallow depth, wave heights in this area are depth-limited, which means that the maximum wave height is fixed regardless of offshore condition.

Figure 6-29: Including development – waves – 100-year return period event

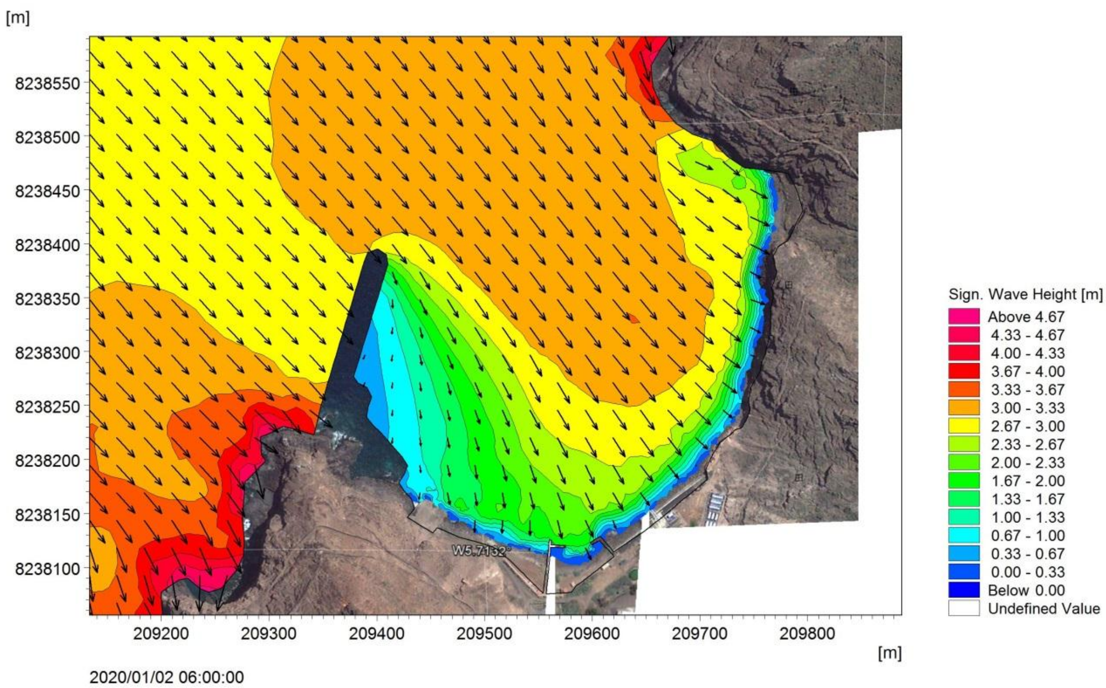
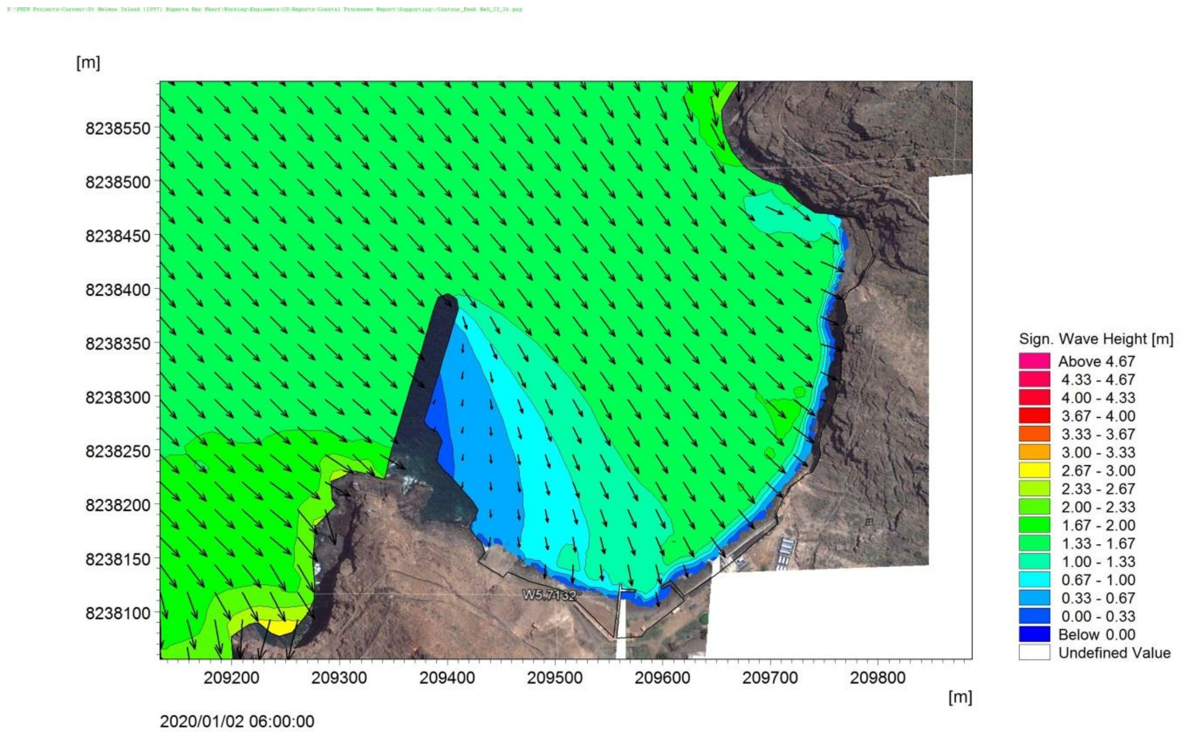


Figure 6-30: Including development – waves – 1-year return period event



6.5.3.2 Hydrodynamics

The current speed and direction at the peak of the 100- and 1-year return period events including the proposed wharf structure are presented in Figure 6-31 and Figure 6-32, respectively. Comparing these to Figure 6-22 and Figure 6-23, it can be observed that the anti-clockwise and clockwise current regime in the status quo is expected to change to become clockwise in the entire bay. This is caused by the wave sheltering effect of the wharf structure, which causes the clockwise wave-driven current in the southern portion of the bay no longer being generated. This means that there is no longer a current with which the clockwise current collides, resulting in the rip current no longer being generated.

Comparing Figure 6-33 and Figure 6-34 to Figure 6-24 and Figure 6-25 respectively, the reversal of the current immediately offshore of the swimming beach is identified. Current speeds in this area remain similar, at around 1 m/s for the 100-year return period event.

On the swimming beach, current speeds remain weak and relatively unchanged. In addition, the difference in current speed between the 100- and 1-year return period event remain marginal.

Figure 6-31: Including development – current – 100-year return period event

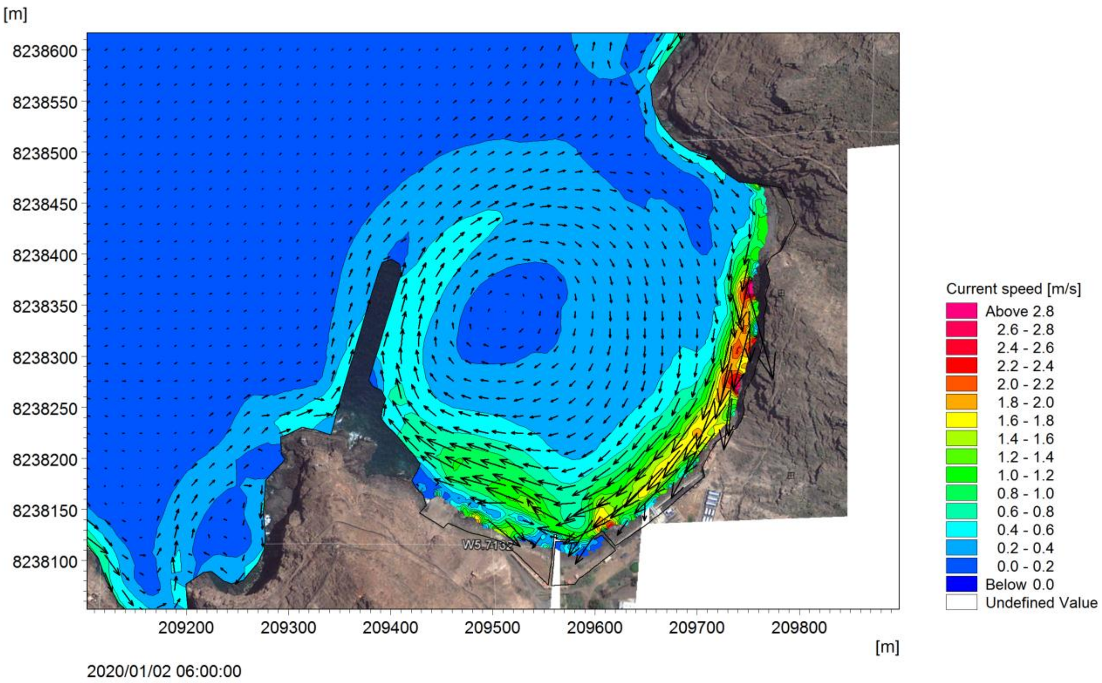


Figure 6-32: Including development – current – 1-year return period event

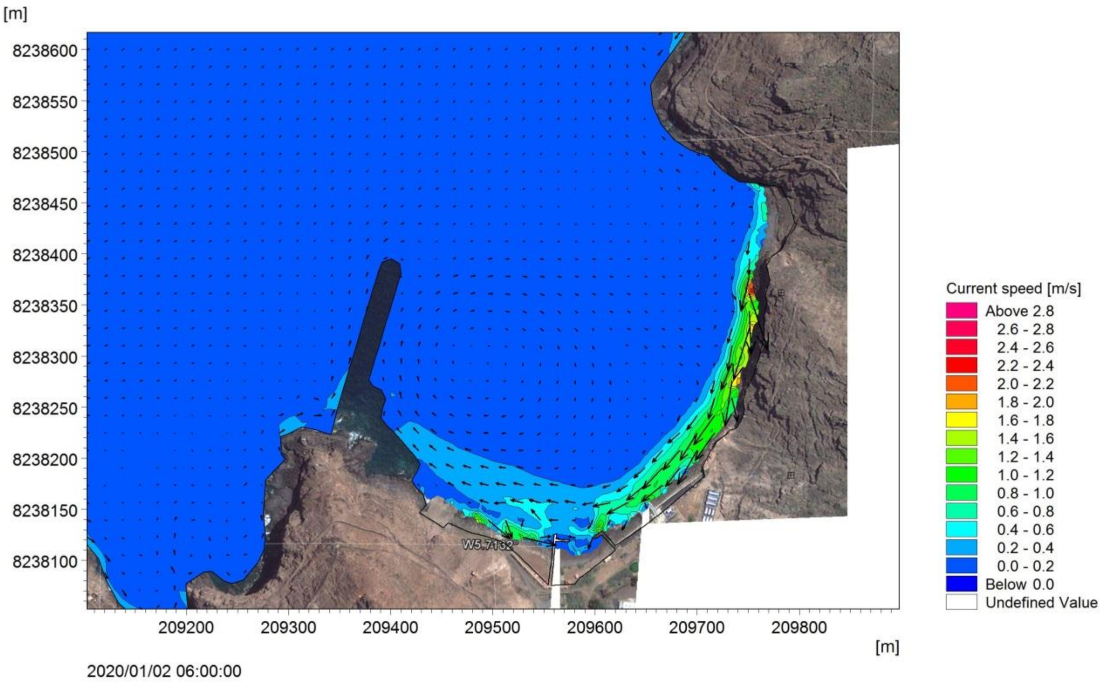


Figure 6-33: Including development – current at swimming beach – 100-year return period event

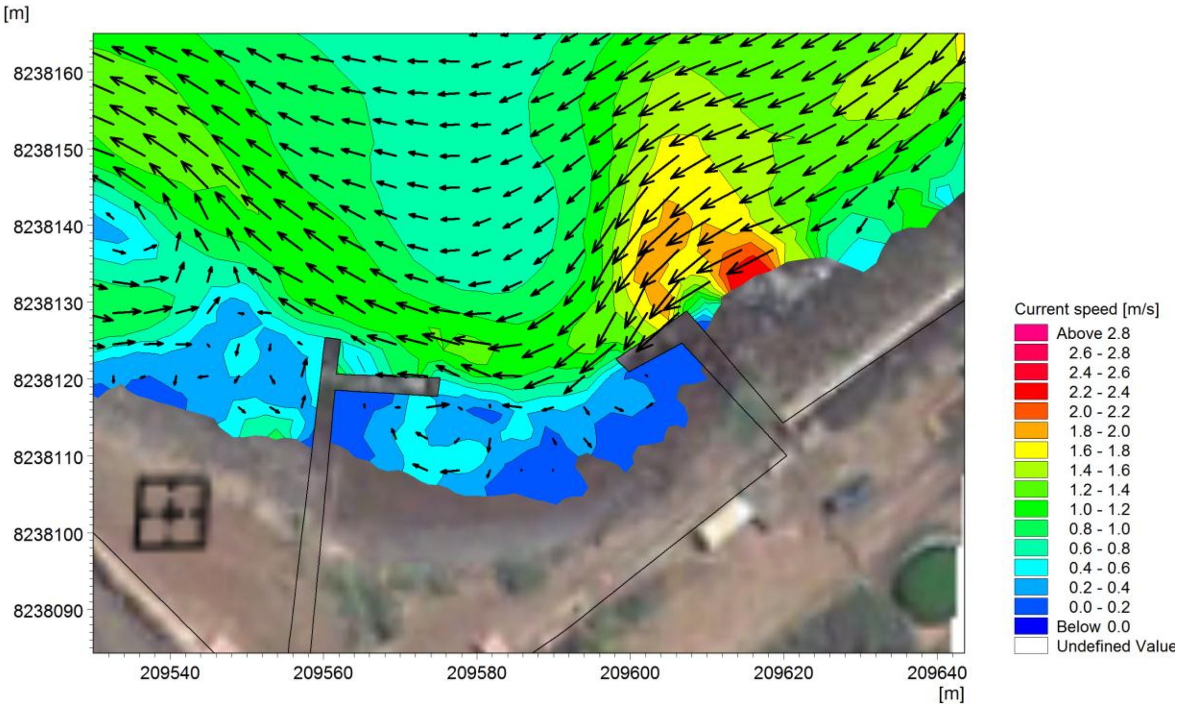
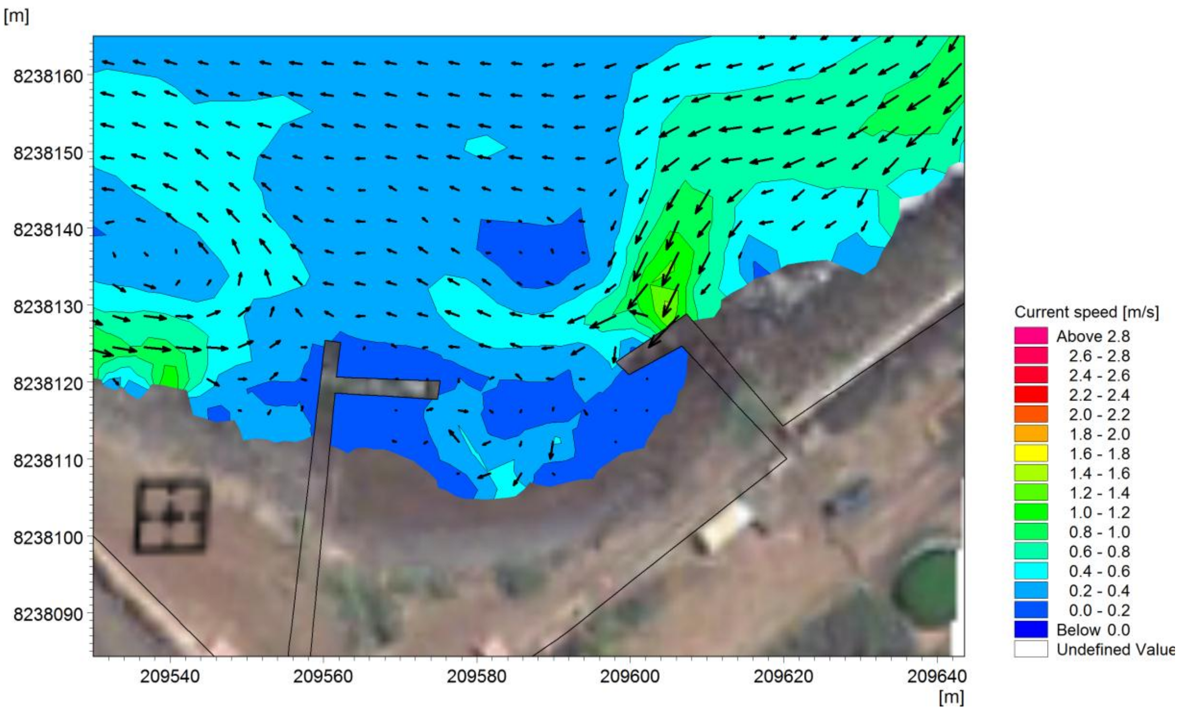


Figure 6-34: Including development – current at swimming beach – 1-year return period event



6.5.3.3 Sediment Transport

The change in bathymetry for the 100- and 1-year return period event including the proposed wharf development is provided in Figure 6-35 and Figure 6-36, respectively. The navigation areas of the proposed development have been superimposed on the figures. From these figures, it is clear that the sediment transport mechanism has changed in response to the change in wave and current climate.

For the 100-year return period event, sedimentation of up to 1.4 m is expected to occur in the lee of the proposed structure. Erosion patterns on the northern extent of the bay are generally expected to remain unchanged, with erosion expected to occur immediately offshore of the north-eastern beach. Similarly, erosion immediately offshore of the south-eastern swimming beach is expected to remain similar to the status quo.

It is noted that due to the expected sedimentation along the south-eastern area of Rupert's Bay, the exposed rock reefs are likely to be covered by sand, and may have an adverse environmental impact.

For the 1-year return period event, sedimentation to the extent of approximately 0.5 m is expected to occur offshore of the south-eastern swimming beach, which is similar to the sedimentation magnitude expected for the status quo situation.

Referring to Figure 6-35, minimal sedimentation of the facility's navigational areas is expected for the 100-year return period event, whilst no sedimentation is expected for the 1-year event. The limited sedimentation is expected to be highest in the south-eastern corner of the berth pocket, reaching approximately 0.1 m during the 100-year return period event. No sedimentation is expected in the approach channel and turning circle for either of the events.

The impact of the proposed development on the stability of the south-eastern swimming beach is presented in Figure 6-37 and Figure 6-38. Similar to the status quo, limited accretion is expected to occur on the beach, whilst erosion to the extent of approximately 0.8 m is expected immediately offshore. As such, the proposed development is not expected to have a significant impact on the stability of the swimming beach in Rupert's Bay.

Figure 6-35: Including development – bed level change – 100-year return period event

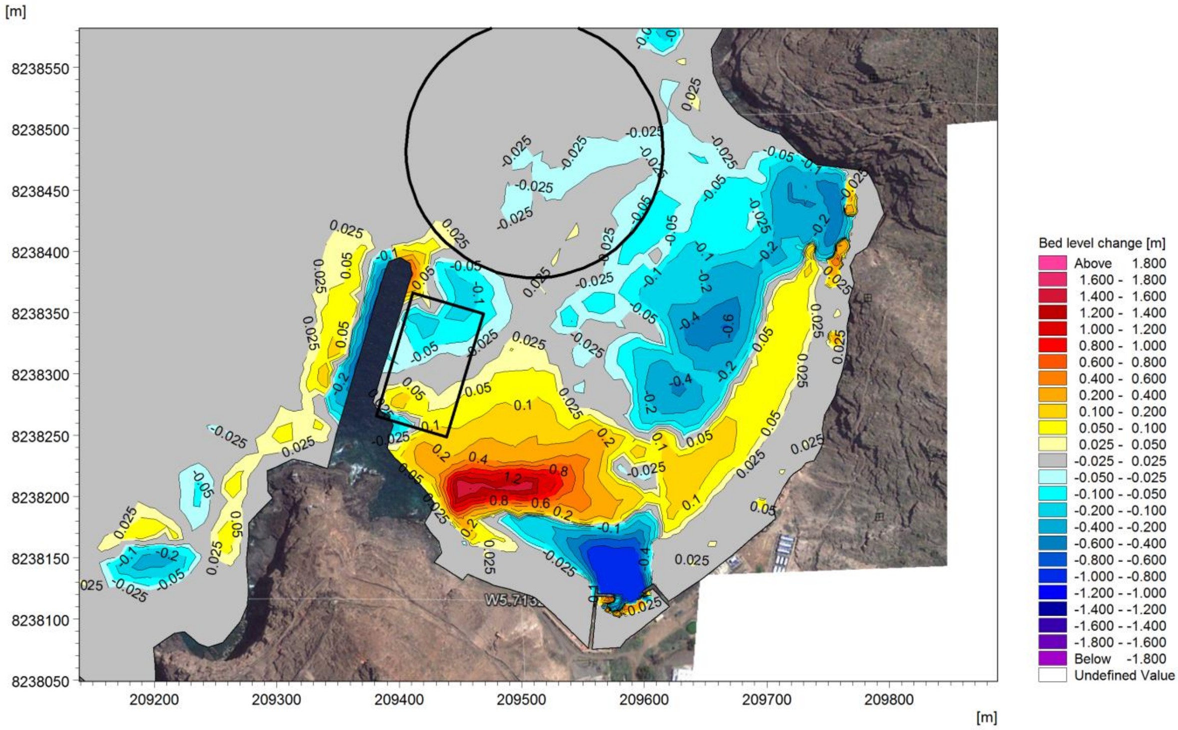


Figure 6-36: Including development – bed level change – 1-year return period event

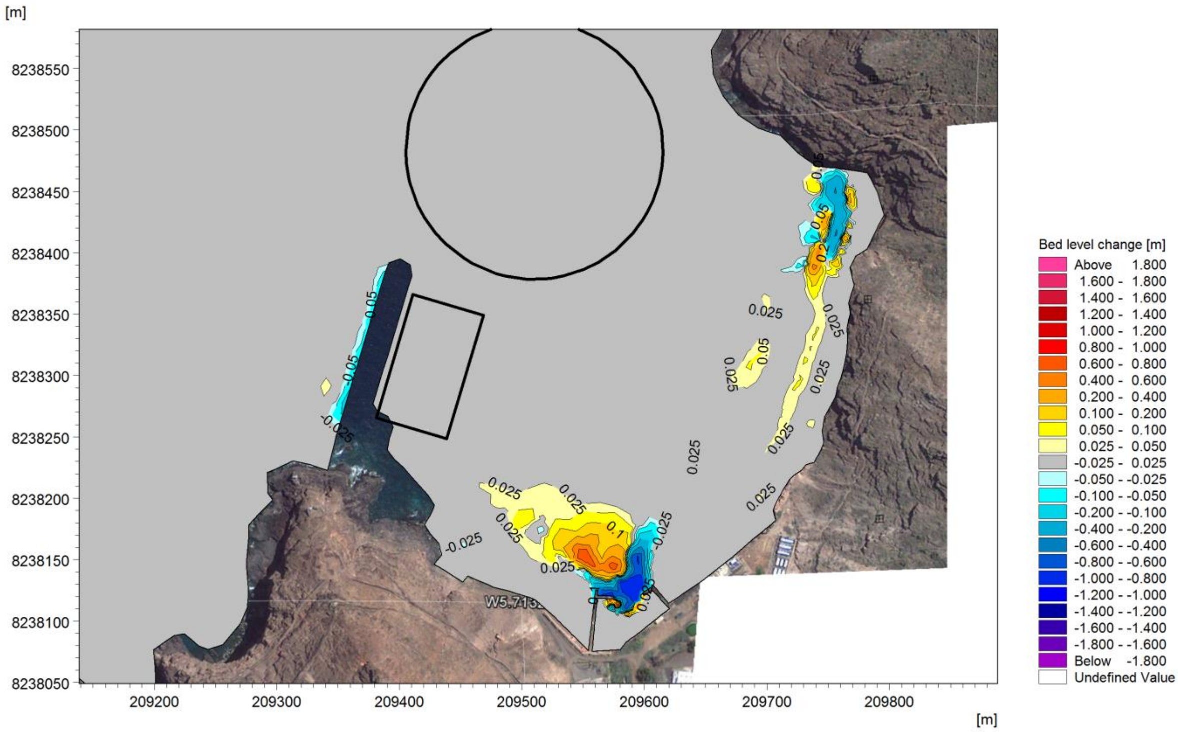


Figure 6-37: Including development – bed level change at swimming beach – 100-year return period event

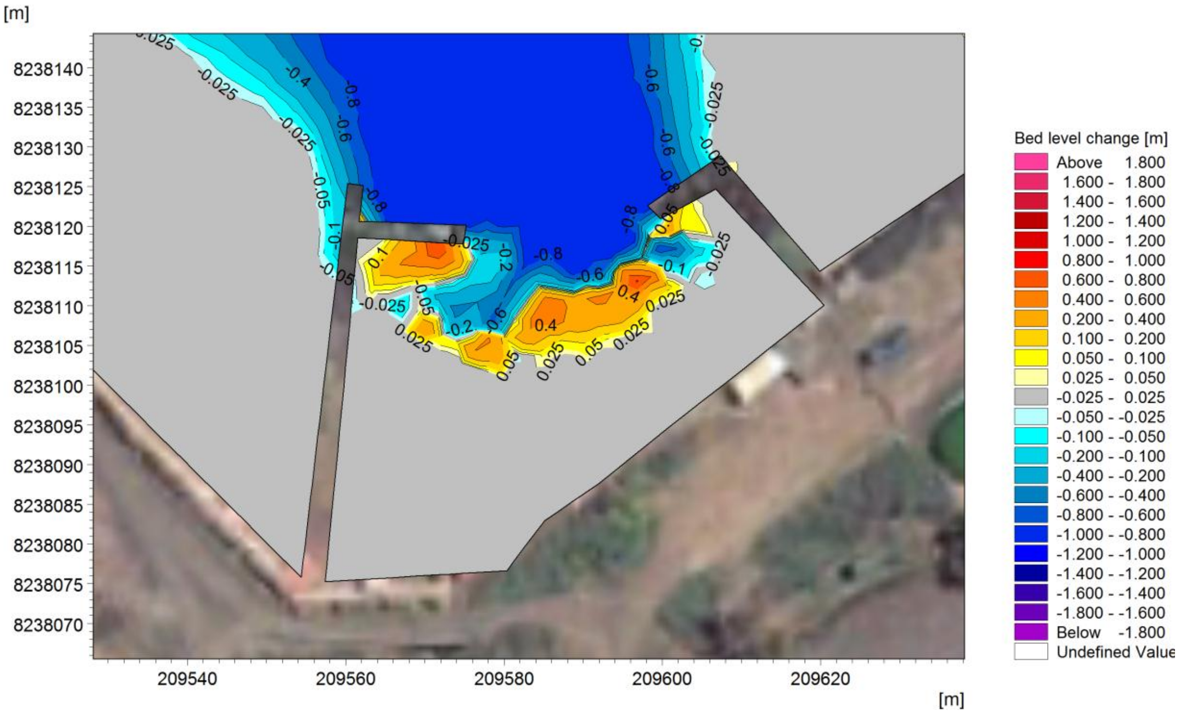
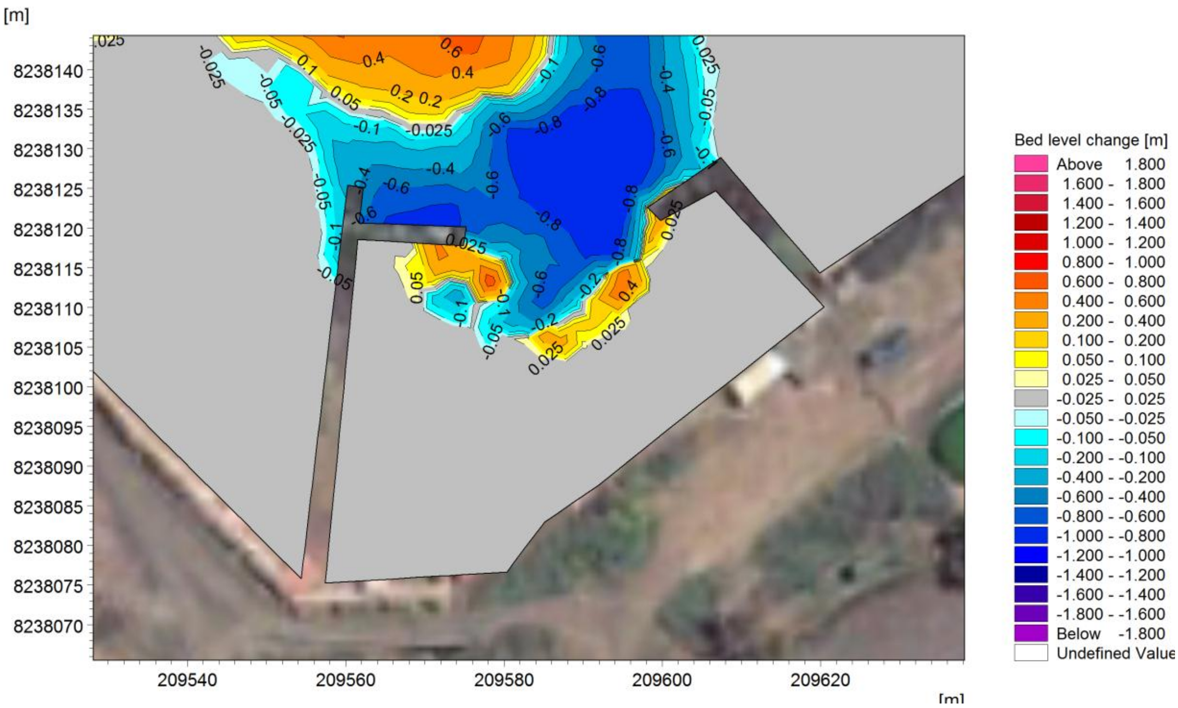


Figure 6-38: Including development – bed level change at swimming beach – 1-year return period event



7. SUMMARY AND RECOMMENDATIONS

The construction of a new airport facility on the island of St. Helena will require the existing port facilities on the island to be upgraded. These upgrades will include the provisions of permanent wharf facilities for handling bulk cargo.

This report has summarized the site conditions in Rupert's Bay, including a detailed analysis of the impact that the proposed development is likely to have on waves, currents and the sediment transport regime.

Available site data has been summarized, including tidal levels, waves, currents, wind, bathymetry, seabed characteristics and sediment properties. The results of this analysis were subsequently used for regional wave modelling, as well as two-dimensional sediment transport modelling.

Regional wave modelling has been performed using offshore hindcast wave data to determine the nearshore wave climate within Rupert's Bay. Calibration has been performed by comparing the simulated waves to measured waves. An Extreme Value Analysis of the modelled nearshore conditions was subsequently performed on the modelled conditions, to determine the design wave conditions for the marine infrastructure. A significant wave height of 4.6 m was determined for the design of the rubble mound breakwater, which has a probability of exceedance of 7% during the 70 year design life of the facility.

Coupled two-dimensional sediment transport modelling has been performed to investigate the direct impact of the proposed development on the wave, current and sediment transport characteristics.

Results of this analysis indicate that the development will result in significant wave sheltering in the southern region of Rupert's Bay. This sheltering results in a changed current pattern in Rupert's Bay, with the rip current that occurs during large wave events in the status quo not being present following the implementation of the proposed development.

The changed wave and current patterns resulting from the implementation of the proposed development result in changes to the sediment transport regime. These changes can be summarized as follows:

- Sedimentation of the facility's navigational area is predicted to occur during storm conditions only. Minimal accretion is expected during operational conditions, with approximately 0.1 m accretion occurring in the south-eastern corner of the berth pocket during the 100-year storm event. Nevertheless, it is recommended that small dredging equipment be included in the development, to facilitate intermittent dredging as and when required.

- 0.5 m to 1.5 m of sedimentation is expected to occur along the south-western edge of Rupert's Bay. Currently, this region is a rocky reef, which, if covered by sand, may change the marine ecology.

- The implementation of the development does not significantly change the waves, currents or sediment transport conditions at the south-eastern swimming beach. It has however been shown that the stability of the beach is critically linked to the presence of the concrete pipeline and offshore breakwater. Failure of maintaining these structures will result in the rapid erosion of the swimming beach, irrespective of the proposed development.

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APPENDIX C

VESSEL MOTIONS REPORT

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

Vessel Motions

REPORT NO. : 1097/02/03 REV B

APRIL 2013

PRESTEDGE RETIEF DRESNER WIJNBERG (PTY) LTD
CONSULTING PORT AND COASTAL ENGINEERS

Rupert's Bay Permanent Wharf – Phase 2: Vessel Motions Report No. 1097/02/03 Rev A					
Revision	**Date**	**Author**	**Checked**	**Status**	**Approved**
A	03 May 2013	RLH/DAS	SAL	For Comment	
B	17 May 2013	RLH/DAS	SAL	For Comment	

BASIL READ

ST HELENA ISLAND

RUPERT'S BAY PERMANENT WHARF - PHASE 2

VESSEL MOTIONS

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1. INTRODUCTION

The construction of a new airport on the island of St Helena will require the existing port facilities on the island to be upgraded. These upgrades will include the provision of permanent wharf facilities for handling bulk cargo, petroleum products, general cargoes and, in the medium to long-term, containers. The site selected for this facility is Rupert's Bay on the North West coast of the island.

This report quantifies the availability of the permanent wharf facilities in regards to typical vessel loading and unloading. Wave penetration modeling was performed in order to inform a high level downtime assessment based on guideline limiting wave height criterion.

Section 2 of this report documents the study approach. The wave penetration modeling and high level vessel downtime assessment are included in Sections 3 and 4, respectively. Conclusions follow in Section 5.

2. STUDY APPROACH

2.1 Wave Penetration Modelling

Wave conditions at the berth were determined by forcing the boundary of a Boussinesq model with discrete events linked to a yearly occurrence. Regional wave modelling as detailed in the Coastal Processes Report (PRDW, 2013a) provided the operational time series of wave parameters at the boundary of the Boussinesq model.

2.2 High Level Downtime Assessment

Preliminary investigations for the assessment of downtime have focused on the percentage occurrence of exceedance of limiting significant wave heights at the berth. This high level approach considers only the significant wave height at the berth and a general description of the direction of wave attack on the vessel. Limiting wave criteria are obtained from the handbook on port design (Thoresen, 2010). Significant wave heights are obtained from the wave penetration modelling study with an associated annual percentage occurrence.

3. WAVE PENETRATION MODELLING

3.1 Model Description

The MIKE 21 Boussinesq Waves (MIKE 21 BW) model (DHI, 2012b) was used for wave propagation and agitation in the bay. The model is based on the enhanced Boussinesq equations and has been further extended to incorporate wave breaking and a moving shoreline.

The enhanced Boussinesq equations incorporated into MIKE21 BW can reproduce the following coastal processes (DHI, 2012b):

- Shoaling
- Refraction
- Diffraction
- Wave breaking*
- Bottom friction*
- Moving shoreline*
- Partial reflection and transmission
- Non-linear wave-wave Interaction
- Frequency spreading
- Directional spreading

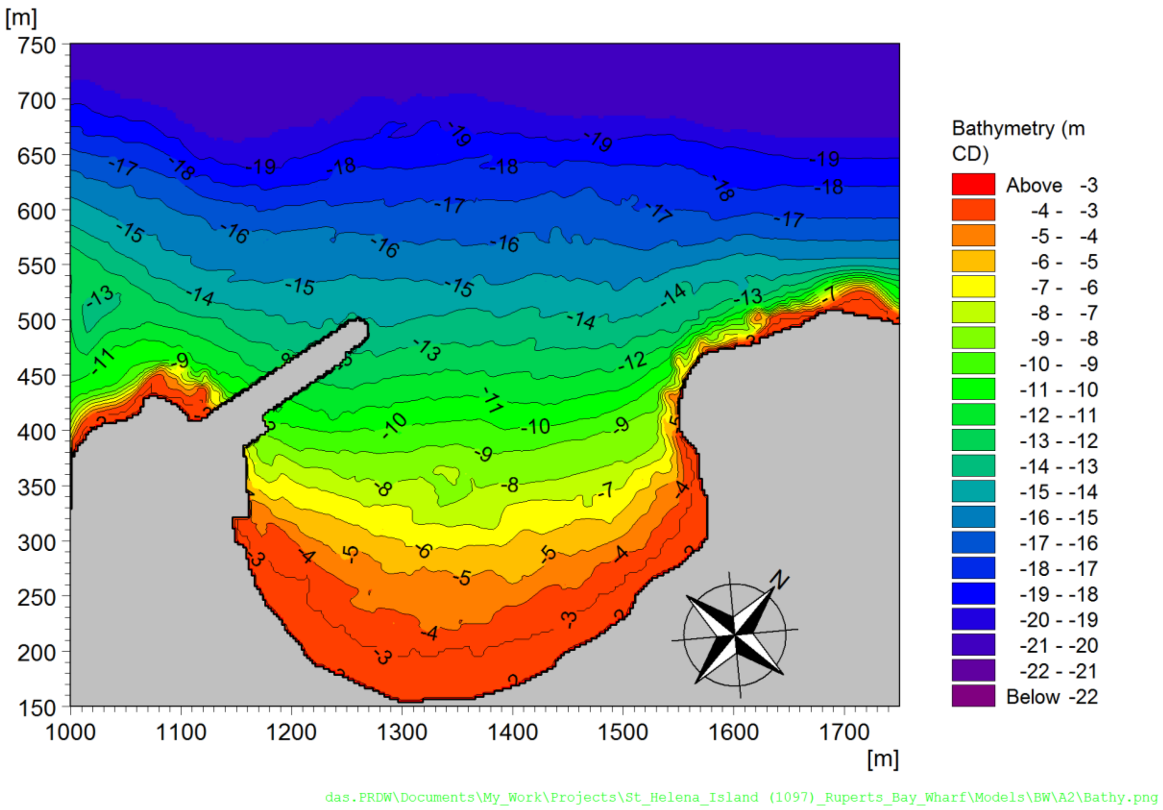
Processes marked with a * were switched off for the models used in the current investigation. The model solves the enhanced Boussinesq equations by an implicit finite difference technique with variables defined on a space-staggered rectangular grid.

3.2 Model Setup

3.2.1 Model Bathymetry

Three bathymetric surveys were used for creation of the model bathymetry shown in Figure 3-1. Within Rupert’s Bay two surveys overlapped, a single-beam bathymetric survey performed in 2006 (Tritan, 2006) and a multi-beam survey performed in 2012 (Tritan, 2012). A datum discrepancy between the two Tritan surveys was discovered. Closer inspection revealed a change in the local control point which necessitated a downward adjustment of 0.22 m to the 2006 survey. In order to consolidate the full datum discrepancy between the two surveys, the 2006 single beam survey required a further downward adjustment of 0.43m. For depths greater than -27 m CD and for the edges of Rupert’s Bay that were not covered by the extent of the two Tritan surveys MIKE C-MAP (DHI, 2012a) digital chart data was used.

Figure 3-1: Rupert's Bay model bathymetry



3.2.2 Numerical Parameters

A grid size of 2.5 m was used for the model. This resolution allowed for enough detail to define the relevant geometry of the land outline and the bathymetry variations. In order to ensure the model was numerically stable a Courant number of 0.28 was used by setting the model time step to 0.05 s.

The model was run for a real time duration equivalent to 2 hours and 15 minutes. The initial 15 minutes was not included in the results and was used as a warm up period. The last 2 hours of the model allowed adequate time for statistical convergence of both long and short wave energy.

3.2.3 Water Levels

A constant water level of +0.5 m CD equivalent to mean sea level was used for all the model runs.

3.2.4 Waves

Random wave input was created from a one dimensional JONSWAP spectrum. Second order corrections to the linear wave generation were enabled due to the importance of non-linear wave characteristics with regards to vessel motions.

The waves were propagated as unidirectional waves from the relevant wave directions. This assumes the waves are long crested and long travelled which is confirmed by the geographical position of the site and the relatively high peak wave period (PRDW, 2013a). The peak period and significant wave height were varied according to the specified event.

3.2.5 Sponge Layers

In order to ensure the entire model did not resonate due to long waves reflecting off the model boundaries, 500 m wide sponge layers, equivalent to 2.5 wavelengths of a 17 s wave, were inserted on the artificial boundaries to absorb any long wave energy that would in reality be reflected out to sea. Note, the full extent of the bathymetry and area taken up by the sponge layer has not been shown in Figure 3-1 above.

3.2.6 Reflection Coefficients

The initial reflection coefficients used in the BW model were set to be the same as used in the wave refraction modelling as detailed in the the Coastal Processes Report (PRDW, 2013a). Due to the orientation of the berth and the shape of Rupert's Bay, it became clear in the wave penetration modelling that reflected waves incident on the beam of the moored vessel were critical to the downtime analysis. The reflection coefficients for the Bay were therefore investigated in more detail to confirm that the coefficients used were realistic.

A review of literature was done to validate the chosen reflection coefficients in Rupert's Bay by applying the guidelines and methods of Thompson et al. (1996), Postma (1989) and Zannutigh and van der Meer (2008).

Thompson et al. (1996) provides a range of typical reflection coefficients compiled from a number of sources. The method of Postma (1989) relates the reflection coefficient to the slope and permeability of the structure and to the wave steepness. However, the low values of wave steepness at Rupert's Bay fall outside the range of applicability of the formula presented by Postma (1989). This method could therefore not be applied with confidence.

Zanuttigh and van der Meer (2008) proposed a formula to predict the reflection coefficient of various coastal structure types. The formula was derived by fitting a hyperbolic tangent curve through a dataset consisting of measured reflection coefficients from rock permeable, rock impermeable, armour unit and smooth slopes. The formula (Equation 5 in Zanuttigh and van der Meer (2008)) relates the reflection coefficient to the surf similarity parameter and two calibration parameters which are expressed as a function (Equation 6 in Zanuttigh and van der Meer (2008)) of the roughness factor used in overtopping research. The surf similarity parameter provides a ratio between the structure slope and the wave steepness.

The Rupert's Bay coastline was divided into sections of equal slope and coastline type, as indicated in Figure 3-2. Along each of the sections, the beach slope was calculated from limited bathymetric and topographical survey information. From these slopes, a maximum, minimum and average slope was obtained for every coastline section. Based on tables provided in Zanuttigh and van der Meer (2008) and engineering judgement, roughness factors were estimated for each of the coastline sections. The formula was then applied to all coastline sections, except sections six (rocky coast with a very steep slope) and nine (outer breakwater) where the value of the surf similarity parameter was outside the range of values in the dataset from which the equation was derived – primarily due to the low values of wave steepness considered here.

The results of the assessment are presented in Figure 3-3. The values from theory combine the guidelines presented in Thompson et al. (1996) and the results of the formula proposed by Zannuttigh and van der Meer (2008). The blue curve represents the values used in the BW model, indicating a conservative selection of reflection coefficients for Rupert's Bay when compared to values obtained from literature.

Figure 3-2: Coastline sections defined for reflection coefficient assessment

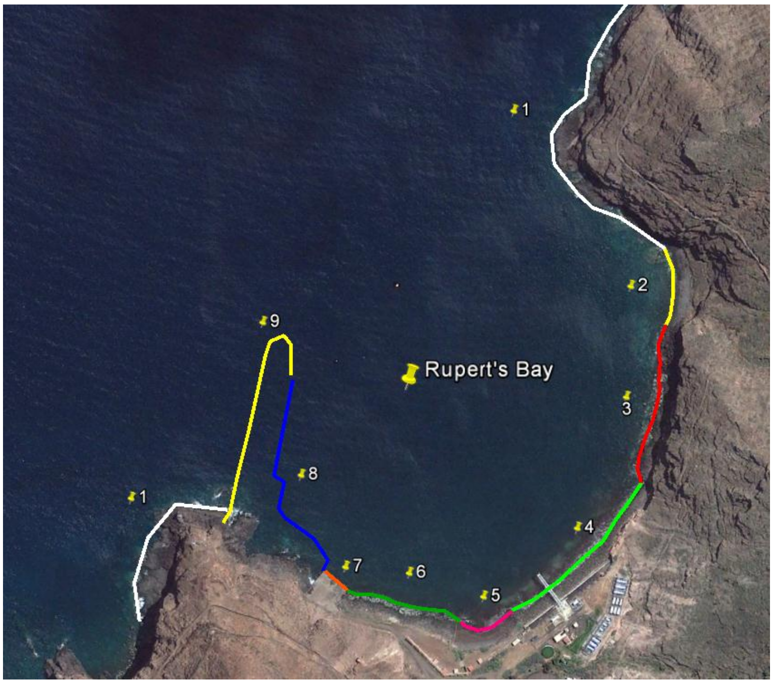
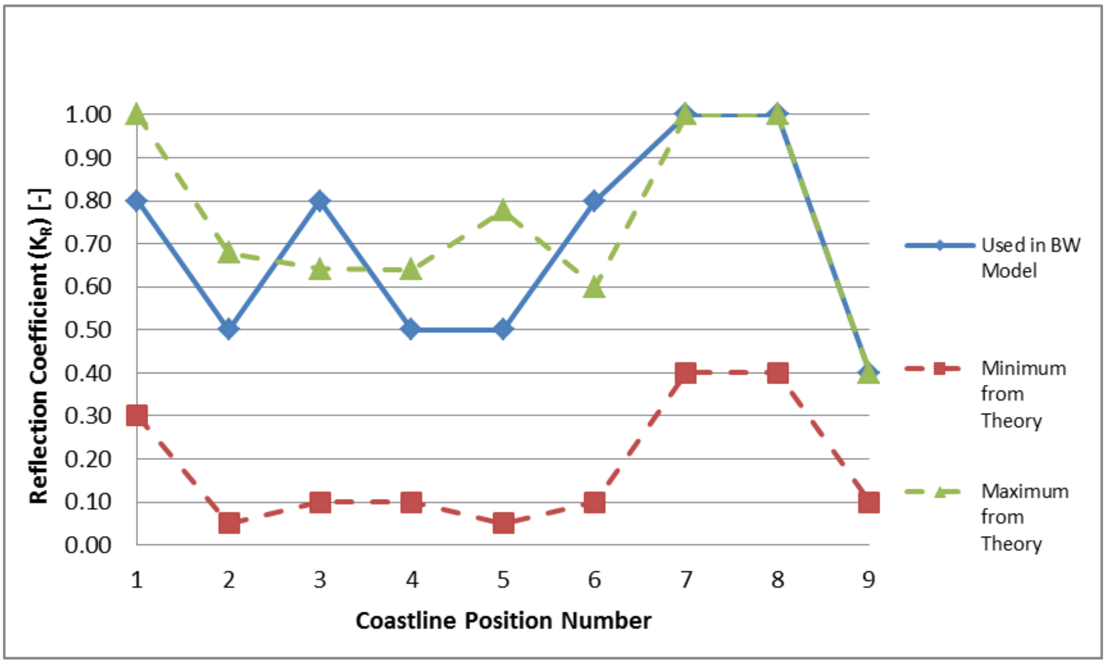


Figure 3-3: Calculated reflection coefficients vs. modelled coefficients



3.3 Discrete Event Selection and Occurrence

The wave climate at the boundary of the BW model was extracted from the spectral wave model as described in the Coastal Processes Report (PRDW, 2013a)

Table 3-1 shows the percentage occurrences of the operational wave climate discretized at the boundary of the Boussinesq model. The percentage boxes highlighted in blue represent the 15 discrete model cases that were run; these were defined as the bins with the highest percentage occurrence. The bin sizes for the discrete cases are +/-0.25 m, +/-10˚ and +/-2 s about the centre of the bin for the significant wave height (H_{m0}), peak wave direction (PWD) and peak period (T_p) bins respectively. In order to decrease the number of model runs, in certain cases wave occurrences were binned conservatively, for instance any incident waves propagating from less than 270˚ were added to the 270˚ bin.

As analysed in the Coastal Processes Report (PRDW, 2013a) there is evidence of highly variable wave groups at the site which can cause the H_{m0} to vary significantly from one hour to the next. The percentage occurrences presented below do not include any allowance for this short-term variability in H_{m0}. However, this is included in the downtime assessment described in Section 4.3.

Table 3-1: Discrete event selection and percentage occurrence

H_{m0} (m)	T_p (s)	PWD (˚)					Percentage Occurence in wave bin (%)
		270	290	310	330	350	
0.5	9	5.6	0.0	0.0	0.0	13.0	93.5
	13	18.8	0.0	5.3	47.7	0.0	
	17	0.2	0.0	0.0	2.9	0.0	
1	9	0.0	0.0	0.0	0.0	0.0	6.2
	13	0.3	0.0	1.0	3.6	0.0	
	17	0.0	0.0	0.4	0.9	0.0	
1.5	9	0.0	0.0	0.0	0.0	0.0	0.3
	13	0.0	0.0	0.3	0.0	0.0	
	17	0.0	0.0	0.0	0.0	0.0	
					Total percentage		100

3.4 Model Results

3.4.1 Discrete Events

Figure 3-4 to Figure 3-6 show disturbance coefficients for typical wave conditions. The series of figures show the effect that a change in PWD (270˚ to 310˚ to 330˚) on the boundary has on wave agitation in the bay and by keeping all other model parameters similar. The expected trend is evident showing the wave agitation coefficient adjacent to the main berth increasing from approximately 0.5 to 0.6 to 0.8 as the wave train has a more direct path into the bay and behind the breakwater.

Figure 3-6 and Figure 3-7 show the effect that an increase in wave period has on the wave agitation coefficient in the bay. Generally the bay experiences slightly higher agitation particularly on the eastern side due to increased refraction, characteristic of a higher period wave.

In general the figures show strong shoaling in the centre of the bay and nodal patterns created by reflected waves forming a standing wave pattern across the basin.

Figure 3-4: Disturbance coefficients for typical wave conditions, PWD = 270˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 18.8 %.

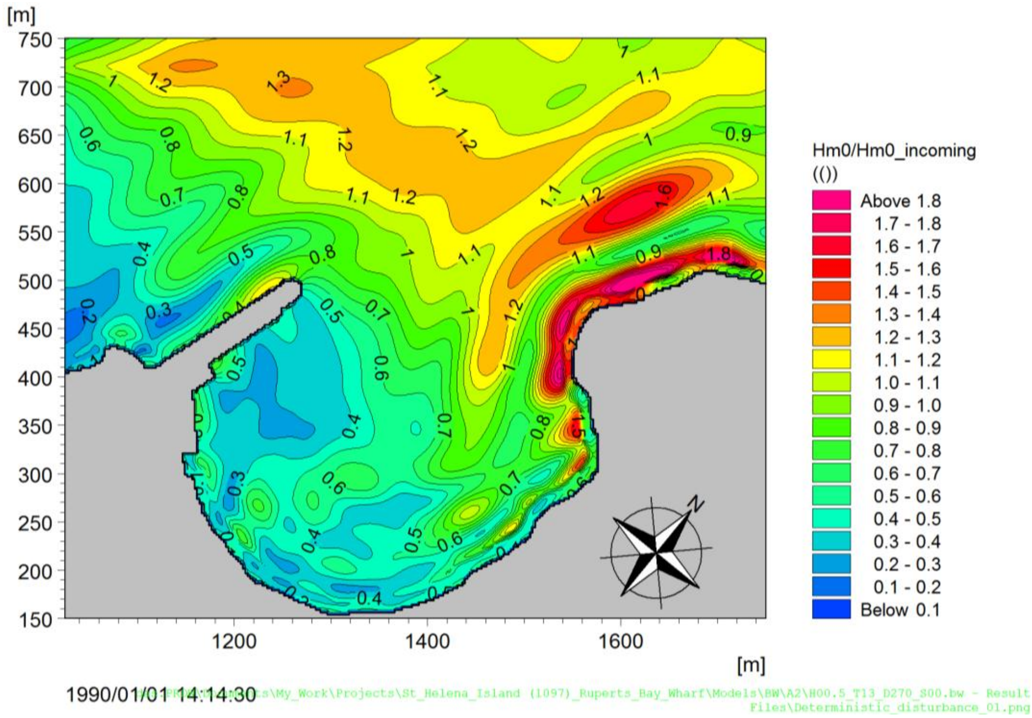


Figure 3-5: Disturbance coefficients for typical wave conditions, PWD = 310˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 5.3 %.

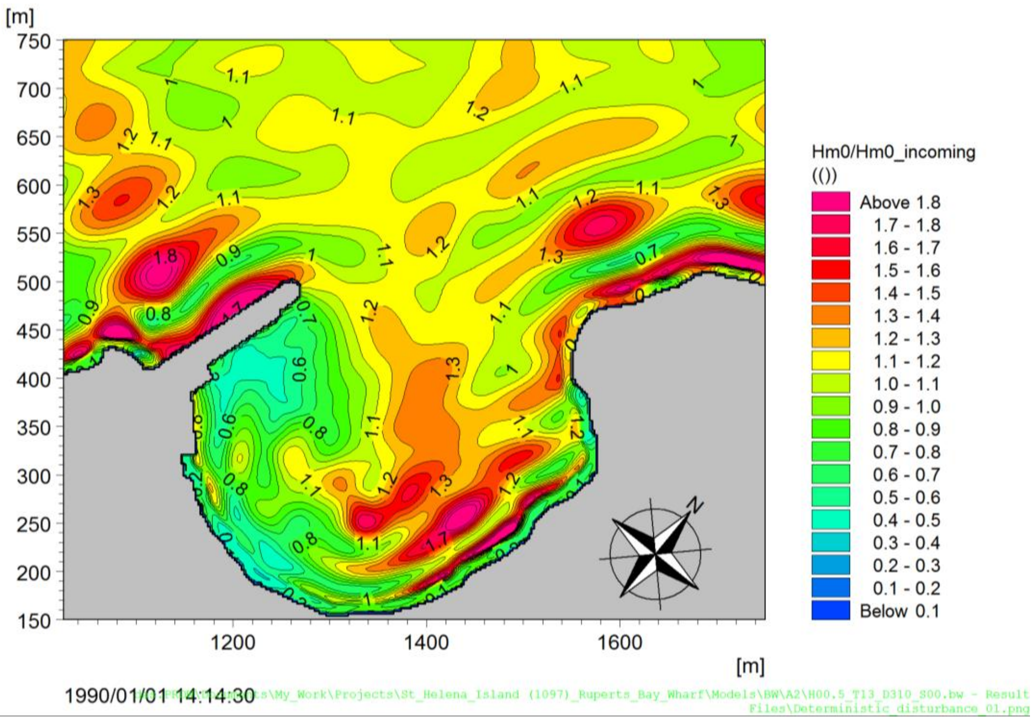


Figure 3-6: Disturbance coefficients for typical wave conditions, PWD = 330˚, H_{m0} = 0.5 m, T_p = 13 s. Yearly occurrence = 47.7 %.

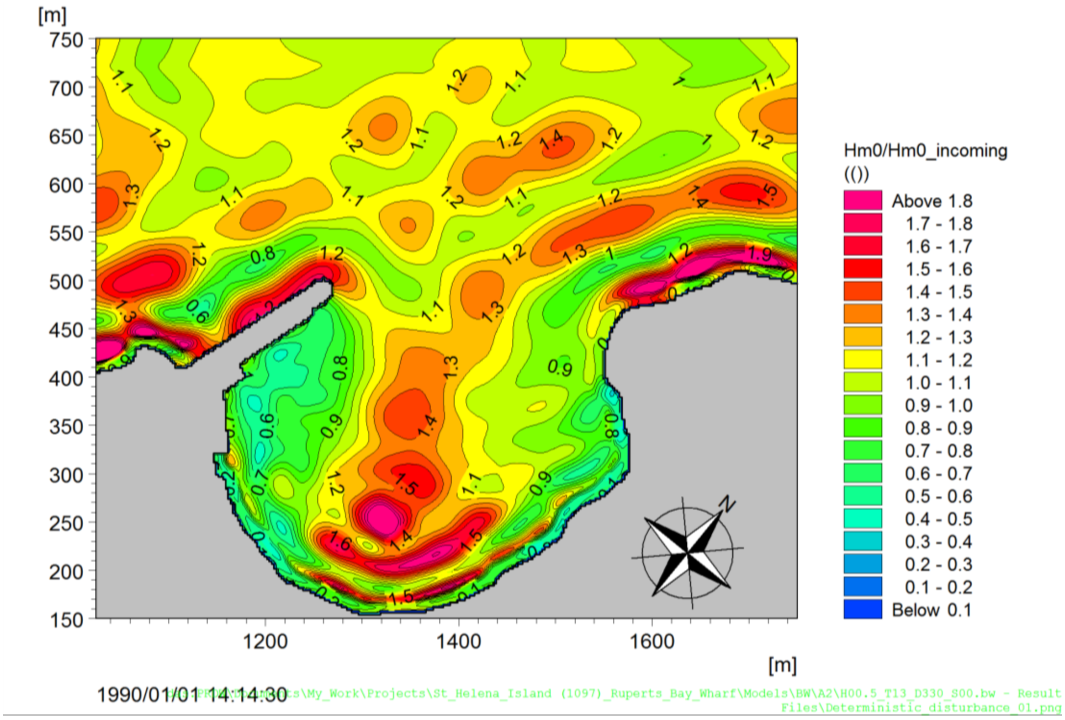
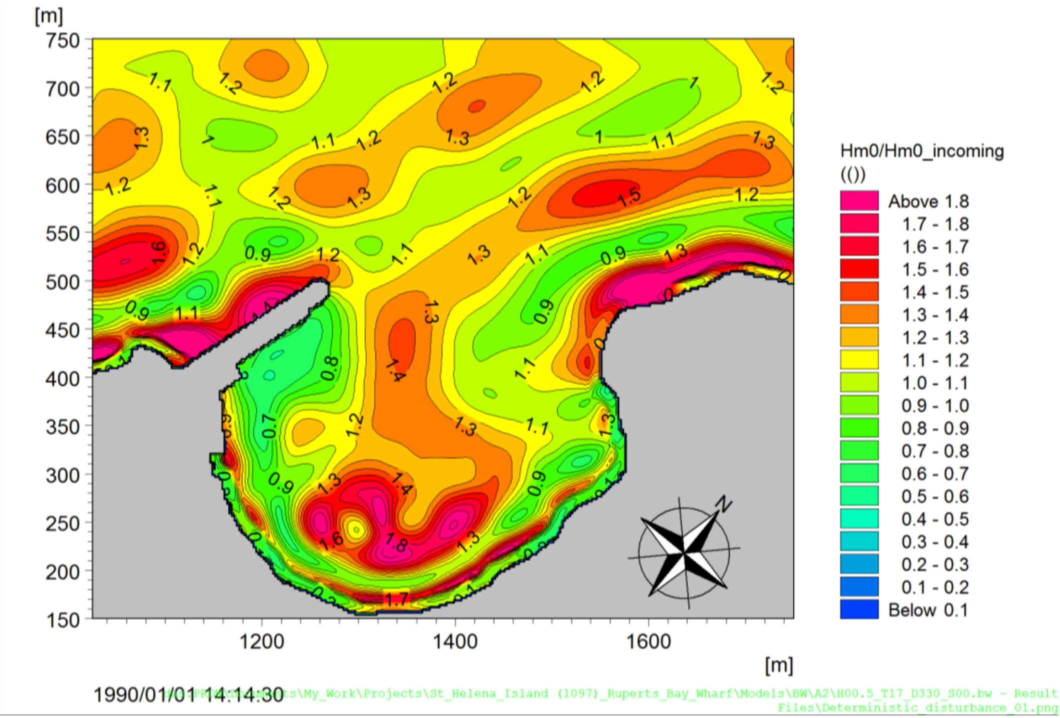


Figure 3-7: Disturbance coefficients for typical wave conditions, PWD = 330˚, H_{m0} = 0.5 m, T_p = 17 s. Yearly occurrence = 2.9 %.



3.4.2 Wave Reflection Investigation

In order to provide an accurate directional split of wave energy at the berth, an investigation into the typical energy present in the reflected and incident wave trains was performed. This was done by repeating the BW model simulation with an absorbing sponge layer along the shoreline of the bay instead of the porosity layer used to simulate the wave reflection. The reflected significant wave height was calculated using the equation below.

$$H_{m0\_total} = \sqrt{H^2_{m0\_incident} + H^2_{m0\_reflected}} \qquad \textbf{Equation 1}$$

Where:

H_{m0_total} = total significant wave height (m)

$H_{m0\_incident}$ = incident significant wave height (m)

$H_{m0\_reflected}$ = reflected significant wave height (m)

Figure 3-8 shows snap shots of the surface elevation for the incident wave only, the combined incident and reflected waves and the isolated reflected wave. Figure 3-9 shows the two model results and the calculated significant reflected wave height.

Figure 3-8: Examples of surface elevation snapshots. Upper: Incident waves (no reflection), Centre: Incident and reflected waves, Lower: Isolated reflected waves. (PWD = 330˚, H_{m0} = 0.5 m, T_p = 13 s).

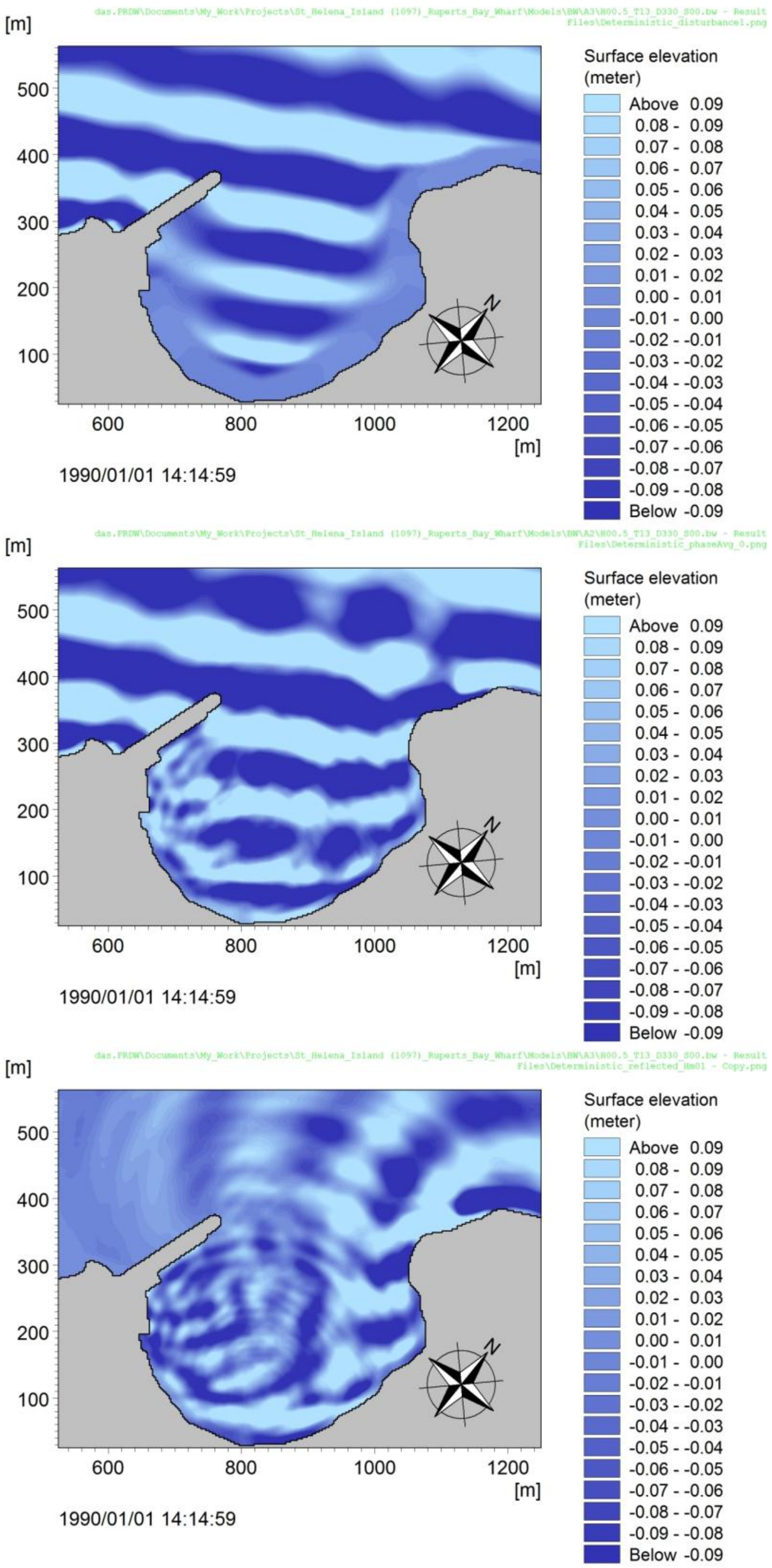
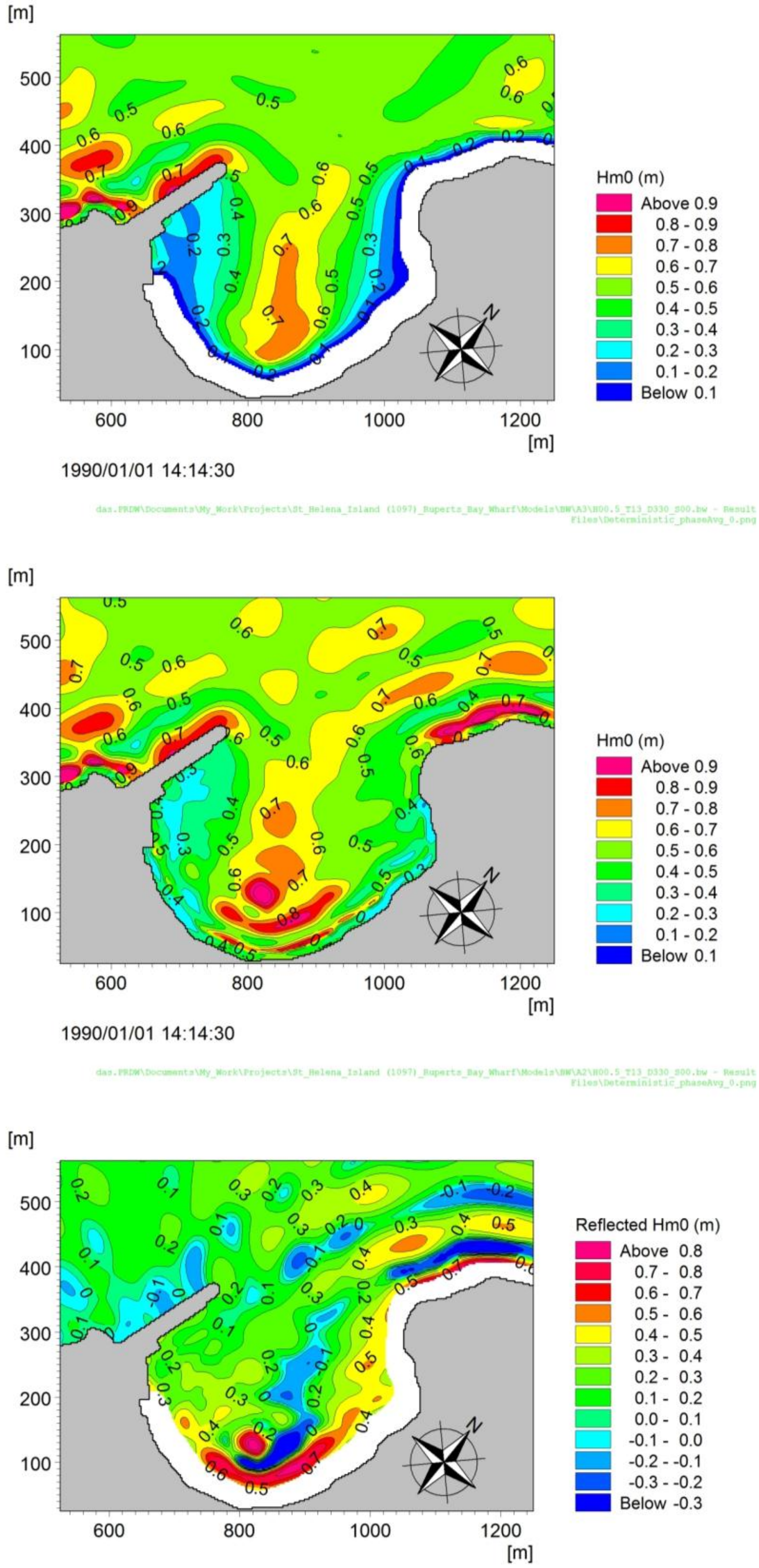


Figure 3-9: Example of significant wave heights showing reflection effect. Upper: Incident significant waves (no reflection), Centre: Incident and reflected significant waves, Lower: Isolated reflected significant waves. (PWD = 330˚, H_{m0} = 0.5 m, T_p = 13 s).



The lower panel of Figure 3-9 shows the reflected significant wave height with negative values near the shore line. This is due to the presence of a reflected standing wave and subsequent node formation in the reflected wave model. The area in the vicinity of the node has less energy than the incident wave model and hence when the energies of the two models are subtracted a negative value is created.

4. HIGH LEVEL DOWNTIME ASSESSMENT

4.1 Introduction

Preliminary investigations for the assessment of downtime have focused on the exceedance of limiting significant wave heights at the berth. This high level approach considers only the significant wave height at the berth and a general description of the direction of wave attack on the vessel. Limiting wave criteria are obtained from the handbook on port design (Thoresen, 2010).

4.2 Limiting Criteria

The limiting criteria are provided in Table 4-1 for a general cargo vessel and a Ro/Ro vessel. It can be seen from the table that the criteria for the Ro/Ro vessel is significantly more stringent than the liming criteria for the general cargo vessel.

Table 4-1: Limiting criteria on significant wave weights

Vessel Type	Limiting wave height H_{m0} in meters	
	0° (head-on or stern-on)	45 to 90°
General Cargo	1.0	0.8
Ro/Ro	0.5	0.3

The recommendation (Thoresen, 2010) is that these values can generally be accepted for wave periods up to 10 s, but for longer wave periods the H_{m0} must be reduced. Further, it is recommended (Thoresen, 2010) that a more realistic criterion would be an expression of the maximum tolerable movement of the ship itself relative to the berth that the mooring system and the cargo handling equipment can tolerate.

4.3 Calculation of Significant Wave Heights

From the BW model a time series of surface elevations for a number of locations along the berth have been processed in order to calculate the significant wave heights. The average significant wave height along the berth has then been calculated from four points for a range of representative offshore conditions, being defined by a significant wave height (H_{m0}), peak wave period (T_p) and peak wave direction (PWD).

Each of these conditions describe a bin (i.e. a single condition representing a discrete range of H_{m0}, T_p and PWD) at the boundary of the BW model in approximately -20 m CD water depth and the percentage occurrence of this bin based on one year of refracted wave conditions. From the BW model therefore, it is possible to obtain a specific condition of H_{m0} and T_p, at the berth with a specific percentage occurrence. This H_{m0} can then be tested against limiting criteria for significant wave heights and provide a value for percentage downtime/availability based on success or failure of the test.

The significant wave heights obtained from the BW model include both the incident and reflected components. From model tests (see Section 3.4.2) it can be seen that the reflected component of the total significant wave height has a very similar magnitude as the incident component. Based on this result, **Equation 1** was then used to split the total wave height predicted by the BW model at the berth into an incident and a reflected wave height as follows:

$$H_{m0\_incident} = 0.707 \times H_{m0\_total}$$

$$H_{m0\_reflected} = 0.707 \times H_{m0\_total}$$

Based on the modelled direction of the reflected wave relative to the berth, it was assumed that the reflected component of the total significant wave height is incident on the beam of the vessel. This assumption and the wave heights calculated and presented in Table 4-2, are then used to determine the percentage availability at the berth. The offshore binned conditions, transformed and separated conditions at the berth and the percentage occurrence for each of these conditions is shown in Table 4-2.

As analysed in the Coastal Processes Report (PRDW, 2013a) there is evidence of highly variable wave groups at the site which can cause the significant wave height to vary significantly from one hour to the next. The difference between measured hourly (based on 20 minute samples) and hindcast refracted 6 hourly average wave heights is shown in Figure 4-1.

Figure 4-1: Difference between short term (20 minutes) and 6 hourly average wave heights

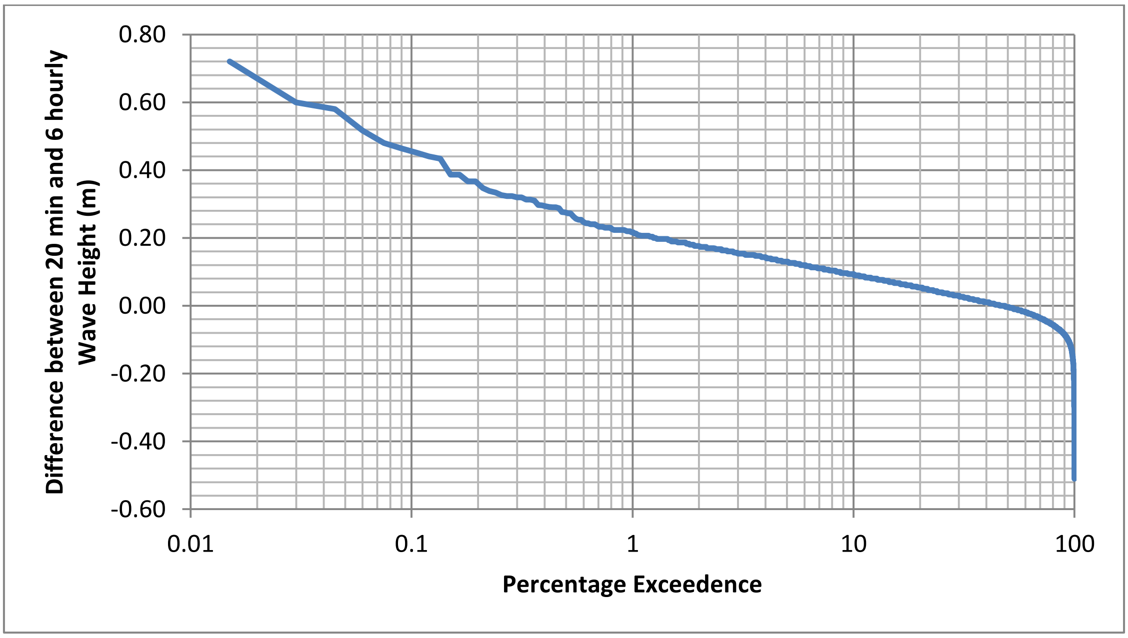


Table 4-2: Wave conditions (6 hourly averages) and percentage occurrence

Offshore conditions			Conditions at berth			Percentage occurrence [%]
H_{m0} [m]	T_p [s]	PWD [° TN]	H_{m0} Total (m)	H_{m0} Incident (m)	H_{m0} Reflected (m)	
0.5	9	350	0.34	0.24	0.24	13.0
0.5	9	270	0.17	0.12	0.12	5.6
0.5	13	330	0.33	0.24	0.24	47.7
0.5	13	310	0.29	0.21	0.21	5.3
0.5	13	270	0.24	0.17	0.17	18.8
0.5	17	330	0.34	0.24	0.24	2.9
0.5	17	310	0.32	0.23	0.23	0.0
0.5	17	270	0.20	0.14	0.14	0.2
1.0	13	330	0.71	0.50	0.50	3.6
1.0	13	310	0.67	0.47	0.47	1.0
1.0	13	270	0.55	0.39	0.39	0.3
1.0	17	330	0.76	0.54	0.54	0.9
1.0	17	310	0.70	0.49	0.49	0.4
1.5	13	310	1.17	0.83	0.83	0.3
1.5	17	330	1.19	0.84	0.84	0.0
1.5	17	310	1.07	0.75	0.75	0.0

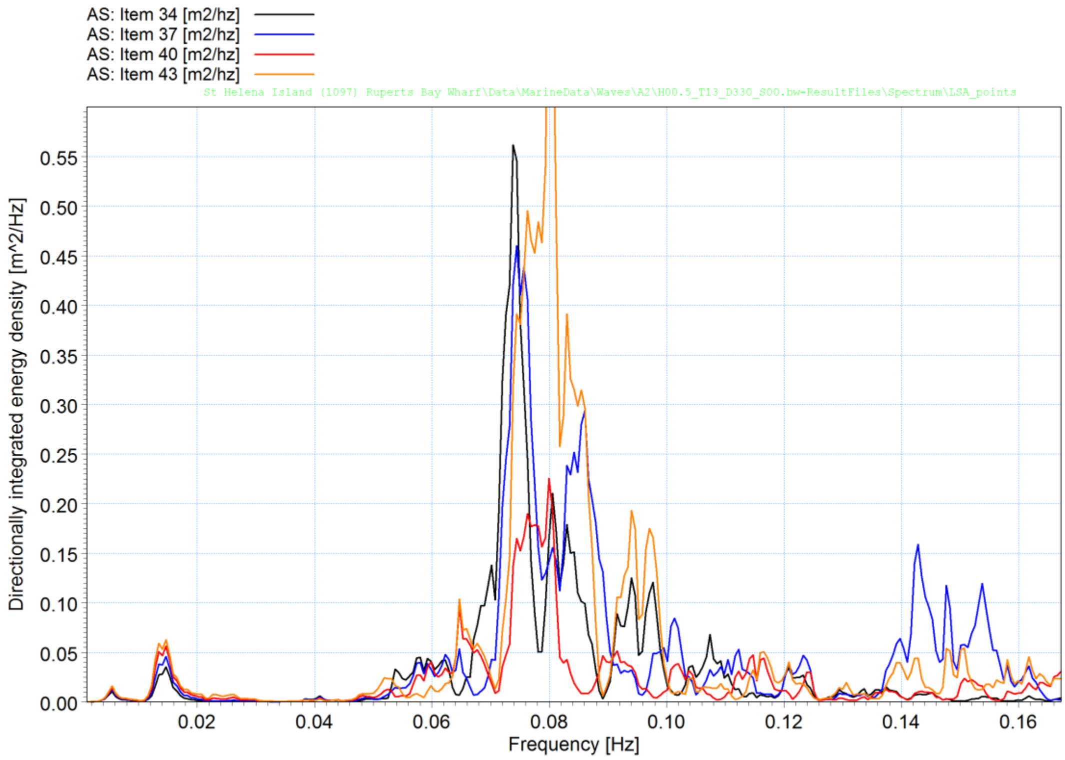
During any 6 hourly period the wave height may be up to 0.7 m more than the average value over the 6 hours. This increase would be associated with a 20 minute period. The expected increase for various percentage exceedences is shown in Table 4-3, indicating that an increase of more than 0.2 m in offshore wave height would only occur a few days per year. The increase in wave height at the berth was estimated based on a weighted average wave penetration coefficient of 0.61 (calculated from Table 4-2).

Table 4-3: Expected increase in wave height over 20 minutes compared to 6 hourly average value

Percentage Exceedance (%)	Increase in Offshore Wave Height (m)	Estimated Increase in Wave Height at the Berth (m)	Associated Number of Days per Year
20	0.05	0.03	73.0
10	0.09	0.05	36.5
5	0.13	0.08	18.3
1	0.22	0.13	3.7

A number of time series of surface elevations along the berth have been analysed in order to get an indication of the resonance frequencies of Rupert's Bay and the energy at the berth. The analysis includes a spectral calculation to determine the energy at different frequencies. Results for a single wave condition (H_{m0} = 0.5 m, T_p = 13 s, PWD = 330 deg) are shown in Figure 4-2. It can be seen that the highest energy is around the frequency of the input wave condition (~0.08 Hz or 13 s), with lesser energy at a frequency of 0.014 Hz (~70 s) and 0.145 Hz (~7s). The energy at ~70 s is surmised to correspond with a resonance period of Rupert's Bay, while the higher frequency (~ 6 s) is believed to correspond to non-linear triad interactions, where energy is transferred to higher frequencies, primarily due to shoaling.

Figure 4-2: Spectral analysis of surface elevations at various locations along the berth for H_{m0} = 0.5 m, T_p = 13 s, PWD = 330 deg



4.4 Downtime Based on Magnitudes of Significant Wave Heights

Percentage downtime has been calculated using the significant wave heights and related percentage occurrence provided in Table 4-2 and comparing these values to the limiting criteria for a general cargo vessel and a Ro/Ro vessel (refer to Table 4-1). The total availability is then calculated simply as the accumulation of the percentage occurrence where the reflected significant wave height is less than the limiting criteria for beam (45 to 90°) waves, as this is the critical condition. This is tabulated below for the general cargo vessel and the Ro/Ro vessel.

Table 4-4: High level downtime assessment for moored vessel motions

Vessel Type	Availability [%] Based on 6 Hourly Average Wave Heights
General Cargo	100%
Ro/Ro	94%

4.5 Discussion of Results

Berth availability for a general cargo vessel and a Ro/Ro vessel has been ascertained based on comparing the H_{m0} at the berth to a limiting H_{m0} criteria suggested by literature (Thoresen, 2010). In order to quantify the beam component of the H_{m0} the wave conditions at the berth have been decomposed into a head-on and beam-on H_{m0} based on the modelled wave reflection. It was found that the reflected H_{m0} has a very similar magnitude to the incident H_{m0}.

The wave data has been discretised into bins and assigned a percentage occurrence for each unique combination of H_{m0}, T_p and PWD. The head-on and beam-on components of the H_{m0} and the percentage occurrence have then been used to determine the percentage availability for the two design vessels.

The implication of short term variability needs to be considered in assessing downtime estimates. Wave heights at the berth can be expected to vary from the average 6 hourly heights used in assessing downtime. At a 1 per cent exceedance level (3.7 days per year) this increase would be 0.13 m or more for a period of approximately 20 minutes (within any given 6 hourly period). It is considered unlikely that this short term variability in wave height will result in a vessel having to leave the berth. Loading efficiency will likely be affected and there may even be some 20 minute windows during which loading has to be stopped. However, the overall effect of short term wave height variability on berth availability and throughput is not expected to be significant.

A noted caveat to the present methodology is that the limiting criteria only loosely take account of wave period. The recommendation provided (Thoresen, 2010) suggest that the limiting conditions should be reduced for wave periods over 10 s, but does not provide any guidance as to how much one can reduce these conditions. As the predominant wave period (~80% occurrence) is greater than ~12 s present estimates of downtime may be optimistic and it is recommended to carry out vessel motion studies to increase the level of confidence in this regard.

5. CONCLUSIONS

Wave penetration modelling has been undertaken in order to determine the wave transformation at the berth for a number of discrete offshore wave conditions associated with a specific percentage occurrence. These H_{m0} have been tested against limiting criteria for a general cargo vessel and a Ro/Ro vessel based on the assumption of equal wave energy from the bow and from the beam.

Availability of the berth has been calculated based on the above calculated wave heights. These preliminary calculations show a high availability for both the general cargo (100% availability) and the Ro/Ro (94% availability) vessels. Short term variability in wave heights may lead to short periods over which loading would be difficult and inefficient. While it is not envisaged that vessels will have to leave the berth due to this short term variability, the loading inefficiency could be interpreted as downtime which would reduce the estimated availability slightly.

Concerns have also been raised that the limiting criteria for wave heights may be too high based on the predominant wave period for the site, and thus the availability may be lower than given above.

In summary the following aspects may lead to increased downtime compared to the present estimates:

- Difficulty with applying a limit to joint occurrence of bow and beam waves
- Short term variability in wave height
- Longer wave periods than that covered by the available guidelines

It is recommended that vessel motion modelling studies be undertaken which can avoid the above limitations. While it may be expected that this will lead to increased downtime it is considered unlikely that availability of general cargo handling will drop below the desired 96 per cent (PRDW, 2013b).

Since downtime appears to be dominated by beam on conditions, it is critical that reflections off beaches and cliffs be modelled accurately. Existing beach profile surveys are considered insufficient and accurate surveys should be conducted for the final design.

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APPENDIX D

CSIR 2D PHYSICAL MODEL STABILITY STUDY

St Helena island - Rupert's Bay, 2D Physical Model Stability Test

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FINAL REPORT

CSIR

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CSIR REPORT **CSIR/BE/IE/ER/2013/0022/B**

St Helena Island

Rupert's Bay Permanent Wharf

2D Physical Model Study

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Published by:

CSIR
P O Box 395
Pretoria
0001
Republic of South Africa

Issued and printed by:

CSIR Built Environment
P O Box 320
7599
Stellenbosch
Republic of South Africa

Revision	Date	Compiled	Status	Approved
01	11/04/2013	K Tulsi	Draft report for internal comments	K Tulsi
02	17/04/2013	K Tulsi	Submitted to consultant for comments	K Tulsi / P O' Connor
03	22/04/2013	K Tulsi	Final Report	K Tulsi

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SUMMARY

The CSIR Coastal & Hydraulics Laboratory was contracted by Prestedge Retief Dresner Wijnberg (PTY) Ltd (referred to as PRDW) to conduct 2D physical modelling tests. The purpose of the tests was to assess Core-Loc™ armour stability and wave overtopping volumes for the breakwater required to protect the permanent cargo handling wharf for Rupert's Bay on St Helena island. The purpose of the wharf structure is to allow for both the landing of construction equipment and supplies during construction of the new airport, and for the provision of permanent facilities for handling bulk cargo, petroleum products, general cargoes and containers in the medium to long-term period. A 2D physical model of the breakwater structure was constructed to verify and refine theoretical and desktop studies.

The study comprised of two-dimensional (2D) tests in a wave flume carried out at a scale of 1:37. This report provides details of the 2D stability and overtopping assessment carried out on the proposed breakwater. The report also covers details of the model construction, wave calibration in the flume, assessments of the cross-sections after each test and wave overtopping measurements.

Numerical wave analysis was carried out by PRDW and provided to CSIR for wave calibration in the flume. The seabed constructed in the flume extended the equivalent of 300 m seaward. The profile of the constructed slope was 1:23. Wave calibration in the flume was at a water depth of -20m CD and -25m CD. Thereafter, the model structure was constructed for testing.

Three cross-sections were tested. The cross-sections varied in filter layer thickness, and toe rock dimensions. The cross-sections tested are described as follows:

Cross-section A is a typical Core-Loc™ breakwater with a crest height of +5.0m CD and berm level at -8.47m CD. The toe has a footing at -14m CD. The Core-Loc™ slope was at 1:1.5.
Cross-section B is similar to Section A with the filter layer thickness reduced from 1.4m to 1.1m. The toe rock was modified between -8.0m CD and -9.8m CD from (300Kg – 700Kg) to (1000Kg – 3000Kg).
Cross-section C is a new cross-section with seaward and rear Core-Loc™ armour. The seaward toe was at -7.1m with the rock grading of (1 tonne – 3 tonne). The leeward toe was at a depth of -4.3m CD with a grading of (300Kg – 700Kg). The seaward Core-Loc™ slope was 1:1.5 and rear slope 1:1.33

Mr K Tulsi compiled this report. Other members of the team included CSIR personnel. Mr F Guerrero from PRDW was present to give guided input during the model study.

SYMBOLS, ABBREVIATIONS AND ACRONYMS

BL	-	Berm Level
CD	-	Chart Datum
CL	-	Crest Level
Core-Loc™	-	Core-Loc is a trademark
EUL	-	Extreme upper limit
ELL	-	Extreme lower limit
H_{max}	-	Maximum wave height
H_{mo}	-	Wave height
Hs	-	Significant wave height
JONSWAP	-	Joint North Sea Wave Analysis Project
M_{em}	-	Median rock distribution
NLL	-	Nominal lower limit
NUL	-	Nominal upper limit
OT	-	Overtopping
OVLD	-	Overload
SWL	-	Still water level
T_p	-	Peak wave period
WL	-	Water Level

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1. INTRODUCTION

1.1. Background

PRDW contracted the CSIR Coastal & Hydraulics laboratory in February 2013 to conduct 2D physical model tests for the Rupert's Bay wharf breakwater. The purpose of this study is to evaluate the Core-Loc armour stability on the seaward and lee slope, toe stability and assess wave overtopping.

This document presents the results of the 2D study for three cross-sections. The supporting specifications for the model tests are documented in report ref: 0-819/0/01 Rev.00, provided to CSIR by PRDW.

One sea bottom profile was constructed and calibrated in the 2D glass flume. Three cross-sections were constructed and tested. Drawings of these cross-sections are provided in Appendix D.

Tests were conducted at the CSIR Coastal & Hydraulics Laboratory, Stellenbosch, South Africa. The model was constructed in a 2D glass flume (GF1). The model was tested at a scale of 1:37 according to the Froude law. Figure 1.1 presents an image of a typical model setup.

1.2. Objectives

The main objectives of the physical model tests were to:

- Evaluate the stability of the Core-Loc units and optimise the cross-section where possible
- Quantify the rate of overtopping above the crest of the structure

1.3. Layout of report

This report summarises the construction and setup of the physical model, the test results obtained during the study and a summary of the observations made during and after the tests.

Chapter 1 gives an introduction, while Chapter 2 provides a description of the facility and equipment used to set up the physical model, including a description of the wave generators and data capturing equipment. The details of the bathymetry construction and calibration of wave conditions are presented in Chapter 3. The 2D test descriptions and observations are presented in Chapter 4. The conclusion of the 2D model tests is given in Chapter 5. A list of references is given in Chapter 6, and Appendices in Chapter 7. The Appendices incorporate information on model scaling and images from the test. All dimensions given in this report refer to prototype dimensions, unless otherwise noted. Symbols and abbreviations used in this document are clarified in the Contents section.

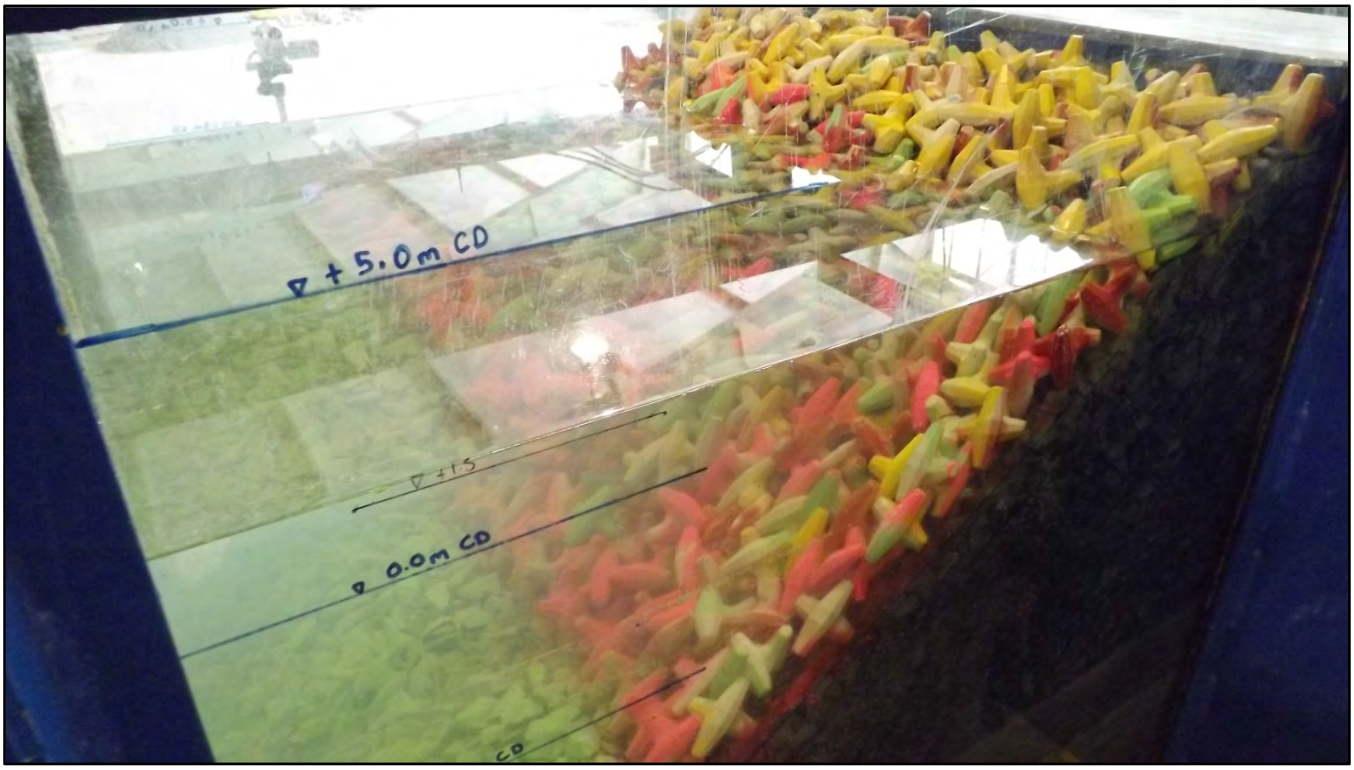

Figure 1.1: Image of a typical model setup

2. CSIR MODEL FACILITY AND EQUIPMENT

This chapter provides details of the 2D glass flume, wavemakers, wave probes, cameras, and overtopping equipment used to conduct the study.

2.1. 2D Glass Flume

The 2D glass flume (GF1) is 0.75 m wide x 30 m long x 1.0 m deep. Figure 2.1 shows an image of the 2D glass flume at the CSIR laboratory.

Figure 2.1: 2D Glass flume used for the tests (GF1)

2.2. 2D Wavemaker

The 2D flume has a single (0.75m wide) paddle with active wave absorption capabilities. Both irregular and regular waves can be generated. The maximum wave height generated in 60cm (model) water depth is approximately 40cm (model scale). Figure 2.2 provides an image of the wavemaker paddle in the glass flume.

The wavemaker program generates regular (sinusoidal) waves and random waves using two methods, those of digitally filtered white noise and summation of sine waves in both real time and using an offline playback method.

The random waves produced by the wavemaker conform to one of two standard spectral shapes, JONSWAP and Pierson-Moskowitz. A user-defined spectrum is also available where a series of spectral densities, separated by a constant frequency increment, can be defined. For this study, a standard JONSWAP spectrum of gamma 3.3 was used.

Figure 2.2: Image of wavemaker paddle mounted on the flume and control computer

2.3. Capacitance Wave Measurement Probes

The waves in the model were measured with capacitance probes coupled to an amplifier. As the water level varies around the probes, so does the voltage reading. By calibration, the voltage readings are coupled to the corresponding water level. The data is simultaneously captured in a binary voltage measurement format. By analysing the probe output, the voltage data is converted to a time-series of the variation in the wave surface elevation, from which the wave parameters are calculated.

The probes are calibrated in three steps covering the wave height expected at the probe. A calibration constant is then derived for the entire length of the probe, which is used to convert model measurements to prototype height measurements. The probes that were used are accurate to 0.5mm (model scale). These probes have a specific recording frequency (e.g. 20Hz).

To record wave conditions offshore, six single probes were used in the 2D flume. Three of the probes offshore of the proposed structure measured wave heights along the -25m CD location and the other three probes near the structure at the natural bathymetry at -20m CD. One other probe was placed on the crest and one behind the structure to measure overtopping. The wave data recorded were spectrally analysed using the GEDAP software. Relevant parameters such as significant wave height (H_s), maximum wave height (H_{max}) and peak period (T_p) were derived. The wave probe setup can be seen in Figure 2.3.

Figure 2.3: Capacitance wave measurement probe array in the 2D flume

2.4. Cameras

Four cameras were used to assist with the analysis process. The cameras were located in front of the structure at the sides and at the rear. Photographs were taken before and after each test and are presented in the powerpoint files for the project. A video of the test is also taken and presented in the powerpoint files. Figure 2.4 shows an image of the camera setup.

Figure 2.4: Camera setup in model

2.5. Wave Overtopping

A probe at the crest of the cross-section (Figure 2.4) recorded waves overtopping the structure. During testing this probe measured the wave height above crest level. The highest of these overtopping waves within the 1000 wave test cycle is reported on in Section 4.

Figure 2.5: Wave overtopping probe at crest of the structure

3. 2D MODEL SETUP

Details of the 2D model preparations and the test methodology are presented in this section.

3.1. Bathymetry

The 2D physical model foreshore profile was constructed at a scale of 1:37. The seabed was constructed using straight edge steel templates along the sides of the glass flume; thereafter it was filled with a light cement sand screed and finished off using a smooth steel float. For intricate detailed sections, templates were drawn on the glass and cement was shaped by hand. This method of construction assumes a uniform bottom roughness over a large area. The wavemaker is located on the boundary of the model at the offshore deep-water end of the flume.

One foreshore profile was constructed for all tests. The seabed profile is sketched in Appendix B. The foreshore seabed profile was built starting at the 16m model chainage (-25m CD prototype depth) with a slope of 1:23 representing the natural bathymetry up to the toe of the structure around -14m CD. Figure 3.1 shows the foreshore profile constructed in the flume.

Figure 3.1: Sea bottom construction prior to wave calibration

3.2. Wave Calibration

The wave conditions and water levels for the tests were specified by PRDW and are tabulated in Table 3.1. The wave specifications are tabulated in Table 3.2. The calibrated wave information during the calibration and testing process is presented in Table 3.3. The recorded wave series is separated into incident and reflected wave series using the Mansard and Funke method (1980).

Table 3.1: Wave Conditions and Parameters

Water level [mCD]	Hmo[m]	Tp[s]	Tp[s]	Tp[s]	Tp[s]
0	3	-	12	16	18
0	4	10	12	16	-
0	4.6	10	12	16	18
0	5		12	16	18
1.5	3	-	12	16	18
1.5	4	10	12	16	-
1.5	4.6	10	12	16	18
1.5	5.0	-	12	16	18

Table 3.2: Wave Condition Achieved During Calibration

Cal No.	Measurement File Name	SWL (mCD)	Prototype Wavemaker Depth (m)	Test Duration (model) (s)	Target Hs (m) @-25mCD	Target T_p (s) @-25mCD	Target Hs (m) @-20mCD	Target T_p (s) @-20mCD	Wave Gain (%)	Calibrated Achieved Hs (m) @-25mCD	Calibrated Achieved T_p (s) @-25mCD	% variation @ -25m	Calibrated Achieved Hs (m) @-20mCD	Calibrated Achieved T_p (s) @-20mCD	% variation @ -20m	Wave Sequence (No.)	Wave cycle time (s)	Measurement Duration (s)	Depth at probe @-25m	Depth at probe @-19 (m)
1	CL01c	1.5	26.5	2187	3.0	13	3.0	13	0.95	3.11	12.93	3%	3.08	12.93	3%	10	273	300	26.5	20.5
2	CL02a	1.5	26.5	2680	3.0	16	3.0	16	0.90	3.13	15.70	4%	3.06	15.70	2%	10	450	500	26.5	20.5
3	Cl03a	1.5	26.5	3174	3.0	19	3.0	19	0.90	3.05	18.31	2%	2.96	18.31	-1%	10	450	500	26.5	20.5
4	CL04a	1.5	26.5	2187	4.0	13	4.0	13	0.95	4.22	12.93	5%	4.17	12.93	4%	10	450	500	26.5	20.5
5	CL05b	1.5	26.5	2680	4.0	16	4.0	16	0.90	4.19	15.70	5%	4.13	15.70	3%	10	450	500	26.5	20.5
6	Cl06a	1.5	26.5	3174	4.0	19	4.0	19	0.90	4.18	18.31	4%	3.98	18.31	-1%	10	450	500	26.5	20.5
7	Cl07	1.5	26.5	2187	4.6	13	4.6	13	0.90	4.77	12.93	4%	4.72	12.93	3%	10	450	500	26.5	20.5
8	Cl08	1.5	26.5	2680	4.6	16	4.6	16	0.90	4.67	15.70	2%	4.63	15.70	1%	10	450	500	26.5	20.5
9	Cl09a	1.5	26.5	3174	4.6	19	4.6	19	0.90	4.84	18.31	5%	4.57	18.31	-1%	10	450	500	26.5	20.5
10	Cl10a	1.5	26.5	2187	5.0	13	5.0	13	0.90	5.19	12.93	4%	5.07	12.93	1%	10	450	500	26.5	20.5
11	Cl11a	1.5	26.5	2680	5.0	16	5.0	16	0.90	5.12	15.70	2%	5.04	15.70	1%	10	450	500	26.5	20.5
12	Cl12a	1.5	26.5	3174	5.0	19	5.0	19	0.90	5.21	18.31	4%	5.00	18.31	0%	10	450	500	26.5	20.5
13	Cl13	0	25	2187	3.0	13	3.0	13	0.90	3.05	13.06	2%	3.00	13.06	0%	10	450	500	25.0	19.0
14	CL14a	0	25	2680	3.0	16	3.0	16	0.90	3.04	16.08	1%	3.04	16.08	1%	10	450	500	25.0	19.0
15	Cl15	0	25	3174	3.0	19	3.0	19	0.90	3.13	19.00	4%	3.06	19.00	2%	10	450	500	25.0	19.0
16	Cl16	0	25	2187	4.0	13	4.0	13	0.90	4.20	13.06	5%	4.10	13.06	3%	10	450	500	25.0	19.0
17	Cl17	0	25	2680	4.0	16	4.0	16	0.90	4.15	16.08	4%	4.10	16.08	2%	10	450	500	25.0	19.0
18	Cl18	0	25	3174	4.0	19	4.0	19	0.90	4.10	19.00	3%	4.10	19.00	2%	10	450	500	25.0	19.0
19	Cl19	0	25	2187	4.6	13	4.6	13	0.90	4.75	13.06	3%	4.75	13.06	3%	10	450	500	25.0	19.0
20	Cl20	0	25	2680	4.6	16	4.6	16	0.90	4.80	16.08	4%	4.74	16.08	3%	10	450	500	25.0	19.0
21	Cl21	0	25	3174	4.6	19	4.6	19	0.90	4.71	19.00	2%	4.70	19.00	2%	10	450	500	25.0	19.0
22	Cl22	0	25	2187	5.0	13	5.0	13	0.90	5.25	13.00	5%	5.17	13.06	3%	10	450	500	25.0	19.0
23	Cl23	0	25	2680	5.0	16	5.0	16	0.90	5.16	16.08	3%	5.07	16.08	1%	10	450	500	25.0	19.0
24	Cl24	0	25	3174	5.0	19	5.0	19	0.90	5.16	19.00	3%	5.07	19.00	1%	10	450	500	25.0	19.0

Table 3.3: Wave Condition Achieved During Testing

Record Number	Test No.	Wavemaker File Name	SWL (mCD)	Target Hs (m) @-25mCD	Target T_p (s) @-25mCD	Target Hs (m) @-20mCD	Target Tp (s) @-20mCD	Wave Gain (%)	Calibrated Achieved Hs (m) @-25mCD	Calibrated Achieved T_p (s) @-25mCD	% variation @ -25m	Test Hs:-25 Acheived (m)	Test Tp:-25 Acheived (s)	% variation @ -10m	Calibrated Achieved Hs (m) @-20mCD	Calibrated Achieved Tp (s) @-20mCD	% variation @ -20m	Test Hs:-20m Acheived (m)	Test Tp:-20m Acheived (s)	% variation @ -20m
1	A1	CL02a	1.5	3.0	16	3.0	16	0.90	3.13	15.70	4%	3.39	15.90	13%	3.06	15.70	2%	3.28	15.90	9%
2	A2	CL05b	1.5	4.0	16	4.0	16	0.90	4.19	15.70	5%	4.55	15.90	14%	4.13	15.70	3%	4.33	15.90	8%
3	A3	Cl08	1.5	4.6	16	4.6	16	0.90	4.67	15.70	2%	5.35	15.90	16%	4.63	15.70	1%	5.05	15.90	10%
	SD	CL02a	1.5	2.0	16	2.0	16	0.55	N/A	N/A	N/A	2.28	15.75	14%	N/A	N/A	N/A	2.16	14.62	8%
4	B1	CL02a	1.5	3.0	16	3.0	16	0.90	3.13	15.70	4%	3.31	15.70	10%	3.06	15.70	2%	3.18	15.70	6%
5	B2	CL05b	1.5	4.0	16	4.0	16	0.90	4.19	15.70	5%	4.56	15.90	6%	4.13	15.70	-4%	4.32	15.90	0%
6	B3	CL04	1.5	4.0	13	4.0	13	0.95	4.22	12.93	5%	4.71	12.92	10%	4.17	15.70	-3%	4.36	12.92	1%
7	B4	CL06	1.5	4.0	19	4.0	19	0.80	4.18	18.31	4%	4.31	18.79	0%	3.98	18.79	-7%	4.23	18.79	-2%
8	B5	CL08	1.9	4.6	16	4.6	16	0.90	N/A	N/A	N/A	4.70	15.90	2%	N/A	N/A	N/A	4.73	16.00	3%
9	B6	Cl19	0	4.6	13	4.6	13	0.90	4.75	13.06	3%	4.84	12.92	5%	4.75	13.06	3%	4.72	12.92	3%
10	B7	Cl20	0	4.6	16	4.6	16	0.90	4.80	16.08	4%	5.08	15.90	10%	4.74	16.08	3%	4.72	15.90	3%
11	B8	Cl21	0	4.6	19	4.6	19	0.90	4.71	19.00	2%	4.79	18.78	4%	4.70	19.00	2%	4.82	18.78	5%
12	B9	CL20	0.5	5.5	16	5.5	16	0.90	-	-	-	-	-	-	-	-	-	-	-	-
	SD	CL02a	1.9	2.0	16	2.0	16	0.40	N/A	N/A	N/A	2.28	15.75	14%	N/a	N/a	N/a	2.16	14.62	8%
13	C1	CL08	1.9	4.6	16	4.6	16	0.85	N/A	N/A	N/A	4.70	15.78	2%	N/A	N/A	N/A	4.69	15.90	2%
14	C2	Cl19	1.9	4.6	19	4.6	19	0.90	N/A	N/A	N/A	4.44	12.92	-4%	N/A	N/A	N/A	4.72	12.92	3%
15	C3	Cl20	0	4.6	16	4.6	16	0.80	4.80	16.08	4%	4.61	15.90	0%	4.74	16.08	3%	4.53	15.90	-2%
16	C4	Cl21	0	4.6	19	4.6	19	0.80	4.71	19.00	2%	4.67	18.78	2%	4.70	19.00	2%	4.57	18.78	-1%

3.3. Test Procedure

At the beginning of each day, all probes to be used for the day's test were calibrated. Proposed changes/ modifications to the cross-section were discussed and implemented where appropriate. The test section was discussed for any changes or modification and the water level was set for the appropriate test. Images were taken before the start of each test. The wavemaker was programmed to generate a test condition and the test was run.

During the test, wave measurements were taken and whenever possible information of wave and structure interaction was documented. If there was a noticeable change in the profile of the cross-section a photo was taken and the event was documented. After every test another series of pictures were taken.

After each test, the videos and photos were compiled along with the data and test observations made during the test. This is reported in a presentation format and sent to the client for review within 24 hours after the test.

At the end of the test series the structure was analysed and if the results were unsatisfactory, an alternate design was presented by the client until a satisfactory solution was obtained. A total of three cross-sections were constructed and assessed. A total of sixteen tests were conducted.

4. TEST RESULTS

4.1. Details of the test sections

Section A

The breakwater construction started at a seabed level of -14m CD. The sketch of the cross-section is shown in Appendix D. The Core-Loc armour units used was 5000 kg. The toe rock used was (300 – 700 Kg). The grading information is presented in Appendix C. Images of the constructed Section A is shown in Figure 4.1. The crest level was +5.0m CD, the toe berm at -8.47m CD. The toe slope extended to -14m CD. Three tests were carried out for Section A.

Figure 4.1: Section A Constructed in the 2D Flume

Section B

Cross-section B was similar to Section A with the filter layer thickness reduced from 1.4m to 1.1m (prototype). The toe rock grading between -8.0m CD and -9.8m CD was increased in mass from (300Kg – 700Kg) to (1000Kg – 3000Kg). Figure 4.2 shows an image of the modified slope.

Figure 4.2: Section B Constructed with a modification of the filter layer and toe

Section C

Cross-section C is a new cross-section with Core-Loc armour units on the seaward and rear slopes. The seaward toe berm was at -7.1m CD with a rock grading of (1 tonne – 3 tonne). The leeward toe berm was at a depth of -4.3m CD extending to -12.5m CD with a grading of (300Kg – 700Kg). The seaward Core-Loc slope was 1:1.5 and rear slope 1:1.33. Figure 4.3 shows an image of cross-section C constructed in the flume.

Figure 4.3: Section C Constructed to test the rear slope stability

The test summary for all tests is tabulated below (Table 4.1). All measurements were taken over 1000 waves. The following section provides a summary of the three cross-sections.

Table 4.1: Test Summary

Record Number	Test No.	SWL (mCD)	Test Hs:- 20m Acheived (m)	Test Tp:- 20m Acheived (s)	% variation @ -20m	Measurement Duration (s)	No. of wave overtopping	Overtopping height in bin 1 (mm)	Overtopping (l/s/m)	Minor Movement <0.5C (Sea)	Moderate Movement 1.0C>0.5C (Sea)	Extreme Movement >1.0C (Sea)	D % (Sea)	Cum D % (Sea)	Minor Movement <0.5C (Rear)	Moderate Movement 1.0C>0.5C (Rear)	Extreme Movement >1.0C (Rear)	D % (Lee)	Cum D % (Lee)
1	A1	1.5	3.28	15.90	9%	2680	105	898	25.30	40	0	0	0.0%	0.0%	-	-	-	0.0%	0.0%
2	A2	1.5	4.33	15.90	8%	2680	180	7200	202.87	60	0	0	0.0%	0.0%	-	-	-	0.0%	0.0%
3	A3	1.5	5.05	15.90	10%	2680	226.0	21200	597.34	100	5	1	1.2%	1.2%	-	-	-	0.0%	0.0%
	SD	1.5	2.16	14.62	8%	1365	N/A	200	22.1	-	-	-	-	-	-	-	-	-	-
4	B1	1.5	3.18	15.70	6%	2680	155	3800	107.1	125	4	0	0.7%	0.7%	-	-	-	0.0%	0.0%
5	B2	1.5	4.32	15.90	8%	2680	180	9800	276.1	75	10	1	2.0%	2.7%	-	-	-	0.0%	0.0%
6	B3	1.5	4.36	12.92	9%	2187	169	6400	221.0	50	0	0	0.0%	2.7%	-	-	-	0.0%	0.0%
7	B4	1.5	4.23	18.79	6%	3174	283	13600	323.7	25	0	0	0.0%	2.7%	-	-	-	0.0%	0.0%
8	B5	1.9	4.73	16.00	3%	2680	289	16200	456.5	12	1	0	0.2%	2.9%	-	-	-	0.0%	0.0%
9	B6	0	4.72	12.92	3%	2187	155	4200	145.0	3	0	0	0.0%	2.9%	-	-	-	0.0%	0.0%
10	B7	0	4.72	15.90	3%	2680	179.0	12400	349.4	7	0	0	0.0%	2.9%	-	-	-	0.0%	0.0%
11	B8	0	4.82	18.78	5%	3174	210.0	10800	257.0	12	0	0	0.0%	2.9%	-	-	-	0.0%	0.0%
12	B9	0.5	5.5	16	-	2680	-	-	-	10	2	3	1.4%	4.3%	-	-	-	0.0%	0.0%
	SD	1.9	2.16	14.62	8%	1365	N/A	N/A	N/A	-	-	-	-	-	-	-	-	-	-
13	C1	1.9	4.69	15.90	2%	2680	-	N/A	N/A	75	1	0	0.2%	0.2%	6	1	0	0.4%	0.4%
14	C2	1.9	4.72	12.92	3%	3174	-	N/A	N/A	75	3	0	0.5%	0.7%	5	0	0	0.0%	0.4%
15	C3	0	4.53	15.90	-2%	2680	155.0	N/A	N/A	1	2	0	0.3%	1.0%	60	3	0	1.2%	1.6%
16	C4	0	4.57	18.78	-1%	3174	123.0	N/A	N/A	1	0	0	0.0%	1.0%	0	0	60	48.0%	49.6%

4.2. Section A

Over the three tests conducted with cross-section A, waves approached the structure steeply to plunge and surge over the crest. Wave overtopping was in the form of frequent green water surges. The crest level was +5.0m CD, therefore significant overtopping was expected for the wave periods of 16s and water level of +1.5m CD. By the end of test three, one Core-Loc unit from +3.0m was displaced to the crest wall. The cumulative damage percentage was 1.2% by the end of the third test. Wave overtopping measurement during Test 1 was 25.3 l/s/m for the 3.0m wave and increased to 597.34 l/s/m during Test 3. Figure 4.4 and Figure 4.5 present images during test section A.

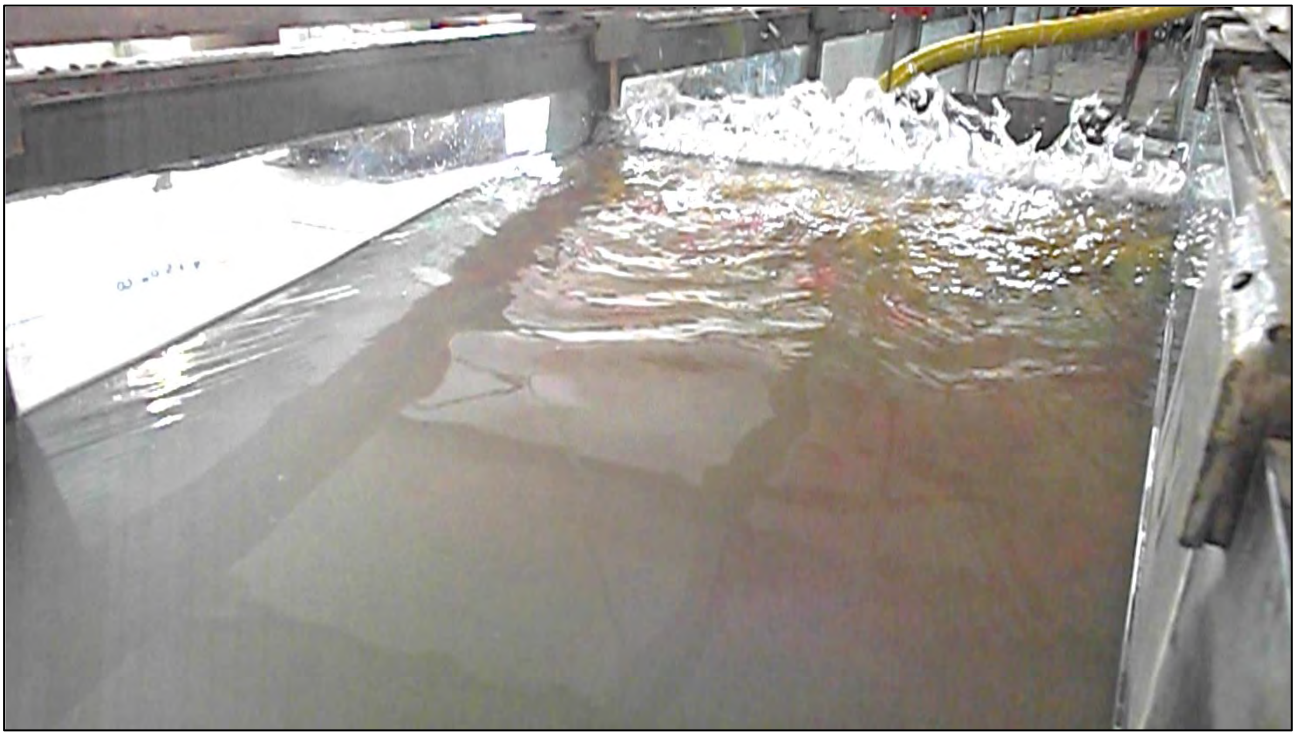

Figure 4.4: Section A: Image of wave overtopping the crest

Figure 4.5: Section A: Image of wave slam on the crest wall

The draw-down of the waves during the three tests reached -5.0m CD. This resulted in disturbances to the toe slope with rock protection of (0.3 tonne - 0.7 tonne). A layer of toe rock between -8.47m CD to -11m CD was displaced during the three tests. This is shown in Figure 4.6.

Figure 4.6: Section A: Image of toe erosion

Based on the erosion noticed on the toe, the cross-section was modified to stabilise the toe berm by increasing the mass of rock from (0.3 tonne – 0.7 tonne) to (1.0 tonne – 3.0 tonne) and have the back row of rocks on the berm resting in-between the voids of the Core-Loc toe. This brought the toe berm to -8.0m CD. Figure 4.7 shows the image of the modified toe berm.

Figure 4.7: Section A to B: Image of larger rocks at berm and Core-Loc toe

4.3. Section B

Section B began with the +1.5m CD water level and then raised to 1.9m and lowered to 0.0m CD to assess the overtopping and stability. Wave overtopping frequency at the structure was similar to section A. Test 1 to Test 3 showed extended settlement of Core-Loc units on the slope. This was visible between Core-Loc rows 1 through to row 5 from the crest. During these tests rearrangement of the (0.3 tonne - 0.7 tonne) rock was also noticed around -10m CD. Test 4 had minor movements of the Core-Locs and the (0.3 tonne – 0.7 tonne) rocks. Test 5 was similar to test 2 with no movements at the toe although the water level was increased to +1.9m CD. Test 6 was the start of the low water level test at 0.0m CD. The (0.3 tonne – 0.7 tonne) rock at the toe had less than 10 rocks displaced. Test 7 had further movement of 10 - 15 toe rocks (0.3 tonne – 0.7 tonne). These rocks moved from around -10m CD to -14m CD. Test 8 had minor movement of less than 5 toe rocks. An additional test was run after Test 8 with wave condition of 5.5m 16s as an overload case (Test 9). Test 9 was run with a water level of +0.5m CD which created erosion pockets at the toe as well as on the core-loc slope. Three core-locs were displaced to the toe during this test.

Throughout the eight tests the (1 tonne – 3 tonne) toe rocks and first row of Core-Loc were stable. On the ninth test the first two rows of (1 tonne- 3 tonne) rocks were displaced. The (300 Kg – 700Kg) layer also failed (Appendix E – Section B), leading to a modification of the toe by extending the coverage area of the (1 tonne – 3 tonne) toe rock down to the seabed. Figure 4.8 through to Figure 4.11 present images of Test section B.

Figure 4.8: Section B Image of wave overtopping during Test B 03

Figure 4.9: Section B Image of wave overtopping during Test B 05

Figure 4.10: Section B Image of wave draw down during Test B 05

Figure 4.11: Section B Image of wave draw down during Test B 08

4.4. Section C

Section C was a series of four tests with two tests at a water level of +1.9m CD and the remaining two at 0.0m CD to assess the rear slope stability against overtopping. Test 1 and 2 projected most of the green water overtopping 10m behind of the deck. Moderate settlement of Core-Locs on the seaward slope was seen between row 1 to row 5. The leeward Core-Loc slope showed minor movement. During test 2 re-arrangement of (0.3 tonne – 0.7 tonne) toe rocks at -4.3m CD was visible. Test 3 and 4 had limited movement of Core-Locs on the seaward slope near the crest along row 2 and minor movement of rocks on the seaward toe.

During the test runs two failure mechanism were identified. The +1.9m CD water level runs showed wave overtopping projecting approximately 10m leeward of the deck making some of the toe rocks unstable. Thereafter, the lower water level runs had wave overtopping land on the deck simply to wash out Core-Locs at the edge. The leeward toe rocks between -4.3m CD to -7.0m CD eroded contributing to the Core-Loc sliding to failure. Core-Loc units from row 1 down to the waterline at 0.0m CD were displaced.

Figure 4.12 through to Figure 4.15 present images during Test section C.

Figure 4.12: Section C Images of wave overtopping projecting 10m leeward during Test C 01

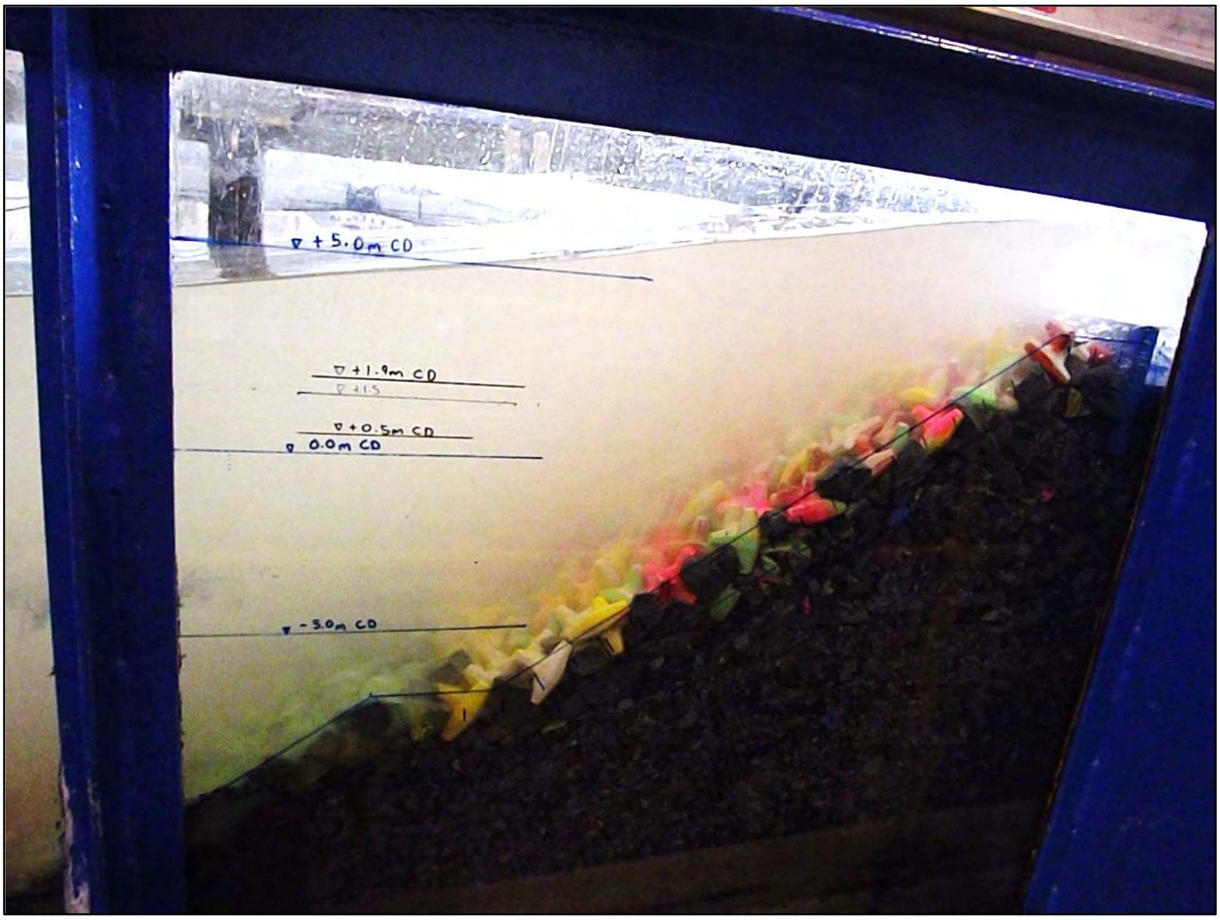

Figure 4.13: Section C Images of wave overtopping crest during Test C 01

Figure 4.14: Section C Image of overtopping on leeward slope during Test C 03 & 04

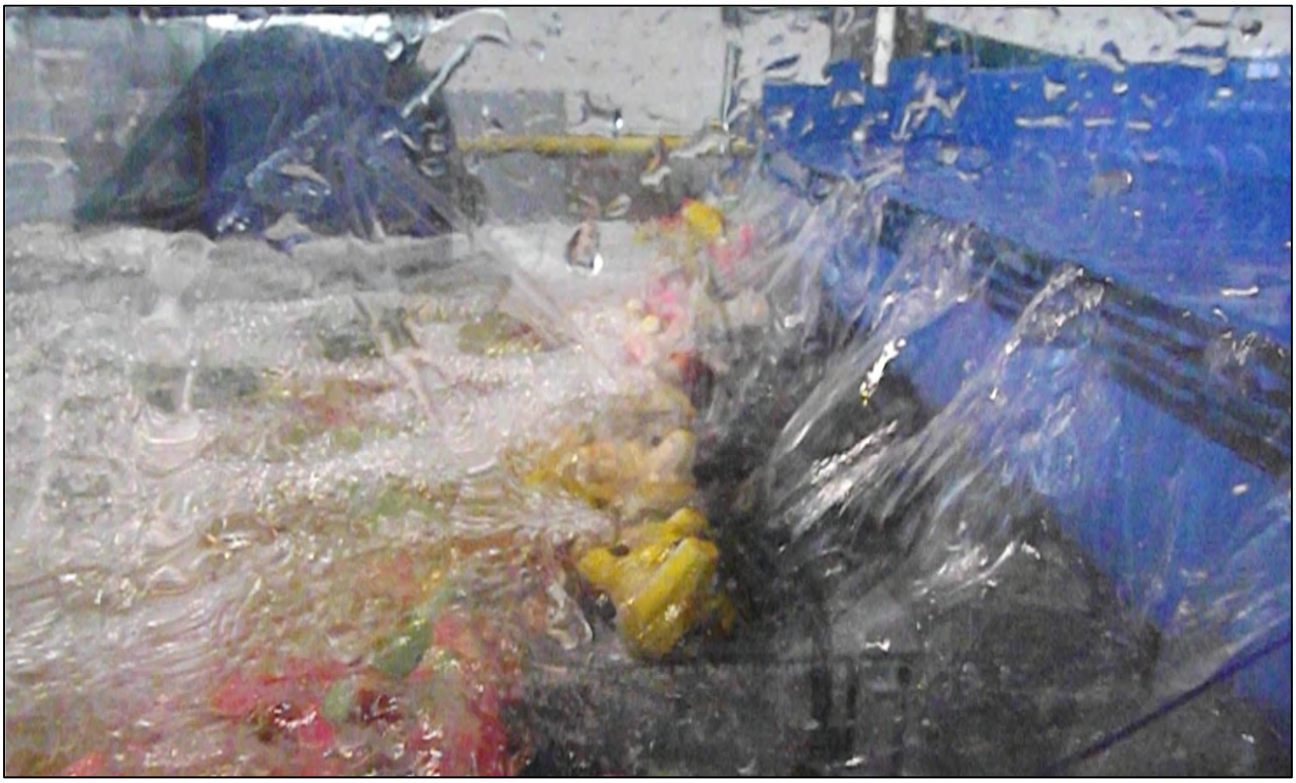

Figure 4.15: Section C Image of displaced Core-Loc units on the leeward slope during Test C 03 & 04

5. CONCLUSION

The CSIR Coastal & Hydraulics Laboratory conducted 2D physical model stability and wave overtopping measurements for the Rupert's Bay Wharf breakwater. The model was constructed at a scale of 1:37. Three cross-sections were tested under various wave heights, wave periods and water levels ranging from admissible to extreme conditions. The stability tests were conducted to assess the hydraulic stability of the 5 tonne Core-Loc units on the seaward and leeward slope, the (0.3 tonne – 0.7 tonne) under layer and the (1 tonne – 3 tonne) rock toe. This study did not focus on the deck wall stability or forces on the deck wall and was constructed fixed to the flume.

5.1. Seaward slope

The seaward slope was initially constructed with (0.3 tonne - 0.7 tonne) rock as the toe supporting the 5 tonne Core-Loc units at a level of -8.47m CD. The Core-Loc units extended from -8.47m CD to +5.0m CD allowing for 21 rows of Core-Locs. During testing section A, it was noticed that the toe rocks were being displaced. Section A was modified to an increased rock mass in section B to (1 tonne – 3 tonne) to attempt to stabilise the slope. The (1 tonne – 3 tonne) rocks supported the first row of Core-Locs. Limited movement of toe rock was seen during Test section B and test section C. Although the toe was stable movement of Core-Locs between rows 1 - 5 from the crest wall was still noticeable during test section B. This movement is partially due to Core-Locs against the flume glass settling due to limited interlocking. Once the units moved into the voids (during test 2) the Core-Locs stabilised and no further displacements were recorded. Test 9 was a destruction test to assess the reserve stability of the core-loc units. Three units were displaced of the slope. During this test, two rows of (1 – 3 tonne) toe rocks were also displaced. Test series section C confirmed the stability of the seaward slope as being stable for the four conditions tested.

5.2. Leeward slope

The leeward slope stability during test series section C was tested. The section failed due to excessive wave overtopping. Wave overtopping projected approximately 10m leeward of the deck. This resulted in displacement of toe rocks. During the lower water level tests, wave overtopping projected onto the deck and rushed off to wash out the Core-loc units. The toe rock displacement and wave overtopping contributed to the leeward Core-loc slope failure.

5.3. Overtopping

Wave overtopping was in the form of green water. The overtopping was excessive. Overtopping measurements are tabulated in Table 4.1.

5.4. Recommendation

The seaward slope tested during section B and section C are considered stable for the test series. The leeward slope tested in series section C requires the toe rock to be made deeper to around -7m CD. The Core-Loc units on row 1 at the deck should be lowered so the flukes of the Core-Loc units do not exceed the deck height to prevent direct wave impact from overtopping. Options to reduce the impact of wave overtopping should be considered such as splitter blocks at the leeward part of the deck. An inclined front seaward wall will allow waves to project further leewards and reduce impact of rear core-loc units.

6. REFERENCES

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7. APPENDICES

Appendix A:	Scaling of Physical Models
Appendix B:	Foreshore Profiles
Appendix C:	Model Setup, Rock Grading
Appendix D:	Cross-sections
Appendix E:	Test Images
Appendix F:	Model Parameters and Damage Definition
Appendix G:	Core-Loc Placement

7.1. Appendix A – Scaling for Physical Models

When scaling a physical model all physical processes should in fact be scaled. This however is not possible for practical reasons and also not necessary in most occasions.

In the physical model all lengths are scaled by a certain factor. Typically, scaled models are constructed to be as large as possible to diminish scale effects. The scaling effects occur because not all physical processes can be scaled down on the same scale. In hydraulic models most scale effects occur because the properties of the water, like density, viscosity and surface tension are not scaled.

Viscosity does not play any significant role in rotational free gravity surface waves. The energy dissipation because of friction with the bottom is not significant for the small distance the waves travel in the model. Therefore the viscosity is neglected and the model is not scaled according Reynolds law of scaling.

The surface tension of the air-water surface can play a role in the wave celerity for small waves. For depths over 2 centimetres and periods of over 0.35 s, this does not play a significant role (Hughes 1995).

In this model, gravity and inertia are the dominating forces that drive the waves. The setup of the model was therefore chosen to ensure similitude with the Froude law of scaling.

For Froude number similarity, F_N prototype must equal $F_{N\ \text{model}}$, thus

$$F_N = {}^{V_p}\!/\!_{\sqrt{gL_p}} = {}^{V_m}\!/\!_{\sqrt{gL_m}} \qquad \text{.........Eqn. A1}$$

$$\therefore {}^{V_p}\!/\!_{V_m} = {}^{\sqrt{gL_p}}\!/\!_{\sqrt{gL_m}} = \sqrt{{}^{L_p}\!/\!_{L_m}} \qquad \text{.........Eqn. A2}$$

Also,

$$\text{And } {}^{V_p}\!/\!_{V_m} = \frac{{}^{L_p}\!/\!_{T_p}}{{}^{L_m}\!/\!_{T_m}} = {}^{L_p}\!/\!_{L_m} \cdot {}^{T_m}\!/\!_{T_p} \qquad \text{.........Eqn. A3}$$

$$\frac{V_p}{V_m} = \sqrt{{}^{L_p}\!/\!_{L_m}} = \frac{L_p}{L_m} \cdot \frac{T_M}{T_P} \qquad \text{.........Eqn A.4}$$

$$\Rightarrow \frac{T_P}{T_M} = \sqrt{{}^{L_p}\!/\!_{L_m}}$$

where:

F_N = Froude Law of similitude [-]
V_p = Velocity in prototype [m/s]
V_m = Velocity in model [m/s]
L_p = Length in prototype [m]
L_m = Length in model [m]
g = Acceleration of gravity [m/s^2]
T_p = Time in prototype [m]
T_m = Time in model [m]

In a similar way, the other scale factors can be derived. A summary is given of all scale factors are given in the following table:

Table A1: Scaling Table For The 2D Models

Variable	2D Scale
Length or distance [m]	n = 37
Time [s]	$n^{1/2}$ = 6.08
Mass [kg]	n^3 = 57104
Volume [m^3]	n^3 = 50653
Force [N]	n^3 = 50653

The first three magnitudes are called the principal magnitudes from which the scaling for the other magnitudes can be derived.

7.2. Appendix B – Foreshore Profiles

Table B1: Bathymetrical Spot Checks

Bathy constructed in flume spot check		
Station	Prototype depth (m CD)	Scaled value from levelling equipment
16	-25.0	0
17	-23.3	45
18	-21.7	88
19	-20.1	132
20	-18.5	175
21	-16.9	219
22	-15.3	262
23	-13.7	306
24	-12.1	349
25	-10.5	393
26	-8.8	437
27	-8.8	437

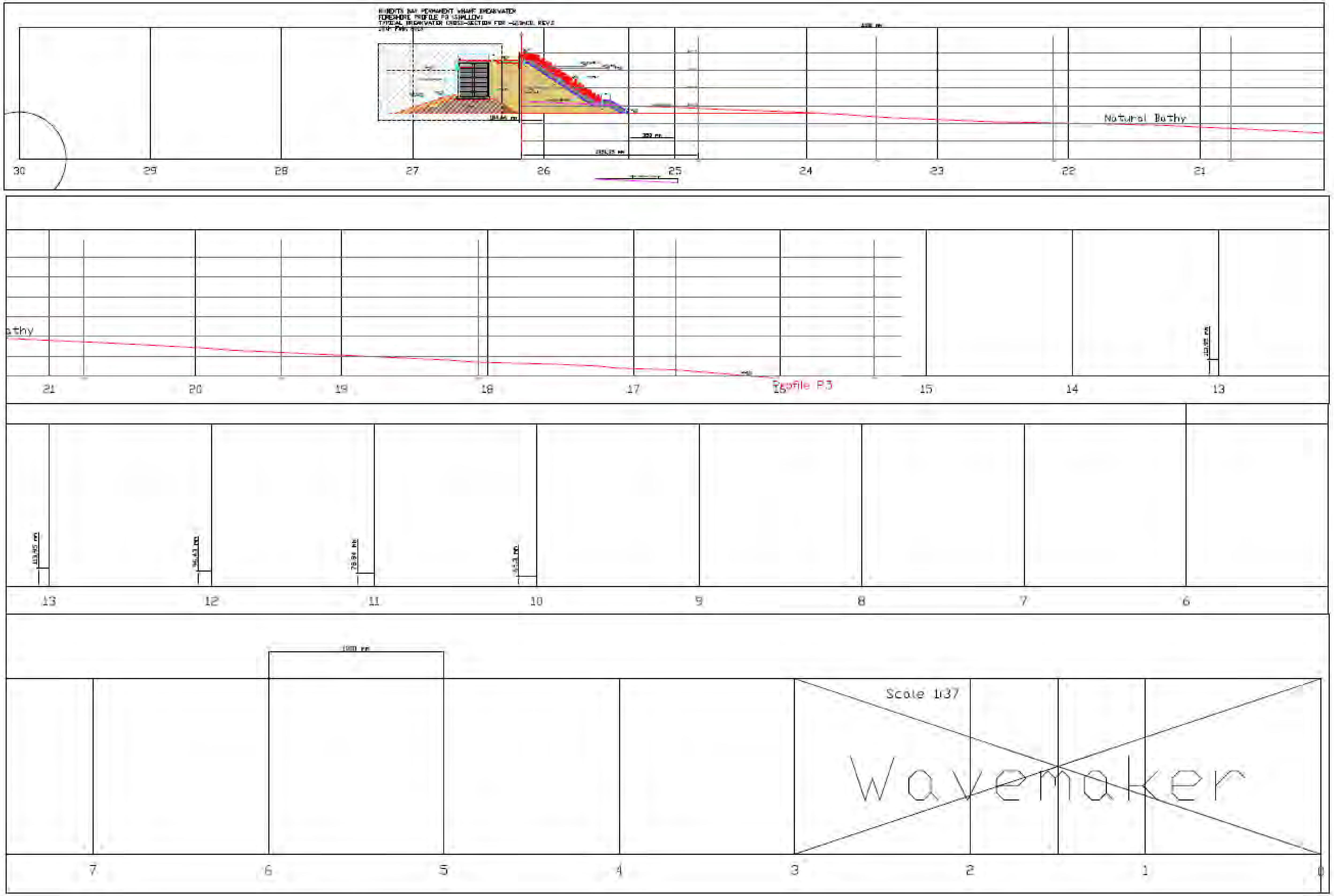


Figure B1: Profile of actual seabed

Model setup

Froude scaling has been applied to obtain the model test characteristics. The Froude number $F_r = {v^2}/{(g \cdot l)}$

$$F_r = \frac{v^2}{(g.l)}$$

is defined as the ratio of gravity and inertia forces; Froude scaling assures that gravity forces are correctly scaled.

Froude scaling is associated with linear length scale, the revetment model is downscaled linearly from the prototype structure; this means that the model geometry is identical to the prototype.

The various rock sizes have been scaled according to their stability number

$$\frac{H_s}{\Delta \cdot D_n},$$

,where H_s is the significant wave height, $\Delta = {\rho_r}/{\rho_w} - 1$ is the relative density and D_n is the nominal diameter. The stability number should be identical in both prototype and model; therefore, the scaling of rock sizes in the model is adjusted with respect to deviations in relative density between model and prototype.

A scale length λ =1:37 has been applied for the revetment model, thus the rock sizes can be determined by:

$$D_{n,m} = \lambda \cdot D_{n,p} \frac{\Delta_p}{\Delta_m}$$

By using the relation between rock mass and rock size,

$$W = \frac{1}{6} \cdot \rho_r \cdot \pi \cdot D_n^3 W = {1}/{6} \cdot \rho_r \cdot \pi \cdot D_n^3 = {1}/{6} \cdot \rho_r \cdot \pi \cdot D_n^3$$

; and assuming that the density of rock is identical between the model and prototype, the previous equation is simplified as:

$$W_m = \lambda^3 \cdot W_p \left(\frac{\Delta_p}{\Delta_m} \right)^3$$

Where λ is the length scale, W is the rock mass and Δ is the relative density; the indices m and p refer to model and prototype, respectively.

1.1. Material properties

Graded rock is applied for the construction of all the layers of the revetment. The specific density considered in the prototype structure for the rock and seawater is 2,650 kg/m^3 and 1,025 kg/m^3, respectively. In the case of the model, such densities are 2,650 kg/m3 and 1,000 kg/m3, respectively.

1.2. Armour layers and Core

The armour rock varies in each profile; therefore, several rock gradings are determined. Detailed information about the required rock grading given by PRDW is given in Table C1.

The grading used for the rock grading is given in Figure C.1. Figure C.2, Figure C.3 and Figure C.4 present the grading achieved in the model. The values have been converted to correspond with the prototype.

Table C1: Rupert's Bay Wharf – Rock Grading

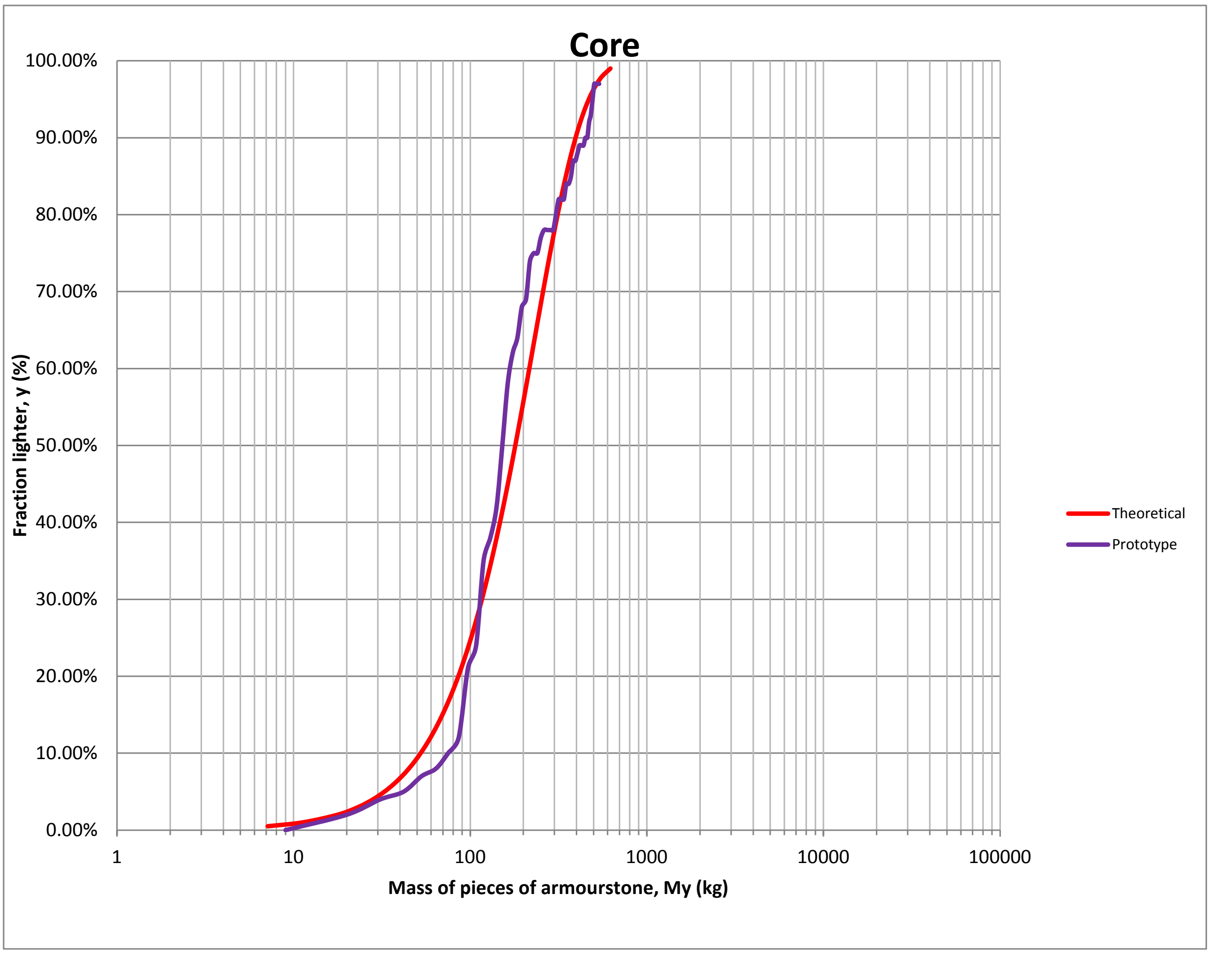


Figure C1: Grading curves for core rock

Table C2: Rupert's Bay Wharf Underlayer Rock

Rupert's Bay Wharf				
Armour		**NLL**	**M50**	**NUL**
Prototype	Weight [kg]	300	500	700

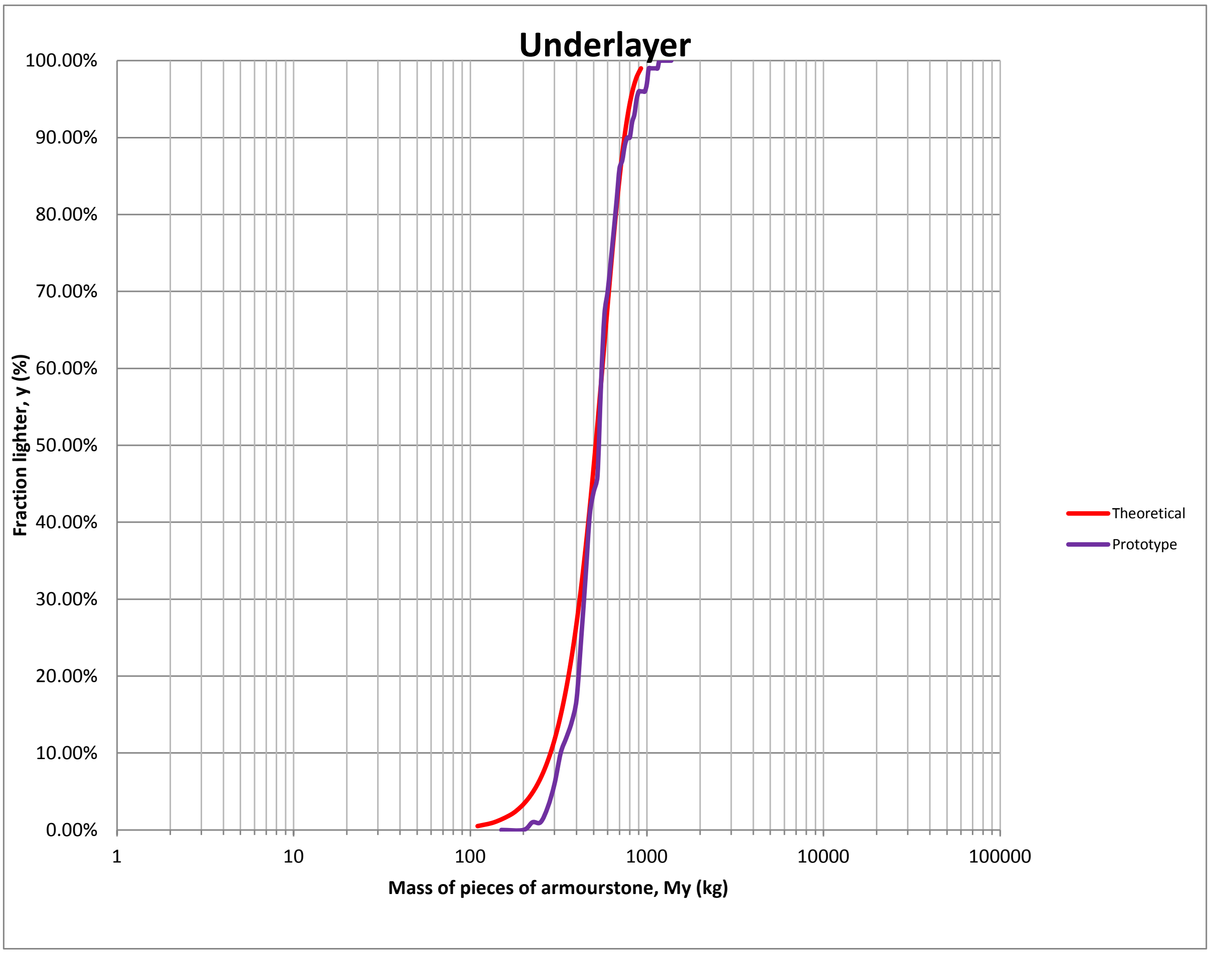


Figure C2: Grading curves for underlayer rock Section A & B

Rupert’s Bay Wharf				
Armour		**NLL**	**M50**	**NUL**
Prototype	Weight [kg]	300	500	700

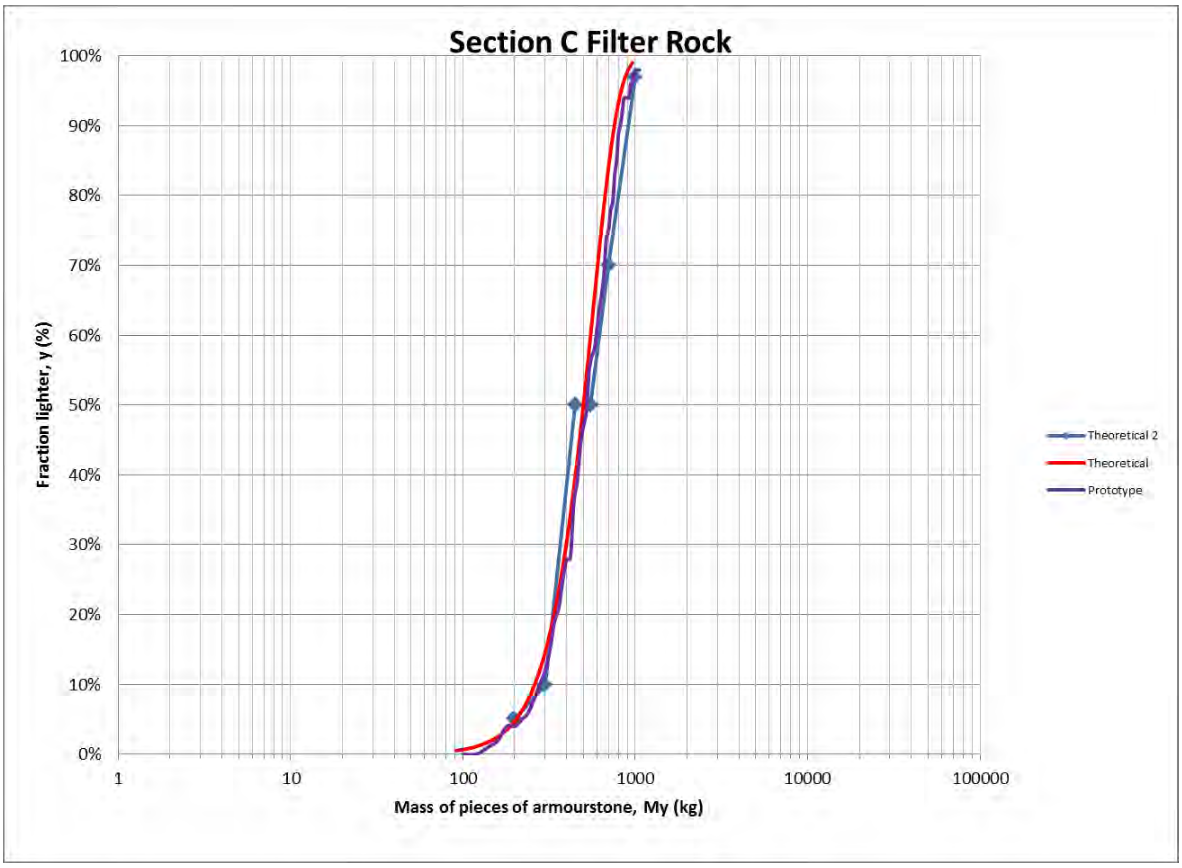


Figure C3: Grading curves for underlayer rock Section C

Rupert’s Bay Wharf				
Armour		**NLL**	**M50**	**NUL**
Prototype	Weight [kg]	1000	2000	3000

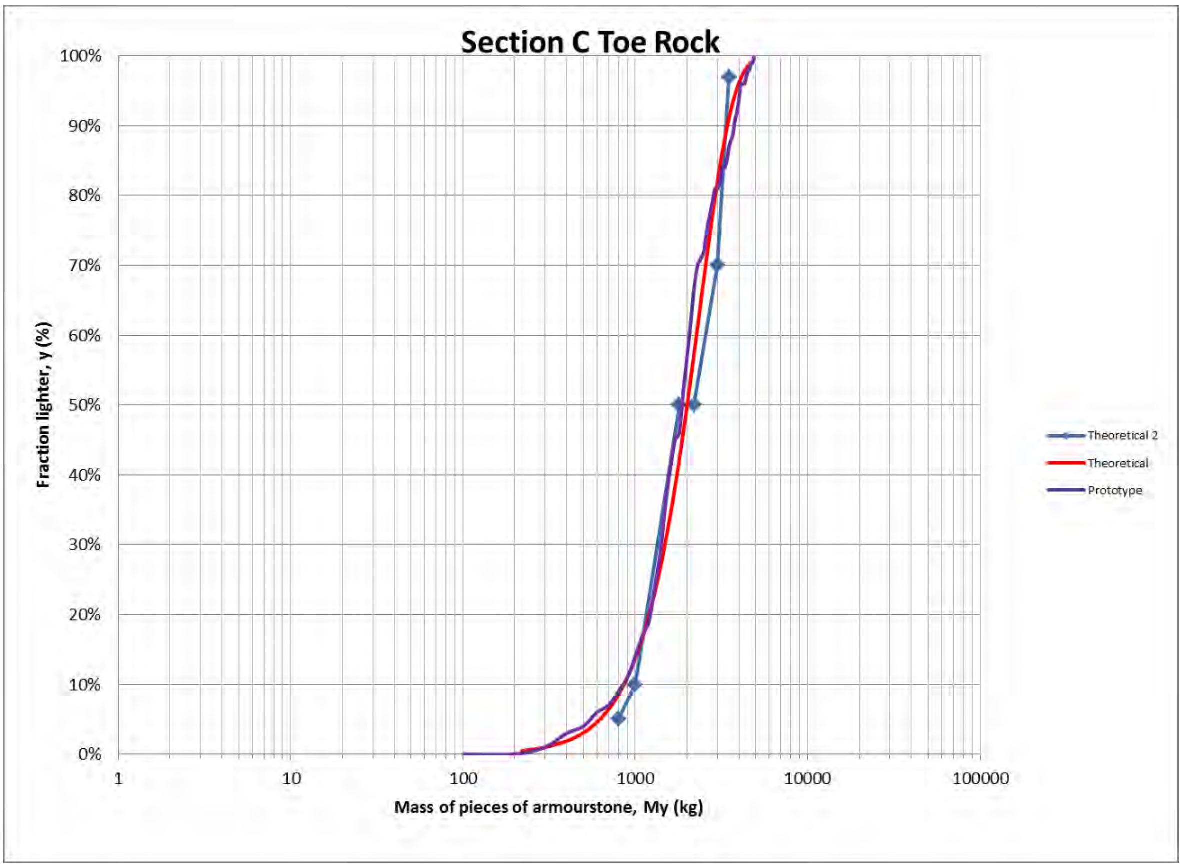


Figure C4: Grading curves for Seaward Toe rock Section B & C

7.4. Appendix D – Cross Sections

Three cross-sections were constructed and tested. The cross-sections are presented in this section.

Figures D1 to D3 illustrates the cross-sections constructed and tested for Rupert's Bay Wahrf

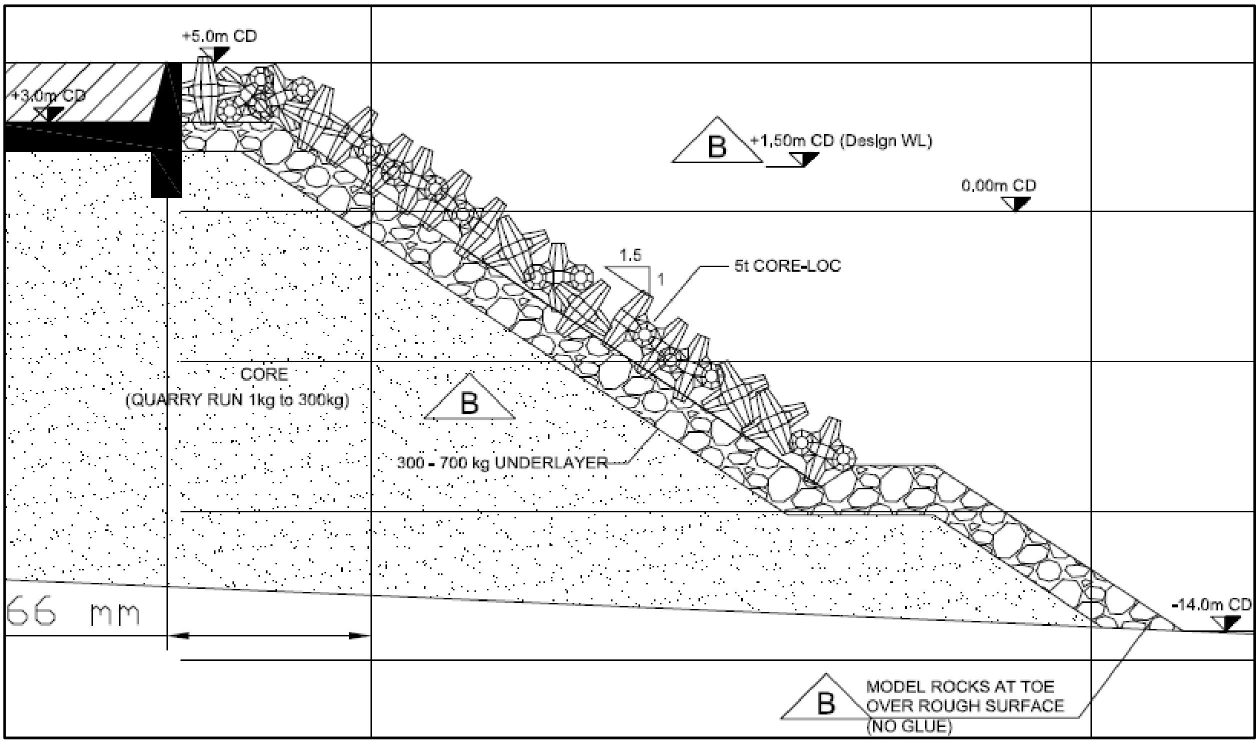


Figure D1: Cross-section A

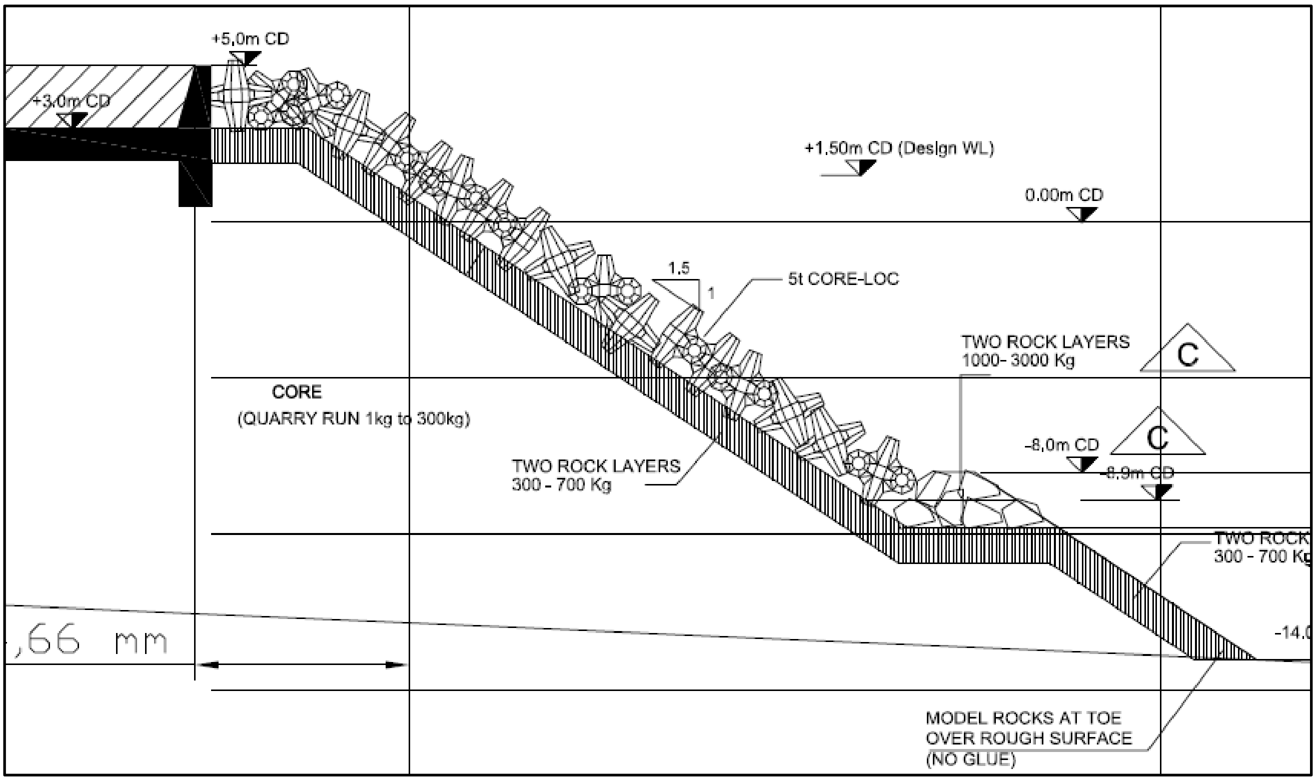


Figure D2: Cross-section B

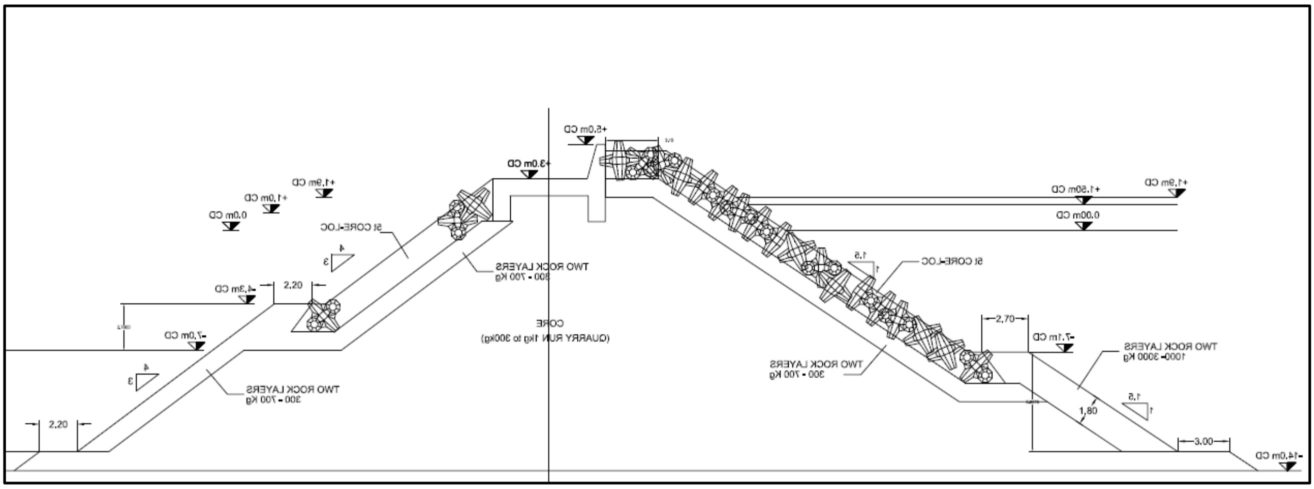


Figure D3: Cross-section C

7.5. Appendix E – Test Images

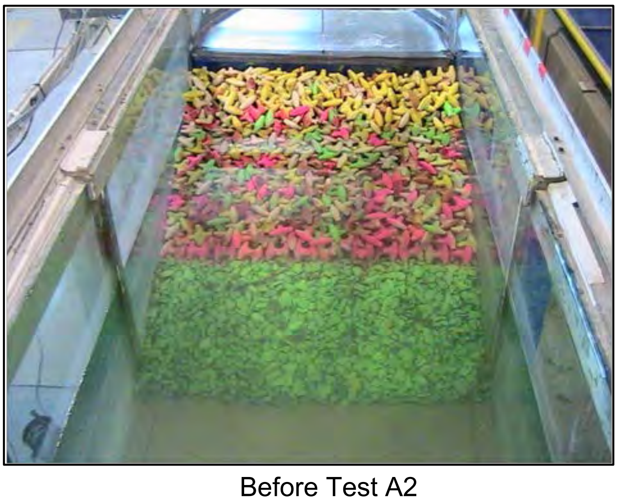
This section presents the images before and after each test for the Rupert's Bay Permanent Wharf.

Section A Test 1, $Hs_{@-20mCD}$: 3.28m, $Tp_{@-20mCD}$: 15.9s, WL=+1.5m CD

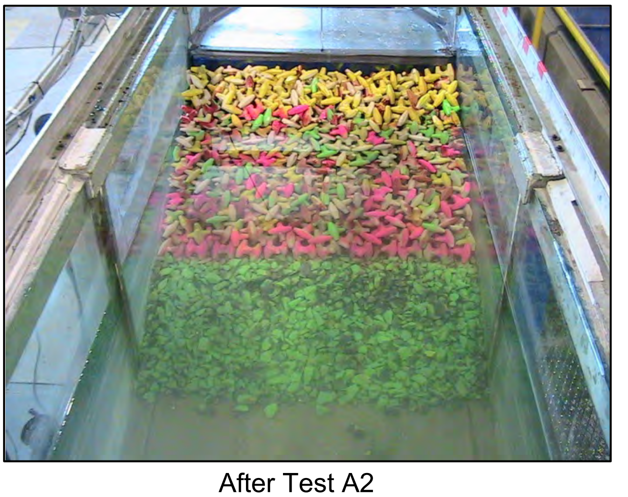
Before Test A1

After Test A1

Section A Test 2, $Hs_{@-20mCD}$: 4.33m, $Tp_{@-20mCD}$: 15.9s, WL=+1.5m CD, BL=-8.47m CD, CL=+5.0m CD



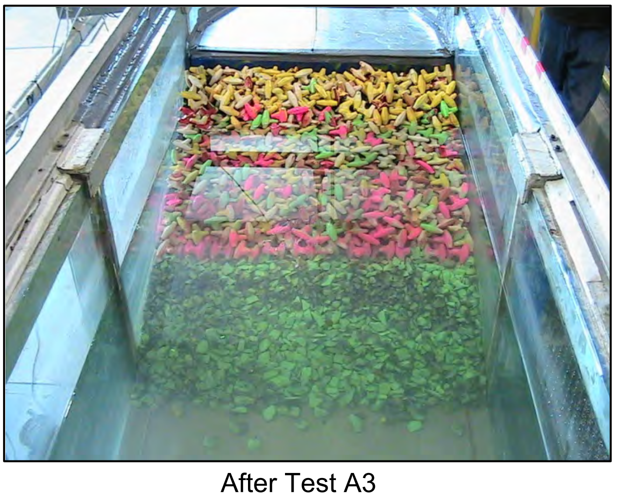
Before Test A2



After Test A2

Section A Test 3, $Hs_{@-20mCD}$: 5.05m, $Tp_{@-20mCD}$: 15.9s, WL=+1.5m CD, BL=+8.47m CD, CL=+5m CD

Before Test A3



After Test A3

Section B Test 1, $Hs_{@-20mCD}$: 3.18m, $Tp_{@-20mCD}$: 15.7s, WL=+1.5m CD, BL=+8.47m CD, CL=+5m CD

Before Test B1

After Test B1

Section B Test 2, $Hs_{@-20mCD}$: 4.32m, $Tp_{@-20mCD}$: 15.9s, WL=+1.5m CD, BL=+8.47m CD, CL=+5m CD

Before Test B2

After Test B2

Section B Test 3, $Hs_{@-20mCD}$: 4.36m, $Tp_{@-20mCD}$: 12.92s, WL=+1.5m CD, BL=+8m CD, CL=+5m CD

Before Test B3

After Test B3

Section B Test 4, $Hs_{@-20mCD}$: 4.23m, $Tp_{@-20mCD}$: 18.79s, WL=+1.5m CD, BL=+8m CD, CL=+5m CD

Before Test B4

After Test B4

Section B Test 5, $Hs_{@-20mCD}$: 4.73m, $Tp_{@-20mCD}$: 16s, WL=+1.9m CD, BL=+8m CD, CL=+5m CD

Before Test B5

After Test B5

Section B Test 6, $Hs_{@-20mCD}$: 4.72m, $Tp_{@-20mCD}$: 12.92s, WL=+0.0m CD, BL=+8m CD, CL=+5m CD

Before Test B6

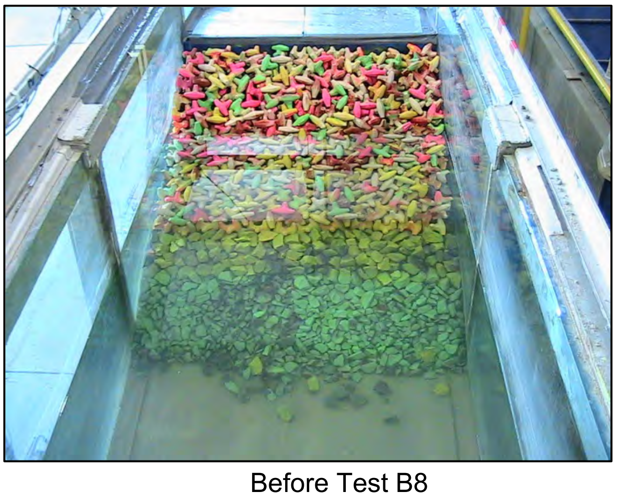
After Test B6

Section B Test 7, $Hs_{@-20mCD}$: 4.72m, $Tp_{@-20mCD}$: 15.9s, WL=+0.0m CD, BL=+8m CD, CL=+5m CD


Before Test B7

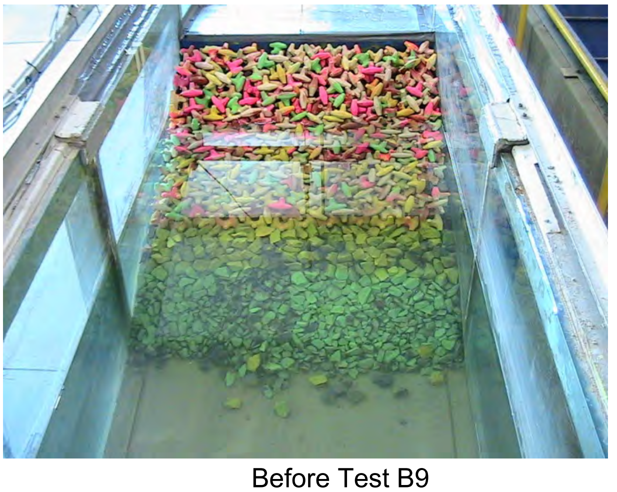

After Test B7

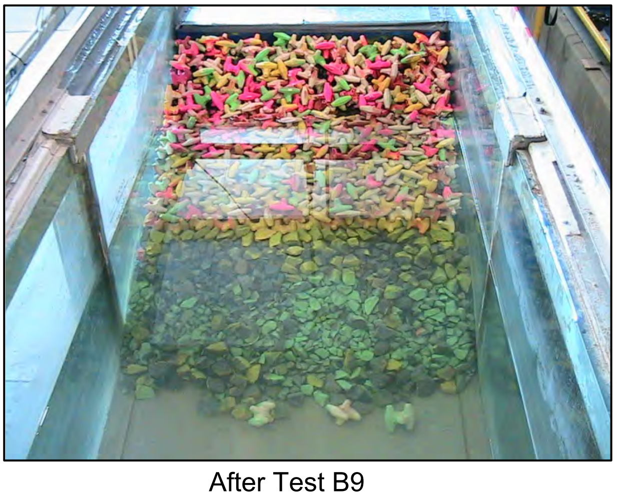
Section B Test 8, $Hs_{@-20mCD}$: 4.82m, $Tp_{@-20mCD}$: 18.78s, WL=+0.0m CD, BL=+8m CD, CL=+5m CD


Before Test B8


After Test B8

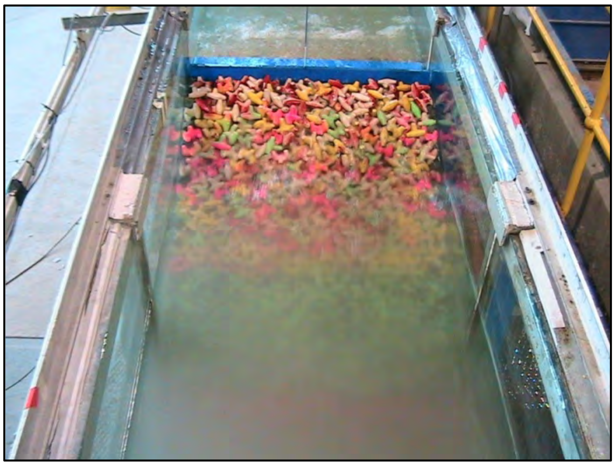
$Hs_{@-20mCD}$: 5.5m, $Tp_{@-20mCD}$: 18.78s, WL=+0.0m CD, BL=+8m CD, CL=+5m CD


Before Test B9


After Test B9

Section C Test 1, $Hs_{@-20mCD}$: 4.69m, $Tp_{@-20mCD}$: 15.9s, WL=+1.9m CD, Seaward BL=-7.1m CD, CL=+5m CD

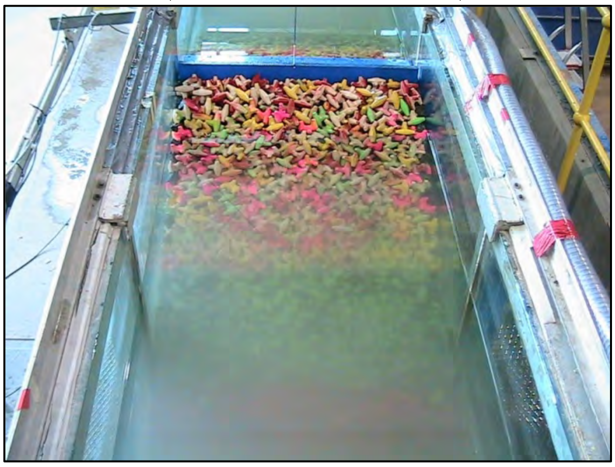
Before Test C1



After Test C1

Section C Test 2, $Hs_{@-20mCD}$: 4.72m, $Tp_{@-20mCD}$: 12.92s, WL=+1.9m CD, Seaward BL=-7.1m CD, CL=+5m CD

Before Test C2



After Test C2

Section C Test 3, $Hs_{@-20mCD}$: 4.53m, $Tp_{@-20mCD}$: 15.9s, WL=+0.0m CD, Seaward BL=-7.1m CD, CL=+5.0m CD

Before Test C3

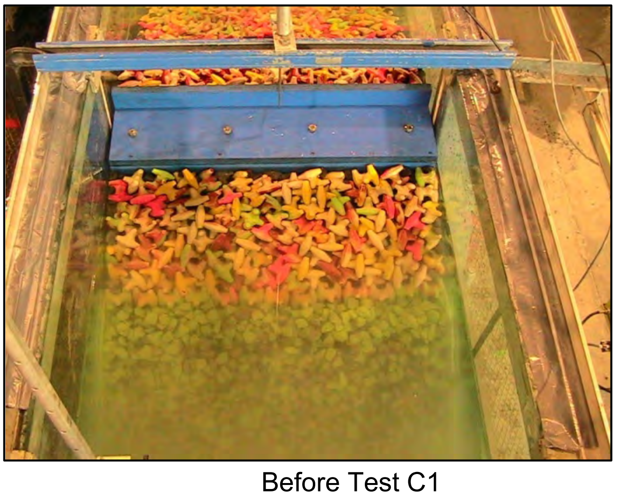
After Test C3

Section C Test 4, $Hs_{@-20mCD}$: 4.57m, $Tp_{@-20mCD}$: 18.78s, WL=+0.0m CD, Seaward BL=-7.1m CD, CL=+5.0m CD

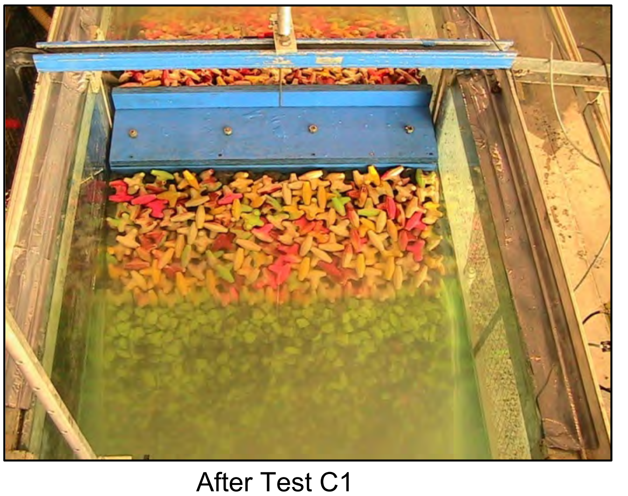
Before Test C4

After Test C4

Section C Test 1, $Hs_{@-20mCD}$: 4.69m, $Tp_{@-20mCD}$: 15.9s, WL=+1.9m CD, Leeward BL=-4.3m CD, CL=+5m CD

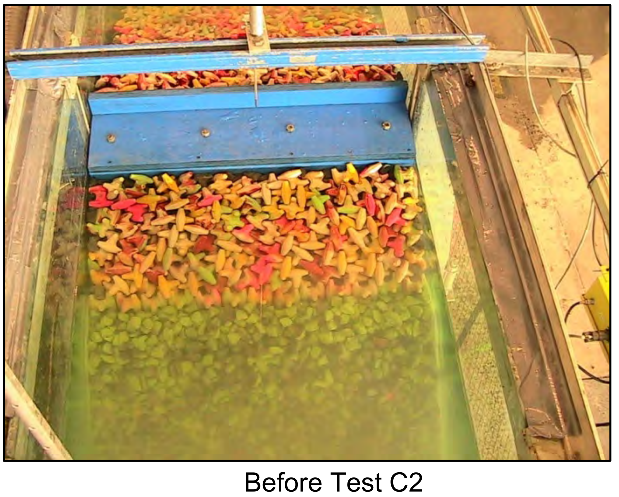


Before Test C1

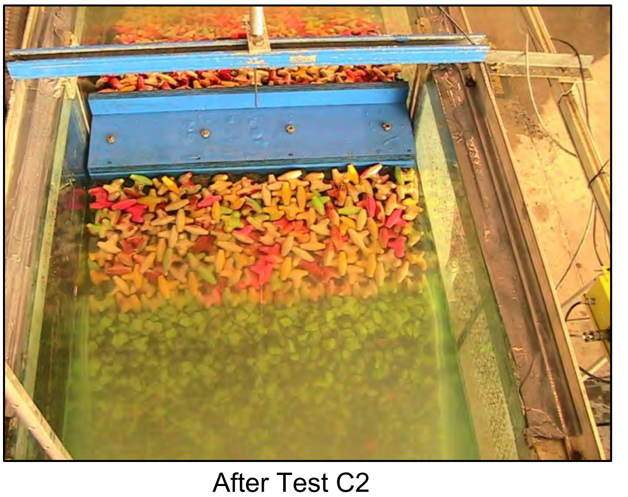


After Test C1

Section C Test 2, $Hs_{@-20mCD}$: 4.72m, $Tp_{@-20mCD}$: 12.92s, WL=+1.9m CD, Leeward BL=-4.3m CD, CL=+5m CD



Before Test C2



After Test C2

Section C Test 3, $Hs_{@-20mCD}$: 4.53m, $Tp_{@-20mCD}$: 15.9s, WL=+0.0m CD, Leeward BL=-4.3m CD, CL=+5.0m CD

Before Test C3

After Test C3

Section C Test 4, $Hs_{@-20mCD}$: 4.57m, $Tp_{@-20mCD}$: 18.78s, WL=+0.0m CD, Leeward BL=-4.3m CD, CL=+5.0m CD

Before Test C4

After Test C4

7.6. Appendix F – Model Parameters and Damage Definition

Table F1 present the formula to determine the relative eroded area and relative displacement to assess the damage level according to the Coastal Engineering Manual 2006.

Table F1: 2D Model Parameters and Damage Definition

Flume width	0.75m
Scale	1 in 37
Relative displacement within an area	$D = \frac{\text{number of displaced units}}{\text{total numbers of units within reference area}}$ The reference area has to be defined and is usually defined as the area between two levels e.g. SWL ± Hs or SWL ± n·Dn
Number of displaced units within a strip with Dn	$N_{od} = \frac{\text{number of units displaced out of armour layer}}{\text{width of the tested section / } D_n}$
Relative number of displaced units within the total height of armour layer	$\frac{N_{od}}{N_a}$ where Na is a strip of width D_n (this is similar to the definition of D, when the total height of the revetment is considered)
Relative eroded area	$S = Ae/D^2n50$

The above is based on the guidelines given in the Coastal Engineering Manual CEM (2006).

Almost all the damage occurs around the SWL, so the number of units missing in the strip will represent the number of units missing around the SWL. One disadvantage of using N_{od} is that it does not take the length of the slope into account (CEM 2006). When using D or D_v as damage, the height of the armour layer plays an important role in the calculation of the damage percentage. A high armour layer with a lot of units might have a low percentage of damage but since most of the damage occurs around a small band around SWL +/- Hs, this low percentage might be misleading. Therefore the use of N_{od} seems the most suitable for use in this case.

To determine the stability of rock, Nod in conjunction with D may be the most suitable analysis approach. Damage criteria have been developed for rock armour (CEM 2006), Table F2. D gives a good indication of the percentage of armour that has moved greater than Dn on the toe, but by definition, it should be based on damage occurring in a horizontal strip centred around the SWL. The toe is, however, at a depth less than SWL, and thus by definition, the failure limits (Table F3) are not applicable to the toe. CEM 2006 gives the classification of damage levels for percentage damage (D) and damage through Nod. S can be interpreted as the number of squares with side length Dn50 which fit into the eroded area, or as the number of cubes with side length Dn50 eroded within a strip width Dn50 of the armour.

Table F2: Classification Of Damage Levels, D And N_{od} (CEM 2006)

Damage Level	Notes			
	D (%)	Nod: Slope Stability	Nod :Toe Stability	
No Damage	0	0.5	0 – 0.5	No units displaced
Initial Damage	0 - 3	-	0.5 – 2.0	Few units are displaced
Intermediate Damage	3 - 5	-	2.0 – 4.0	Units are displaced but no exposure of the under layer or filter layer
Failure	>=20	>=2.0	>=4.0 (Severe)	The under layer or filter layer is exposed to wave attack.

Table F3: Classification Of Damage Level By S For Two Layer Armour (Van Der Meer 1988)

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

Test No.	Measurement File Name	Cum D %
1	A1	0.0%
2	A2	0.0%
3	A3	1.2%
	SD	-
4	B1	0.7%
5	B2	2.7%
6	B3	2.7%
7	B4	2.7%
8	B5	2.9%
9	B6	2.9%
10	B7	2.9%
11	B8	2.9%
12	B9	
	SD	-
12	C1	0.2%
13	C2	0.7%
14	C3	1.0%

7.7. Appendix G – Core-Loc Placement

The most recent guidance is that a target packing density of 0.6 is appropriate for all sizes of Core-Loc Units. This replaces the guidance given in CHL-97-6, which indicates that slightly lower packing densities are appropriate for larger units. It is anticipated that the general Placement Grid shown in Figure 8 of CHL-97-6 will be followed and that the Core-Loc units shall be placed in accordance with the Core-Loc Technical Guidance (Report CHL-97-6) to achieve a target packing density of 0.6.

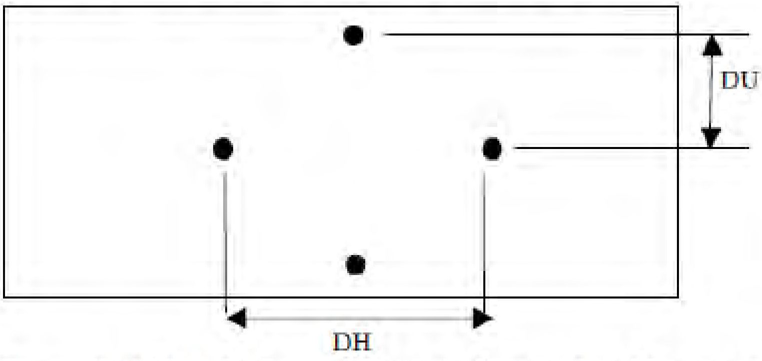
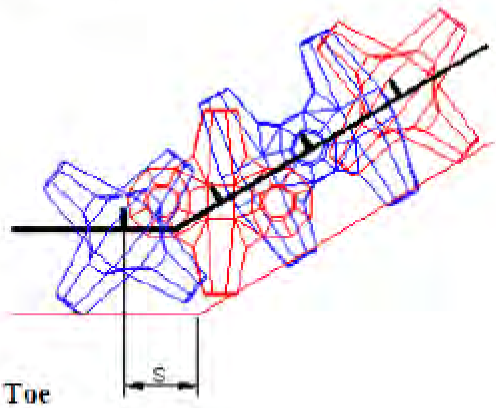


Figure 8 of CHL-97-6 Schematic of Core-Loc Placement Grid

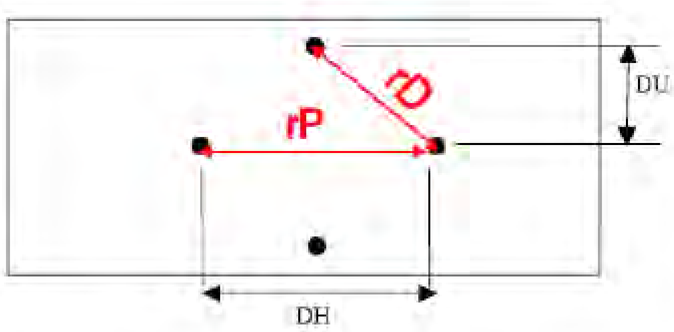
Summary Packing Density and Placement Grid Dimensions

Volume	**Target Packing Density**	**DH**	**DU**
All volumes	0.60	1.11C	0.55C

The toe row will be set out at the distance s measured from the bottom of the slope, where s = 0.5xDU. The bottom of the slope is taken to be the bottom of the slope formed by the top surface of the under layer rock (shown by the the thin red line in the diagram below).



Section through Core-Loc Toe



rP and rD are used in the gridgenerator 3D placement application that runs within Autocad
$rD = ((0.5\ DH)^2 + DU^2)^{0.5}$
$rP = DH$

Packing density formulae:

$\phi = n\ V^{2/3}$

$n = 1/\ (DH \cdot DU)$

n = number of units per m^2
ϕ = number of units per D_n^2
$D_n = V^{1/3}$

Area of triangle:
$A = ((s(s-a)(s-b)(s-c))^{0.5}$ where $s=0.5(a+b+c)$
and a, b and c are the side lengths of the triangle

$n=0.5/A$

Figure G1: Image of First placement

This was checked by counting all the units. The top two rows had 15 units each. Two units from each were taken out as indicated by the arrows.

Figure G2: Counting of placed units

The final image before testing is shown below

Figure G3: Final placement before Test 1

APPENDIX E

ICEBREAK CONSULTING ENGINEERS –

QUARRY RUN FOR CORE MATERIAL

TECHNICAL MEMO

Ruperts Bay – Core material

Prepared for:	Prestedge Retief Dresner Wijnberg (Pty) Ltd, Anton Holtzhausen Pr Eng
Prepared by:	IceBreak Consulting Engineers, Sigurður Sigurðarson
Date/Revision:	10 June 2013 / Rev. B
Regarding:	Use of quarry run for core material

1. Introduction

IceBreak Consulting Engineers has been inquired by PRDW about the use of quarry run as core material for a breakwater in Ruperts Bay on St Helena Islands in the south Atlantic. The basis for this work are 11 photos from the Ruperts Bay quarry, Figures 1 and 2 of this memo, and an excel sheet with measurements of density and water absorption of stones collected from the quarry face after blasting, included as Figure 5 of this memo.

2. Quarry run as core material in the Rock Manual

The terminology of core materials in the Rock Manual, 2007, defines “quarry run” as including all granular material found in the quarry blast-pile that can be picked up in a typical loading shovel.

According to the Rock Manual the decision whether to specify no fines removed or fines removed requires an understanding of the performance of the different type of materials, key parameters often being the porosity or the risk of damage to the structure through piping or internal erosion. The considerations that favour the removal of fines include instability because of clay minerals in the fines and shear strength and liquefaction potential. Still it is stated that there is a lack of guidance on the subject.

The Rock Manual continues that removal of fines significantly increases porosity and permeability, so where it is desired to reduce wave transmission (as is the case for most breakwaters) it should only be necessary to remove fines if geotechnical design factors make it necessary.

3. Quarry run as core material in international projects

Screening of core material is quite common in international projects. With reference to the recommendations of the Rock Manual the question remains, if it is always necessary?

One of the reasons for screening the quarry run in many projects is that the material has to be transported long distances. As the transport is usually paid for in tonnes, it can be economical to screen the fine material. When the transport distance is short and / or the project is paid for in m^3, then the presence of fines is not a major concern and it might not be economical to screen the material.

In several international projects where IceBreak has been involved the Rock Specifications have limited the fines in core material with an upper and lower limits passing a 100 mm sieve to 30% and 5%.

4. North Atlantic practise with quarry run as core material

In Iceland, all coastal rubble mound structures, breakwaters, revetments and shore protection structures, are constructed from basaltic quarries, except for two structures where gabbroic rock is used. All of these structures make a use of dedicated armourstone quarries where the distance from quarry to construction site varies from being adjacent to the structure up to about 30 km. Except for one of these structures they are all designed locally in Iceland and no screening of quarry run has been deemed necessary.

Similar is true for the Faroe Islands, also basaltic rock, and for Norway, with rock of various types, as gneiss, granite and gabbroic, screening of quarry run is usually not needed, deemed necessary or economical.

5. Quarry run from Ruperts Bay quarry in comparison with Icelandic quarries

The provided photos from the Ruperts bay quarry are included as Figures 1 and 2 of this memo. The photos show both dense rocks from the inner parts of the lava flows as well as more porous rock from the outer parts of the lava flows and breccia inbetween the lava flows. Evidence of fracturing from blast holes can be seen.

According to the provided information the density of the rock from Ruperts Bay varies from 2.7 t/m^3 for the dense rock to 2.5 t/m^3 for the medium dense rock to 2.3 t/m^3 for the porous rock, Figure 5 at the back of this memo. The water absorption varies from about 1% to about 5%. IceBreak is not fully convinced that the measurements of density and water absorption follow international standards.

For comparison Figures 3 and 4 show show 9 photos from a basaltic quarry in Iceland, the Eystri-Sólheimaheiði quarry used for construction of the Vik groyne in south Iceland in 2011. This quarry was neither very good nor very bad in comparison with other Icelandic quarries. (As a matter of fact it is chosen for comparison as it is the only quarry I have information about where I am located at the time of writing this memo) The quarry yield prediction of stones heavier than 1 tonne was 25 to 30% and it proved to be achievable during production.

Similar to the Ruperts Bay quarry there are zones with dense rock with large fracture pattern and zones with more fractures, as well as fracturing around blast holes. The density and water absorption were measured from the dense part of the quarry as 2.9 t/m^3 and 1.5-2%.

6. Technical Specification

With reference to density the Ruperts Bay quarry clearly fulfils all standards. The question is if the water absorption is too high. Usual criterion for water absorption of armourstone varies from 1% to 3%, sometimes differentiating from armour to under layers. For core material higher often higher criterion is accepted.

As there is no danger of degradation of basaltic rock in the core of the breakwater there is no reason to have a criterion on the water absorption. In Iceland there is no criterion for absorption of quarry run as core material.

7. Volumes in the breakwater and the need for quarrying

In total the project volume is about 77,000 m^3 of placed rock, of which 5.4% should be 1 to 3 tonne, Class I, and 11.0% 0.3 to 0.7 tonne, Class II, or 16.4% in Classes I and II.

The photos from the Ruperts Bay quarry do not show any stone in Class I. On the other hand pictures 9 and 10, Figure 2, show a potential source of Class I rock. From the photos alone it is not possible to estimate if this will account for 5.4% of the total quarried material. That has to be onsite.

It seems reasonable to widen Class II to include all rock from 0.3 to 1 tonne. That adds to the yield in Class II. Given that the quarry yields close to what is needed in Class I then it is likely that the contractor has to be careful in sorting all Class II stone from the quarry run, as in that case Class II could be critical for the quarrying. The reason for this statement is that IceBreak has often experienced projects where the contractor was not careful in sorting the smaller stones from the core in the beginning of the project but needed rock in those classes later on in the project.

It is worth pointing out that the need for extra quarrying is very sensitive to small margins in the yield percentage, especially for Class I. If the quarry yields 5% instead of 5.4% then the extra quarrying is about 6,000 m^3 and if it is 4.4% then 17,000 m^3. Pricing for extra quarrying should take to removal of overburden, drilling and blasting, loading on truck, driving less than 1 km and tipping. Only the material used in the breakwater should be priced for sorting of material.

8. Conclusion

The Rock Manual only recommends screening of quarry run for circumstances that are not valid for the Ruperts Bay project. The main reason for screening quarry run in many projects is economical and that is not the case for this project.

There is a long history of not screening quarry run in the northern Atlantic with many rock types and especially with basaltic rock in Iceland and the Faroe Islands. The Ruperts Bay quarry looks very similar to medium quality basaltic quarries in Iceland.

If required, it is possible to make a measurement of the fines passing 100 mm sieve. In that case it is important to choose the sample carefully and include in the calculations stones that would be in the quarry run in prototype but are avoided for the test.

Even though the porous rock has water absorption higher than usually accepted for armourstone there is no technical reason to exclude material from the porous parts of the quarry from the quarry run.

In Icelandic projects based on basaltic quarries the technical specifications for quarry run as core material only state that all material has to be blasted from the quarry, no organic material should be blended with the blasted material. Usually we do not allow more stones to be taken from the quarry run than are needed for the actual project. With a long record of successful projects in Iceland we do neither require any measurement of fines in our projects nor any test on physical or mechanical properties. For the Rupert Bay project it is recommended to do several simple tests with 100 mm sieve as described above, two to three tests early in the project and then one test per 10.000 m^3 of core material.

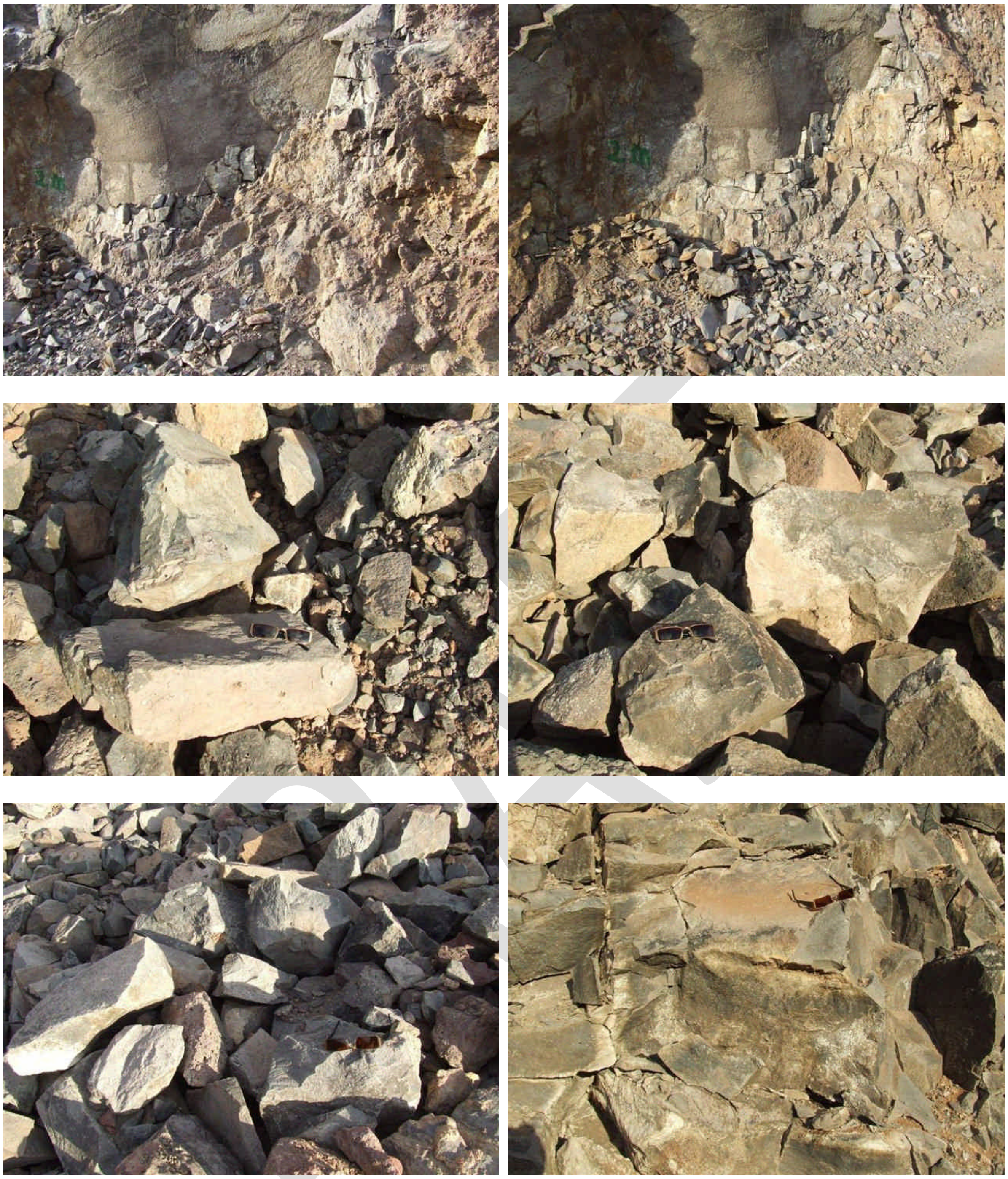


Figure 1. Photos 1 to 6 from the Ruperts Bay quarry on St Helena Islands.

Figure 2.Photos 7 to 11 from the Ruperts Bay quarry on St Helena Islands.

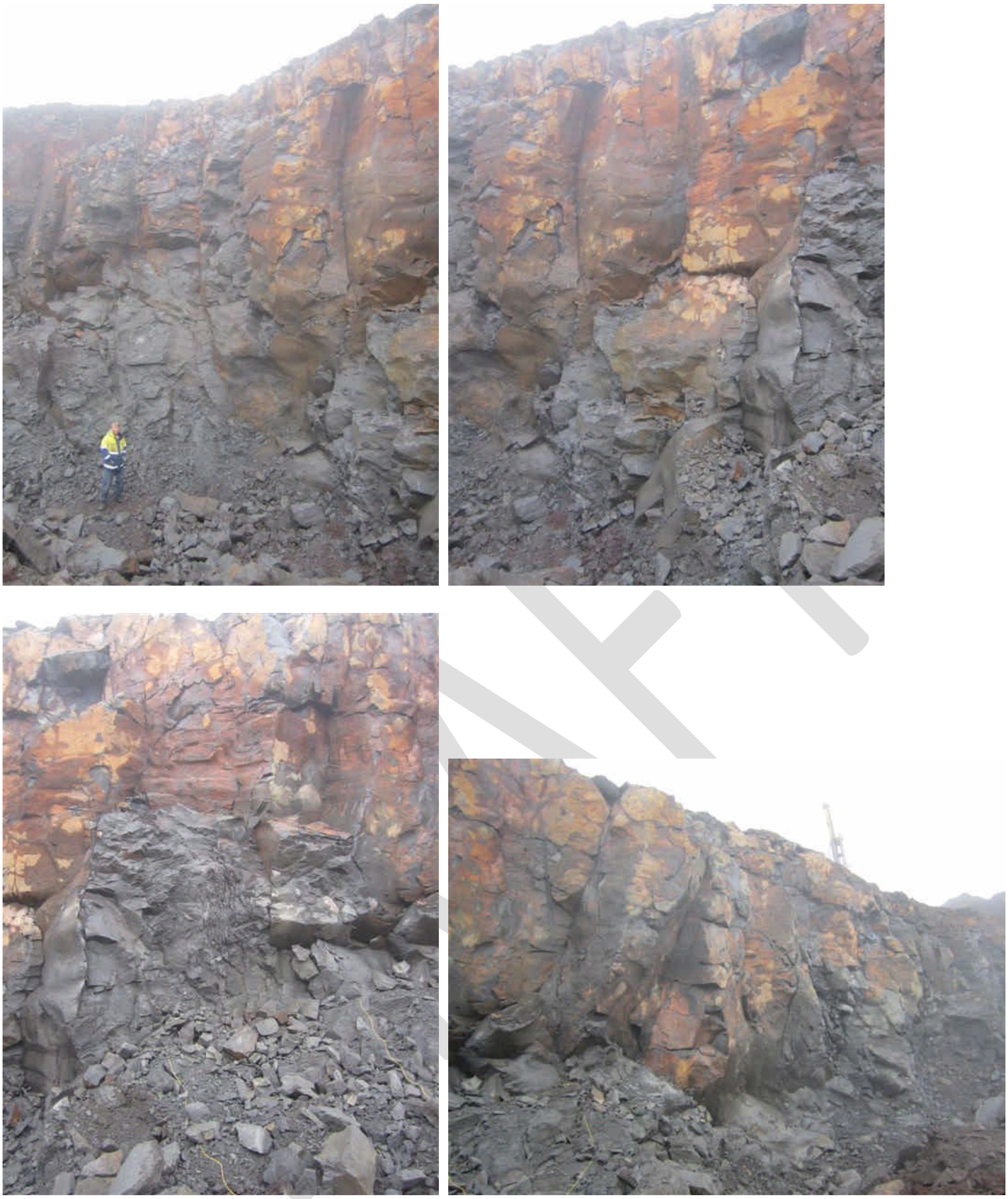


Figure 3. Photos 1 to 4 from the Eystri-Sólheimaheiði quarry for the Vik groyne in south Iceland.

IceBreak Consulting Engineers

Figure 4. Photos 5 to 9 from the Eystri-Sólheimaheiði quarry for the Vik groyne in south Iceland.

15/04/2013

ROCK DENSITY Rupert's B/P

Visually high-dense rock at 25°C

Specimen no	Dry mass	Mass after soaking	Mass submerged	Volume	Dens. Of stone (kg/m3)	Water absorption (%)
1	231,3	233,5	149,2	84,1	2.736	0,95
2	440,8	446	279,9	164,6	2.646	1,18
3	187	189,1	119,8	69,1	2.691	1,12
4	250,6	252,2	161,2	90,5	2.746	0,64
5	186	187,9	119,8	67,4	2.723	1,02
6	222,4	223,2	144,4	78,2	2.814	0,36
7	145,7	149,3	94	54,7	2.627	2,47
8	263,2	267,8	169,5	98	2.670	1,75
9	129,5	131,4	84,1	46,7	2.730	1,47
10	334	338,5	215,4	123,4	2.705	1,35
$\bar{x}$					2.709	1,23
S_n					54	0,59

Visually medium-dense rock at 25°C

Specimen no	Dry mass	Mass after soaking	Mass submerged	Volume	Dens. Of stone (kg/m3)	Water absorption (%)
1	163,8	166,3	105,1	60,4	2.669	1,53
2	123,4	126,2	78,7	46,5	2.590	2,27
3	272,9	283,8	168,5	113,8	2.360	3,99
4	201	204,1	127,8	76,1	2.627	1,54
5	204,4	211,2	129,7	80	2.501	3,33
6	149,1	156,4	92,9	62,1	2.341	4,90
7	201,3	205	126,5	77,7	2.557	1,84
8	105,6	108,9	66,7	41,6	2.495	3,13
9	116,4	118,9	73,7	44,7	2.568	2,15
10	233,4	237	149,3	87,9	2.654	1,54
$\bar{x}$					2.536	2,62
S_n					113	1,17

Visually porous-dense rock at 25°C

Specimen no	Dry mass	Mass after soaking	Mass submerged	Volume	Dens. Of stone (kg/m3)	Water absorption (%)
1	138,7	143,8	83,2	60	2.282	3,68
2	90,5	97,8	56,2	41	2.169	8,07
3	185,8	194,8	116,1	78,2	2.354	4,84
4	106,7	109,3	63,8	44,6	2.338	2,44
5	120,3	125,4	74,5	50,1	2.357	4,24
6	109,1	114,7	65,8	48,2	2.225	5,13
7	125	132,1	76,5	53,3	2.242	5,68
8	90,9	94,4	55,5	38,9	2.330	3,85
9	98,6	105,1	58,5	44,4	2.110	6,59
10	179,3	185,6	107,5	77,1	2.289	3,51
$\bar{x}$					2.269	4,80
S_n					83	1,65

NOTE: Samples were collected at the quarry face after blasting.
Quarrying may therefore contain a varying mix of these rock types.
Proportions difficult/impossible to estimate.

Figure 5. Density and water absorption measurements from the Ruperts Bay quarry.