Wellington Airport Runway Extension
Surf Break Impact Assessment

Numerical Modelling, Preliminary Mitigation Investigations and Feasibility Study

Draft Report

October 2015

The expert in **WATER ENVIRONMENTS**

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Wellington International Airport Ltd

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Source: Hamish Waterhouse Photography

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CONTENTS

FIGURES

TABLES

APPENDICES

1 Executive Summary

Wellington International Airport Limited (WIAL) commissioned DHI Water and Environment Ltd (DHI) to carry out a numerical modelling study to assess the impact of a proposed airport runway extension on the wave climate, swimmer safety and surf quality of the surf breaks at Lyall Bay. It was also requested that DHI provide recommendations for mitigation against any negative effects and methodologies for monitoring to confirm any potential effects of the works and/or success of mitigation features.

The numerical modelling assessment framework utilized the non-linear 2D Boussinesq Wave Model (MIKE 21 BW) coupled with DHIs surfing amenity analysis model, OPTISURF, to provide a detailed assessment of potential changes to surfing amenity caused by the airport runway extension.

Surfing amenity impact investigations were subdivided into the three inshore surf spots 'The Corner', 'Middle Beach' and 'Western Beach'. An additional surf spot called "Airport Rights" is located in front of the existing runway within the extent of the proposed extension.

It was found that the airport runway extension would not cause a noticeable sheltering or reduction in wave height along the three inshore surf spots.

However, the design of the steep rock revetment, which encloses the airport extension, is expected to reduce wave refraction to the east which will reduce the overall 'peakiness' of waves propagating further into the bay.

In the absence of nearshore rocky outcrops or reefs the surf amenity at the Western and Middle beaches are, to a large degree, dependent on the peakiness of the incoming wave field in order to generate surfable waves. With the reduction in wave peakiness caused by the proposed airport runway extension, the reduction in surf rides is expected to be between 14-29% for Middle Beach and 18-27% at West Beach.

At the surf spot called 'The Corner' the wave quality is primarily governed by the diffracted wave field. The diffracted wave field extends from the rock formation beneath the spur breakwater, creating a favourable angle between incoming waves and the local bed contours. The reduction in wave peakiness affects this spot, with an expected reduction of total number of surf rides of 4-8%.

For all three surf spots, negative impacts are expected to be the largest for long period swell. The surfing amenity provided at Airport Rights will be lost.

Changes in the location and magnitude of the circulation current near The Corner may cause moderate changes to local bed bathymetry. In nearshore areas of Lyall Bay the overall small differences in significant wave heights and change in wave induced currents poses negligible impact to changes to swimming safety.

Based on the information available to DHI at this stage of the project, and subject to further investigations, it is considered that a submerged wave focusing structure may be the safest and most cost effective approach for mitigating and potentially further enhancing surfing amenity in Lyall Bay. A strictly preliminary design of a wave focusing structure was developed and assessed to demonstrate the feasibility of a wave focusing structure for Lyall Bay. The model outputs indicate that the wave focusing structure would not only mitigate against the impacts of the airport extension but also has the potential to enhance surfing amenity for Lyall Bay. Further detailed investigations using state-of-the-art methods would need to be undertaken to ensure that wave breaking patterns are in line with project performance objectives and without causing adverse shoreline impacts.

DHI recommends that a field monitoring campaign is carried out, that can measure and confirm the findings of this study or future investigations. The monitoring campaign methodology should be consistent for the approaches used to assess conditions pre and post construction and should document changes in local wave field, nearshore bathymetry and surfing amenity.

2 Introduction

WIAL have commissioned DHI to carry out a numerical modelling study to assess the impact of a proposed airport runway extension on the wave climate, swimmer safety and surf quality of the surf breaks at Lyall Bay. It was also requested that DHI provide recommendations for mitigating against any potential negative effects and to provide a monitoring approach to confirm the effects of the works and/or success of the mitigation options. It was requested that feasibility assessment was undertaken for the most suitable mitigation solution if mitigation was required.

2.1 Description of Study Area and Study Background

The suburb of Lyall Bay is located in the southern part of Wellington, New Zealand close to the centre of the city. The southern extent of the suburb is fringed by a bay (also called Lyall Bay). Located to the east of the bay, in the suburb of Rongotai, is Wellington Airport, which officially opened in 1959 with a single runway of 1945 m in length.

WIAL propose to extend the current airport runway further south into Lyall Bay to accommodate Code E Aircraft and the occasional Code F (particularly A380) Aircraft. It is proposed that the runway will be extended to 2,300 m.

Lyall Bay is important for the Wellington region, as it is close to the city and also contains a quality surf break, called 'The Corner'. The Corner is located in the east of Lyall Bay, close to the breakwater built for the current runway. The Corner is considered a very good, reasonably consistent, left hander for intermediate level surfers which breaks on the sand banks that have built up along the edge of the breakwater. It can produce a quick tube section in optimum conditions, followed by a steep workable wall. When conditions at The Corner are good it can get very crowded. Without the runway break wall, The Corner surf break would not exist in its current form (i.e. wave breaking would be more similar to the rest of the bay).

There is another surf break to the south of the runway called 'Airport Rights'. Airport Rights is an exposed reef break off the southern end of the runway which only breaks in very large swell and is for expert level surfers only. When the wave does break, it is typically a short ride, which normally ends in a powerful close out.

The rest of the beach is made up of average beach breaks which can be good during certain conditions, but without the consistency of The Corner. The breaks ranging from the Maranui end of the bay to the west, (close to The Corner), are very dependent on the variable sand banks within Lyall Bay, but are typically good for surfers of all levels. For this reason Lyall Bay is also considered a nursery surf break where people are able to learn to surf without the dangers that can be associated with surf breaks for surfers with more experience and advanced abilities.

The Lyall Bay surf breaks are indicated in **Figure 2-1**, while the proposed airport runway extension is presented in **Figure 2-2**.

It is understood there is a concern from the local surfing community about the possibility of the airport runway extension impacting on the quality of the surf breaks. Any adverse effects on the surf break (especially The Corner) could be considered a major loss to the surfing community and even a small perceived impact on the breaks could be unwelcome. Also of concern is the potential impact on any currents (or rips) within Lyall Bay due to the airport runway extension and the implications of this for swimmer safety.

The National Institute of Water and Atmospheric Research (NIWA) were engaged by WIAL, to carry out an assessment for the airport runway extension and the potential impact to coastal processes in Lyall Bay. This focused on the impacts of the airport runway extension with regard to both hydrodynamic and sediment transport processes within Lyall Bay. Some of the main findings from this work have been used as the basis of the DHI assessment. Inputs (i.e. bathymetry and wave data) were provided by NIWA from their assessment.

Figure 2-1 Lyall Bay surf breaks - The Corner, Airport Rights and Western and Middle beaches.

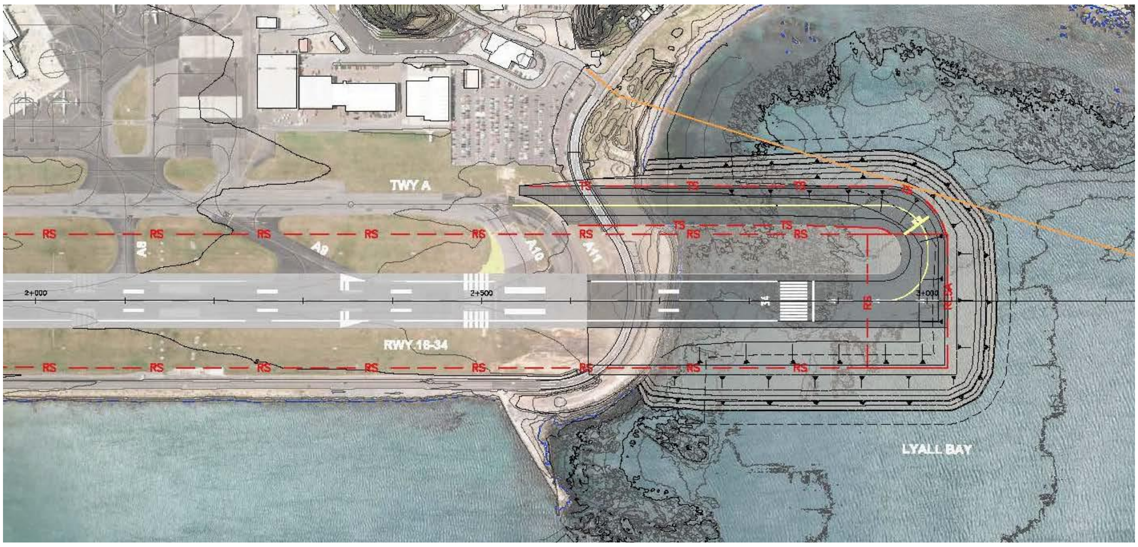

Figure 2-2 Proposed airport runway extension (Source: AECOM – South Runway Extension – Plan and Profile).

2.2 DHI Profile and Relevant Expertise

DHI is an independent, not for profit, research, technology and consulting organisation established in Denmark and today represented in all regions of the world. Our objectives are simple: to advance technological development, governance and competence within the fields of water, environment and health. In 2004 DHI opened an office at the business enterprise centre at Massey University Auckland and we now have offices in Wellington and Christchurch.

DHI have completed a number of projects involving investigation of surfing amenity and concept design of surfable structures as a key project consideration. Three of DHI's previous projects that have addressed surfing amenity are outlined below.

2.2.1 Palm Beach Shore Project Concept Design (Australia)

Project Aim

Palm Beach has been identified as being one of the Gold Coast beaches currently under the greatest risk with regards to loss and damage to beach front properties due to beach erosion during significant wave events. In 2013/14 DHI were commissioned by City of Gold Coast to assess the effectiveness of three proposed erosion control measures (DHI, 2014). An option of beach nourishment combined with a submerged control structure (SCS) was selected for the concept design phase of the study. In addition to providing shoreline protection two key design requirements for the SCS included 1) no negative impact on the surf amenity at Palm Beach, or 2) improve the surf amenity at Palm Beach.

Methodology

The development of a new type of SCS structure was completed by DHI as part of this project. The impact of the SCS on surfing amenity at Palm Beach was investigated using MIKE21 BW Boussinesq wave modelling and DHI's surfing amenity program OPTISURF. The non-linear Boussinesq wave model MIKE21 BW was initially used to calculate the detailed changes in wave transformation processes and wave breaking processes occurring along Palm Beach due to the presence of the SCS. Based on long term records of nearshore wave conditions four representative scenarios were selected to undergo detailed surfing amenity impacted investigations by comparing the SCS layout versus the (no SCS) baseline. The wave boundary condition for each event was modelled as a directional wave spectrum using spectral wave parameters and directional standard deviation from the spectral wave model results.

The program OPTISURF was used to calculate key surfing amenity parameters from the calculated wave field. OPTISURF utilises the instantaneous wave field output to track the moving transition point between unbroken and broken waves. Each of these points is referred to as a pocket point and marks the zone adjacent to where the wave is breaking, which marks the optimum position for a surfer during a wave ride.

For each time step, each pocket point was tracked back to its previous position if it exists. The vector length between the two points describes the minimum average speed the surfer would have had to maintain in order to keep surfing the wave. If the average speed exceeds 10m/s, the particular section of the wave is considered nonsurfable and the surf ride is terminated. Subsequently a new ride is initialised for the new pocket point.

Using OPTISURF, the following surfing amenity parameters were evaluated over the course of one hour of wave action respectively for the baseline case and for the SCS design:

- Number of surfable waves
- Maximum length of each surf ride
- Maximum wave height of each ride

Outcomes

A comparison of the spatial distribution of the rides and associated wave height with and without the SCS ‘design is presented in **Figure 2-3**. Based on the surfing impact assessment study it was concluded that the SCS is expected to produce surfable rides of 50 – 70 m length on a regular basis occasionally exceeding 90 m. Surfable wave heights are expected to be substantially larger at the SCS compared to elsewhere, which could potentially increase consistency of surfable conditions.

The SCS was found to produce an approximate 300 m shadow zone in its lee with substantially smaller wave heights. OPTISURF largely predicted an increase in surfable waves when the SCS is present for the scenarios. The SCS will introduce a diffracted wave field in the shadow zone which could potentially introduce new smaller surfable wave peaks. The wave height of these breaks will be significantly smaller compared to outside of the shadow zone.

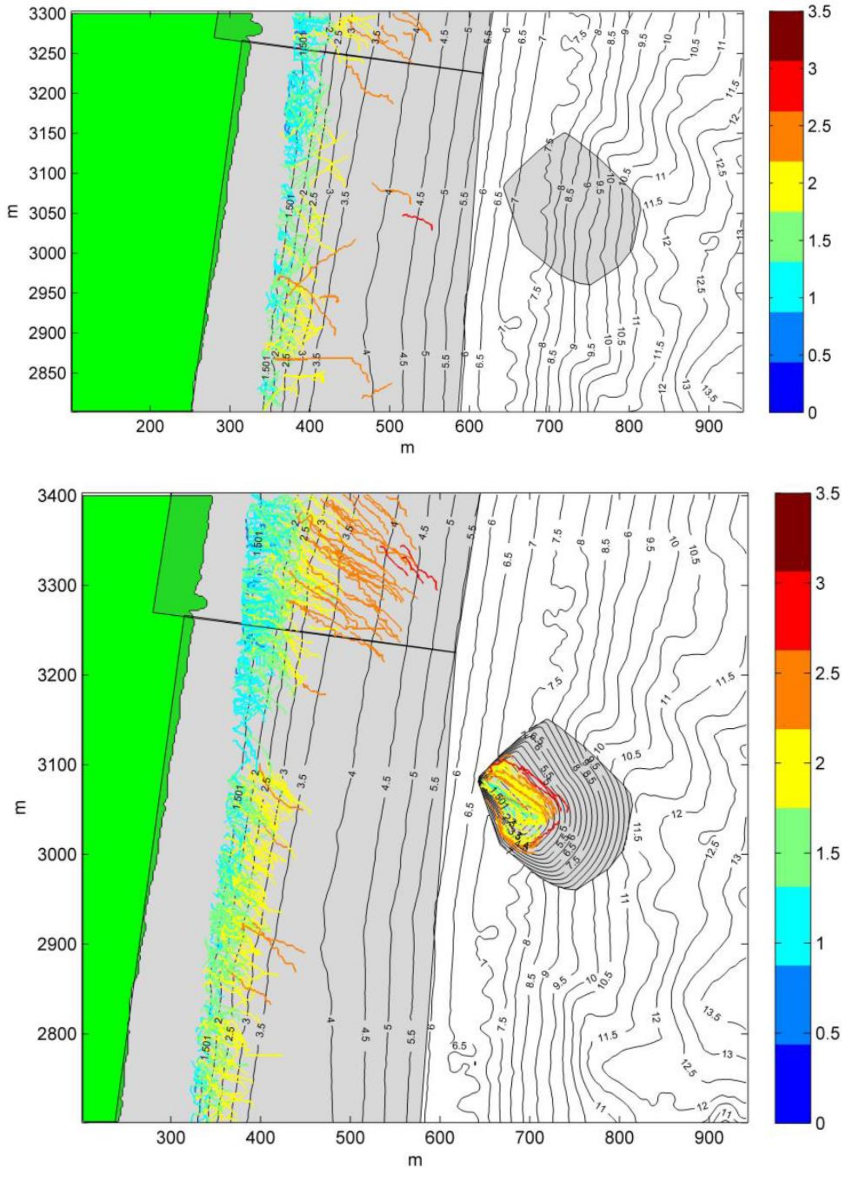

Figure 2-3 Spatial distribution of the surfable wave sections without SCS (top) and with the SCS (bottom).

2.2.2 Whakatane Coastal Processes Study (New Zealand)

Project Aim

DHI were commissioned by Whakatane District Council to investigate the effectiveness of a range of options to achieve the best result in terms of improved navigation through the Whakatane Harbour Entrance while taking into consideration other recreational and environmental constraints, of which surf amenity at the river mouth was one (DHI, 2011).

Methodology

MIKE21 BW Boussinesq wave modelling was used to calculate the detailed wave transformation and breaking for two selected offshore wave condition scenarios. Scenario 1 included a highly energetic condition while scenario 2 comprised of the most frequently occurring wave condition. OPTISURF was then used to analyse the surf quality of the produced wave field.

For each breaking wave the transition point between broken and unbroken wave was traced until the wave breaking terminated. Each cycle was logged as a surfable ride along with information on the ride length, peeling speed and breaking wave height.

Outcomes

An example of the output from OPTISURF for the river mouth is presented in **Figure 2-4.** The results revealed key features of the surfing break such as the take-off zone in proximity to the navigation channel. Both scenarios were found to provide ride lengths of over 400m. Variations in ride return periods were distinguished for each scenario, being of 1 ride in every 5.5 minutes for scenario 1 and 12 minutes for scenario 2.

The analysis highlighted a strong link between the orientation, length and continuation of the river mouth bar and the quality of the surfing amenity foremost expressed in terms of the potential length of the surf ride. The project is currently on hold with a detailed assessment of potential options awaiting further approval. Further stages of surfability assessment would involve the proposed navigation works on changes to the bathymetry, wave direction and changes in the wave driven currents, requiring more scenarios to be considered and assessed.

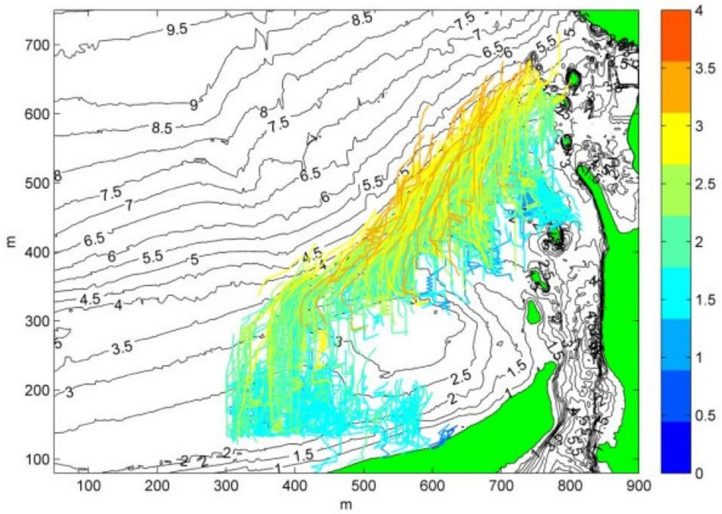

Figure 2-4 OPTISURF Illustration of the length and size of all possible surf rides over the course of 1 hour – Whakatane (NZ) The colour scale indicates the maximum breaking height experienced along the ride.

2.2.3 Hanstholm Port Expansion (Denmark)

Project Aim

DHI were commissioned by Grontmij A/S to investigate the potential impact to surfing, wind surfing and kite surfing amenity at five surf spots caused by a proposed expansion of Port of Hanstholm, Denmark (DHI, 2012). Due to the significant seaward extension of the port, it was expected that the nearby surf spots could experience a reduction in wave heights due to the increased sheltering during some wave conditions.

Methodology

Based on a preliminary screening study two wave scenarios were selected to represent the expected reduction in surfing amenity following predicted wave height reduction in the lee of the proposed port. Surfing amenity assessment for the two representative surf events was undertaken using the numerical models MIKE 21 BW and OPTISURF.

Outcomes

Results from OPTISURF show a reduction in potential ride height and length of surfable waves (see **Figure 2-5**). OPTISURF enabled the quantification of the reduced surfability to be documented at individual sections of the coast, representative of the various surf breaks. The reduction of the number of high quality surfable waves at the break directly in the lee of the structure was found to be significant and almost all surfable rides with wave heights of more than 1.5m and ride lengths of more than 200m were predicted to be eliminated for the scenarios tested.

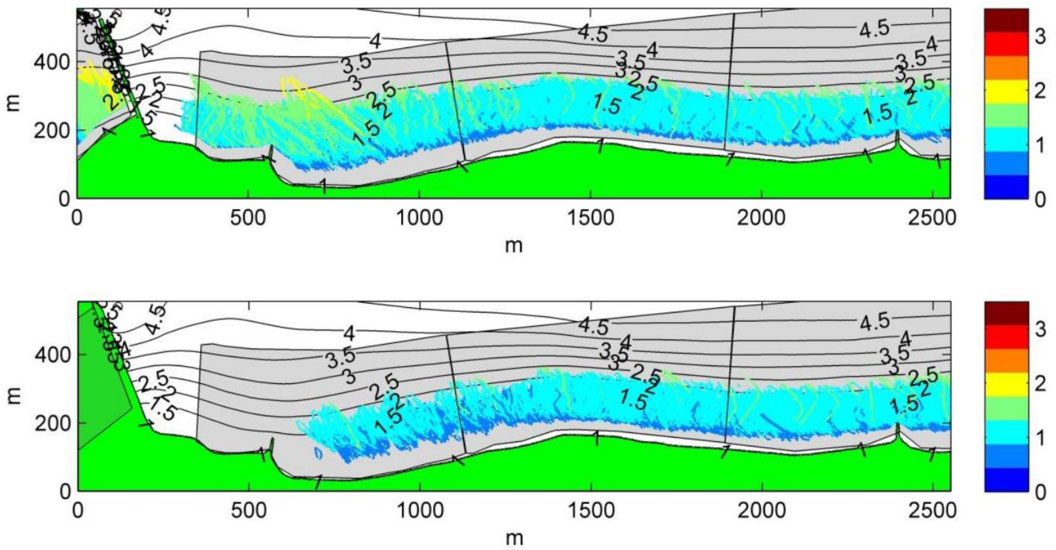

Figure 2-5 OPTISURF wave height and track distribution from the current layout (top) and future layout (bottom) inclusive of the proposed port, over a selected scenario simulation period of one hr.

2.3 Study Objectives and Overview of Methodology

The main objectives of this study (and how these were accomplished) were as follows:

- Review of NIWA's coastal hydrodynamics and sediment processes impact assessment report and inputs provided by NIWA for this study.

  A review was undertaken of the report and inputs provided by NIWA to identify key assumptions and findings of the NIWA work and implications for this study and to confirm the quality of any inputs provided to DHI.

- Generation of a long term (one year) wave data set for Lyall Bay.

  This was achieved by developing a wave model of the southern Wellington coastline (including Baring Head) and then using the wave model to transform one year of wave data from Baring Head to a selected location within Lyall Bay.

- Assess the effects of the works on Lyall Bay, with emphasis on the following:

  - Effects on the wave climate; and
  - Effects on swimmer safety of the beach (from changes to wave driven currents);
  - Effects on the surf break quality.

  This was achieved by developing a detailed wave model of Lyall Bay. For selected wave events the wave climate (phase averaged significant wave height) was compared with and without the airport extension. Three scenarios were then identified from historical photographic evidence and generated wave data for Lyall Bay that were deemed to capture a wide range of various surfing conditions. These selected scenarios were simulated with the detailed wave model to predict changes to wave induced currents with the airport extension and any implications for swimmer safety. The impacts to surfable rides with the airport extension were also predicted for three scenarios for the west beach, middle beach and The Corner to assess effects to the surf break quality.

- Provide recommendations for avoiding, remedying or mitigating any predicted effects of the works.

  Four mitigation options were considered to mitigate any potential impacts of the airport extension on surf break quality for Lyall Bay. This including identifying the advantages and disadvantages associated with each option. A preliminary feasibility assessment for the preferred option was then undertaken.

- Recommend methodologies for a monitoring plan to confirm predicted effects of works and/or success of mitigation features.

  A methodology has been proposed which could be used as a basis to fulfil any resource consent conditions required to confirm the findings of numerical investigations and prove the success of any proposed mitigation solutions. Additional data collection requirements for any detailed design of a mitigation option were also highlighted.

A suite of numerical models has been developed and applied for this study. **Table 2-1** presents an overview of all the numerical models that were used for this study. The models and modelling approach are discussed in more detail in Section 4.

Table 2-1 Outline of models applied in study.

Model	Purpose	Input Data
MIKE 21 SW Model	A wave model of southern Wellington coastline to transform Baring Head wave data to Lyall Bay and generate one year time series of wave data.	Lyall Bay bathymetry provided by NIWA. CMAP bathymetry data. Baring Head and Lyall Bay wave data provided by NIWA.
MIKE 21 BW Model	Detailed wave model of Lyall Bay for predicting how waves approach the shoreline and break within Lyall Bay for selected wave scenarios. Model also provides prediction of wave induced currents.	Lyall Bay bathymetry provided by NIWA. Selected wave scenarios.
OPTISURF	Tool for assessing possible surfable waves from outputs of detailed wave model.	Output from MIKE 21 BW.

3 Review of Previous Work, Provided Inputs and Resulting Project Limitations

This study was reliant on a number of assessments, assumptions and data provided by external parties. This section outlines the previous work and inputs that were provided to DHI to form the basis for our study.

3.1 Previous Work

DHI have reviewed the following reports to gain an understanding of previous studies undertaken in the study area, which are applicable for this phase of work associated with the Wellington Airport Runway Extension:

- NIWA (2015) *Wellington Airport Runway Extension, Coastal Hydrodynamics and Sediment Processes (Lyall Bay)*, Prepared for Wellington International Airport Ltd, Draft, February 2015.
- NIWA (2014) *Evans and Lyall Bay Bathymetry Report*, Prepared for Wellington International Airport Ltd, January 2014.

3.1.1 Sediment Transport Modelling

NIWA assessed changes to hydrodynamic and sediment transport behaviour within Lyall Bay with the airport runway extension, using Deltares Delft3D numerical modelling suite (NIWA, 2015).

The hydrodynamics were simulated using a combination of Delft3D-FLOW and Delft3D-WAVE. The hydrodynamic model was calibrated with measured tidal levels and currents.

Both data and model predictions indicate that tidal currents within Lyall Bay are exceptionally weak and dominated by wind or wave driven currents. From the assessment of the predicted changes to current behaviour within Lyall Bay, NIWA predict that the proposed airport runway extension is likely to have only a minor localised effect on the residual tide/wind circulation within the middle section of Lyall Bay and a negligible effect on net circulation in the nearshore area off Lyall Bay Beach. For wind driven circulation, NIWA predict that the circulation patterns driven by the northerly wind will remain largely unaffected by runway construction. An impact is predicted for the southerly wind, however NIWA conclude this is unlikely to affect morphological change or flushing of the inner part of Lyall bay.

The effects of the airport runway extension on sediment transport behaviour were investigated using a combination of Delft3D-SED and Delft3D-MOR modules. An eight week period was simulated which contained at least one condition (a strong southerly gale of 22 m/s) which would promote significant sediment transport within Lyall Bay. For this event for both the existing situation and with the airport runway extension, it was predicted there was significant erosion inshore for Lyall Bay with significant deposition offshore, behaviour which is consistent with a previous study for the site.

NIWA predicted that there will not be a significant net change to sea bed bathymetry, following the construction of the airport runway extension, with only some localised changes in vicinity of the extended runway. This is a key assumption for this DHI study.

3.1.2 High Resolution Wave Modelling

In NIWA (2015) high resolution wave modelling was carried out using the phase-resolving wave model ARTEMIS by TELEMAC. ARTEMIS belongs to the family of wave models based on solving Berkhoff's version of the elliptic mild-slope equations. The model is originated in linear wave theory and is able to model the following processes:

- Bottom refraction,
- Diffraction by obstacles,
- Depth induced wave breaking,
- Bottom friction, and
- Full or partial reflections against structures.

The ARTEMIS wave model accounts for irregular sea states by superposition of monochromatic waves and wave breaking is handled separately for each wave frequency. Despite belonging to an older breed of wave models, this type of wave models is well suited to provide a fast first level investigation into potential wave penetration processes into sheltered areas such as Lyall Bay.

However in order to assess impacts on surfing amenity the wave model is not suitable for modelling the breaking wave field of a real sea state to a level that is required. The model is also not able to resolve wave induced circulation currents which affects the nearshore wave field and is important to assessing swimming safety.

For comparison, a DHIs non-linear Boussinesq wave model MIKE was run for the same incident wave as one of NIWA's Artemis simulations presented in NIWA 2015 (Section 5.7.2). The scenario considered an offshore wave height of 1.5m s with a 12s period. It is not clear if the input wave conditions refers to a monochromatic wave input or a superimposed spectrum where values refer to Hs and Tp. Wave direction in the Artemis simulation was not provided but based on illustrations appears to be from due 180° south. Based on same illustrations it appears that incident waves were uni-directional without any directional spreading.

Wave boundary conditions in the MIKE21 BW model assumed a JONSWAP spectrum with a significant wave height of 1.5 m and spectral peak period of 12 seconds. The directional spreading index was set to 16 to represent a typical swell condition. A detailed description of the setup of the MIKE21 BW model is presented in Section 4.1. A comparison between the two models is provided in **Figure 3-1** and **Figure 3-2**.

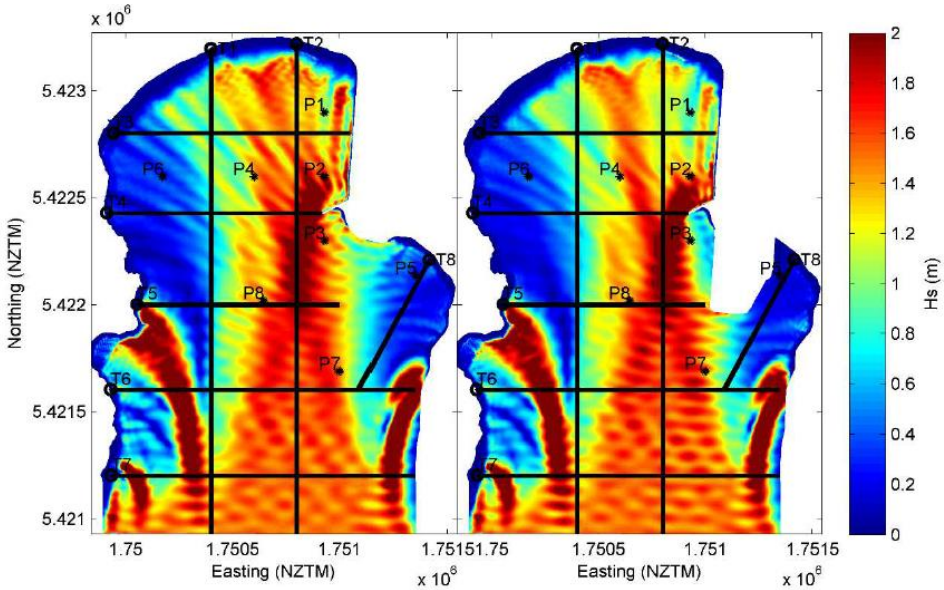

Figure 3-1 Artemis Simulation, Significant Wave Height Plot (NIWA, 2015).

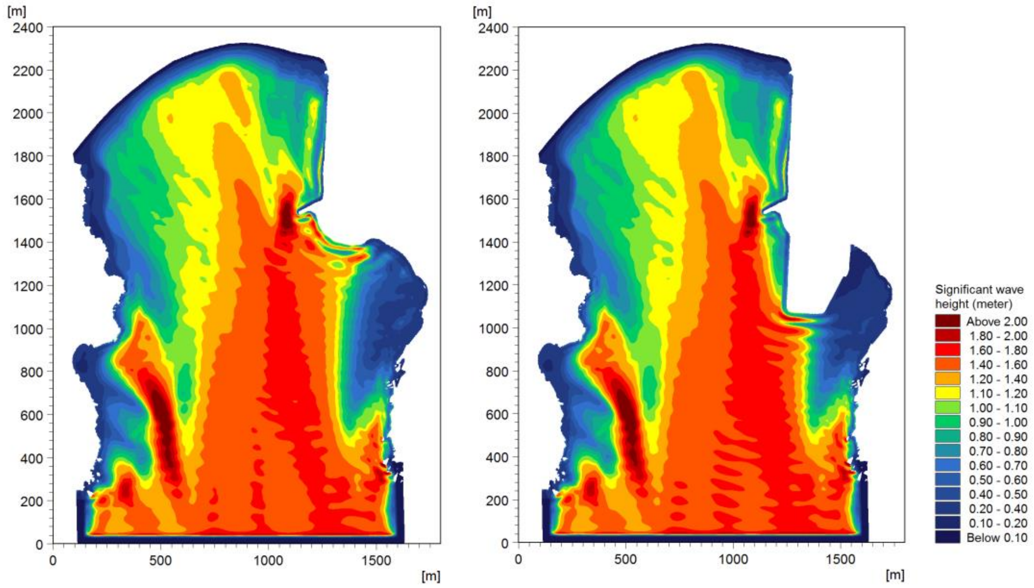

Figure 3-2 MIKE21 BW Simulation, Significant Wave Height Plot.

From **Figure 3-1** and **Figure 3-2** it can be observed that the general agreement between the two models is fair. Both models predict significant wave refraction to occur along both the western and eastern sides of the Bay. The models also agree on the overall location of localized wave focusing zones. It appears however that MIKE21 BW predicts a much more gradual transition in wave height compared to ARTEMIS where gradients are very sharp. The most significant differences occur along the western side of the bay close to P6 in **Figure 3-1** where ARTEMIS predicts wave heights as low as 0.2 m in areas MIKE21 BW predicts 1 m. Without the possibility of validating model results with measured data it is not possible to proclaim which model is the more accurate, but it is DHIs expectations that primary discrepancies are caused due to limitations in the replication of a realistic sea state at the offshore boundary in ARTEMIS.

3.2 Provided Inputs

The following datasets have been provided to DHI as inputs for this study.

Wave Data

Wave data from three instruments deployed within the study area were provided to DHI by NIWA. These include data from Site 1 (located in the nearshore in close proximity to the runway), Site 2 (located off of the headland to the southeast of Waitaha Cove) and Site 3 (located approximately 1.5km offshore of Baring Head). An overview of the deployment locations is shown in **Figure 3-3** with the coordinates provided in **Table 3-1**.

There is some uncertainty in the accuracy of the wave directions collected at Site 1, where wave directions as low as 164 degrees were recorded. DHI believe that the Site 1 directions seem unlikely and believe there may have been issues with the ADCP compass or effects from the nearby steel structures or perhaps the mounting frame. We believe this may have resulted in a 10 -15 degree offset in recorded wave directions. This is investigated in more detail in Appendix A.

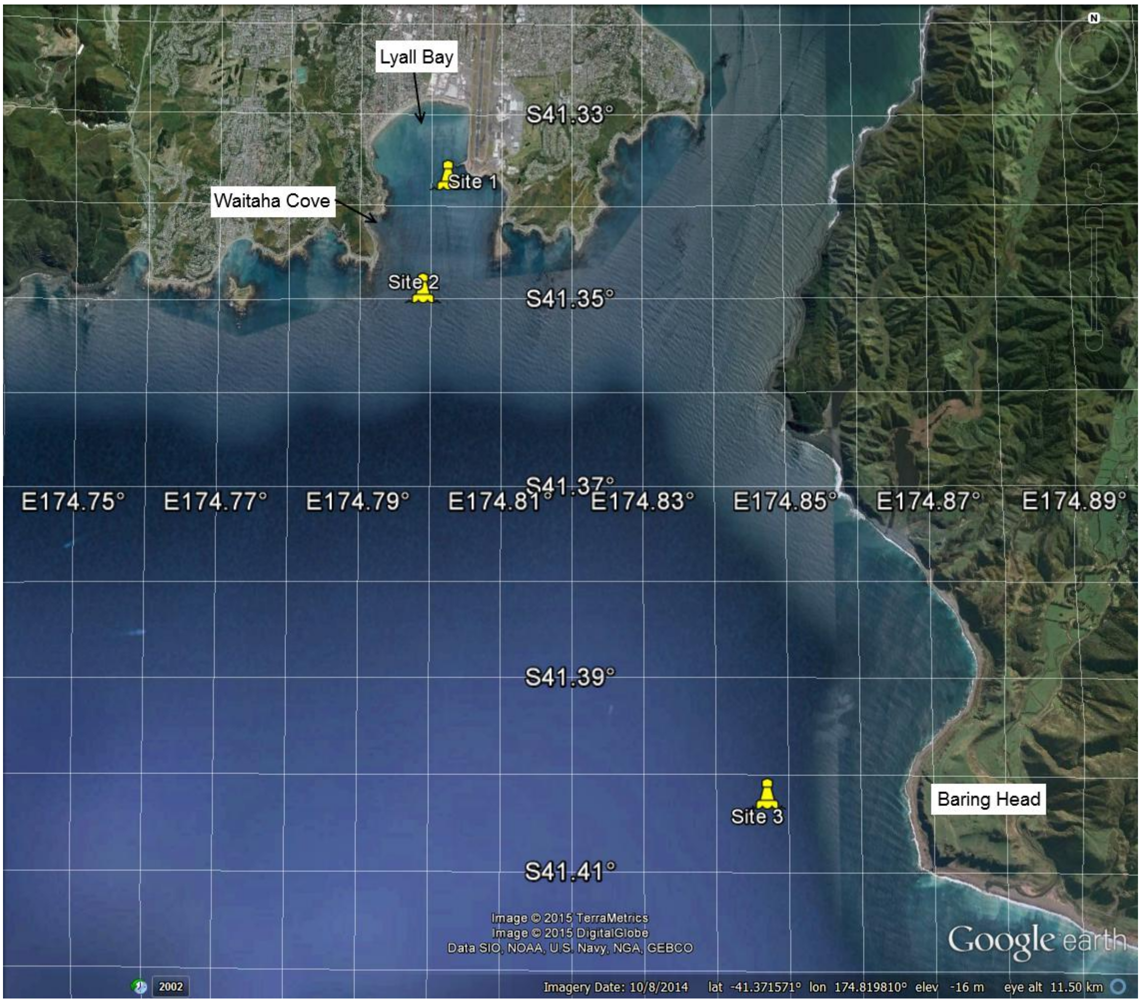

Figure 3-3 Locations of wave data.

Table 3-1 Locations of wave data used to transform waves from Baring Head.

Site No.	Instrument	Longitude	Latitude	Deployment Period
1	ADCP (nearshore)	174.8023	-41.3366	18-Aug – 09 Oct 2014
2	Dobie (offshore)	174.7989	-41.3489	04 Sept – 09 Oct 2014
3	Baring Head Waverider	174.8470	-41.4020	30 Jan 2014 – 18 Feb 2015 *

* For this study data from the waverider buoy at Baring Head has only been used for the period where direction was also recorded.

Bathymetry Data

Bathymetry data of Lyall Bay was supplied by NIWA, while the proposed airport extension layout was supplied by WIAL. The NIWA bathymetry set consisted of 25 m gridded bathymetry, shoreline LIDAR and 1 m gridded multi-beam bathymetry around the existing airport rock wall. An overview of the spatial coverage and resolution of the bathymetry coverage provided by NIWA is presented in **Figure 3-4**.

The 25 m gridded bathymetry was interpolated from multi-beam transect surveys with an average spacing of 25 m and 7 to 11 meters in near-shore areas. There is a reasonable gap in data sets between the gridded data and shoreline data within the near shore zone. Small bar features within the nearshore which may improve surf quality (especially during times of small wave heights) cannot be resolved with this data set. This is discussed further in Section 7.

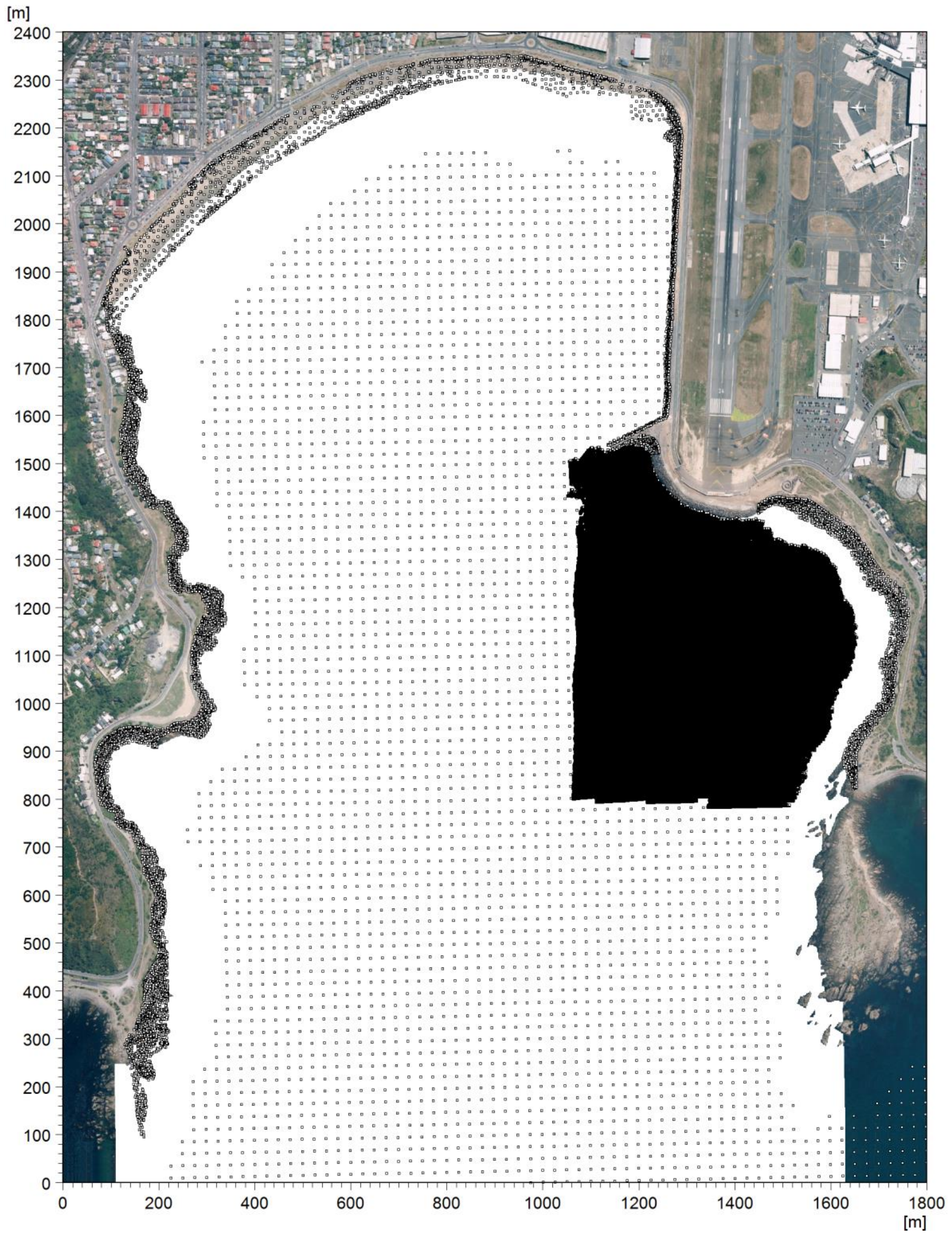

Figure 3-4 Provided Lyall Bay raw bathymetry.

Wind Data

Wind data from Wellington Airport was provided by MetService for the period 1st January 2013 to 1st March 2015. This data is presented in **Figure 3-5** and indicates predominant northerly and southerly winds.

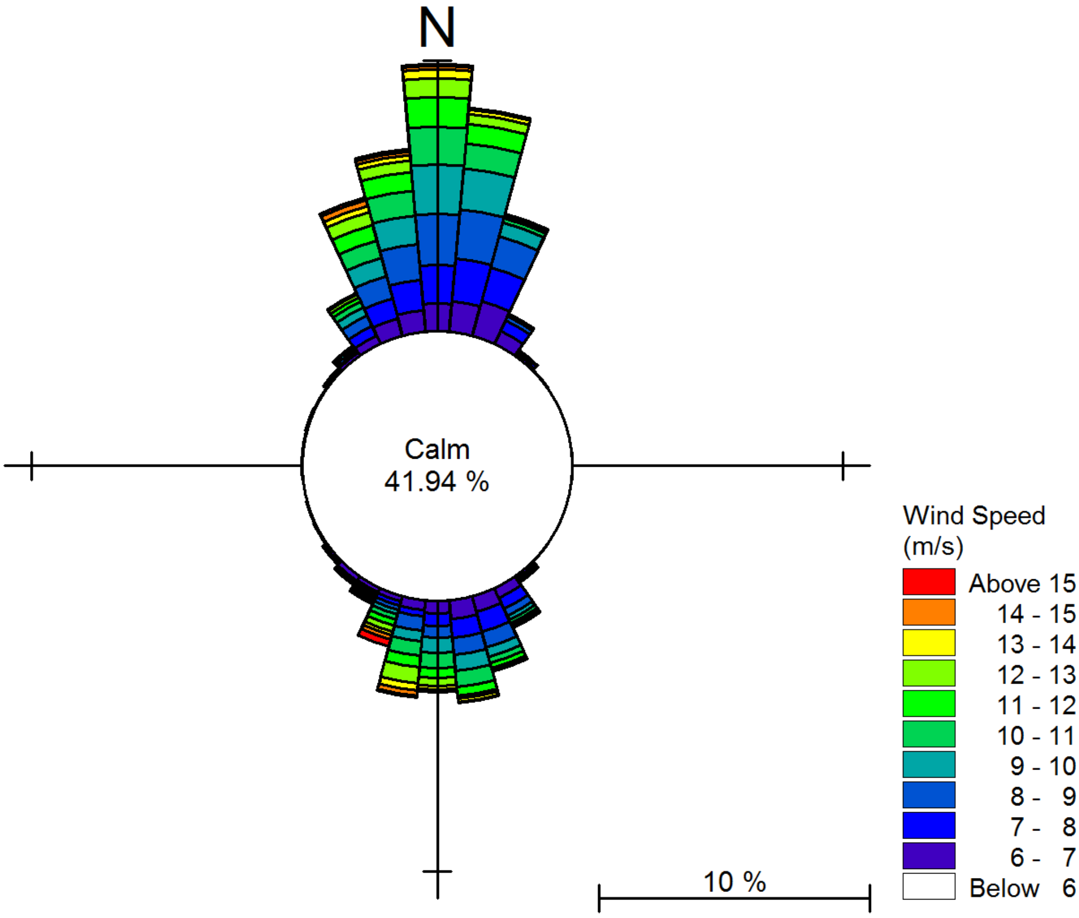

Figure 3-5 Wind Rose for Wellington International Airport (01/01/2013 – 01/03/2015).

Evidence of Recent Surf Events

Photographic evidence of 13 recent reasonable quality surf events was supplied by WIAL through a request to the public on Wellington Boardrider's Facebook page. Only events that occurred within the period 30th January 2014 to 18th February 2015 were considered for this study because this is the only period where wave direction data was available from the waverider buoy at Baring Head.

3.3 Project Limitations

Due to the reliance on a number of assessments, assumptions and data provided by external parties, there are a number of limitations associated with the present study. These are as follows:

- No details were provided for the calibration of the wave component of the Lyall Bay hydrodynamic model developed by NIWA.
- Offshore wave conditions at the entrance of Lyall Bay are calculated using a wave transformation method that relies on the accuracy of measured wave data provided by NIWA. No information about detailed wave spectra (frequency and directional distribution) was provided.
- Nearshore wave transformations and accuracy of surfing amenity predictions depends significantly on the accuracy and resolution of the bathymetric survey data provided by NIWA. The resolution of the nearshore areas in most places is only 25 x 25 m and no coverage exists for the nearshore surf zone within Lyall Bay.
- The assessment of impacts due to the proposed airport runway extension relies on NIWAs conclusion in recent studies that ‘negligible impacts to nearshore sediment transport processes and bed morphology are expected’. No calibration of the sediment transport models used to make this conclusion was undertaken due to the cost involved with collecting appropriate data for calibration.
- WIAL has provided a historic record of dates for recent surf events in Lyall Bay that were considered good quality with assistance from the local board riders. Determination of lower bounds for surfing reoccurrence, statistical analysis and the selection of scenarios has been reliant on the accuracy of these records.
- Quantification of surfing amenity in this study has been targeted to address surfers across all skill levels and is based only on reoccurrence frequency, wave face height and length of ride. Distinctions between challenging fast breaking hollow waves and slow mellow waves was considered outside the scope of the current study and subject to further detailed investigations.

4 Modelling Assessment Framework

This section outlines the tools that were used to assess the effects of the proposed airport runway extension on wave climate, swimmer safety and surf quality within Lyall Bay.

4.1 Boussinesq Wave Modelling

The modelling assessment framework utilizes the non-linear 2D Boussinesq Wave Model MIKE21 BW. MIKE21 BW is one of the most advanced wave models currently available in the industry and is ideally suited to the calculation of wave transformations and breaking in nearshore coastal areas.

The study was carried out using Wellington Vertical Datum and New Zealand Transverse Mercator coordinate system. The computational domain was discretized by a Cartesian grid aligned with true north, with an extent of 1800 m by 2400 m and a horizontal grid spacing of 1.5 meters.

The model bathymetry was generated from raw data sets provided by NIWA. Hue te Taka Peninsula (Moa Point) was geo-rectified from Google Earth and interpolation was required along the shoreline of Lyall Bay due to nearshore gaps of the provided data. The Boussinesq model domain with its interpolated bathymetry is shown in **Figure 4-1**. Bathymetry for the airport extension was generated from data in a dxf file provided by NIWA. The bathymetry for the Boussinesq model domain including the airport extension is shown in **Figure 4-2**.

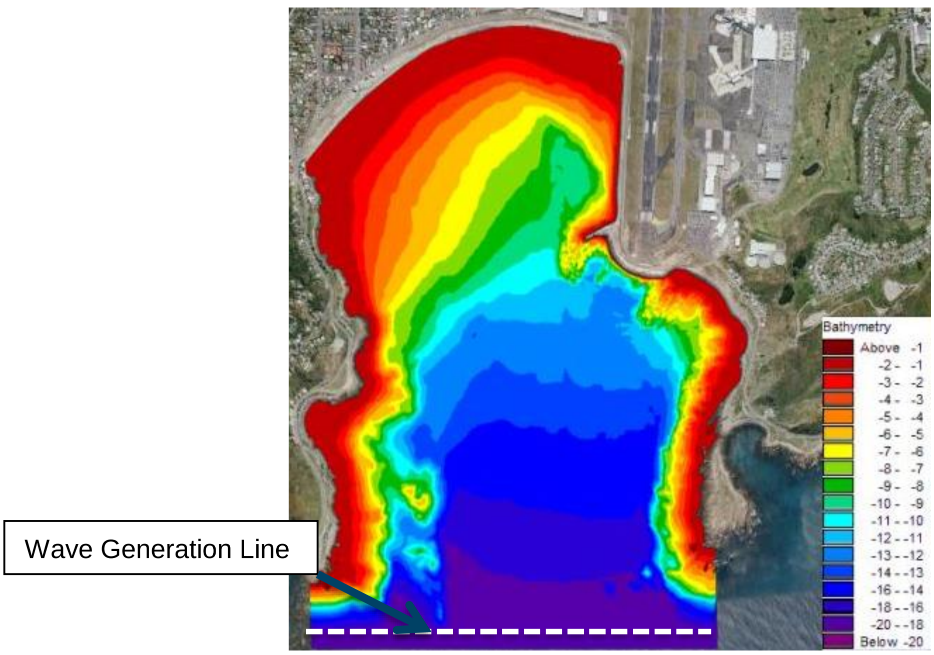

Figure 4-1 Boussinesq Model Existing Bathymetry.

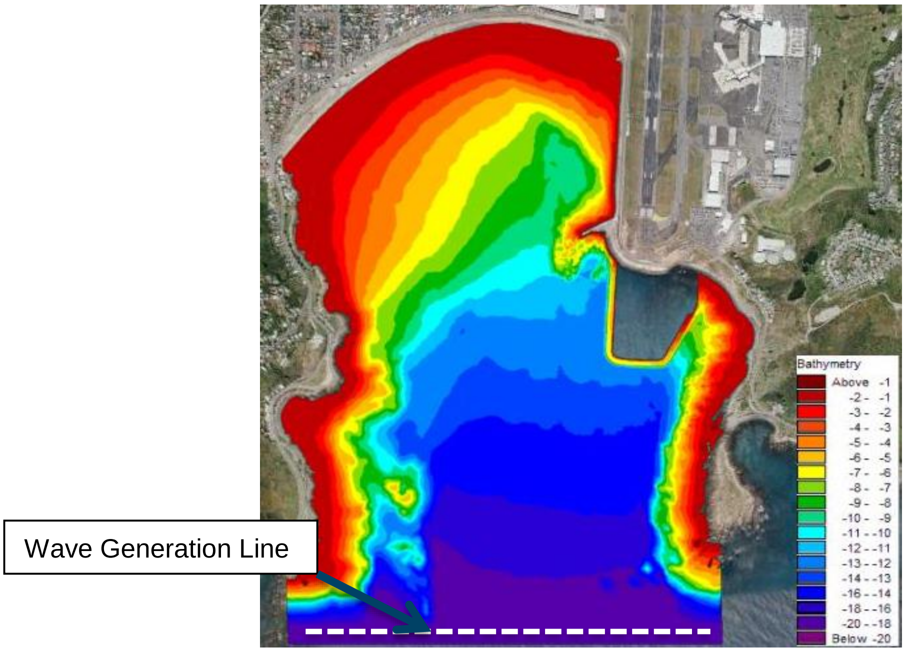

Figure 4-2 Boussinesq Model Airport Extension Bathymetry.

The simulation period for each scenario was set to 40 minutes, in order to allow for adequate time windows for spectral and OPTISURF analysis, while also allowing sufficient time for model warm up period (approximately 20 minutes).

Each wave boundary condition was applied as an internal wave generation line close to the southern boundary and was represented by a fully directional and irregular sea-state adopting a standard JONSWAP spectrum. A second order correction scheme was applied to the incident wave train to prevent the release of spurious harmonics in the weakly non-linear wave conditions at the boundary. A $\cos^n$ directional spreading distribution with n=16 was used in order to mimic the limited directional spreading of typical swell events.

The maximum water depth was set to 19.7 m to assure a constant water depth along the wave generation line. A sponge layer was positioned along non-reflective boundaries to prevent gradual build-up of wave energy in the domain. The minimum water depth along beaches and shallow areas was set to 1m below MSL followed by a thin sponge to prevent wave refection.

A moving shoreline (run-up) is an option in MIKE21 BW, however this was not included in this study, due to time and budget constraints. The resulting impact on nearshore wave dynamics is considered to be very small.

4.2 Offshore Wave Transformation

A long term dataset of wave height, period and direction at the entrance to Lyall Bay was required in order to determine representative wave characteristics for Lyall Bay and provide realistic wave boundary conditions for the Boussinesq wave model.

From the recent wave data collation exercise undertaken by NIWA, one month and two month data sets were available from Lyall Bay. This study required at least a one year wave dataset at the entrance of the bay. Therefore a wave transformation study was completed to provide directional wave data at the entrance of Lyall Bay.

The wave transformation modelling was undertaken using our MIKE 21 Spectral Wave (SW) Model FM which is a state-of-the-art numerical tool for prediction and analysis of wave climates in offshore and coastal areas. A detailed description of the Wave transformation process is presented in Appendix A.

4.3 OPTISURF Surfing Amenity Modelling

DHI's surfing analysis program OPTISURF was used to calculate key surfing amenity parameters based on the outputs from the Boussinesq wave model. OPTISURF utilises the instantaneous wave field output to track the moving steep transition zone between unbroken and broken waves, which most often marks the optimum position (surf term: 'the pocket') for a surfer during a wave ride. In the following sections this position is referred to as a pocket point.

For each time step the program keeps track of all active surf rides occurring in the domain. Note that some waves offer the possibility for multiple rides to be executed. This can include a simultaneous left and right hand ride created by a breaking wave peak. Other waves can break over multiple sections before reaching the shoreline.

For each active surf ride, the program keeps a track of the minimum speed the surfer will have to maintain in order to keep ahead of the breaking wave front. The program also logs time series of the wave face height and the wave steepness at the position of the surfer and the surfer's ground speed, see **Figure 4-3**. The maximum ground speed achievable depends on both the detailed wave breaking characteristic and the skill level of the surfer. Maximum surfer speeds of more than 10 m/s are uncommon. Yet some fast wave sections are still passable if the surfer can predict them in advance and move faster during prior wave sections.

If the maximum surf speed exceeds a predefined maximum, the particular section of the wave is considered to be closing out or breaking too fast to be surf able and the surf ride is terminated. Consequently, if possible, a new ride is initialised for subsequent wave sections. Due to the site specific nature of maximum surf speeds, it is often recommended to compare threshold values to site specific measurements and observations.

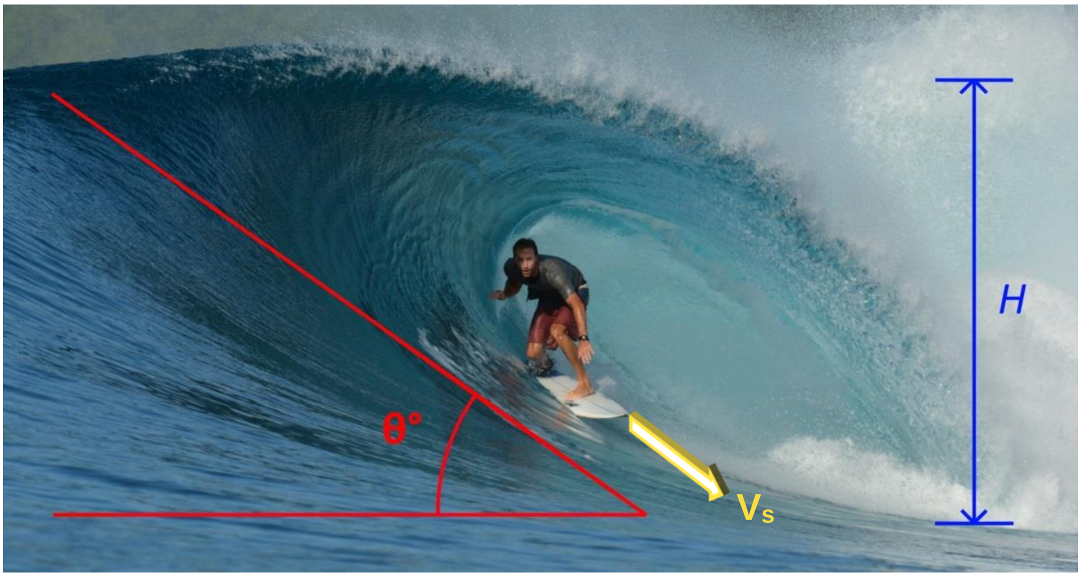

Figure 4-3 Plunging wave diagram with face height (H), wave steepness (θ) and surfer ground speed (V_s).

An example of a graphical output from a time step processed in OPTISURF is presented in **Figure 4-4**. The pocket points with corresponding wave face height are represented by the coloured squares and tracks from the previous few time steps are shown in black. Minimum limits for wave face height, length and steepness can be specified as filters in order to limit the number of visualised surf trajectories.

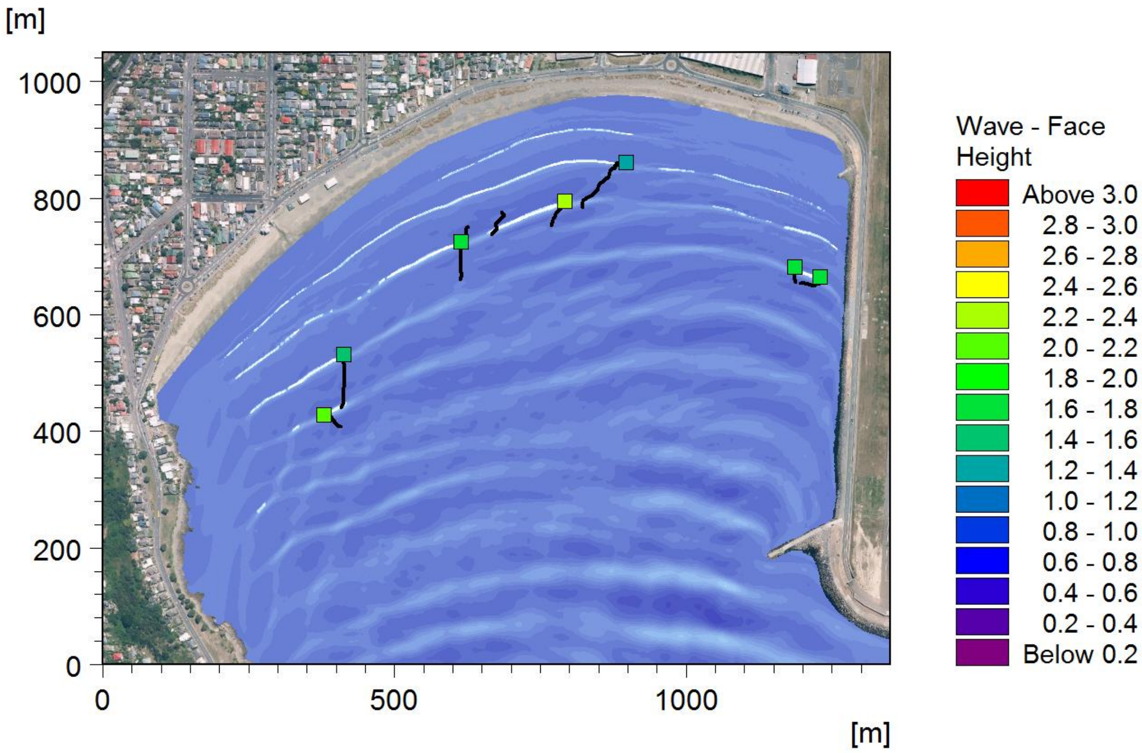

Figure 4-4 OPTISURF Animation Snapshot.

Using OPTISURF, the following surfing amenity parameters were evaluated over the course of 20 minutes of wave action respectively for the existing bathymetry and for the airport extension.

- Number of surf able waves
- Maximum length of each surf ride
- Maximum wave face height of each ride

Result statistics can be split up into multiple sub areas marking different surf spots. In this project, the study area was split up into three areas defined as ‘The Corner’, ‘Middle Beach’ and ‘West Beach’. These areas are overlaid on the existing bathymetry contours in **Figure 4-5**. The surf spot ‘Airport Rights’ was not included as it is located within the extent of the proposed extension.

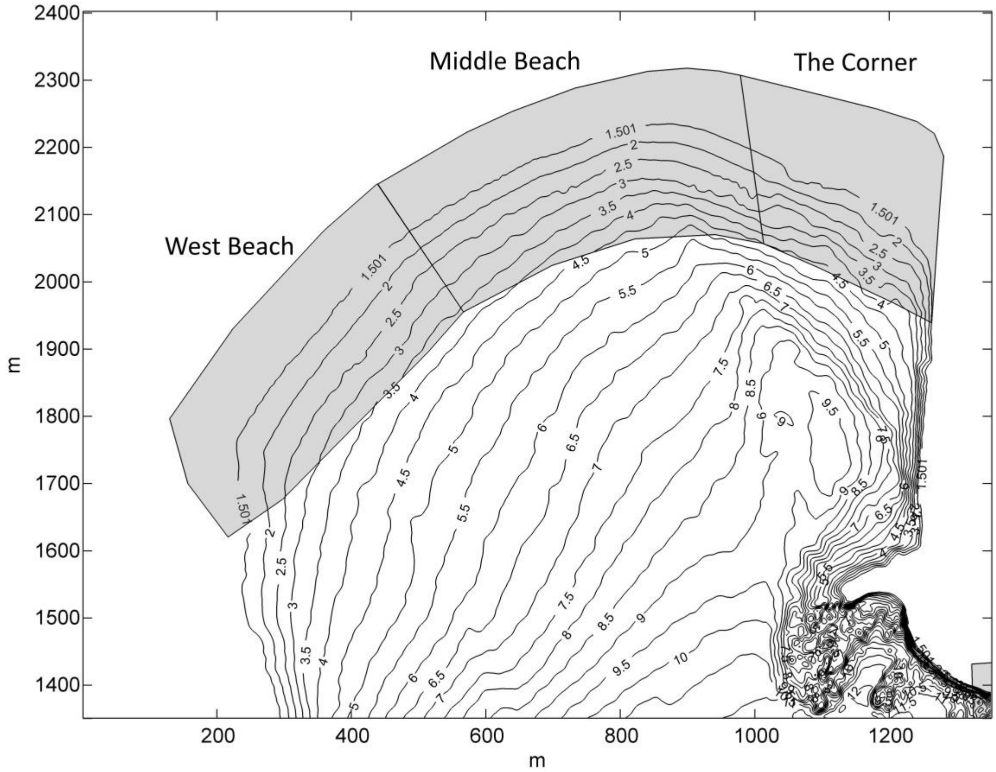

Figure 4-5 Defined Surf Spots in Lyall Bay.

5 Assessment of the Effects of the Works on Lyall Bay

The section outlines the assessment of the effects of the proposed airport runway extension on wave climate, swimmer safety and surf quality within Lyall Bay.

5.1 Scenario Selection

Wave conditions produced from the wave transformation study and corresponding wind measurements from Wellington Airport were used to provide an estimate the envelope of surfable wave conditions for Lyall Bay. Transformed wave data and historic wind data was available from 30th January 2014 to 17th February 2015. During this period, 13 photographed surf events of varying sizes and quality were made available by WIAL with assistance from local board riders. Each recorded scenario is presented in **Table 5-1**.

In order to provide a realistic quantification of the frequency and distribution of surfable events, surf criteria was established to provide the lower bounds of when surfing was expected to be possible somewhere in Lyall Bay. Based on the abundance of recorded surf events it was decided to base the surfing criteria on the minimum wave height and period featuring in the record.

In addition it was set as requirement that surfing events could only occur during daytime hours and during episodes of offshore wind or light onshore wind. A minimum of two consecutive hours of sustained conditions was set as a requirement of a day to be justified to contain a surf event. An overview of the surf criteria requirements are given below.

Minimum Surf Criteria (Good Conditions)

- Minimum Significant Wave Height (Hs) of 0.8 meter,
- Minimum Spectral Peak Period (Tp) of 11 seconds,
- The wind direction must originate from the North (±90°) or wind speed must be less than 6 m/s,
- The event must last for at least 2 hours, and
- Occur during average daylight hours between 6 am and 6 pm.

In addition an increasingly stringent surf criterion was defined to provide a rough estimate of the distribution of surfing days were conditions were expected to be very good to excellent.

Optimum Surf Criteria (Very Good Conditions)

- Minimum Significant Wave Height (Hs) of 1.5 meter,
- Minimum Spectral Peak Period (Tp) of 11 seconds,
- The wind direction must originate from the North (±90°)
- The event must last for at least 2 hours, and
- Occur during average daylight hours between 6 am and 6 pm.

Wave conditions outside of this surf envelope generally consist of primarily of periods of strong onshore winds or small wind waves. Surfing in Lyall Bay is expected to be possible outside of the envelope, but is not expected to represent quality conditions.

A total of 365 days were included in the suitability assessment. A total of 125 (34%) of these days provided conditions fulfilling the minimum surf criteria which corresponds to approximately

2.4 days per week. Very good surf conditions occur about 10% of the time or approximately 3.2 days per month. The distribution from the surf criteria analysis is illustrated in **Figure 5-1** below.

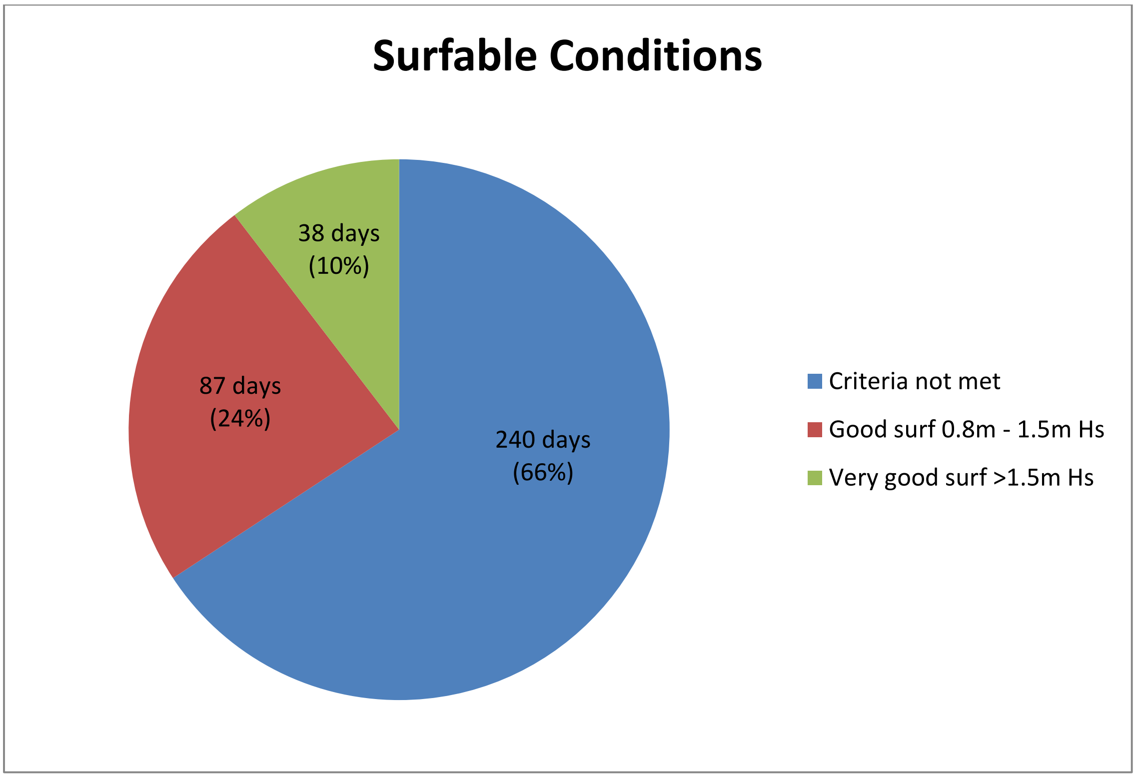

Figure 5-1 Distribution of surf criteria analysis.

Each historic surf event was subsequently grouped into wave event bins with dimensions of 0.5m (Hs) x 1s (Tp) x 20° (MWD). The corresponding recurrence of each event was then calculated using the one year wave dataset. **Table 5-1** shows the mean conditions for each photographed event with the total recurring days for each historic wave event.

Table 5-1 Photographed Event Recurrence.

Date	Recurring Event Days per Year	H_s	T_p	MWD
29/3/2014	35	1.2	12.0	188°
23/2/2014	33	0.8	11.3	184°
1/6/2014	32	1.3	11.3	189°
17/11/2014	27	1.3	12.3	184°
10/8/2014	16	1.5	12.2	179°
10/5/2014	14	1.7	11.3	184°
12/5/2014	9	1.7	14.1	183°
25/9/2014	6	1.7	14.3	186°
5/10/2014	5	2.0	13.4	191°
18/10/2014	3	0.8	14.9	185°
9/9/2014	1	2.5	15.4	187°
7/2/2015	1	2.9	12.2	189°
4/3/2014	1	3.5	11.3	195°

Based on the reoccurrence analysis of the reported surf events it was decided to select three historic events that represented a sufficient spreading in both reoccurrence frequency and offshore wave conditions. Each selected scenario is marked in yellow in **Table 5-1**. The aim of the selection was to capture differences in responses to the airport extension for as wide a range of various surfing conditions as possible using only 3 events. The three selected wave scenarios are presented in **Table 5-2**.

Table 5-2 Selected Wave Scenarios.

	H_s	T_p	MWD	Matching Event
Scenario 1	2.5m	15.4s	187º	9/9/2014
Scenario 2	1.3m	11.3s	189º	1/6/2014
Scenario 3	3.5m	11.3s	195º	4/3/2014

Figure 5-2 Photograph of Lyall Bay Corner 9/9/2014 (Scenario 1) (Source: Hamish Waterhouse Photograph).

Scenario 1 was selected as a rare but excellent surf event for Lyall Bay representing very high quality surfing conditions at the location as observed in **Figure 5-2**. This day was frequently mentioned on the Wellington Boardrider’s Facebook page by local surfers and several images were posted. The corresponding offshore wave conditions are large and with a very long spectral peak period of 15.4 seconds. This type of surf event would only be suitable for intermediate to advanced surfers.

Figure 5-3 Photograph of Lyall Bay 01/06/2014 (Scenario 2).

Scenario 2 (**Figure 5-3**) represents one of the most common good surf conditions in Lyall Bay. Offshore wave heights are almost half of those in Scenario 1 and the spectral peak period is 11.3 seconds. This type of surf event would be suitable for beginner to advanced surfers.

Figure 5-4 Photograph of Lyall Bay Corner 04/03/2014 (Scenario 3).

Scenario 3 (**Figure 5-4**) was selected to represent events with large wave conditions but with a shorter wave periods than Scenario 1. Representing the largest wave heights among the recorded events it was also of key interest if noticeable changes to wave induced current patterns could be expected. This type of event is just as uncommon but in combination the two scenarios provides some level of coverage of the expected responses of large swell events in Lyall Bay. This type of surf event would only be suitable for intermediate to advanced surfers.

5.2 Modelling Results - Wave Climate

The wave climate inside Lyall Bay is often dominated by relatively calm conditions due to its sheltered location to the Pacific Ocean. However, it also regularly experiences southerly ground swell events with significant wave heights exceeding two meters and spectral peak periods of 10 -15 seconds. Over the period 18th February 2014 to 17th February 2015 significant wave heights at the entrance of Lyall Bay exceed 1.0m 46% of the time and 2.0m 13% of the time.

Groundswell arriving from the Pacific Ocean undergoes a significant refraction process before reaching the entrance of the Bay which results in a very small directional spreading of the nearshore wave climate. The orientation of local bed contours causes the calculated dominant wave direction at the Bay entrance to be south to south-southwest (Appendix A). In comparison wave directions measured at the Baring Head wave buoy were south-southwest to southwest. An overview of the wave directions is provided in **Figure 5-5**.

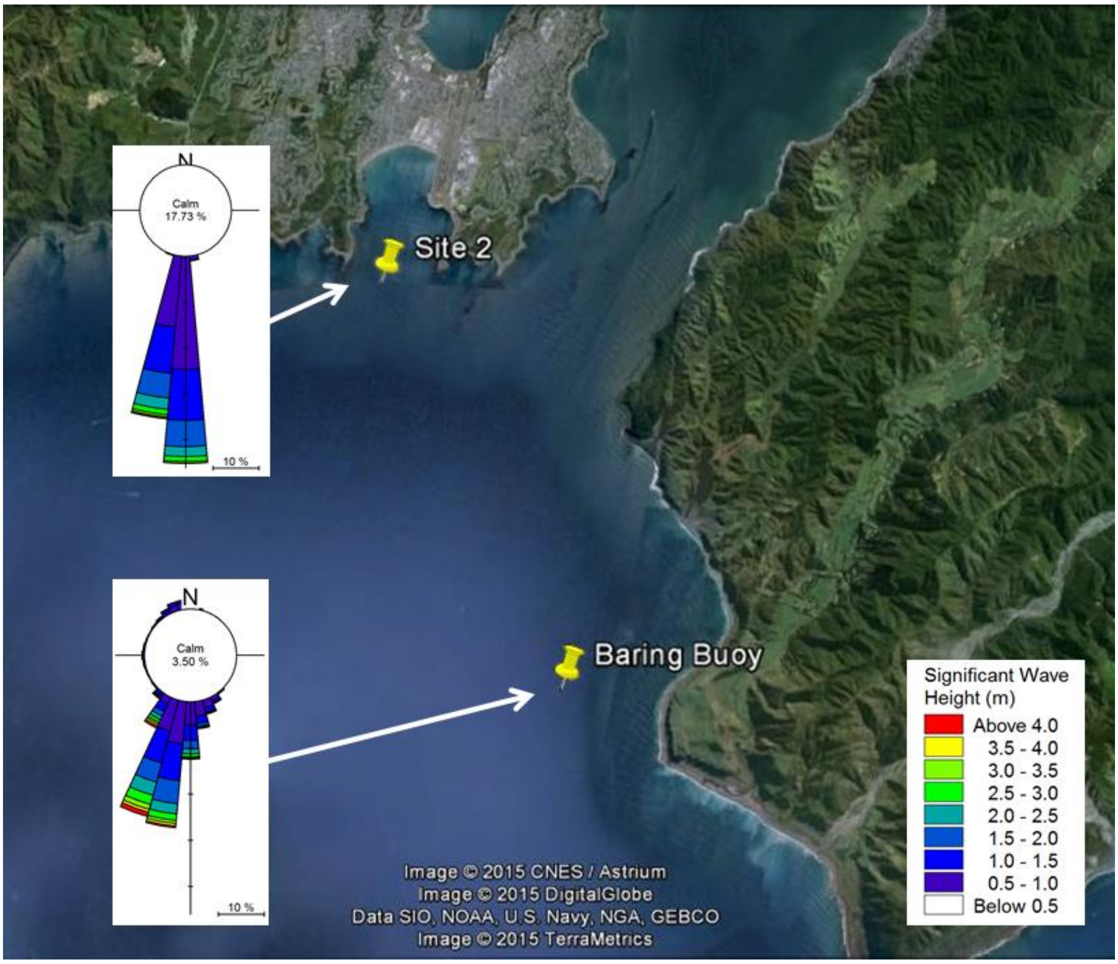

Figure 5-5 Wave Rose for Baring Head Measured and Lyall Bay Transformed Data (30/01/2014 – 18/02/2015).

The MIKE21 BW wave model was used to calculate the detailed wave behaviour inside the Bay for each of the 3 selected scenarios. The wave direction for all events is south to south-southwest. Spatial 2D maps of distribution of significant wave height were produced in order to provide a well-known proxy for comparing the existing bathymetry to the future layout containing the airport runway extension. Comparisons are presented in below in **Figure 5-6** to **Figure 5-11** for Scenarios 1, 2 and 3.

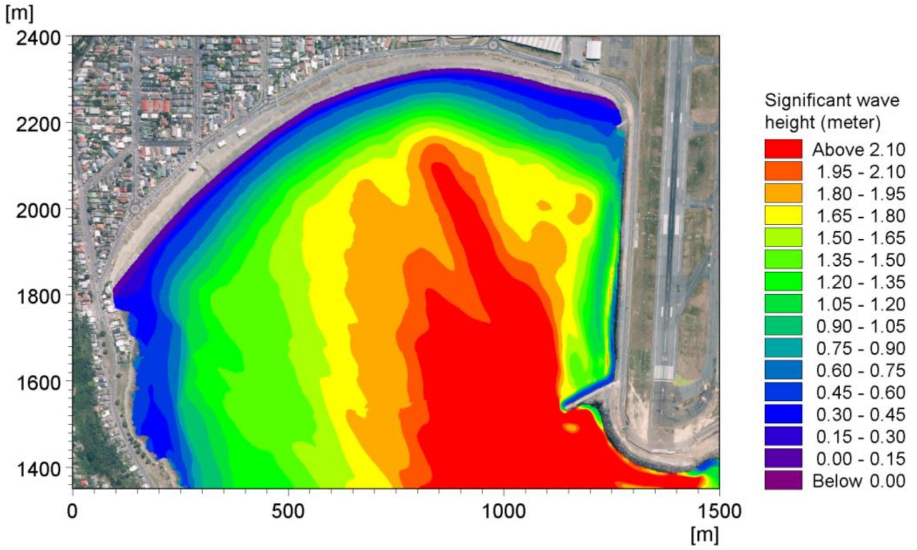

Figure 5-6 Phase Averaged Significant Wave Height Scenario 1: Existing Bathymetry.

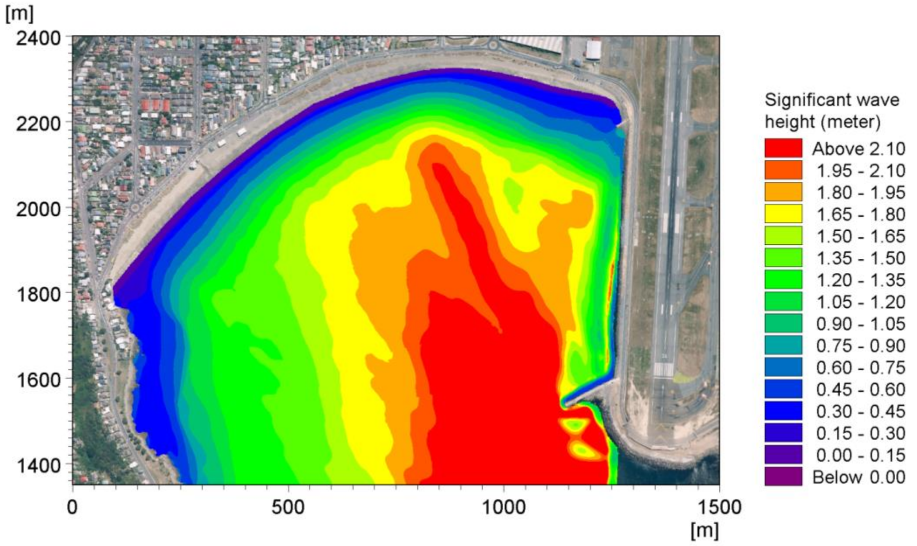

Figure 5-7 Phase Averaged Significant Wave Height: Scenario 1 (Airport Extension).

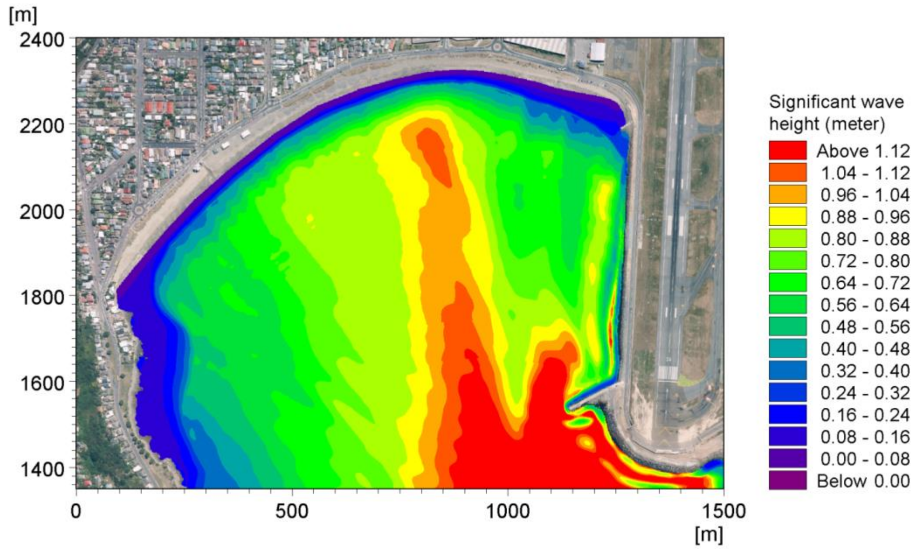

Figure 5-8 Phase Averaged Significant Wave Height: Scenario 2 (Existing Bathymetry).

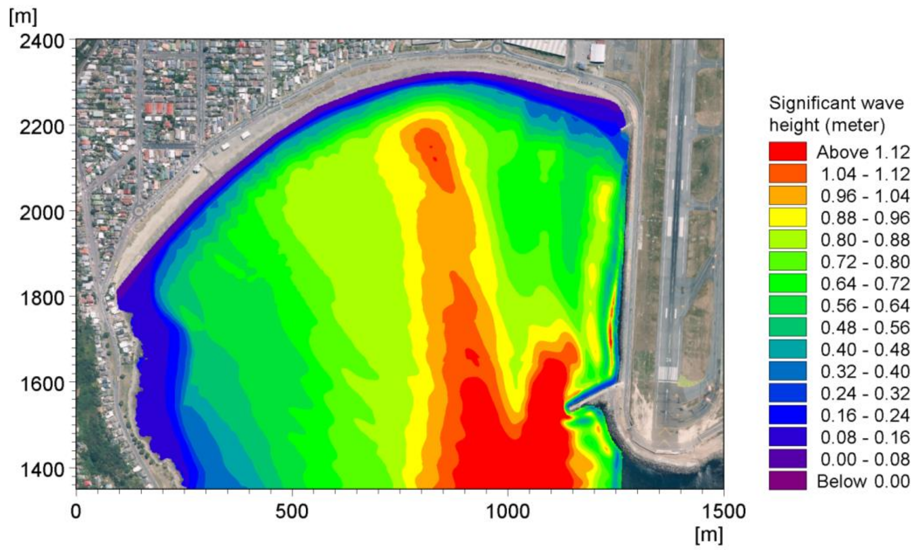

Figure 5-9 Phase Averaged Significant Wave Height: Scenario 2 (Airport Extension).

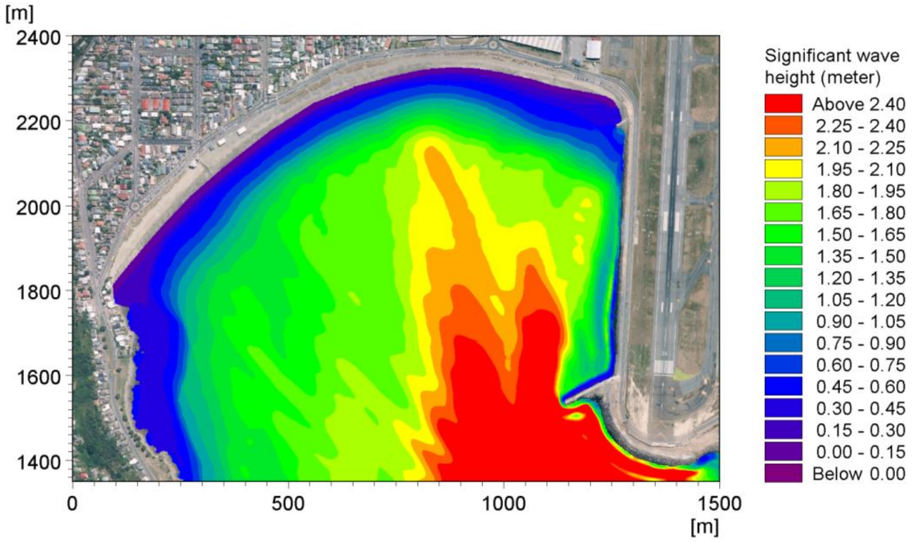

Figure 5-10 Phase Averaged Significant Wave Height: Scenario 3 (Existing Bathymetry).

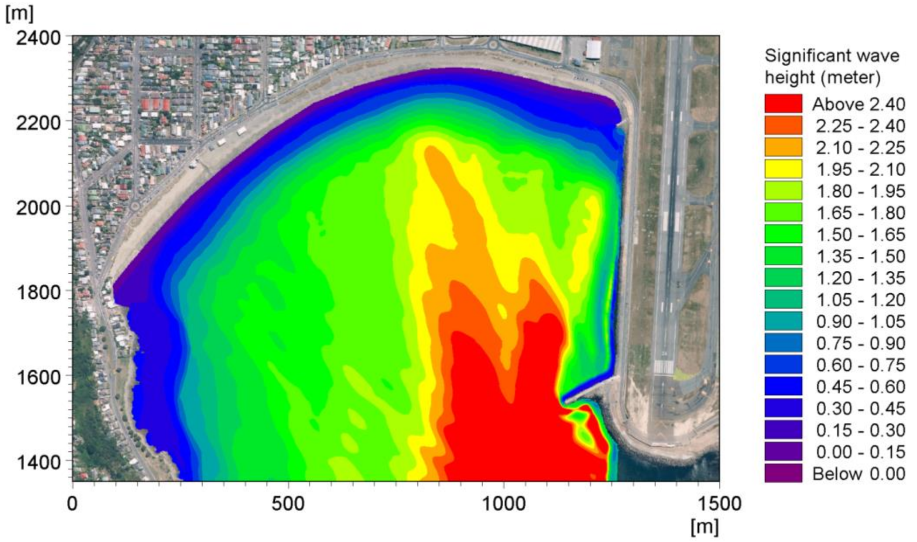

Figure 5-11 Phase Averaged Significant Wave Height: Scenario 3 (Airport Extension).

As the airport runway extension does not provide a direct obstruction to the incoming wave train it is not surprising that the overall changes to the nearshore wave field are small for all three scenarios simulated.

The dominant change to the wave field is caused by a slight reduction in the amount of wave energy that gets refracted towards the east. Hence the net wave energy propagating north of the spur breakwater actually slightly increases with the extension for all scenarios and most noticeable directly north of the Spur Breakwater.

5.3 Modelling Results – Wave Induced Currents

For Lyall Bay changes in swimming safety is governed by changes in nearshore breaking wave height and resulting changes in nearshore wave induced currents commonly referred to as sweep (longshore) and rip (outgoing cross shore) currents.

In the MIKE21 BW model, dynamic and phase resolving wave/current interaction is incorporated implicitly into the governing equations and as a result time averaged wave driven currents can be extracted automatically. Comparisons in time averaged wave induced currents are presented for each scenario in **Figure 5-12** to **Figure 5-20**.

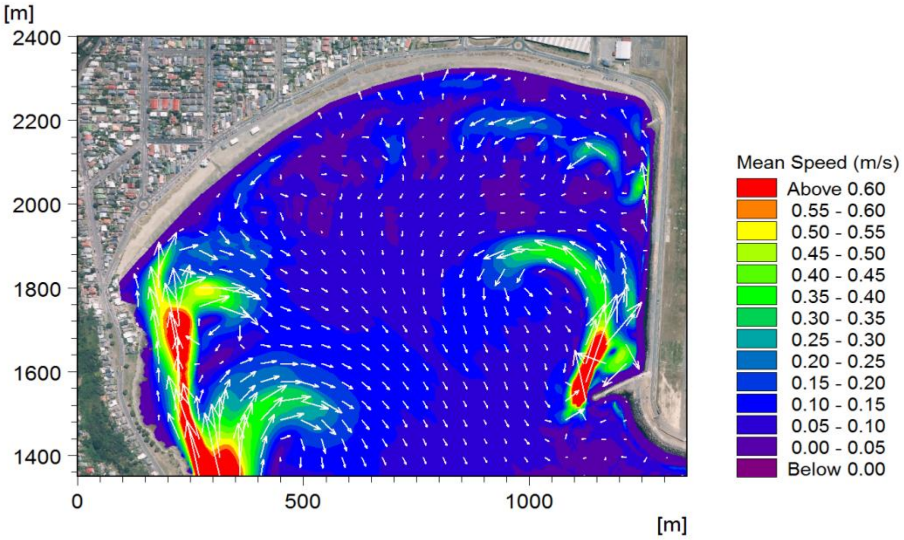

Figure 5-12 Wave Induced Currents Scenario 1: Existing Bathymetry.

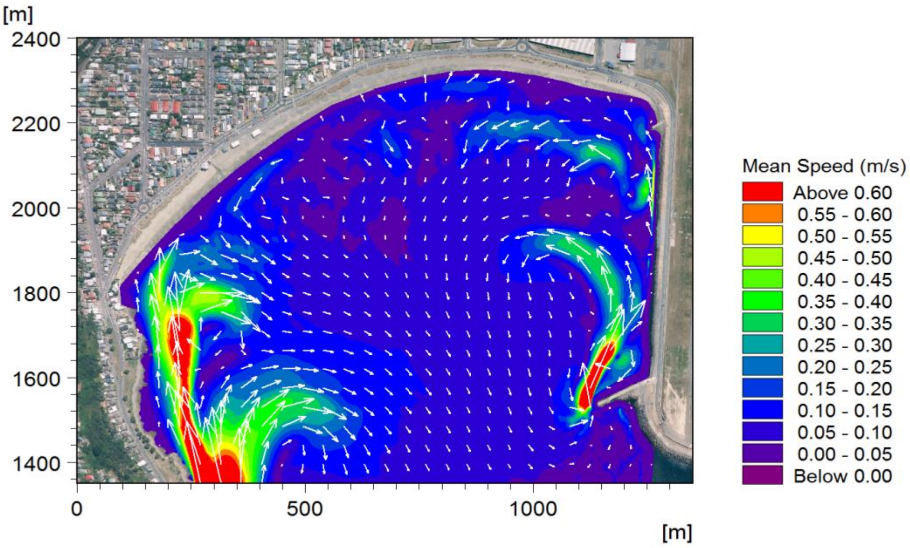

Figure 5-13 Wave Induced Currents Scenario 1: Airport Extension.

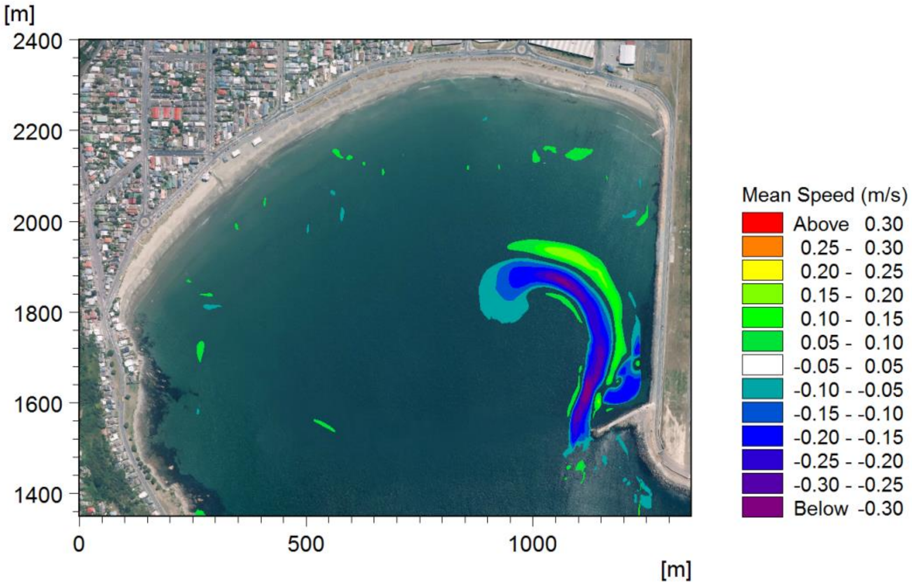

Figure 5-14 Wave Induced Currents: Scenario 1 (Difference).

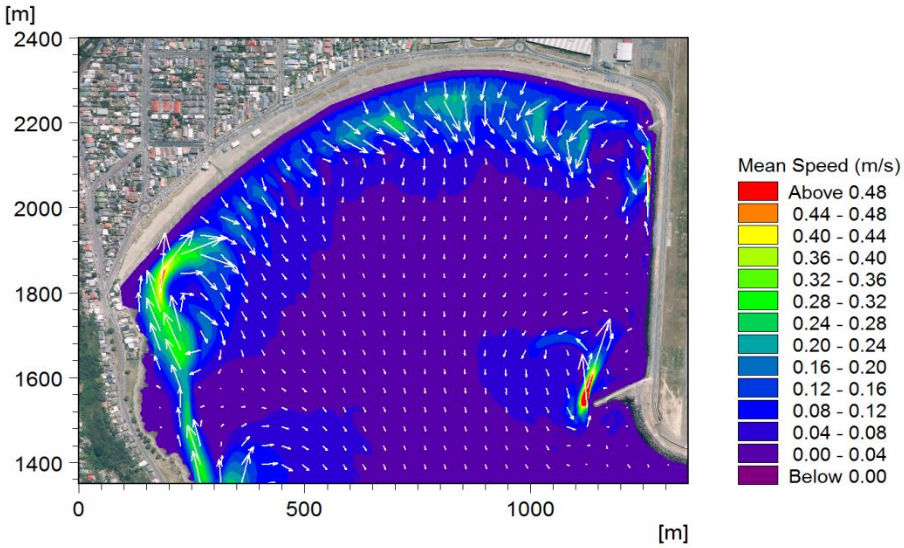

Figure 5-15 Wave Induced Currents: Scenario 2 (Existing Bathymetry).

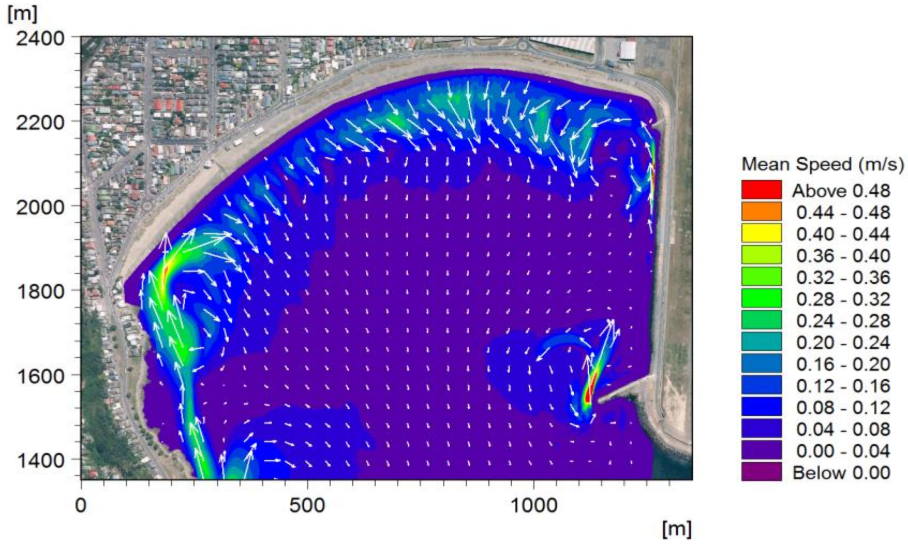

Figure 5-16 Wave Induced Currents: Scenario 2 (Airport Extension).

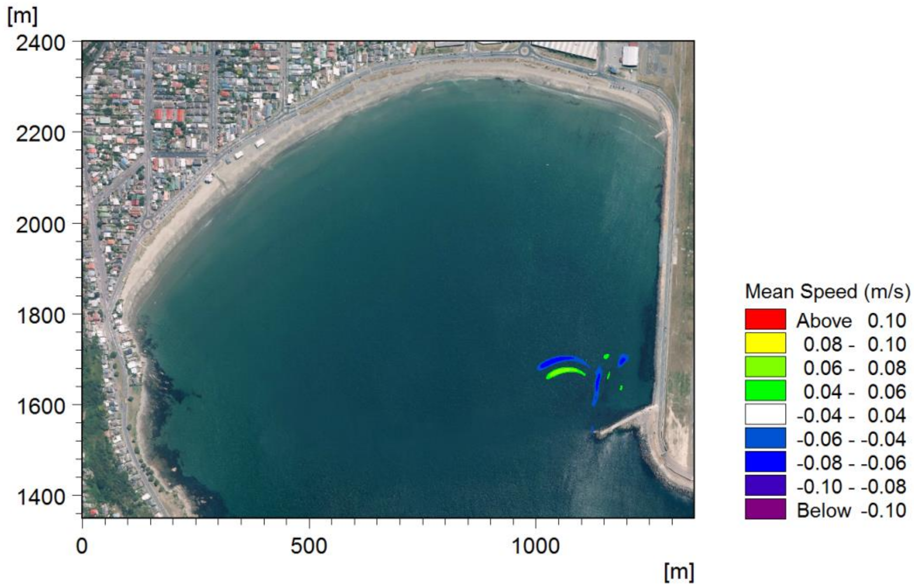

Figure 5-17 Wave Induced Currents: Scenario 2 (Difference).

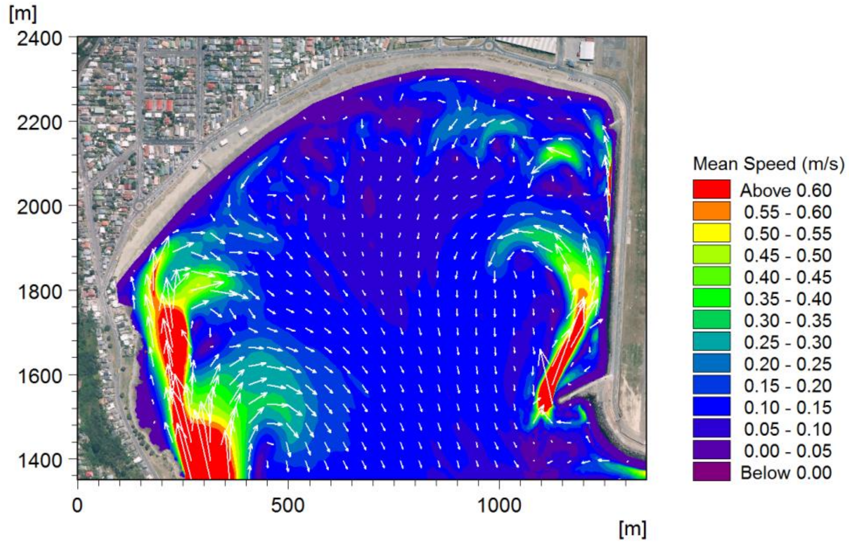

Figure 5-18 Wave Induced Currents: Scenario 3 (Existing Bathymetry).

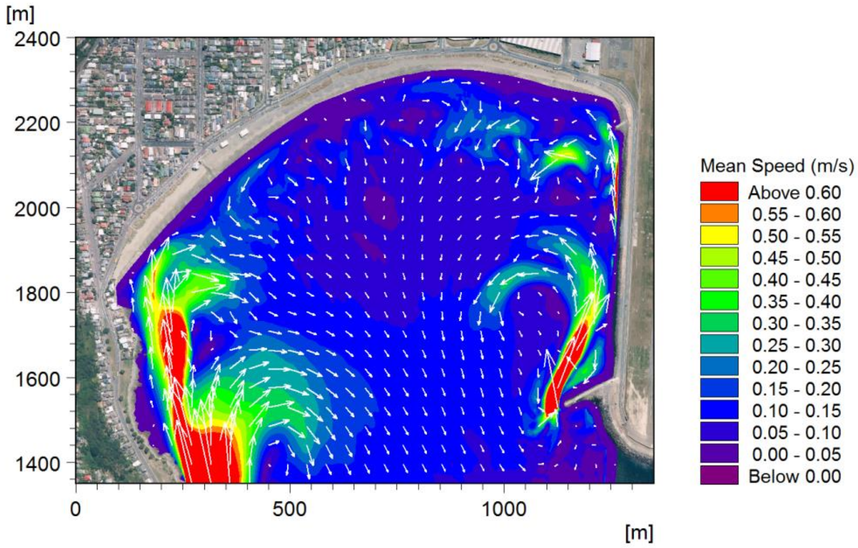

Figure 5-19 Wave Induced Currents: Scenario 3 (Airport Extension).

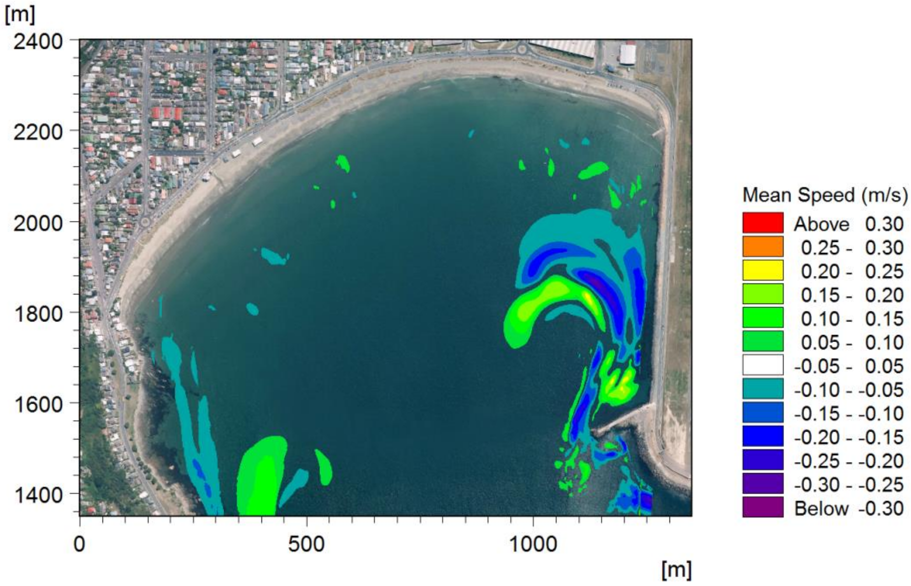

Figure 5-20 Wave Induced Currents: Scenario 3 (Difference).

From the 2D plots it is observed how current magnitude increases with wave height. As a result the maximum wave induced currents are largest for Scenario 3 followed closely by Scenario 1. Maximum nearshore current speeds are approximately 20% smaller. For all layouts strong circulation cells are observed along the western and eastern corner of the Bay and in the vicinity of the spur breakwater. The middle of the Bay contains several smaller rip current cells with locations varying between scenarios.

It is noticed that for Scenario 1 and 3 the circulation cell extending north of the spur breakwater is slightly weaker after the Airport extension. At the same time the location of the current shifts approximately 50 m north for Scenario 1 and 100 m south for Scenario 3. The change in local current patterns at The Corner may cause small localized bed changes, but will require a long term morphology assessment in order to estimate if any notable changes in bathymetry is to be expected.

The previous sediment transport modelling presented in NIWA (2015) did not predict any morphological change expect for local scouring. However this modelling did not include morphological updates of the bathymetry which is required to capture subtle changes in bar orientations.

The effect of the changed current patterns on wave surfability is implicitly captured in the MIKE21 BW/OPTISURF simulations (assuming a fixed bed).

In nearshore areas of Lyall Bay the overall small differences in significant wave heights and change in wave induced currents poses a negligible impact to changes in swimming safety.

5.4 Modelling Results - Surf Break Quality

OPTISURF model simulation results for existing conditions and the proposed airport extension were processed and compared to quantify the changes to the wave field in Lyall Bay. The colour of each surf track represents the maximum wave face height experienced by the surfer during each ride-able wave. The maximum average surfer ground speed was set to 15 m/s. It is acknowledged that this upper threshold value is larger than what is commonly considered achievable (~10 m/s). As expected most surfable rides in this study fall below this value.

However due to uncertainties regarding nearshore bathymetry is was decided to include wave rides with average speeds up to 15 m/s.

5.4.1 Scenario 1

OPTISURF was run for each 20 minute Boussinesq output to quantify the surfable waves in Scenario 1. Waves with ride lengths below 50 m and maximum face height below 1.8 m were filtered out. All remaining surfable tracks for the existing and extension layouts are overlaid on the bathymetry contours in **Figure 5-21** and **Figure 5-22** respectively. The maximum wave face height represents the colour of each track.

Figure 5-23 to **Figure 5-25** show the distribution of surfable waves, ride length and maximum wave face height. A separate plot was generated for the three defined surfing areas, 'The Corner', 'Middle Beach' and 'West Beach', shown in **Figure 4-5**.

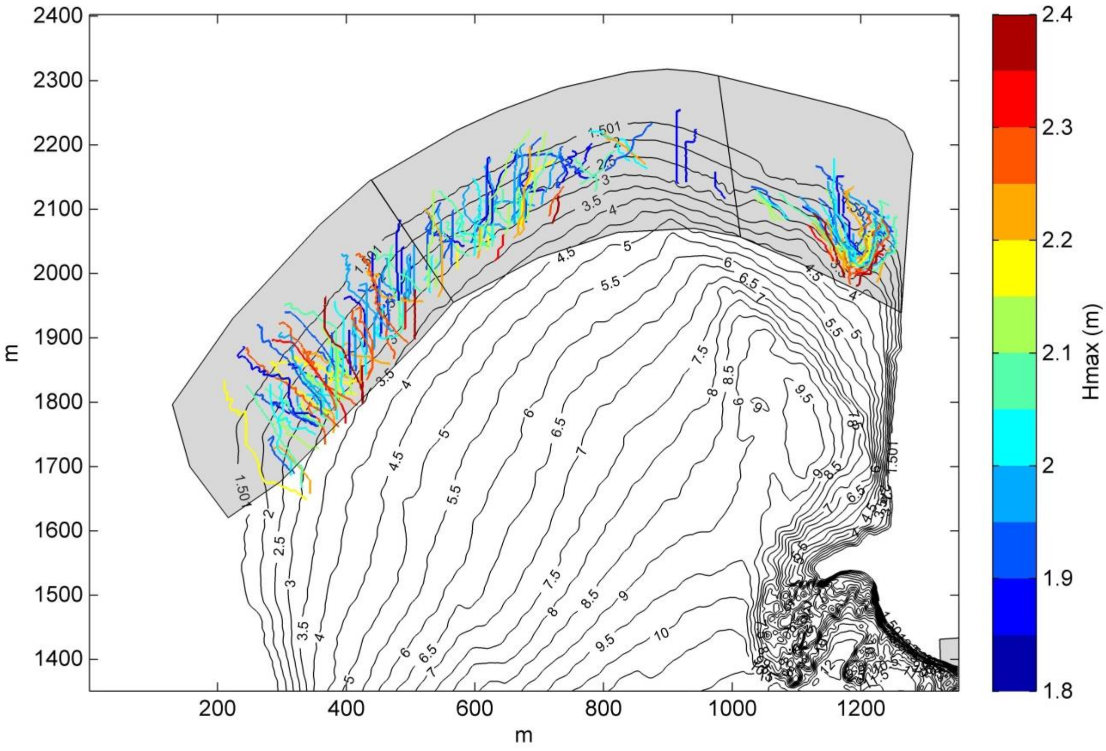

Figure 5-21 OPTISURF Track Plot: Scenario 1 (Existing Bathymetry).

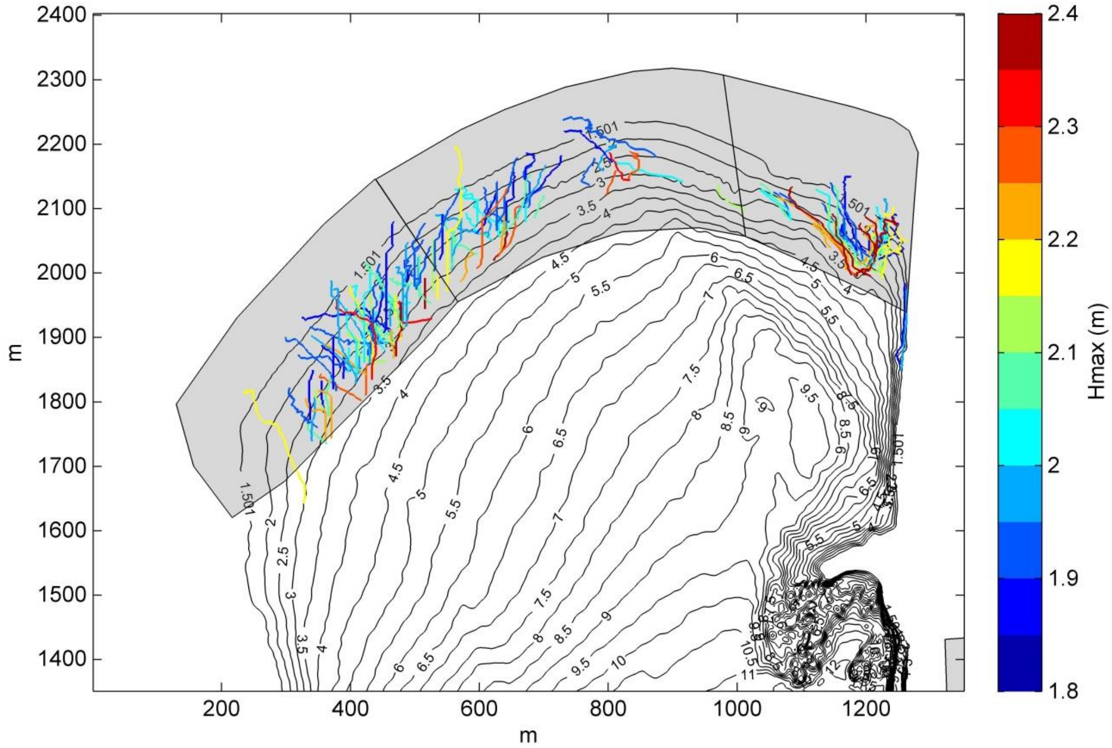

Figure 5-22 OPTISURF Track Plot: Scenario 1 (Airport Extension).

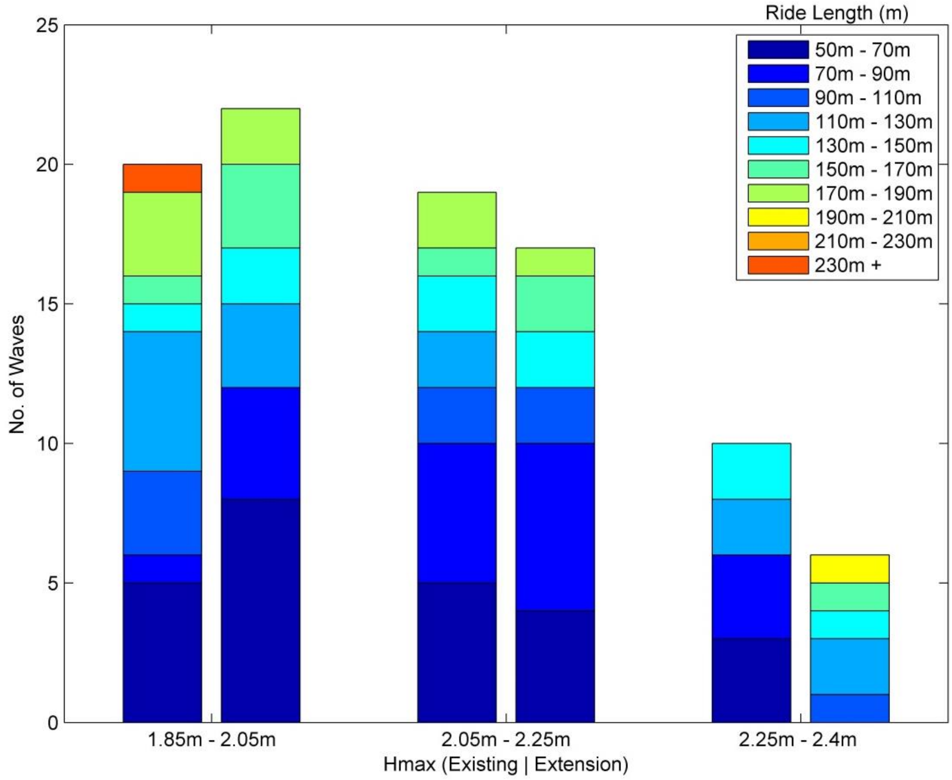

Figure 5-23 OPTISURF Statistics Scenario 1: 'The Corner'.

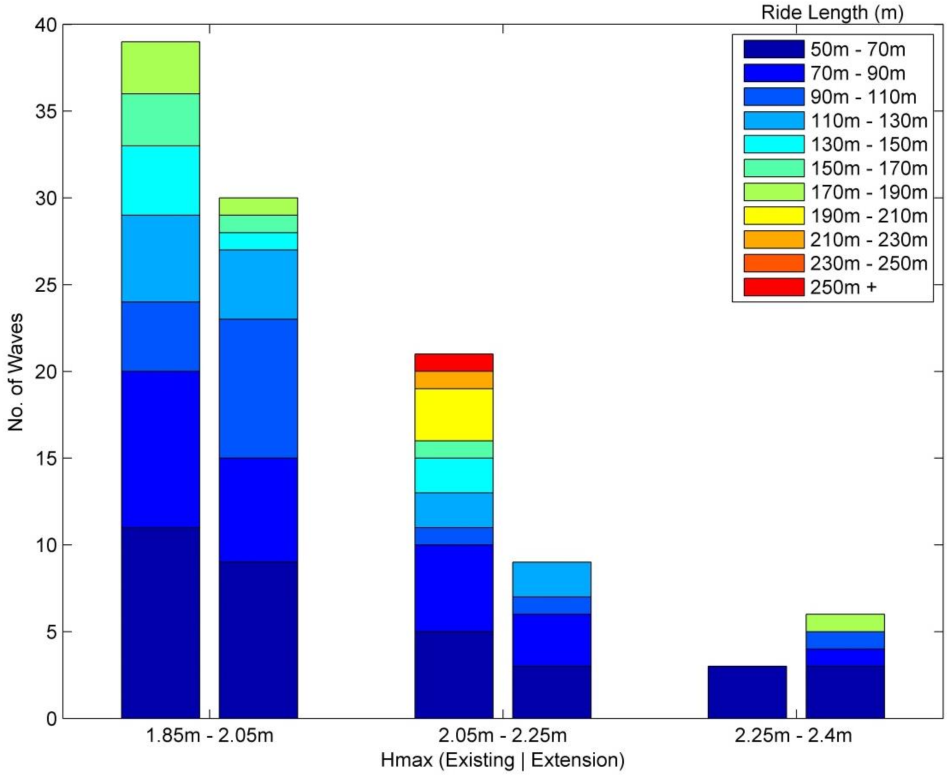

Figure 5-24 OPTISURF Statistics Scenario 1: Middle Beach.

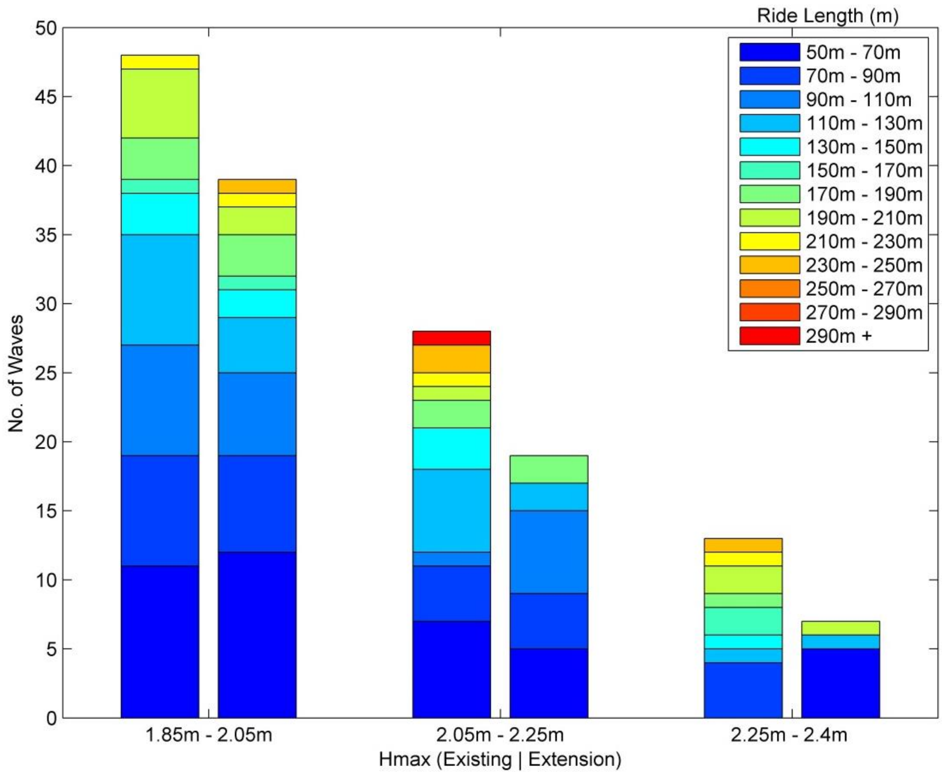

Figure 5-25 OPTISURF Statistics Scenario 1: West Beach.

Table 5-3 Percentage Change in Wave Conditions Scenario 1.

	H_{max} (%)	L (%)	No. of waves (%)
The Corner	-1.2%	0.2%	-8.2%
Middle Beach	0.1%	-14.8%	-28.6%
West Beach	-1.3%	-16.4%	-27.0%

As shown in **Table 5-3**, there is a significant drop in the number of waves for scenario 1, with the majority of the impact in the Middle Beach and West Beach areas. This trend is visible in **Figures 5-21**, **Figure 5-22** and **Figure 5-23**. There is also a reduction in the ride length for these beach areas, 14.8% and 16.4% less for the Middle and West Beach areas respectively. The longest ride is reduced by at least 40 m in each area. In terms of wave height there is little impact, with a maximum reduction of 1.3% for the West Beach.

5.4.2 Scenario 2

OPTISURF was run for each 20 minute Boussinesq output to quantify the surfable waves in Scenario 2. Waves with ride lengths below 50 m and maximum face height below 1.0 m were filtered out. All remaining surfable tracks for the existing and extension layouts are overlaid on the bathymetry contours in **Figure 5-26** and **Figure 5-27** respectively. The maximum wave face height represents the colour of each track.

Figure 5-28 to **Figure 5-30** show the distribution of surfable waves, ride length and maximum wave face height. A separate plot was generated for the three defined surfing areas, 'The Corner', 'Middle Beach' and 'West Beach', shown in **Figure 4-5**.

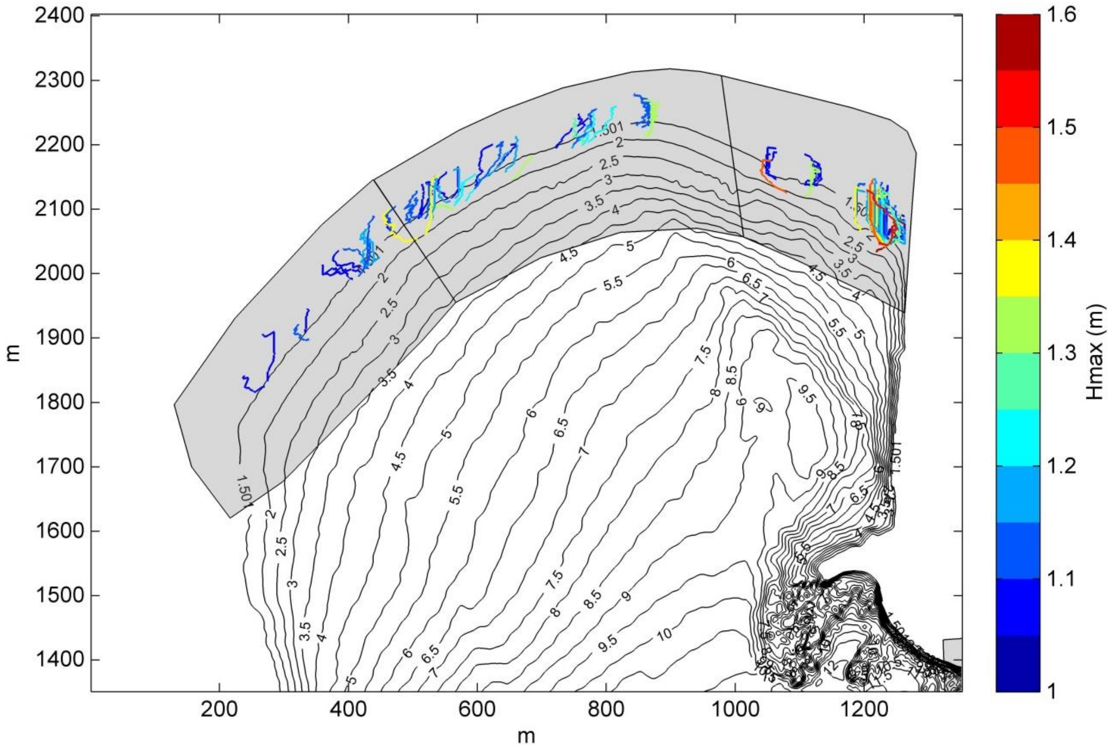

Figure 5-26 OPTISURF Track Plot: Scenario 2 (Existing Bathymetry).

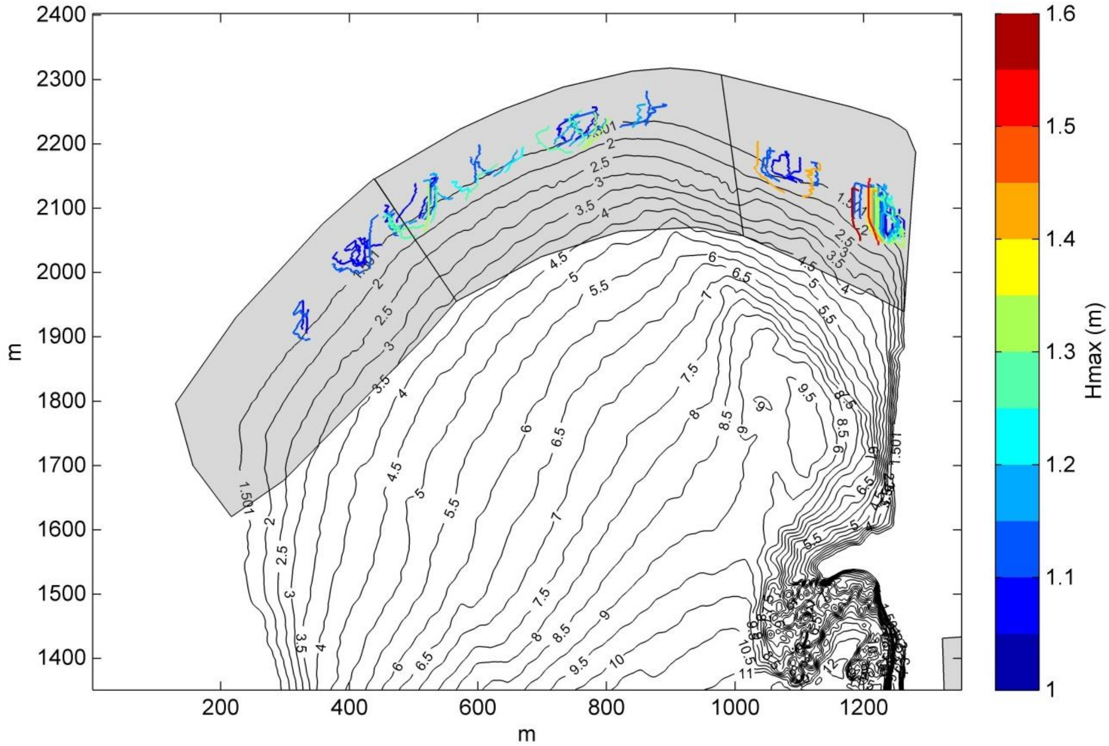

Figure 5-27 OPTISURF Track Plot: Scenario 2 (Airport Extension).

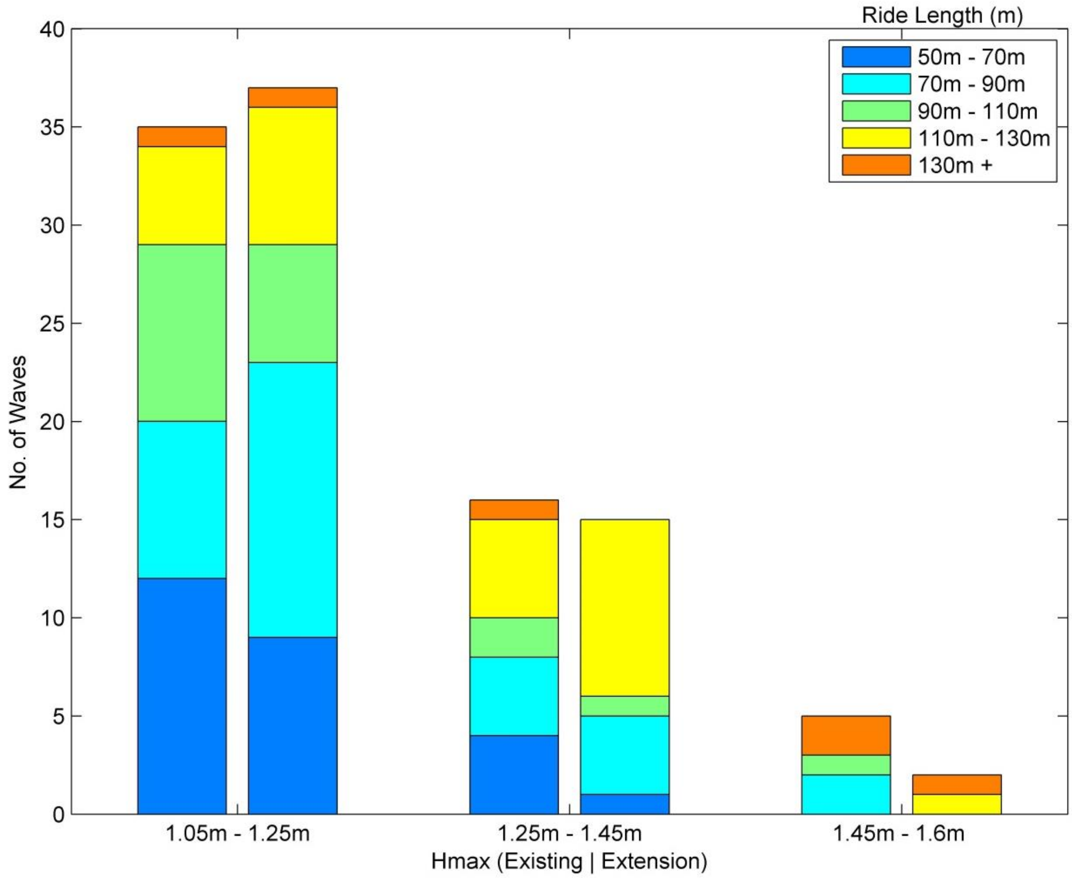

Figure 5-28 OPTISURF Statistics Scenario 2: ‘The Corner’.

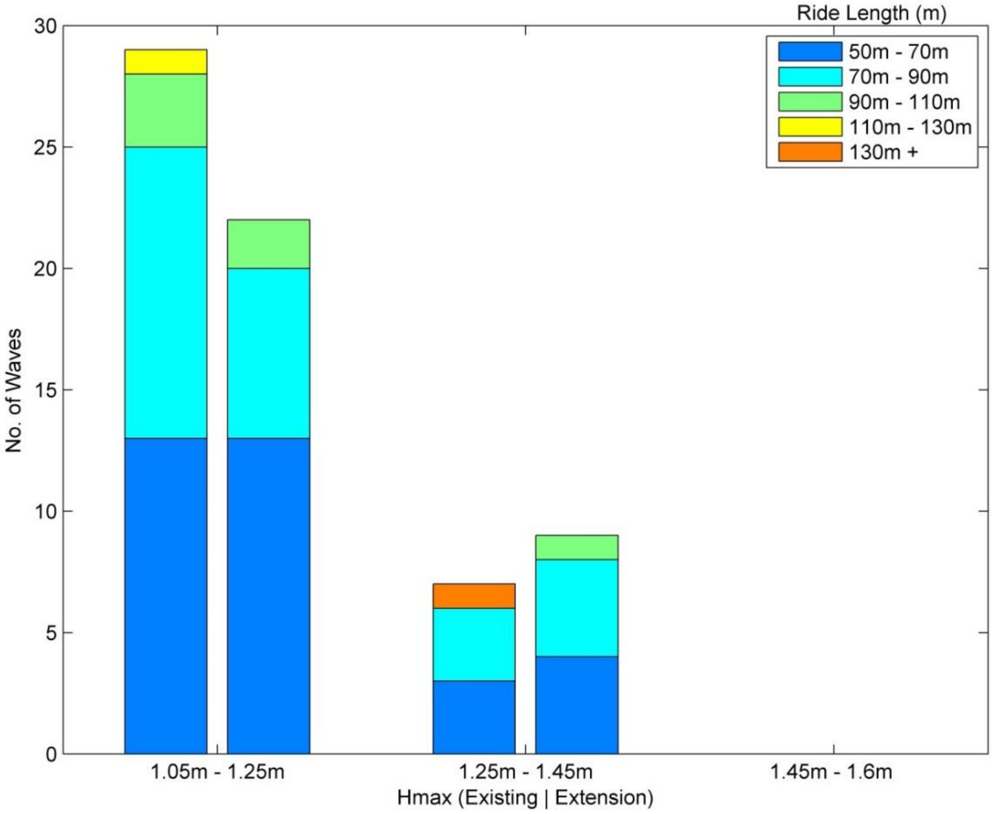

Figure 5-29 OPTISURF Statistics Scenario 2: Middle Beach.

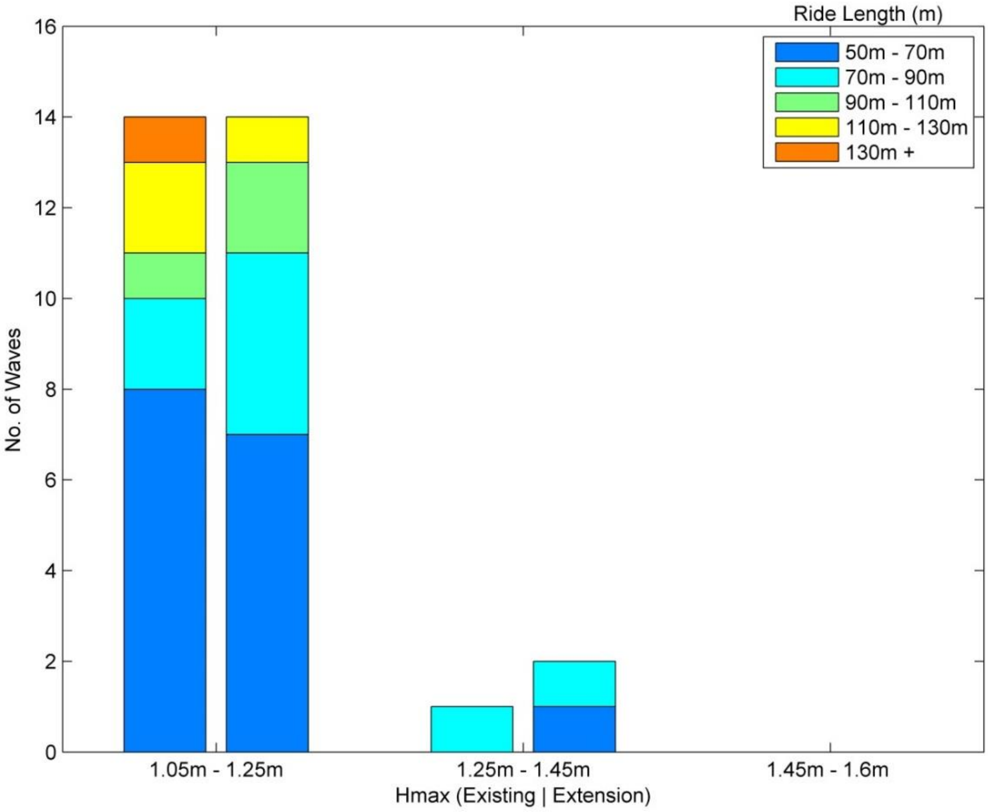

Figure 5-30 OPTISURF Statistics Scenario 2: West Beach.

Table 5-4 Percentage Change in Wave Conditions Scenario 2.

	H_{max} (%)	L (%)	No. of waves (%)
The Corner	-1.0%	4.1%	-3.6%
Middle Beach	0.8%	-6.1%	-13.9%
West Beach	-0.6%	-6.3%	6.7%

For Scenario 2 there is little impact on the wave height (see **Table 5-4**), with a maximum reduction of 1% for The Corner and an increase of 0.8% for the Middle Beach. Mean ride length increases slightly for The Corner by 4.1%, however drops for the Middle and West Beaches with reductions of 6.1% and 6.3% respectively. There is a reduction in the number of waves for The Corner and The Middle Beach and an increase of one wave for the West Beach. Again, the Middle Beach area is most impacted with a 13.9% reduction in waves.

5.4.3 Scenario 3

OPTISURF was run for each 20 minute Boussinesq output to quantify the surfable waves in Scenario 3. Waves with ride lengths below 50m and maximum face height below 2.0m were filtered out. All remaining surfable tracks for the existing and extension layouts are overlaid on the bathymetry contours in **Figure 5-31** and **Figure 5-32** respectively. The maximum wave face height represents the colour of each track.

Figure 5-33 to **Figure 5-35** show the distribution of surfable waves, ride length and maximum wave face height. A separate plot was generated for the three defined surfing areas, ‘The Corner’, ‘Middle Beach’ and ‘West Beach’, shown in **Figure 4-5**.

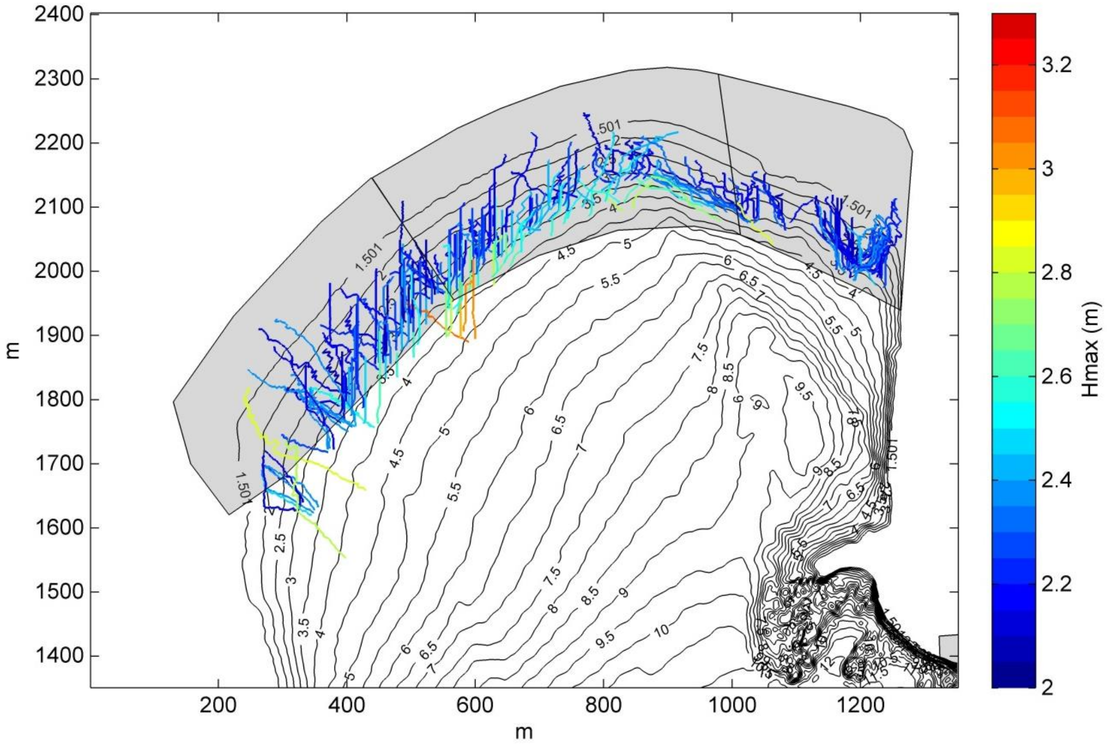

Figure 5-31 OPTISURF Track Plot: Scenario 3 (Existing Bathymetry).

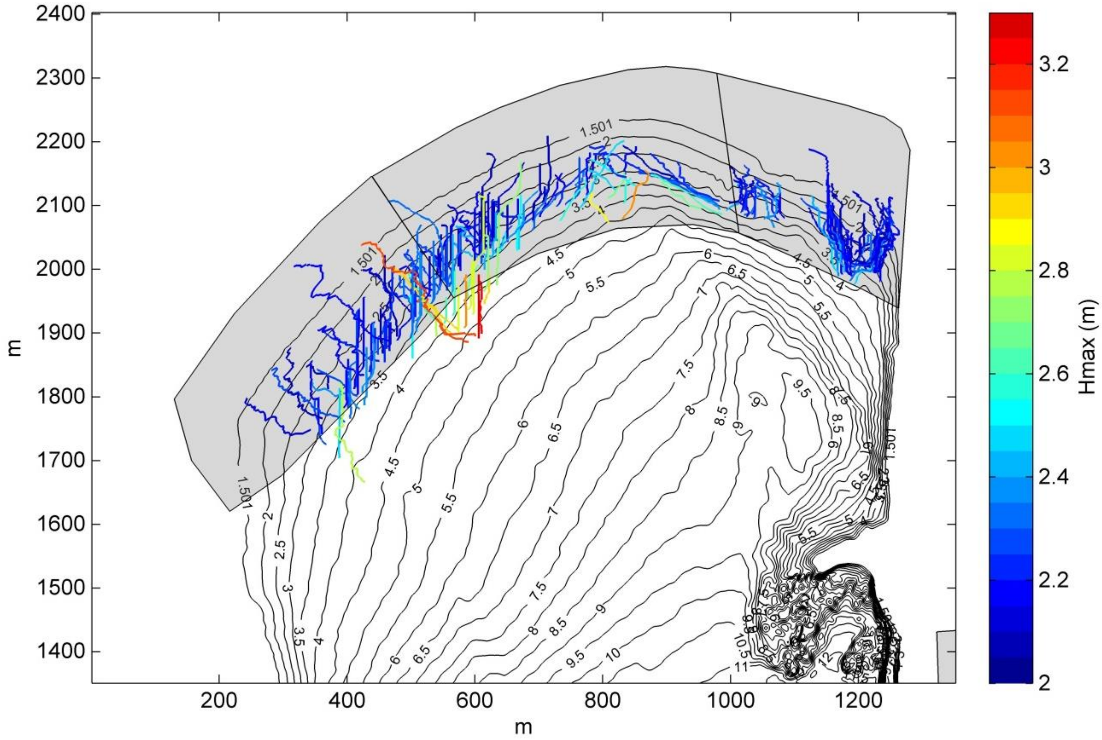

Figure 5-32 OPTISURF Track Plot: Scenario 3 (Airport Extension).

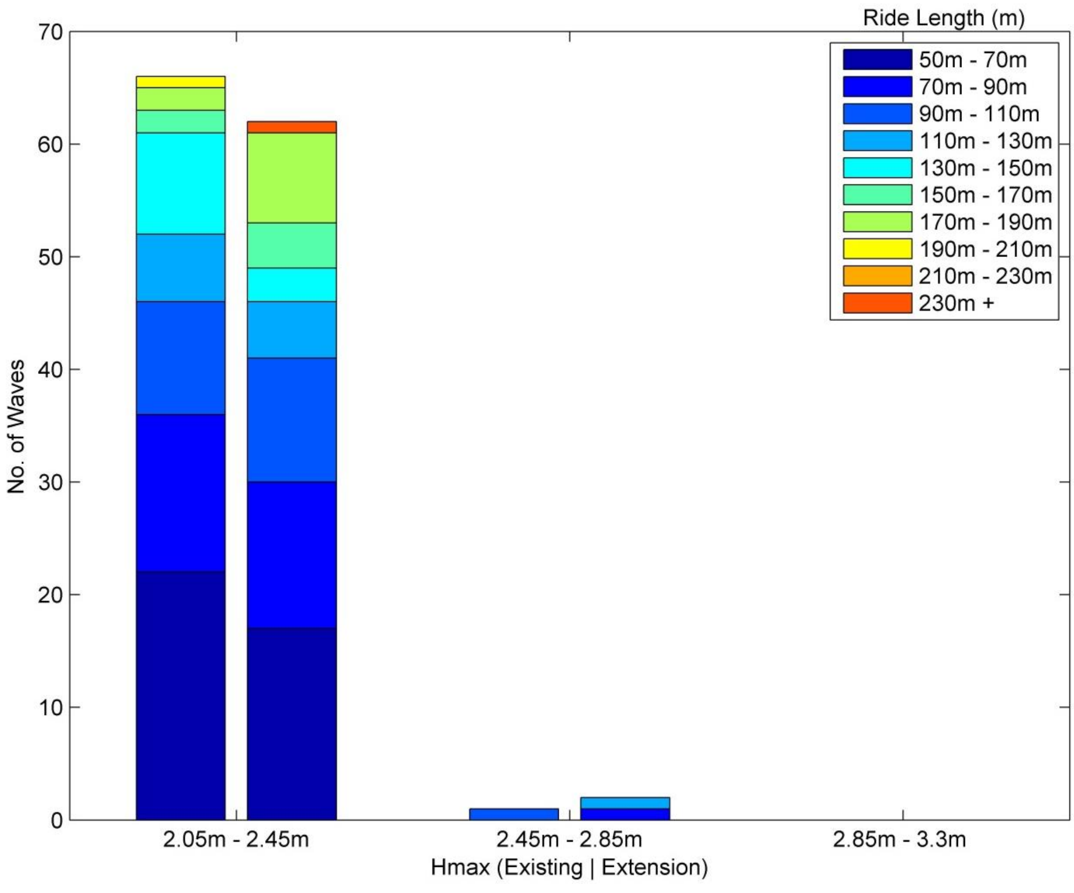

Figure 5-33 OPTISURF Statistics Scenario 3: ‘The Corner’.

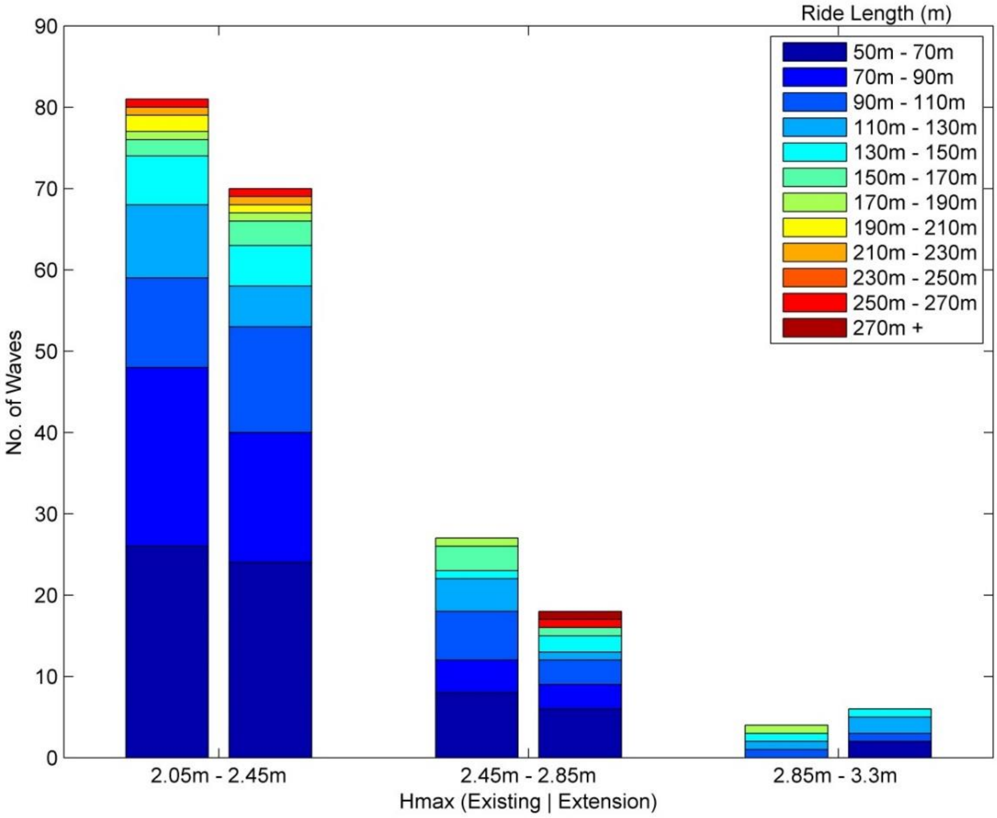

Figure 5-34 OPTISURF Statistics Scenario 3: Middle Beach.

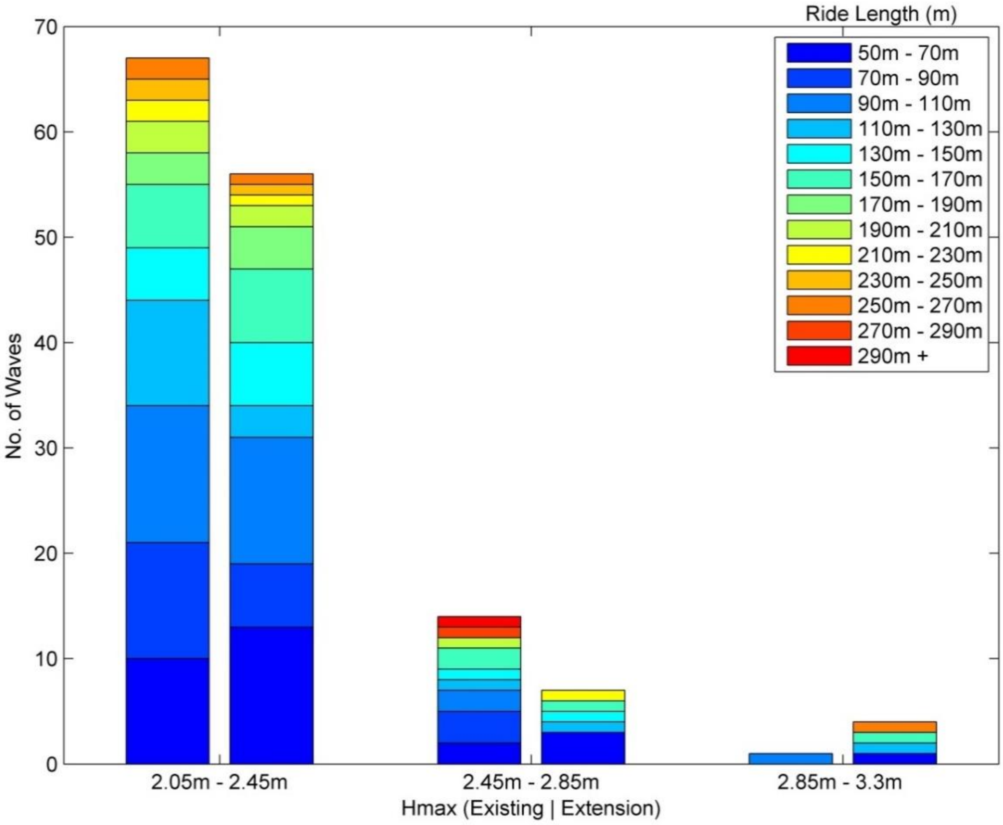

Figure 5-35 OPTISURF Statistics Scenario 3: West Beach.

Table 5-5 Percentage Change in Wave Conditions Scenario 3.

	H_{max} (%)	L (%)	No. of waves (%)
The Corner	-0.3%	9.6%	-4.5%
Middle Beach	-0.3%	0.9%	-16.1%
West Beach	0.5%	-3.9%	-18.3%

Again for this scenario the impact on wave height is small (see **Table 5-5**), with less than a 0.5% increase for the West Beach and a 0.3% reduction for The Corner and the Middle Beach. Ride length is increased for The Corner and the Middle Beach by 9.6% and 0.9% respectively. For these two areas the maximum ride length increases by 40m. In the West Beach there is a ride length reduction of 3.9%, and a maximum reduction of 40m.

Similar with Scenario 1 there is a significant reduction in the number of waves, with 16.1% for the Middle Beach and 18.3% for the West Beach. **Figure 5-33**, **Figure 5-34** and **Figure 5-35** confirm this same trend observed with Scenario 1.

5.5 Summary of Anticipated Impacts to Surfing Amenity

Open ocean waves undergo tremendous regional refraction and diffraction processes before arriving at the entrance to Lyall Bay. The combination of these phenomena most often creates a much more directional uniform sea state compared to exposed open ocean beaches especially during significant groundswell events with large wave periods.

Upon entering the Bay, wave refraction starts to develop along the western and eastern beaches. This process effectively causes an east-west gradient in wave energy to either side, which drives a resulting transverse wave diffraction process.

The transverse wave diffraction processes can be observed as small to moderate perturbations of the incoming wave train also commonly referred to as wave peakiness. The process increases with wave period and will be most dominant during large swell events. A high level of wave peakiness is often very important for generating surf quality along coastlines with straight or gently varying bed contours. A further description of this phenomenon is provided in Appendix B.

The proposed airport runway extension will extend over the surf spot Airport Rights and the surfing amenity of that spot will be lost, since the bathymetry south of extension is too deep to generate a surfable breaking wave. For this reason there was no reason to carry out a detailed modelling assessment for this break.

Based on current predictions of dominant wave directions at the Lyall Bay entrance, the proposed airport runway extension is not expected to cause noticeable sheltering and resulting reduction in wave height along the inshore surf spots. However the steep rock revetment enclosing the extension is expected to reduce wave refraction to the east which will reduce the overall wave peakiness of wave propagating further into the Bay.

At The Corner the wave quality is governed primarily by the diffracted wave field that extends from the rock formation beneath the spur breakwater and causes a favourable angle between incoming waves and the local bed contours. Therefore, the reduction in wave peakiness only affects this spot to a lesser degree with an expected reduction of characteristic surf rides of 4-8%.

In the absence of nearshore rocky outcrops or reefs the surf amenity at the Western and Middle beaches are, to a large degree, dependent on the peakiness of the incoming wave field in order to generate surfable waves. With the reduction in wave peakiness caused by the proposed airport extension, the reduction in surf rides is expected to be 14-29% for Middle Beach and 18-27% at West Beach.

For all three surf spots negative impacts are expected to be the largest for large wave periods (long period swell).

6 Recommendations for Avoiding, Remedying or Mitigating Effects of Works

Based on the expected key requirements of the airport runway extension it is not considered likely that the potential surfing amenity impacts can be avoided or reduced through altering the core design of the layout as this would require a shortening of the proposed airport runway extension.

It could, however, be possible to introduce one or more mitigation options that could compensate for the loss of surfing amenity or potentially even improve it above baseline conditions.

As mentioned in previous sections of the report, the wave climate is often dominated by long period swell with very little directional spreading both of which is an ideal foundation for the successful implementation of artificial surf enhancing structures.

This study has considered the following four mitigation options and listed advantages (Pros) and disadvantages (Cons) associated with each option. The assessment is strictly preliminary and subject to detailed investigation by further studies and stakeholder feedback.

- Option 1 - Spur Breakwater Removal
  This option would consider the partial full removal of the spur breakwater located south of The Corner.
  ***Pros*:** Will increase local wave heights at The Corner.
  ***Cons*:** May require upgrade of inshore rock revetment. May cause wave reflection from the inshore revetment back into The Corner, which could negatively impact surf quality. Reducing the wave diffraction may actually reduce surf quality. There are potential environmental concerns associated with dredging the underlying rocky outcrop.

- Option 2 - Submerged Surfing Reef Offshore of Airport Runway Extension
  This option consists of constructing an artificial surfing reef located south of the airport runway extension. The structure would most likely extend from the eastern side of the bay.
  Pros: Will create a new surf spot which could potentially accommodate very large and powerful surf conditions. No expected impact to existing surf spots. Not expected to cause shoreline impact.
  Cons: Would potentially be required to extend into deep water and require a large amount of construction material to build. Would have to sustain occasional very large wave events. Safety concerns regarding surfer impact on reef structure or entrapment will have to be addressed.

- Option 3 - Nearshore Submerged Surfing Reef
  This option would consist of a surfing reef located either at Middle or Western Beach. The purpose of the structure would be to improve the post-extension surfing amenity at one of these two surf spots.
  Pros: Will improve the consistency and quality of surf able waves where it is built. Will be built in shallower water and require less material than the offshore option and be exposed to a less energetic wave climate.
  Cons: May cause shoreline impact. Safety concerns regarding surfer impact on reef structure or entrapment will have to be addressed.

- Option 4 - Submerged Wave Focusing Structure (SWFS)
  This option would be similar to Option 3 but unlike a normal surfing reef waves will not break on the structure itself. The structure will be built in slightly deeper water with a wide focusing platform and a deeper crest level. The structure will refract waves on both sides towards its centre thereby generating a localized wave peak which will break on the natural

inshore bathymetry.
Pros: Will improve the consistency and quality of surfable waves inshore of where it is built. Will be built in shallower water and require less material than option 2 and potentially also less than Option 3. Wave breaking will not occur on the structure itself thereby potentially reducing safety concerns.
Cons: May cause shoreline impact. Surf breaking pattern may be more difficult to control compared to Option 3. May require a large footprint to generate sufficient focusing.

Based on the information available to DHI at this stage of the project, and subject to further investigations, it is considered that Option 4 may be the safest and most cost effective approach for mitigating and potentially further enhancing surfing amenity in Lyall Bay. The SWFS uses an alternative method for increasing localized wave peakiness. The affected section of beach will be limited but the amount of localized wave amplification and wave breaking intensity could potentially be enhanced well beyond current conditions and produce consistent surfing conditions of good quality. A preliminary concept visualization using the MIKE21 BW model is presented in **Figure 6-1** and **Figure 6-2**.

Detailed investigations using state-of-the-art methods can be used to assure that wave breaking patterns are in line with project performance objectives and without causing adverse shoreline impacts. Construction material should most likely be based on rock similar in size to that used for the extension rock revetment. The absence of wave breaking on the hard structure itself will reduce safety concerns associated with SWFS design.

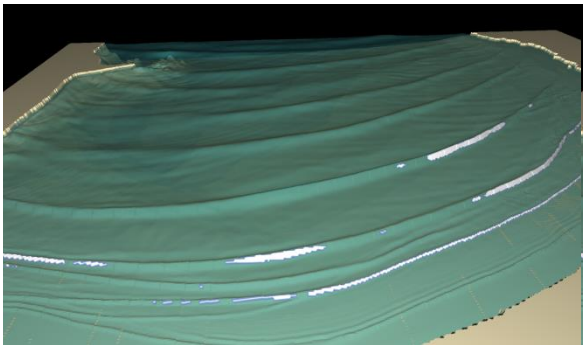

Figure 6-1 Lyall Bay Airport Extension without SWFS.

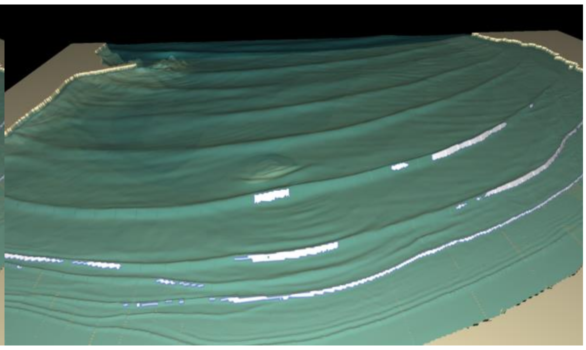

Figure 6-2 Lyall Bay Airport Extension with SWFS.

It should be noted that in addition to associated technical implications, the potential feasibility of each of the four mitigation option will require careful consideration with regards to variations in expected construction costs and stakeholder requirements. As a result DHIs recommendation for a suitable mitigation option should be regarded as preliminary and subject to further assessment and stakeholder engagement.

7 Monitoring to Confirm Effects of Works

DHI recommends that a field monitoring campaign is carried out that can measure and confirm the findings of this assessment or future associated investigations.

The monitoring campaign methodology should be consistent in the methods used to assess conditions pre and post construction and should document changes in local wave field, nearshore bathymetry and surfing amenity.

7.1 Wave Field Investigations

Additional wave data is scheduled to be collected in October 2015. This section will be completed after this data has been collected and analysed.

7.2 Nearshore Bathymetry

NIWA has provided a dataset of the existing bathymetry for Lyall Bay. Nearshore areas are mostly covered by a relatively coarse 25 m bathymetric grid and do not cover the inner surf zone. The relatively coarse dataset and the missing data, limit the ability to account for detailed nearshore sand bank configurations, which may be important to surfing amenity and could in some way be affected by the changes in local wave climate.

Figure 7-1 illustrates this missing data with OPTISURF tracks from Scenario 1 overlaid on the raw bathymetry plot **Figure 3-4**. Note the second half of most tracks in the West Beach area and some entire tracks in the Middle Beach area rely on interpolation over missing data. The Corner is reasonably well covered by the 25m bathymetric grid, however its resolution is still relatively coarse compared to what would be preferred for this type of study.

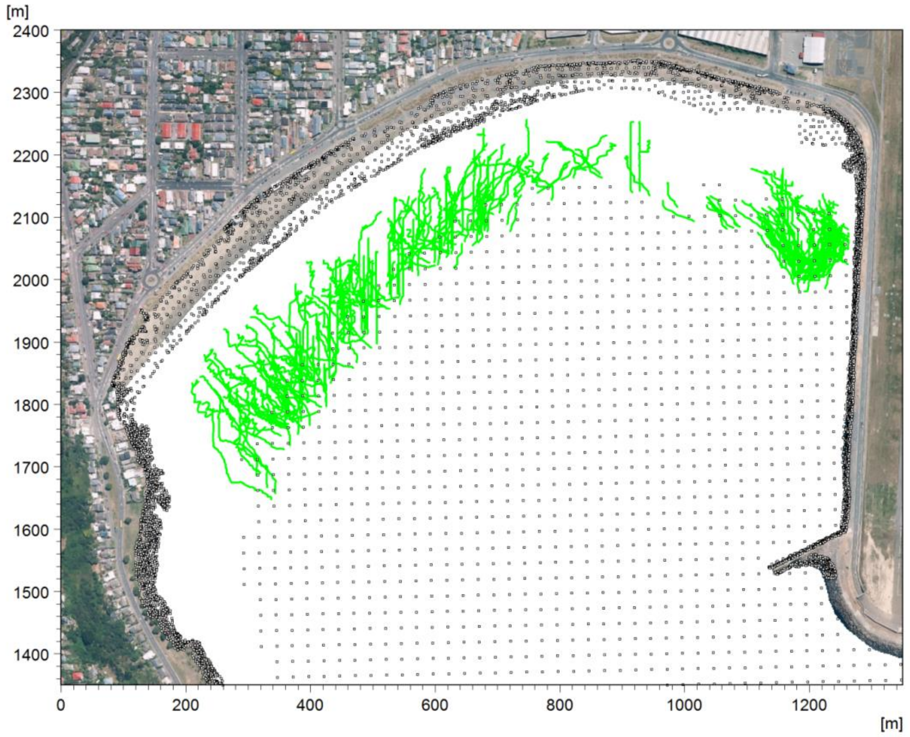

Figure 7-1 Scenario 1 OPTISURF Overlaid on Raw Bathymetry.

It is recommended that a LiDAR or multi-beam survey is carried out (during calm conditions) in order to capture the full bathymetry layout. Ideally several surveys with four to six months spacing could be supplemented to document cyclic natural changes in nearshore bed morphology. The survey should be repeated four to six months after construction allowing for affirmation of expected changes to the sediment budget inside the bay.

7.3 Surfing Amenity

Historically, evidence for the quality of a surf break has been reliant on either photos or anecdotal observations, with inherent flaws with both types of information. Anecdotal observations have the potential to be exaggerated and/or the frequency of occurrence higher than what occurred in reality. Photos, although providing a great snap shot in time of the quality of a breaking wave (typically taken at point when wave begins to break), do not provide further information about the ultimate fate of the wave and the actual characteristics of the ride (such as length of ride) are only a guess.

Recent advances in compact waterproof tracking technology have made it possible to quantify surfing amenity in an unprecedented cost effective manner. Small cheap water proof tracking devices fitted with high frequency GPSs and accelerometers can be fitted to the boards of more than 10 surfers during a surf session during which all of their surf rides (lengths, speed and location) will be recorded. Output data is directly comparable with OPTISURF output and can be used to confirm model predictions for the pre and post construction layout. An example of measured output is presented in **Figure 7-2**.

This type of monitoring could be used as a basis to fulfil any consent conditions required to confirm the findings of numerical investigations and prove the success of any proposed mitigation solutions.

DHI have had great experiences with coordinating these types of monitoring campaigns in the recent past on the Australian Gold Coast.

We recommend that WIAL consider moving forward with this type of data collection quickly to insure a good data set for the existing conditions is obtained

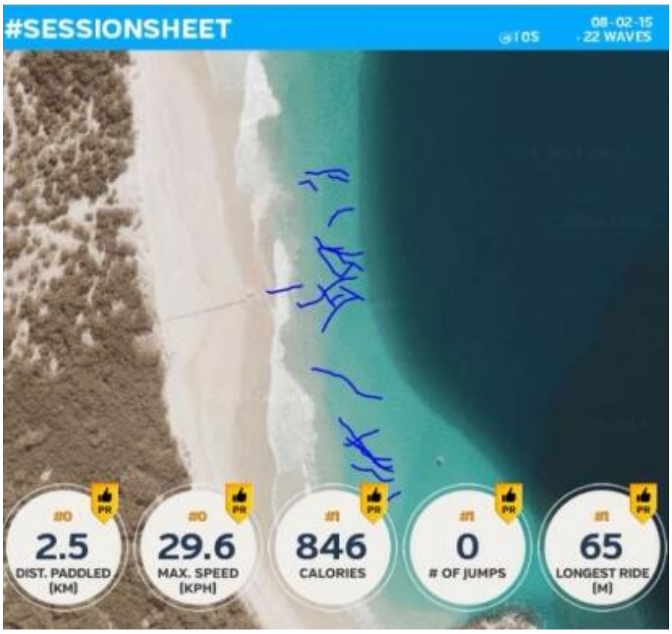

Figure 7-2 Surf Session sheet presenting an overview statistic of measured surf rides (1 surfer) for South Stradbroke Island, Australia.

8 Submerged Wave Focusing Structure Feasibility Assessment

The section outlines the feasibility assessment undertaken for a submerged wave focusing structure for Lyall Bay including an overview of the concept and a preliminary design.

8.1 Wave Focusing Concepts and Surf Quality

In areas of featureless nearshore bathymetry, ‘closeout’ waves are regularly experienced. Such closeout waves allow a surfer only a very short ride before the full length of the wave crest breaks at the same time. Surfing conditions can be significantly improved by reducing the number of closeouts and increasing the number of waves that initiate breaking from a pronounced ‘peak’ thus offering a longer ride. The process of wave focusing (a magnification of wave height) across offshore bathymetry can result in the occurrence of defined peaks in the leeward surf zone.

Put into natural context, the unique layout of both South Stradbroke Island (**Figure 8-1** and **Figure 8-2**) and Duranbah Beach as surf breaks can be attributed to their local and offshore bathymetry. Submerged ebb tidal delta depositional shoals are located offshore at both of these locations. An ebb tidal delta is a sand bar or shoal formed in the seaward side of an inlet by the combined action of wave and tide dominated processes (Masselink and Hughes, 2003). This shallower section of bathymetry refracts incoming waves and can focus (and magnify) wave energy onto various locations of the leeward beach. The presence of submerged ebb tidal delta depositional shoals at both of these locations plays a significant role in breaking up the long, straight swell lines at these beaches and creating magnified larger waves that break from distinctive peaks with a suitable peel rate for surfing (**Figure 8-2**).

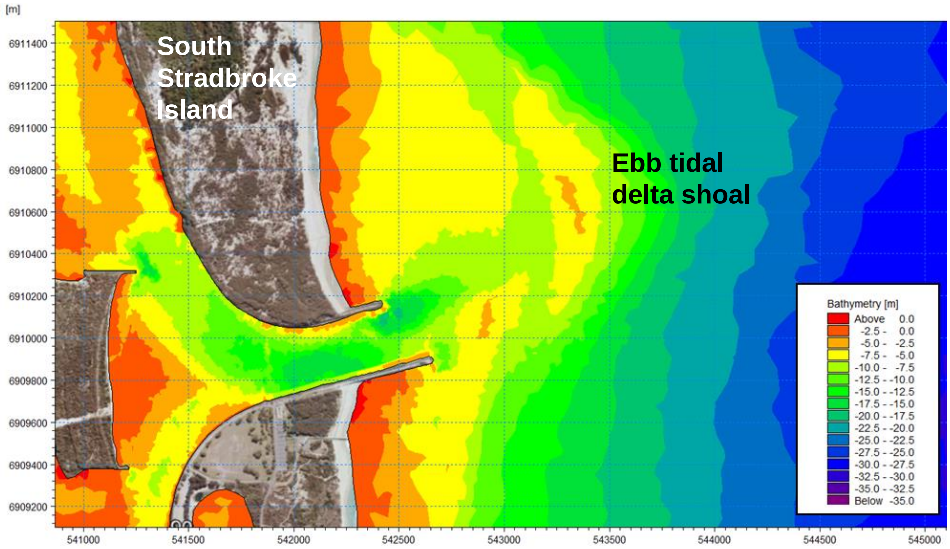

Figure 8-1 Ebb tide delta shoal offshore of South Stradebroke Island.

Figure 8-2 Wave breaking from defined peaks at South Stradebroke Island (Wannasurf, 2015).

A number of studies have investigated the influence of offshore bathymetry on wave focusing in the lee of the bathymetric feature and subsequent improvements of wave breaking quality lending to improved surfing conditions (West (2002), Beamsley and Black (2003), Pitt (2009) and Pitt 2010)).

Pitt (2009) investigated the influence of natural offshore rock formations of raised bathymetry, referred to as 'bomboras', on adjacent surfing conditions. Within New South Wales, Australia twenty six of the most popular surfing locations were found to have bombora formations offshore, including but not limited to; North Narrabeen, Old Bar, Pelicans, Bellambi, Bendalong, Narrawallee and Congo. Other popular surfing spots further afield also have the same bathymetry arrangement of bombora controlled beach breaks, including; Pitta St. Peregian, (Queensland, Australia), Woolamai (Victoria, Australia), 13th Beach (Victoria, Australia) and Gisborne Pipe (New Zealand). At these locations surfing is undertaken over sand in the lee of the bomboras.

Pitt (2009) measured and documented the influence and advantages of other such offshore bomboras from a surfing perspective, concluding:

- Bathymetric features of bomboras encourage wave refraction and wave amplification to 'pre-condition' breaking waves.
- The bomboras focus advancing waves to a more certain location on the beach.
- Wave heights in the lee of bomboras were larger than a factor 1.25 depending on wave period and swell angle.

- Waves leeward of bomboras are more likely to break as a peak (rather than close out) and therefore offers a longer length of ride.

West (2002) undertook a study into the potential application of designing artificial wave focusing reefs to enhance surfing in the lee of an offshore structure. The concept is illustrated in **Figure** 8-3. West (2002) investigated the required artificial reef dimensions and established a parameter referred to as the West-Cowell surfing reef factor which relates reef cross-section to wave focusing.

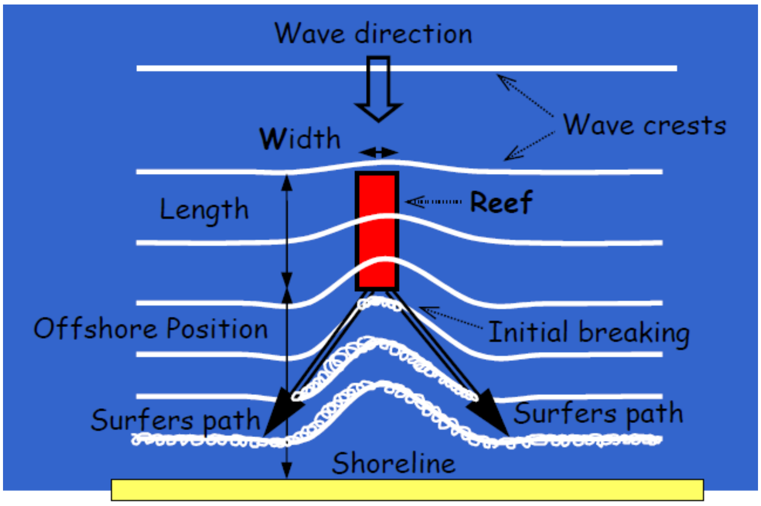

Figure 8-3 Concept of a wave focusing reef (West, 2002).

An example of anthropogenic modifications to offshore bathymetry resulting in an increase of wave focusing is reported by Bancroft (1999) in a performance monitoring study of the Cable Station artificial surfing reef in Western Australia. The Cable Station artificial surfing reef was completed in December 1999 and funded by the Government of Western Australia. The reef was designed at the University of Western Australia for the sole purpose of recreation (Pattiaratchi, 2003) and more specifically, the improvement of surfing conditions (West (2002) and Strauss (2011)). The main intention of the reef was to generate improved surfing conditions by wave breaking on the reef. However, the shore break leeward of the reef was also reported by Bancroft (1999) to have been improved by the focusing effect of the artificial reef and more rideable waves were being produced.

8.2 Feasibility for Lyall Bay

8.2.1 Preliminary Design Functionality

A strictly preliminary design of a wave focusing structure was developed to demonstrate the potential feasibility of a wave focusing structure for Lyall Bay. The purpose of the structure is to focus wave energy towards a shoreward location along the middle or western section of Lyall Bay thereby forming a local wave peak. This will result in larger waves, longer ride lengths and an increase in overall surf amenity. A strictly conceptual layout of how such structure could look like is shown in **Figure 8-4** and **Figure 8-5**. Approximate dimensions of the structure are given in **Table 8-1**.

The main feature of the structure is a curved shoaling platform designed to refract approaching waves towards the middle, focusing wave energy into a peak in the lee of the structure which will cause waves to break over the natural inshore bathymetry. The crest level of the structure is to be sufficiently deep so that waves only break on the crest during rare occasions of very large waves. The structure must be positioned a distance sufficiently west of The Corner to avoid any reducing in wave energy propagating into the existing surf break.

To avoid adverse effects on the shoreline and The Corner surf break, while also enhancing surf amenity in Lyall Bay and ensuring the structural integrity of structure, the dimensions and location of the structure would need to be optimised, through a concept design study.

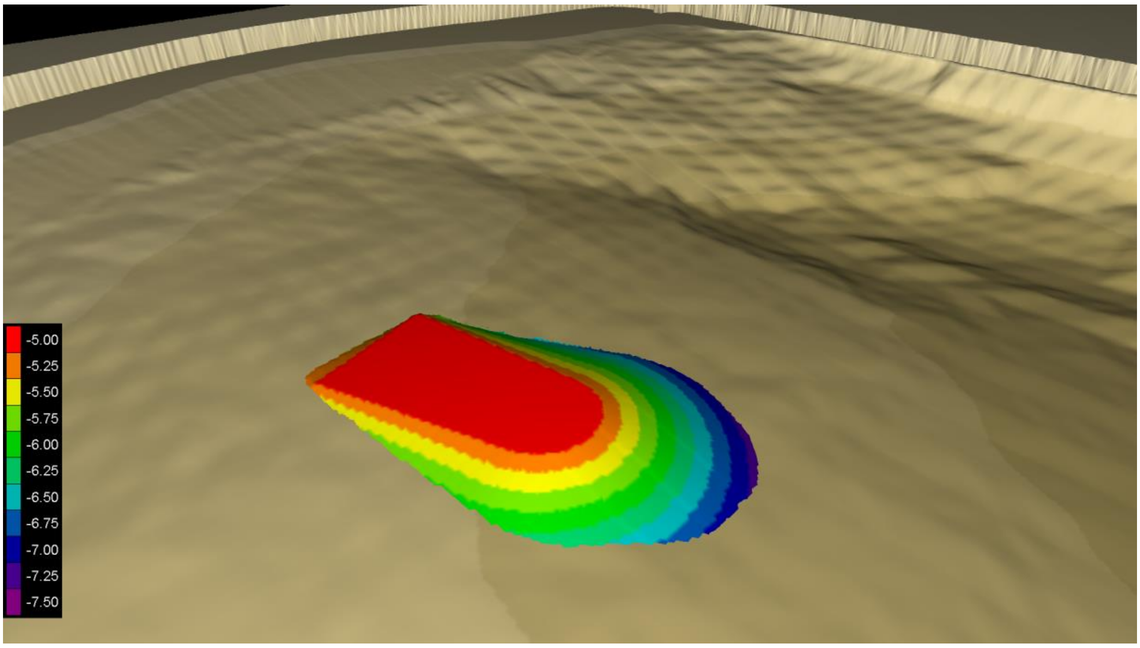

Figure 8-4 Conceptual Submerged Focusing Structure Geometry.

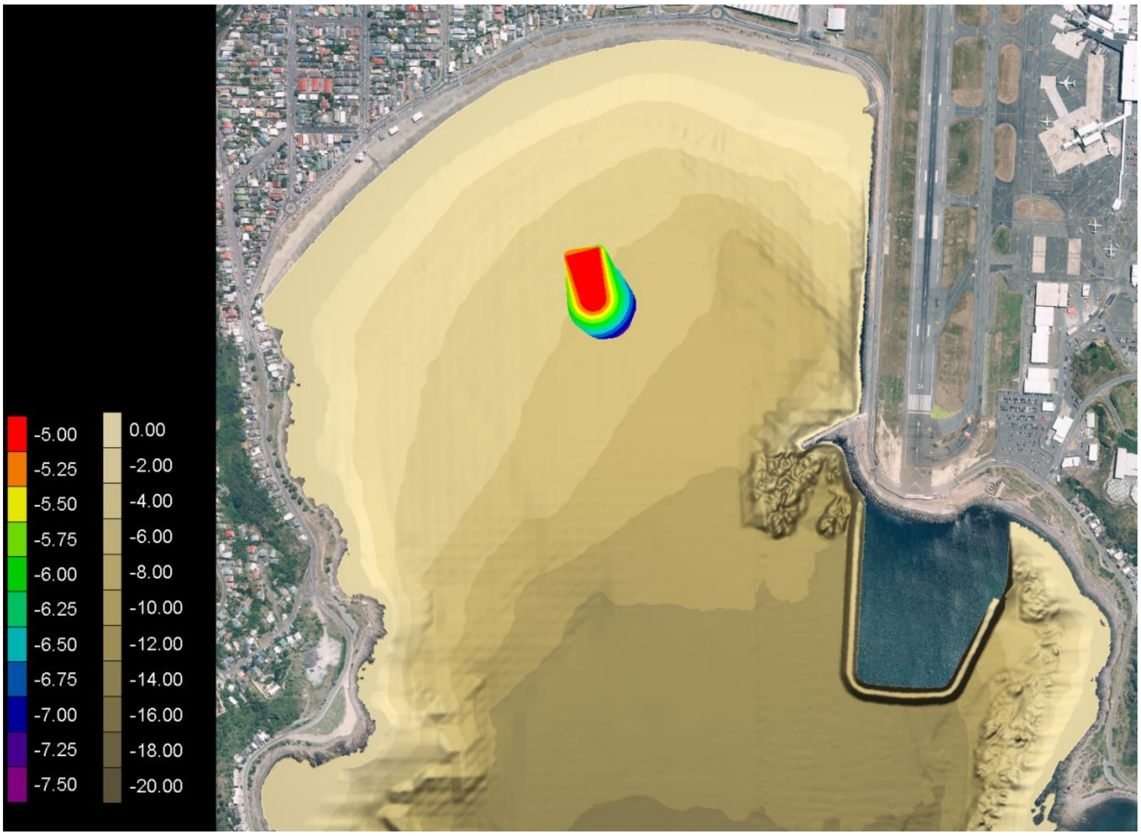

Figure 8-5 Submerged Focusing Structure Geometry (Plan View).

Table 8-1 Approximate Dimensions of Submerged Focusing Structure.

Specification	Value
Volume	17,000m^3
Footprint	20,000m^2
Crest Depth	-5.0m
Length	180m
Width	140m
Toe Slope	1 in 30
Distance from shore to deepest point	450m

8.2.2 Preliminary SWFS OPTISURF Results

OPTISURF was run for Scenario 1 (**Table 5-2**) with and without the submerged focusing structure. **Figure 8-6** and **Figure 8-7** demonstrate the longer right and left rides with larger wave face heights that form in the lee of the structure. Note that no rides are present over the structure as waves in general do not break on the crest.

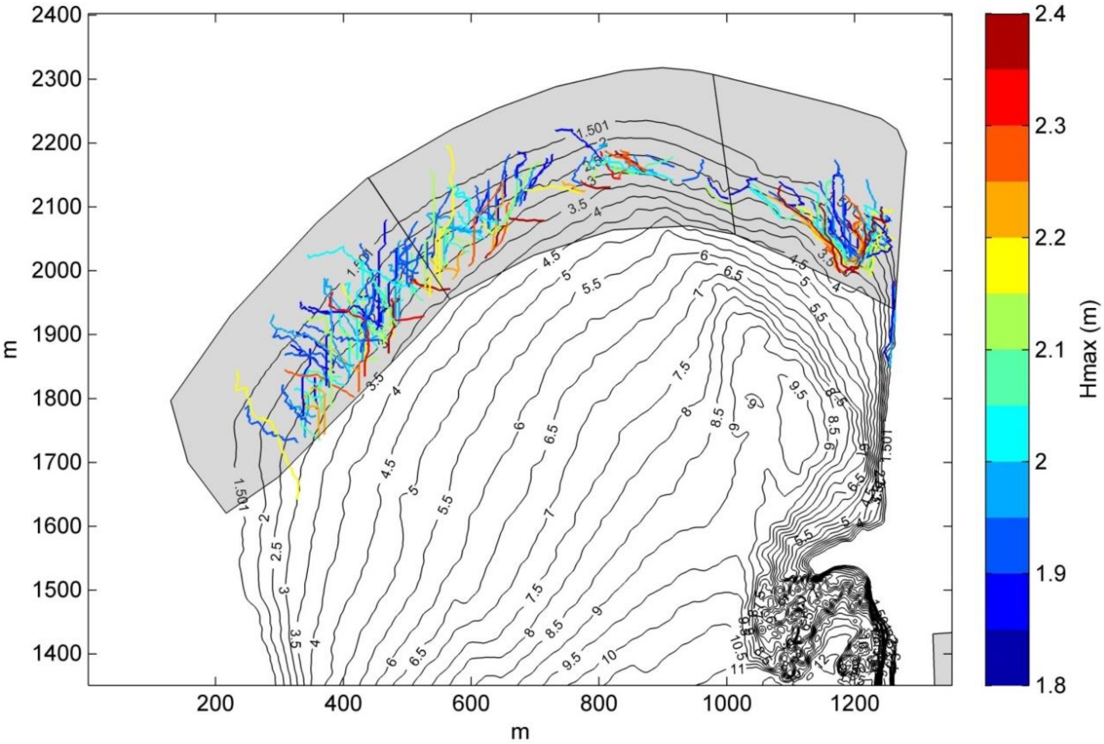

Figure 8-6 OPTISURF plot for Scenario 1 Airport Extension.

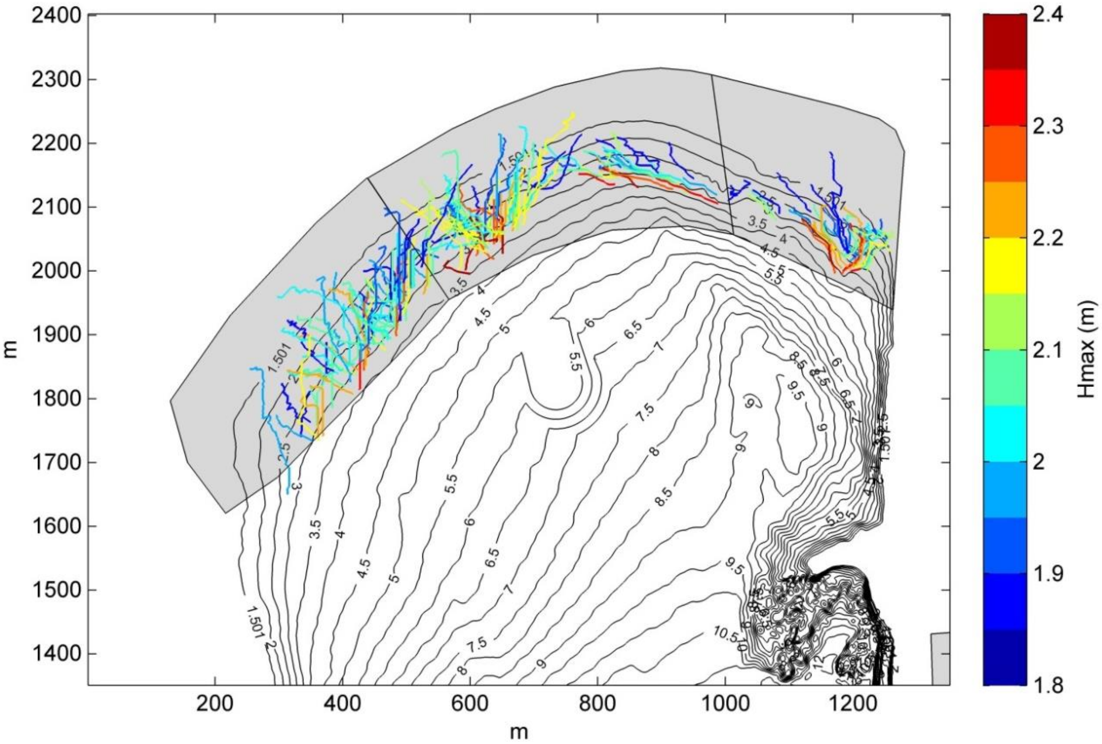

Figure 8-7 OPTISURF plot for Scenario 1 Airport Extension with Submerged Focusing Structure.

Figure 8-8 and **Figure 8-9** consists of a filtered version of **Figure 8-6** and **Figure 8-7** where only surf rides with maximum face heights greater than 2.1m are presented. The filtered plots makes it easy to illustrate improved surfing conditions at Middle Beach provided by the focusing structure shown by the increased frequency, length and wave face height of surfable waves along this part of the bay. A slight reduction in ride number and wave height can be observed in The Corner as caused by the SWFS. Although the impact is relatively small, further revisions of the placement of the structure would involve ways to eliminate this negative impact to the surf quality most likely by moving the structure further to the west.

Significant wave height plots were also included to demonstrate the effect of the structure on overall wave height. **Figure 8-10** and **Figure 8-11** show significant wave height distribution for Scenario 1 with the Airport Extension with and without the focusing structure respectively.

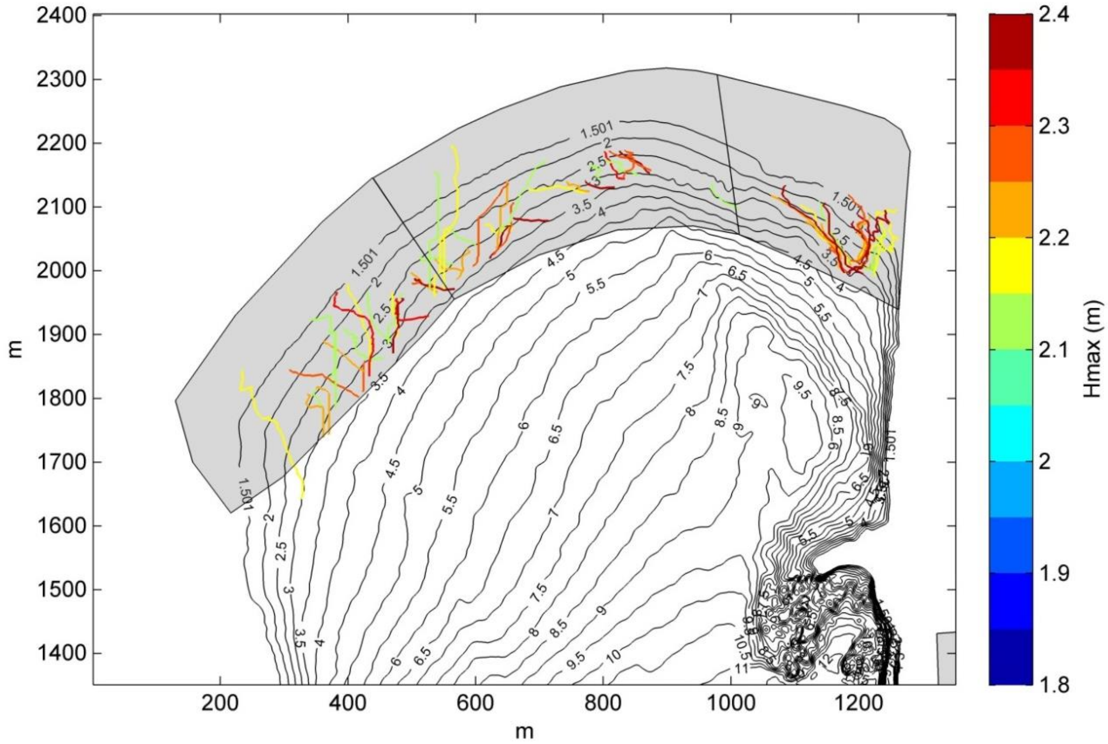

Figure 8-8 OPTISURF plot for Scenario 1 Airport Extension (2.1m Minimum).

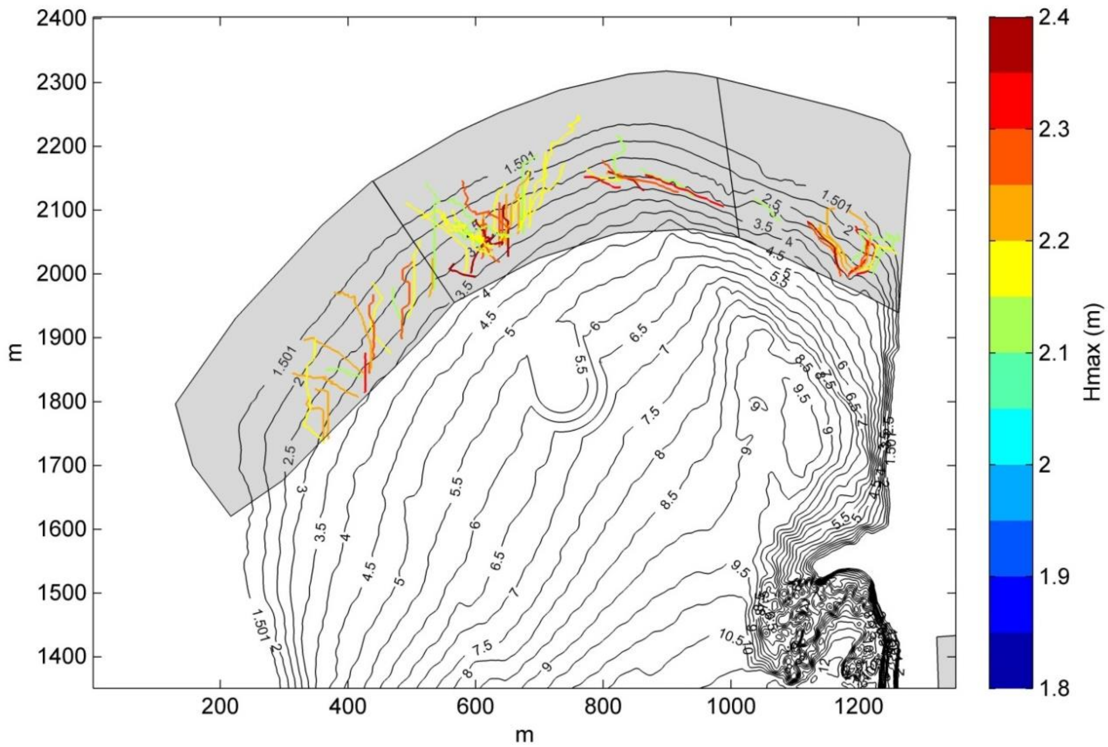

Figure 8-9 OPTISURF plot for Scenario 1 Airport Extension with Submerged Focusing Structure (2.1m Minimum).

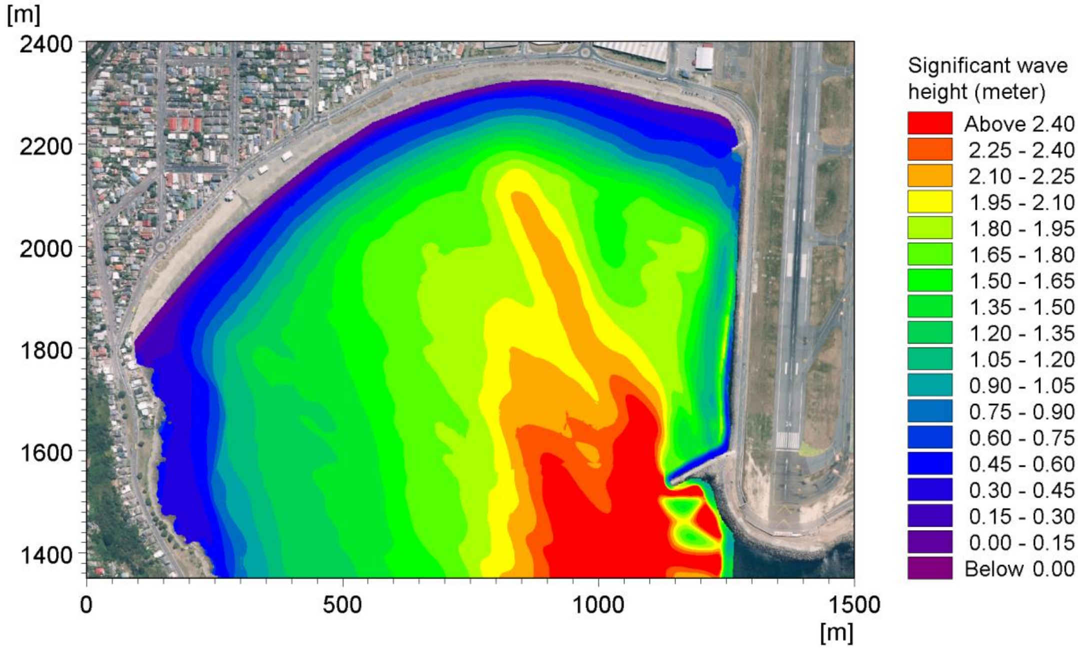

Figure 8-10 Phase Averaged Significant Wave Heights for Scenario 1 Airport Extension.

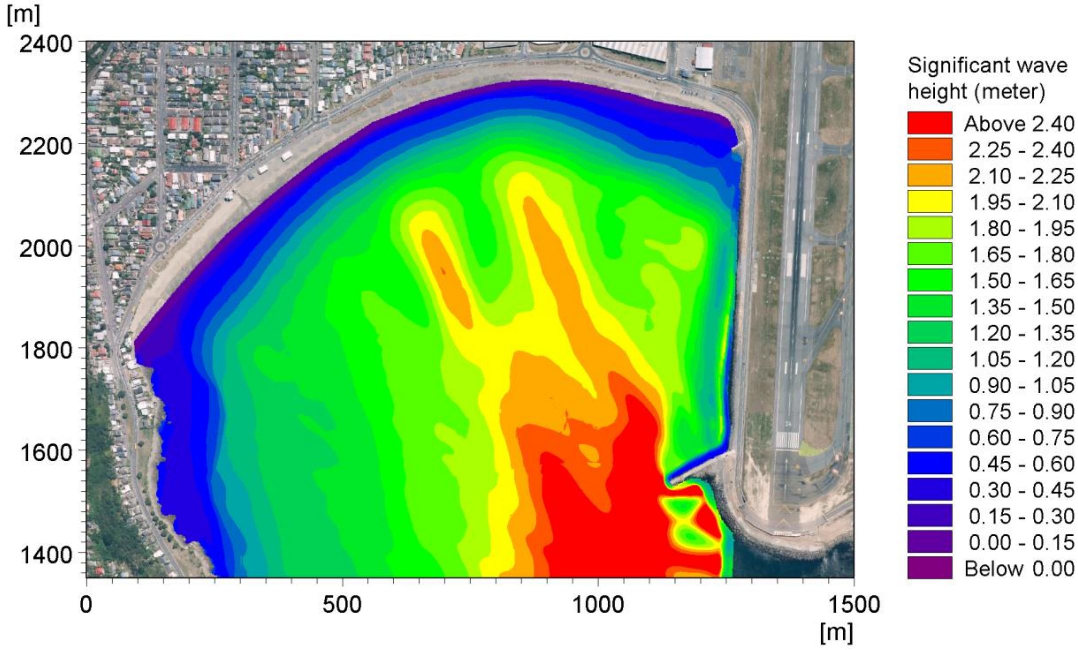

Figure 8-11 Phase Averaged Significant Wave Heights for Scenario 1 Airport Extension with Submerged Focusing Structure.

8.2.3 Preliminary Design Considerations

8.2.3.1 Material Selection

It is recommended that rock would be the most suitable material for constructing the wave focusing structure due to its proven durability in the marine environment and its ability to meet design objectives. Rock has a long history in coastal defence structures and protection works throughout the world. It is a proven material for marine construction and provides excellent levels of durability. Numerous detached and submerged breakwaters have been constructed around the world for shoreline protection.

A recent review carried out in DHI (2014) found that all reviewed submerged reef designs using geotextile sand containers had faced problems with bag breaking and stability. Until further improvements are documented DHI does not recommend the use of geotextile containers for submerged structures.

The choice of construction material will result in some level of porosity of the structure, which will affect both structural stability and wave dissipation over the structure. In conventional rock armouring structures a high porosity can be desirable as it improves stability and limits the size requirements. Porosity of rock breakwaters can be up to 0.4.

However for a SWFS it is recommended to limit porosity in order not to introduce too much wave energy dissipation over the structure, which will work against the intended wave focusing principle.

A large body of research has been completed on the stability of rock structures in the marine environment, including submerged breakwaters. Typically used equations for assessing the stability of breakwaters and revetments are Hudson (1959) and Van der Meer (1988). Van der Meer (1991) also looked at submerged breakwaters and developed equations for submerged structures. However this work was only undertaken for 1 in 2 slopes, so is not relevant for the proposed gentle 1 in 30 sloping face of the SWFS.

Generally, shore protection structures are looking to be as cost efficient as possible by reducing the volume of rock required; therefore the slopes are rarely flatter than 1:4. Accordingly limited work has been undertaken for low sloping rock structures. Some research has been undertaken by De Waal and Van der Meer (1992) on the testing of 1:3, 1:6 and 1:8 slopes. This was primarily focused on wave run-up which validated Van der Meers (1988) formula.

Using Van Der Meers formula the required minimum rock armouring D50 diameter becomes approximately 0.7 – 0.8 m. The number assumes a conservatively low permeability of 0.2 and a depth limiting maximum wave height of 5.5 meters with a T_z of 12 to 15 seconds. The rock density has been assumed to be 2700 kg/m^3 (Greywacke). If the rock slope was assumed to be 1:8 the rock sizes could be as large as 1.5 m in diameter.

It is emphasised that the above estimates can only be utilised as a very rough guideline. Detailed investigations backed up by scaled physical model tests are recommended in order to confirm appropriate sizes for the rock armouring layer.

8.2.3.2 Material Sourcing

The airport extension will be protected with rock revetment structures comprised of rock units require a large scale source of rock. Therefore logistics for providing the additional amount of rock for the wave focusing structure is expected to be not problematic.

8.2.3.3 Material Placement

It is important the machinery used for placement can be operated with a high level of accuracy in order to assure that the construction layout meets the design specifications. One alternative is to use a barge mounted long-arm excavator (**Figure 8-12**). An interim construction layout of the airport extension may be adopted to use the extended rock revetment walls to provide temporary shelter and mooring for the barge during construction.

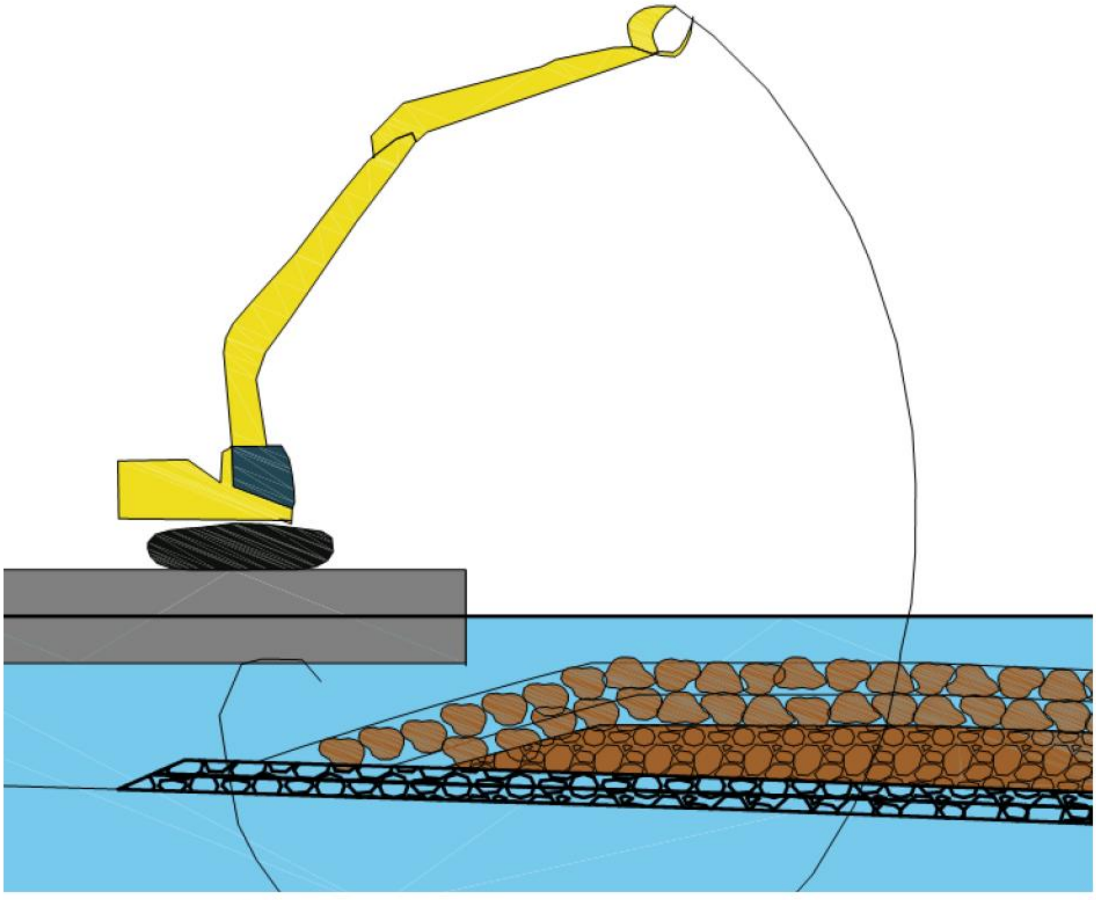

Figure 8-12 Barge mounted long-arm excavator (Image sourced from DHI (2014)).

8.2.3.4 Costing

Construction costs will depend highly on the local availability of suitable material and the rock size required for detailed design. Mobilization costs and allowance for downtime due to bad weather all contribute significantly to the overall budget and requires detailed assessment. Due to the existing concurrent construction of the airport extension it is considered possible that some of these costs could be reduced substantially.

The WIAL has provided an estimate of $80/tonnes for rock for the airport extension project.

Assuming a low porosity of 0.2, rock density of 2700 kg/m3 the estimated material cost for a 17,000m^3 structure is NZD $2.9 million.

Detailed investigations of cost optimization options regarding the SWFS are highly recommended as this considered one of the key limiting factors with regards to its overall feasibility.

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APPENDICES

APPENDIX A – Offshore Wave Transformation

A.1 Offshore Wave Transformation

Since there is no long term wave data set for Lyall Bay, a wave transformation study was required to provide directional wave data at the entrance of Lyall Bay. From this data an analysis of the different types of wave conditions that will occur for Lyall Bay and the frequency for when these wave conditions occur could be carried out. Although wave data has been collected at Baring Head since 1995, directional data has been collected since 30th January 2014. As a result only a year wave data set could be generated for Lyall Bay. This was deemed a satisfactory length of time to determine the representative range of conditions likely to occur at Lyall Bay.

A.2 Summary of Available Wave Data

A data collation exercise was undertaken to source suitable wave datasets within the study area. Wave data from three instruments deployed within the study area were provided to DHI by NIWA. These include data from Site 1 (located in the nearshore in close proximity to the runway), Site 2 (located off of the headland to the southeast of Waitaha Cove) and Site 3 (located approximately 1.5km offshore of Baring Head). An overview of the deployment locations is shown in **Figure A1** with the coordinates provided in **Table A1**.

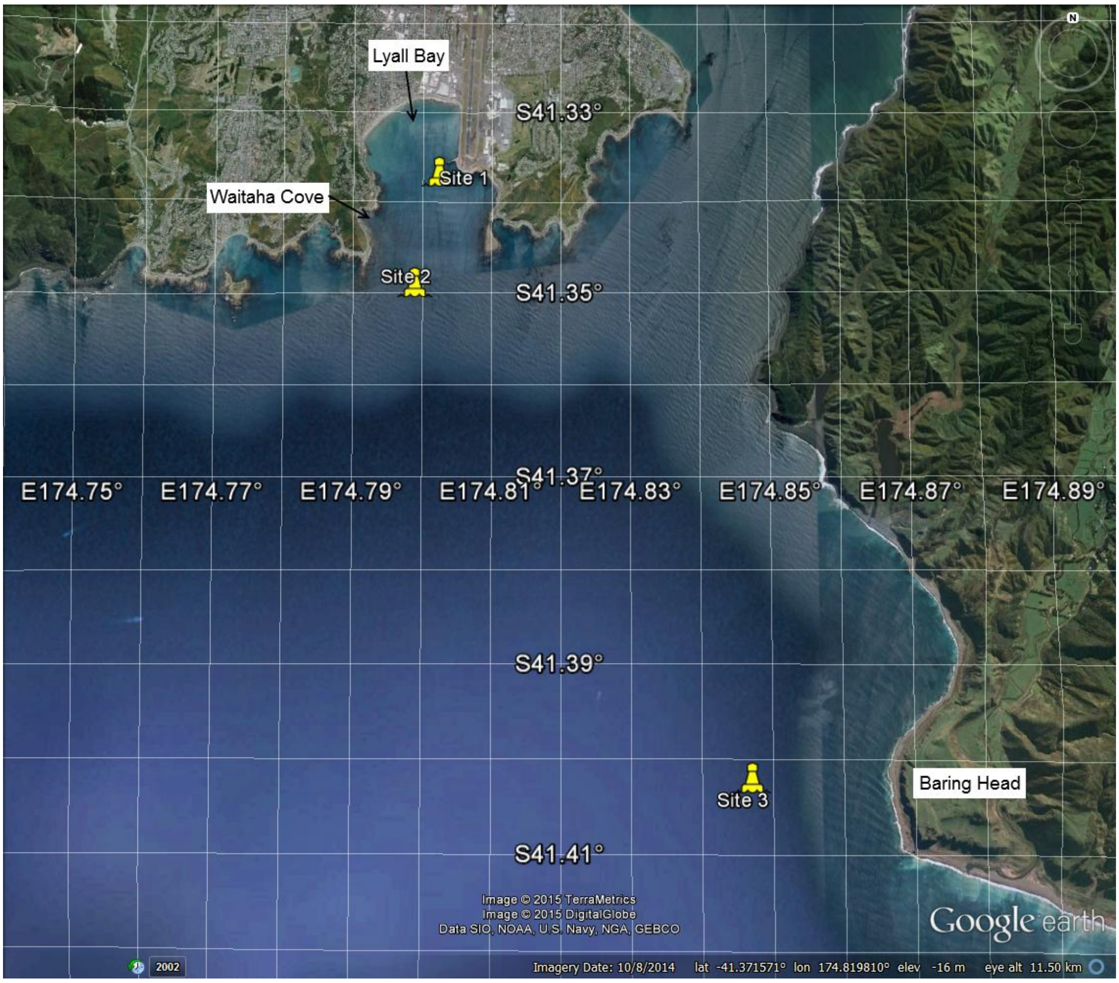


Figure A1 Locations of wave data

Table A1 Locations of wave data used to transform waves from Baring Head

Site No.	Instrument	Longitude	Latitude	Deployment Period
1	ADCP (nearshore)	174.8023	-41.3366	18-Aug – 09 Oct 2014
2	Dobie (offshore)	174.7989	-41.3489	04 Sept – 09 Oct 2014
3	Baring Head Waverider	174.8470	-41.4020	30 Jan 2014 – 18 Feb 2015 *

* For this study data from the waverider buoy at Baring Head has only been used for the period where direction was also recorded.

Wave data at Sites 1 and 2 were measured using an Acoustic Doppler Current Profiler (ADCP) device for the duration of 4 to 6 weeks. At Site 1 significant wave height (Hs), peak wave period (Tp) and mean wave direction (MWD) were measured, compared to Hs and zero crossing period (Tz) at Site 2. Data measured at Site 3 includes Hs, Tp, mean period (Tm02) and peak wave direction (Dirp). Data from Site 3 has only base used in this study since the 30th January 2014 when wave direction has been recorded.

A.3 Development of Wave Conditions

Between the 30th January 2014 and the 18th February 2015 the waverider buoy off of Baring Head recorded 17993 wave conditions. During this period many of these wave conditions had similar characteristics with regard to wave height, period and direction. In order to reduce the computational overhead of running each wave condition during the period a matrix approach was undertaken to bin the data.

To generate a matrix of suitable wave conditions from the Baring Head waverider buoy data (Site 3) the data was binned for directions between 130 to 320 degrees (true north) in 5 degree increments, and for peak wave periods between 2 to 18 seconds in 1 second increments. A total of 654 conditions were identified.

These wave conditions were used to provide boundary conditions for the wave transformation modelling. An overview of the raw peak wave period and wave direction data at Baring Head in, in addition to the 654 wave conditions within the matrix. These conditions are shown in **Figure A2**.

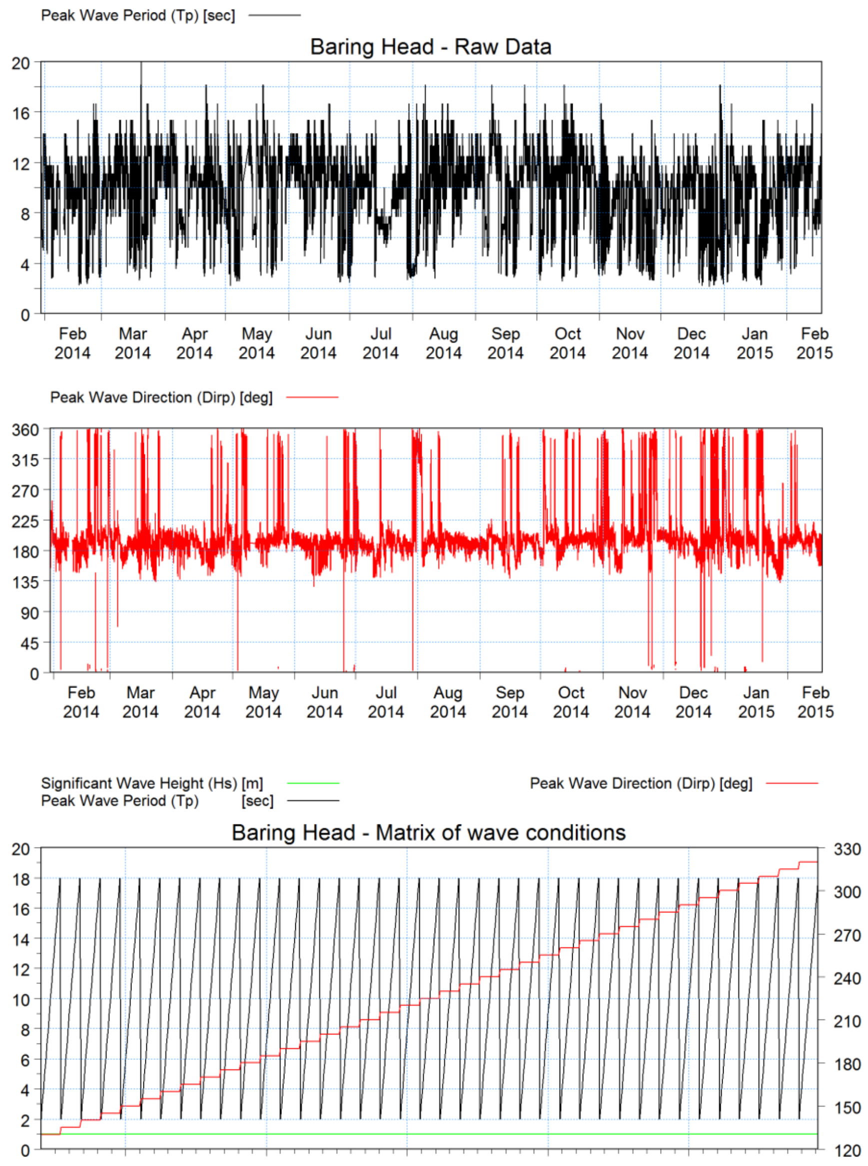


Figure A2 Summary of raw wave data (top and middle plots) and matrix of wave conditions (bottom plot) for Baring Head

A.4 Wave Transformation Modelling Methodology

The wave transformation modelling was undertaken using our MIKE 21 Spectral Wave (SW) Model FM which is a state-of-the-art numerical tool for prediction and analysis of wave climates in offshore and coastal areas. The main computational features of MIKE 21 SW include:

- Fully spectral and directionally decoupled parametric formulations,
- Source functions based on state of the art third generation formulations,
- Instationary and quasi-stationary solutions,
- Optimal degree of flexibility in describing bathymetry and ambient flow conditions using depth-adaptive and boundary-fitted unstructured mesh,
- Coupling with hydrodynamic flow model for modelling of wave-current interaction and time-varying water depth,
- Flooding and drying in connection with time-varying water depths,
- Cell centred finite volume technique,

- Fractional step time-integration with a multi-sequence explicit method for the propagation, and
- Extensive range of model output parameters.

A.4.1 SW Model Setup

This section provides details of the SW model setup for the wave transformation modelling.

Domain extent and model bathymetry

The domain extent of the SW model extends from near Cave Bay to the west and the Rimutaka Forest Park to the east. The model includes from Wellington Harbour extending approximately 18km offshore from the harbour entrance.

The bathymetry data for the SW model was compiled from two sources, (i) post processed 25m gridded bathymetric data of Lyall Bay provided by NIWA (2014) and (ii) sounding data from Jeppesen Marine's C-MAP Worldwide Electronic Chart Database Professional+ (MIKE by DHI, 2014) to cover the remainder of the model domain extent.

An overview of the SW model bathymetry is shown in **Figure A3** for the entire domain extent and a zoom in for Lyall Bay in **Figure A4**. The wave buoy locations have also been included for reference.

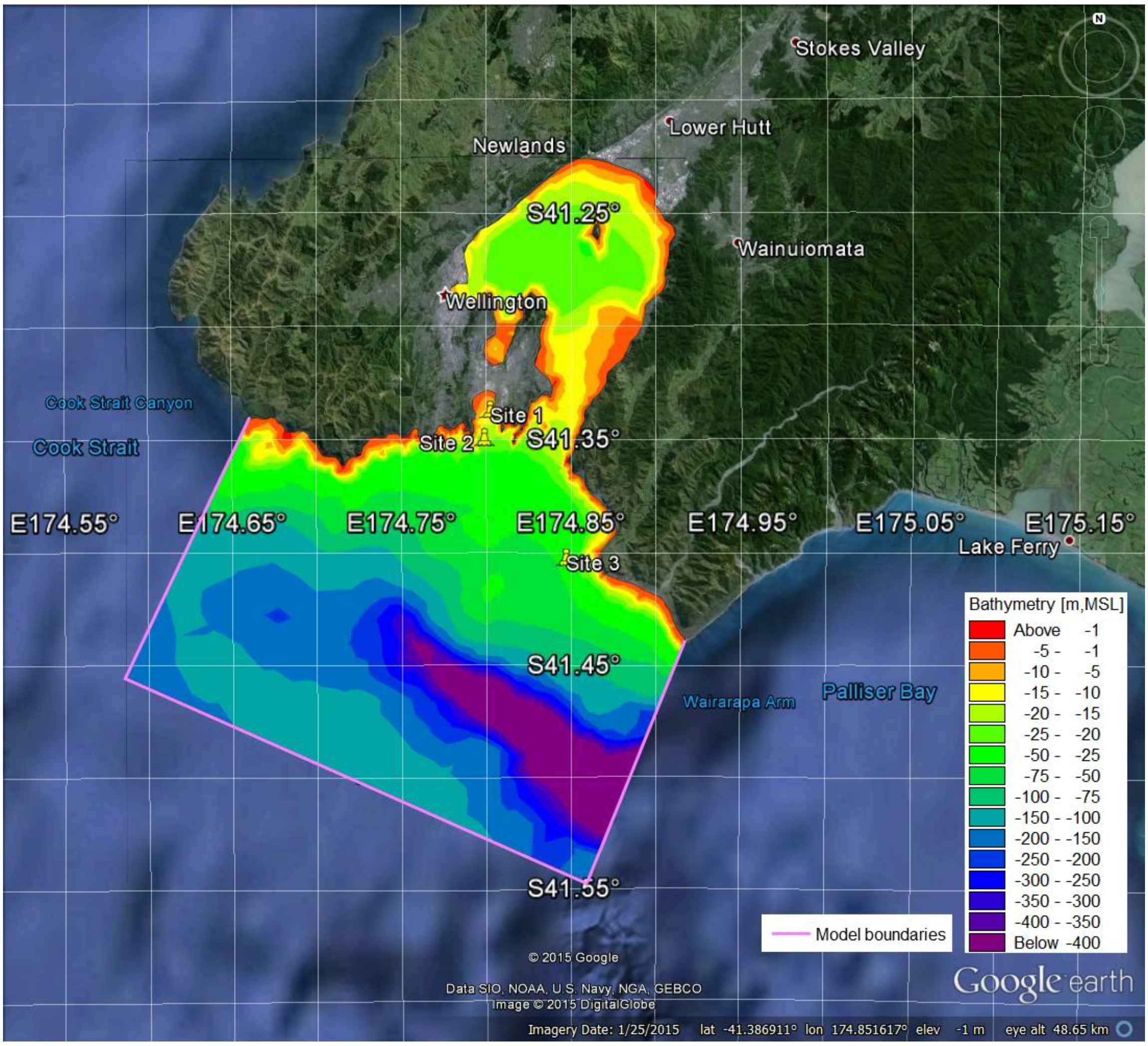


Figure A3 SW model domain extent, bathymetry and boundaries

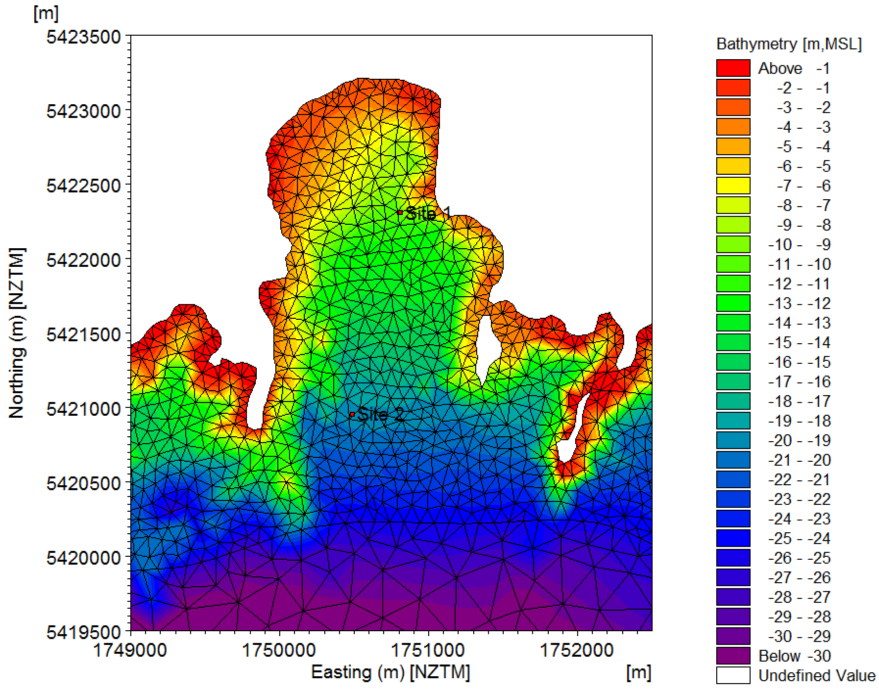


Figure A4 SW model bathymetry and mesh for Lyall Bay

Model parameters

The SW model was run using a fully spectral formulation in quasi stationary mode. The directional spectral discretisation was set to 5 degrees, and a minimum frequency of 20 seconds was applied. Diffraction, wave breaking and white capping have been included within the model setup. A spatially varying bottom friction map was applied to the model domain, specifying a lower bed roughness inside Lyall Bay. Wind, variations in water levels and current conditions were not concluded within the model setup.

Model boundaries

The 654 wave conditions from the Baring Head waverider buoy data defined in Section A.3 were applied to the three offshore boundaries within the model (as shown by the pink line in Figure A3). The significant wave height was set to 1m for all wave conditions and a suitable directional spreading factor for swell waves was applied.

Model outputs

Timeseries data, including significant wave height, peak wave period, wave period T02, peak wave direction and directional standard deviation, were saved at each of the three wave buoy locations. The same parameters were also saved as spatial data covering the entire model domain.

A.4.2 SW Model Results

The raw SW model results (prior to wave transformation) are shown in **Figure A5** for Hs, Tp and MWD at each of the three wave buoy sites. As expected the wave heights increase towards the shore, as the bathymetry shallows. The Tp is typically the same at all three sites, with some variation visible for the smaller wave conditions. The trend in MWD results varies for wave conditions with directions approximately less than 180 degrees and approximately greater than 180 degrees. Where wave conditions at Baring Head (Site 3) are less than 180 degrees this typically results in wave directions at Sites 1 and 2 being smaller than Site 3 as the waves diffract around the headland. For wave conditions with directions greater than 180 degrees (with a greater westerly component) MWDs are greater at Site 3 compared to Sites 1 and 2, due to refraction which tends to ‘swing’ the wave crest to an alignment parallel with the bathymetry contours.

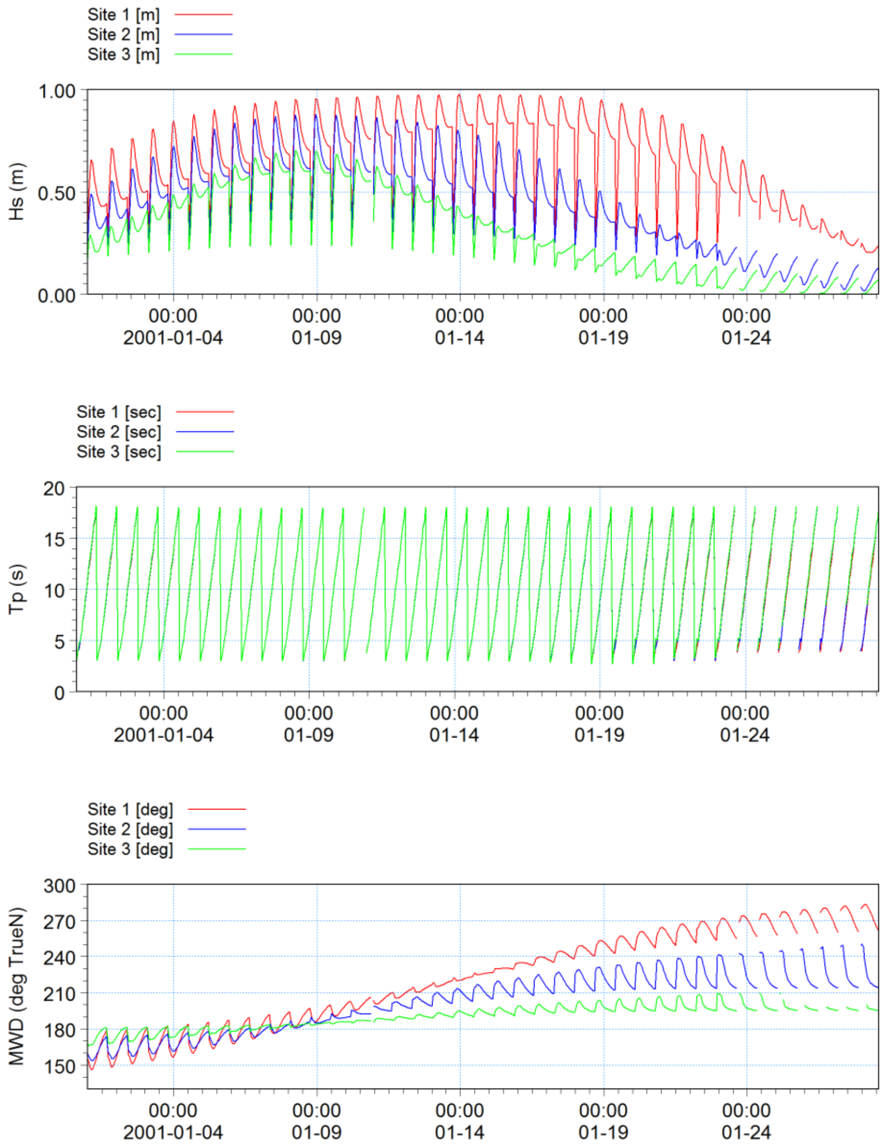


Figure A5 SW model results shown for Hs, Tp and MWD at Sites, 1, 2 and 3

A.5 Wave Transformation Methodology

Following completion of the SW model simulation for the 654 wave conditions MATLAB was used to generate 3D surfaces for Hs and MWD to transform data from Site 3 to Sites 1 and 2. The following steps were undertaken to generate the wave transformation surfaces:

- Run the SW model for all 654 wave conditions,
- Extract Hs and MWD data from the model output at Sites 2 and 3,
- Calculate the ratio between Sites 3 and 2 for Hs and MWD for all wave conditions,
- Take the ratios and bin the conditions at Site 3 in 0.1 second increments for Tp and 0.5 degrees increments for MWD,

- Extrapolate and interpolate the conditions to create a surface which covers all possible wave conditions,
- Use the 3D surface to look up Hs, Tp and MWD values at Site 2 (Note: the MATLAB program will interpolate between the four nearest grid points of binned data to further refine the value obtained),
- Repeat the process for Site 1.

The wave transformation surfaces for Site 2 are shown in **Figure A6** and **Figure A7** for Hs, **Figure A8** and **Figure A9** for Tp, and **Figure A10** and **Figure A11** for MWD.

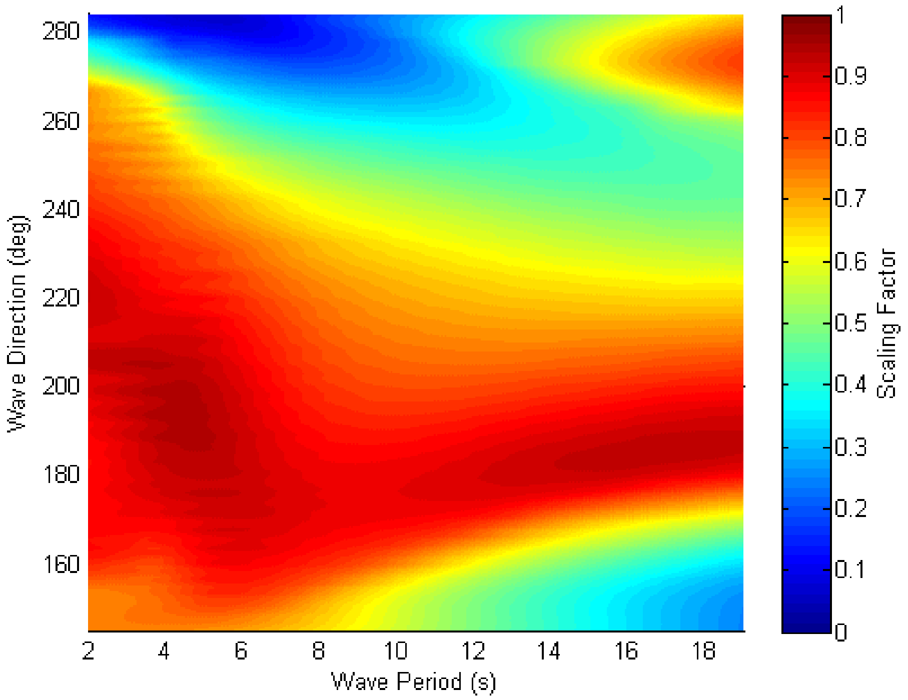


Figure A6 Transformation surface for Hs at Site 2

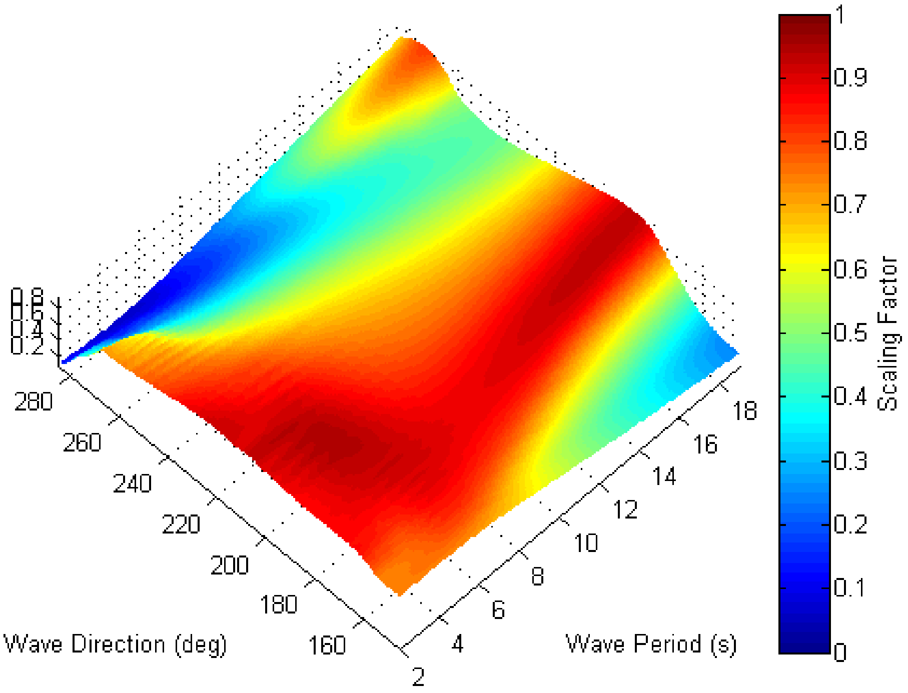


Figure A7 3D transformation surface for Hs at Site 2

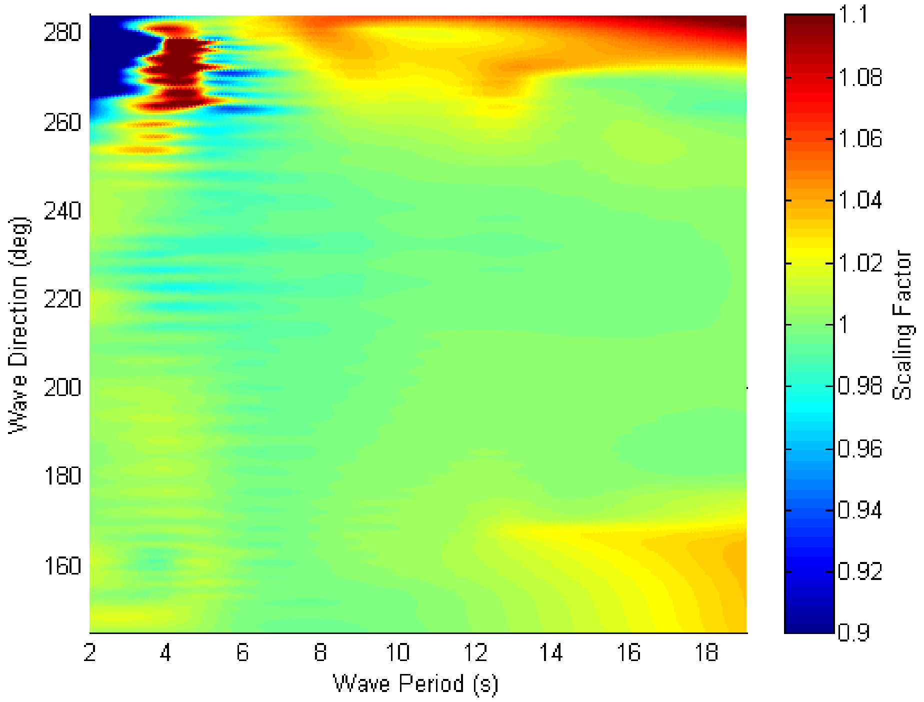


Figure A8 Transformation surface for Tp at Site 2

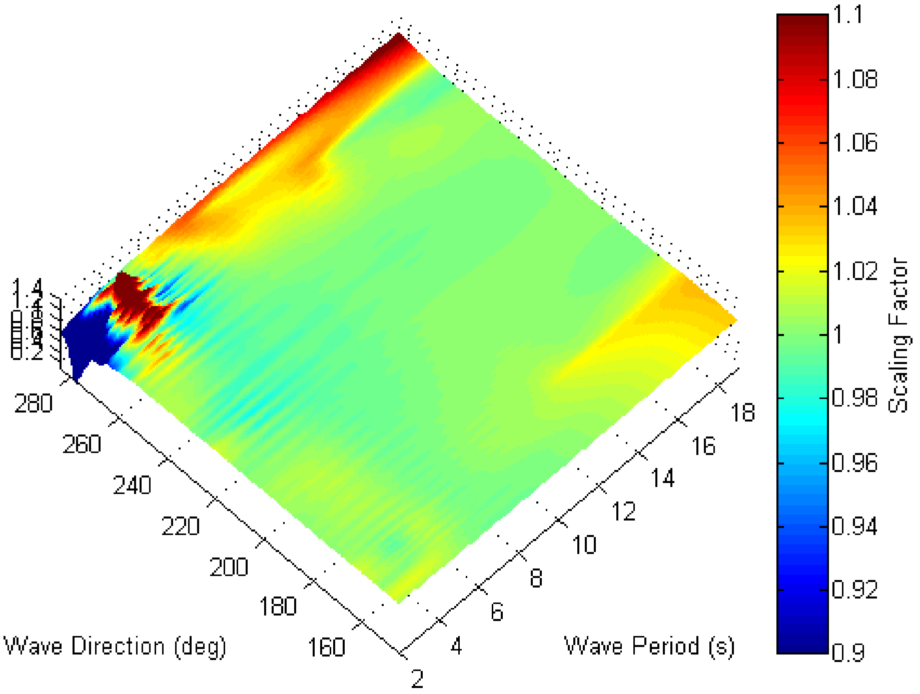


Figure A9 3D transformation surface for Tp at Site 2

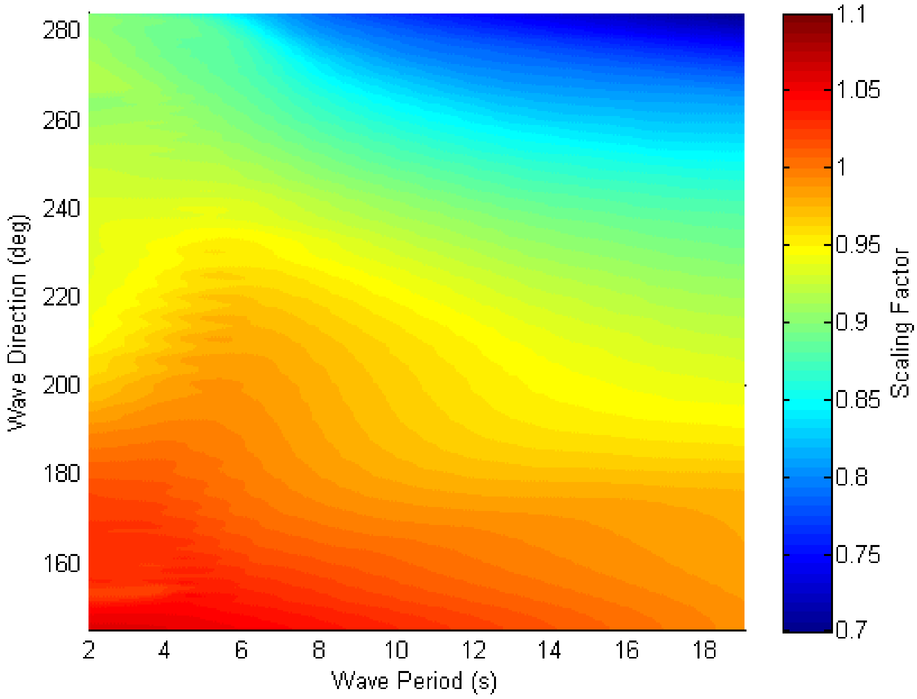


Figure A10 Transformation surface for MWD at Site 2

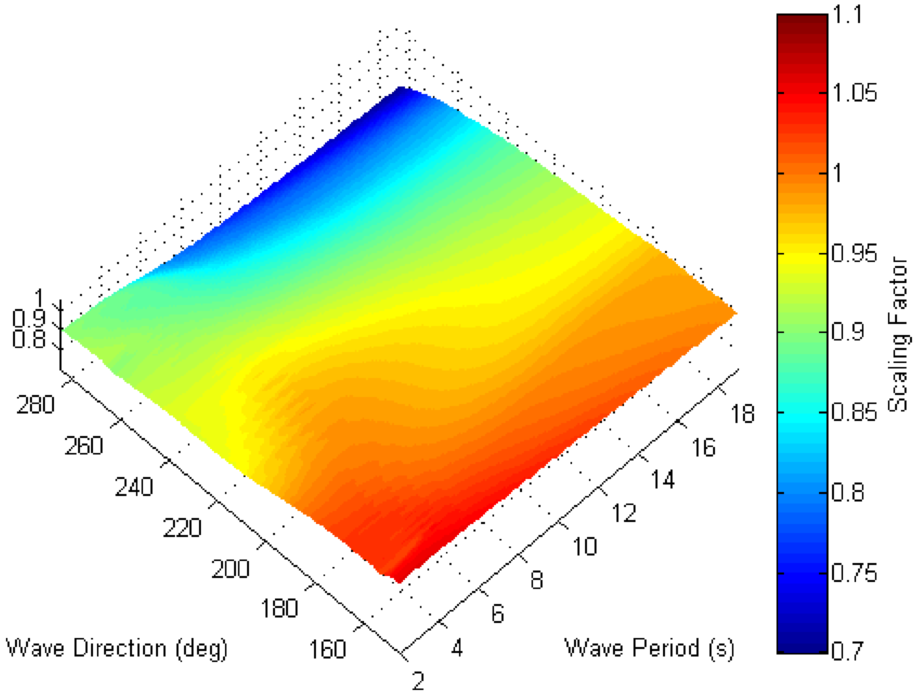
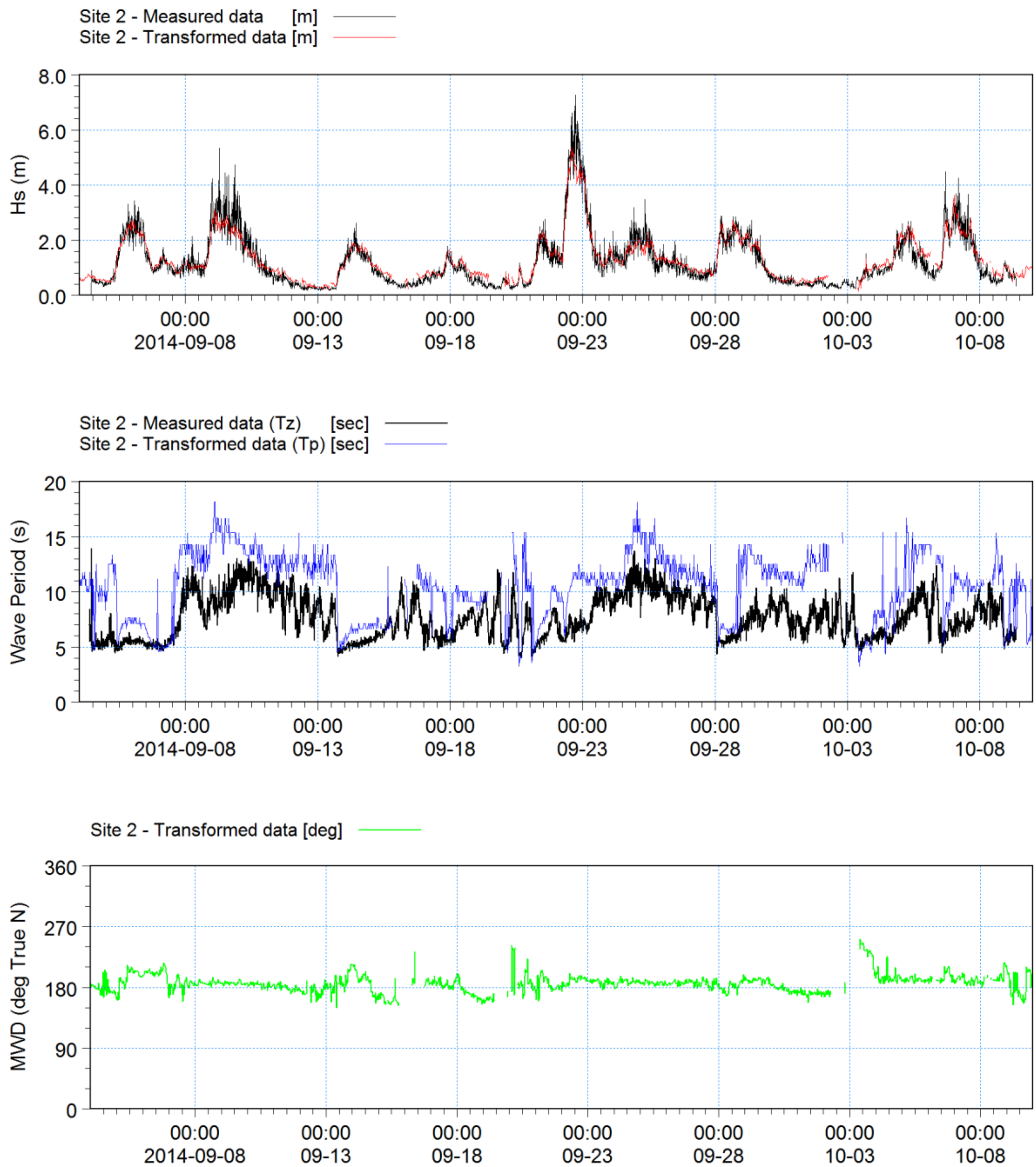


Figure A11 3D transformation surface for MWD at Site 2

A.6 Wave Transformation Results

The wave transformation results have been plotted against the measured data available at Sites 2 and 1, shown in **Figure A12** and **Figure A13**.

The agreement between measured and transformed data at Site 2 is generally good. The trend in Hs is captured well. A slight underestimation of transformed wave heights is evident during the two largest wave events around the 9th and 22nd September 2014. Only mean wave period (Tz) was measured at Site 2, hence the discrepancies between the measured and transformed data. The trend between the two datasets is clearly evident. Wave direction was not recorded at Site 2, hence, only the transformed data has been plotted.



Note: For wave period measured Tz is plotted against transformed Tp, as only Tz was measured at Site 2. Wave direction was not recorded at Site 2.

Figure A12 Wave transformation results at Site 2

To provide further confidence in the wave transformation approach the results have also been compared at Site 1, where Hs, Tp and MWD were measured. The model results show good agreement with the measured data for Hs and Tp. There is a mean offset of 12 degrees between the measured (172 degrees) and modelled (184 degrees) MWD. From recent correspondence, NIWA confirms an 8° bias most like caused by faults of the instrument. Given the location of Site 1 and the orientation of Lyall Bay the modelled MWD appears to be more reasonable.

Note the gaps in modelled data are typically associated with wave conditions from directions between 320 to 130 degrees or with less than a two second peak wave period as these have been excluded from the wave condition matrix generated.

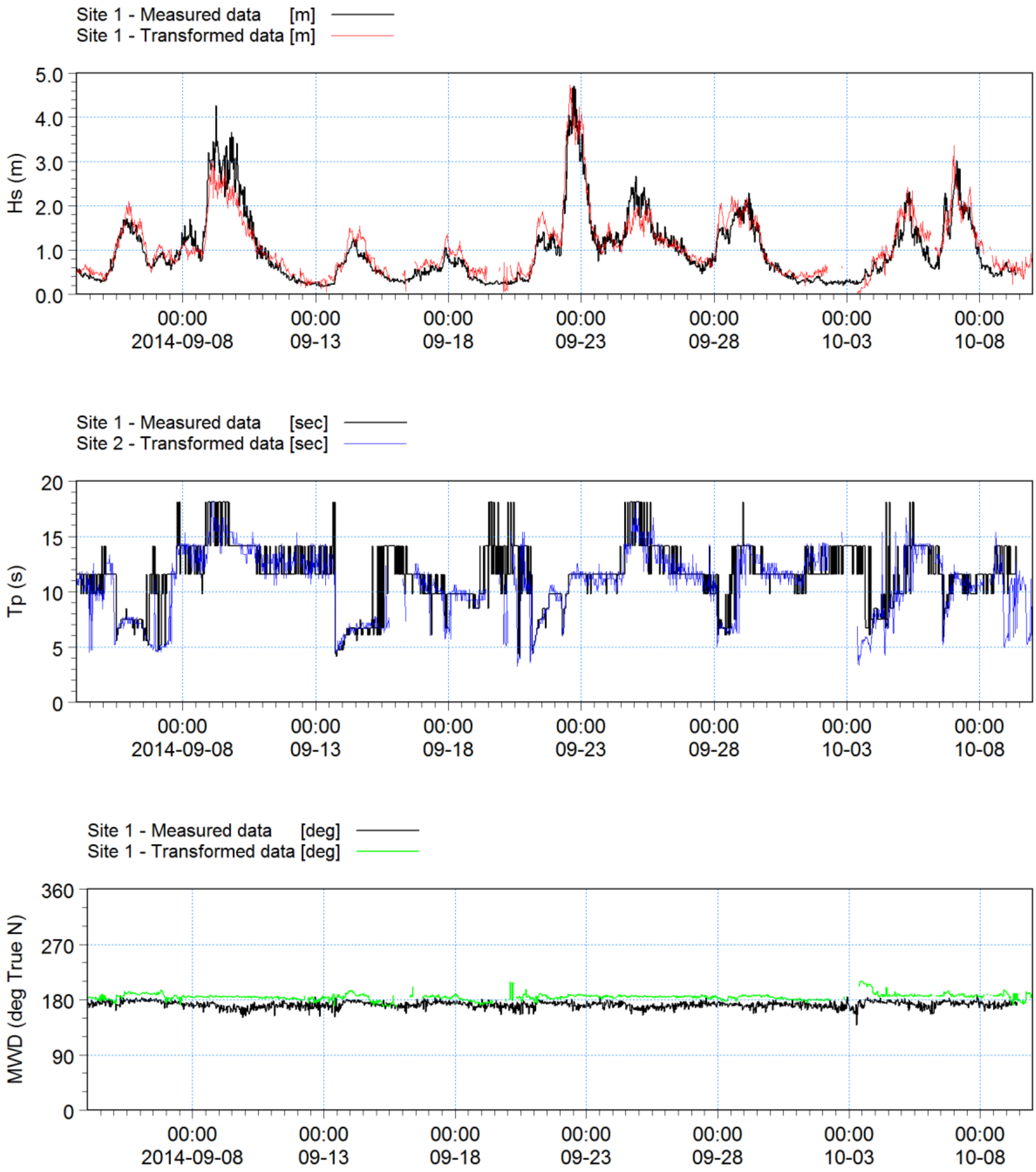


Figure A13   Wave transformation results at Site 1

APPENDIX B– Wave Diffraction Tests

B.1 Wave Diffraction Tests

When waves propagate into Lyall Bay energy is dissipated to both the east and west due to wave refraction. The phenomenon causes a transverse gradient in wave energy which drives a transverse diffraction wave process, which can be seen as a perturbation on the incident wave train causing wave peakiness.

To illustrate the effect transverse diffraction has on the wave field two tests were carried out using a simplified Boussinesq model domain. The tests used unidirectional 2nd order monochromatic Stokes waves with wave periods representing the upper and lower bounds of the Scenario test (see **Table B1**).

Table B1 Diffraction Test Scenarios

	H_s	T_p	WD
Test 1	1m	15.4s	270º
Test 2	1m	11.3s	270º

The modelling assessment framework utilizes the non-linear 2D Boussinesq Wave Model MIKE21 BW. The domain is discretized by a Cartesian Grid with an extent of 960m by 2660m and a horizontal grid spacing of 4 meters.

The simulation period for each test was set to 20 minutes.

Bathymetry consists of a straight channel with fully reflecting walls along the first 580m. The remaining domain is surrounded by sponge layers to allow transverse diffraction of wave energy. Water depth was set to 10m to represent conditions near the middle of Lyall Bay where the diffraction processes is expected to be limited by the airport runway extension. The Boussinesq model domain with this bathymetry is shown in **Figure B1**.

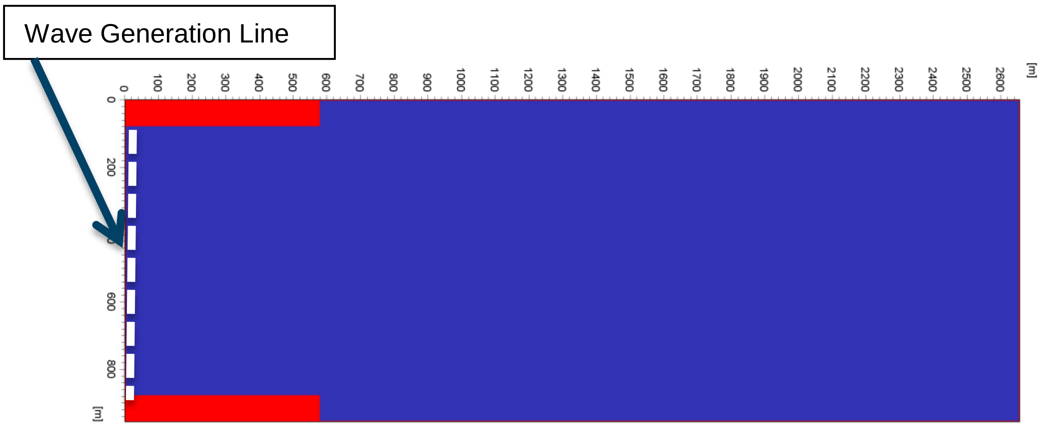


Figure B1 Diffraction Test Bathymetry

B.2 Diffraction Test Results

Instantaneous outputs of surface elevation are displayed in 3D for visualisation of the perturbations caused by wave diffraction (see **Figure B2** and **Figure B3**). A distinct difference is visible between test 1 and 2, confirming the effect wave period has on these perturbations. A cross sectional view is also provided in **Figure B4** for comparison between both tests. The extraction location is marked with a red dashed line in the 3D plots.

The transverse wave diffraction process drives a small propagating wave travelling perpendicular to the primary wave train. This can be seen in **Figure B2** and **Figure B3** where it is observed how the wave peakiness changes over time as long as diffraction persists. From **Figure B4** is observed how the wave height of the diffracted wave is approximately 10 – 25% of the incident primary waves (before diffraction). It is also observed that the length of the transverse diffracted wave is dependent on the wave period of the primary waves.

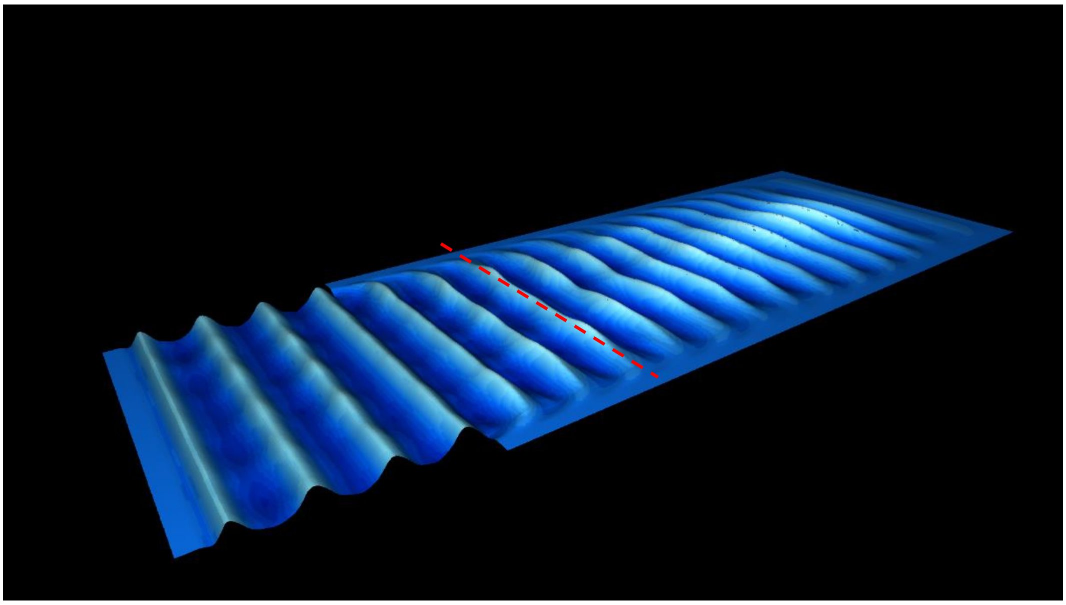


Figure B2 Visualisation of Diffraction Test 1: 1m Hs, 15.4 seconds

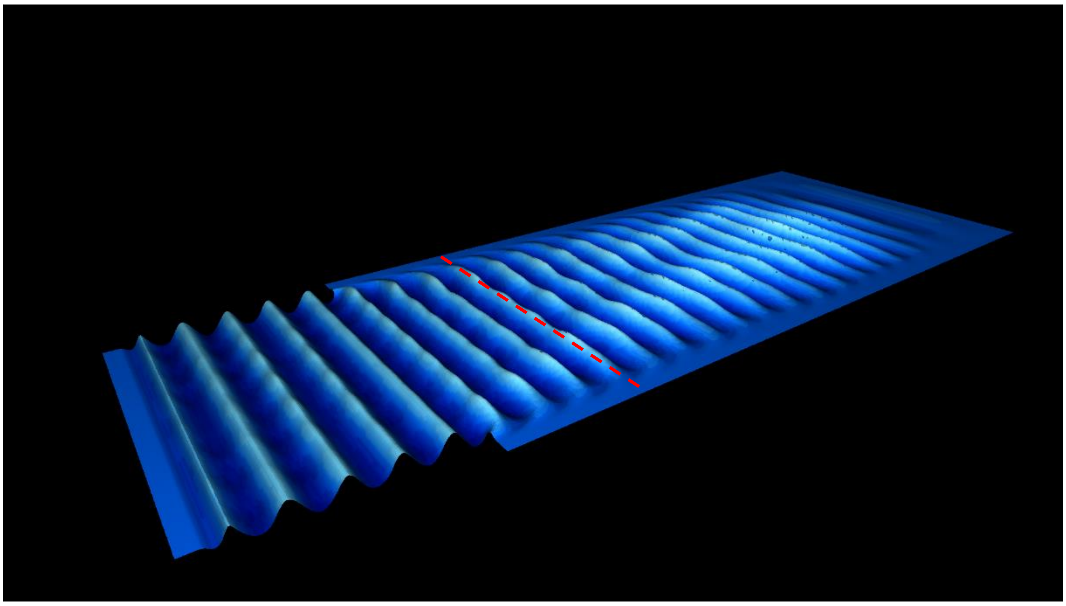


Figure B3 Visualisation of Diffraction Test 2: 1m Hs, 11.3 seconds

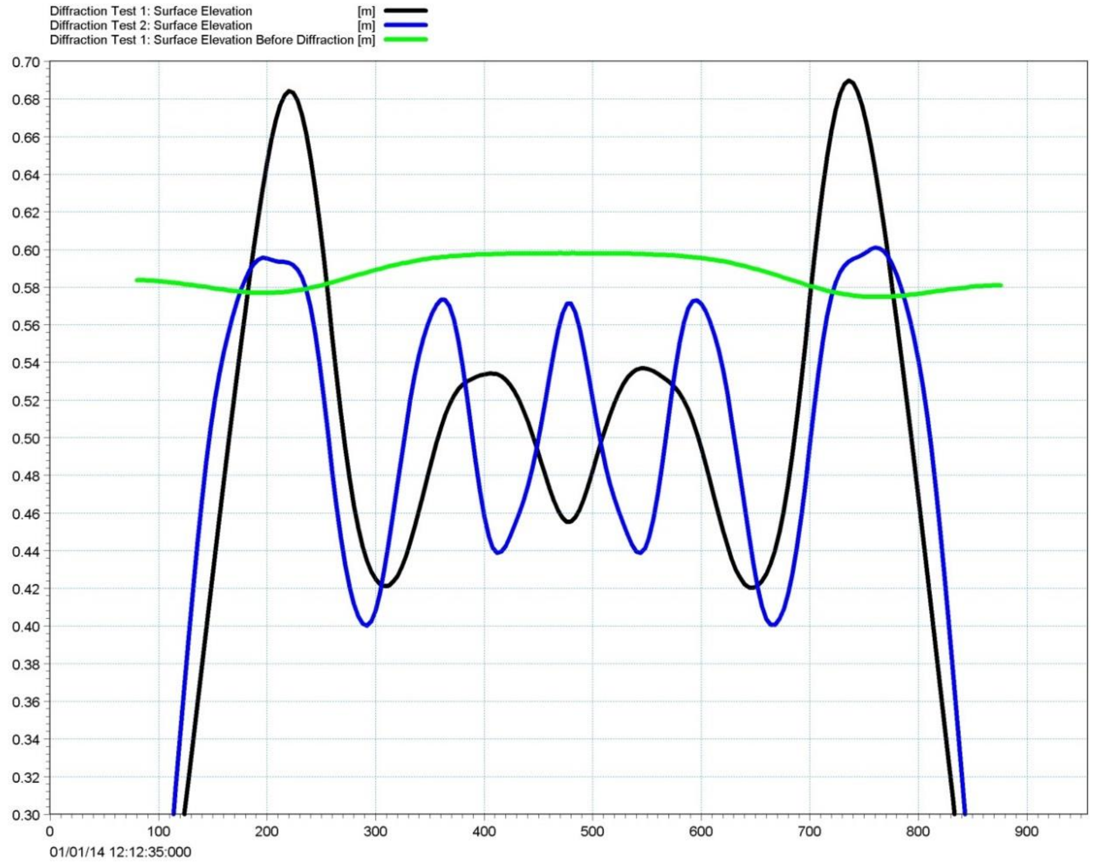


Figure B4 Diffraction Test Cross Section Comparison