

Metocean Report

New Marine Facilities to Service Mandorah and Cox Peninsula

ZMD01890



Prepared for
Department of Infrastructure, Planning and
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Table of Contents

1	Introduction	1
1.1	Background	1
1.2	Site Location	1
1.3	Project Objectives	1
1.4	Purpose and Structure of this Report	1
1.5	Reference Documents	3
2	Project Datum and Conventions	5
2.1	Units	5
2.2	Coordinate system	5
2.3	Vertical Reference Level	5
2.4	Time Reference	5
2.5	Direction Convention	5
2.6	Average Recurrence Interval	5
3	Existing Information	7
3.1	Metocean Data	7
3.2	Numerical Modelling	9
4	Data Collection	10
4.1	Methodology	10
4.2	Water Level	11
4.3	Waves	11
4.4	Currents	12
5	Site Environmental Conditions	13
5.1	Climate	13
5.2	Oceanography	18
6	Hydrodynamic Conditions	23
6.1	Hydrodynamic Model	23
6.2	Currents	31
6.3	Pre vs Post-Development Current Patterns	33
7	Wave Conditions	40
7.1	SWAN Wave Modelling System	40
7.2	Operational Wave Climate Study	40
7.3	Pre vs Post-Development Wave Conditions	48
8	Extreme (Cyclonic) Conditions	61
8.1	Cyclones	61
8.2	Design Wave Characteristics	64
8.3	Cyclonic Storm Tide Levels	67
9	Physical Modelling Studies	68
10	Harbour Agitation	69

10.1	Purpose	69
10.2	Guidelines for Acceptable Wave Conditions	69
10.3	SWASH Wave Modelling	72
11	References	81

Appendices

Appendix A ADCP Logsheets

Appendix B Current Vector Maps

Appendix C Physical Modelling Report

Tables

Table 1-1	Coastal processes investigations need, objectives and relevant standards/guidelines.	2
Table 2-1	Probability of Equalling or Exceeding Different ARI Events for Given Design Lives	6
Table 4-1	ADCP deployment details.	10
Table 5-1	Monthly and annual wind at Darwin airport station, 1985 to 2019.	16
Table 5-2	Directional (non-cyclonic) extreme wind speeds, 10-minute average measured 10m above ground level.	17
Table 5-3	Tide planes for Darwin Harbour (National Tides Centre, 2020)	19
Table 5-4	Global mean sea level rise projections in AR5 (IPCC, 2013)	21
Table 5-5	Present day cyclonic and non-cyclonic extreme storm tide levels derived for the site	22
Table 5-6	Design storm tide levels	22
Table 8-1	Cyclone wave modelling results	62
Table 8-2	Extreme significant wave height and peak wave period along the main and lee breakwaters (breakwater points) and inside the harbour (inner points)	66
Table 8-3	Design Storm Tide Levels – Cyclones	67
Table 10-1	AS3962 Criteria for a 'Good' Wave Climate in a Small Craft Harbour	71
Table 10-2	AS3962 Criteria for a 'Excellent Wave Climate in a Small Craft Harbour	71
Table 10-3	AS3962 Criteria for a 'Moderate' Wave Climate in a Small Craft Harbour	71
Table 10-4	Summary of wave conditions defined as input in the boundary of the wave agitation models for all three options	72
Table 10-5	The significant wave height extracted at the nominated output locations	80

Figures

Figure 1-1	Locality Plan	4
Figure 3-1	Existing metocean dataset locations	8
Figure 4-1	ADCP deployment location	10
Figure 4-2	Measured water level at the ADCP site	11
Figure 4-3	Measured wave statistics at the ADCP site	11
Figure 4-4	Measured depth average current speed and direction at the Mandorah ADCP 2019/2020	12
Figure 5-1	Monthly variation of the mean maximum, minimum, 9am and 3pm temperature based on Darwin airport data	13
Figure 5-2	Monthly variation of the mean 9am and 3pm relative humidity based on Darwin airport data	14
Figure 5-3	Monthly variation of mean rainfall and mean number of days of rain ≥ 1 mm based on Darwin airport data	14
Figure 5-4	Annual wind rose based on Darwin airport wind data (1985 to 2019)	15
Figure 5-5	Seasonal wind roses based on Darwin airport wind data (1985 to 2019)	16
Figure 5-6	Storm Tide Schematic [Courtesy: QLD Government]	18
Figure 5-7	AR5's (IPCC, 2013) global mean sea level rise projections	21
Figure 6-11	The location of wave and current data extraction point at Mandorah	31
Figure 6-12	Tidal current rose over the modelling period (100 days) at the extraction point	32
Figure 6-13	Time series of water level, current speed and current direction over the modelling period at the extraction point	32
Figure 6-14	Spring tide ebb flow vector plot – Pre development	34
Figure 6-15	Spring tide ebb flow vector plot – Post development	35
Figure 6-16	Spring tide ebb flow vector plot – Pre vs post development	36
Figure 6-17	Spring tide flood flow vector plot – Pre development	37
Figure 6-18	Spring tide flood flow vector plot – Post development	38
Figure 6-19	Spring tide flood flow vector plot – Pre vs post development	39
Figure 7-1	SWAN model grids: Beagle Gulf (red), Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Beagle Gulf bathymetry	41
Figure 7-2	SWAN model grids: Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Darwin Harbour bathymetry	42
Figure 7-3	Wind and data extraction points from Darwin airport and CSIRO wave model	43
Figure 7-4	Comparison of measured and modelled wave parameters at the Mandorah ADCP (2020)	44
Figure 7-5	Comparison of measured and modelled significant wave height at the 2013-2015 RBR measurement locations	45
Figure 7-6	Annual offshore wave rose based on CSIRO hindcast wave data at CSIRO Extraction Point 2 (1979 to 2019)	46
Figure 7-7	Seasonal offshore wave roses based on CSIRO hindcast wave data at CSIRO Extraction Point 2 (1979 to 2019)	47
Figure 7-8	Annual total wave rose at the extraction point (1985 to 2019)	48
Figure 7-9	Wave vector plot during spring and summer, north-north-westerly conditions – Pre development	50
Figure 7-10	Wave vector plot during spring and summer, north-north-westerly conditions – Post development	50

Figure 7-11 Wave vector plot during spring and summer, north-north-westerly conditions – Pre vs post development	51
Figure 7-12 Wave vector plot during spring and summer, northerly conditions – Pre development	52
Figure 7-13 Wave vector plot during spring and summer, northerly conditions – Post development	53
Figure 7-14 Wave vector plot during spring and summer, northerly conditions – Pre vs post development	54
Figure 7-15 Wave vector plot during autumn and winter, north-north-easterly conditions – Pre development	55
Figure 7-16 Wave vector plot during autumn and winter, north-north-easterly conditions – Post development	56
Figure 7-17 Wave vector plot during autumn and winter, north-north-easterly conditions – Pre vs post development	57
Figure 7-18 Wave vector plot during autumn and winter, south-easterly conditions – Pre development	58
Figure 7-19 Wave vector plot during autumn and winter, south-easterly conditions – Post development	59
Figure 7-20 Wave vector plot during autumn and winter, south-easterly conditions – Pre vs post development	60
Figure 8-1 Tracks of the 25 most severe cyclones that have affected Darwin since 1964	61
Figure 8-2 Design wave extraction locations along the breakwaters	64
Figure 8-3 Design wave extraction locations inside the harbour	65
Figure 10-1 Wave Direction Classifications for Marinas and Harbours (AS3962)	70
Figure 10-2 The Six Degrees of Freedom of Vessel Motion. Image source: Varela et al, 2011.	70
Figure 10-3 SWASH wave model grid and bathymetry utilised for the wave agitation modelling	73
Figure 10-4 SWASH wave modelling result for 1-year ARI waves from ESE direction	74
Figure 10-5 SWASH wave modelling result for 1-year ARI waves from SE direction	75
Figure 10-6 SWASH wave modelling result for 1-year ARI waves from E direction	76
Figure 10-7 SWASH wave modelling result for 50-year ARI waves from ESE direction	77
Figure 10-8 SWASH wave modelling result for 50-year ARI waves from SE direction	78
Figure 10-9 SWASH wave modelling result for 50-year ARI waves from E direction	79

1 Introduction

1.1 Background

The existing Mandorah Jetty and Boat Ramp are key infrastructure facilities servicing the Mandorah, Wagait and Belyuen communities, as well as visitors to the area from Darwin and surrounds. The Mandorah Jetty supports the only ferry service from Darwin to the Cox Peninsula. The Mandorah boat ramp is located just south of the Jetty, with both facilities sharing a common onshore carpark facility.

Previous studies (Jacobs, 2016) identified shortcomings in the design of the existing facilities, such as Disability Discrimination Act (DDA) non-compliance of the Jetty and investigated various options to improve the ferry berthing facility and boat ramp. The recommendation from those studies included building a breakwater, new floating pontoon and boat ramp at a location just north of the existing jetty. A further study (Jacobs, 2019) undertook field surveys and marine modelling to determine the optimised location, configuration and size of rock breakwater, groyne, gangway/pontoon and boat ramp structures. That study investigated various sizes and configurations of these facilities, with siting of the pontoon based on available water depths for future ferry operations and to avoid dredging. This resulted in an expensive rock groyne configuration requiring undesirable commuter access across the rock groyne and stricter design criteria.

Cardno (NT) Pty Ltd (Cardno) were engaged to undertake value engineering, design and documentation services for these new marine facilities to service the Mandorah and Cox Peninsula communities (the Project).

1.2 Site Location

Mandorah is located near the eastern tip of the Cox Peninsula in the Northern Territory, approximately six kilometres to the west of Darwin. Access to Mandorah from Darwin is via the regular ferry service, or by driving approximately 120 km along the road network. The locality is shown in **Figure 1-1**.

1.3 Project Objectives

The key project objectives for DIPL include selection and design of a facility that:

- > Minimises construction and whole-of-life costs;
- > Allows for uninterrupted ferry service operation during construction;
- > Provides a safe and enjoyable passenger experience; and
- > Addresses issues faced by the existing Mandorah Jetty and boat ramp. These issues include the highly exposed nature of the site, DDA non-compliance, passenger safety issues and safety and sedimentation issues of the existing boat ramp.

1.4 Purpose and Structure of this Report

The purpose of this document is to detail the metocean investigations that were carried out to inform the design and environmental approvals associated with the new marine facilities at Mandorah. These include the following:

- > Development of wave and hydrodynamic models incorporating cyclone modelling to determine critical (or threshold) metocean design constraints; and
- > Joint probability analysis to assess likelihood and severity of extreme water levels occurring with extreme wave conditions.

Details on the need and requirements for outputs derived from coastal processes investigations are summarised in **Table 1-1**.

Table 1-1 Coastal processes investigations need, objectives and relevant standards/guidelines.

Data type	Data need	Data quality objective	Relevant standards/guidelines
Waves	Design of structures for wave loads	Confident prediction of wave related forces on components of facility (breakwaters, pontoons, piles etc.) to optimize their location, inform their design, set their damage criteria, estimate maintenance across facility lifecycle.	AS 3962-2001 (Section 4.8.2, 4.11) AS 4997-2005 (Section 5.9)
	Assessment/design for operational conditions	Assessment of wave climate within facility to optimize design and inform operability.	AS 3962-2001 (Section 4.8.2.1, 4.12)
	Local sediment transport	Incorporate wave conditions into sediment transport modelling to ensure the effect of waves on this process are allowed for.	AS 3962-2001 (Section 2.3.2.7)
	Input parameters for physical modelling	Determine range of wave parameters to force in physical modelling for the purpose of sensitivity testing the design of breakwater segments.	
Currents	Design of structures for current loads	Confident prediction of current related forces on components of facility (breakwaters, pontoons, piles etc.) to optimize their location, inform their design, estimate maintenance across facility lifecycle.	AS 3962-2001 (Section 4.8.4, 4.11) AS 4997-2005 (Section 5.5)
	Assessment/design for operational conditions	Assessment of currents within, and approaching, facility to optimize design and inform operability.	AS 3962-2001 (Section 4.9, 4.10)
	Local sediment transport	Use hydrodynamic modelling to assess local sediment transport to inform design and predict maintenance requirements across facility lifecycle. Use modelling to predict construction impacts.	AS 3962-2001 (Section 2.3.2.7)
Water level	Design levels of structures	Extreme value analysis, modelling of water levels, setup, storm surge to dictate required heights/levels.	AS 3962-2001 (Section 2.3.2.3, 2.3.2.4, 2.3.2.5) AS 4997-2005 (Section 5.8)
	Operational constraints	Forecasting of water levels to determine ramp slope compliance (i.e. DDA compliance).	AS 3962-2001 (Section 3.6.2)
	Drive physical process modelling	Calculated water level record may be used to drive hydrodynamic modelling. I.e. using tidal constituents.	AS 3962-2001 (Section 2.3.2.6)
	Sea level rise assessment	Factor predicted sea level rise into design of facility for full lifecycle	AS 3962-2001 (Section 2.3.2.9) AS 4997-2005 (Section 4.6)
Wind	Design of structures for wind loads	Analysis of wind data for design constraints, loads on maritime structures and estimate maintenance across facility lifecycle.	AS 3962-2001 (Section 2.3.2.1) AS 4997-2005 (Section 5.4)
	Drive physical process modelling	Modelled winds to drive physical process modelling (e.g. wave/cyclone modelling)	AS 3962-2001 (Section 2.3.2.1)

The document has been structured as follows:

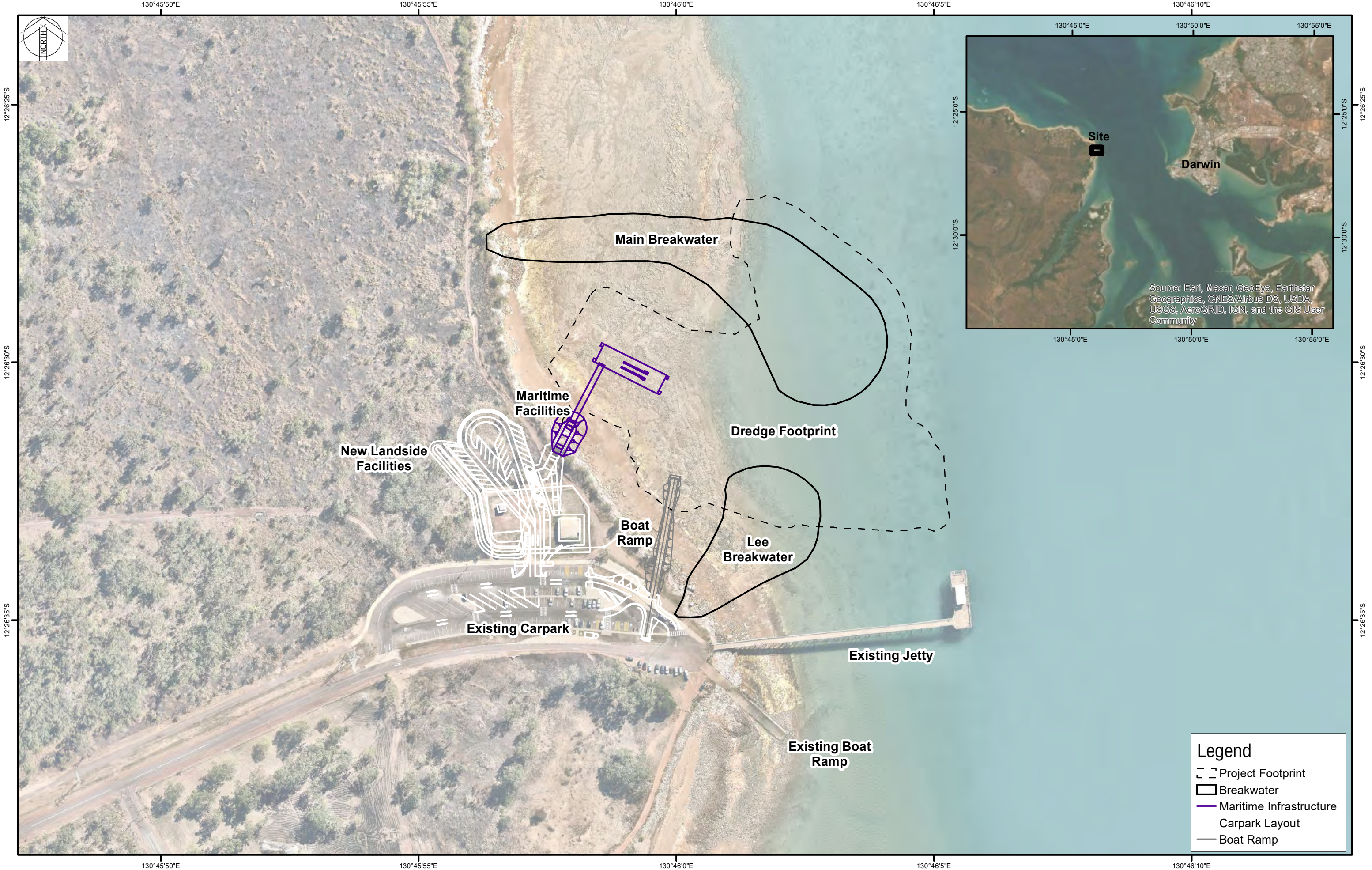
- > This Section (1) introduces the project and context and purpose of this report;

- > **Section 2** details project datums and conventions used in the report;
- > **Section 3** outlines existing information relevant to the project;
- > **Section 4** describes field investigations/data collection undertaken as part of the project to fill relevant data gaps;
- > **Section 5** described the general environmental conditions of the site;
- > **Section 6** details hydrodynamic conditions for the site and project;
- > **Section 7** details wave conditions for the site and project;
- > **Section 8** details extreme (cyclonic) conditions for the site and project;
- > **Section 9** references physical modelling studies associated with the project; and
- > **Section 10** investigates harbour agitation.

1.5 Reference Documents

Various investigations and studies have been carried out as part of the overall development and design project to date. This report documents a component of these investigations. The following documents are referenced in, or should be given due consideration when reading, this report:

- > Directly relevant:
 - *Design Report* (Cardno, 2022a): Details the design requirements, basis, inputs, calculations and outcomes for the new proposed infrastructure, which relies heavily on the details in this report;
 - *Sediment Transport Report* (Cardno, 2023): Details siltation, sedimentation and plume dispersion investigations associated with the project. The investigations are underpinned by the modelling described in this report;
- > Relevant to the overall project:
 - *Environmental Referral Report* (Cardno, 2022b): This document provides the necessary details to refer the project to the NT EPA;
 - *Geotechnical Report* (Cardno, 2022c): Details the geotechnical information and investigations for the project site that inform design of the new proposed infrastructure;
 - *Marine Environment Report* (Cardno, 2022d): Details the marine environment (including physical properties related to sediment) and ecology relevant to the project site. The results of sediment transport investigations are important for understanding potential impacts to the marine environment detailed in the report; and
 - *Draft Dredging and Disposal Management Plan* (Stantec, 2023a): Defines the proposed dredging actions and necessary controls to avoid or minimise risk for, and impact to, the marine environment.
 - *Supplementary Environmental Report* (Stantec, 2023b): Details the outcomes of the supplementary information requested to refer the project to the NT EPA;



2 Project Datum and Conventions

2.1 Units

The SI System of Units has been used.

2.2 Coordinate system

All coordinates are in MGA zone 52 and are based on the GDA94 spheroid.

2.3 Vertical Reference Level

All elevations are in meters and referenced to Australian Height Datum (AHD), unless otherwise noted.

2.4 Time Reference

All data related to time are given in Australian Central Time (ACST), which is GMT + 9.5 hours.

2.5 Direction Convention

- > **Flow:** Flow directions refer to the direction towards which the flow occurs. Directions of the flow are always given clockwise with respect to true north. The unit is degrees.
- > **Wave direction:** The direction from which the wave is coming (i.e. coming from convention), and is measured clockwise from true north. A wave with a direction of 0° is coming from the north, and 90° is coming from the east etc.
- > **Wind direction:** The direction from which the wind is blowing (i.e. coming from convention), and is measured clockwise from true north. A wind with a direction of 0° is coming from the north, and 90° is coming from the east etc.

2.6 Average Recurrence Interval

This report presents an overview of the met-ocean conditions at Mandorah. The met-ocean conditions are presented in the form of statistics, wind and wave roses and extreme value estimates.

This report defines extreme metocean conditions based upon an average recurrence (or return) interval (ARI). The ARI is defined as the average interval of time, normally in years, in which the condition would be expected to be exceeded once. Design events are typically defined in this manner. For example, a 50-years ARI event is, on average, exceeded once every 50 years. It should be noted that it is possible, although not probable, that this extreme event could be exceeded during the first year of the life of the structure, or may not be exceeded at all over the life of the structure. Table 2-1 presents the probability of occurrence of events up to the 1,000-years ARI event for given design lives. For a 50-years design life there is an approximate 40% probability that a 100-years ARI event or larger will occur within the design lifetime of the structure. A 1,000-years ARI event has a 5% chance of occurring (or being exceeded) during a typical 50-years design life of a coastal structure.

Table 2-1 Probability of Equalling or Exceeding Different ARI Events for Given Design Lives

Average Recurrence Interval Event	Probability of Occurrence in 1 year Period	Probability of Occurrence in 2 years Period	Probability of Occurrence in 5 years period	Probability of Occurrence in 10 years Period	Probability of Occurrence in 25 years Period	Probability of Occurrence in 50 years Period	Probability of Occurrence in 100 years Period
1 year	63.2%	86.5%	99.3%	100.0%	100.0%	100.0%	100.0%
2 years	39.3%	63.2%	91.8%	99.3%	100.0%	100.0%	100.0%
5 years	18.1%	33.0%	63.2%	86.5%	99.3%	100.0%	100.0%
10 years	9.5%	18.1%	39.3%	63.2%	91.8%	99.3%	100.0%
25 years	3.9%	7.7%	18.1%	33.0%	63.2%	86.5%	98.2%
50 years	2.0%	3.9%	9.5%	18.1%	39.3%	63.2%	86.5%
100 years	1.0%	2.0%	4.9%	9.5%	22.1%	39.3%	63.2%
200 years	0.5%	1.0%	2.5%	4.9%	11.8%	22.1%	39.3%
300 years	0.3%	0.7%	1.7%	3.3%	8.0%	15.4%	28.3%
400 years	0.2%	0.5%	1.2%	2.5%	6.1%	11.8%	22.1%
500 years	0.2%	0.4%	1.0%	2.0%	4.9%	9.5%	18.1%
1000 years	0.1%	0.2%	0.5%	1.0%	2.5%	4.9%	9.5%

3 Existing Information

3.1 Metocean Data

The following metocean datasets have been obtained by Cardno and have been incorporated to inform and/or determine design constraints for the Project (see also **Figure 3-1**):

> Wave:

- o Non-directional wave data (2 Hz water level measurements) collected by Cardno between 16/08/2013 and 16/01/2015 at Mandorah (12°26.205' S, 130°46.241'E);
- o Directional wave data (ADCP) collected by AIMS between 03/07/2010 and 30/04/2019 at an offshore location in Darwin Harbour (686490mE, 8634892mN);
- o Directional wave data (ADCP) collected by Cardno between 23/11/2012 and 05/06/2013 at the INPEX dredge spoil ground location (701124mE, 8647132mN); and
- o Directional wave data (ADCP) collected by Cardno between 23/11/2012 and 20/03/2013 at Weed Reef (701124mE, 8647132mN);

> Currents:

- o Current Meter (Nortek Aquadopp) data collected by Astute Surveying at Mandorah between 01/09/2017 and 07/10/2017 (12°26.513' S, 130°46.125'E);
- o Current data (ADCP) collected concurrently with waves at the three ADCP sites described above;
- o Current data (ADCP) collected by Cardno between 09/04/2013 and 09/05/2013 at Fort Hill Wharf (699976mE, 8620515mN);
- o Current data (ADCP) collected by Cardno between 09/04/2013 and 08/05/2013 at Middle Arm Channel (700678mE, 8615496mN);
- o Current data (ADCP) collected by Cardno between 27/09/2012 and 20/03/2013 at Mandorah (692850mE, 8624360mN);
- o Current data (ADCP) collected by Cardno between 28/09/2012 and 21/03/2013 at Upper East Arm (714477mE, 8613172mN); and
- o Current data (ADCP) collected by Cardno between 03/04/2013 and 08/05/2013 at Wickham Point (714477mE, 8613172mN);

> Water level:

- o Fort Hill Wharf Tide Gauge Data recorded between 1992 and 2019;
- o Water level gauge data collected by Astute Surveying at Mandorah between 01/09/2017 and 07/10/2017 (12°26.513' S, 130°46.125'E);
- o Water level measurements (2Hz) collected by Cardno between 16/08/2013 and 16/01/2015 at Mandorah (12°26.205' S, 130°46.241'E); and
- o Water level data collected concurrently with all ADCP sites described above;

> Wind

- o BoM Darwin Airport anemometer data between 1985 and 2019; and
- o Fort Hill Wharf anemometer data between 1992 and 2019.

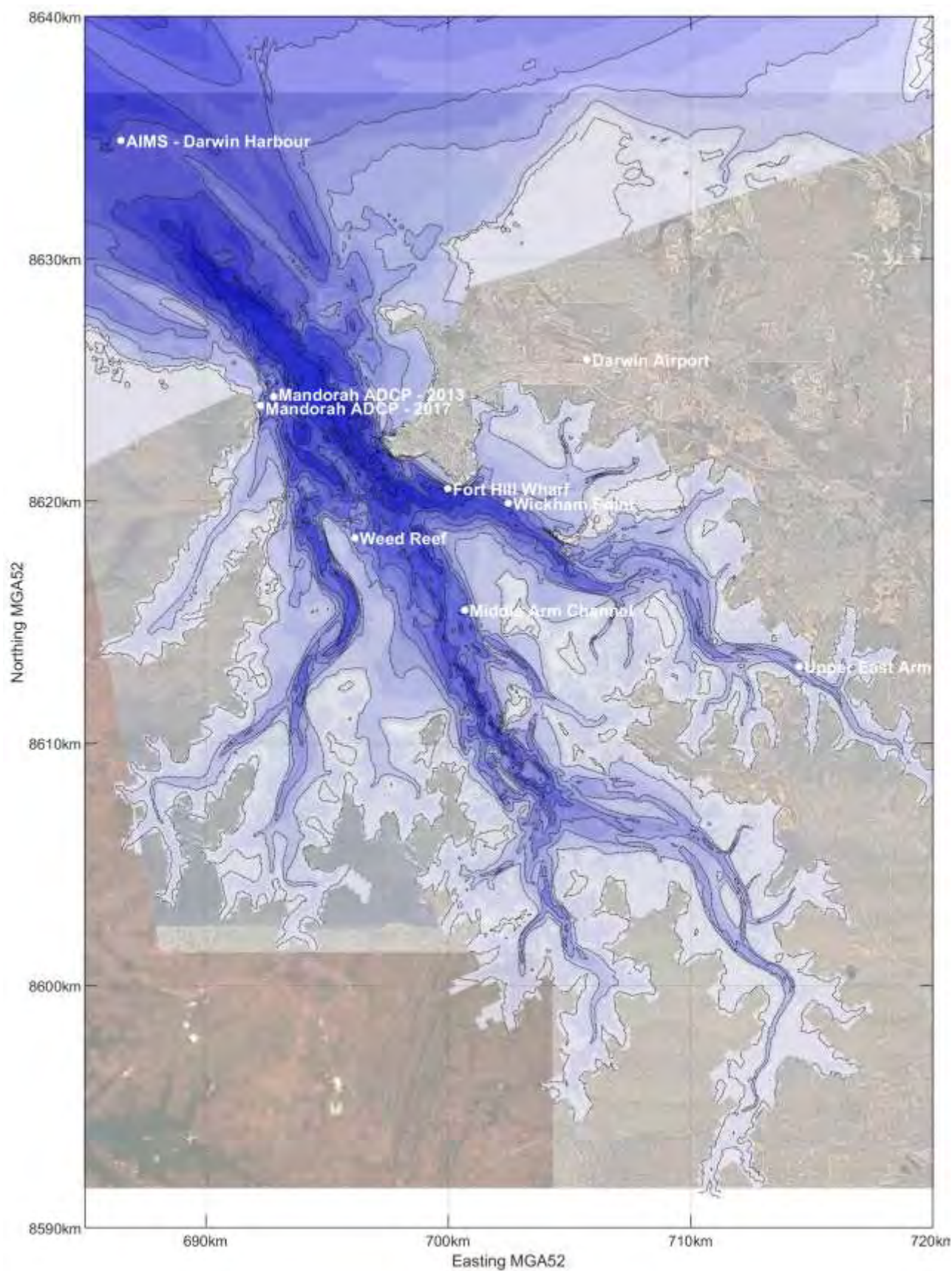


Figure 3-1 Existing metocean dataset locations

3.2 Numerical Modelling

Jacobs (2019) undertook preliminary wave and hydrodynamic modelling to assess site conditions and provide preliminary design constraints for a concept harbour layout. The extent of modelling included:

- > Baseline wave transformation Model (SWAN model used to transform ambient and extreme offshore wave climate to Site location); and
- > Baseline hydrodynamic model (using Jacobs existing MIKE 21 FM hydrodynamic model of Darwin Harbour to estimate water levels and currents at Site location).

However, a range of limitations were identified in this modelling. Subsequently, Cardno have developed and refined its own numerical models and calculated design constraints independently. The results of Jacobs (2019) modelling allow for a crosscheck of its outputs.

4 Data Collection

4.1 Methodology

A 1200 kHz, waves-enabled Acoustic Doppler Current Profiler (ADCP) was deployed at the project site for a period of approximately 4 months, to measure waves, currents and water levels at the project site during the wet season. The purpose of data collection was to collect metocean (particularly wave) data directly at the project site, which has not been collected previously. The instrument measured wave conditions continuously for 20 minutes every hour. Water level and current speed and direction, in 0.5m bins through the water column, were measured every 20 minutes. The instrument was initially deployed on 12/12/2019 at the location shown in **Figure 4-1**. The final recovery date, from this location was 22/04/2020. **Table 4-1** provides details on the deployment periods, including exact deployment location, recovery dates/times and data return. There was data lost during the first deployment due to a data corruption issue, caused when the primary memory card of the instrument became full and it switched to the secondary data card. Deployment and recovery log sheets for the ADCP are presented in **Appendix A** of this report.

Table 4-1 ADCP deployment details.

Time Period	Easting (MGA z52)	Northing (MGA z52)	Deployment date/time	Recovery date/time	Data return
Deployment 1	692214	8624001	12/12/2019 07:00	02/03/2020 8:00	77%
Deployment 2	692214	8624001	02/03/2020 10:00	22/04/2020 7:30	100%



Figure 4-1 ADCP deployment location

4.2 Water Level

Water level measurements have been plotted against the mean water level for the deployment in **Figure 4-2**. The record demonstrates the site's large tidal range, approximately 7 metres for the observation period. As expected under a large tidal regime, water level appears to be predominately governed by tides. The influence of other drivers, such as storm surge, wind and wave setup, is minor by comparison.

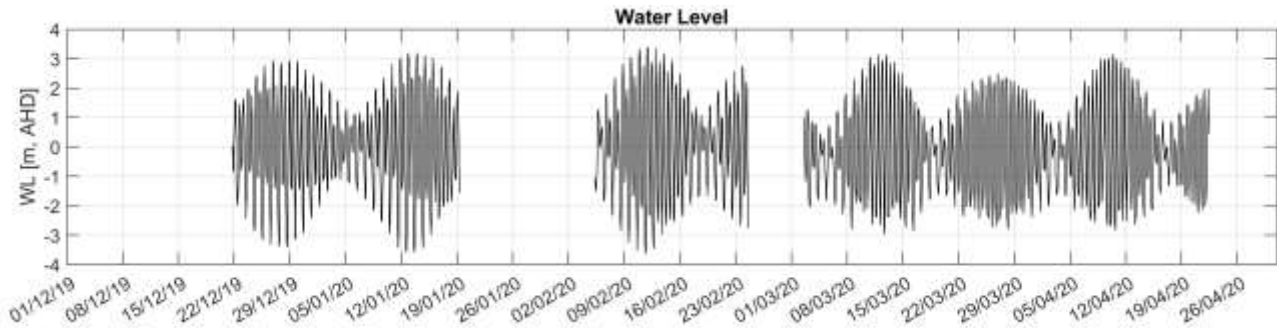


Figure 4-2 Measured water level at the ADCP site

4.3 Waves

Timeseries of the significant wave height, peak wave period and peak wave direction at the deployment site are presented in **Figure 4-3**. The records demonstrate a low energy wave climate over the deployment period, predominately comprised of locally generated seas originating from the north to north-east. Maximum significant wave heights measured for the period were just over 1 metre.

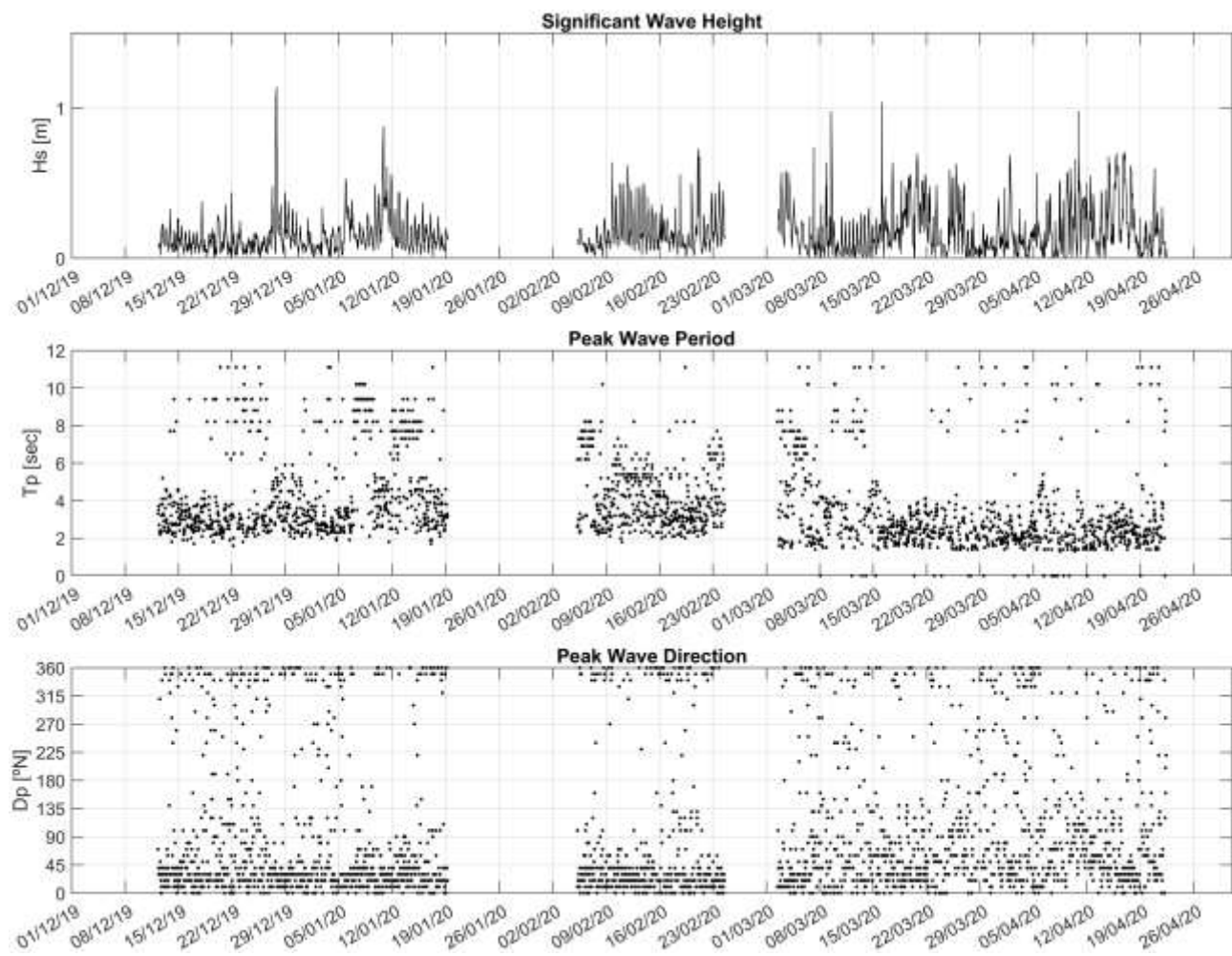


Figure 4-3 Measured wave statistics at the ADCP site

4.4 Currents

Timeseries of depth-averaged current speed and direction are plotted in **Figure 4-4**. The records demonstrate that the current regime is driven predominantly by tidal movement. The prevailing currents at the deployment location are alongshore in both directions. The tidal asymmetry is reflected in the record with stronger currents flowing to the north (ebb tide). The maximum depth averaged current speed recorded during the observation period was just over 1 m/s.

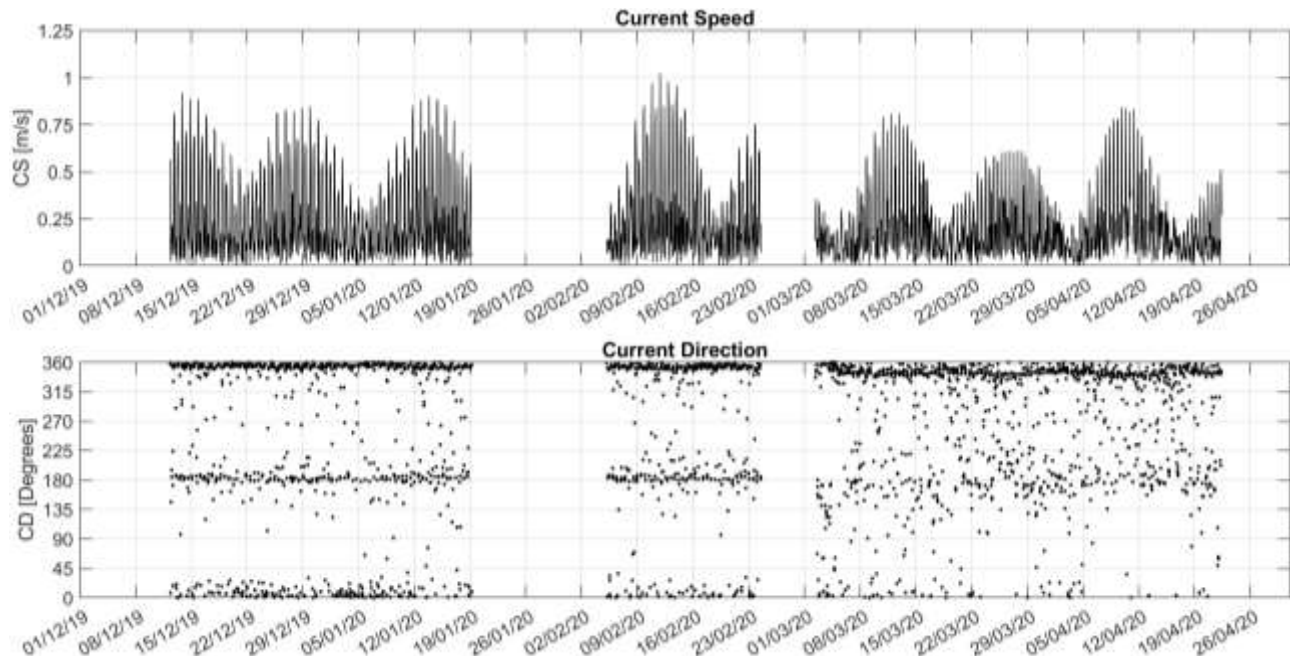


Figure 4-4 Measured depth average current speed and direction at the Mandorah ADCP 2019/2020

5 Site Environmental Conditions

5.1 Climate

5.1.1 Precipitation and Temperature

Darwin has a tropical climate with distinct wet and dry seasons and the average maximum temperature is similar all year round. The driest period of the year, seeing only approximately 5 mm of monthly rainfall on average, is between May and September. In the coolest months of June and July, the daily minimum temperature may dip as low as 14 °C, but very rarely lower, and a temperature lower than 10 °C has never been recorded in Darwin. The extreme upper and lower temperatures at the Darwin airport station have ranged from 38.9 °C to 10.4 °C.

The highest minimum temperature on record is 29.7 °C; while the lowest maximum temperature on record is 21.1 °C. Mean annual maximum and minimum temperature at the Darwin Airport station is recorded to be 32.1 °C and 23.2 °C, respectively. The hottest month is November, just before the onset of the main rain season. Because of humidity levels, the heat index sometimes rises above 45 °C, while the actual temperature is usually below 35 °C. The average temperature of the sea also ranges from 25.8 °C in July to 31.5 °C in December.

The wet season is associated with tropical cyclones and monsoon rains. The majority of rainfall occurs between December and March (the southern hemisphere summer), when thunderstorms are common and afternoon relative humidity averages over 70 percent during the wettest months. It does not rain every day during the wet season, but most days have plentiful cloud cover; January averages under 6 hours of bright sunshine daily. The dry season runs from about May to September, during which nearly every day is sunny, and afternoon relative humidity averages around 30%.

Variation of the mean maximum, minimum, 9am and 3pm temperature at different months of the year as per temperature data recorded at Darwin airport station is presented in Figure 5-1. Figure 5-2 also shows the monthly variation of mean 9am and 3pm relative humidity at this station. In addition, monthly variation of mean rainfall and mean number of the days of rain ≥ 1 mm is provided in **Figure 5-3**.

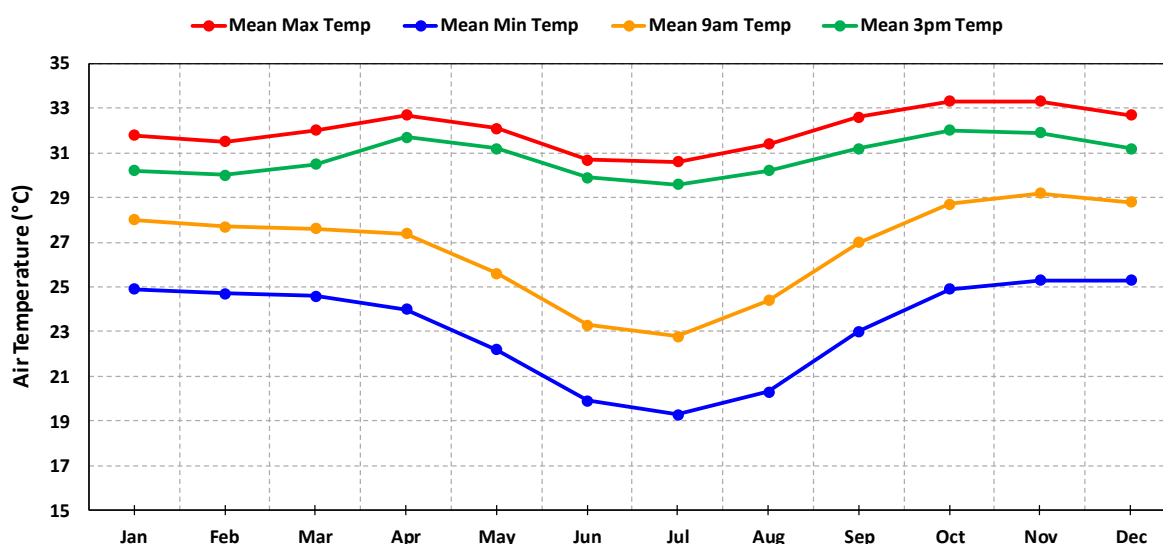


Figure 5-1 Monthly variation of the mean maximum, minimum, 9am and 3pm temperature based on Darwin airport data

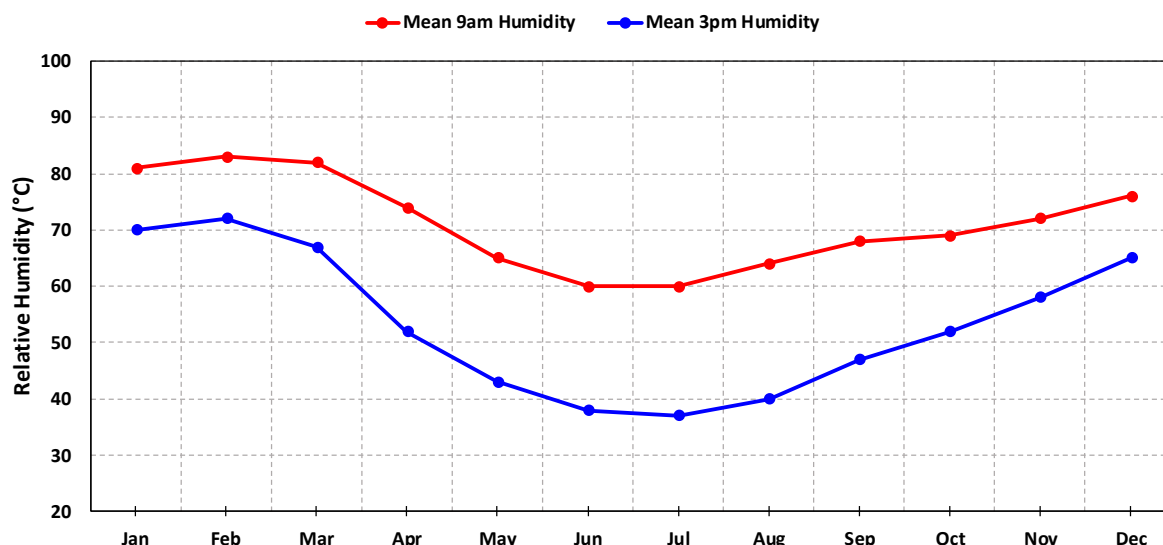


Figure 5-2 Monthly variation of the mean 9am and 3pm relative humidity based on Darwin airport data

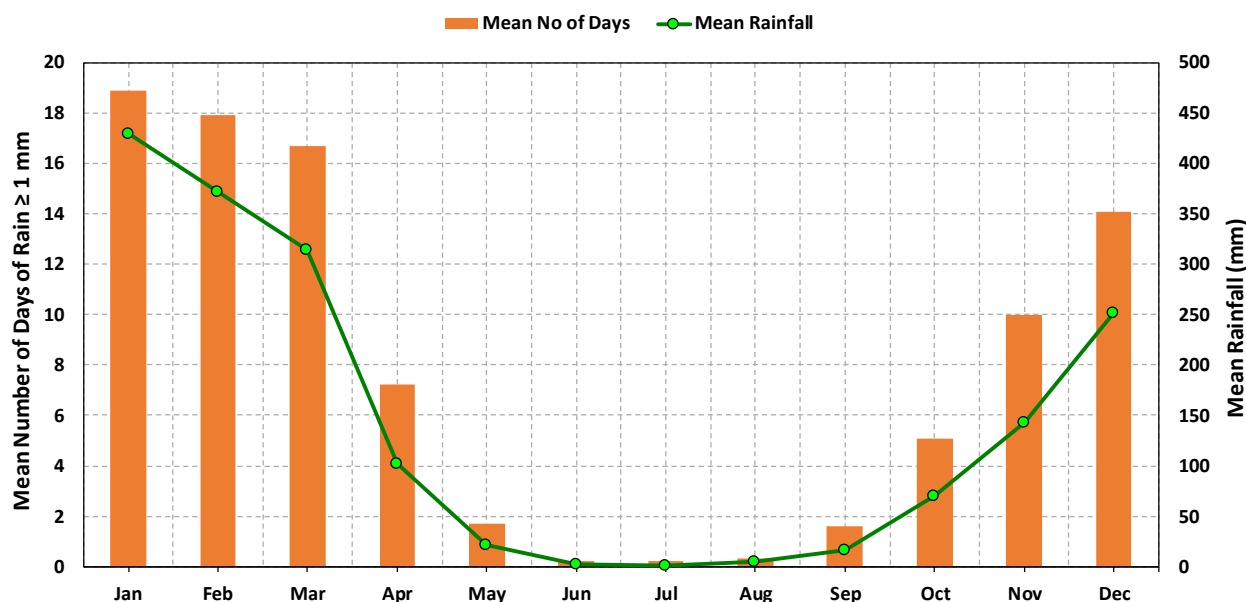


Figure 5-3 Monthly variation of mean rainfall and mean number of days of rain ≥ 1 mm based on Darwin airport data

5.1.2 Wind

5.1.2.1 Main Wind Directions and Corresponding Wind Speeds

Measured wind data at Darwin Airport from 1985 to 2019 has been used to derive the wind climate at the project site. The location of the wind data extraction point is shown in **Figure 3-1**.

5.1.2.2 Main Wind Directions and Corresponding Wind Speeds

Darwin airport wind data, as provided by the BoM, were analysed in terms of wind speed (10-minute average) and directions for annual, seasonal and monthly distributions. The annual and seasonal distributions of the joint occurrence of wind speed and direction are presented in **Figure 5-4** and **Figure 5-5**, respectively. These winds have been converted to a 10 minute, 10m wind speed over water.

The results of the monthly analysis of Darwin airport wind data is also presented in **Table 5-1**. The average wind speed is relatively constant throughout the year, ranging from 6.7 m/s in April to 7.7 m/s in January and February. The average annual wind speed is 7.2 m/s. The maximum wind speeds during the wet season (1st of November to 30th of April) are generally higher than those during the dry season (1st of May to 31st of October). This is to be expected due to the cyclonic activity and tropical lows that occur during the wet season.

The wind direction also varies throughout the year, displaying a seasonal variation. The winds are predominantly from the west to north (270-360 °N) from the start of September to the end of February (spring and summer) and predominantly from the east to the south east (90-135 °N) from the start of March to the end of the August (Autumn and Winter). The months of March, August and September show the transition from the easterly winds to the westerly winds. This variation in winds is consistent with the formation of the Indo – Australian monsoon.

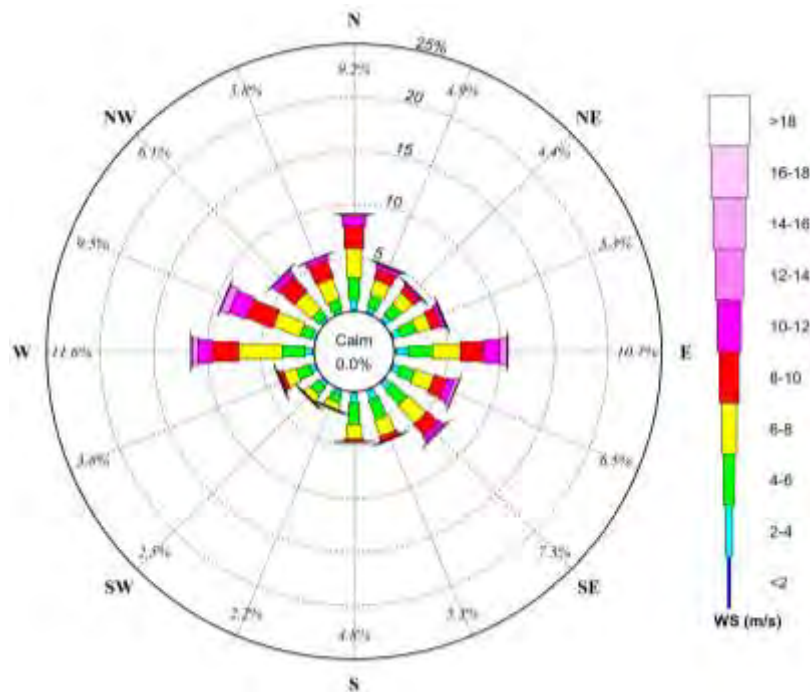


Figure 5-4 Annual wind rose based on Darwin airport wind data (1985 to 2019)

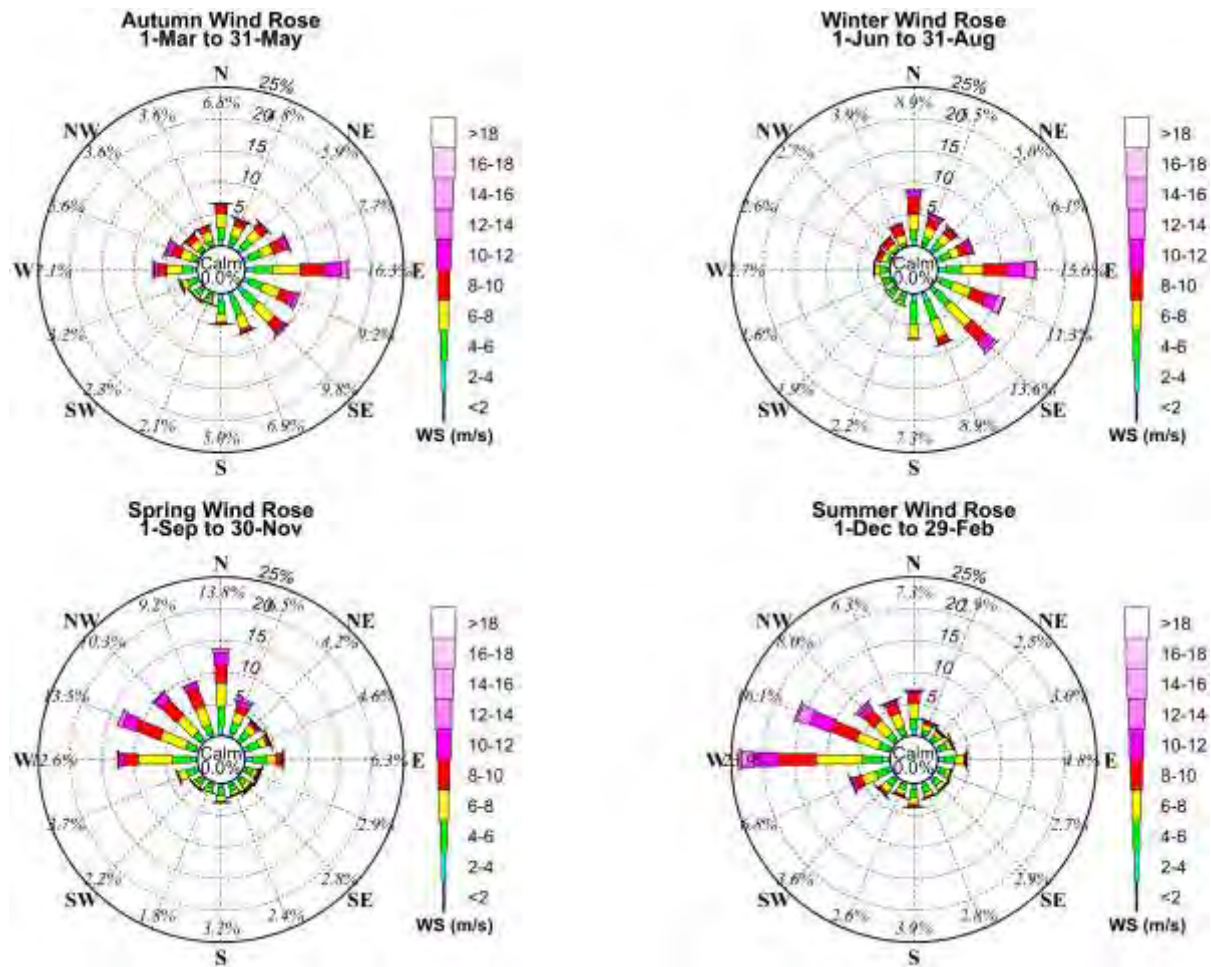


Figure 5-5 Seasonal wind roses based on Darwin airport wind data (1985 to 2019)

Table 5-1 Monthly and annual wind at Darwin airport station, 1985 to 2019.

Time Period	Wind Speed (10 min average in m/s)		Average Direction	Dominant Direction			
	Average	Maximum		North	East	South	West
January	7.7	24.1	239	21%	11%	13%	55%
February	7.7	34.1	243	18%	11%	11%	60%
March	6.9	28.9	208	22%	22%	16%	39%
April	6.7	27.4	142	20%	50%	18%	13%
May	7.1	17.4	129	17%	53%	25%	5%
June	7.3	21.3	139	16%	49%	30%	5%
July	7.2	17.4	147	22%	43%	27%	7%
August	7.1	17.4	166	28%	35%	21%	16%
September	7.1	19.8	208	36%	21%	11%	32%
October	7.3	27.9	221	40%	16%	7%	37%
November	6.9	20.8	226	32%	16%	11%	41%
December	7.2	24.1	222	24%	18%	12%	46%
Annual	7.2	34.1	190	25%	29%	17%	30%

In order to determine the extreme wind speed statistics, peak wind speeds were extracted from the dataset using a 'peak over threshold' analysis. From here, extreme wind speeds were determined by undertaking extreme value analyses (EVA). EVA has the objective of estimating an expected value of an extreme event that would be equalled or exceeded once in a long period of time (Goda, 2000). The return period is defined as the average duration of time between the occurrences of extreme values exceeding a certain threshold value.

In order for the EVA to generate probabilistic quantification of the wind speed, it was necessary to apply the use of statistical models. The Weibull distribution is one of the most widely used distributions for the analysis of lifetime and time-to-event data (Robinson et al, 2014). The two-parameter Weibull probability function is very popular for such applications due to the wide range of shapes that it can take. There are numerous methods available to fit the distribution and estimate the shape parameters, including (but not limited to) the method of moments, maximum likelihood estimation, or the least squares linear regression method. Of these methods, the maximum likelihood estimation is often favoured by statisticians and engineers because its characteristics can be examined mathematically (Goda, 2000) and does not require the assumption of a plotting position formula. Therefore, for this study the EVA is undertaken using the maximum likelihood method, fitting to the Weibull Distribution.

The resultant non-cyclonic extreme wind speeds are presented below in **Table 5-2**. This analysis indicates that the strongest winds tend to come from the west.

Table 5-2 Directional (non-cyclonic) extreme wind speeds, 10-minute average measured 10m above ground level.

Return Period (years)	Directional Extreme Wind Speeds (m/s)								Omni Direction
	N	NE	E	SE	S	SW	W	NW	
1	13.8	14.9	13.8	16.8	14.4	14.3	16.3	15.9	18.9
2	14.1	16.0	14.1	17.4	15.3	15.2	17.6	17.5	20.5
5	15.1	18.5	15.1	18.7	17.2	17.2	20.3	19.6	23.4
10	16.4	20.8	16.4	20.0	18.9	19.0	23.1	21.2	25.9
25	18.7	24.3	18.7	22.0	21.4	21.8	27.3	23.4	29.7
50	20.9	27.3	20.9	23.6	23.5	24.1	31.0	25.0	32.8
100	23.6	30.5	23.6	25.4	25.7	26.6	35.0	26.6	36.0
200	26.7	33.9	26.7	27.4	28.1	29.2	39.3	28.2	39.4
500	31.4	38.8	31.4	30.1	31.4	33.0	45.4	30.4	44.1

5.1.2.3 Gust Factors

Based on Australian guidelines for the design of maritime structures (AS 4997), wind actions on wharves and wharf buildings and on stored materials or vehicles should be designed in accordance with Australian Standard, Structural design actions, Part 2: Wind actions (AS/NZS 1170.2). As per AS 4997, it is also stipulated that wind actions on vessels and floating structures may be designed using a wind speed based on a 30 second gust rather than basic wind speeds due to 3 second gusts. This is because floating structures have a delayed response to wind loads. The 30-second wind speed may be taken as 0.87 times the relevant basic wind speed as specified in AS/NZS 1170.2.

In summary, design wind speed in terms of the 3 second gust shall be considered for the design of wharves, wharves buildings, stored materials and vehicles, whilst the wind actions on the vessels and floating structures should be designed using a wind speed based on a 30 second gust.

Based on Guidelines for converting between various wind averaging periods in tropical cyclone conditions (World Meteorological Organization, 2010), a factor of 1.52 is recommended to convert 10-minute average wind speed to 3 second gust wind. In order to determine the design wind speed in terms of 3 second and 30 second gust wind, results from Table 5-2 should be multiplied by 1.52 and 1.33 (1.52×0.87), respectively.

Note that the above wind speeds have been applied to the coastal processes' investigations. For derivation of wind loads on structures, the wind speeds outlined in AS/NZS 1170 will be used.

5.2 Oceanography

5.2.1 Water Levels

Coastal water levels in the study area are dominated by the astronomical tide. However, significant short duration variations from the predicted tide can occur. During tropical cyclone events, these variations are referred to as storm tide. Storm tides are the combination of astronomical tides and the increase in the still water level due to storm, as shown in **Figure 5-6**. The physical processes that contribute to storm tide include:

- > Wind Set-up;
- > The Inverse Barometer Effect;
- > Wave Set-up; and
- > Wave Run-up – which is normally addressed separately from storm tide.

In addition to storm tide, water levels in the study area will be affected by longer term physical processes that act over timescales ranging from weeks to decades to millennia. These include:

- > Long term mean sea level rise due to climate change;
- > Variations in Mean Sea Level (MSL) due to regional oceanographic and atmospheric phenomena such as the El Niño Southern Oscillation (ENSO) and the Pacific Decadal Oscillation (PDO); and
- > Long term Eustatic and Tectonic Changes to mean sea level.

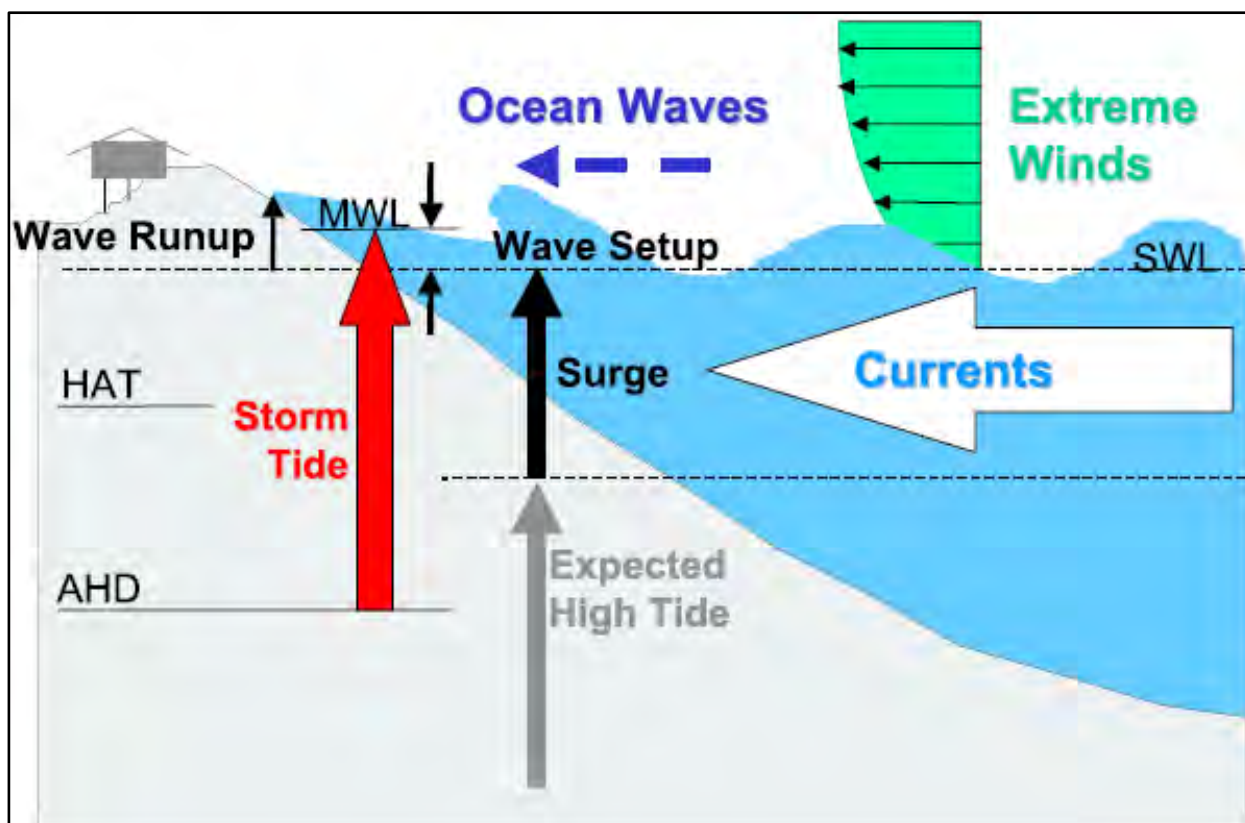


Figure 5-6 Storm Tide Schematic [Courtesy: QLD Government]

5.2.1.2 Tides

Tides in Darwin Harbour are semi-diurnal, which means it experiences two high tides and two low tides per day. Tidal planes for Darwin harbour (National Tides Centre, pers. comm 2020) are presented in **Table 5-3**, relative to Australian Height Datum (AHD) and Chart Datum (CD).

Table 5-3 Tide planes for Darwin Harbour (National Tides Centre, 2020)

Tidal Plane	Level (m, CD)	Level (m AHD)
Highest Astronomical Tide (HAT)	8.17	4.07
Mean High Water Springs (MHWS)	7.05	2.94
Mean High Water Neaps (MHWN)	5.13	1.02
Mean Sea Level (MSL)	4.24	0.13
Australian Height Datum (AHD)	4.105	0.00
Mean Low Water Neaps (MLWN)	3.34	-0.76
Mean Low Water Springs (MLWS)	1.42	-2.68
Lowest Astronomical Tide (LAT)	-0.01	-4.11

5.2.1.3 Non-Tidal Water Level Variations

Wind Set-up

Non-periodic variations from the astronomical tide are typically associated with the effect of wind on sea level. When the wind blows over an open body of water, drag forces develop between the air and the water surface. These drag forces are proportional to the square of the wind speed. The result is that a wind drift current is generated. This current may transport water towards the coast, where it then piles up causing wind set-up. Wind set-up is inversely proportional to depth, which means that it becomes more severe where depths are shallow near the coast.

Additionally, along-coast wind driven currents may also cause a set-up through the Coriolis Effect and refraction.

The Inverse Barometer Effect

The 'inverse barometer effect' is the result of the effect that atmospheric (also known as barometric) pressure has on local sea levels. When the barometric pressure increases, it applies a local depressing force on sea levels. However, when barometric pressure drops, such as during severe tropical cyclone events, it causes local sea levels to temporarily rise, typically by up to 1 cm for each hecta-Pascal (hPa) drop in pressure below the regional average, (which is typically about 1006-1012 hPa). The actual increase depends on the speed of the meteorological system and 1 cm is only achieved if it is moving slowly. The phenomenon causes daily variations from predicted tide levels up to 5 cm. However, during severe tropical cyclones, where the central barometric pressure can be in the vicinity of 950-980 hPa, local sea levels may increase temporarily by tens of centimetres.

Wave Set-up

When waves break on a beach, they produce wave set-up, which is an increase in the nearshore water level above the still water elevation of the sea (**Figure 5-6**). Wave set-up can be considered as a piling up of water against the shoreline that is caused by breaking waves causing a transfer of kinetic to potential energy.

As waves approach the shoreline and the water depth decreases, wave height and wave radiation stresses increase due to wave shoaling. The resulting positive gradient in radiation stress must be balanced by a negative water surface gradient, known as wave set-down. However, in the surf zone where waves break,

wave heights and radiation stresses then decrease as wave energy dissipates, and hence the negative gradient in radiation stress must be balanced by a positive water surface gradient resulting in an elevated water level shoreward of the surf zone. This is known as wave set-up.

Wave setup can be particularly severe during storm events, when the effects of big waves and the resultant wave set-up can be a significant contributor to the overall resultant storm tide inshore of the breaker zone.

5.2.1.4 Wave Run-up

Wave run-up is the vertical distance that a wave will run up a beach or coastal structure after it breaks. Wave run-up height is defined as the difference between the maximum elevation that a wave runs up the beach or a coastal structure, and the still water level (comprised of the astronomical tide plus storm surge). Wave run-up level is defined as the maximum vertical elevation of the run-up. Generally speaking wave run-up heights depends on the wave height and length, the slope of the sea bed, the form of the shoreline. Wave run-up heights and levels therefore change on a wave by wave basis.

5.2.1.5 Sea Level Rise

It is generally accepted that climate change will impact coastal flooding in the future. The predominant mechanisms are:

- > Sea level rise; and
- > Cyclone intensification.

According to the Intergovernmental Panel on Climate Changes (IPCC) fifth assessment report (AR5) released in 2013, the thermal expansion of the oceans and glacial melting have been the dominant contributors to 20th century global mean sea level rise. AR5 makes projections for global mean sea level rise during the 21st century. Projections are made for a number of different global greenhouse gas emissions scenarios, known as Representative Concentration Pathways (RCP's). AR5 states that *"It is very likely that the rate of global mean sea level rise during the 21st century will exceed the rate observed during 1971–2010 for all Representative Concentration Pathway (RCP) scenarios due to increases in ocean warming and loss of mass from glaciers and ice sheets"*. The AR5's Global Mean Sea Level Rise (GMSLR) projections are reproduced in Figure 5-7 and Table 5-4.

The Queensland Coastal Hazard Technical Guideline Determining Coastal Hazard Areas (DEHP, 2013) states that the implications of projected sea level rise for Queensland's coast include a progressive worsening of the apparent severity and frequency of storm tide inundation that will cause inundation to occur further inland. The Queensland Department of Environment and Heritage Protection presently adopts a Mean Sea Level Rise (MSLR) projection of 0.8 m above present day levels by 2100. This is in line with the AR5 (IPCC, 2013) RCP8.5 emission scenario.

Based on the current research and sea level rise horizons adopted within Australia it is recommended to include an allowance of 0.3m in 2050 and 0.8m in 2100. This translates to an allowance of 0.43m at 2075 – aligning to the 50-year design life.

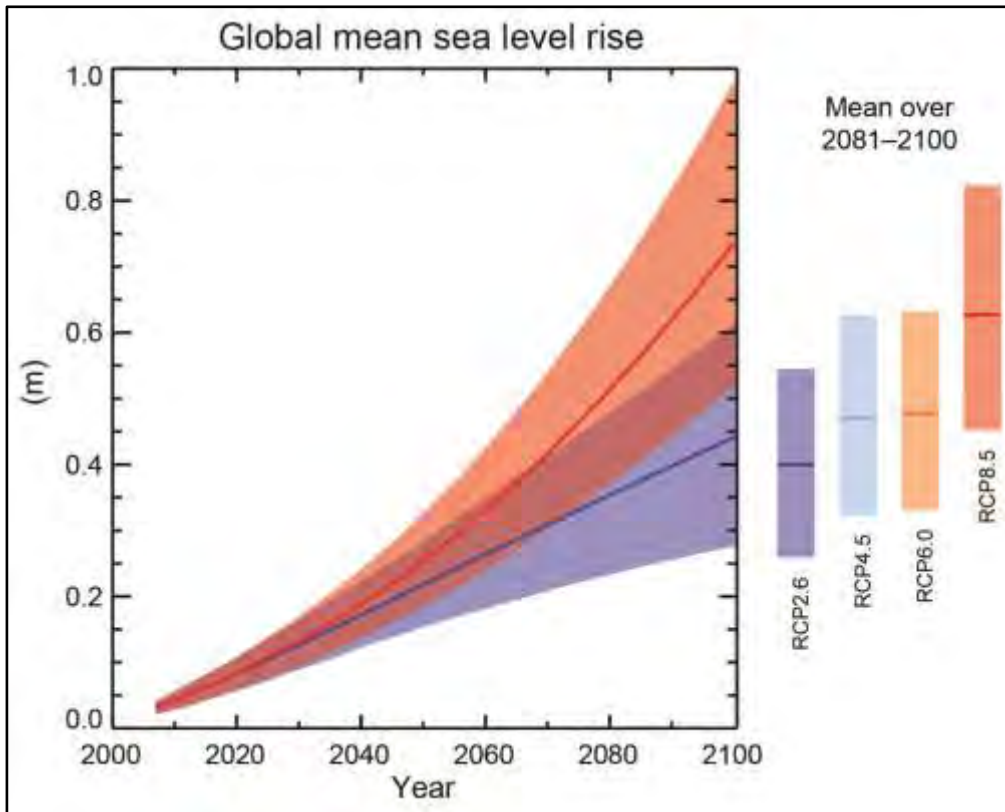


Figure 5-7 AR5's (IPCC, 2013) global mean sea level rise projections

Table 5-4 Global mean sea level rise projections in AR5 (IPCC, 2013)

Scenario Name	Global Annual Greenhouse Gas Emissions Scenario	MSLR Mean Over 2046-2065 Mean and likely (>66% probability) range (m)	MSLR Mean Over 2081-2100 Mean and likely (>66% probability) range (m)	MSLR In 2100 Mean and likely (>66% probability) range (m)
RCP2.6	Emissions peak between 2010 and 2020, and decline substantially thereafter.	0.24 (0.17 to 0.32)	0.40 (0.26 to 0.55)	0.44 (0.28 to 0.61)
RCP4.5	Emissions peak around 2040, then decline.	0.26 (0.19 to 0.33)	0.47 (0.32 to 0.63)	0.53 (0.36 to 0.71)
RCP6.0	Emissions peak around 2080, then decline.	0.25 (0.18 to 0.32)	0.48 (0.33 to 0.63)	0.55 (0.38 to 0.73)
RCP8.5	Emissions continue to rise throughout the 21 st century.	0.30 (0.22 to 0.38)	0.63 (0.45 to 0.82)	0.74 (0.52 to 0.98)

5.2.1.6 Extreme Water Levels

Cardno has recently carried out a metocean study in Darwin harbour for the Darwin Fort Hill project (Cardno, 2019). Measured water level data was available for Darwin Fort Hill from 1992 to present. Cardno undertook an extremal analysis on this data following the Pugh & Vassie (1978) method. The method allows derivation of probability of occurrence of extreme sea levels through a combination of the respective probability functions of surge and the astronomical tide. This method implicitly accounts for the likelihood of extreme water levels occurring as the result of different combinations of storm surge and astronomical tide level (**Table 5-5**). Site specific cyclonic storm tide levels have also been derived for the site (refer to **Section 8.3**). These values are also presented below for comparison.

Table 5-5 Present day cyclonic and non-cyclonic extreme storm tide levels derived for the site

Return Period (years)	Non-cyclonic (m, AHD)	Cyclonic (m AHD)	Recommended Design Storm Tide Levels (m AHD)	Recommended Design Storm Tide Levels (m CD)
1	3.97		3.97	8.08
2	4.02		4.02	8.13
5	4.08	3.01	4.08	8.19
10	4.12	3.55	4.12	8.23
25	4.17	4.03	4.17	8.28
50	4.21	4.32	4.32	8.43
100	4.24	4.57	4.57	8.68
200	4.28	4.78	4.78	8.89
500	4.33	5.03	5.04	9.14

Considering an allowance of 0.43m in 2075 due to effects of sea level rise, the design storm tide level (future extreme water levels) for the project was calculated and is presented in Table 5-6.

Table 5-6 Design storm tide levels

Return Period (years)	Present Day Design Storm Tide Levels		2075 Design Storm Tide Levels	
	(m AHD)	(m CD)	(m AHD)	(m CD)
1	3.97	8.08	4.40	8.51
2	4.02	8.13	4.45	8.56
5	4.08	8.19	4.51	8.62
10	4.12	8.23	4.55	8.66
25	4.17	8.28	4.6	8.71
50	4.32	8.43	4.75	8.86
100	4.57	8.68	5.00	9.11
200	4.78	8.89	5.21	9.32
500	5.04	9.14	5.47	9.58

6 Hydrodynamic Conditions

6.1 Hydrodynamic Model

In order to determine the operational tidal currents and cyclone storm surge levels at the site, numerical models were used. A hydrodynamic model was used to define the sea-level and currents under tidal conditions only, and with tropical cyclone wind and pressure forcing in addition to the tides. The following section describes the model set up.

6.1.1 Tides, Sea Level and Currents

For the purposes of this study, the Delft3D hydrodynamic model system has been utilised. Delft3D has been developed by Deltares in The Netherlands. The Delft3D modelling system includes wind, pressure, tide and wave forcing, currents, stratification, sediment transport and water quality descriptions and is capable of using irregular rectilinear or curvilinear coordinates. In this case, the hydrodynamic model was operated in 2D depth-averaged mode.

Many nearshore areas include shallow inter-tidal regions; consequently, Delft3D includes a robust and efficient wetting and drying algorithm to handle this process. This is particularly important for Darwin Harbour, where there are extensive mangrove and intertidal areas.

Delft3D provides the facility to adopt a modelling approach known as domain decomposition. Domain decomposition is a technique in which a model is divided into several smaller model domains that are dynamically coupled. Domain decomposition is widely recognised as an efficient and flexible tool for the simulation of complex physical processes.

Domain decomposition was applied in the model to allow efficient modelling. A coarse grid model was set up of the Darwin estuary entrance which meet finer grid domain extended north of Darwin to Melville and Bathurst Islands. Successively finer meshes were then adopted to give a resolution able to resolve the features of the channels and intertidal areas. The arrangement of these grids is shown in **Figure 6-1** and **Figure 6-2**. The astronomical tide was applied at three open boundaries of the Darwin Entrance model as a time series. The boundaries were generated from the Beagle Gulf model.

The Beagle Gulf model domain is shown in **Figure 6-1**. The tides were specified along the open boundaries of the Beagle Gulf hydrodynamic model using the 13 constituents from the FES2004 global tidal model (Lyard et al., 2006). The tides are provided as complex amplitudes of earth-relative sea-surface elevation for eight primary (M2, S2, N2, K2, K1, O1, P1, Q1) and two long period (Mf, Mm) harmonic constants.

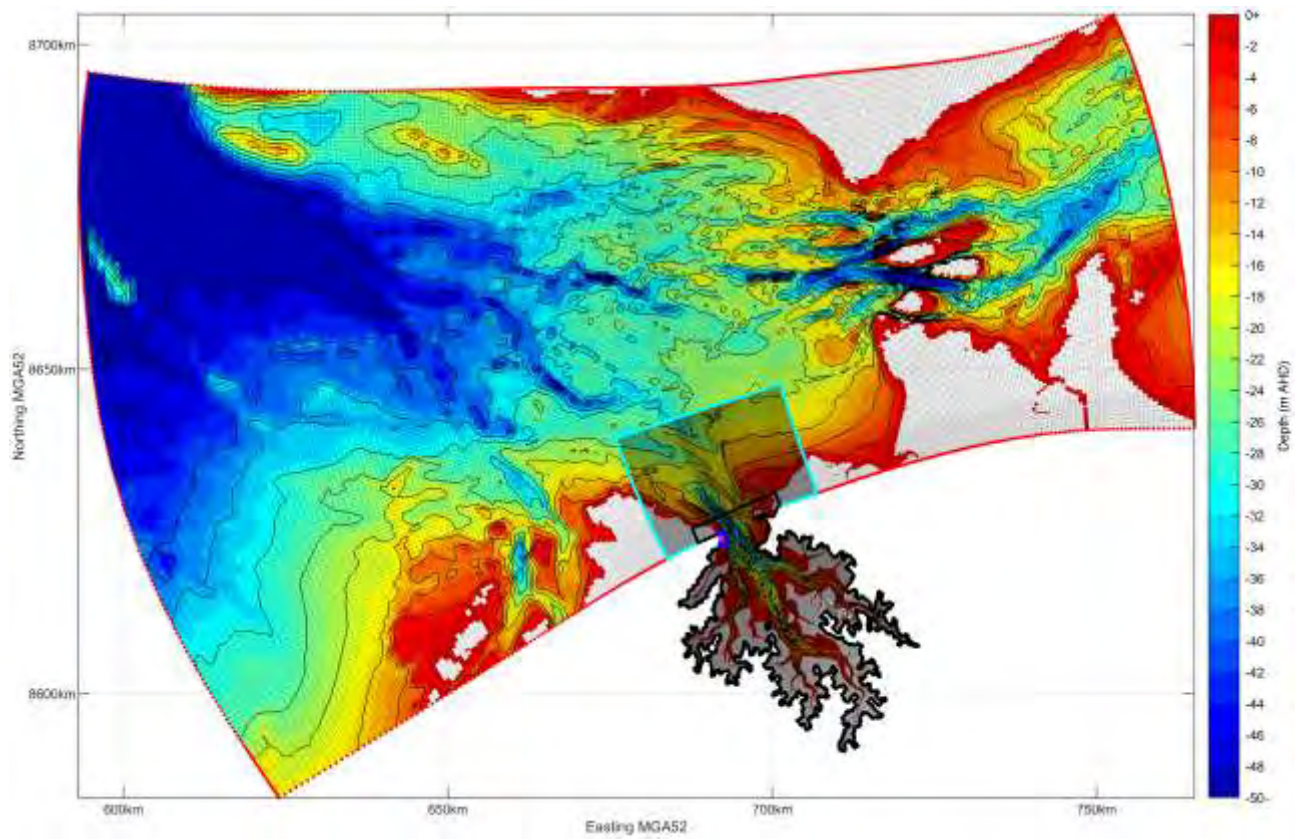


Figure 6-1 Hydrodynamic model grids: Beagle Gulf (red), Entrance (cyan), Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Beagle Gulf bathymetry

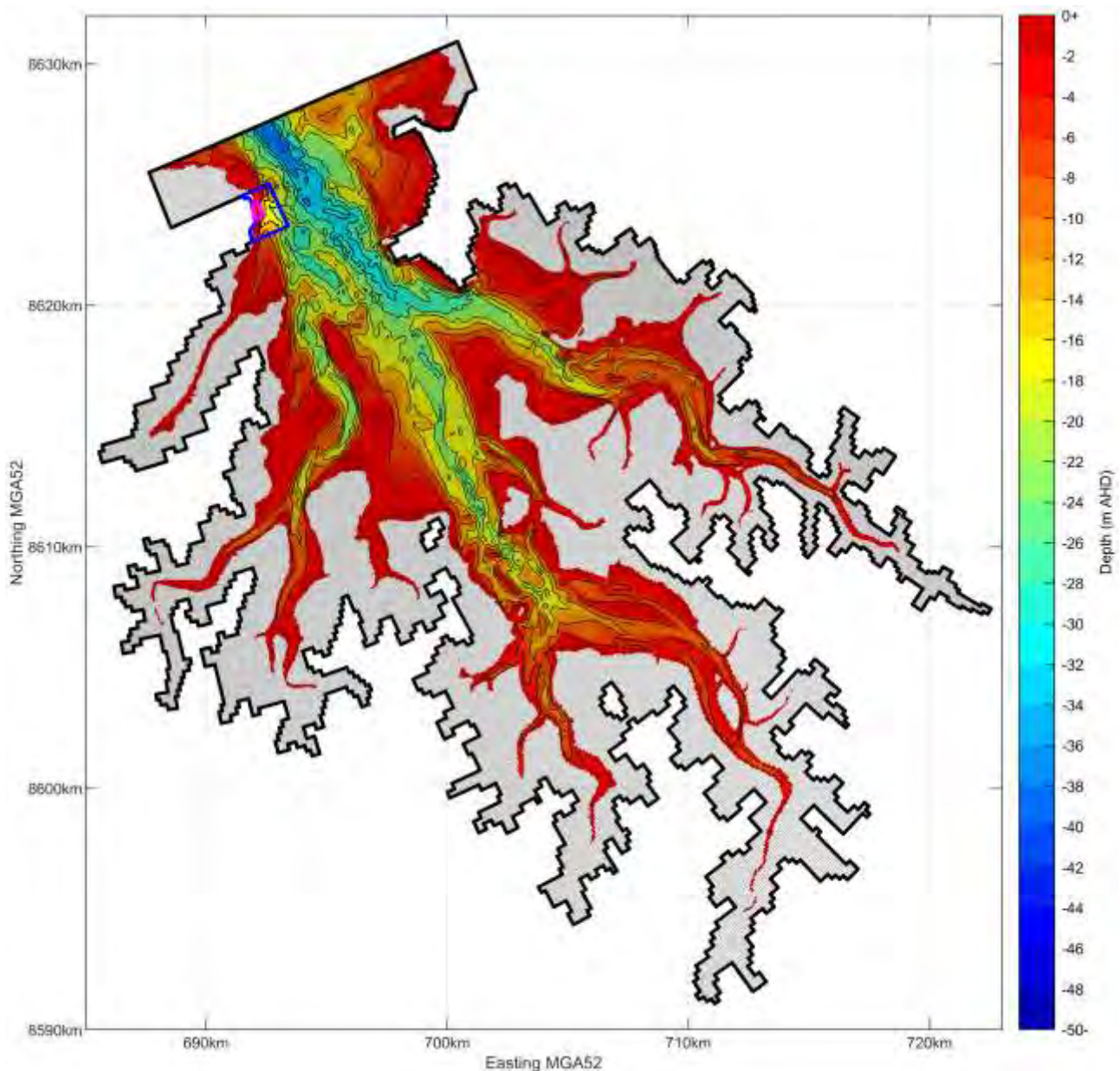


Figure 6-2 Hydrodynamic model grids: Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Darwin Harbour bathymetry

6.1.2 Cyclone Surge

Historical cyclone track data was obtained from the Bureau of Meteorology cyclone track database (BoM, 2019). From this a total of twenty five cyclones that affected the study area were identified.

The wind and pressure fields for these cyclones were prepared using the Holland wind model developed for the Bureau of Meteorology. The model includes first-order asymmetry based on the forward speed of the cyclone. This model is considered to provide the most realistic description of cyclonic wind fields for the Australian region. Cyclone parameters required for the Holland wind model were extracted from the existing literature where available; for example, Bureau of Meteorology cyclone data and sea level pressure charts as well as several publications: DNRM (2001), McConochie (2004) and Harper et al. (2001).

The empirical formula proposed in Holland (1980) has been used to determine the 'B' shape parameter value - it was adopted to be 1 when data were not available. Neutral pressure (ambient atmospheric pressure at the periphery of the cyclone) has been taken to be 1012hPa.

Storm surge was modelled using the FLOW hydrodynamic module of Delft3D. Cyclone wind and pressure fields were applied to the hydrodynamic grid and storm surge heights were calculated using the predicted astronomical tide for the timing of each cyclone.

6.1.3 Model Skill Assessment

Model Limitations and Accuracy

Limitations of the modelling undertaken for this study are due to the main inputs into the models such as the tidal boundary obtained from the global tidal model which does not take into account regional scale residual water levels.

As with all numerical modelling techniques, there is a requirement to make certain assumptions and there exists some limitation in the application of the numerical scheme to certain hydrodynamic settings. The Delft3D model applied to this study is capable of integrating wind, pressure, tide and wave forcing, three-dimensional currents, stratification, sediment transport and water quality descriptions. The numerical scheme is intended for regional and local scale hydrodynamic investigations, but not for finer scale local turbulence problems. Such an application would require the use of a high level CFD model. One of the initial limitations evident from the model setup was the large area that is required to be described by the model while providing adequate detail over the particular area of interest. To overcome these conflicting requirements domain decomposition was utilised, however, model size is still a concern and must always be kept manageable so as to ensure efficient run times.

To gain confidence that the model is describing the hydrodynamic conditions at the site well, a calibration and verification exercise was undertaken and is presented below.

Model Calibration and Validation

The hydrodynamic model results have been compared to measured water levels, current speeds and directions at the six sites within Darwin Harbour, listed in **Table 6-1**, during the period of February to May 2013.

Table 6-1 Measurements in Darwin Harbour used for model validation

Locations	East	North	From	To
Mandorah ADCP (2020)	692214	8624001	12/12/19	22/4/20
Mandorah ADCP (2017)	692253	8623943	1/9/17	7/10/17
Mandorah ADCP (2013)	692797	8624311	24/12/12	20/3/13
Fort Hill Wharf ADCP	699976	8620515	9/4/13	9/5/13
Middle Arm Channel ADCP	700678	8615496	9/4/13	8/5/13
Upper East Arm ADCP	714477	8613172	28/9/12	21/3/13
Weed Reef ADCP	696147	8618483	23/11/12	20/3/13
Wickham Point-ADCP	702473	8619911	3/4/13	8/5/13

The comparisons of modelled and measured water levels and currents are presented below in **Figure 6-3** through **Figure 6-10**. The comparison shows that the hydrodynamic model achieves good agreement with the measurements at all of the measurement locations.

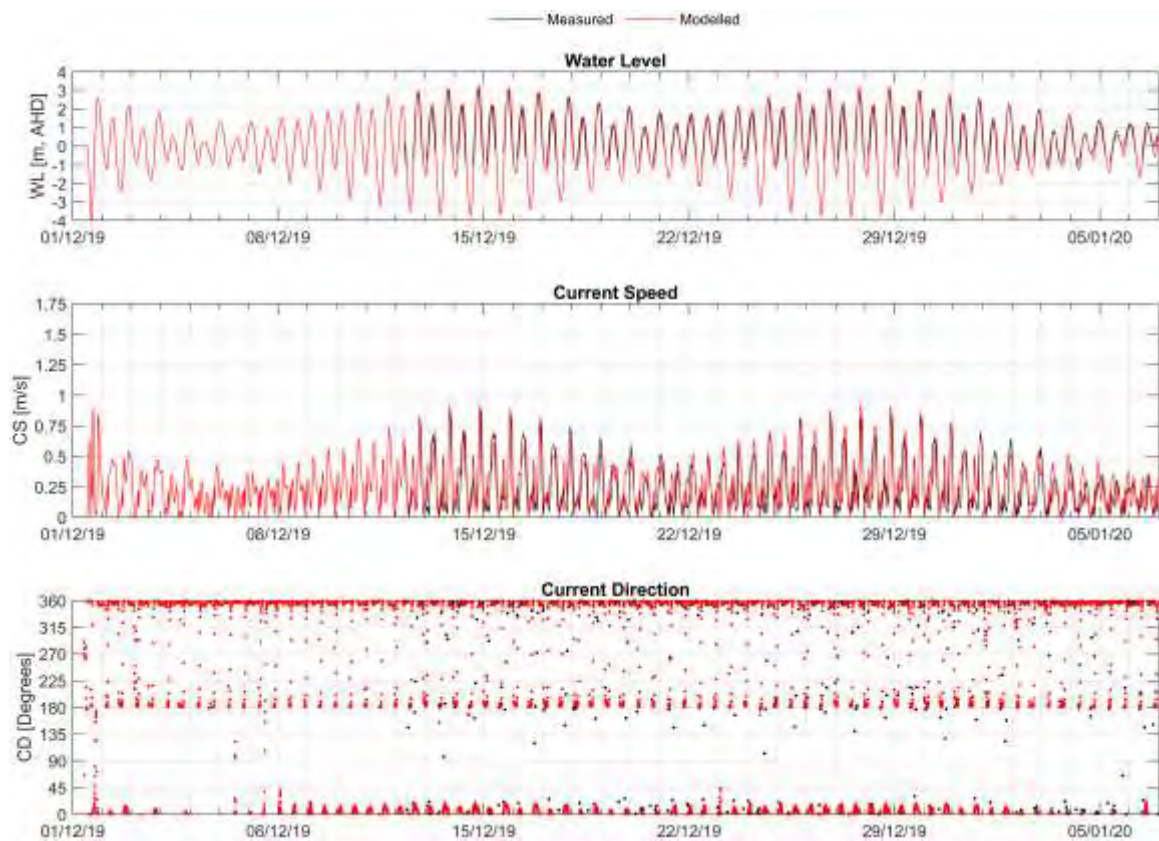


Figure 6-3 Comparison of measured and modelled depth averaged current speeds and directions at the Mandorah ADCP 2020

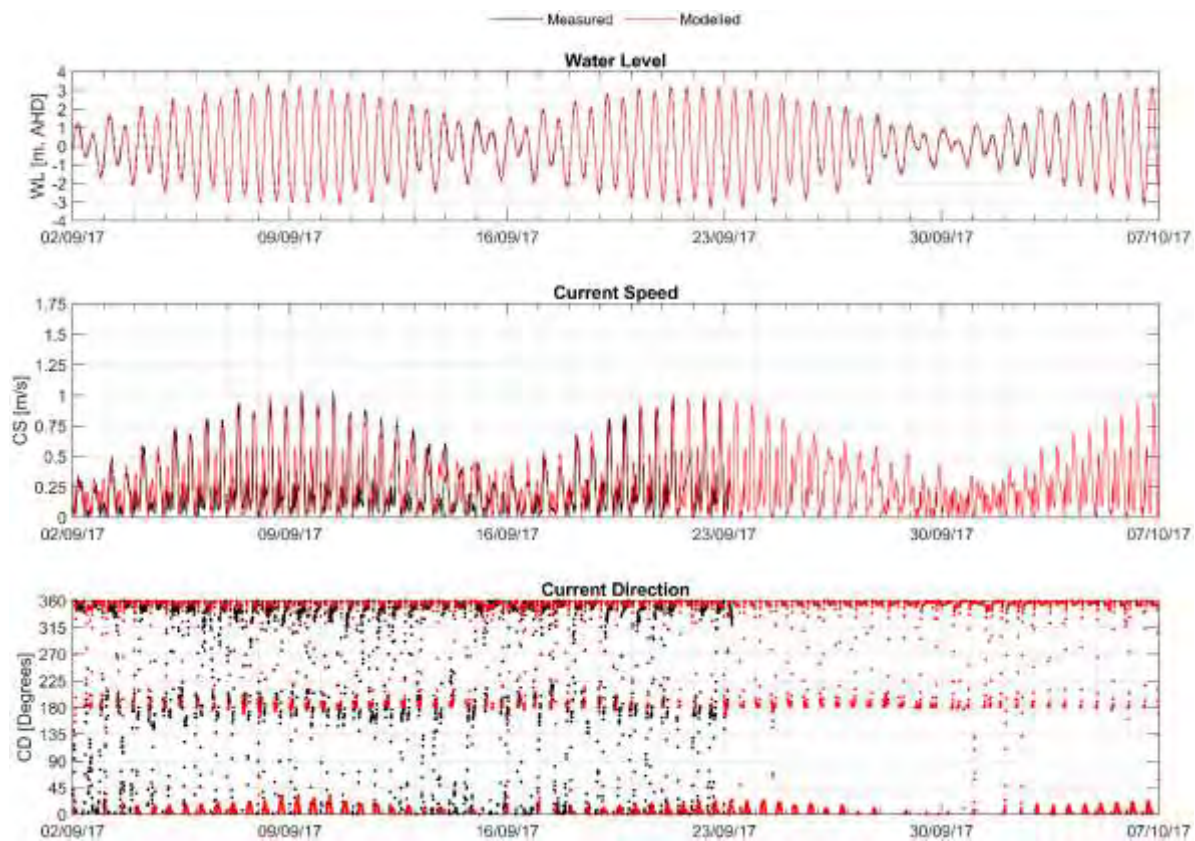


Figure 6-4 Comparison of measured and modelled current speeds and directions at the Mandorah ADCP 2017

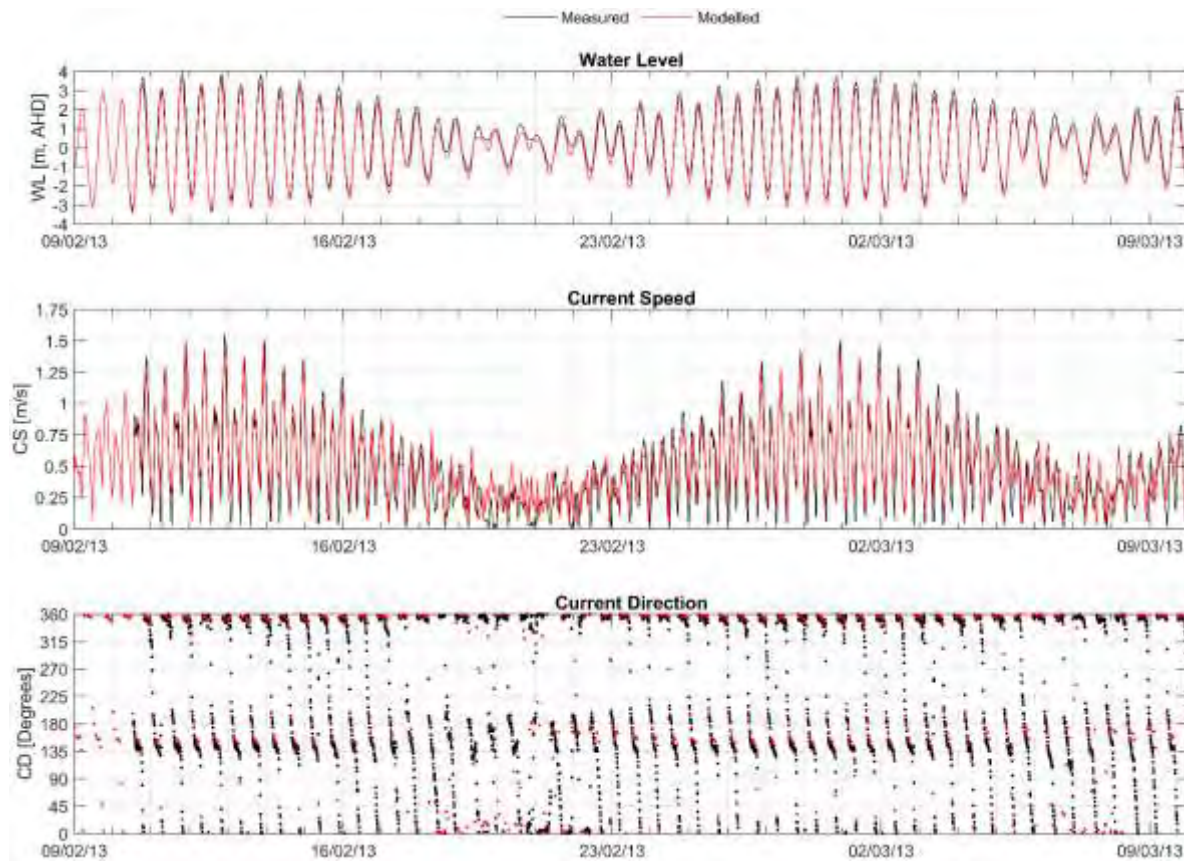


Figure 6-5 Comparison of measured and modelled current speeds and directions at the Mandorah ADCP 2013

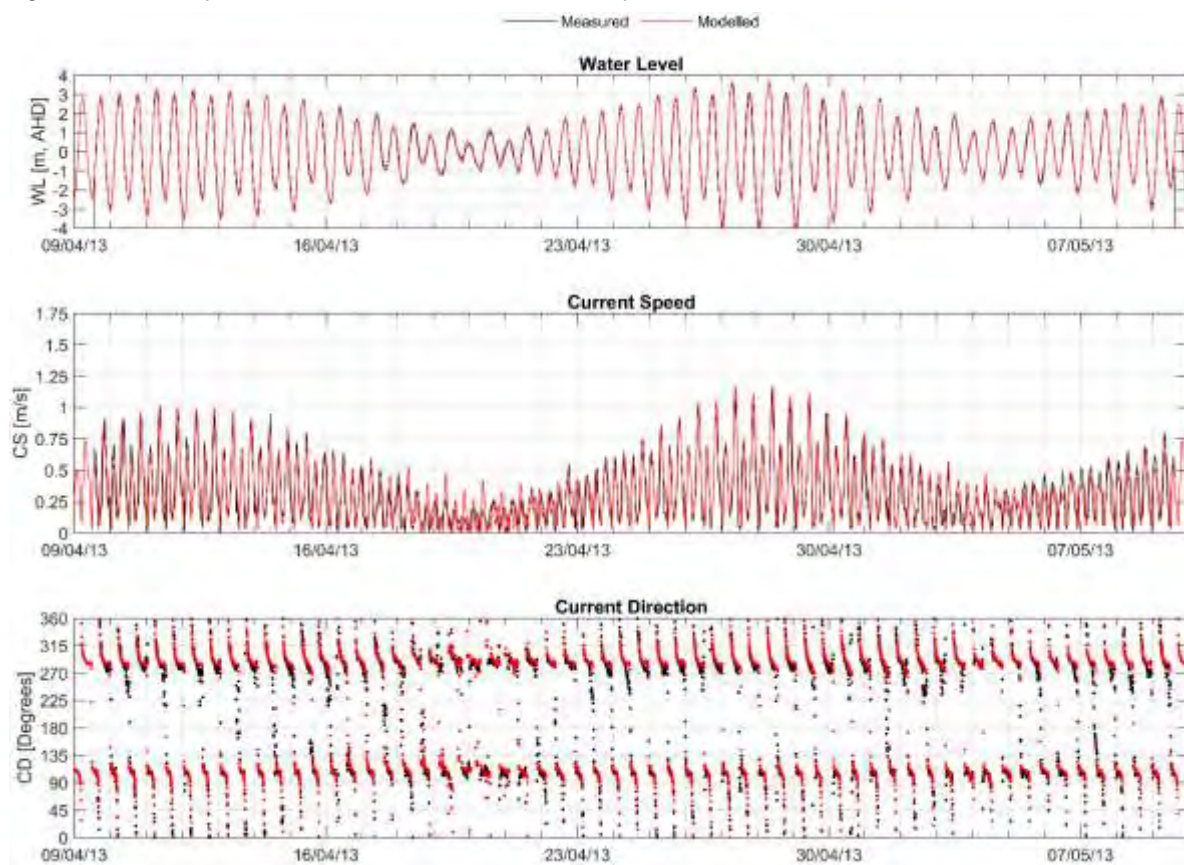


Figure 6-6 Comparison of measured and modelled current speeds and directions at the Fort Hill Wharf ADCP

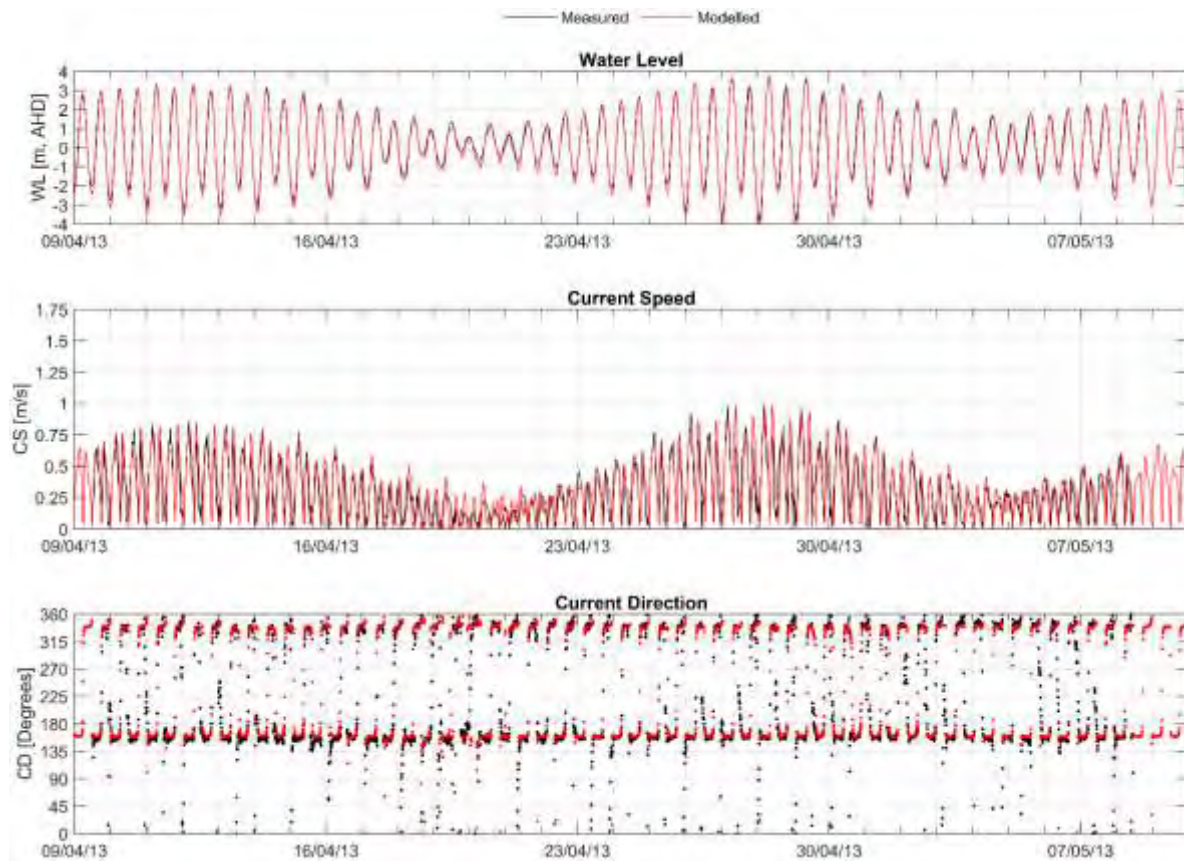


Figure 6-7 Comparison of measured and modelled current speeds and directions at the Middle Arm Channel ADCP

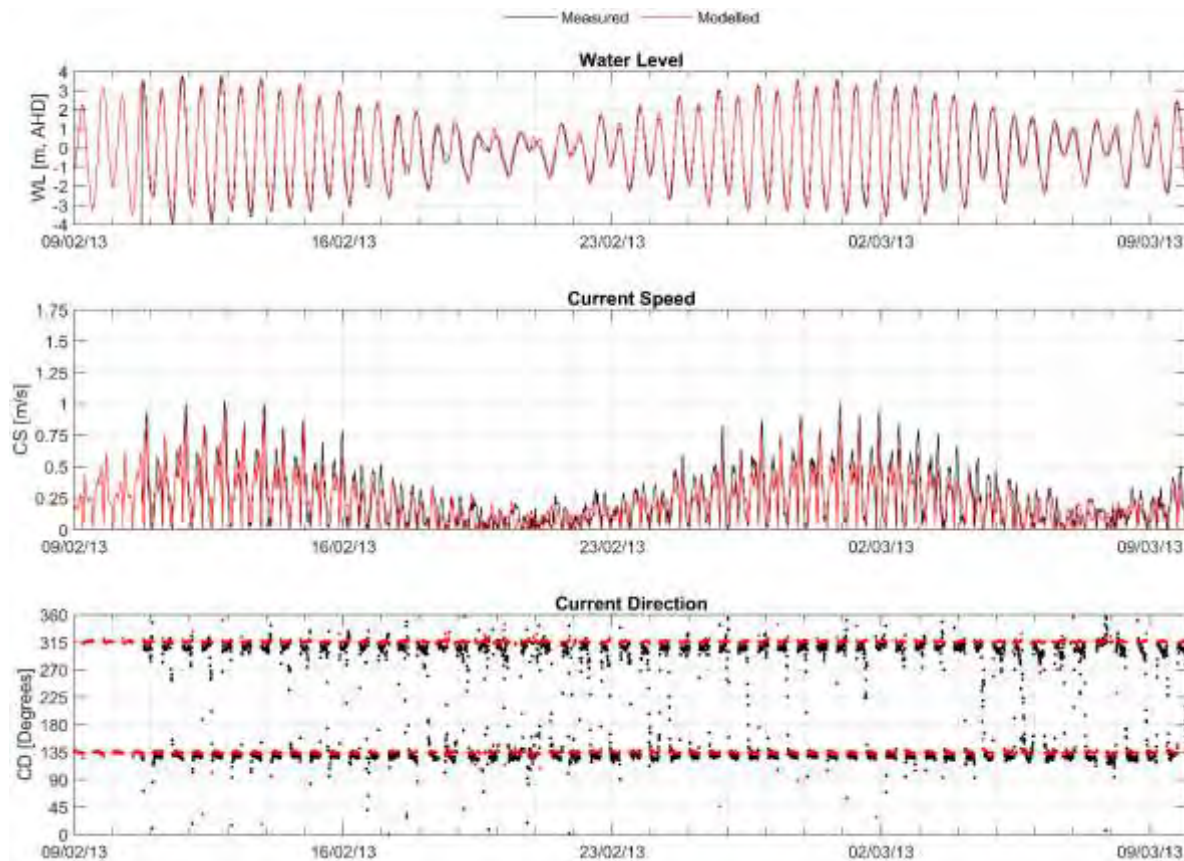


Figure 6-8 Comparison of measured and modelled current speeds and directions at the Upper East Arm ADCP

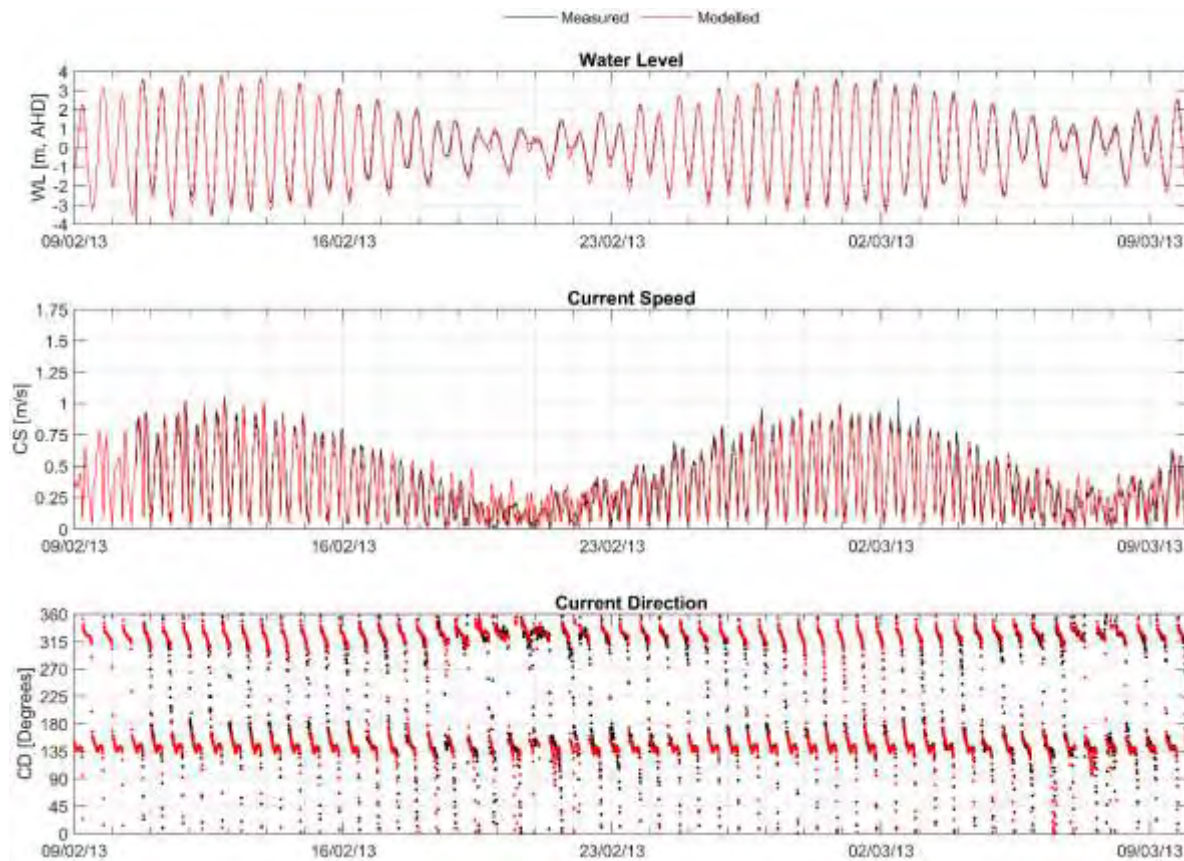


Figure 6-9 Comparison of measured and modelled current speeds and directions at the Weed Reef ADCP

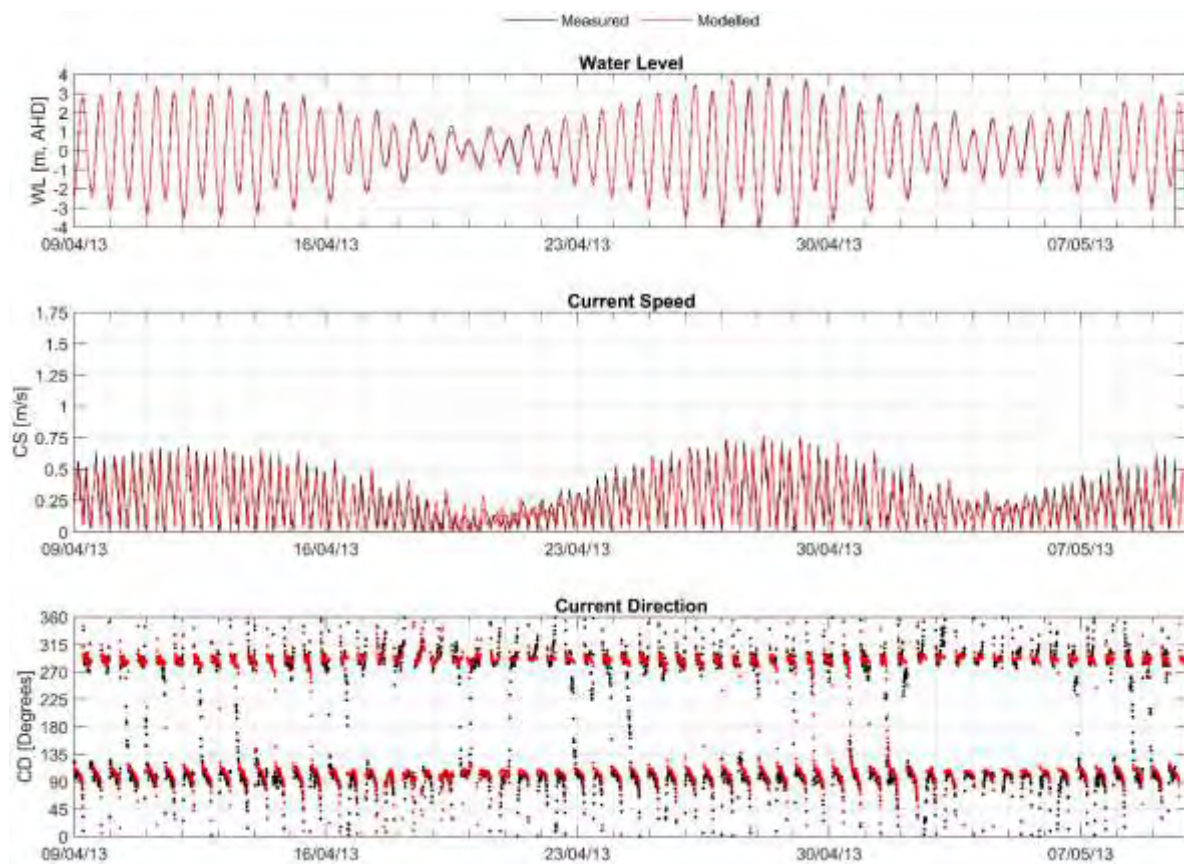


Figure 6-10 Comparison of measured and modelled current speeds and directions at the Wickam Point ADCP

6.2 Currents

The major driver of currents in Darwin Harbour is the tide. Currents at the Project site have been extracted from a tidal hydrodynamic model at the extraction location show in **Figure 6-11**, below. The model was run for a 3-month period covering February, March and April of 2013, using the astronomical tides along the boundary at a water depth of around 12.0 m AHD. **Figure 6-12** depicts the tidal current rose produced at the Project site (depicts direction current heading). **Figure 6-13** shows the time series of water level, depth average current speeds and direction at around 12m depth fronting the site.



Figure 6-11 The location of wave and current data extraction point at Mandorah

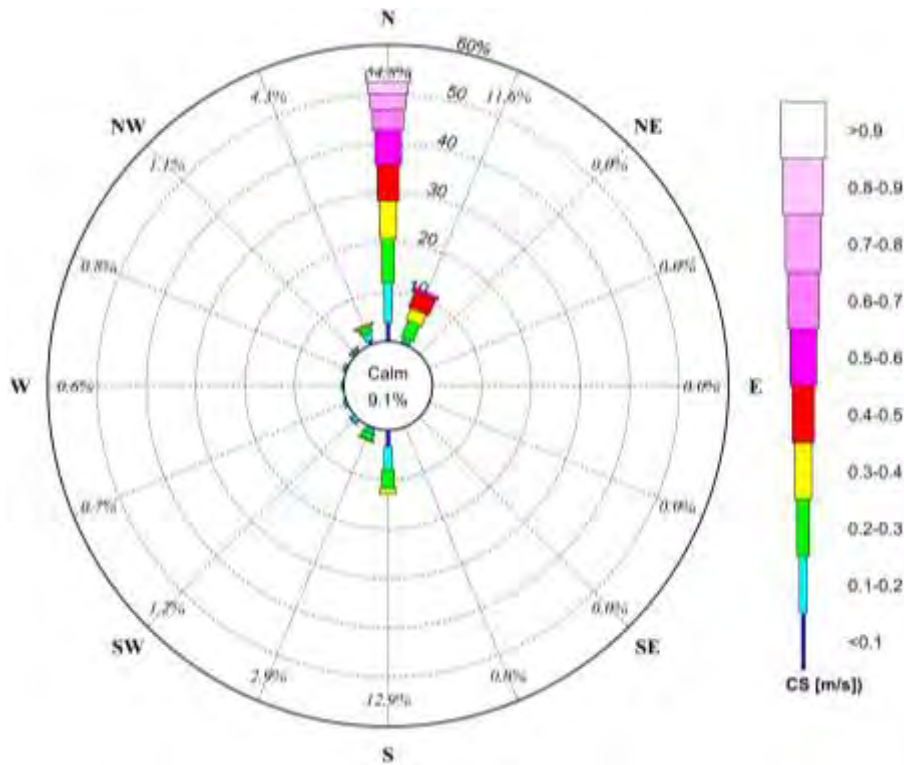


Figure 6-12 Tidal current rose over the modelling period (100 days) at the extraction point

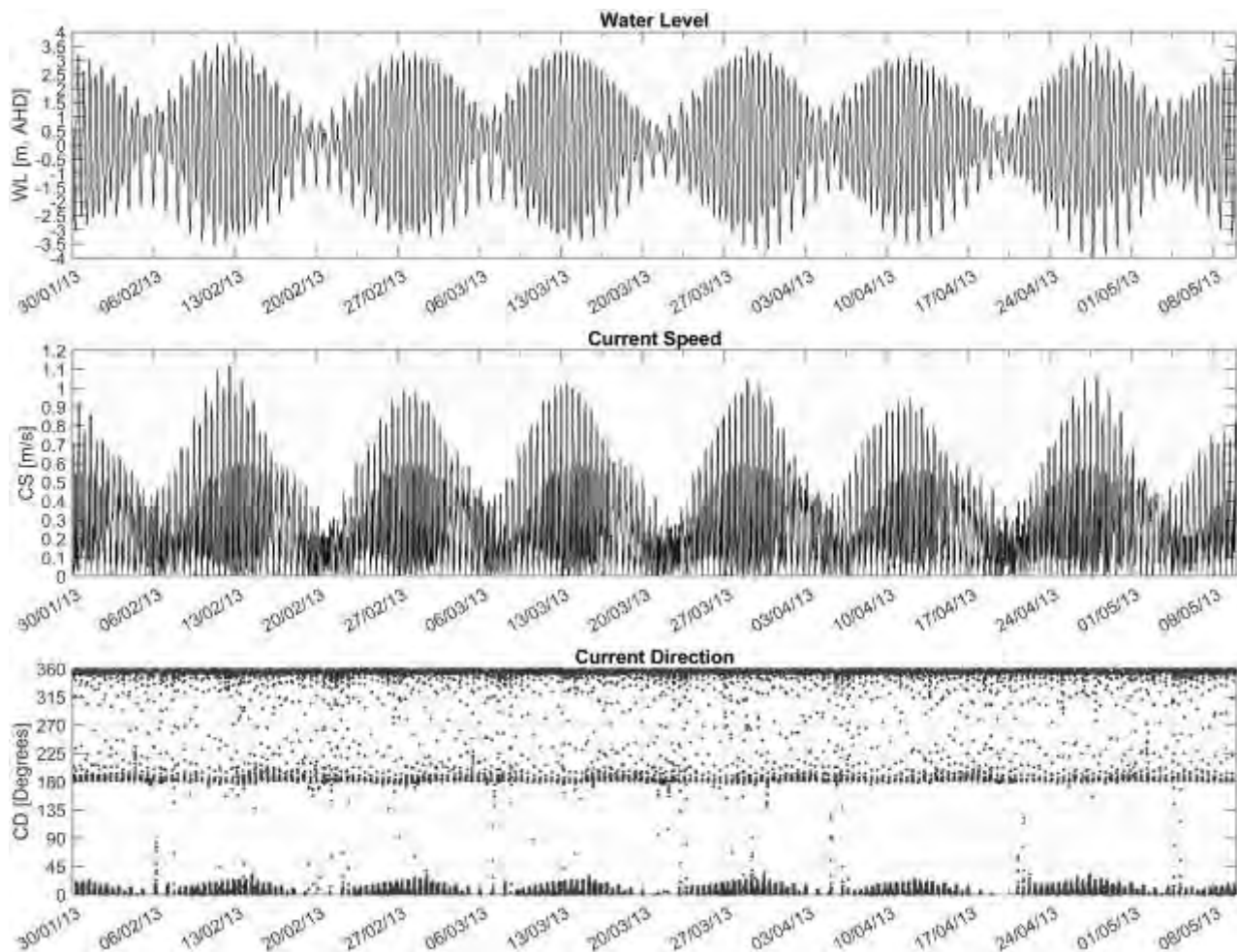


Figure 6-13 Time series of water level, current speed and current direction over the modelling period at the extraction point

6.3 Pre vs Post-Development Current Patterns

The proposed harbour design was incorporated into the hydrodynamic model to demonstrate its effect on local currents. **Figure 6-14** and **Figure 6-15** show the pre and post development magnitude and vector plot of spring tide ebb flow, respectively. **Figure 6-16** shows the pre vs post development difference plots during spring tide ebb flow. Spring tides are associated with the greatest water movement and, therefore, highest current speeds in Darwin Harbour. **Figure 6-17** and **Figure 6-18** show the pre and post development magnitude and vector plot of spring tide flood flow, respectively. **Figure 6-19** shows the pre vs post development difference plots during spring tide ebb flow. Additional vector maps and difference plots for various tidal flow conditions are presented in **Appendix B** of this report.

Offshore the site, currents are typically directed towards the north during an ebb tide and towards the south during a flood tide. Generally, ebb currents are stronger than flood currents. Ebb tidal currents at the output location are predicted to reach up to 1.1 m/s during a spring tide and flood currents approach 0.4 m/s. In the nearshore waters adjacent the proposed harbour, the flood flow current circulations are reversed, and tend to flow toward the north.

The inclusion of the proposed marina has a localised impact on the tidal currents through the area. The interaction between the harbour and net northerly current circulations creates a sheltered environment in the lee of the northern breakwater, which extends approximately 500m alongshore. Such environments are conducive of low sediment mobility, leading to the accumulation of material on the seabed. A similar, though less pronounced interaction is evident in the lee of the southern breakwater, leading to a reduction in current speeds in the nearshore waters to south of the harbour. Within the harbour, the current vectors show a reduction in peak tidal current speeds up to approximately 80%. Peak tidal currents at the harbour entrance predominantly flow toward the north directional sector and are in the order of 0.4 m/s.

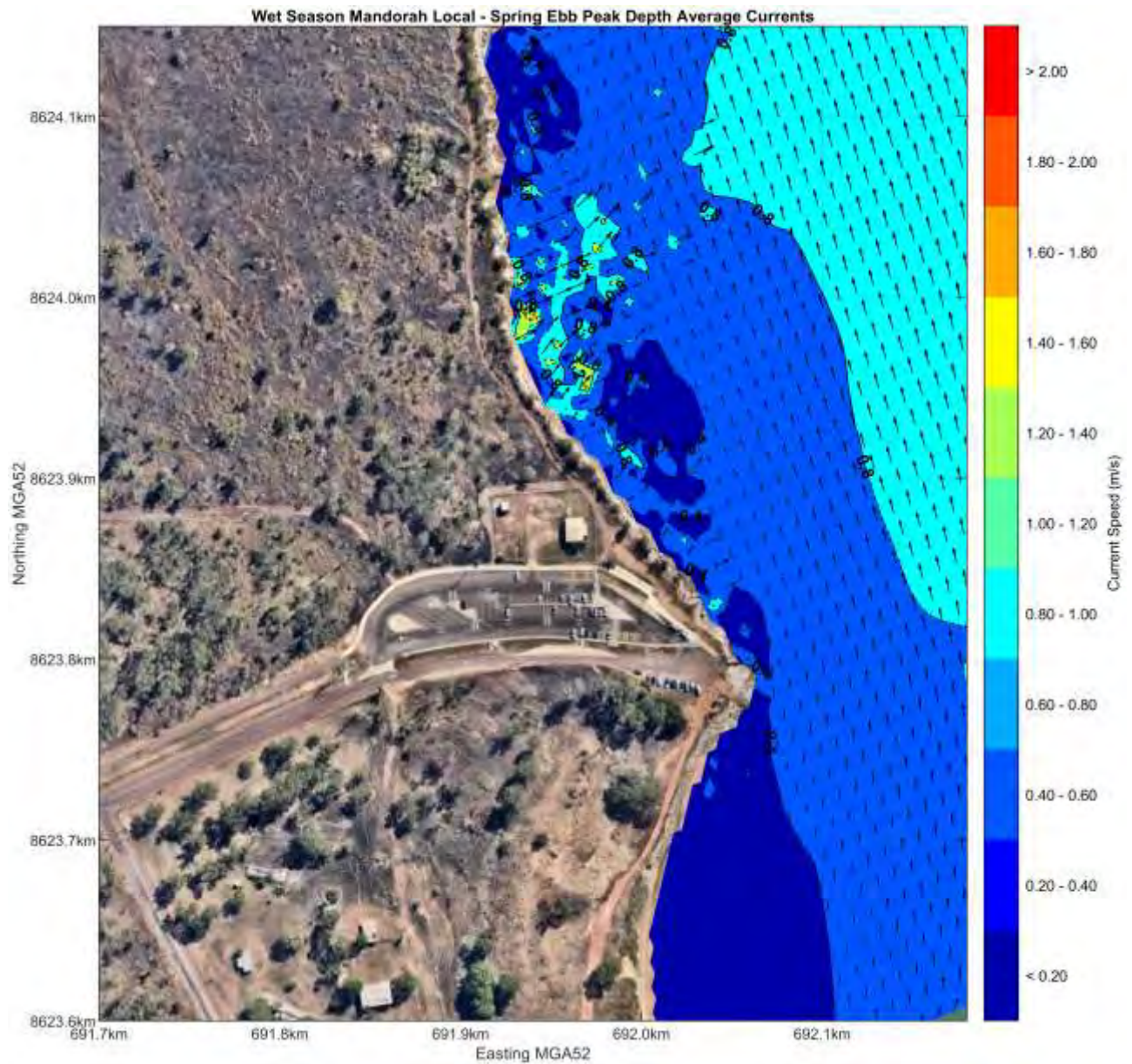


Figure 6-14 Spring tide ebb flow vector plot – Pre development

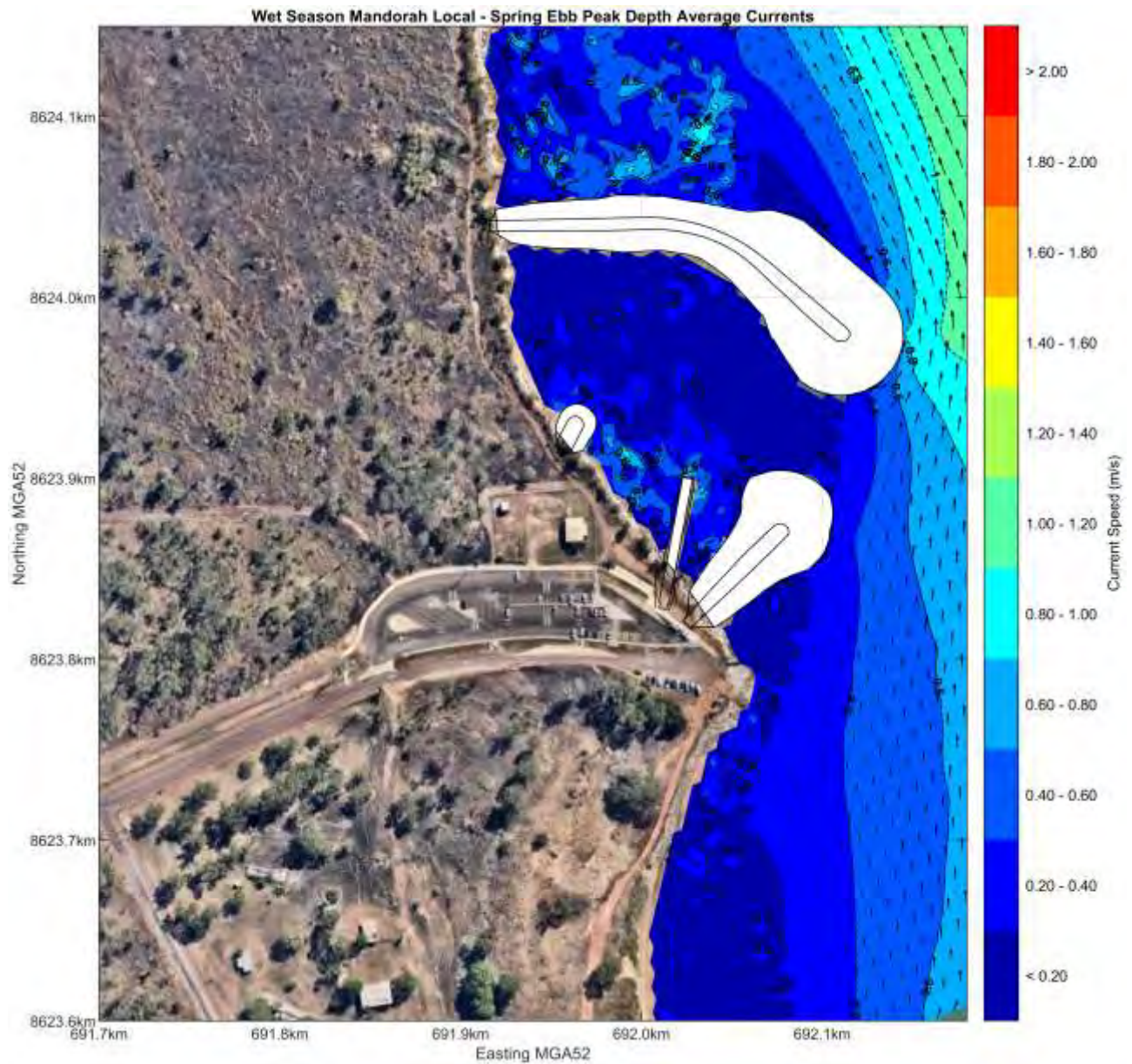


Figure 6-15 Spring tide ebb flow vector plot – Post development

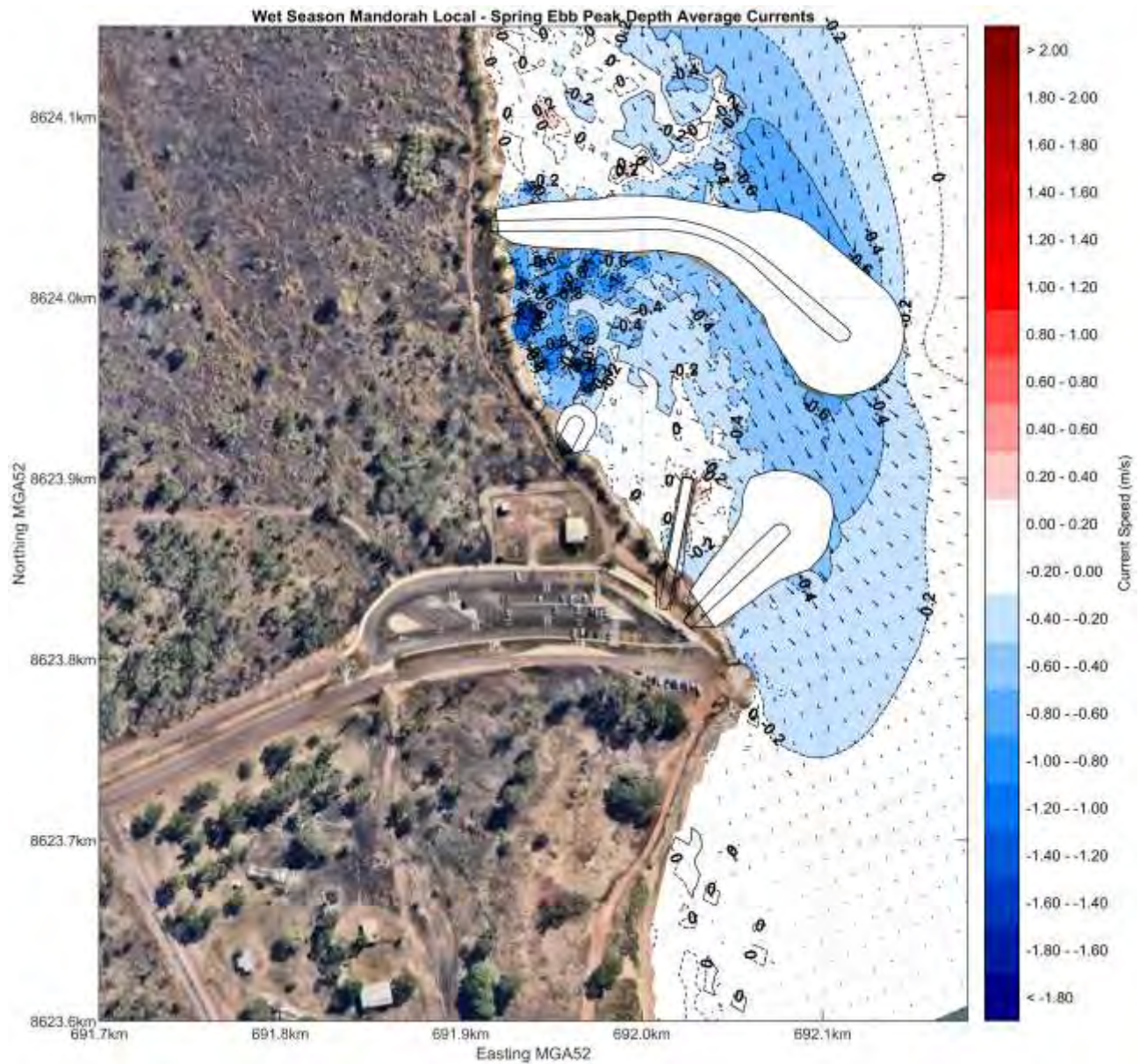


Figure 6-16 Spring tide ebb flow vector plot – Pre vs post development

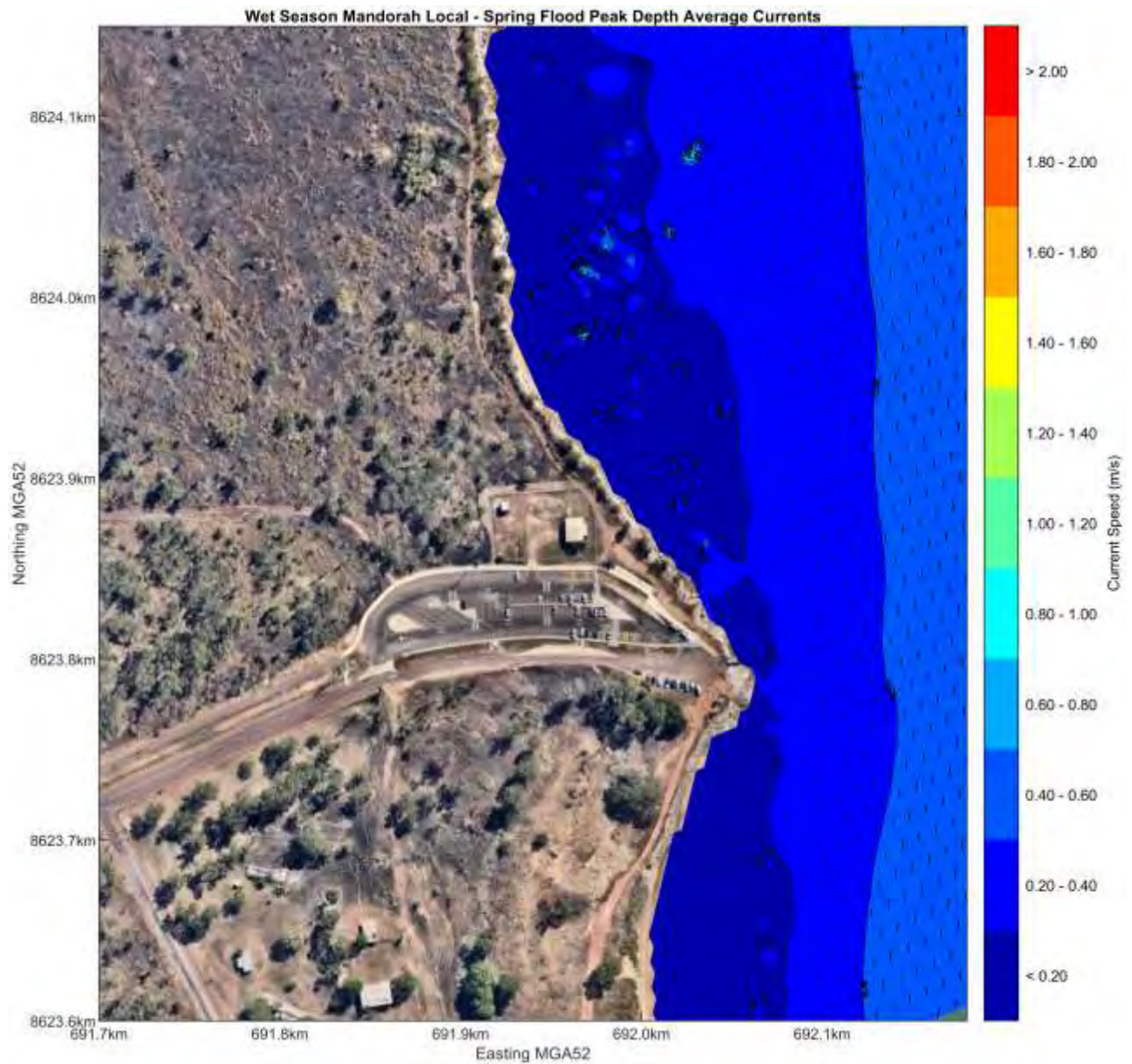


Figure 6-17 Spring tide flood flow vector plot – Pre development

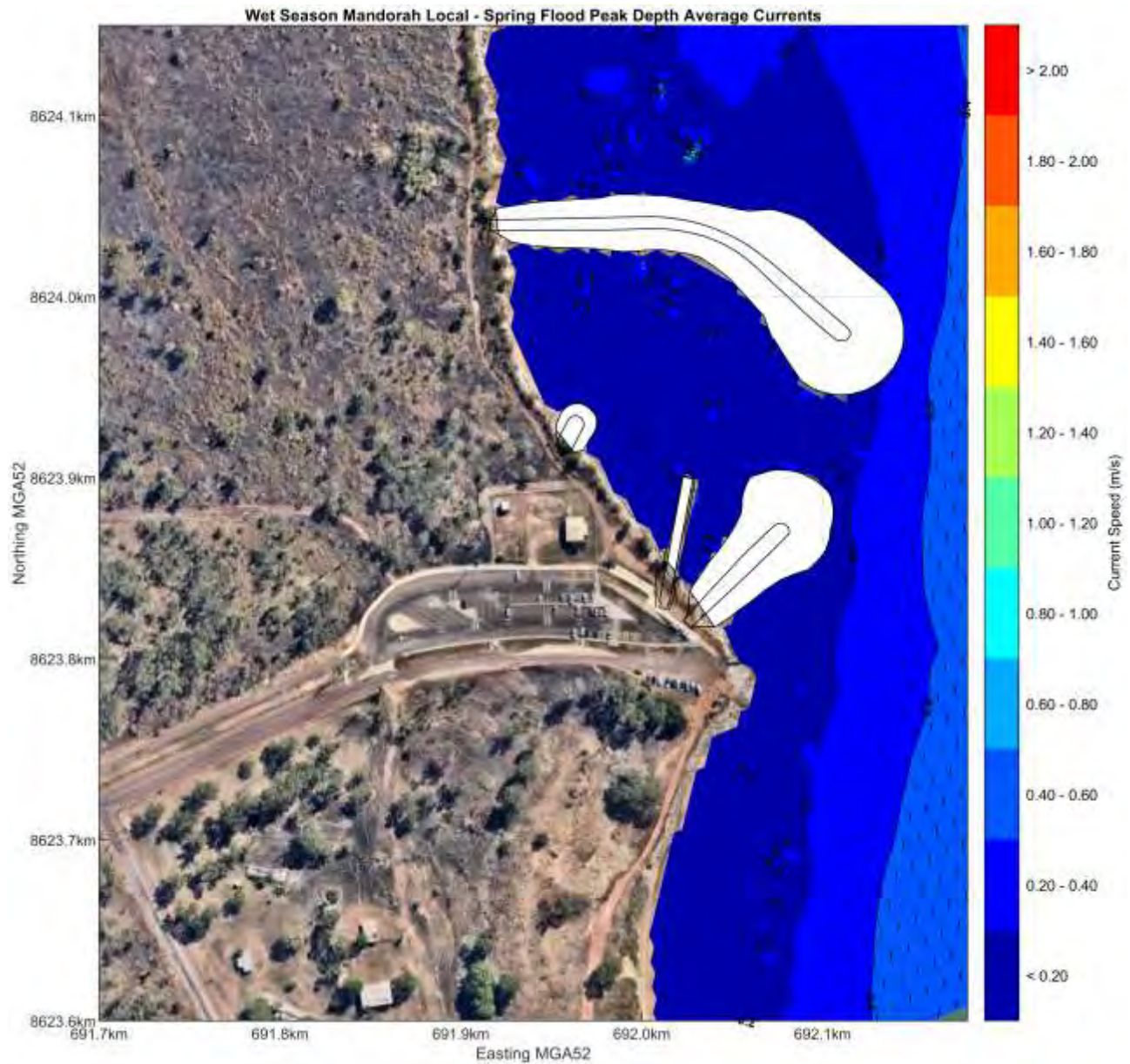


Figure 6-18 Spring tide flood flow vector plot – Post development

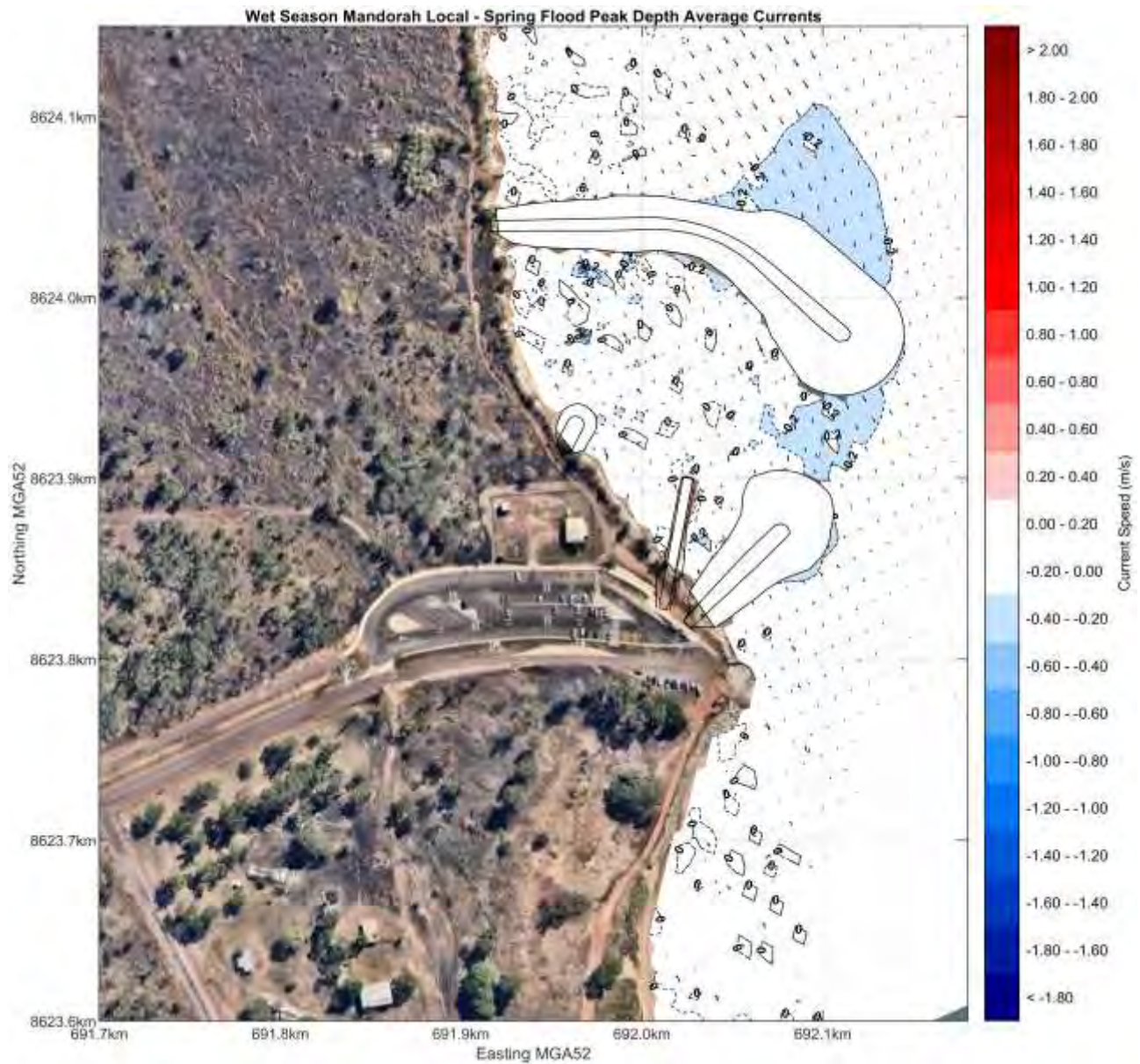


Figure 6-19 Spring tide flood flow vector plot – Pre vs post development

7 Wave Conditions

In order to determine the wave climate at the Mandorah location, it is necessary to utilise a long-term time series of wave data prepared for the site. Long-term measured wave data does not exist: therefore, Cardno has undertaken numerical wave modelling using its existing and validated Delft3D-Wave (SWAN) model of the study area. This model is used to develop a synthetic hindcast modelled time series of wave conditions within the study area. Where applicable, the model was updated with more recent bathymetric survey, and refined as required.

Wave conditions at Mandorah are governed by two simultaneously occurring sets of wave conditions:

- > Longer period, oceanic swell waves from the Beagle Gulf that penetrate into the Darwin Harbour; and
- > Shorter period, wind waves generated locally by winds blowing across Darwin Harbour.

Modelling, therefore, requires a wind wave hindcast as well as a swell wave hindcast.

7.1 SWAN Wave Modelling System

The wave model Cardno applied in this study is based on the third-generation wind/wave modelling system, SWAN, which is incorporated as a module into the Delft3D modelling system. This model was developed at the Delft Technical University and includes wind input (local sea and developing storm cases), combined sea and swell, offshore wave parameters, refraction, shoaling, non-linear wave-wave interaction, a full directional spectral description of wave propagation, bed friction, white capping, currents and wave breaking. SWAN also includes phase-averaged diffraction based on the model of Holthuijsen et al (2007).

SWAN includes a nested grid capability that allows coarser grids in deeper water and finer grids in shallow water where better definition of seabed form and depth variation are needed, and may use the same or different grids from those used in the hydrodynamic modelling. Output from the model includes significant wave height, dominant wave direction, spectral peak and mean periods and (optionally) the full directional wave spectra. It can be coupled with Delft3D to include the inter-action between waves and tidal/wind driven flows. For cyclones the non-steady-state version is applied so that the spatially and temporally varying cyclonic wind-fields are included in wave development and propagation.

7.2 Operational Wave Climate Study

A local wind wave hindcast was undertaken to assess the operational (non-cyclonic) wave climate at the study site. The purpose of this assessment was to determine the local wave conditions at facility on both a seasonal and an annual basis. Both swell and wind waves were modelled using a 'round the clock' approach. This entails using the numerical model to undertake simulations and to determine wave transfer coefficients for a full suite of local wind speeds, directions and water levels. The modelling included:

- > Wind speeds ranging from 4 m/s to 32 m/s - at 4 m/s intervals;
- > Wind directions from north, clockwise through to south back to north (around the clock) - at 11.25 degrees intervals;
- > Different tidal levels from LAT to HAT - at 0.5 m intervals.

The SWAN wave model comprised of four nested grids, covering the full range of available fetches at Mandorah. The wave model grid system is shown in **Figure 7-1** and **Figure 7-2**, which consists of a coarse Beagle Gulf grid with a resolution of 1,200m. Nested within the Beagle Gulf grid include a 270m resolution Darwin Harbour grid, and two local grids in the vicinity of Mandorah, with resolutions of 100m and 20m. The operational wave climate study utilised the three finer grids and CSIRO extraction point 1 in **Figure 7-3**. The Beagle Gulf grid and CSIRO extraction point 2 were utilised to assess changes in the local sediment transport pathways, attributed to the construction of the marina (refer Cardno, 2023).

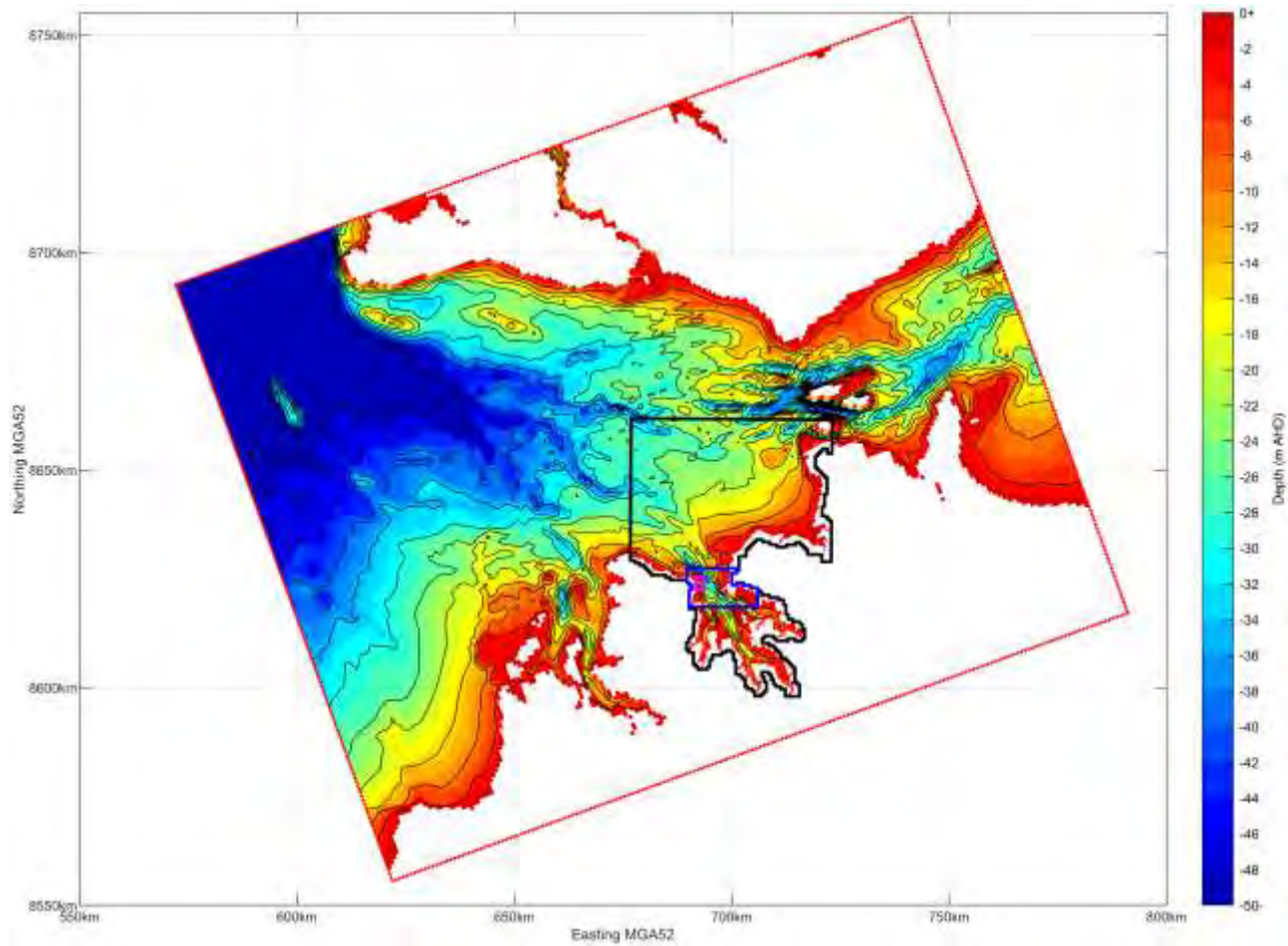


Figure 7-1 SWAN model grids: Beagle Gulf (red), Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Beagle Gulf bathymetry

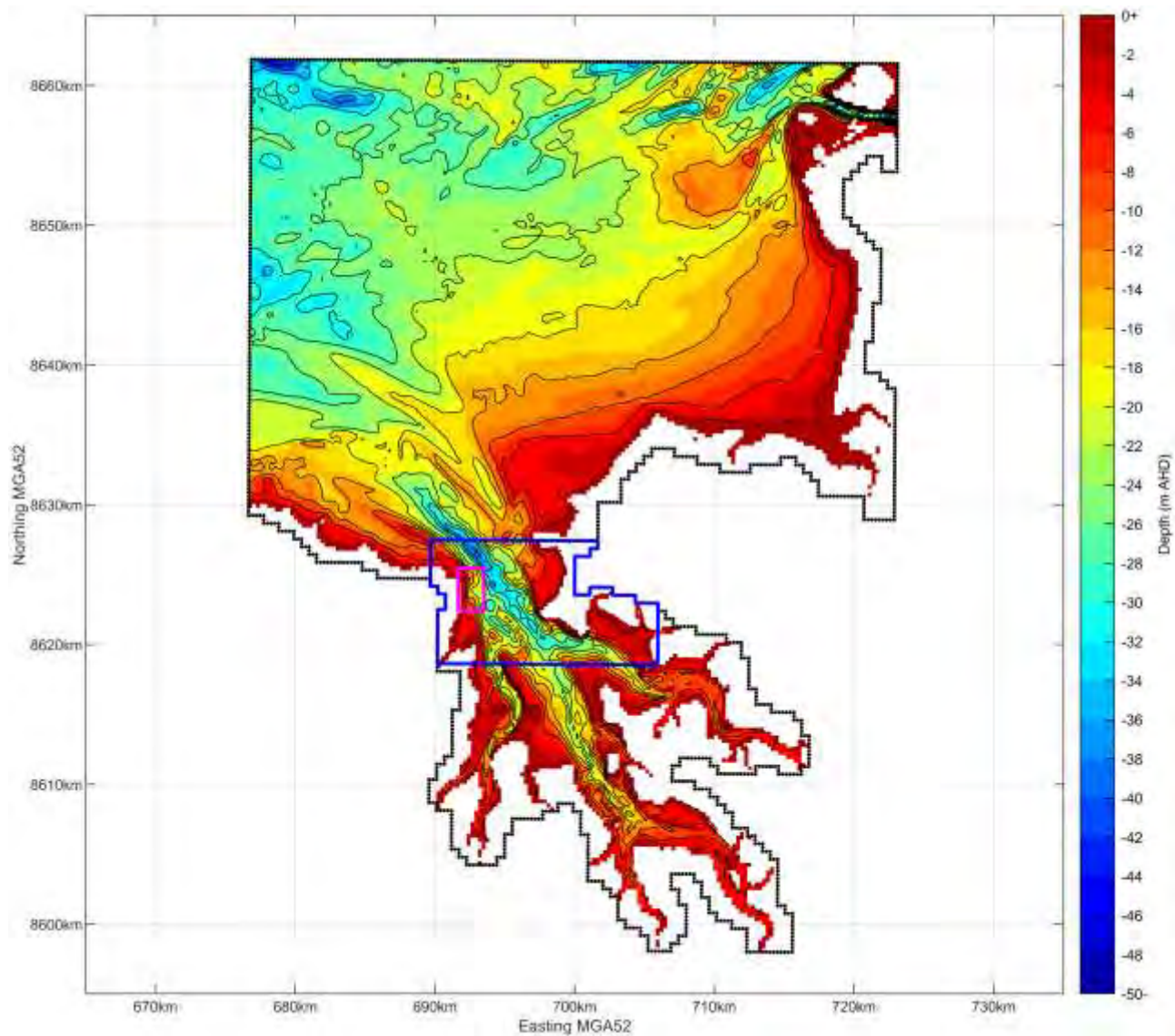


Figure 7-2 SWAN model grids: Darwin Harbour (black), Mandorah (blue) & Mandorah Local (magenta). Colour contours showing Darwin Harbour bathymetry



Figure 7-3 Wind and data extraction points from Darwin airport and CSIRO wave model

7.2.2 Model Skill Assessment

Model Limitations and Accuracy

Limitations of the modelling undertaken for this study are due to the main inputs into the models such as the wind and wave boundaries obtained from the global wave model and the Darwin Airport Met-Station. Spatially constant (but variable in time) wind fields have been applied over the domain which does not take into account any local orographic effects.

To gain confidence that the model is describing the wave conditions at the site well, a calibration and verification exercise was undertaken and is presented below.

Model Calibration and Validation

The wave model results have been compared to measured significant wave heights and directions at the four sites within Darwin Harbour, listed in **Table 7-1**, during the period of August 2013 to March 2015, as well the ADCP site data collected between December 2019 and April 2020.

Table 7-1 Measurements in Darwin Harbour used for model validation

Site	East	North	From	To
Mandorah ADCP (2020)	692214	8624001	12/12/2019	22/4/2020
Mandorah RBR	692415	8624508	17/8/2012	15/2/2015
Fannie Bay 1 RBR (FAN)	697324	8624997	09/8/2012	15/2/2015
Wickham Point RBR (WIC)	703687	8617551	08/8/2012	15/2/2015
Weed Reef ADCP (WEED REEF)	696147	8618483	23/11/2012	20/3/2013

The comparisons of modelled and measured wave parameters are presented below in **Figure 7-4** and **Figure 7-5**. The comparison shows that the wave model achieves good agreement with the measurements at all of the measurement locations.

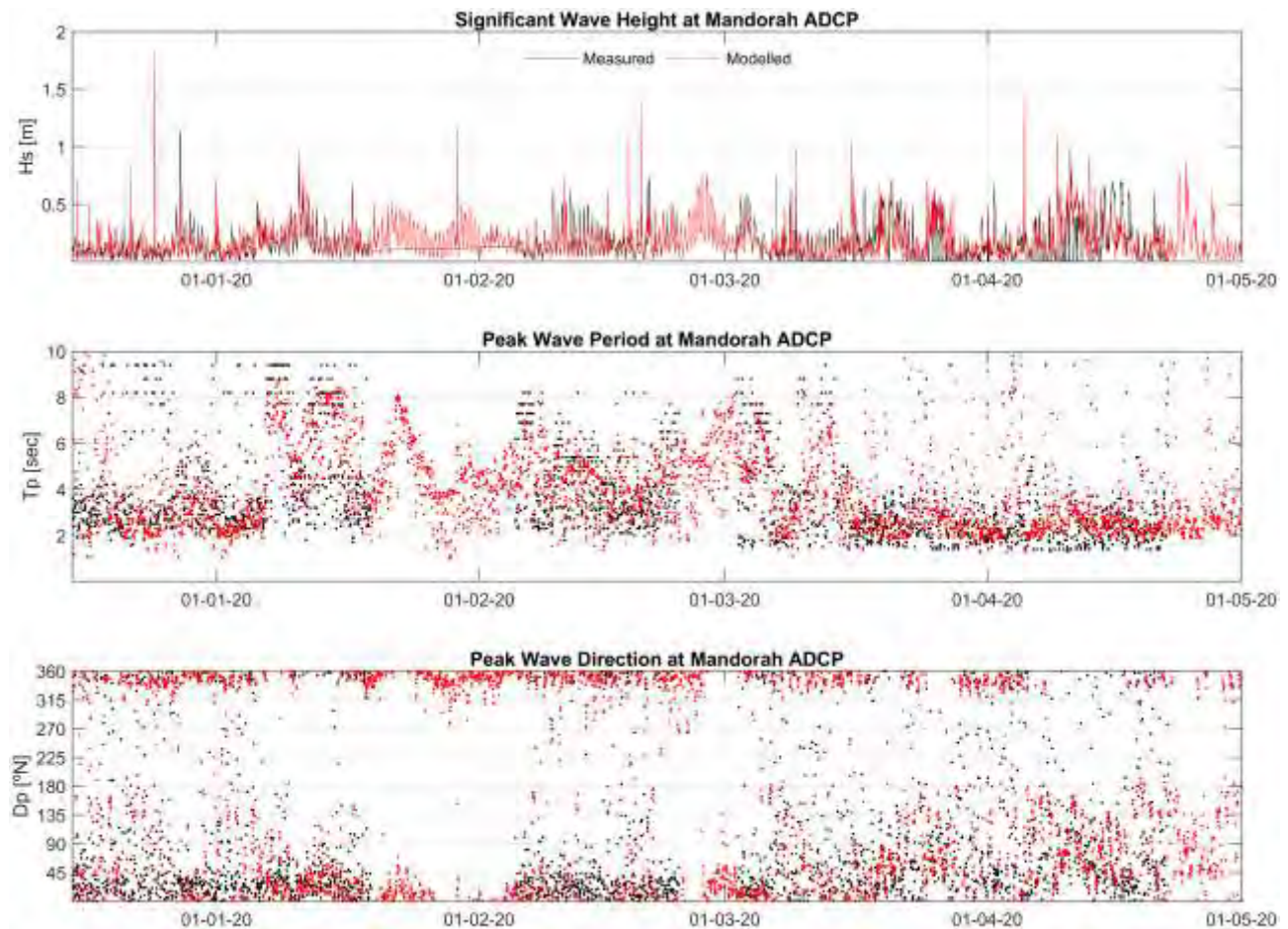


Figure 7-4 Comparison of measured and modelled wave parameters at the Mandorah ADCP (2020)

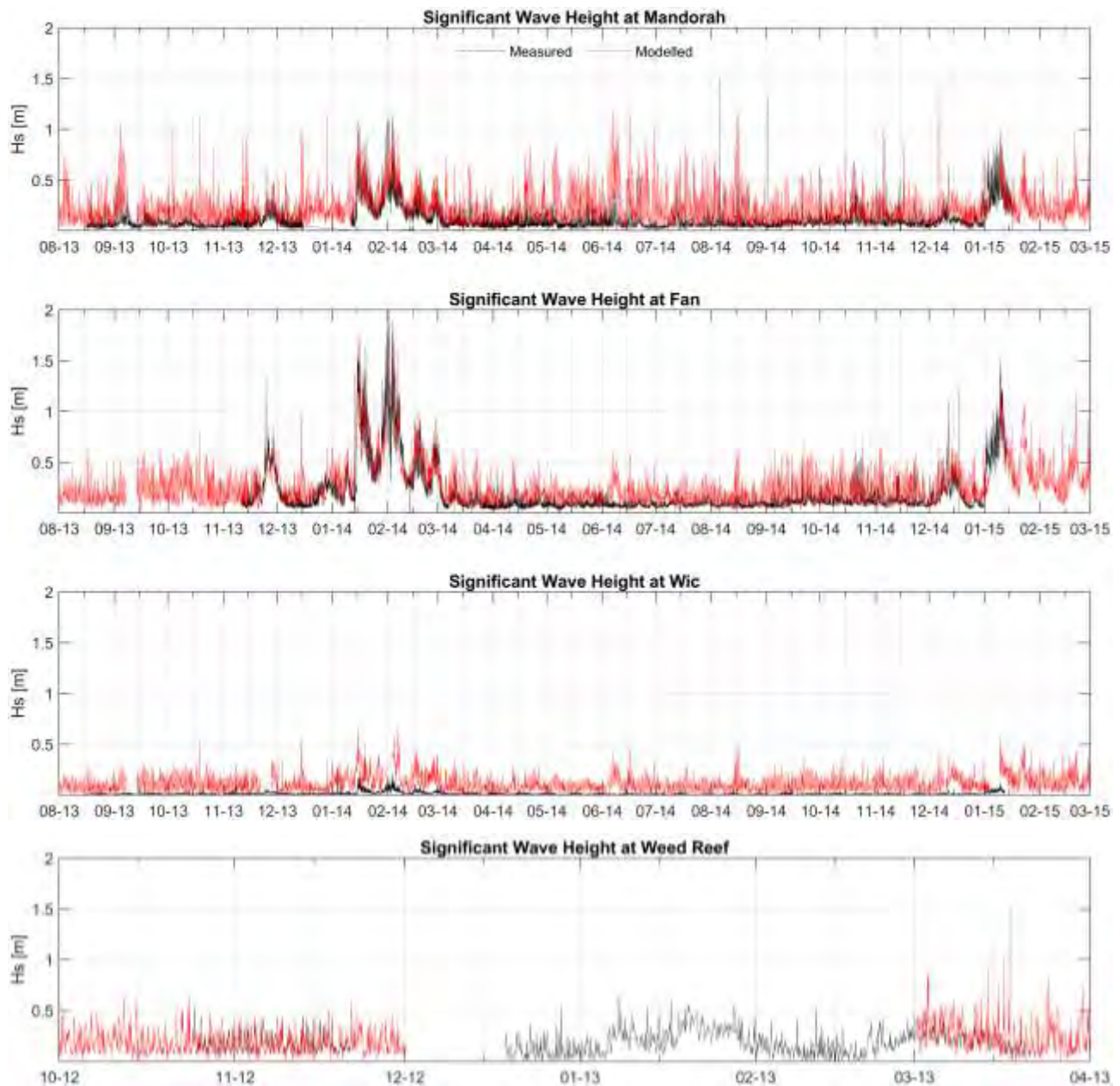


Figure 7-5 Comparison of measured and modelled significant wave height at the 2013-2015 RBR measurement locations

7.2.3 Offshore Wave Data

Offshore wave data was obtained from the CSIRO's (Commonwealth Scientific and Industrial Research Organisation) wave model at the Beagle Gulf (130.2°E, -12.2°S). The 40-year hindcast data covers the period from 1979 to 2019 at 1-hourly intervals. The location of the wave data extraction point use for the operational wave climate study is shown in **Figure 7-3** (CSIRO Extraction Point 2). The annual and seasonal directional distribution of significant wave heights at the offshore location are presented in **Figure 7-6** and **Figure 7-7**.

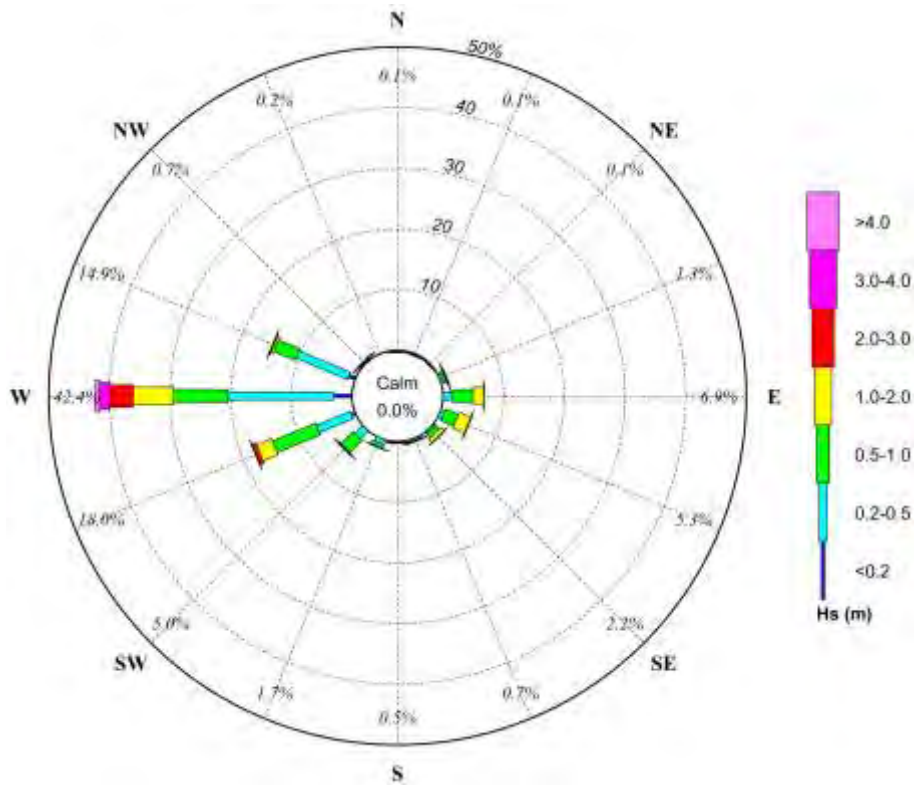


Figure 7-6 Annual offshore wave rose based on CSIRO hindcast wave data at CSIRO Extraction Point 2 (1979 to 2019)

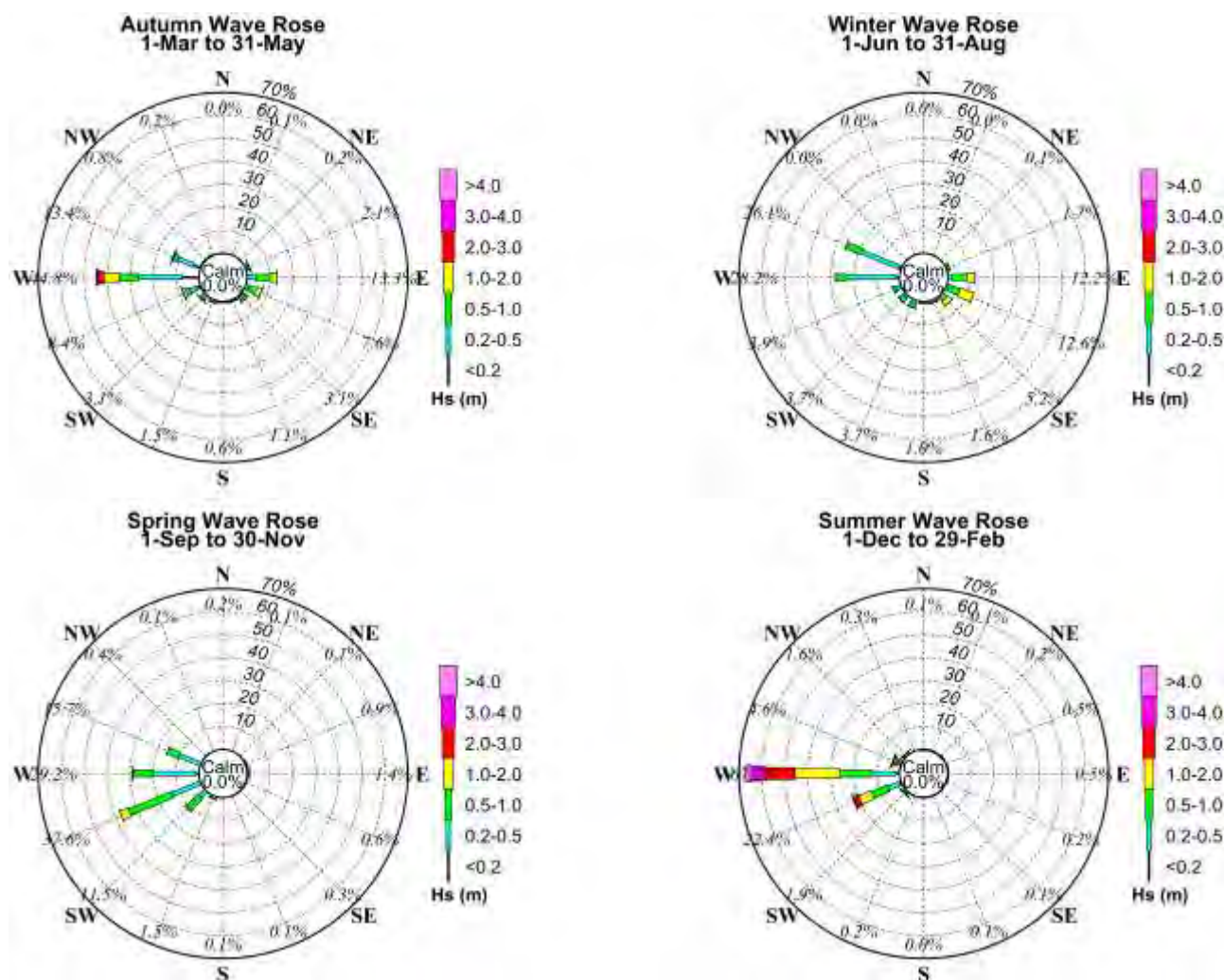


Figure 7-7 Seasonal offshore wave roses based on CSIRO hindcast wave data at CSIRO Extraction Point 2 (1979 to 2019)

7.2.4 Nearshore Wave Data

The output from the operational wave climate study was used to generate a timeseries of total, wind sea and swell wave data at the project site. The location of wave data extraction point at the project site is shown in **Figure 6-11**. **Figure 7-8** presents the total, wind and swell wave climate at the site in terms of an annual wave roses.

As per the modelling results, swell waves typically approach the site from the north to north-east, however wind sea waves approach the site from different directions, mainly from north-north-west to south-east. The seasonal distribution of swell waves is almost the same throughout year, but the wind sea waves typically approach the site from the north-north-east to south-east during autumn and winter, and from the north-north-west to north-north-east during spring and summer.

Moreover, the results of the wave modelling indicate that swell waves at the site are typically small, and significant wave heights of the order of 5cm or less occurs for nearly 69% of the time. Swell significant wave height is predicted to not exceed 1.0m. Only around 3.5% of the time the swell significant wave height is higher than 0.25m. Conversely, the wind wave climate is much more energetic. Wind (sea) waves are predicted to approach 2.0m (significant height) in very rare events, and for over 55% of the time the sea significant wave height is higher than 0.25m.

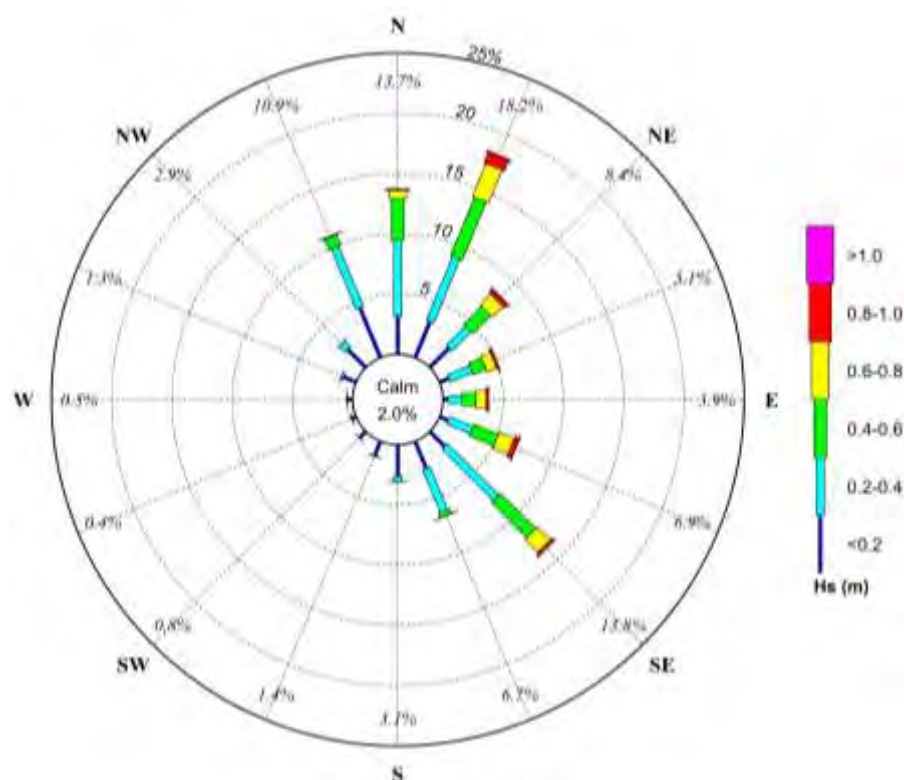


Figure 7-8 Annual total wave rose at the extraction point (1985 to 2019)

7.3 Pre vs Post-Development Wave Conditions

The proposed harbour design was incorporated into the SWAN wave model to demonstrate its effect on local wave conditions. Representative wave conditions during spring/ summer and autumn/ winter have been extracted from the operational wave climate study. **Figure 7-9** and **Figure 7-10** show the pre and post development significant wave magnitude and vector plots during north-north-westerly conditions, respectively. **Figure 7-11** shows the pre vs post development difference plots during north-north-westerly conditions. **Figure 7-12** and **Figure 7-13** show the pre and post development significant wave magnitude and vector plots during northerly conditions, respectively. **Figure 7-14** shows the pre vs post development difference plots during northerly conditions. Waves approaching the site from these directional sectors is representative of the prevailing wave conditions typically experienced during spring and summer at the site.

Figure 7-15 and **Figure 7-16** show the pre and post development significant wave magnitude and vector plots during north-north-easterly conditions, respectively. **Figure 7-17** shows the pre vs post development difference plots during north-north-easterly conditions. **Figure 7-18** and **Figure 7-19** show the pre and post development significant wave magnitude and vector plots during south-easterly conditions, respectively. **Figure 7-20** shows the pre vs post development difference plots during south-easterly conditions. Waves approaching the site from these directional sectors is representative of the prevailing wave conditions typically experienced during autumn and winter at the site.

The inclusion of the proposed harbour has a relatively small and localised impact on the wave climate through the area. Localised changes are largely isolated to the areas adjacent the northern and southern breakwaters, due the sheltering afforded by the structures. The extent of the local sheltering is most pronounced during south-easterly wave conditions, extending to the north of the harbour's footprint by approximately 250m.

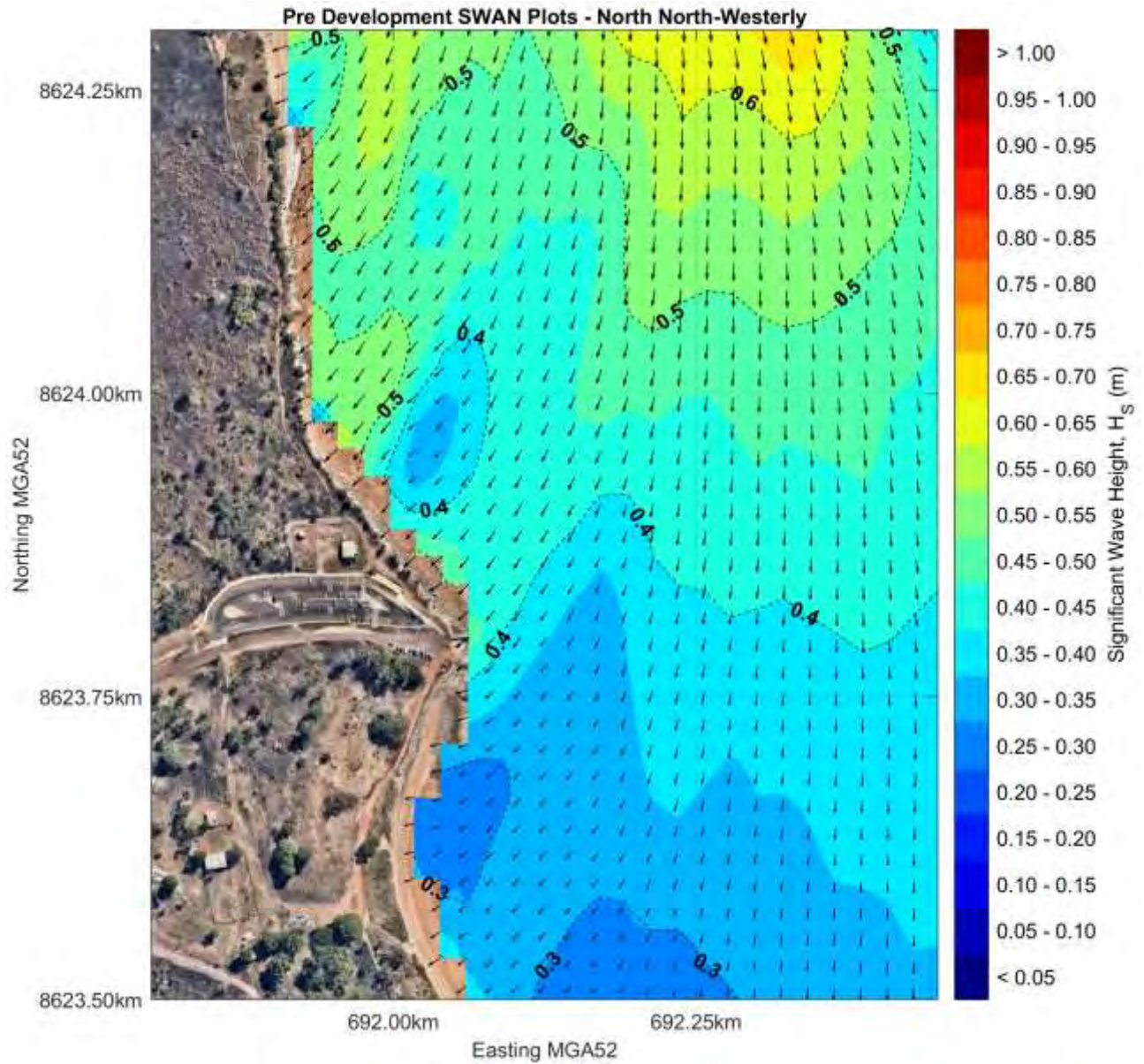


Figure 7-9 Wave vector plot during spring and summer, north-north-westerly conditions – Pre development

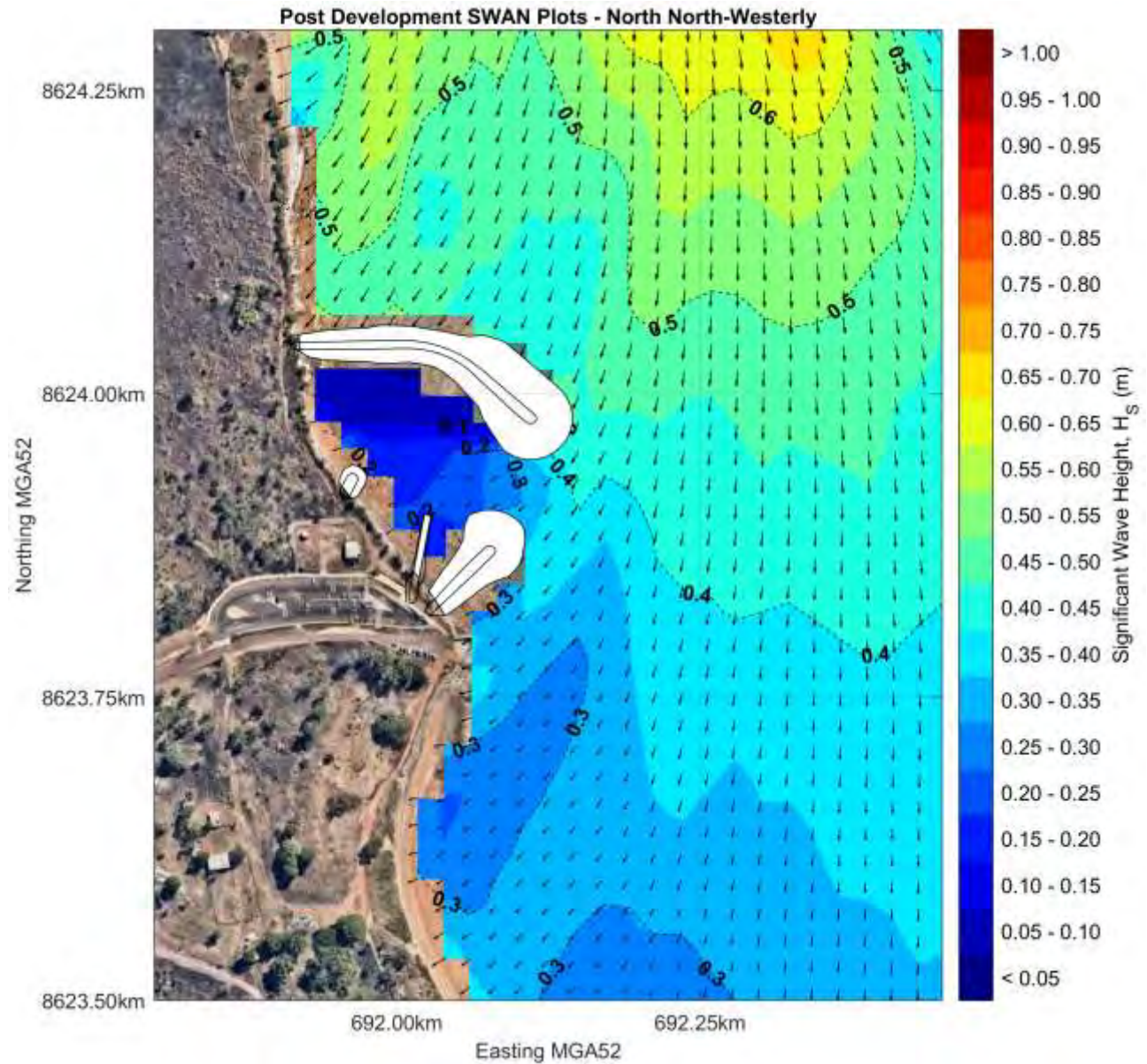


Figure 7-10 Wave vector plot during spring and summer, north-north-westerly conditions – Post development

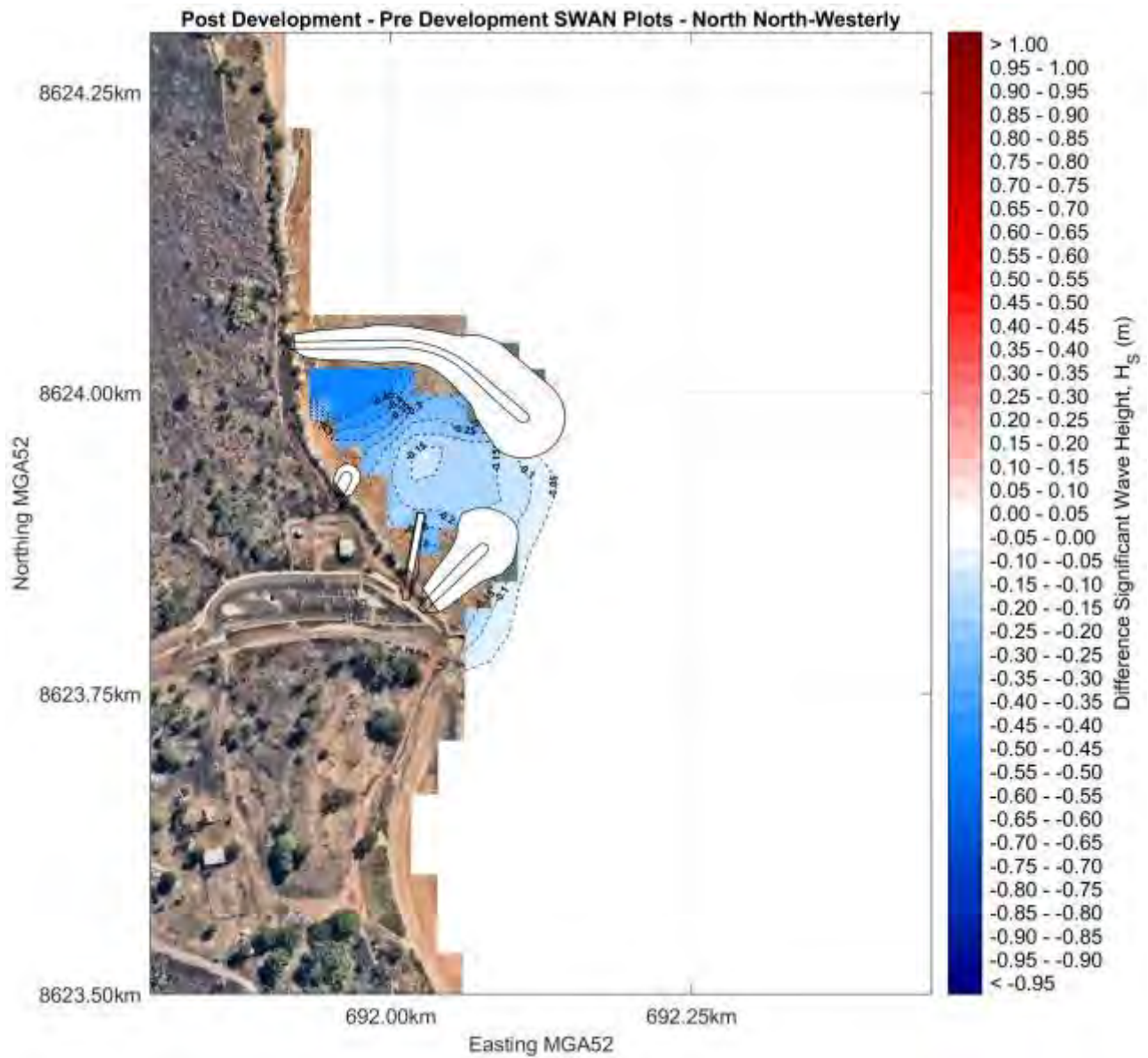


Figure 7-11 Wave vector plot during spring and summer, north-north-westerly conditions – Pre vs post development

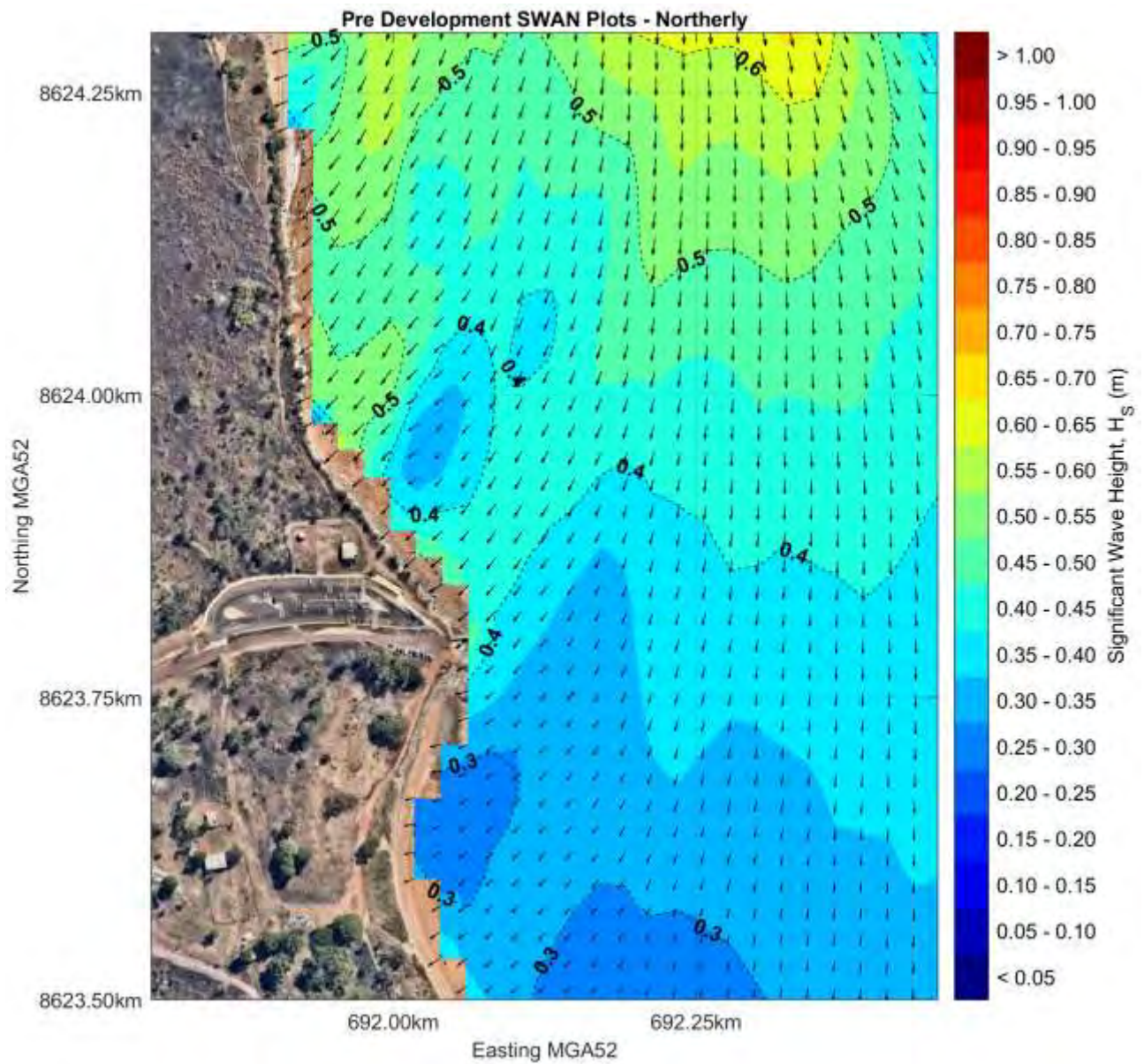


Figure 7-12 Wave vector plot during spring and summer, northerly conditions – Pre development

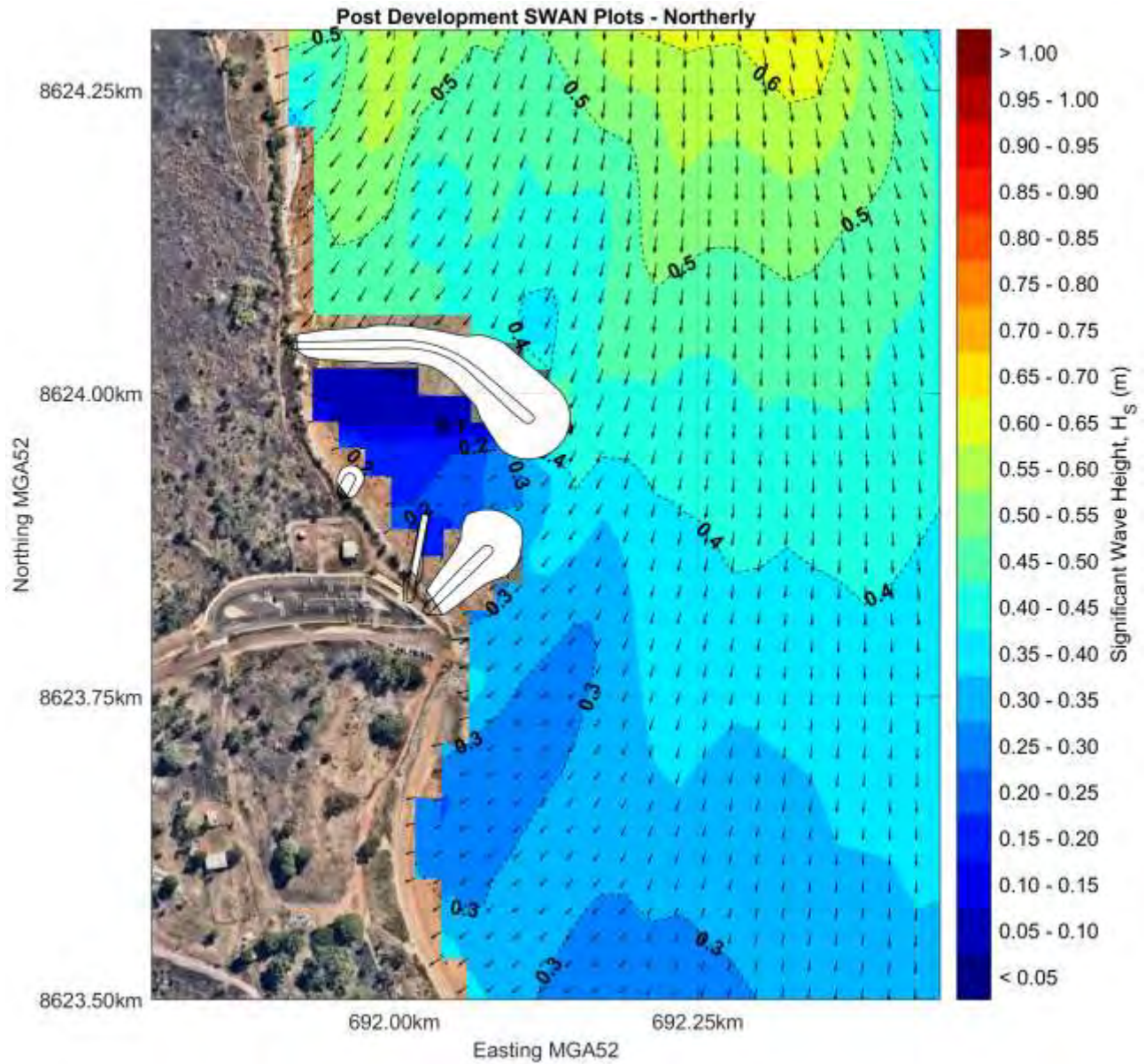


Figure 7-13 Wave vector plot during spring and summer, northerly conditions – Post development

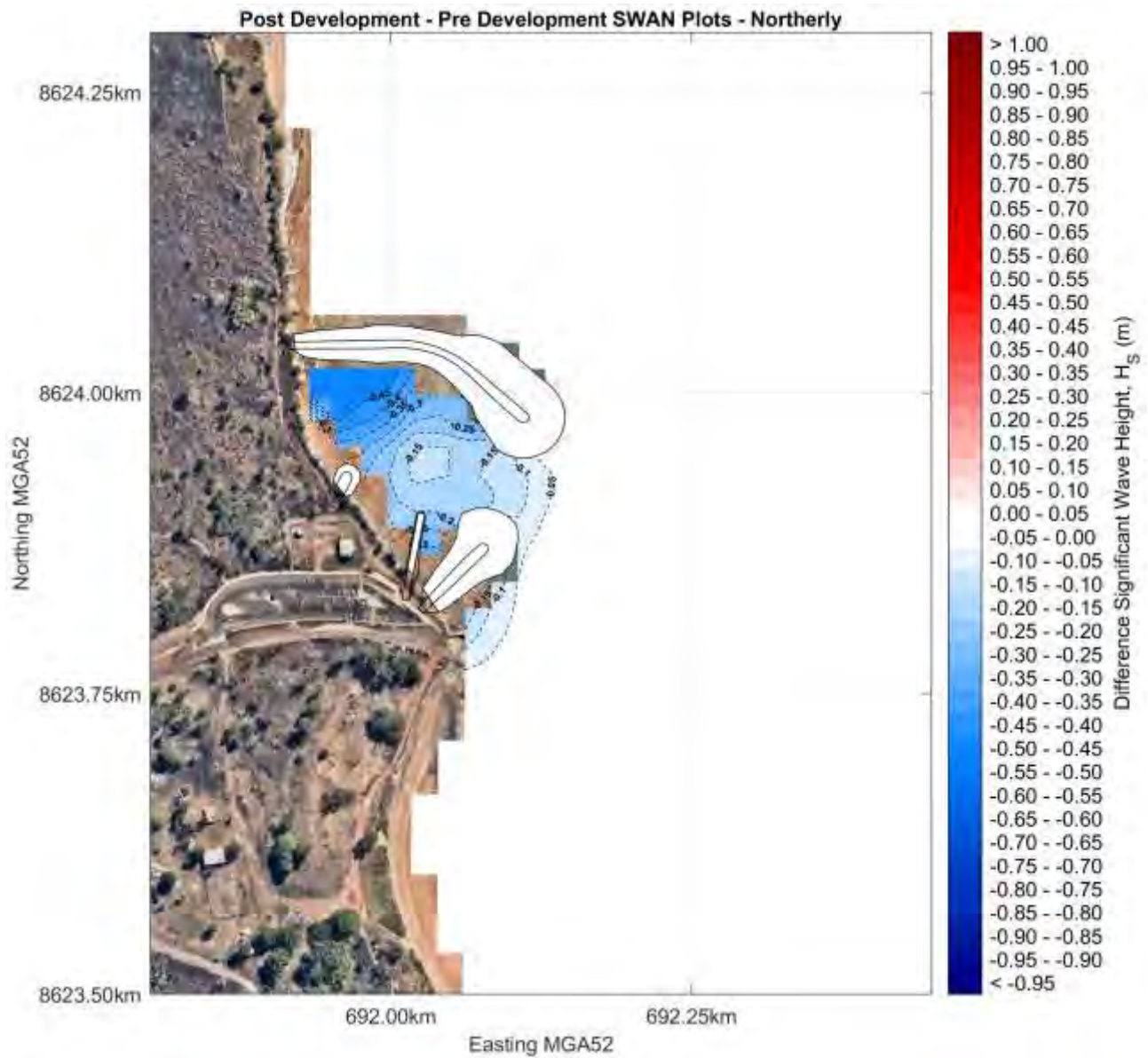


Figure 7-14 Wave vector plot during spring and summer, northerly conditions – Pre vs post development

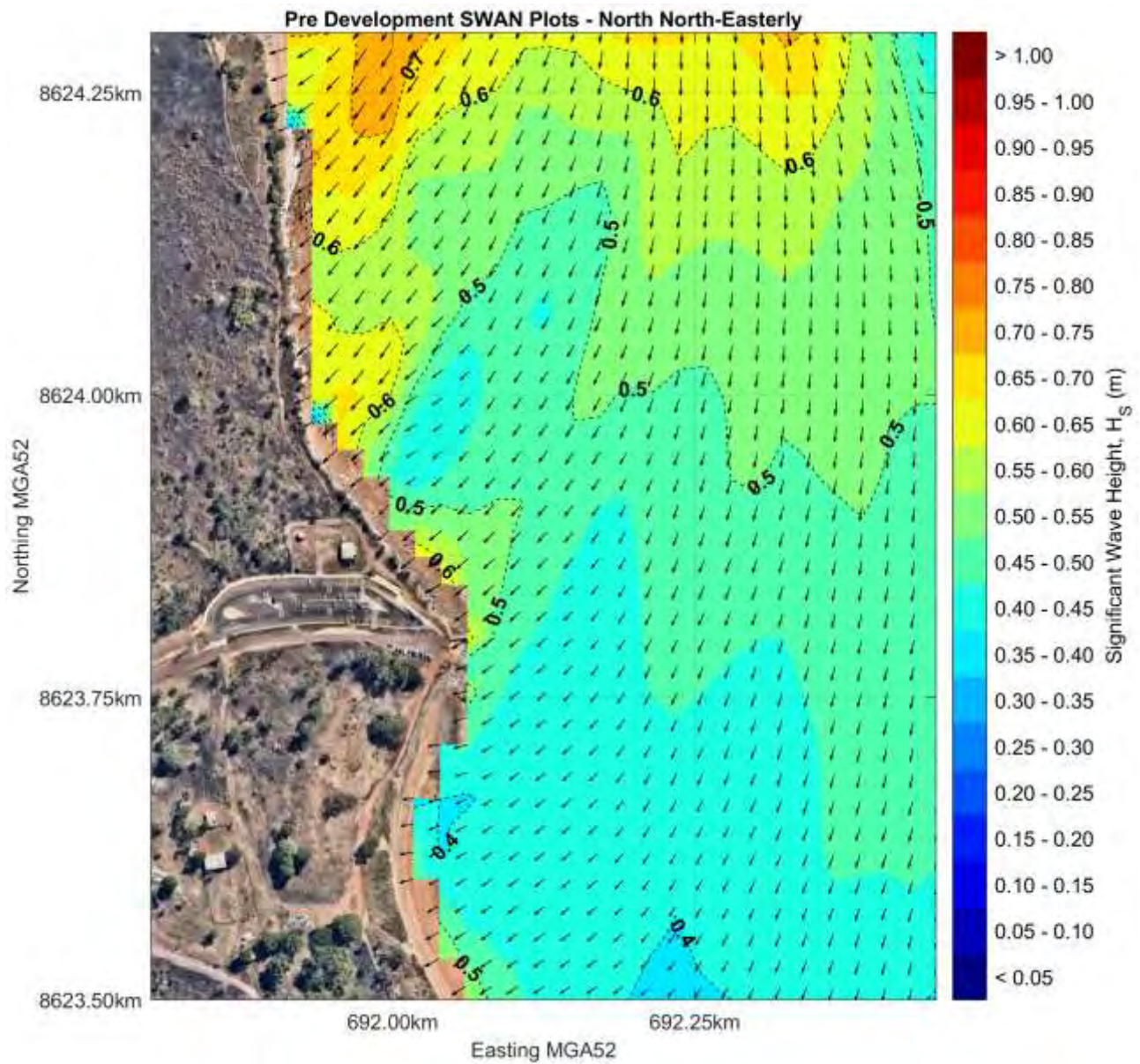


Figure 7-15 Wave vector plot during autumn and winter, north-north-easterly conditions – Pre development

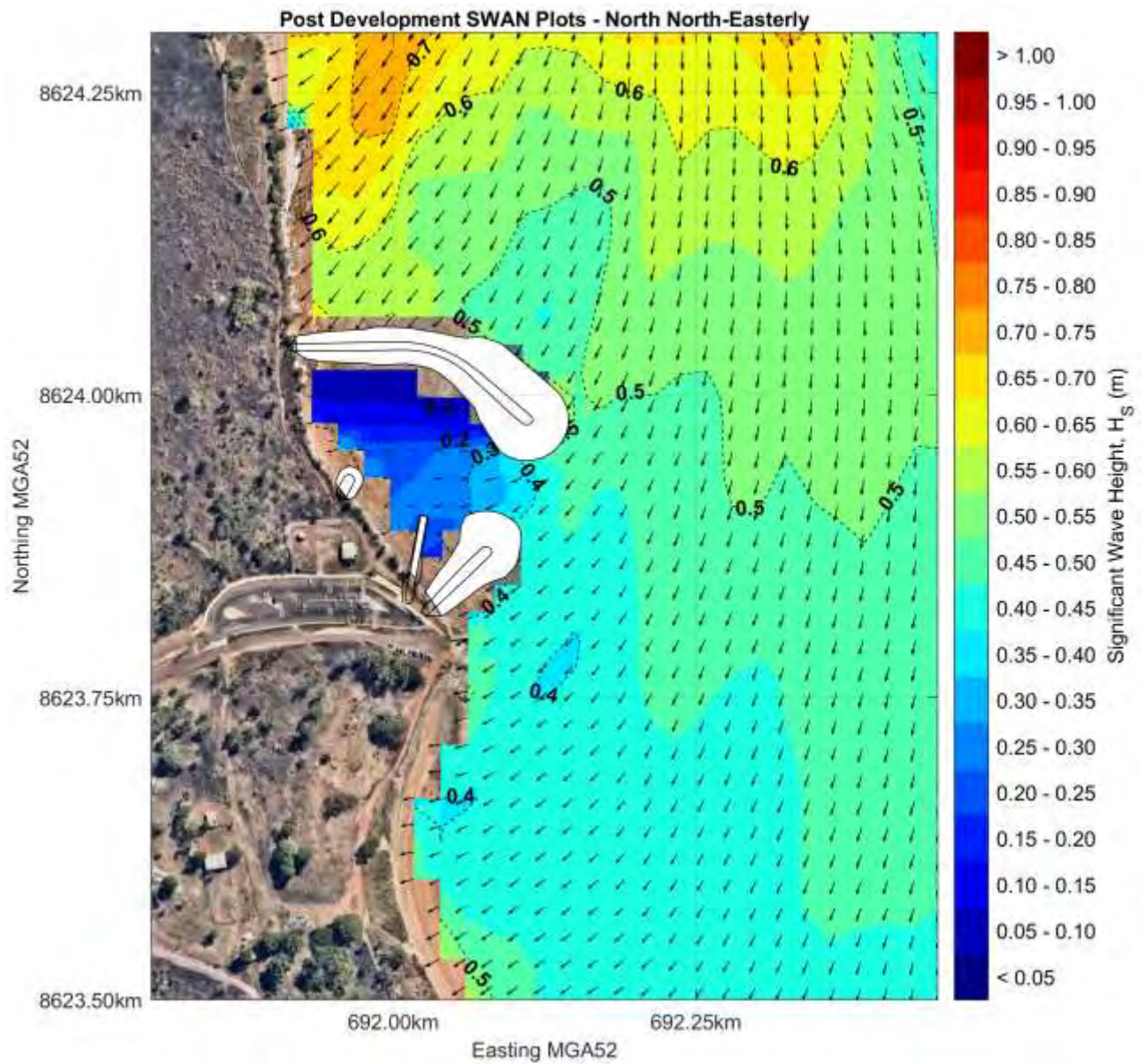


Figure 7-16 Wave vector plot during autumn and winter, north-north-easterly conditions – Post development

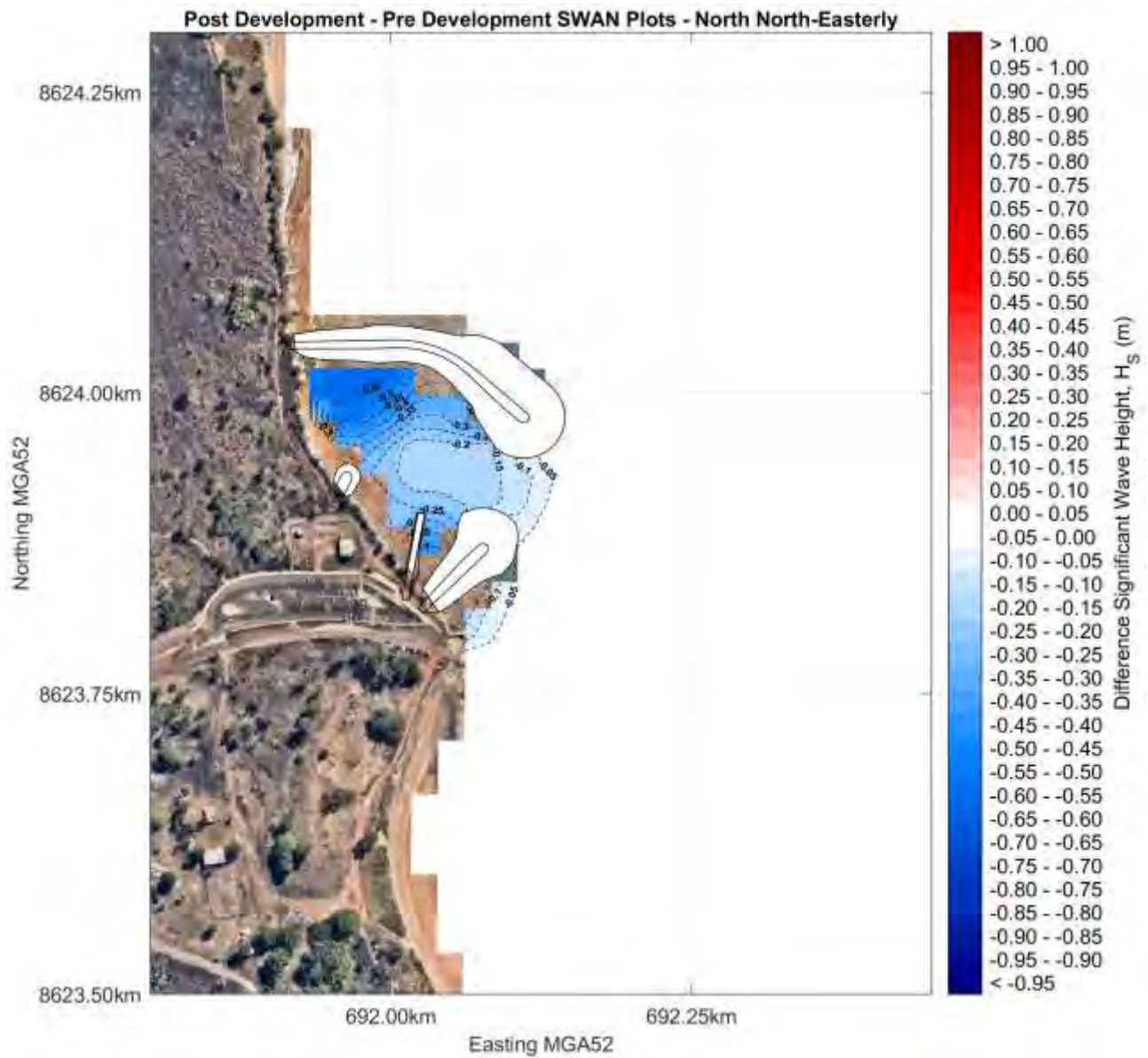


Figure 7-17 Wave vector plot during autumn and winter, north-north-easterly conditions – Pre vs post development

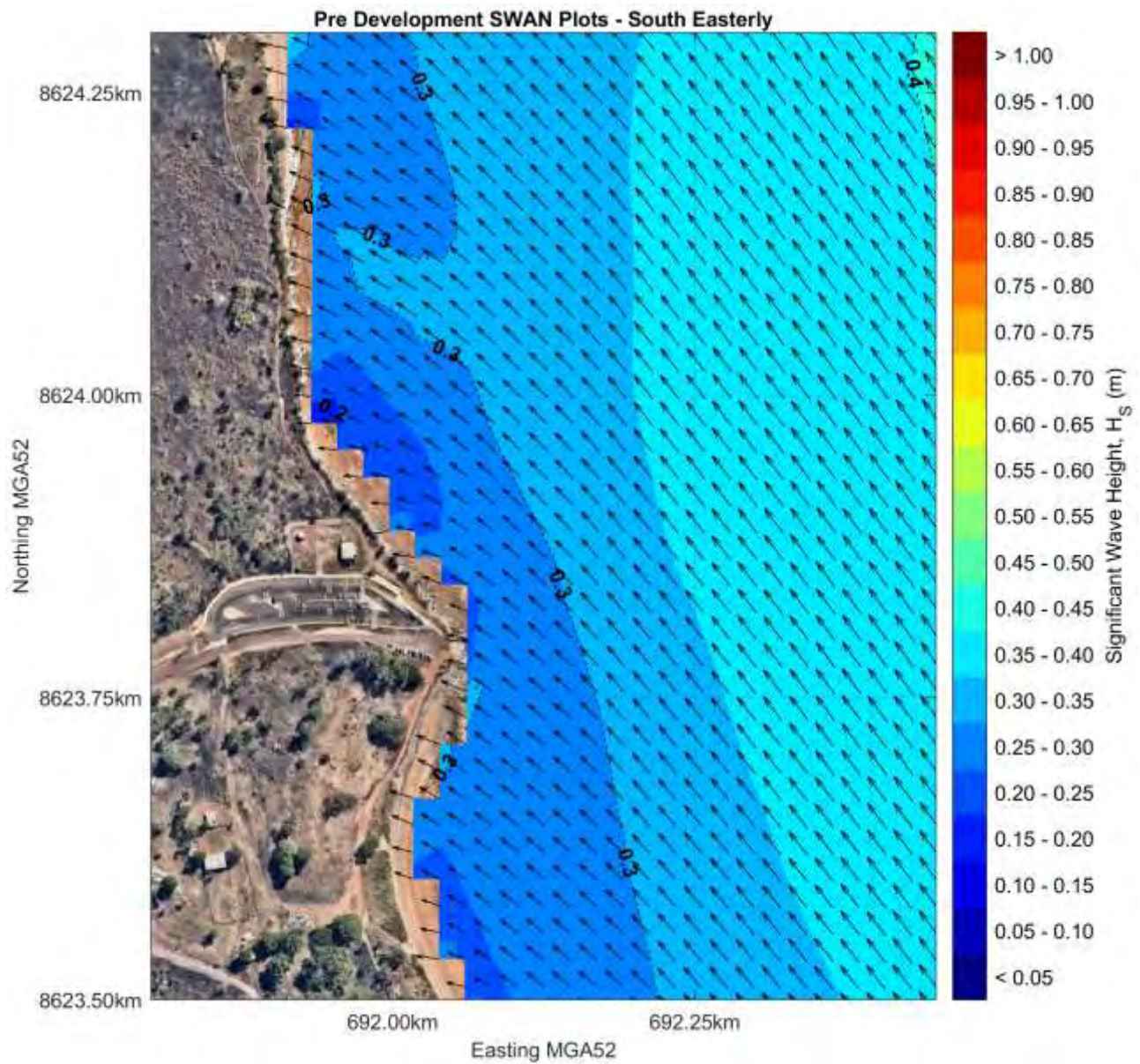


Figure 7-18 Wave vector plot during autumn and winter, south-easterly conditions – Pre development

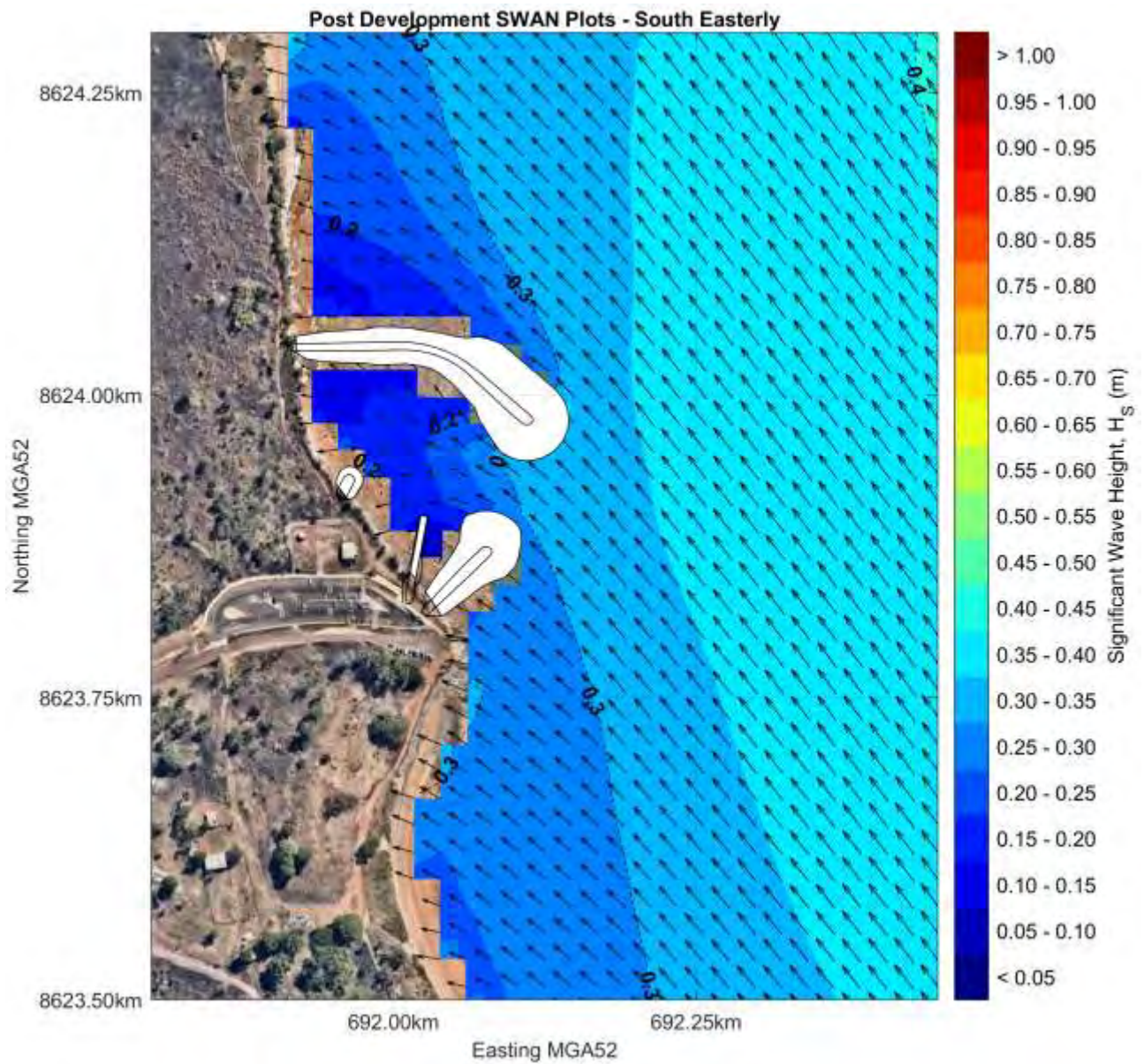


Figure 7-19 Wave vector plot during autumn and winter, south-easterly conditions – Post development

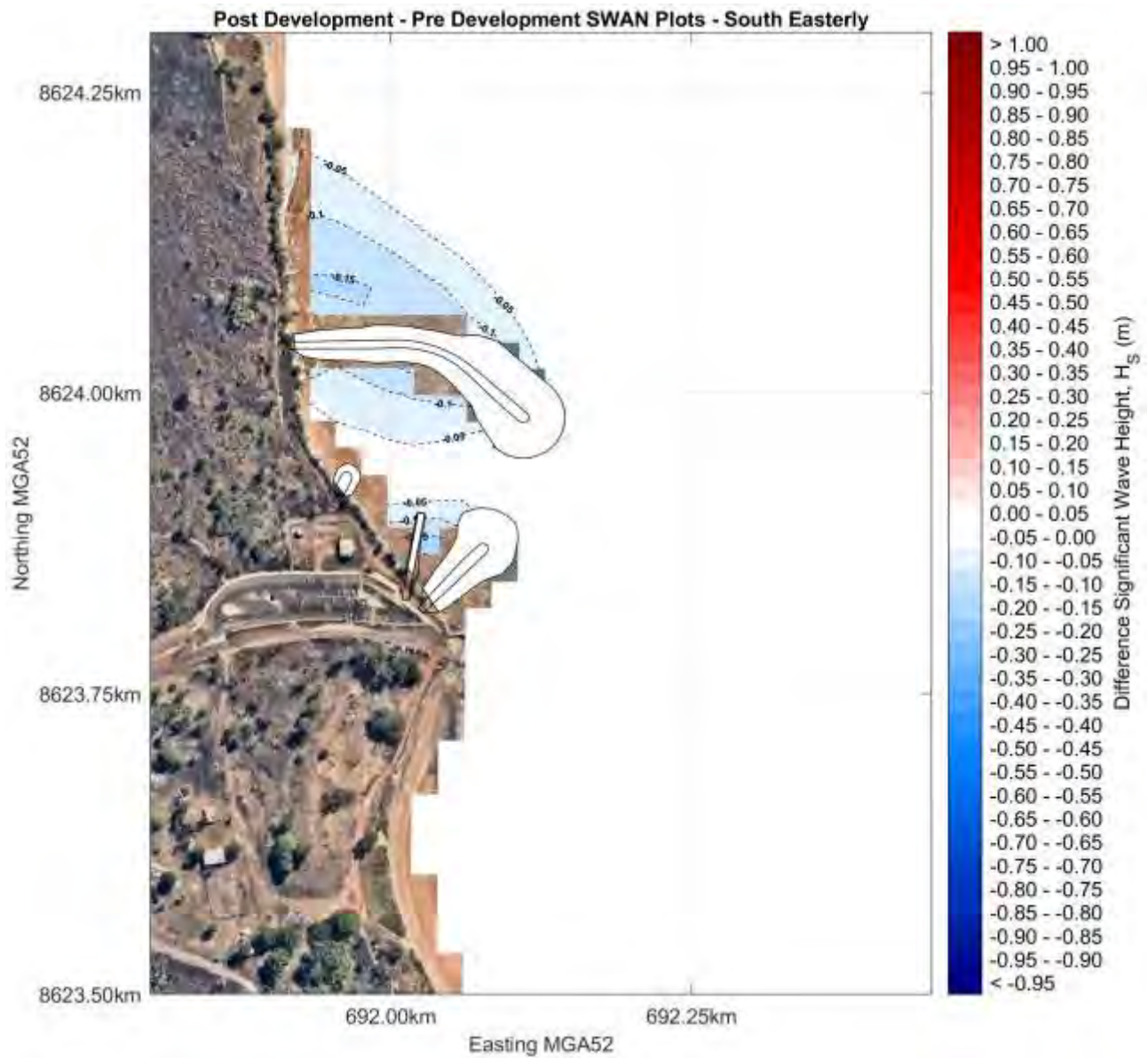


Figure 7-20 Wave vector plot during autumn and winter, south-easterly conditions – Pre vs post development

8 Extreme (Cyclonic) Conditions

8.1 Cyclones

Tropical cyclones in the region mostly form from lows within the monsoon trough, between November and April. There are on average 7.7 days per season when a cyclone exists in the Northern Territory Region. The north western Gulf of Carpentaria near Gove has the highest concentration of cyclone days. The Gulf of Carpentaria averages two cyclones a year, while the Arafura and Timor Seas average one a year. Cyclones in the Gulf of Carpentaria move very erratically, whereas those in the Arafura and Timor Seas tend to follow more regular tracks to the southwest. Over half the cyclones generated in the Northern Region move either southwest or southeast into adjoining regions.

Design wave conditions at this site are affected by tropical cyclones. Storm tide and wave set-up will have a significant influence on wave heights propagating to the nearshore areas where the Mandorah new marine facilities will be constructed. So, a site-specific cyclone modelling study (coupled wave and hydrodynamic modelling) was carried out to determine cyclone associated metocean design parameters such as design wave height, period and direction as well as current speed and storm tide level. For this purpose, historical cyclone track, wind and pressure has been obtained from the Bureau of Meteorology cyclone track database (BoM, 2019). Only cyclones that have occurred since 1964 and affected the study area were selected. These are the historical cyclones that are most likely to have affected the Darwin region to the greatest extent. The tracks of the 25 most severe cyclones (over the last 55 years) which were modelled in this study are presented below in **Figure 8-1**.

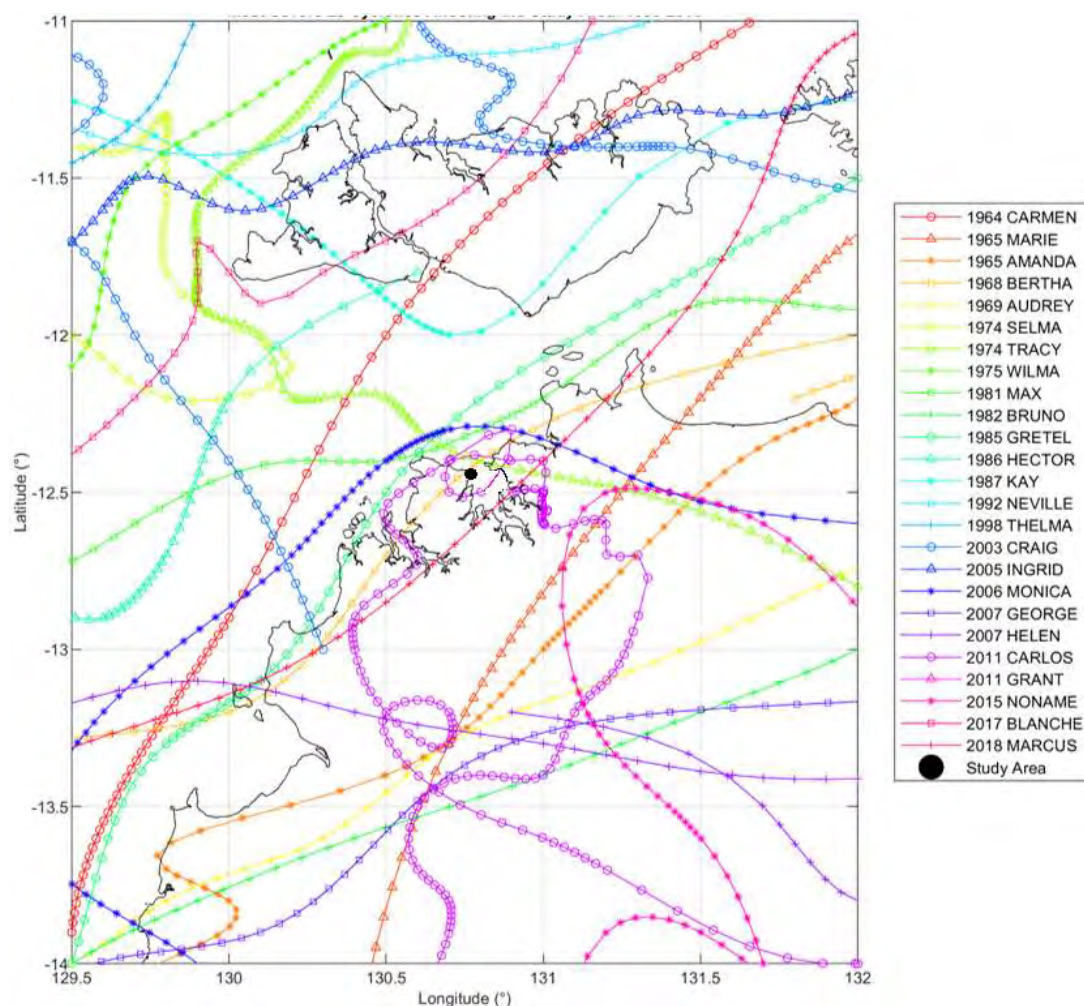


Figure 8-1 Tracks of the 25 most severe cyclones that have affected Darwin since 1964

8.1.2 Hindcast Cyclone Modelling

In order to determine design wave and storm tide criteria at selected ARI, cyclonic storm tide and wave modelling was undertaken for each of the 25 historical tropical storm events listed in Figure 8-1. Spatially variable wind fields were utilised, with the wind field described using the Bureau of Meteorology best track data and the Holland wind field model for the southern hemisphere.

The model output comprised spatially variable time-series in terms of water level (WL), current speed (CS), wave height (H_s), wave period (T_{m01} and T_p) and mean wave direction. Table 8-1 presents the peak water level, current speed and significant wave heights and the associated peak wave periods and direction from each time-series at the output location for each of the selected cyclones

Peak cyclonic significant wave heights at the study site are generally in the region of 2.5 m or less. Notable exceptions to this are Cyclone Tracy, which reached modelled significant wave heights of up to 3.6 m at the site (see Table 8-1), and Cyclone Selma which reached 3.0m.

Similarly, storm tide levels at the site are generally less than 4.0m above MSL. The modelling indicates that Cyclones Thelma and Ingrid caused significant storm tide events, reaching up to 5m and 4.7m above MSL, respectively.

Table 8-1 Cyclone wave modelling results

BoM Cyclone ID	Waves			Hydrodynamics	
	H_{m0} (m)	T_p (s)	Direction (° N)	WL (m, MSL)	Current Speed (m/s)
1964 Carmen	1.77	5.7	17.4	1.55	0.51
1965 Marie	1.02	4.5	164.3	2.69	1.07
1965 Amanda	1.24	5.1	353.0	2.29	0.53
1968 Bertha	1.49	5.7	6.6	3.12	0.70
1969 Audrey	1.59	5.1	355.1	3.42	0.95
1974 Selma	2.99	9.5	34.6	3.30	0.85
1974 Tracy	3.57	5.7	37.6	1.21	0.75
1975 Wilma	2.30	6.5	21.9	3.30	0.96
1981 Max	2.52	7.4	22.5	3.03	0.98
1982 Bruno	0.93	4.5	18.2	1.96	0.53
1985 Gretel	2.14	5.7	18.8	2.17	0.89
1986 Hector	2.19	5.7	17.4	2.27	0.50
1987 Kay	2.49	5.1	73.5	2.12	0.64
1992 Neville	2.14	5.1	78.0	3.00	0.77
1998 Thelma	2.01	5.7	30.1	4.96	1.61
2003 Craig	3.57	6.5	25.9	3.15	0.82
2005 Ingrid	2.20	6.5	42.1	4.72	1.45
2006 Monica	2.34	5.7	12.3	3.04	0.60

BoM Cyclone ID	Waves			Hydrodynamics	
	H _{m0} (m)	T _p (s)	Direction (° N)	WL (m, MSL)	Current Speed (m/s)
2007 George	1.36	4.5	11.2	3.40	0.96
2007 Helen	1.75	7.4	7.2	1.69	0.35
2011 Carlos	1.47	5.1	354.1	2.86	0.92
2011 Grant	0.91	4.5	141.4	3.65	1.15
2015 Noname	1.29	4.5	355.2	3.13	0.75
2017 Blanche	2.01	6.5	27.5	3.50	0.87
2018 Marcus	1.24	3.9	138.5	3.10	0.68

8.2 Design Wave Characteristics

Peak wave conditions were extracted from each cyclone at 25 output locations along the main and lee breakwaters (**Figure 8-2**), and 6 locations coinciding with key infrastructure within the harbour (**Figure 8-3**). The results were used to undertake an extreme value analysis based on Weibull and Gumbell extreme value distributions using the maximum likelihood method and a range of peak over threshold values. The distribution which provided the best fit to the data was the Weibull Distribution with a peak over threshold of 50% of the highest value in the hindcast dataset. These techniques were used to calculate the design cyclonic wave conditions at each output location. Results of the wave climate analyses in terms of significant wave height and associated wave period are presented for recurrence intervals of 5, 10, 20, 25, 50, 100, 200 and 500-years ARI in **Table 8-2** below. Wave periods have been derived from a correlation analysis between H_s and T_p .

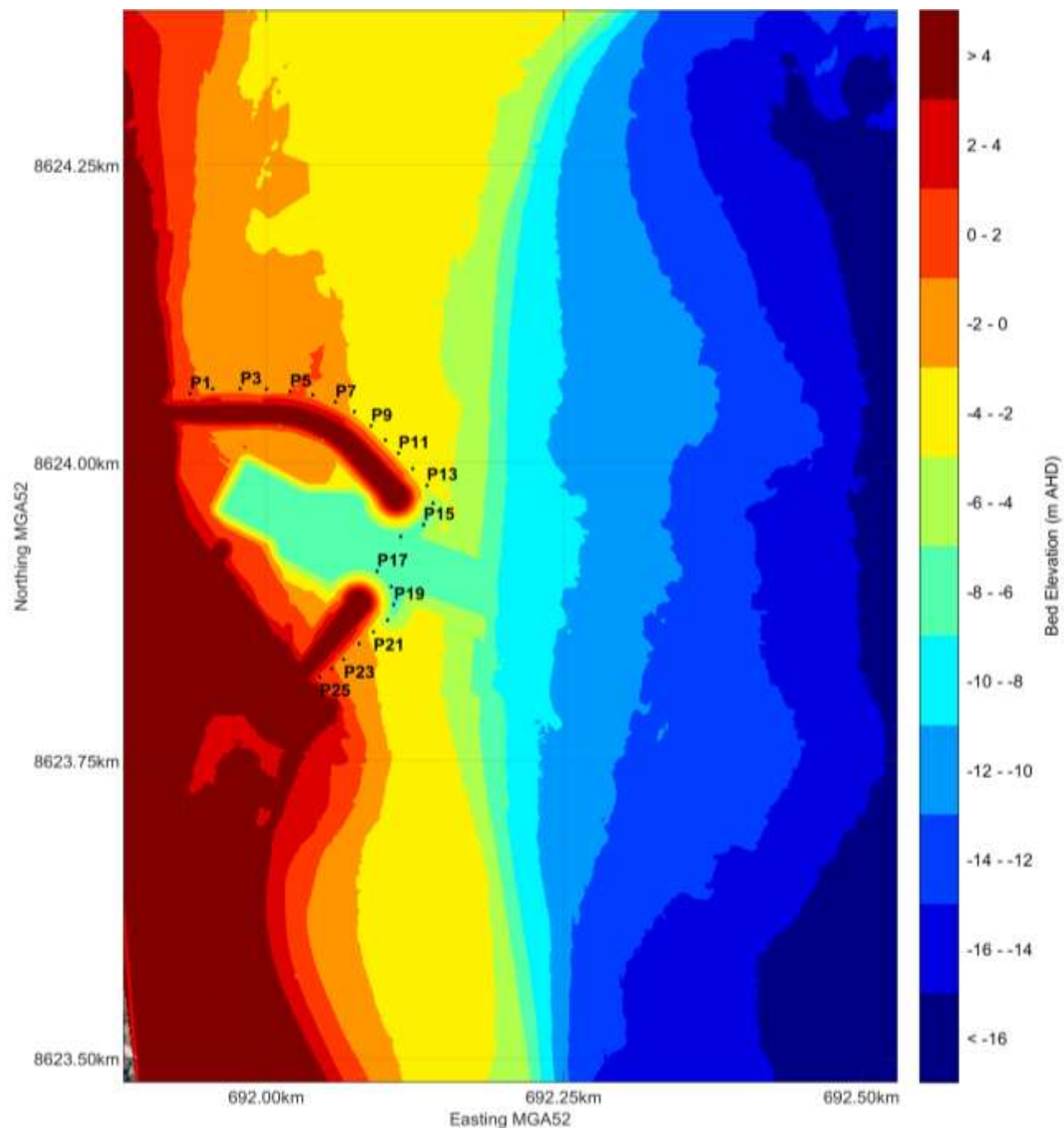


Figure 8-2 Design wave extraction locations along the breakwaters

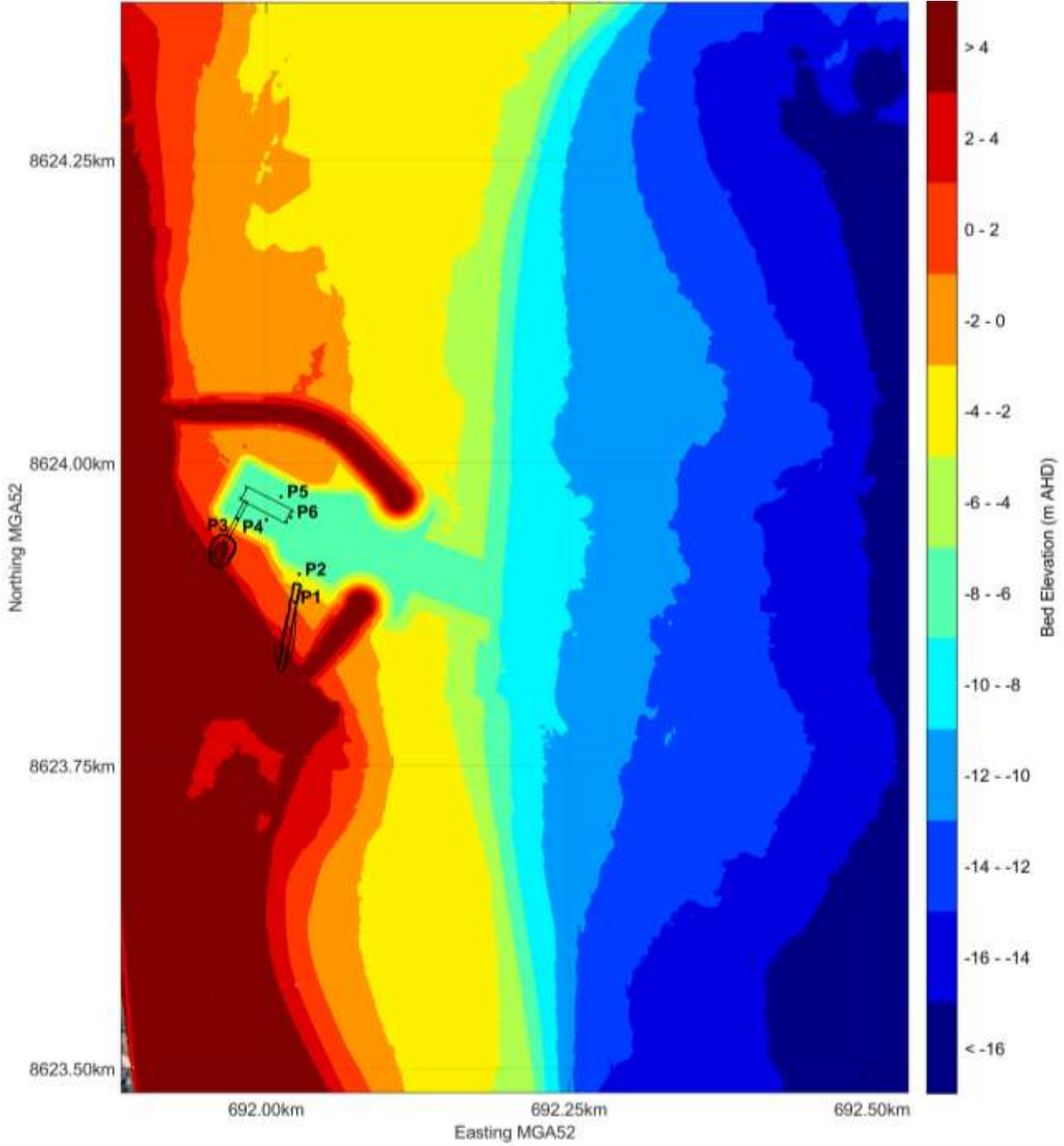


Figure 8-3 Design wave extraction locations inside the harbour

Table 8-2 Extreme significant wave height and peak wave period along the main and lee breakwaters (breakwater points) and inside the harbour (inner points)

ARI (yrs)		5		10		20		25		50		100		200		500	
Point No.		H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)	H _{m0} (m)	T _p (s)
Breakwater Points	1	0.90	6.3	1.07	6.3	1.19	6.3	1.22	6.3	1.31	6.3	1.38	6.3	1.45	6.3	1.52	6.3
	2	1.19	6.2	1.36	6.4	1.48	6.6	1.51	6.6	1.60	6.7	1.67	6.7	1.72	6.8	1.79	6.9
	3	1.24	6.5	1.42	6.8	1.54	7.0	1.57	7.1	1.66	7.2	1.73	7.3	1.78	7.4	1.85	7.6
	4	1.28	6.3	1.45	6.4	1.57	6.5	1.60	6.5	1.68	6.6	1.75	6.6	1.81	6.7	1.87	6.7
	5	1.35	6.2	1.53	6.3	1.65	6.4	1.68	6.4	1.76	6.5	1.83	6.5	1.88	6.5	1.95	6.5
	6	1.32	6.5	1.51	6.8	1.65	7.1	1.68	7.1	1.78	7.3	1.86	7.4	1.93	7.6	2.01	7.7
	7	1.49	6.6	1.75	7.2	1.93	7.6	1.97	7.7	2.10	8.0	2.21	8.3	2.31	8.5	2.42	8.8
	8	1.68	6.6	1.98	7.3	2.20	7.8	2.25	7.9	2.41	8.3	2.54	8.6	2.66	8.9	2.79	9.2
	9	1.76	6.5	2.08	7.2	2.31	7.7	2.37	7.8	2.54	8.2	2.68	8.5	2.81	8.7	2.95	9.0
	10	1.80	6.5	2.13	7.2	2.37	7.7	2.43	7.8	2.61	8.2	2.76	8.5	2.89	8.8	3.04	9.1
	11	1.83	6.6	2.17	7.3	2.41	7.7	2.47	7.9	2.65	8.2	2.80	8.5	2.93	8.8	3.09	9.1
	12	1.83	6.6	2.18	7.3	2.44	7.9	2.51	8.0	2.71	8.4	2.88	8.8	3.03	9.1	3.21	9.5
	13	1.86	6.5	2.22	7.3	2.50	7.8	2.58	8.0	2.79	8.4	2.98	8.8	3.15	9.2	3.36	9.6
	14	1.83	6.1	2.19	6.7	2.47	7.1	2.55	7.3	2.77	7.6	2.96	7.9	3.13	8.2	3.34	8.5
	15	1.69	6.1	2.07	6.6	2.37	7.0	2.46	7.2	2.70	7.5	2.91	7.8	3.11	8.1	3.34	8.4
	16	1.38	5.5	1.74	6.2	2.03	6.7	2.11	6.9	2.35	7.3	2.57	7.7	2.76	8.1	3.00	8.5
	17	1.38	5.7	1.78	6.4	2.10	6.9	2.20	7.0	2.47	7.5	2.71	7.9	2.94	8.3	3.21	8.7
	18	1.52	6.1	1.92	6.9	2.25	7.6	2.34	7.8	2.61	8.4	2.85	8.9	3.08	9.4	3.35	9.9
	19	1.53	6.0	1.91	6.8	2.20	7.5	2.29	7.7	2.53	8.2	2.75	8.6	2.95	9.1	3.19	9.6
	20	1.47	5.7	1.83	6.3	2.11	6.8	2.19	6.9	2.42	7.3	2.62	7.6	2.81	7.9	3.03	8.3
	21	1.32	5.6	1.64	6.3	1.88	6.8	1.95	6.9	2.15	7.3	2.32	7.7	2.48	8.0	2.67	8.4
	22	1.16	6.1	1.45	6.7	1.67	7.2	1.74	7.3	1.92	7.6	2.07	7.9	2.22	8.2	2.39	8.6
	23	1.05	6.2	1.30	6.8	1.50	7.3	1.55	7.4	1.71	7.8	1.86	8.2	1.99	8.5	2.14	8.8
	24	0.90	6.8	1.11	7.4	1.28	7.8	1.33	8.0	1.47	8.3	1.59	8.7	1.70	9.0	1.84	9.4
	25	0.60	6.3	0.76	7.0	0.88	7.5	0.91	7.7	1.00	8.1	1.08	8.4	1.15	8.7	1.24	9.1
Inner Points	1	0.62	6.7	0.83	7.3	0.99	7.9	1.03	8.0	1.16	8.4	1.28	8.8	1.39	9.2	1.52	9.6
	2	0.81	6.3	1.11	7.0	1.36	7.6	1.44	7.7	1.65	8.2	1.85	8.7	2.03	9.1	2.25	9.7
	3	0.59	5.7	0.79	5.9	0.95	6.1	0.99	6.1	1.13	6.2	1.25	6.4	1.35	6.5	1.49	6.6
	4	0.71	4.8	0.93	5.3	1.10	5.6	1.15	5.7	1.29	6.0	1.41	6.3	1.52	6.5	1.66	6.8
	5	0.69	5.0	0.86	5.0	0.98	5.1	1.01	5.1	1.10	5.1	1.18	5.2	1.24	5.2	1.31	5.2
	6	0.76	4.9	0.97	5.2	1.13	5.3	1.17	5.4	1.29	5.5	1.40	5.6	1.49	5.7	1.60	5.8

8.3 Cyclonic Storm Tide Levels

Similarly, design storm tide levels with respect to mean sea level and the project datum are provided below in **Table 8-3**. The values presented below were fitted to the Weibull Distribution with a peak over threshold of 2.5 m (above MSL), because this distribution provided the best fit to the data.

Table 8-3 Design Storm Tide Levels – Cyclones

ARI Years	WL m, MSL	95% Confidence Limits		WL m, CD
		lower	Upper	
10	3.51	3.16	3.86	7.70
20	3.90	3.36	4.44	8.09
30	4.11	3.46	4.77	8.30
40	4.26	3.52	5.01	8.45
50	4.38	3.56	5.19	8.57
60	4.47	3.60	5.33	8.66
70	4.54	3.63	5.46	8.73
80	4.61	3.65	5.57	8.80
90	4.67	3.67	5.66	8.86
100	4.72	3.69	5.75	8.91
200	5.05	3.79	6.30	9.24
300	5.23	3.84	6.63	9.42
400	5.36	3.87	6.86	9.55
500	5.47	3.90	7.04	9.66

9 Physical Modelling Studies

Physical modelling was undertaken for the project by the University of New South Wales' Water Research Laboratory, to confirm the integrity of the nominated breakwater designs. The metocean conditions detailed in this report were interpreted and applied in the physical modelling studies. The physical modelling report is provided in **Appendix C** of this report and the results of physical modelling have been interpreted to refine the breakwaters design.

10 Harbour Agitation

10.1 Purpose

In the design of harbours and boat ramps for ferries and small watercraft, it is necessary to limit the height of waves that can impinge on vessels berthed in the harbour or boats being launched and retrieved. This limitation is necessary to ensure that the harbour is a safe haven for users and to prevent damage of vessels and harbour infrastructure.

The purpose of this modelling is to assess the performance of the design from a wave tranquillity perspective and derive metocean design parameters for maritime structures within the harbour. In order to investigate the penetration of waves into the harbour basin, wave agitation modelling was undertaken using the SWASH Wave model system developed by Deltares.

10.2 Guidelines for Acceptable Wave Conditions

There are guidelines available that set forth recommended criteria for berthing conditions in small craft harbours. These guidelines are based on decades of observations of vessel motion in small craft harbours, and how that motion is caused by waves of particular height, period, length and direction – relative to a moored vessel. Most of the relevant guidelines used in Australia and globally are relatively consistent in terms of their recommended berthing conditions. These predominant guidelines are:

- > AS3962: Guidelines for Design of Marinas (2020);
- > The PIANC Guidelines for Marina Design (2016); and
- > The ASCE Planning and Design Guidelines for Small Craft Harbours (2012).

Australian guideline (AS3962) will be used as the predominant basis for this investigation.

10.2.1 AS3962: Guidelines for Design of Marinas

The wave climate criteria set out in AS3962 is based on wave period (>2 seconds, is the common case), and the directionality of the incident (or incoming) waves that are experienced by vessels berthed in the harbour. These directions are referred to as head seas (waves incident on the bow or stern), beam seas (waves incident directly on the side) or oblique seas (waves incident at an oblique angle). A graphical depiction of these wave directions is provided in **Figure 10-1**, and a depiction of the six degrees of vessel movement is provided in **Figure 10-2** for reference. These wave directions are defined in AS3962 as:

- > **Head Seas:** Sea condition with waves approaching up to 22.5° from the vessel centre-line. These seas will cause vessels to surge, heave and pitch;
- > **Beam Seas:** Sea condition with waves approaching from within 22.5° from abeam. These seas will cause vessels to sway, heave and roll; and
- > **Oblique Seas:** Sea conditions that are not head seas or beam seas. These seas may cause a range of combinations of vessel movements.

Because most vessels are longer than they are wide, waves of a given height will generate a greater degree of roll motion (when approaching from beam-on) than pitch motion (when approaching from head-on). Excessive roll motion can reduce the operability of berthed vessels and result in safety issues for crew and passengers. Consequently, most guidelines recommend lower wave heights for beam seas than head seas (with oblique seas being somewhere in the middle).

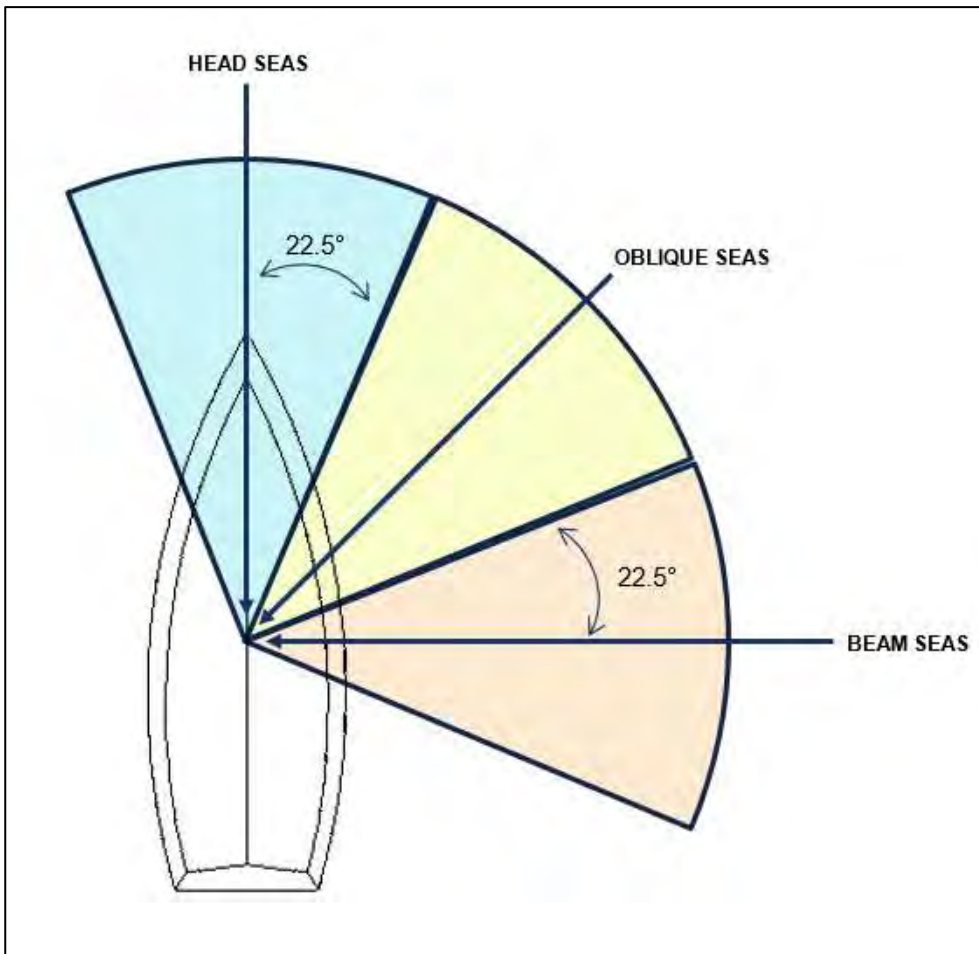


Figure 10-1 Wave Direction Classifications for Marinas and Harbours (AS3962)

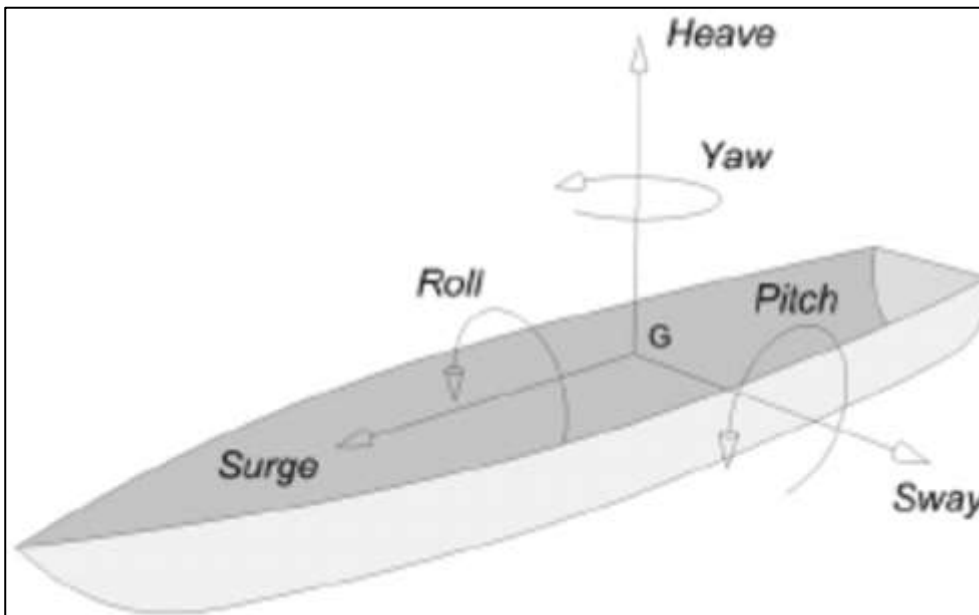


Figure 10-2 The Six Degrees of Freedom of Vessel Motion. Image source: Varela et al, 2011.

Most marinas and harbours in Australia are designed to limit wave conditions for moored vessels, such that the extreme wave conditions satisfy the criteria provided in *Australian Standard AS3962: Guidelines for Design of Marinas* (AS, 2001). AS3962 provides guidelines for three levels of acceptable wave conditions within marinas, which are referred to as “excellent”, “good” and “moderate” conditions. These guidelines have been adapted from Mercer et al (1982).

Table 10-1, extracted from AS3962, provides guidelines for “good” wave conditions. Note that different criteria are provided for waves approaching moored boats from different directions relative to the moored-vessel alignment; as described earlier. AS3962 states that “excellent” conditions are factored down by 25% from “good”, and “moderate” conditions are factored up by 25% - as given in Table 10-2 and **Table 10-3**.

These criteria have been applied to the assessment of the harbour in terms of the suitability of at-berth wave conditions. That assessment has required wave heights and wave crest patterns at berth locations to be determined through the modelling procedures described in **Section 10.3**.

Table 10-1 AS3962 Criteria for a ‘Good’ Wave Climate in a Small Craft Harbour

Direction and Peak Period of Design Harbour Wave	Significant Wave Height	
	Wave Event Exceeded Once in 50 Years	Wave Event Exceeded Once in 1 Year
Head Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.3m wave height
Head Seas Greater than 2s	Less than 0.6m wave height	Less than 0.3m wave height
Oblique Seas Greater than 2s	Less than 0.4m wave height	Less than 0.3m wave height
Beam Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.3m wave height
Beam Seas Greater than 2s	Less than 0.25m wave height	Less than 0.15m wave height

Table 10-2 AS3962 Criteria for a ‘Excellent Wave Climate in a Small Craft Harbour

Direction and Peak Period of Design Harbour Wave	Significant Wave Height	
	Wave Event Exceeded Once in 50 Years	Wave Event Exceeded Once in 1 Year
Head Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.23m wave height
Head Seas Greater than 2s	Less than 0.45m wave height	Less than 0.23m wave height
Oblique Seas Greater than 2s	Less than 0.3m wave height	Less than 0.23m wave height
Beam Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.23m wave height
Beam Seas Greater than 2s	Less than 0.19m wave height	Less than 0.11m wave height

Table 10-3 AS3962 Criteria for a ‘Moderate’ Wave Climate in a Small Craft Harbour

Direction and Peak Period of Design Harbour Wave	Significant Wave Height	
	Wave Event Exceeded Once in 50 Years	Wave Event Exceeded Once in 1 Year
Head Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.38m wave height
Head Seas Greater than 2s	Less than 0.75m wave height	Less than 0.38m wave height
Oblique Seas Greater than 2s	Less than 0.5m wave height	Less than 0.38m wave height
Beam Seas Less than 2s	<i>Conditions not likely to occur during this event</i>	Less than 0.38m wave height
Beam Seas Greater than 2s	Less than 0.31m wave height	Less than 0.19m wave height

As part of the proposed facility at Mandorah, a boat launching ramp will also be constructed inside the harbour basin for the launching and recovery of boats. As per AS3662, boat launching ramps should be located and aligned as follows:

- > Aligned into the dominant waves from swell, sea and boat wash; and

- > Sheltered from waves larger than 0.2 m.

10.3 SWASH Wave Modelling

SWASH is a robust phase resolving model and it requires small grid sizes and time steps. SWASH includes refraction, shoaling, diffraction, bed friction, wave breaking and frequency-direction spectral input. It can include partial and full wave reflectivity – rock breakwaters, attenuators and vertical walls.

10.3.1 Model Setup

To estimate wave conditions in and around the harbour, the detailed phase resolving SWASH model was applied. The SWASH model set-up consisted of a 1.25m resolution rectilinear grid extending 300-350m offshore of the harbour entrance, and 250-350m either side of the proposed harbour. Grid extent is 900m in the alongshore direction and 650m in the cross-shore direction. **Figure 10-3** depicts the SWASH wave models' grid and bathymetry.

It is noted that the wave agitation modelling was undertaken under different wave conditions for waves from E, ESE and SE directions and for 1-year and 50-year ARI storm events. The summary of the wave conditions including the significant wave height (H_s), peak wave period (T_p) and peak wave direction (PWD) defined as input in the boundary of the wave agitation models is presented in **Table 10-4**. In total, 6 simulations were undertaken to evaluate the tranquillity of the harbour. It is worth mentioning that all models were run under Mean High Water Springs (MHWS=2.91 AHD) condition – a conservative approach.

Table 10-4 Summary of wave conditions defined as input in the boundary of the wave agitation models for all three options

ARI (Years)	Significant Wave Height, H_s (m)	Peak Wave Period, T_p (s)	Peak Wave Direction, PWD (° N)
1	1.07	3.8	90
1	1.08	3.9	112.5
1	1.07	3.9	135
50	1.45	4.5	90
50	1.44	4.6	112.5
50	1.53	5.0	135

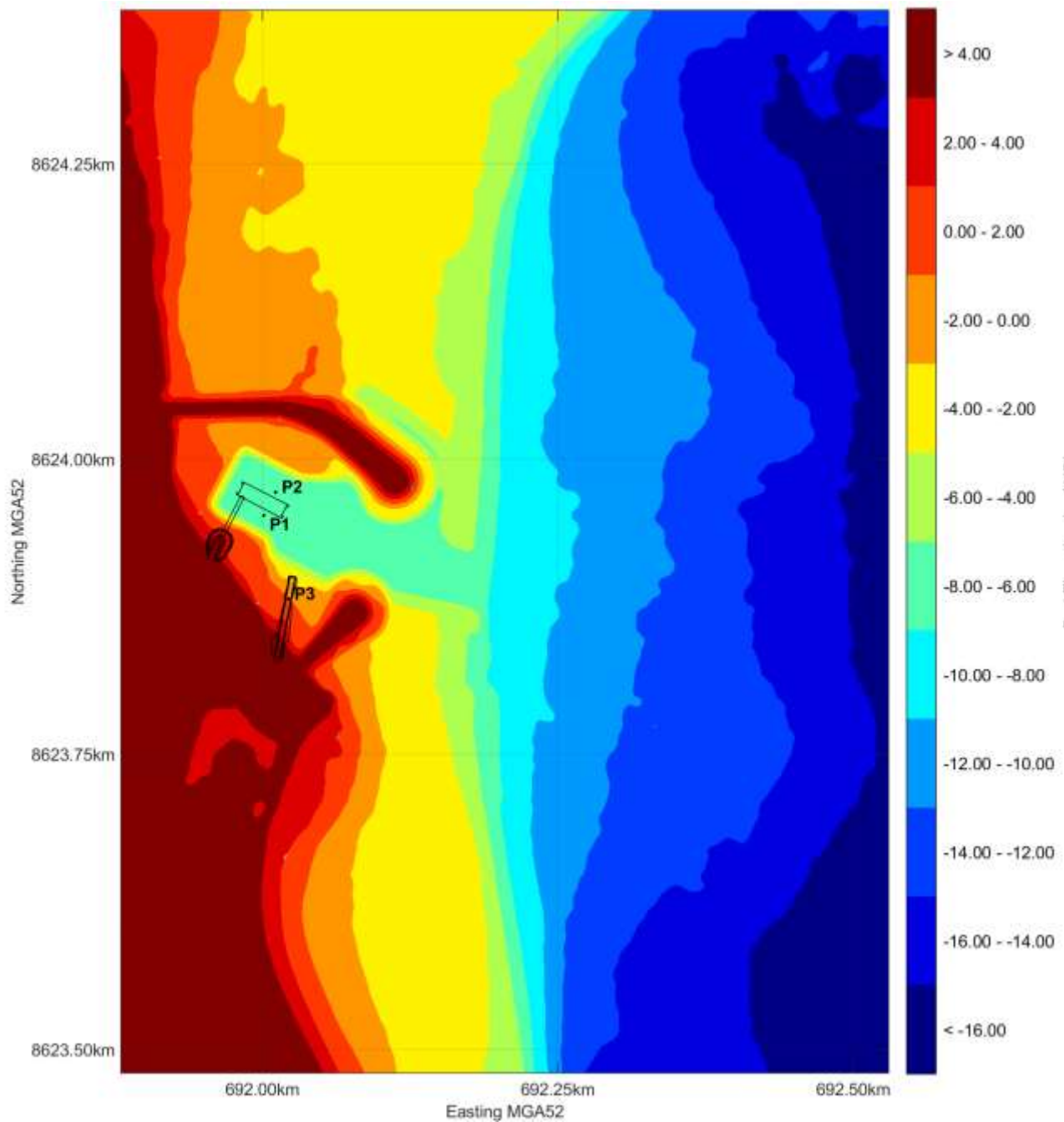


Figure 10-3 SWASH wave model grid and bathymetry utilised for the wave agitation modelling

10.3.2 Modelling Results

The results of the SWASH wave modelling provided a series of wave maps under different wave conditions outlined above. The modelling results for each scenario listed in **Table 10-4** are presented below in **Figures 10-4 to 10-9**. Due to the relatively short wave periods being resolved, some numerical dissipation was evident along the model boundary. As such, the results from the 1-year ARI events have been scaled to ensure the design significant wave height is sustained within the model domain.

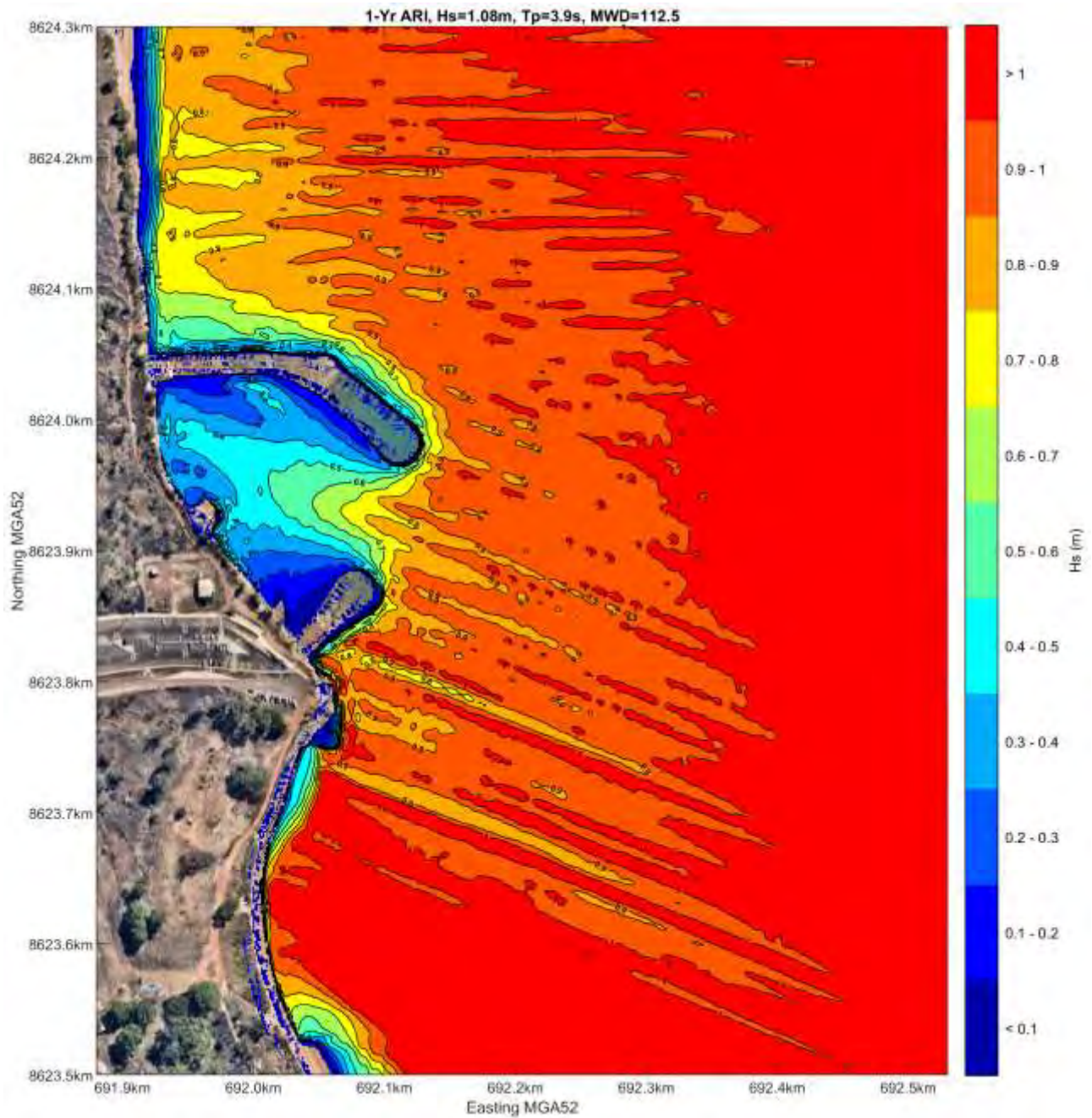


Figure 10-4 SWASH wave modelling result for 1-year ARI waves from ESE direction

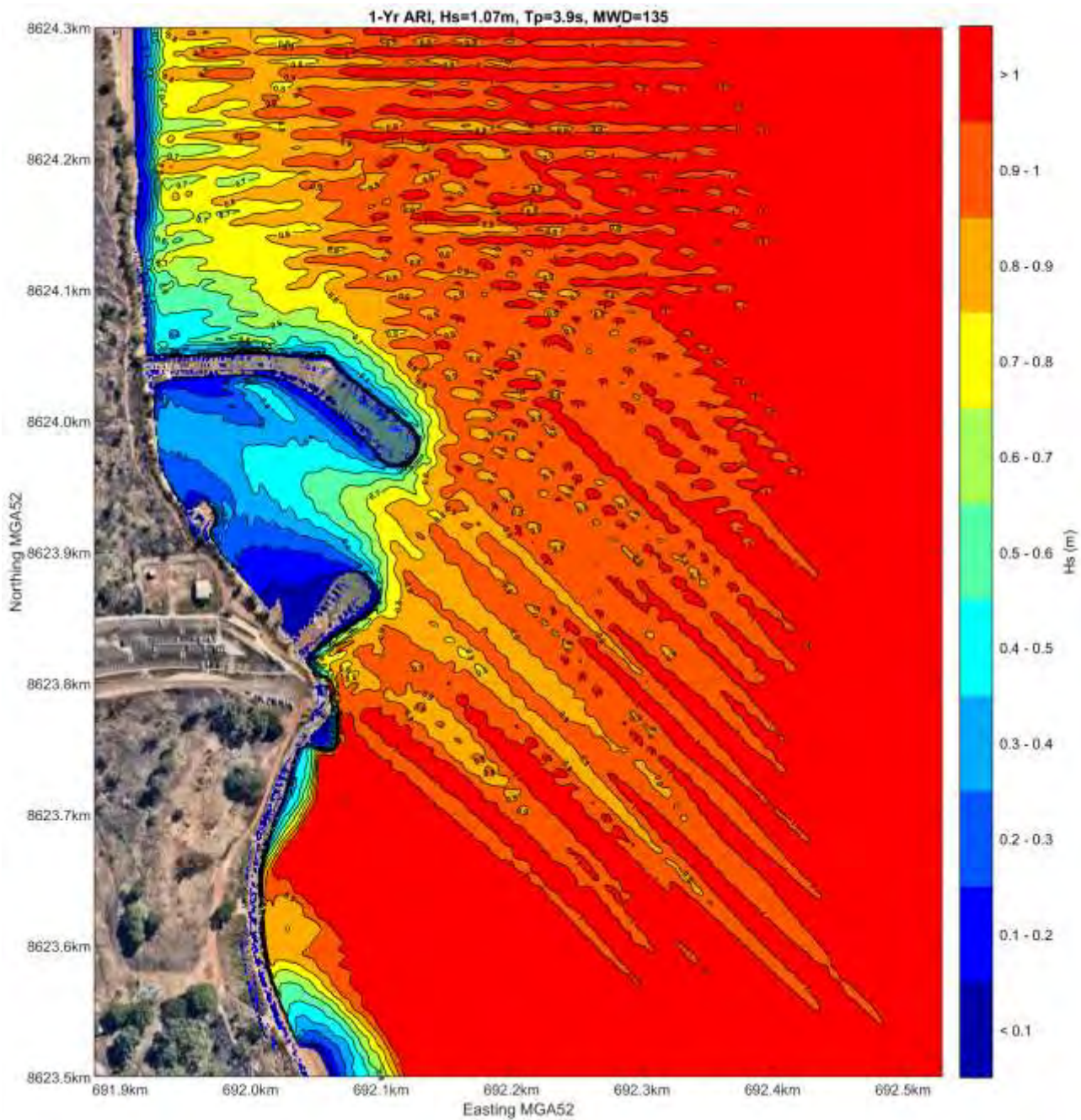


Figure 10-5 SWASH wave modelling result for 1-year ARI waves from SE direction

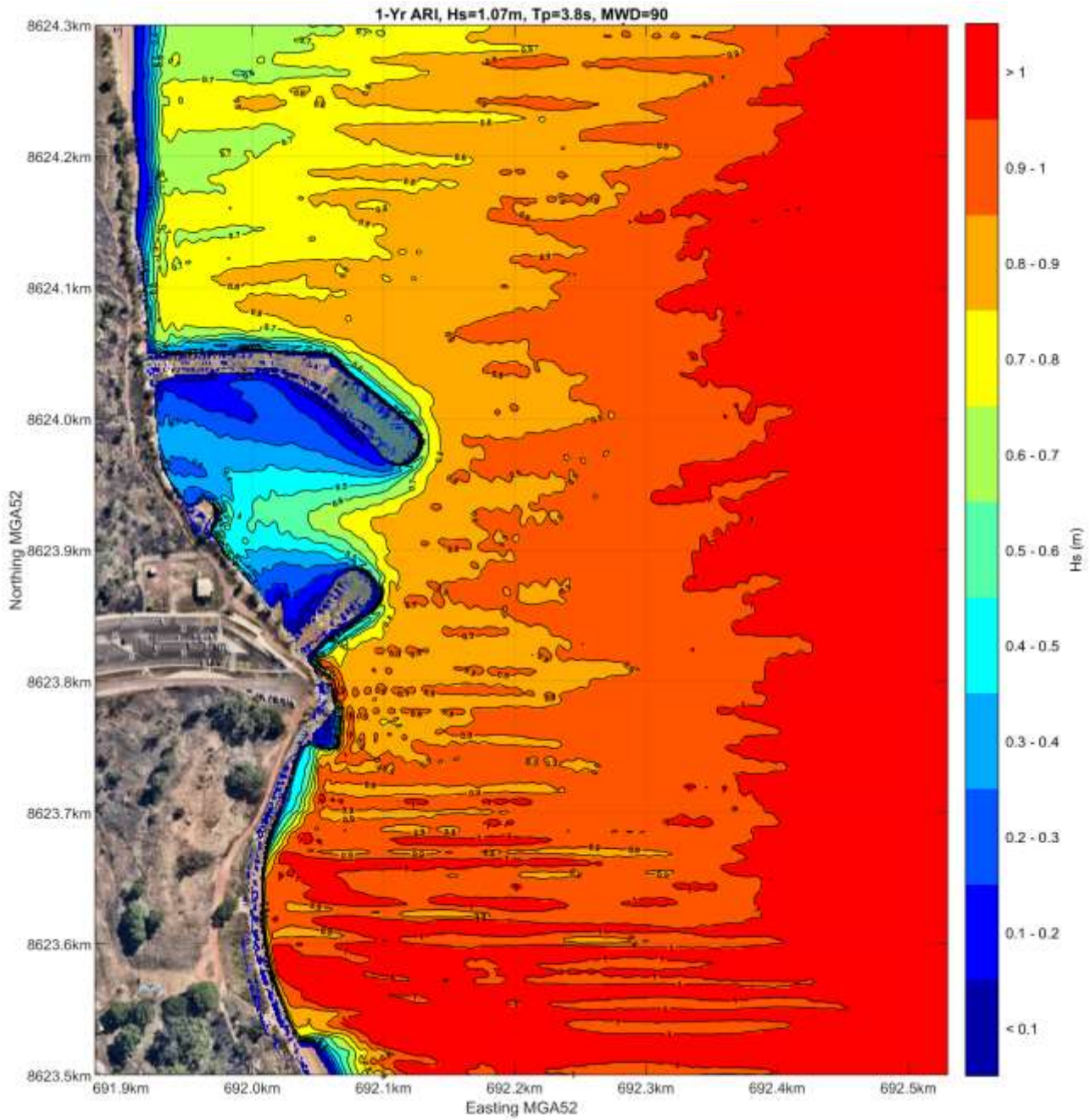


Figure 10-6 SWASH wave modelling result for 1-year ARI waves from E direction

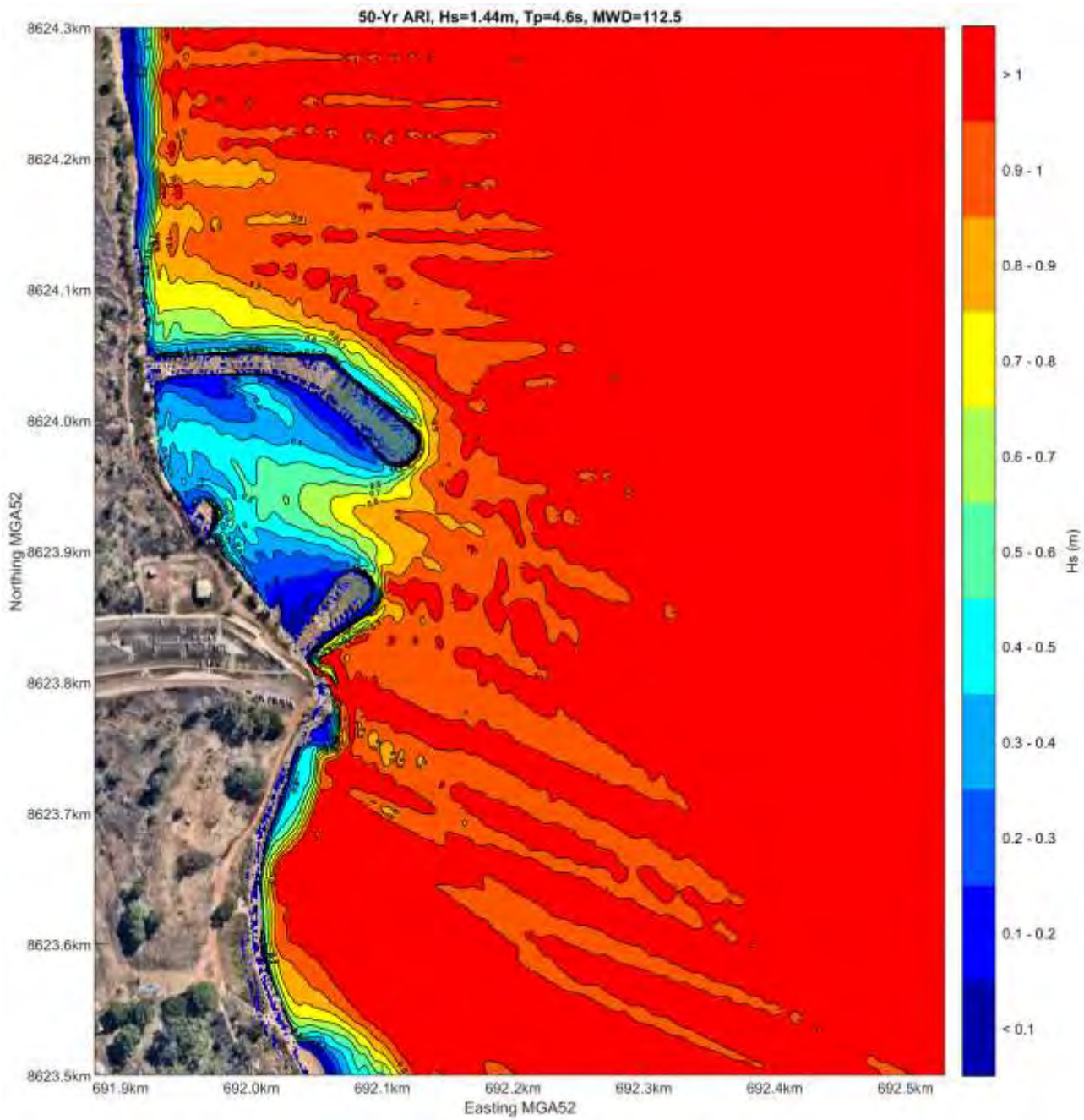


Figure 10-7 SWASH wave modelling result for 50-year ARI waves from ESE direction

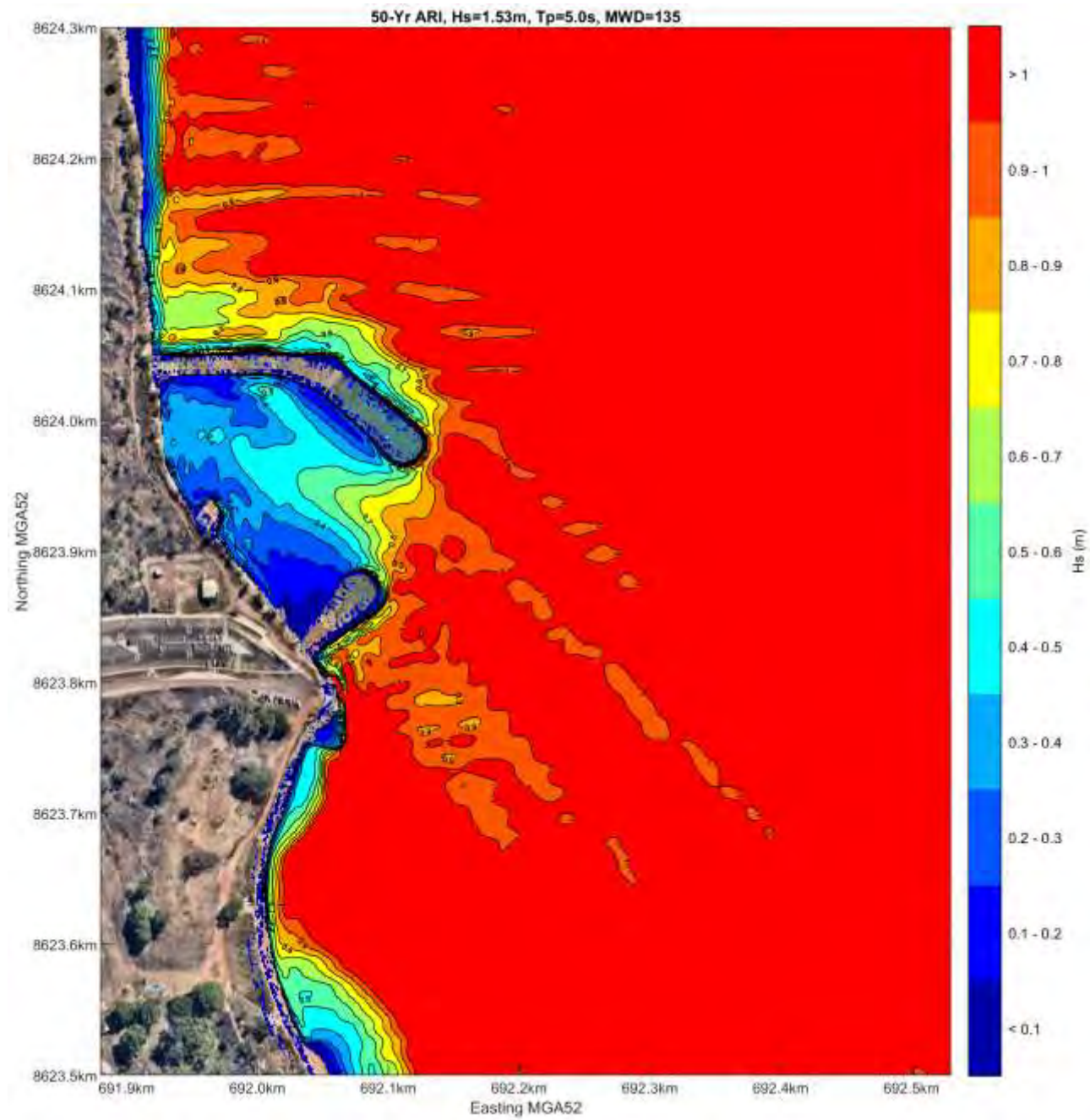


Figure 10-8 SWASH wave modelling result for 50-year ARI waves from SE direction

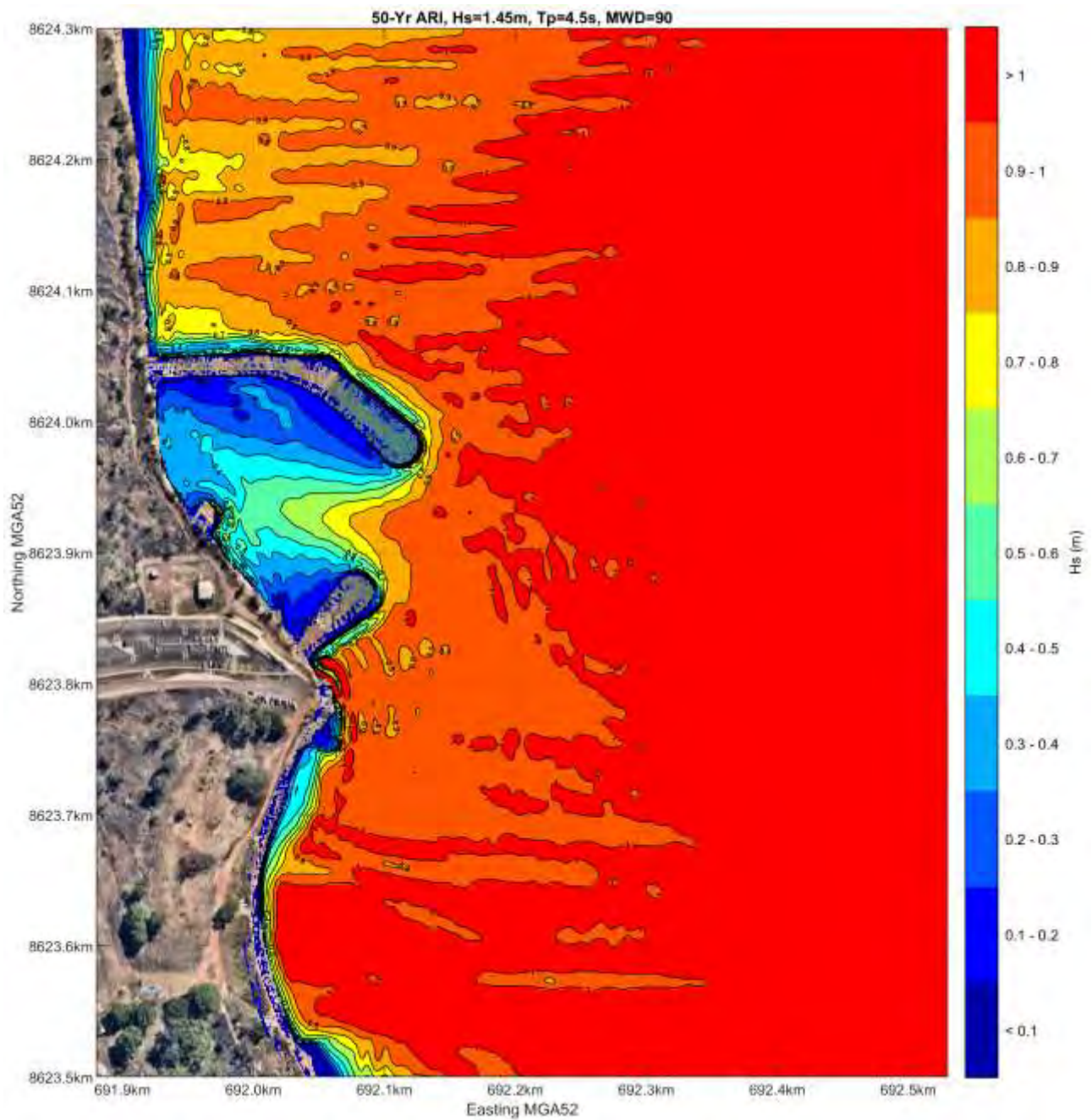


Figure 10-9 SWASH wave modelling result for 50-year ARI waves from E direction

In order to assess the wave climate inside the harbour based on the tranquillity criteria outlined in AS3962, the significant wave height was extracted in 3 different locations inside the harbour (as shown previously in **Figure 10-3**) and then compared to the tranquillity criteria. The result of this assessment is presented in **Table 10-5**.

Table 10-5 The significant wave height extracted at the nominated output locations

Point	1-Yr ARI PWD = 90	1-Yr ARI PWD = 112.5	1-Yr ARI PWD = 135	50-Yr ARI PWD = 90	50-Yr ARI PWD = 112.5	50-Yr ARI PWD = 135
P1	0.44/ 0.30	0.49/ 0.34	0.41/ 0.29	0.51	0.50	0.42
P2	0.36	0.49	0.43	0.42	0.50	0.43
P3	0.26	0.19	0.11	0.31	0.22	0.18

The compliance of the ferry berth (P1 and P2) and boat ramp (P3) with respect to the criteria set out in AS3962, will not only be dependent upon the wave heights, but also on orientation of the facilities (i.e. are the facilities exposed to head seas, oblique seas, or beam seas). In order to determine whether waves approaching each facility constituted head, oblique, or beam seas, a two-step approach was adopted. In the first instance, the orientation of the facilities was determined by examination of the proposed layout. Secondly, the approximate angle of the incoming waves was determined at each facility based on the outcomes of SWASH wave modelling for different wave directions. The angle of wave approach relative to each berth, θ , then was determined based on the wave crest pattern using the surface elevation extracted from SWASH wave modelling. As discussed earlier, head-seas occur when waves approach within 22.5° from the vessel centre-line (bow or stern), beam-seas occur when waves approach from within 22.5° from beam, (or $>67.5^\circ$ from the bow) and oblique seas are those that arrive from between these two conditions. Based on the modelling results and orientations of the facilities, head seas will impinge on both vessels berthed at the ferry berth, and vessels launching and retrieving from the boat ramp.

Based on the modelled wave height within the harbour, the ferry terminal (points P1 and P2) will satisfy “good” wave conditions (head seas) for the 50-year ARI cases, but are larger than moderate wave conditions (head seas) for the 1-year ARI cases. The boat (point P3) ramp satisfy the “excellent” wave conditions (head seas) for the 50-years ARI cases and “good” wave conditions for the 1-year ARI cases. The wave conditions assessed in the model are based on the non-cyclonic wave climate. The facility is not expected to comply with AS3962 conditions under extreme cyclonic events.

Note that the operability requirements stipulated in AS3962 relate to marinas, which typically include small vessels which are moored there for the majority of the time. At Mandorah, the facility will not be used for permanent berthing or bunkering – so the AS3962 criteria would be considered to be stringent. Alternate criteria for ferries are provided in the PIANC document “Harbour Approach Channels – A Design Guide”, which indicates that when waves approach head on the limiting wave condition for loading and unloading of passengers is $H_s = 0.5\text{m}$. Based on the modelling, this condition is expected to exceeded 1 or 2 times a year, on average (i.e. the 1 year ARI non-cyclonic wave height is estimated to be 0.49m at the berth). This is a significant improvement to the existing jetty, which considering the beam on alignment and the modelled wave climate would have at least 30% downtime when using a limiting significant wave height of 0.3m for beam on waves.

AS 3662 stipulates that boat launching ramps should be sheltered from waves larger than 0.2 m, however there is no information provided in the Standard as to the frequency of occurrence of the 0.2 m wave.

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APPENDIX

A

ADCP LOGSHEETS

RDI WH ADCP LOG SHEET

INSTRUMENT S/N	18740	JOB NO.:	DZ1910
FREQUENCY	1200KHz + WAVES	By (initials)	DRS
DATE DEPLOYED	12/12/2019	LOCATION	MANDORAH
DEPLOYMENT NAME	Man 1	BOTTOM TRACKING	☐ ☐
CONFIGURATION FILES	JOB FOLDER		

SETUP

CURRENTS

Depth.....6.....m
 Duration.....65.....days
 Ensemble Int.....60.....mins
 Ping/Ens.....50.....
 No. Cells.....23.....
 Cell Length.....0.5.....m
 No. Batteries.....1.....
 Memory Size.....512.....

WAVES

Burst Length.....20.....Mins
 Burst Int.....60.....Mins
 Samples/Bst.....2400.....
 Min Period Non-Dim.....1.71.....s
 Dirn.....1.93.....s
 Temp.....32.....°C
 Salinity.....35.....ppt

RESULTS

First Cell.....1.05.....m
 Last Cell.....12.05.....m
 Max Range.....14.58.....m
 S.Dev.....0.98.....cm/s
 Storage.....279.54.....MB
 Power.....678.58.....Wh
 Battery usage.....1.5.....

BATTERIES

Serial No. Installed.....1 X 44 VOLT..... By.....DRS..... Date.....11/12/2019.....
 Prior Usage.....NIL.....

MEMORY

Cards Installed (number and size).....2 X 256 MB..... By.....DRS..... Date.....11/12/2019.....
 Start: RR.....0.....files RS.....250.....used/.....0.....free
 End: RR.....2.....files RS.....88.....used/.....162.....free

CHECKS (RUN BBTALK)

(Before and After, tick boxes)

CONNECT (END) ☒ ☒
 INFO (PSØ) ☒ ☒
 SYSTEM TEST (PA) ☒ ☒

BAUD: 9600 TEST LOG FILES (F3)
 MANDORAH ADCP Dep 1.doc

SENSOR TESTS (PC2)-

Pitch	☒	☒	Temperature	☒	☒
Roll	☒	☒	Compass Rotation	☒	☒

BEAM RUB (PC1) 1 ☒ ☐ 2 ☒ ☒ 3 ☒ ☒ 4 ☒ ☒

BOTTOM TRACKING (If applicable)

TIMING

ADCP TIME ☒ TIME ZONE ACST

DEPLOYMENT

TIME ON 18:35 11/12/19 ADCP COORDS: 12°26.482', 130°46.103'
 TIME IN WATER 07:04 12/12/19 CLUMP COORDS: 12°26.485', 130°46.078'
 TIME OFF LOCATION 07:06 12/12/19

RECOVERY

TIME ON LOCATION 08:00 2/3/20
 TIME OUT OF WATER 08:12 2/3/20
 TIME STOPPED 14:00 2/3/20
 DATE READ/FILE NAME MAN 1-000, MAN 2-000

COMMENTS

PINGER DTIS 707 - 27KHz

RDI WH ADCP LOG SHEET

INSTRUMENT S/N	18738	JOB NO.:	DZ1910
FREQUENCY	1200 KHz + WAVES	By (initials)	JB
DATE DEPLOYED	2/3/20	LOCATION	MANDORAH
DEPLOYMENT NAME	Man 2 -	BOTTOM TRACKING	☐ ☐
CONFIGURATION FILES	JOB FOLDER		

SETUP

CURRENTS

Depth.....6.....m
 Duration.....65.....days
 Ensemble Int.....60.....mins
 Ping/Ens.....50.....
 No. Cells.....23.....
 Cell Length.....0.5.....m
 No. Batteries.....1.....
 Memory Size.....1GB.....

WAVES

Burst Length.....20.....Mins
 Burst Int.....60.....Mins
 Samples/Bst.....2400.....
 Min Period Non-Dim.....1.71.....s
 Dirn.....1.93.....s
 Temp.....32.....°C
 Salinity.....35.....ppt

RESULTS

First Cell.....1.05.....m
 Last Cell.....12.05.....m
 Max Range.....14.58.....m
 S.Dev.....0.98.....cm/s
 Storage.....279.54.....MB
 Power.....678.58.....Wh
 Battery usage.....1.5.....

BATTERIES

Serial No. Installed.....1x44V.....By DRS/JB.....Date 1/3/20
 Prior Usage.....NIL.....

MEMORY

Cards Installed (number and size).....1x1GB.....By DRS/JB.....Date 1/3/20
 Start: RR.....files RS.....used/.....free
 End: RR.....files RS.....used/.....free

CHECKS (RUN BBTALK)

(Before and After, tick boxes)

CONNECT (END) ☒ ☒
 INFO (PSØ) ☒ ☒
 SYSTEM TEST (PA) ☒ ☐

BAUD: 9600 TEST LOG FILES (F3)
 Mandorah ADCP_Dep2.doc

SENSOR TESTS (PC2)-

Pitch ☒ ☒ Temperature ☒ ☒
 Roll ☒ ☒ Compass Rotation ☒ ☒

BEAM RUB (PC1) 1 ☒ ☒ 2 ☒ ☒ 3 ☒ ☒ 4 ☒ ☒

BOTTOM TRACKING (If applicable)

TIMING

ADCP TIME ☒ TIME ZONE ACST

DEPLOYMENT

TIME ON 15:15 1/3/20 12° 26.482', 130° 46.103'
 TIME IN WATER 08:30 10:00 2/3/20
 TIME OFF LOCATION 10:10

RECOVERY

TIME ON LOCATION 07:00 22/4/20
 TIME OUT OF WATER 07:30 22/4/20
 TIME STOPPED 13:00 22/4/20
 DATE READ/FILE NAME MAN1_000

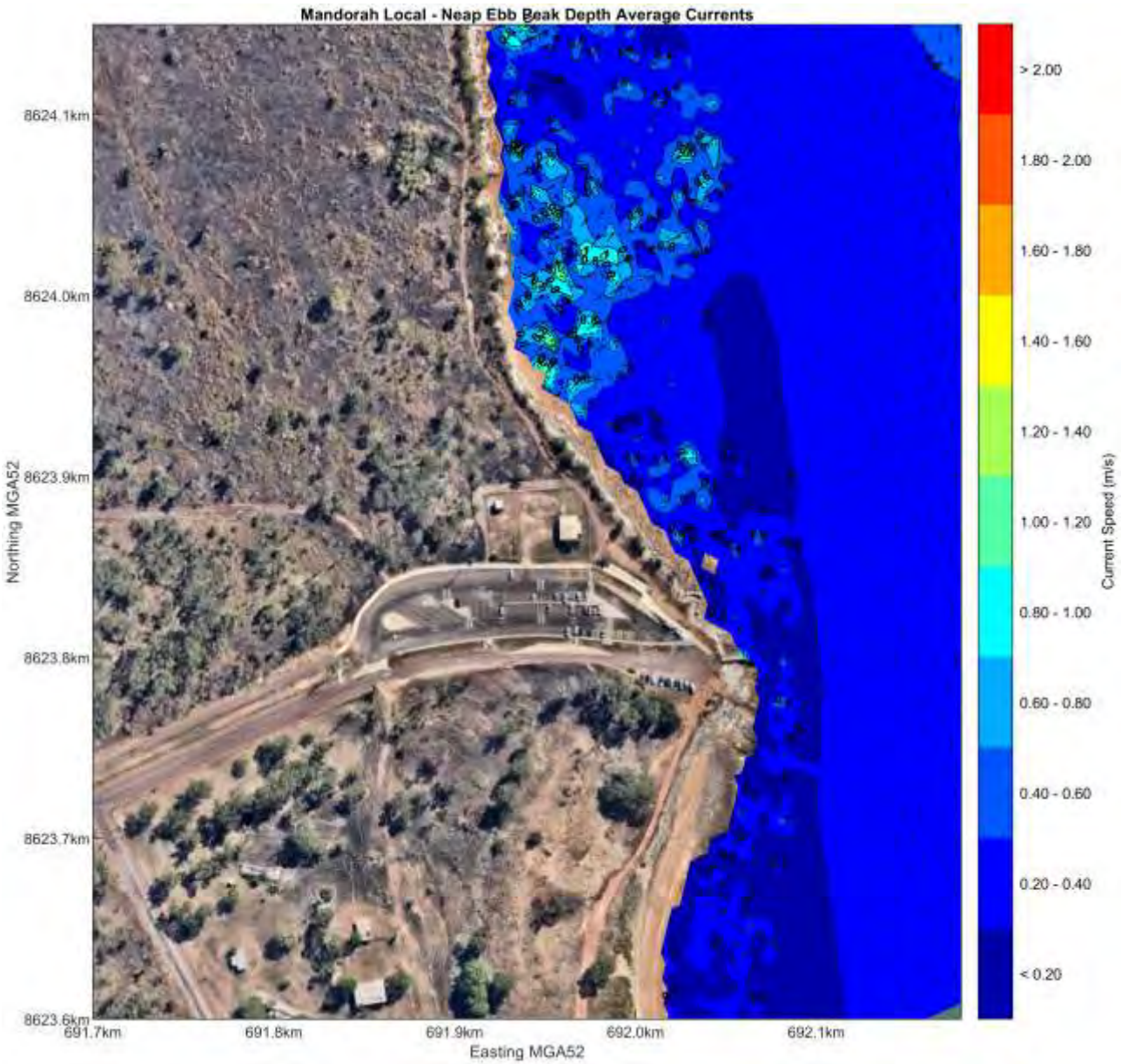
COMMENTS

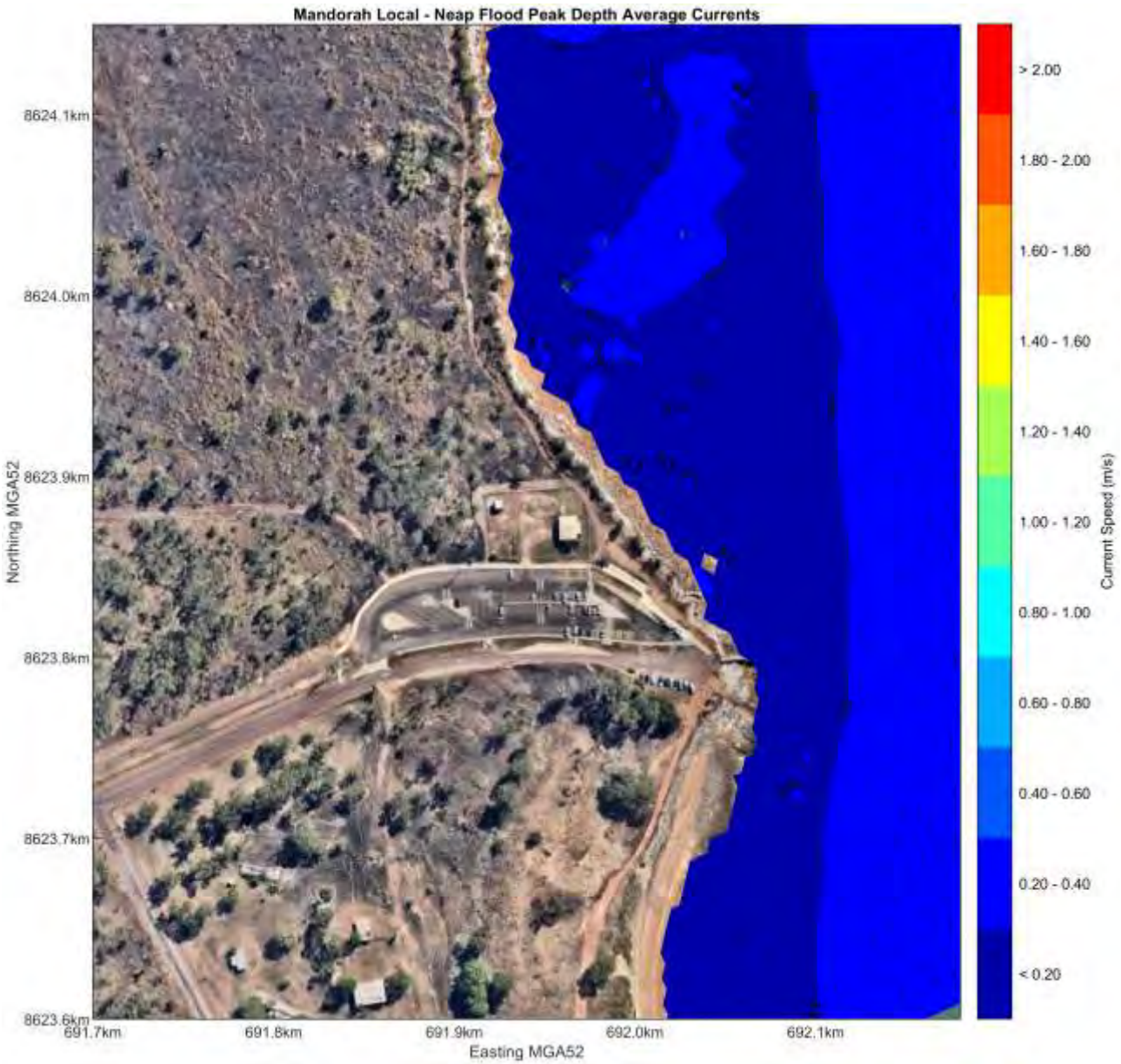
PINGER - 37 kHz

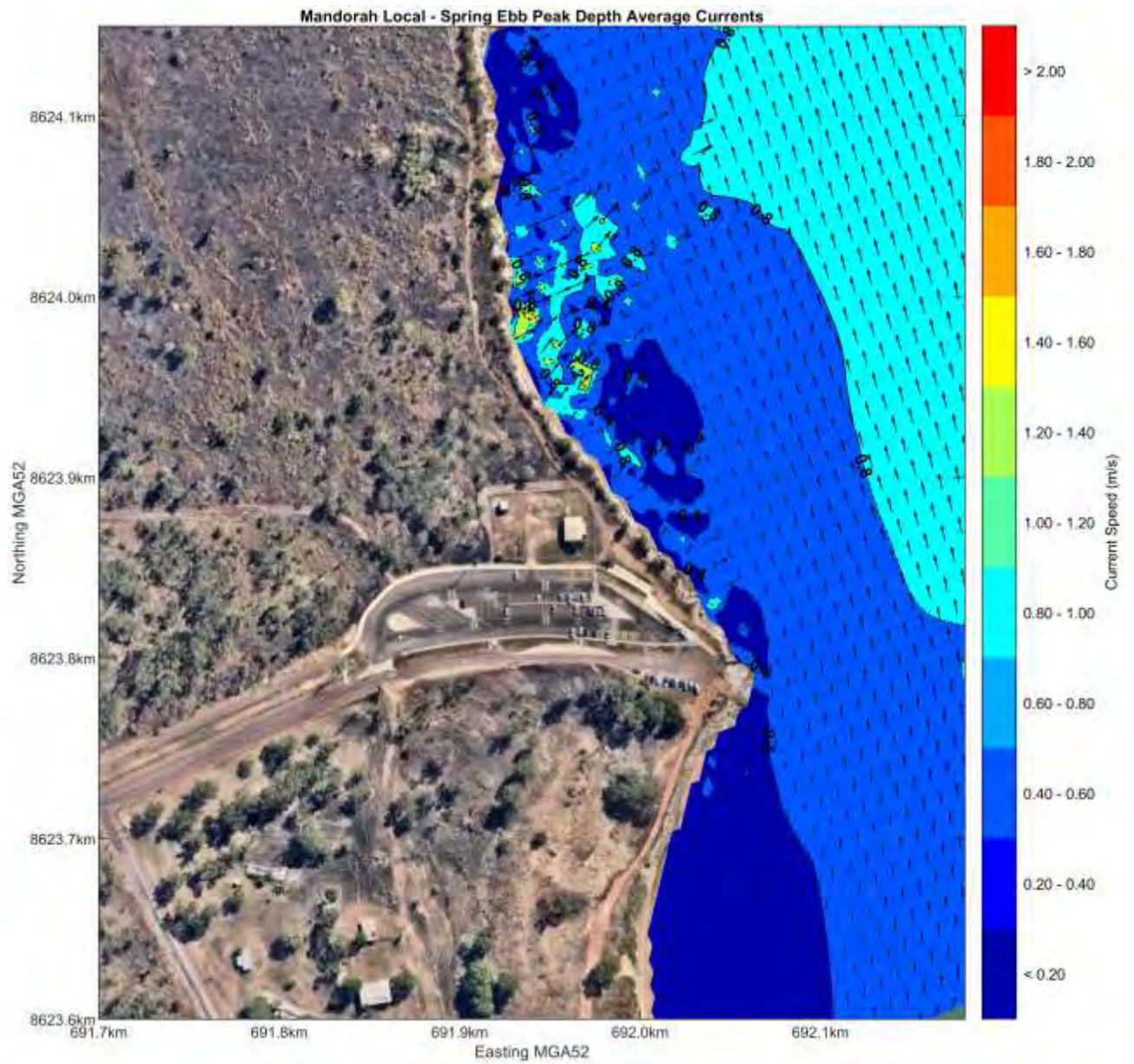
APPENDIX

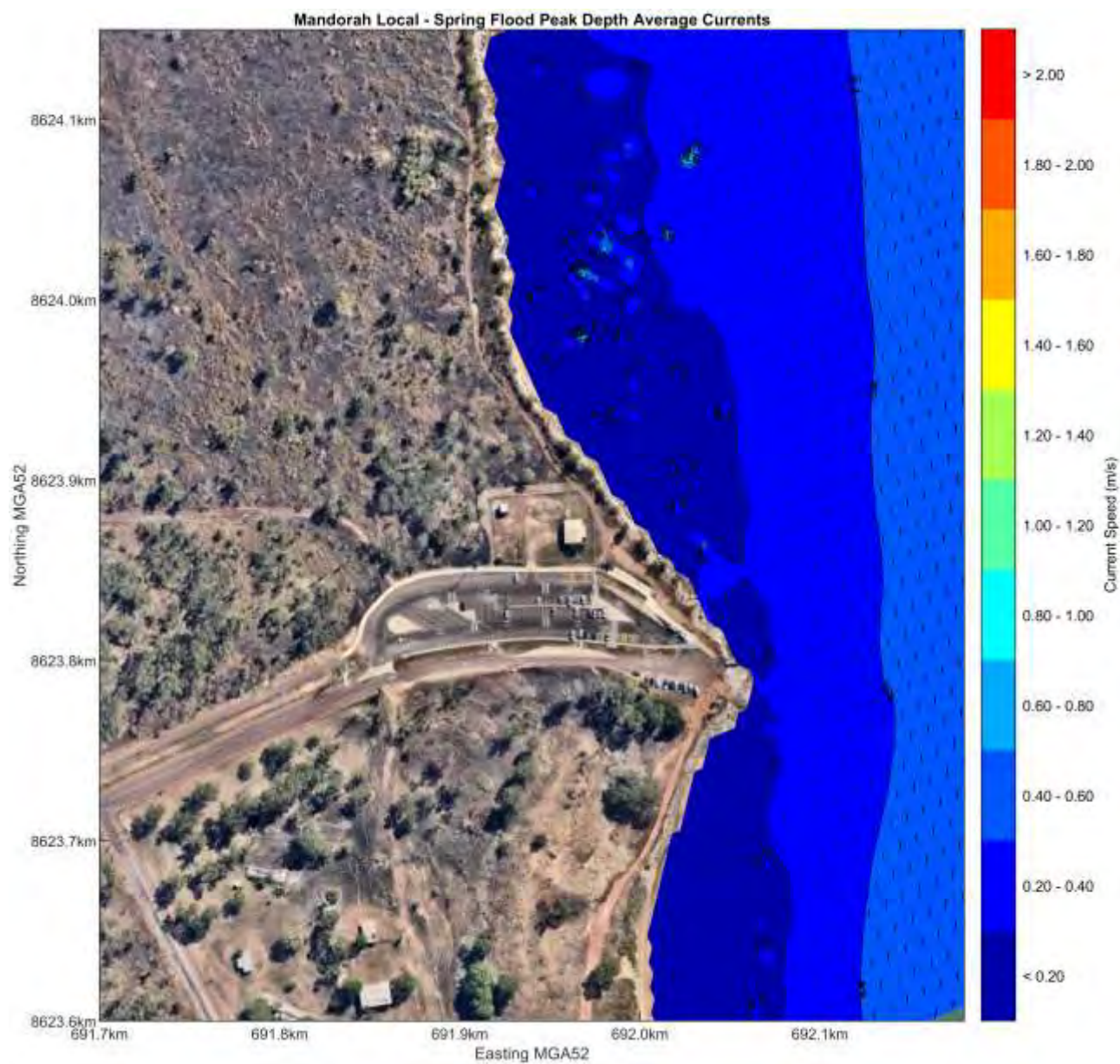
B

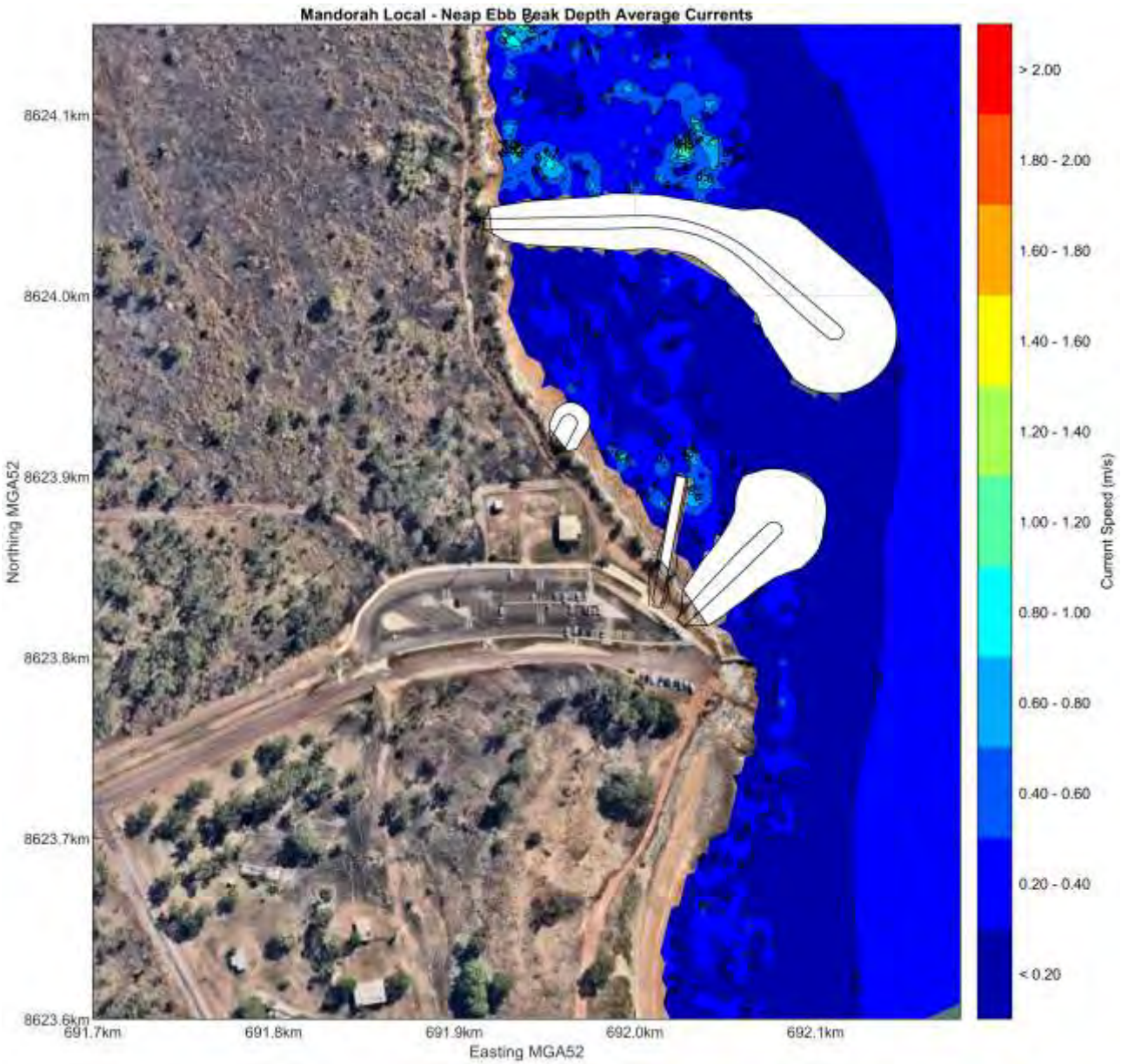
CURRENT VECTOR MAPS

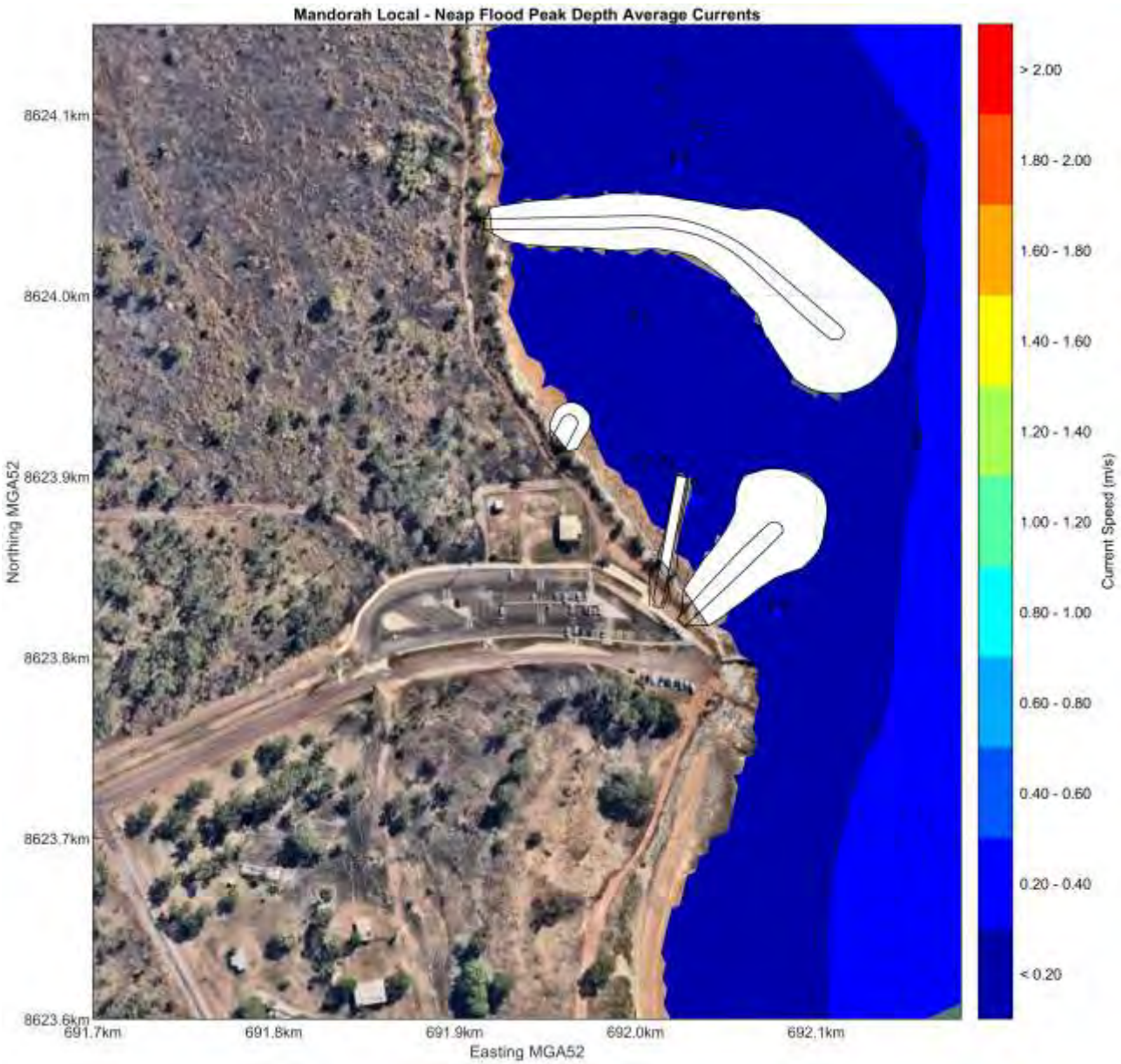


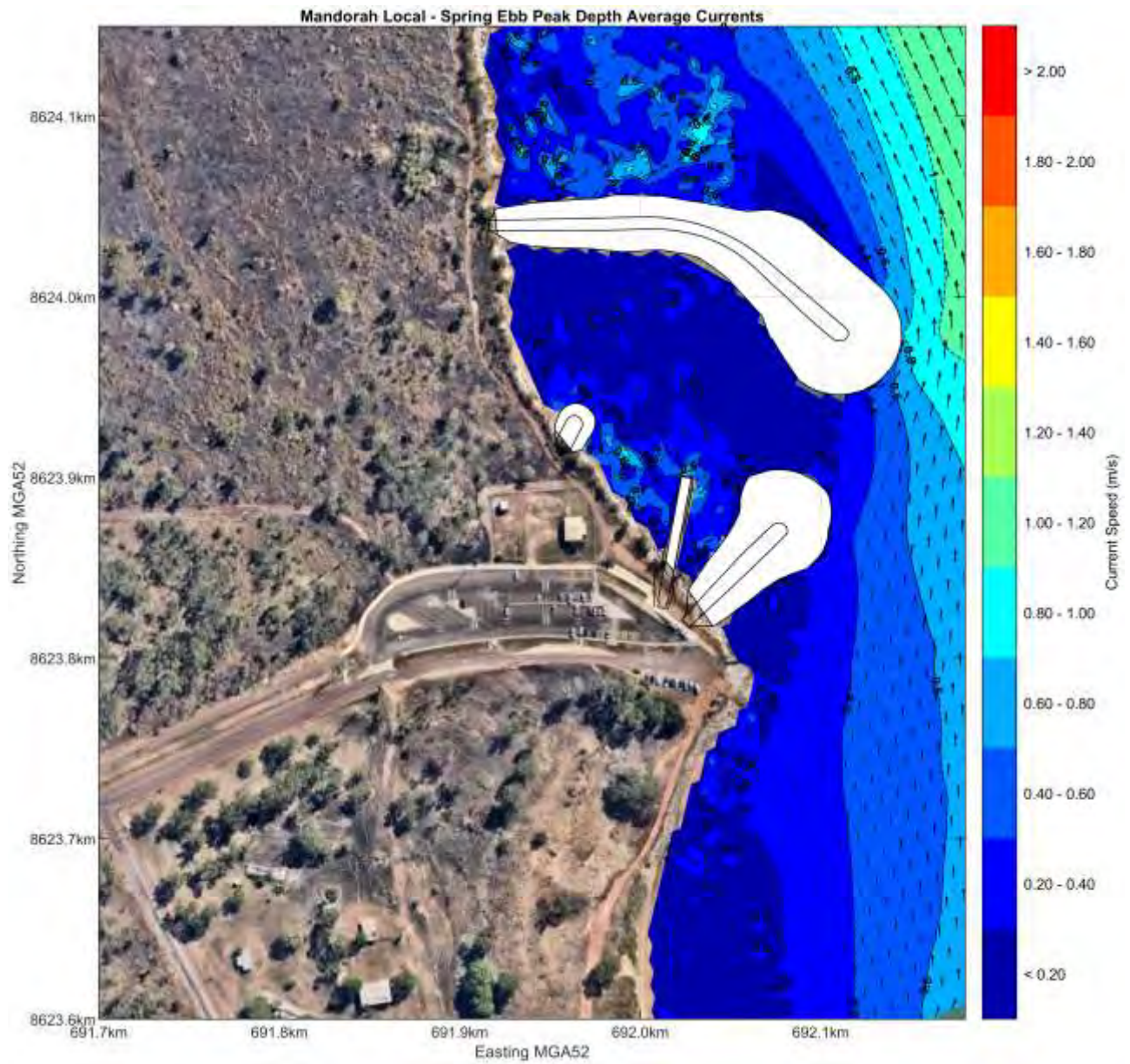


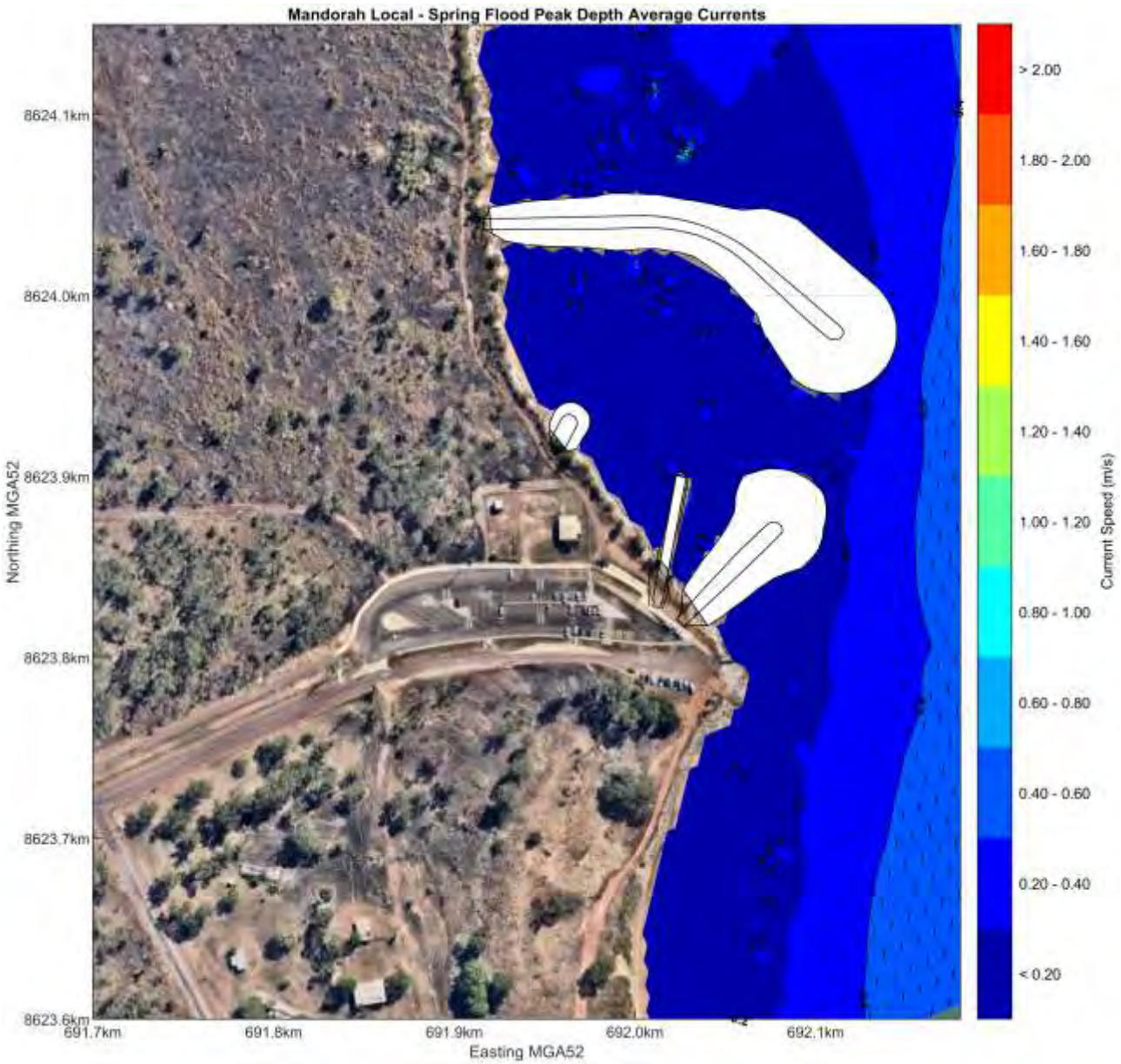












APPENDIX

C

PHYSICAL MODELLING REPORT

Mandorah Breakwater Physical Modelling

WRL TR 2022/03, May 2022

By F Flocard, J Chan, M Deiber and M Blacka



UNSW
Water Research
Laboratory



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SYDNEY



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SYDNEY

Mandorah Breakwater Physical Modelling

WRL TR 2022/03, May 2022

By F Flocard, J Chan, M Deiber and M Blacka

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Contents

1	Introduction	1
2	Study objectives	4
3	Design conditions	5
4	Model design, setup and operation	6
4.1	Testing facility	6
4.2	Model design and scaling	6
4.3	Flume setup	7
4.3.1	<i>Bathymetry</i>	7
4.3.2	<i>Breakwater armour material</i>	9
4.4	Data collection and analysis	10
4.4.1	<i>Wave data</i>	10
4.4.2	<i>Rock armour stability</i>	11
4.5	Breakwater model design and construction	12
4.5.1	<i>Stage 1 – 2D models</i>	12
4.5.2	<i>Stage 2 – Quasi 3D model</i>	15
5	Wave climate calibration	17
6	Stage 1: 2D testing	18
6.1	M5A 2D breakwater section testing	18
6.1.1	<i>Overview of M5A testing</i>	18
6.1.2	<i>Wave measurements for M5A testing</i>	19
6.1.3	<i>Wave transmission analysis for test section M5A</i>	20
6.1.4	<i>Rock armour damage assessment for test section M5A</i>	22
6.2	M5B 2D breakwater section testing	31
6.2.1	<i>Wave measurements for M5B testing</i>	32
6.2.2	<i>Wave transmission analysis for test section M5B</i>	33
6.2.3	<i>Rock armour damage assessment for test section M5B</i>	34
7	Stage 2 quasi 3D testing	41
7.1	Overview of quasi 3D testing	41
7.2	Wave measurements and transmission for quasi 3D testing	42
7.3	Rock armour damage assessment	43
8	Conclusions	52
9	References	54
Appendix A	Rock gradings	A-1
A1	2D Model - M5A	A-1
A2	2D Model - M5B	A-2
A3	Quasi 3D model	A-4

List of tables

Table 3.1 Target wave and water level conditions for physical modelling	5
Table 4.1 Model scale ratios	6
Table 5.1 Calibrated wave conditions for <i>Nearshore 3PA</i> (reflection removed) at -3.8 m AHD	17
Table 5.2 Calibrated wave conditions for <i>Offshore 3PA</i> (reflection removed) at -12.5 m AHD	17
Table 6.1 Measured wave statistics for Test Conditions 1 to 8, test section M5A	19
Table 6.2 Wave transmission for the five largest overtopping events of Test Condition 3	21
Table 6.3 Wave transmission for the five largest overtopping events of Test Condition 4	21
Table 6.4 Wave transmission for the five largest overtopping events of Test Condition 5	21
Table 6.5 Average wave transmission for the five largest overtopping events during Test Conditions 3, 4 and 5	22
Table 6.6 Test section M5A summary of damage assessment and overtopping type	23
Table 6.7 Summary of test conditions run for section M5B	31
Table 6.8 Measured wave statistics for testing of section M5B	32
Table 6.9 Wave transmission for the five largest overtopping events of Test Condition 5	33
Table 6.10 Test Section M5B summary of damage assessment and overtopping type	35
Table 7.1 Summary of test conditions ran for section M5B	42
Table 7.2 Measured wave statistics for quasi 3D testing	42
Table 7.3 Summary of armour damage for quasi 3D testing	44

List of figures

Figure 1.1 Project site location [Source: Google Earth]	1
Figure 1.2 Overview of the Mandorah marine facilities [Source: Cardno]	2
Figure 2.1 Breakwater plan view showing cross section locations	4
Figure 4.1 Mandorah facility layout with considered bathymetry transects	7
Figure 4.2 Mandorah bathymetric profiles (green) and WRL proposed 2D flume bathymetry model	8
Figure 4.3 Overview of the bathymetry built in the wave flume relative to project site	9
Figure 4.4 Armour layer 6 to 10 T rock grading curve	10
Figure 4.5 Armour layer 0.3 to 1 T rock grading curve	10
Figure 4.6 Layout plan of wave probes for wave climate calibration	11
Figure 4.7 Comparison between the M5A and modified M5B cross-sections	12
Figure 4.8 Construction sequence of the M5A cross-section	13
Figure 4.9 Construction sequence of the M5B cross-section	14
Figure 4.10 Modified M6 and M5/M6 transition section designs as tested	15
Figure 4.11 Q3D model layout	15
Figure 4.12 Construction sequence of the Q3D model	16
Figure 6.1 Layout of wave probes for Stage 1 testing (2D breakwater sections M5A and M5B)	18
Figure 6.2 Completed M5A cross-section model, including armour stone counts	22
Figure 6.3 Images from Test Condition 2 showing seaside (top) and leeside (bottom) armour	24
Figure 6.4 Images from Test Condition 3 showing seaside (top) and leeside (bottom) armour	25
Figure 6.5 Images from Test Condition 4 showing seaside (top) and leeside (bottom) armour	26
Figure 6.6 Images from Test Condition 5 showing seaside (top) and leeside (bottom) armour	27
Figure 6.7 Annotated plan view image following completion of Test Condition 5	28
Figure 6.8 Images from Test Condition 6 showing seaside (top) and leeside (bottom) armour	28
Figure 6.9 Images from Test Condition 7 showing seaside (top) and leeside (bottom) armour	29
Figure 6.10 Images from Test Condition 8 showing seaside (top) and leeside (bottom) armour	30
Figure 6.11 Images from additional test with 6 to 10 T primary armour on seaward toe	30
Figure 6.12 Comparison of transmitted wave conditions between breakwater M5A and M5B	34
Figure 6.13 Completed M5B cross-section model, including armour stone counts	35
Figure 6.14 Images from Test Condition 3 showing seaside (top) and leeside (bottom) armour	36
Figure 6.15 Images from Test Condition 4 showing seaside (top) and leeside (bottom) armour	37
Figure 6.16 Images from Test Condition 5 showing seaside (top) and leeside (bottom) armour	38
Figure 6.17 Images from Test Condition 6 showing seaside (top) and leeside (bottom) armour	39
Figure 6.18 Images from Test Condition 8 showing seaside (top) and leeside (bottom) armour	40
Figure 7.1 Layout of wave probes for Stage 2 testing (quasi 3D breakwater roundhead)	41
Figure 7.2 Quasi 3D model before (top) and after (bottom) Test Condition 1	45
Figure 7.3 Quasi 3D model before (top) and after (bottom) Test Condition 3	46
Figure 7.4 Quasi 3D model armour displacement during Test Condition 3	47
Figure 7.5 Quasi 3D model before (top) and after (bottom) Test Condition 5	48
Figure 7.6 Quasi 3D model armour displacement during Test Condition 5	49
Figure 7.7 Quasi 3D model before (top) and after (bottom) Test Condition 6	50
Figure 7.8 Quasi 3D model armour displacement during Test Condition 6	51

1 Introduction

Cardno has been contracted by the NT Department of Infrastructure, Planning and Logistics (DIPL) to deliver detailed design of new marine facilities at Mandorah, and is currently working on the design for breakwater structures to provide coastal protection of a new jetty and boat ramp. Mandorah is located on the Cox Peninsula on the western side of Darwin Harbour (Figure 1.1).



Figure 1.1 Project site location [Source: Google Earth]

The works to be completed within the package of new marine facilities at Mandorah are illustrated in Figure 1.2 and include:

- Establishment of a sheltered harbour through the construction of the new Main (northern) Breakwater and Lee (southern) Breakwater
- Dredging of a navigation channel and turning basin
- Establishment of a new jetty for vessel berthing
- Establishment of a new boat ramp

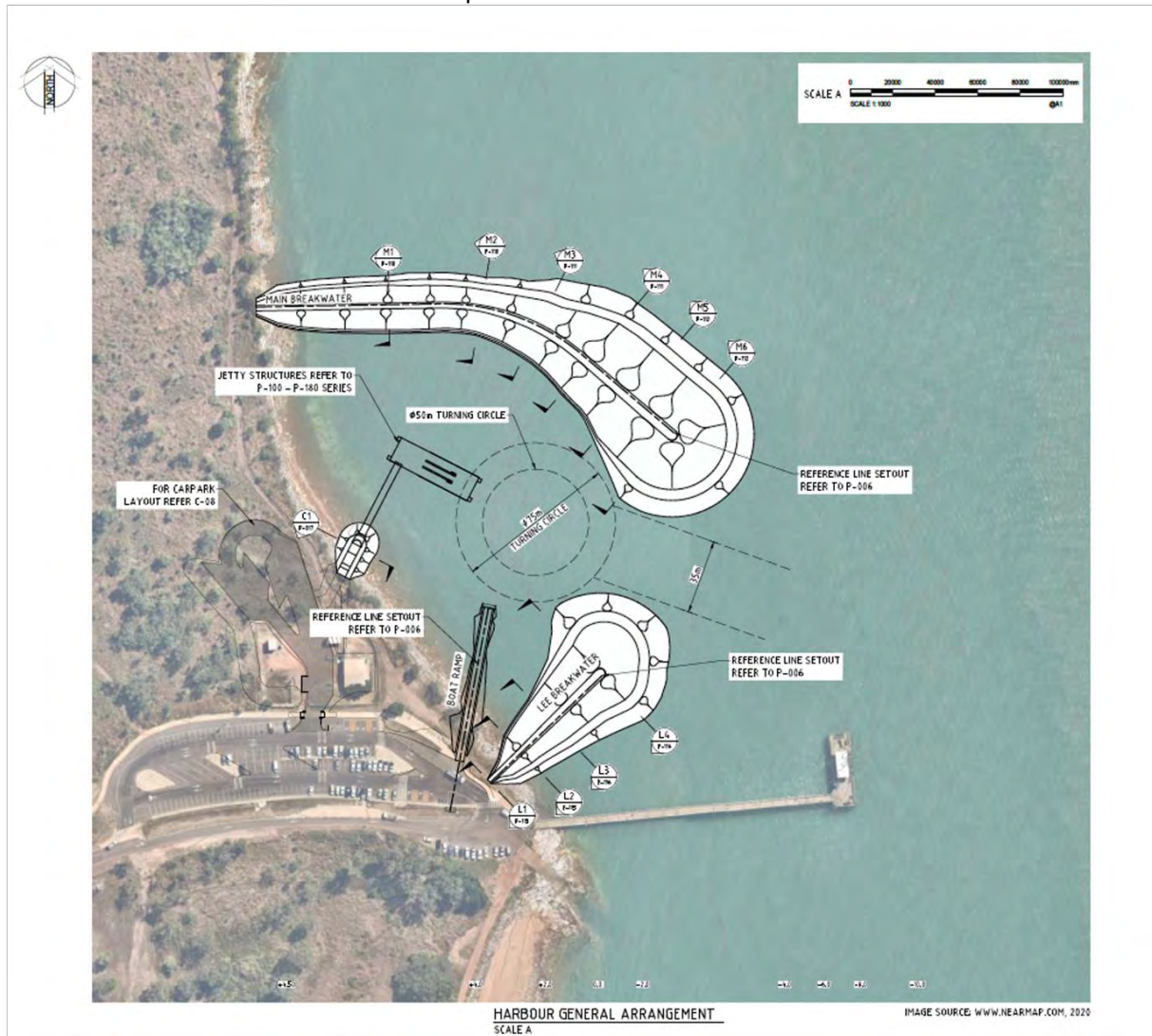


Figure 1.2 Overview of the Mandorah marine facilities [Source: Cardno]

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Cardno to complete a physical modelling investigation to support the design of the Main Breakwater.

This scale physical modelling, conducted as 2D and quasi-3D flume modelling, investigated a range of parameters for the new marine facilities development including:

- Nearshore wave processes
- Breakwater armour stability and cross-section design
- Wave transmission through overtopping and diffraction

All dimensions in this report are provided at prototype (real world) scale (unless specified as model scale). The vertical datum used in this report is the Australian Height Datum (AHD), and horizontal coordinates based on GDA94 MGA Zone 52.

2 Study objectives

The physical modelling program was divided into two stages:

- Stage 1: 2D modelling of the Main Breakwater trunk
- Stage 2: Quasi 3D modelling of the Main Breakwater head

The primary objective of Stage 1 was to verify and optimise the cross-section design for the trunk of the Main Breakwater. This included the assessment of rock armour stability for the primary armour on both seaward and leeward sides of the trunk, as well as the toe armour, under wave attack and overtopping. Wave energy transmission to the leeside of the breakwater trunk due to overtopping of the breakwater and wave transmission through the porous layers of rock armour was also assessed during Stage 1. Two different cross-sections were tested within the 2D modelling, allowing armour size optimisation and increased overall armour stability. The first 2D cross-section is referred to as M5A in this report, while the second cross-section is referred to as M5B. Both sections were representative of the outer zone of the Main Breakwater trunk at location M5 as shown in Figure 2.1.

The main objective of Stage 2 was to verify overall primary armour and toe armour stability around the Main Breakwater head. This model explored the 3D interaction of wave processes (such as wave breaking and diffraction) and armour around the breakwater head, as well as the potential effects of a localised lower seabed from the planned dredging arrangement for the approach navigation channel.

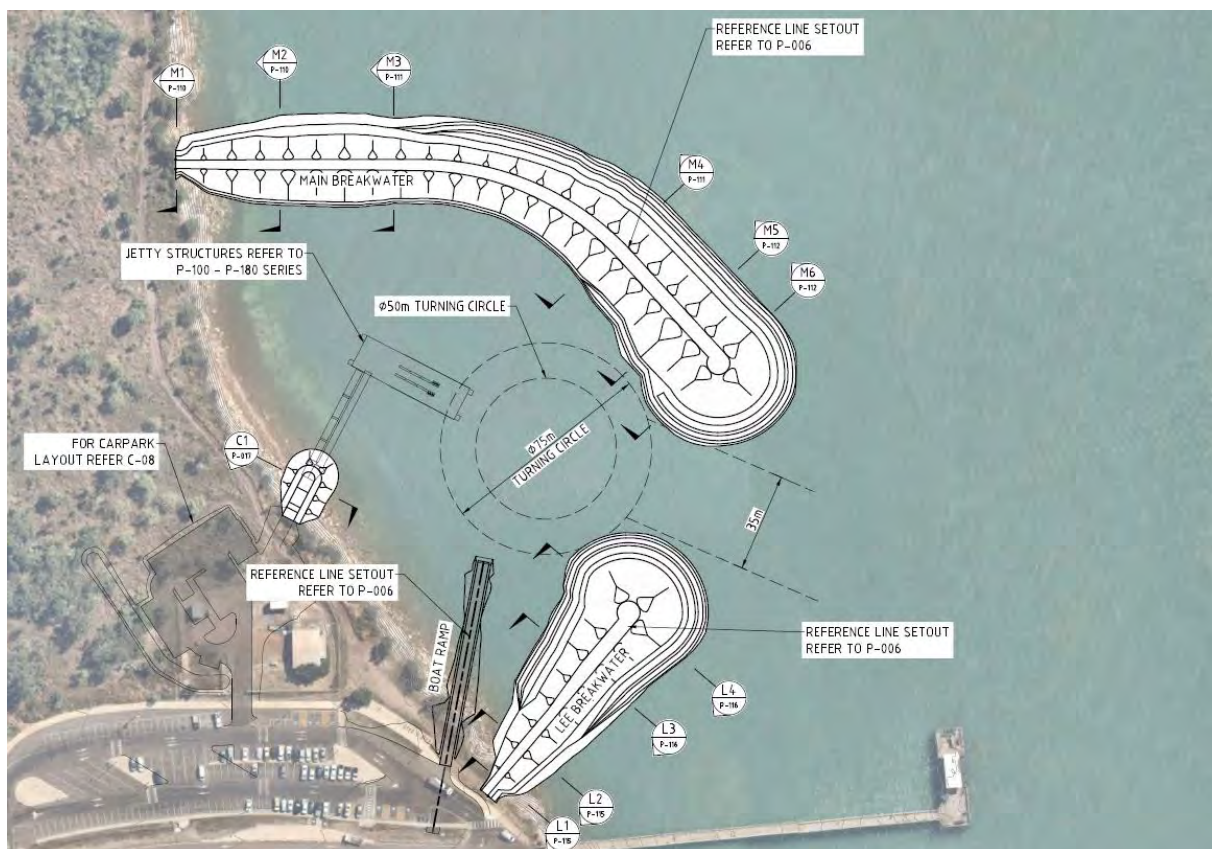


Figure 2.1 Breakwater plan view showing cross section locations

3 Design conditions

Cardno provided a range of target wave and water level conditions to be simulated in the physical modelling investigation, on the basis of their numerical wave modelling for the site. Each of these target test conditions were closely calibrated within the wave flume by WRL. The numerical wave model output location considered for the target test conditions was located close to the toe of the Main Breakwater at a seabed level of -3.8 m AHD (referred to herein as “nearshore” conditions). The target wave and water level conditions at the nearshore location are summarised in Table 3.1, with test events ranging from 1 year average recurrence interval (ARI) through to 500 year ARI, with and without an allowance for future sea level rise (SLR). Several low water level (LWL) test conditions were also explored, as some aspects of breakwater design can be more sensitive under these conditions (such as toe armour stability).

For the remainder of this report, specific test conditions are referred to by their “Test Condition” reference number as shown in Table 3.1.

Table 3.1 Target wave and water level conditions for physical modelling

Test Condition	Wave Condition	Water Level Condition	H _{m0} (m)	T _p (s)	WL (m AHD)
1	1 yr ARI	1 yr ARI + SLR	1.86	6.5	3.44
2	50 yr ARI	10 yr ARI + SLR	2.79	8.4	4.07
3	100 yr ARI	25 yr ARI + SLR	2.98	8.8	4.46
4	200 yr ARI	50 yr ARI + SLR	3.15	9.2	4.75
5	500 yr ARI	100 yr ARI + SLR	3.36	9.6	5.05
6	100 yr ARI	LWL no SLR	2.98	8.8	2.50
7	200 yr ARI	LWL no SLR	3.15	9.2	2.75
8	500 yr ARI	LWL no SLR	3.36	9.6	3.00

Notes:

H_{m0} = Significant wave height, determined as 4 x the square root of the zeroth-order moment of the wave spectrum

T_p = Spectral peak wave period

4 Model design, setup and operation

4.1 Testing facility

All testing carried out in this study was completed in WRL's 3 m wide wave flume. This wave flume measures 32 m in length, 3.0 m in width and 1.3 m in depth. The flume walls are primarily constructed of rendered and painted brick. A bathymetry was fabricated from a combination of waterproof plywood sheets over a treated pine sub-frame, and an aggregate filled base that was capped with fibre-reinforced concrete.

For the Stage 1 2D testing component of the project, breakwater test sections were positioned within an internal mini flume that had a width of 1 m, and was centred within the main flume. This restricted model test sections to a width of 1 m (standard for 2D breakwater testing), compared with using the full 3 m width of the flume, so as to ensure efficient model construction and analysis. The use of the mini flume also assists with dissipation of unwanted reflected wave energy within the flume. For the Stage 2 Quasi 3D breakwater roundhead component of the testing, the full 3 m width of flume was utilised.

The flume has a paddle type wave generator powered by a 30 kW hydraulic piston system. This system is capable of generating both monochromatic and irregular wave spectra. Representative wave sequences were generated by WRL's bespoke wave modelling software and the generated wave signal was fed to the wave paddle via the National Instruments LabVIEW software package.

Wave data was collected at a range of locations in the flume using capacitance wave probes. High frequency water level data from the wave probes was recorded on a PC using the National Instruments LabVIEW software package, and the time series of water level data was post processed to produce wave statistics for each location using the Mathworks MATLAB software package.

4.2 Model design and scaling

Model scaling was based on geometric similarity between the prototype (real world) and the model. An undistorted length scale of 1:28 was selected after consideration of the model design constraints (such as available flume space, wave maker capacity and potential scale effects). The scaling relationship between length and time was determined by Froudian similitude, with the relevant scale ratios (prototype divided by model) being adopted for the model as shown in Table 4.1.

Table 4.1 Model scale ratios

Parameter		Unit	Scaling ratio (prototype value / model value)
Length ratio	(L_r)	m	28.00
Time ratio	$L_r^{0.5}$	s	5.29
Velocity ratio	$L_r^{0.5}$	m/s	5.29
Volume ratio	L_r^3	l	21,952.00
Overtopping rate ratio	$L_r^{3/2}$	l/s/m	148.16
Mass ratio	L_r^3	kg	21,952.00
Adjusted rock mass ratio	*	kg	31,521.20

*The scaling ratio of rock mass was determined using the Hudson method (Hughes, 1993) to account for the differences between prototype (seawater) and model (freshwater) water densities, and prototype (2500 kg/m^3) and model (2650 kg/m^3) rock densities.

Design of all aspects of the model including selection of the model scale was completed in accordance with best practice physical modelling guidance provided in:

- Hydralab III (2007a), Guidelines for wave modelling in flumes and basins: Hydraulic model testing in waves, Deliverable NA3.1-1, EC contract no. 022441 (RII3)
- Hydralab III (2007b), Guidelines for physical model testing of breakwaters: Rubble mound breakwaters, Deliverable NA3.1-2, EC contract no. 022441 (RII3)

4.3 Flume setup

4.3.1 Bathymetry

A 2D bathymetric profile was constructed to scale in the wave flume using surveyed bathymetric data provided by Cardno (20211011_Mandorah_Bathymetry.XYZ). To establish a suitable model bathymetry design, an initial review of the bathymetry data was performed by extracting multiple profiles normal to the breakwater sections of interest (i.e. Main Breakwater head and trunk) and originating from the reference points 8 to 12 (Figure 4.1). The analysis of the bathymetry profiles was performed over a distance of 500 m north-east of the breakwater toe (Figure 4.2), extending greater than three wave lengths seaward of the breakwater. This is the recommended minimum bathymetry length to be included within breakwater physical models in accordance with physical modelling guidelines (Hydralab III, 2007b), so as to ensure accurate simulation of wave processes in the nearshore zone.



Figure 4.1 Mandorah facility layout with considered bathymetry transects

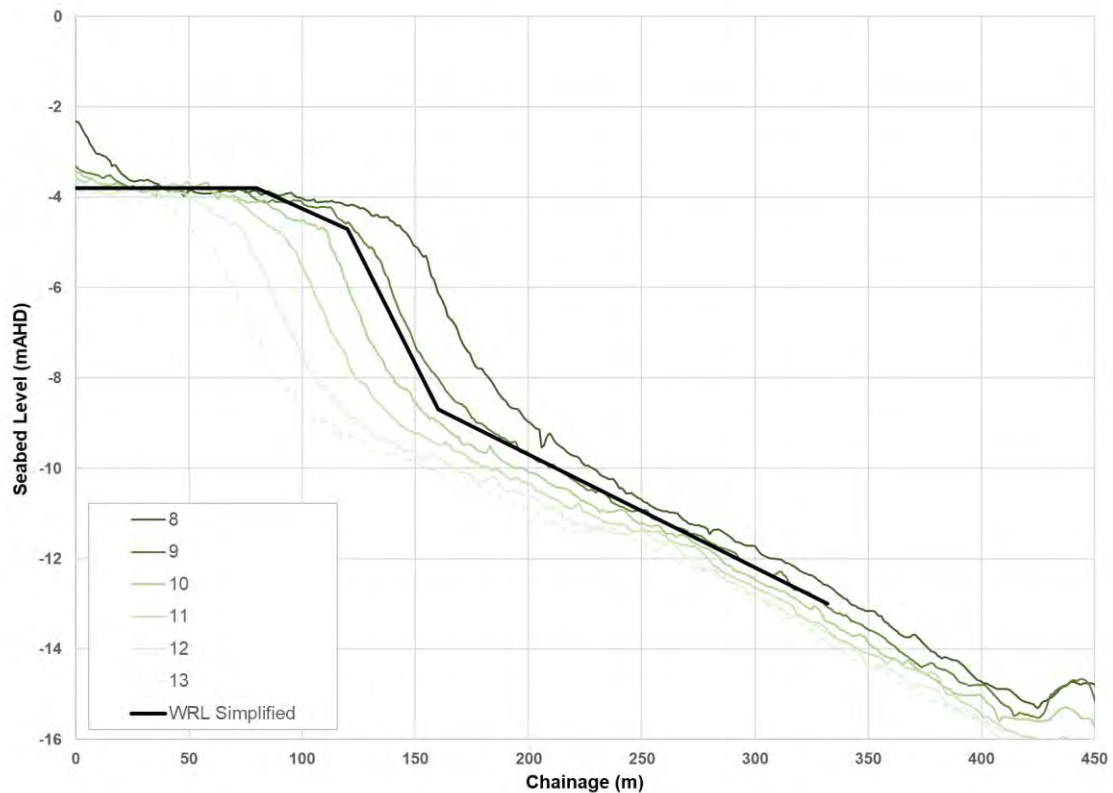


Figure 4.2 Mandorah bathymetric profiles (green) and WRL proposed 2D flume bathymetry model

The reviewed bathymetric profile data (Figure 4.2) showed that the offshore bathymetry could be simplified and modelled as:

- initially flat over a distance of 80 m (ch. 0 to 80 m) at -3.8 m AHD
- dropping at a slope of 1V:40H (ch. 80 to 120 m) between -3.8 m AHD and -4.7 m AHD
- dropping at a slope of 1V:10H (ch. 120 to 160 m) between -4.7 m AHD and -8.7 m AHD
- dropping at a slope of 1V:44H (ch. 160 to 330 m) between -8.7 m AHD and -13 m AHD

Beyond chainage 330 m offshore from the toe of the breakwater, the flume bathymetry was extended with a 1V:10H slope to transition down to the fixed concrete floor of the flume (at a depth of -22 m AHD), approximately 5 m in front of the wave paddle as per physical modelling guidelines (Hydralab III, 2007b).

The 2D model bathymetry was designed with an extended nearshore horizontal section (-3.8 m AHD) to also allow modelling of sections of the breakwater trunk which are fronted by a wider flat section of seabed.

Figure 4.3 shows the position of the wave flume relative to the site, as well as the layout of the modelled bathymetry. A 3 m long dissipative beach was fitted across the back wall of the flume to minimise wave reflections during wave climate calibration and testing.



Figure 4.3 Overview of the bathymetry built in the wave flume relative to project site

4.3.2 Breakwater armour material

Model rocks for the underlayer, toe armour and primary armour were sorted based on mass distributions specified by Cardno and adjusted between the different test series. The scaling relationships for armour mass considered the differences in prototype and model densities due to the use of fresh water in the model and were determined using the technique of Hudson et al. (1979) as recommended by Hydralab (2007b).

Four different design rock armour gradings were provided by Cardno and specified as the target armour stone mass ranges for each grading, namely 6 to 10 T, 1 to 3 T, 0.3 to 1 T, and 60-300 kg. Based on these specifications, WRL obtained target mean armour stone mass values (M_{50}) for each grading by following the Rosin-Rammler distribution in *The Rock Manual* (CIRIA, CUR, CETMEF, 2007).

Armour material for each target grading was sorted by WRL using a specialist computer-controlled rock sorting machine. Examples of achieved model armour distribution curves for the two primary armour rock classes used on the Stage 2 Quasi-3D model are presented in Figure 4.4 and Figure 4.5. All rock grading distribution curves (mass or diameters) that were used within the modelling program are provided in Appendix A.

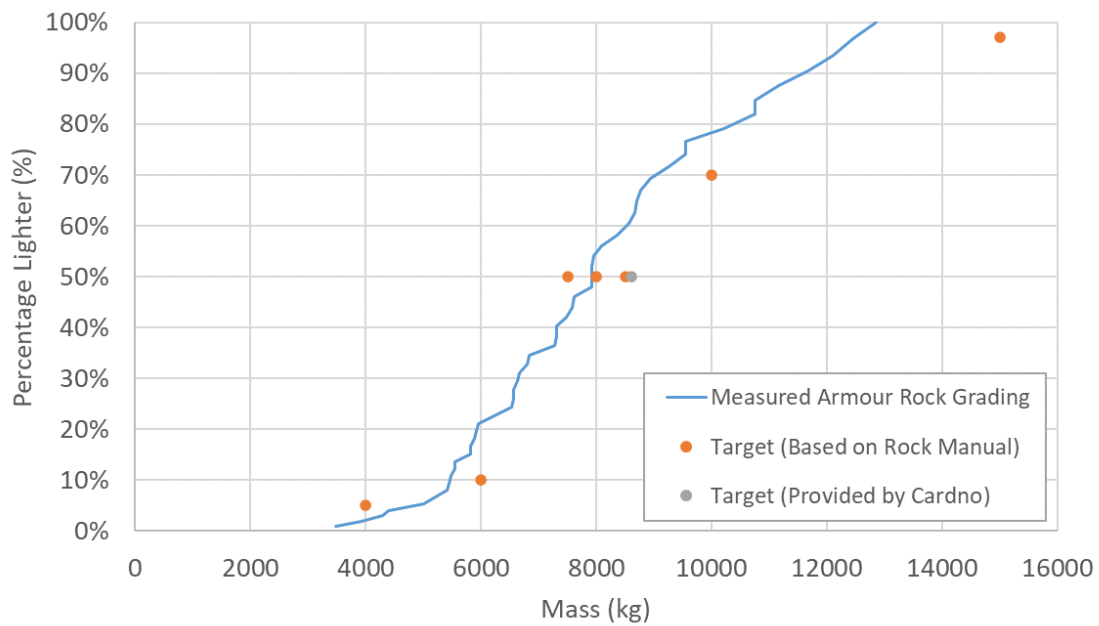


Figure 4.4 Armour layer 6 to 10 T rock grading curve

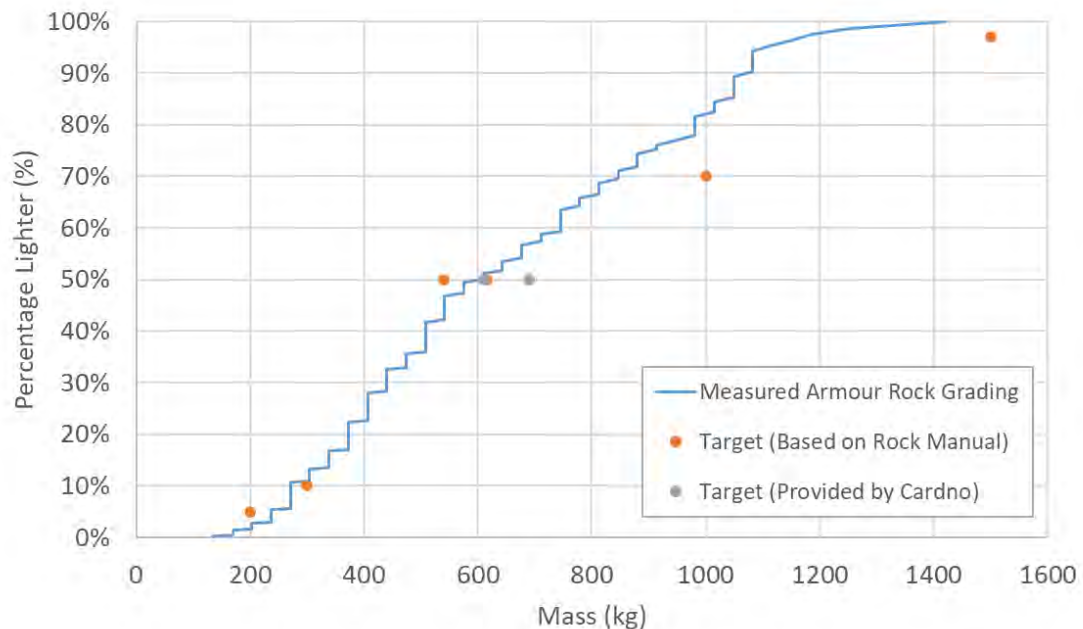


Figure 4.5 Armour layer 0.3 to 1 T rock grading curve

4.4 Data collection and analysis

4.4.1 Wave data

During each test, water surface elevation data was recorded at various locations within the flume using capacitance type wave probes. This data was logged at high frequency (200 Hz) using the National Instruments LabVIEW software package installed on a laboratory PC. Based on the time series of water surface elevations recorded by each wave probe, zero crossing and spectral analysis of the incident waves was able to be completed using the Mathworks MATLAB software package, which allowed calculation of the statistical wave distribution at each location.

Each target wave and water level condition (Table 3.1) was initially calibrated within the wave flume once the 2D model bathymetry was fabricated, but prior to construction of the model breakwater. The calibration was performed with a total of six wave probes, installed in two separate 3-probe arrays (3PA) along the centre line of the flume. These 3-probe arrays were used to separate the incident waves from the reflected waves (off the wave flume back wall and dissipative beach) using the Mansard and Funke (1980) method.

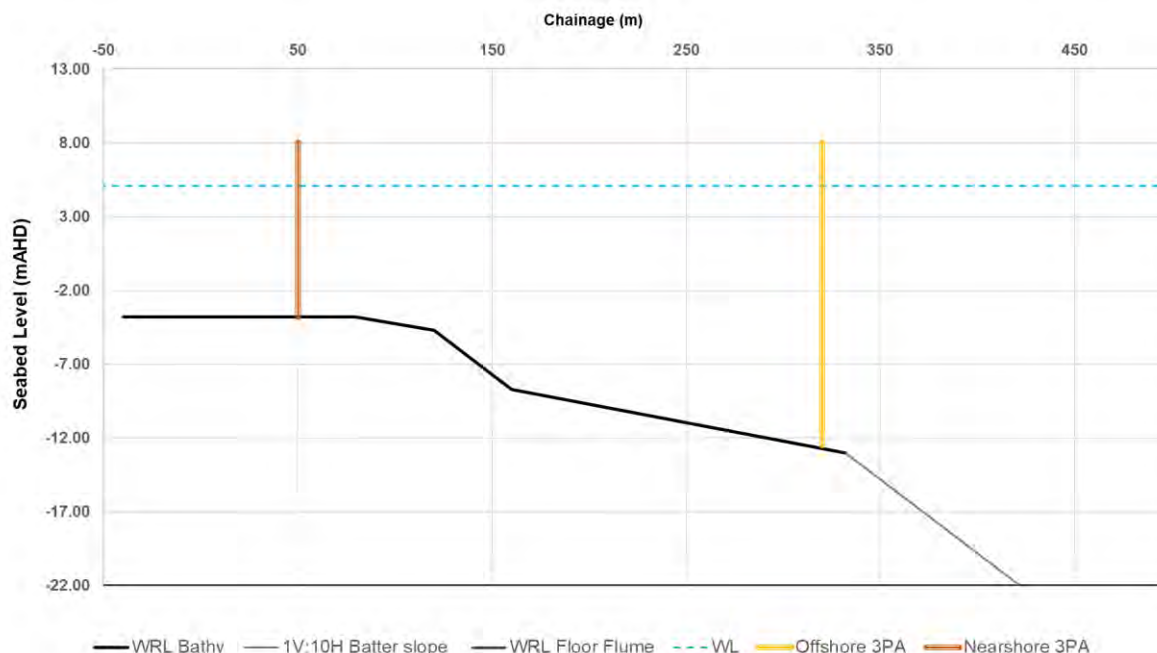


**Figure 4.6 Layout plan of wave probes for wave climate calibration
(indicative water level shown, which varied across the different target test conditions)**

The nearshore 3 probe array (“*Nearshore 3PA*”) had its seaward probe located above the flat portion of the bathymetry at -3.8 m AHD, and was positioned 30 m landward from the last bathymetric slope change. The *Nearshore 3PA* array was used for the wave climate calibration, with the recorded wave height parameter H_{m0} at the *Nearshore 3PA* used to adjust the wave amplitude to match the target wave conditions provided by Cardno. The results of the wave climate calibration process are presented in Section 5 of the report.

The Offshore 3 probe array (“*Offshore 3PA*”) had its seaward probe located above the -12.5 m AHD seabed contour (Figure 4.6). The Offshore array was predominantly used for comparison purposes and to assess wave breaking processes across the flume bathymetry.

The wave paddle drive signals and associated wave time series were generated using a JONSWAP 3.3 spectral shape and duration of 3 hours at prototype scale (Hasselmann et. al., 1973).

4.4.2 Rock armour stability

In accordance with the *Coastal Engineering Manual* (U.S. Army Corps of Engineers, 2006) and *The Rock Manual* (CIRIA, CUR, CETMEF, 2007), displacement of individual rocks was measured by initially placing and counting the units in different coloured bands running across the face of the breakwater, then counting the number that were displaced from each section of the structure throughout the test. This count was completed using a combination of photographic, video and visual observations

taken before, during and after each test. Armour damage was determined by expressing the displaced stones/units as a percentage of the total number in the armour layer.

4.5 Breakwater model design and construction

4.5.1 Stage 1 – 2D models

The two different breakwater designs tested in Stage 1 of the modelling program were constructed to the design drawings and specifications as provided by Cardno. Key details for each of the 2D cross-section designs (M5A and M5B), are provided in Figure 4.7. The main changes that were made between M5A and M5B included:

- Reduction in crest level from 8.8 to 7.7 m AHD and increase in crest width
- Reduction in seaward slope primary armour mass from 10 to 15 T down to 6 to 10 T
- Increase in leeward slope primary armour mass from 0.3 to 1 T up to 1 to 3 T

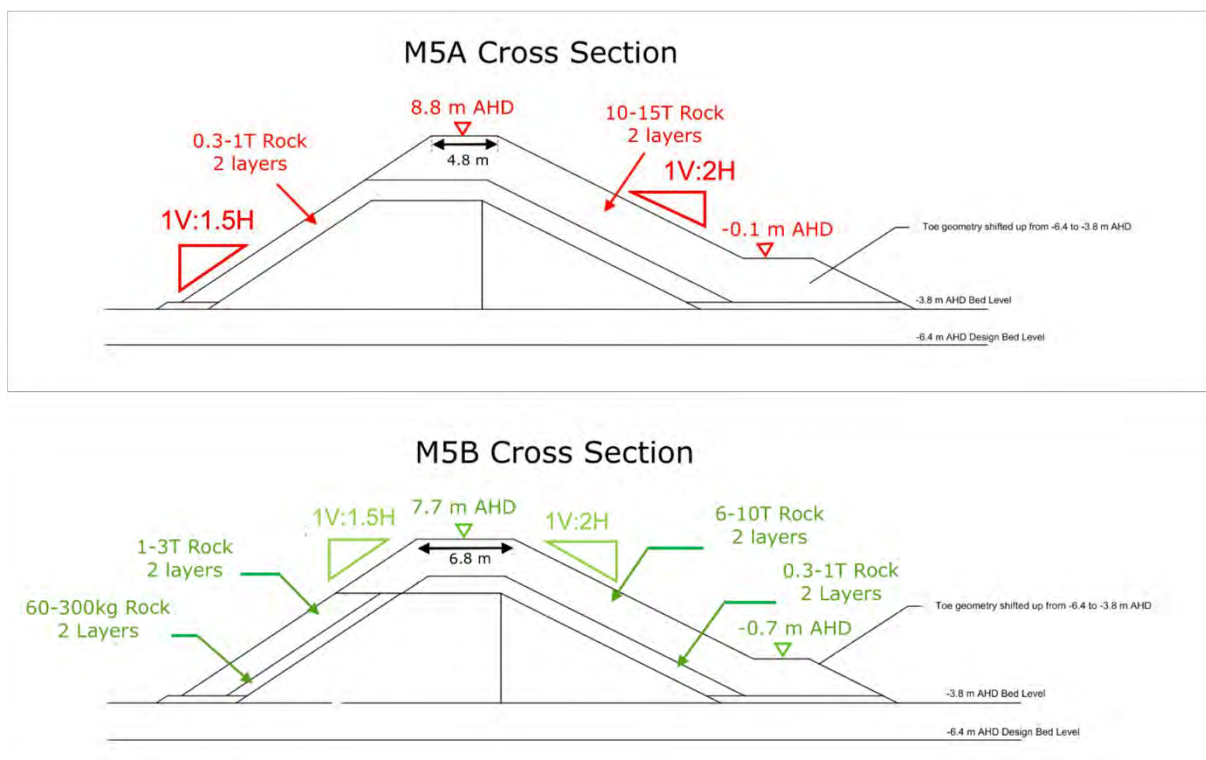
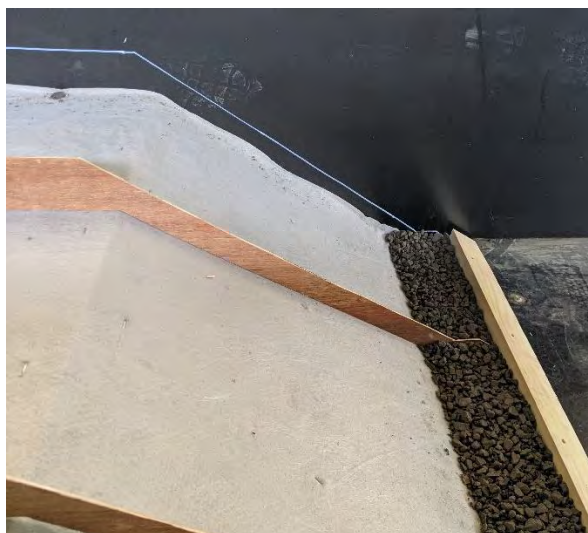


Figure 4.7 Comparison between the M5A and modified M5B cross-sections

The proposed breakwater core is largely impermeable. As such the core for the 2D models was constructed with an impermeable hollow timber box. The surface of the box was covered with geotextile material to provide a more representative roughness interface between the core and the underlayer. An image sequence showing construction of the M5A breakwater section is provided in Figure 4.8. An image sequence showing construction of the M5B breakwater section is provided in Figure 4.9.



(a) Core box lined with geotextile



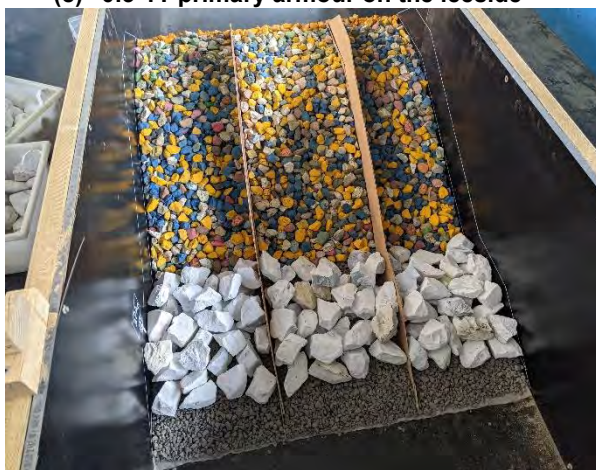
(b) Quarry run on the leeside toe



(c) 0.3-1T primary armour on the leeside



(d) 0.3-1T secondary armour on the seaside



(e) 10-15T primary armour on the seaside



(f) 10-15T primary armour on the crest

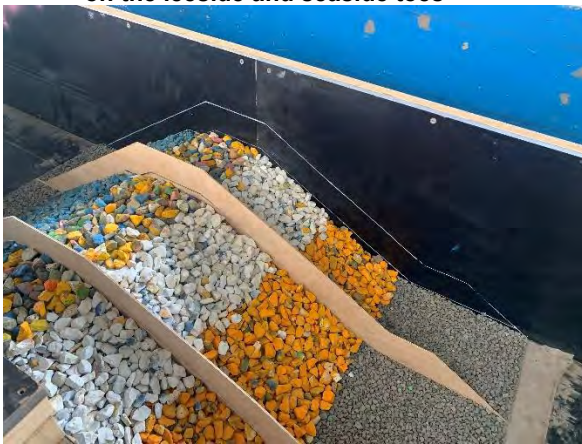
Figure 4.8 Construction sequence of the M5A cross-section



(g) Core box lined with geotextile and quarry run on the leeside and seaside toes



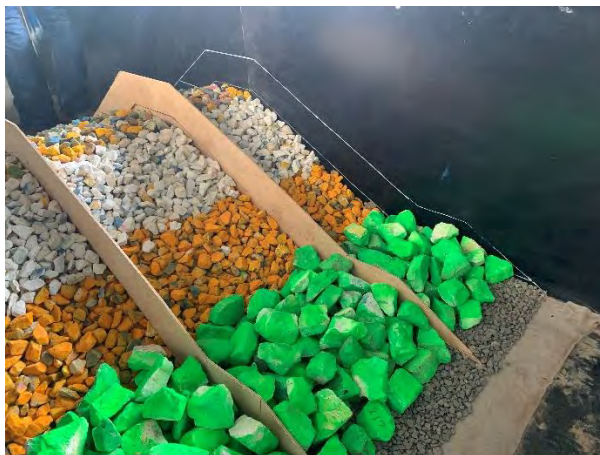
(h) 60-300kg leeside secondary armour (blue) and 0.3-1T seaside secondary armour (yellow/white)



(i) Secondary armour completed with primary armour templates in position



(j) 1-3T leeside primary armour (white/yellow)



(k) 6-10T primary armour on the seaside toe



(l) 6-10T primary armour completed

Figure 4.9 Construction sequence of the M5B cross-section

4.5.2 Stage 2 – Quasi 3D model

The quasi 3D Main Breakwater head model included the roundhead and most of the trunk spanning between sections M5 and M6 as show on Figure 2.1. The M6 and M5/M6 transition section designs upon which the quasi 3D model was based are shown in Figure 4.10. The arrangement of the quasi 3D head model within the flume is shown in Figure 4.11.

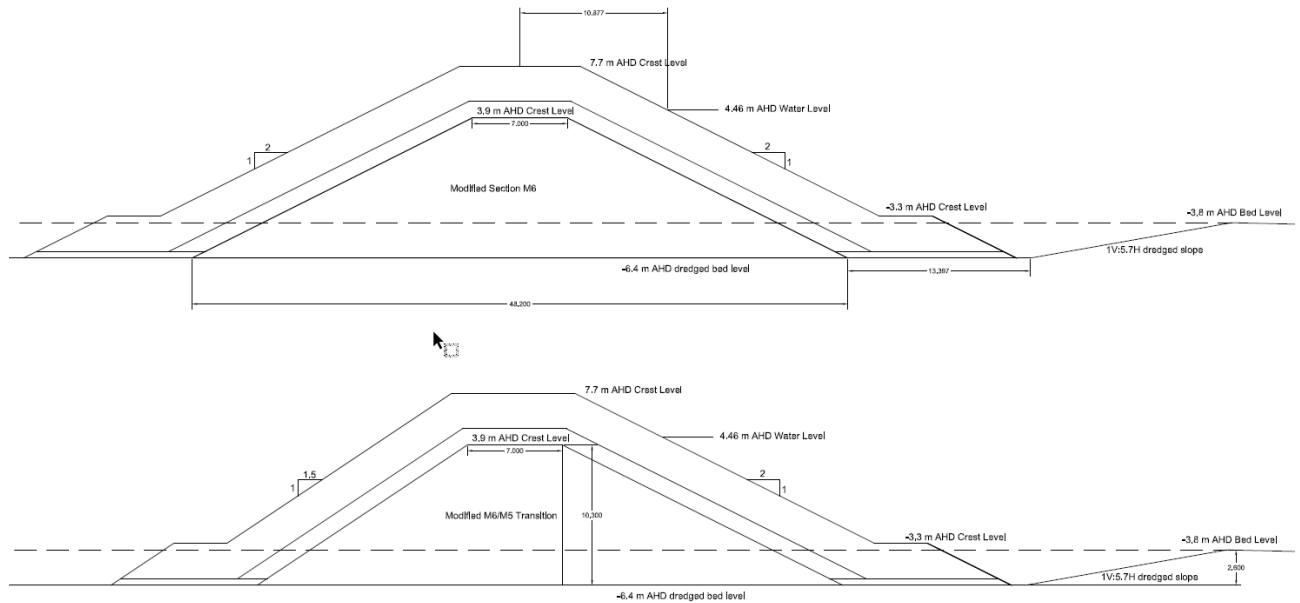
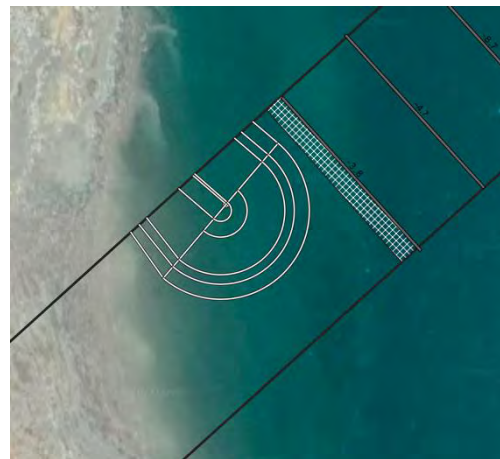


Figure 4.10 Modified M6 and M5/M6 transition section designs as tested



a. Q3D model position in flume



b. Plan View of the Tested Section

Figure 4.11 Q3D model layout

The breakwater roundhead was constructed on a flat surface at -6.4 m AHD corresponding to the design dredged depth of the navigation channel and breakwater head footprint. The top of the toe was situated at -3.3 m AHD and the crest at +7.7 m AHD. The primary armour layer was constructed from 6 to 10 T rocks with an underlayer of 0.3 to 1 T rocks. The seaward and roundhead slopes were 1V:2H and transitioned to 1V:1.5H slope at the flume wall on the leeward side.

The core of the breakwater roundhead model was made of concrete and covered by a layer of geotextile to increase the friction on the slope. The underlayer was placed on top of the geotextile. Two rows and two layers of 15 T rocks were placed on top of the underlayer along the flume's wall, so as to minimise boundary effects on the model armour stability analysis. Finally, the armour layer was built on top of the underlayer and divided into eight zones to help identifying and reporting damage. Photos of the breakwater roundhead construction are shown in Figure 4.12.

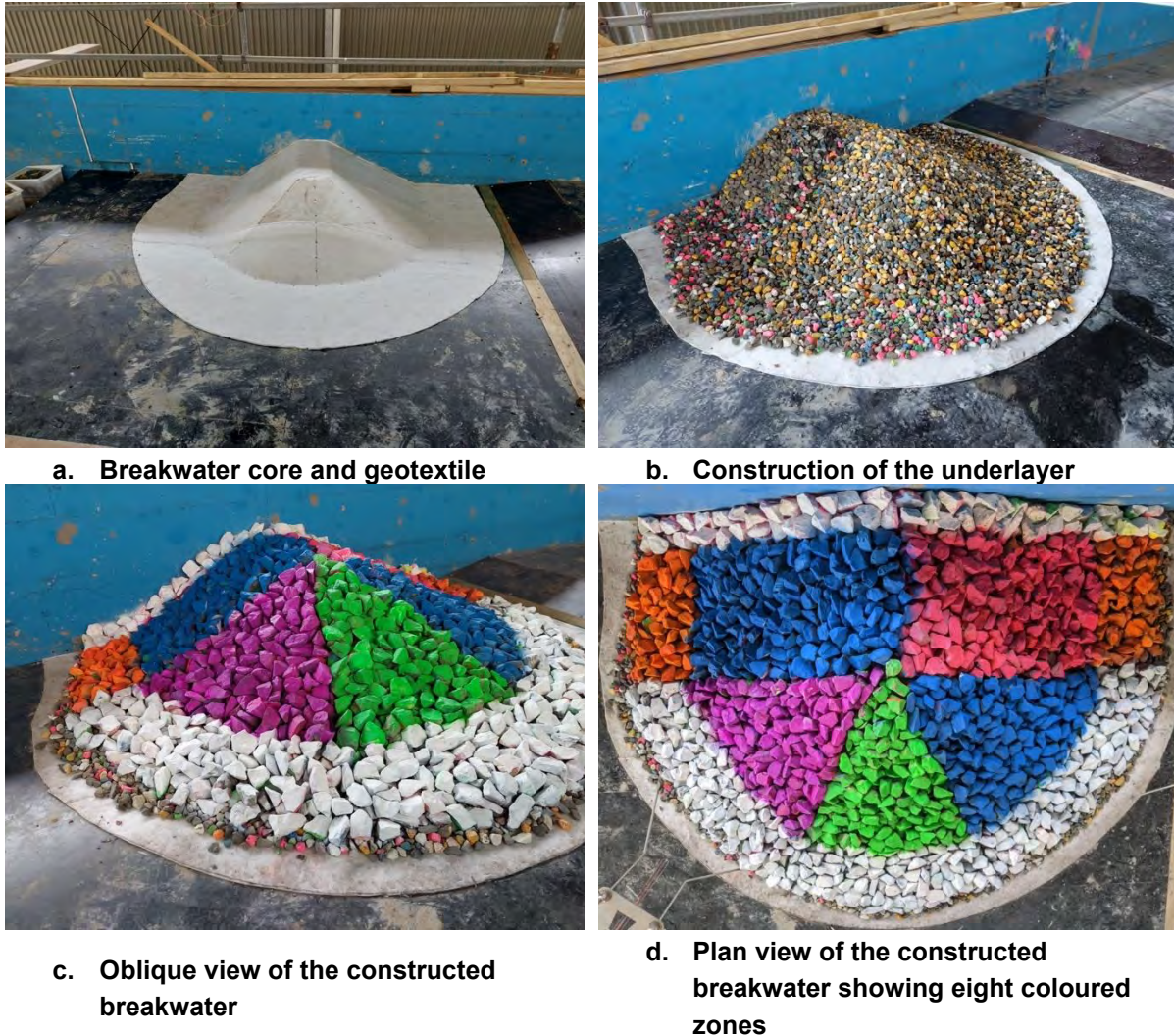


Figure 4.12 Construction sequence of the Q3D model

5 Wave climate calibration

A summary of measured wave statistics for all eight calibrated wave/water level scenarios are presented in Table 5.1 (*Nearshore 3PA*) and Table 5.2 (*Offshore 3PA*) at prototype scale. Location of the wave probes relative to the 2D bathymetry was provided in Figure 4.6.

Table 5.1 Calibrated wave conditions for *Nearshore 3PA* (reflection removed) at -3.8 m AHD

Test Condition	H _{m0} (m)	T _p (s)	H _{1/3} (m)	T _z (s)	H _{mean} (m)	H _{10%} (m)	H _{5%} (m)	H _{2%} (m)	H _{1%} (m)	H _{max} (m)
1	1.87	6.74	1.82	5.71	1.16	2.31	2.55	2.80	2.94	3.33
2	2.84	8.50	2.76	7.09	1.80	3.29	3.52	3.75	3.88	4.50
3	3.03	8.87	2.95	7.20	1.87	3.52	3.75	4.02	4.22	5.13
4	3.23	9.26	3.16	7.58	2.05	3.76	4.00	4.23	4.40	5.03
5	3.40	9.42	3.31	7.44	2.10	4.01	4.27	4.55	4.73	5.42
6	2.97	8.81	2.84	7.02	1.94	3.25	3.42	3.61	3.72	4.09
7	3.21	9.32	3.04	7.11	2.09	3.42	3.59	3.75	3.86	4.16
8	3.47	9.65	3.25	7.27	2.25	3.70	3.88	4.08	4.20	4.79

Table 5.2 Calibrated wave conditions for *Offshore 3PA* (reflection removed) at -12.5 m AHD

Test seq #	H _{m0} (m)	T _p (s)	H _{1/3} (m)	T _z (s)	H _{mean} (m)	H _{10%} (m)	H _{5%} (m)	H _{2%} (m)	H _{1%} (m)	H _{max} (m)
1	1.92	6.60	1.84	5.31	1.16	2.32	2.55	2.81	2.97	3.30
2	2.84	8.45	2.74	6.73	1.75	3.39	3.69	4.06	4.37	5.33
3	3.04	8.71	2.95	7.03	1.82	3.73	4.13	4.61	4.93	7.06
4	3.21	9.20	3.09	7.41	1.98	3.80	4.14	4.54	4.77	5.40
5	3.33	9.54	3.22	7.66	2.06	4.02	4.41	4.94	5.34	6.69
6	3.27	8.69	3.17	7.02	2.00	3.99	4.38	4.86	5.26	7.06
7	3.63	9.14	3.49	7.33	2.25	4.29	4.67	5.14	5.44	6.29
8	4.16	9.47	4.04	7.57	2.57	5.07	5.55	6.18	6.64	8.19

As expected, increased wave breaking could be observed for the Low Water Level conditions (LWLs) 6, 7 and 8 between the *Offshore 3PA* and *Nearshore 3PA* locations, requiring an increase in wave conditions generated off the wave paddle to match target wave heights at the *Nearshore 3PA* location (identical to target wave conditions 3, 4 and 5). It can be seen in Table 5.2 that the increase in wave paddle energy resulted in an offshore increase in wave height of 10 to 25% measured at the *Offshore 3PA* location when comparing SLR and LWL conditions.

The calibrated wave conditions presented in Table 5.1 and Table 5.2, and their associated wave paddle settings, were subsequently used for all breakwater armour stability tests.

6 Stage 1: 2D testing

Measurements collected during the 2D Main Breakwater testing in Stage 1 of the program focussed on:

- Wave transmission through the structure and via wave overtopping
- Stability of rock armour

The arrangement of wave probes during this stage of the program was similar to the wave climate calibration phase, as shown in Figure 6.1. The locations of the wave probes comprising the *Offshore 3PA*, were kept unchanged from the calibration phase to allow direct comparison between wave conditions (with test structure in place) and the calibrated wave conditions (no structure). A single wave probe “*Seaward Toe WP*” was positioned 6 m offshore from the seaward toe of the structure, to provide information on the wave conditions directly in front of the breakwater. Two additional probes, “*Leeward 1 WP*” and “*Leeward 2 WP*”, were positioned 8 m and 25 m from the leeward toe of the structure respectively, to assess wave transmission generated by overtopping and waves passing through the rock armour layers.

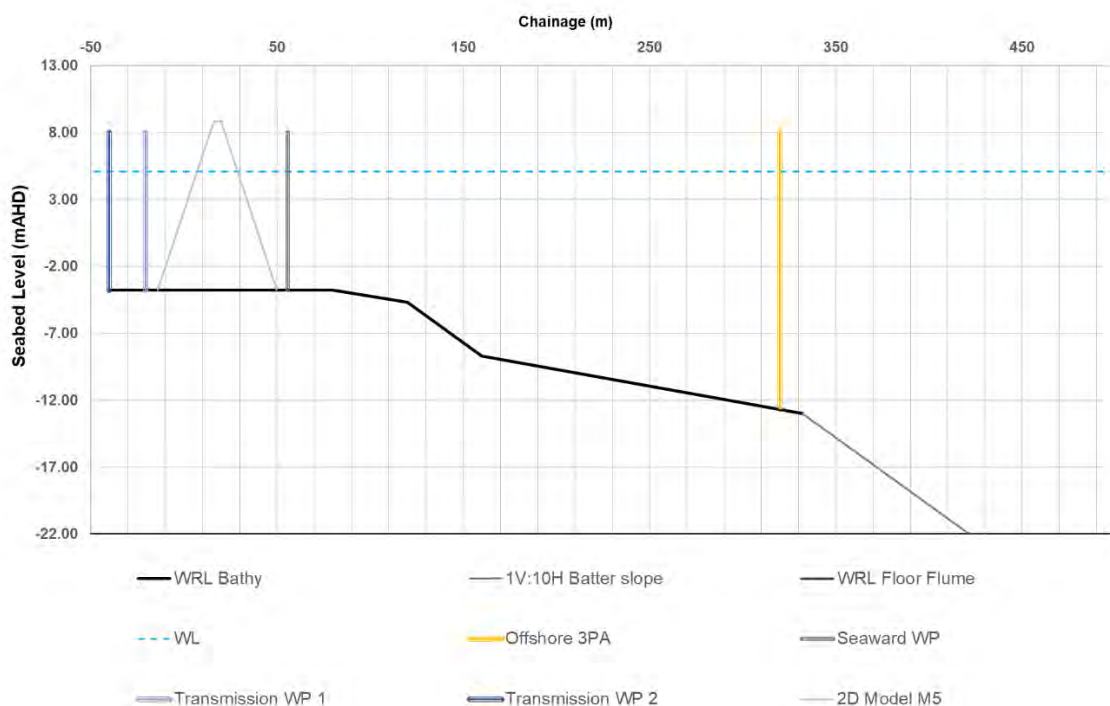


Figure 6.1 Layout of wave probes for Stage 1 testing (2D breakwater sections M5A and M5B)

6.1 M5A 2D breakwater section testing

6.1.1 Overview of M5A testing

The 2D Main Breakwater test section M5A was constructed within the flume and subjected to each of the Test Conditions 1 to 8, with each test running for 3 hours (prototype) duration. Minor adjustments to the model breakwater setup throughout the program included:

- Between Tests 1 and 2, the mini-flume was extended leeward by 2 m and the back wall of the flume equipped with a dissipative beach to reduce effects of wave reflections within the flume
- Between Test Conditions 5 and 6, a minor rebuild of the crest and leeside of the breakwater was conducted to ensure the overall stability of the upper sections and the leeside of the breakwater during the subsequent testing of low water level Test Conditions 6 to 8.

6.1.2 Wave measurements for M5A testing

A summary of the wave statistics obtained at each of the wave probes for Test Conditions 1 to 8, is provided below in Table 6.1. It should be noted that based on the measured wave statistics, wave conditions behind the breakwater for the LWL tests were similar or slightly larger than for the SLR counterparts. This is explained by an increase in wave setup caused by the need to create more energetic offshore wave conditions to match the target nearshore wave design conditions, as well as a progressive reshaping of the breakwater crest (i.e. slumping) allowing for more wave energy to go over the crest and through the armour of the breakwater.

Table 6.1 Measured wave statistics for Test Conditions 1 to 8, test section M5A

Test Condition	Location ¹	H _{m0} (m)	T _p (s)	H _{1/3} (m)	T _z (s)	H _{mean} (m)	H _{1%} (m)	H _{max} (m)
1	Leeward 2	-	-	-	-	-	-	-
	Leeward 1	-	-	-	-	-	-	-
	Seaward Toe	1.77	6.9	1.78	5.3	1.07	3.06	3.59
	Offshore 3PA	1.89	6.5	1.82	5.0	1.07	3.05	3.47
2	Leeward 2	0.45	26.1	0.37	10.5	0.25	0.56	0.66
	Leeward 1	0.44	103.1	0.34	12.3	0.22	0.60	0.68
	Seaward Toe	3.32	8.4	3.42	7.3	2.16	5.34	6.05
	Offshore 3PA	2.85	8.3	2.75	6.7	1.77	4.25	5.37
3	Leeward 2	0.52	48.5	0.42	10.6	0.28	0.61	0.69
	Leeward 1	0.48	76.9	0.36	12.3	0.23	0.67	0.97
	Seaward Toe	3.64	8.7	3.79	7.6	2.38	6.01	7.13
	Offshore 3PA	3.08	8.7	2.98	7.0	1.88	4.94	7.27
4	Leeward 2	0.54	70.5	0.44	9.8	0.29	0.66	0.80
	Leeward 1	0.48	141.1	0.35	10.5	0.21	0.62	0.79
	Seaward Toe	3.76	9.2	3.89	7.9	2.46	6.12	6.90
	Offshore 3PA	3.17	9.3	3.06	7.3	1.96	4.82	5.35
5	Leeward 2	0.57	47.4	0.47	8.7	0.30	0.74	0.97
	Leeward 1	0.50	154.4	0.38	8.8	0.22	0.79	1.51
	Seaward Toe	3.86	9.7	3.99	8.0	2.50	6.44	7.27
	Offshore 3PA	3.24	9.5	3.12	7.5	1.99	5.06	7.06
6	Leeward 2	0.57	38.9	0.45	12.1	0.30	0.70	0.87
	Leeward 1	0.57	45.3	0.48	18.2	0.28	0.77	0.83
	Seaward Toe	3.57	8.9	3.83	7.4	2.41	5.28	5.88
	Offshore 3PA	3.23	8.6	3.13	7.0	1.98	5.27	7.27
7	Leeward 2	0.60	32.7	0.47	12.4	0.33	0.75	0.89
	Leeward 1	0.59	48.6	0.49	18.1	0.29	0.79	0.94
	Seaward Toe	3.86	9.2	4.13	7.8	2.68	5.55	6.04
	Offshore 3PA	3.58	9.1	3.46	7.3	2.22	5.41	6.35
8	Leeward 2	0.67	105.5	0.50	13.8	0.34	0.83	0.94
	Leeward 1	0.68	82.0	0.55	19.4	0.32	0.91	1.06
	Seaward Toe	4.30	11.2	4.46	8.1	2.95	6.11	6.69
	Offshore 3PA	4.16	9.5	4.01	7.4	2.52	6.46	8.11

Notes:

1. Only the offshore wave measurement location (*Offshore 3PA*) allowed for the separation of the incident and reflected waves. The reflected wave signal could not be removed from the three additional single wave probes located closer to the tested structure and were used to provide information on wave transmission, namely *Leeward 1*, *Leeward 2* and *Seaward Toe*. As such, the *Seaward Toe* measurements which contain both incident and reflected wave energy, are slightly larger than the *Offshore 3PA* values which have the component of reflected wave energy removed.
2. Wave period values for the Leeward probes 1 and 2 are on average overestimated due to the intermittent nature of overtopping. Observed $H_{\max}$ are likely to be overestimated due to the high level of white water spray and splashing hitting the leeside wave probes.

6.1.3 Wave transmission analysis for test section M5A

Wave conditions on the leeside of the breakwater cross-section were assessed to provide preliminary information regarding transmitted wave energy. During testing it was observed that transmitted waves in the lee of the breakwater were caused by a combination of wave overtopping over the crest of the structure and wave transmission through the porous layers of rock armour at the crest.

The effects of overtopping over the structure and wave transmission through the structure were assessed by analysing the recorded timeseries of the water surface on the leeward side of the tested 2D model (refer to Figure 6.1 for wave probe layout). A number of adjustments were made to the mini flume between Tests 1 and 2 in order to minimise wave record contamination from reflected waves moving within the flume on the leeward side of the model. Although these improvements were only implemented after Test 1, this had no consequence on wave transmission results for test 1 as only negligible overtopping of the crest could be observed during testing for this wave climate.

Wave transmission data was analysed in two ways. Firstly, wave statistics were produced via zero-crossing analysis ($H_{1/3}$, T_z , H_{mean} , $H_{1\%}$, $H_{\max}$) and wave spectral analysis (H_{m0} , T_p) for the full recorded time series of data logged for each test. A summary of the wave statistics obtained at each of the wave probes for Test Conditions 1 to 8 is provided in Table 6.1. This approach considered the wave statistics for the full recorded test timeseries, and therefore included both the occurrences of wave overtopping, as well as the periods of tranquillity between overtopping occurrences.

Secondly, a more focussed analysis was completed which considered only the wave transmission records for the five largest overtopping occurrences during each test. These records were manually extracted from the full test data set and analysed to obtain a more thorough understanding of wave transmission generated by overtopping. The five largest overtopping events and their associated wave heights at the *Seaward Toe WP* and *Leeward WP* locations, were determined from the water surface time series extracts, based on the peak/trough levels of each water surface time series. This approach differs to the zero-crossing/spectral analysis used to obtain wave statistics in Table 6.1, and subsequently, the results from the two different analysis techniques should not be compared. This second type of analysis was completed only for the more extreme wave/water level test conditions (Test Conditions 3 to 5), as they had the most significant overtopping. Wave transmission over/through the breakwater was calculated as the ratio of wave heights between the *Leeward WP* and the *Seaward Toe WP*. A summary of the results from analysing the five largest events for Test Conditions 3 to 5 is provided in Table 6.2 to Table 6.4. Additionally, an average of the wave transmission results for the five largest overtopping events is provided in Table 6.5.

Based on analysis of the five largest overtopping events, wave transmission (*Leeward WP* vs *Seaward WP*) over/through the breakwater was on average around 10% at *Leeward 1 WP* (8 m from the leeward

toe) and 6% at the *Leeward 2 WP* (25 m from the leeward toe). The larger transmission for some events at *Leeward 1 WP* is likely due to direct splashing from overtopping, that wasn't observed to reach *Leeward 2 WP* during the largest overtopping events. The measurements at *Leeward 2 WP* are therefore considered to provide a more robust estimate of wave transmission generated by overtopping.

Table 6.2 Wave transmission for the five largest overtopping events of Test Condition 3 (Section M5A)

Event wave height (m) at the Seaward Toe (ST) ($H_{\text{event, ST}}$)	Event wave height (m) at Leeward 1 (L1) ($H_{\text{event, L1}}$)	Event wave height (m) at Leeward 2 (L2) ($H_{\text{event, L2}}$)	Leeward 1 transmission ($H_{\text{trans, L1}} / H_{\text{event, ST}}$)	Leeward 2 transmission ($H_{\text{trans, L2}} / H_{\text{event, ST}}$)
6.2	0.6	0.5	10%	9%
6.3	0.3	0.3	5%	5%
6.3	0.4	0.4	7%	6%
7.0	0.9	0.4	13%	6%
7.1	0.5	0.4	6%	6%

Table 6.3 Wave transmission for the five largest overtopping events of Test Condition 4 (Section M5A)

Event wave height (m) at the Seaward Toe (ST) ($H_{\text{event, ST}}$)	Event wave height (m) at Leeward 1 (L1) ($H_{\text{event, L1}}$)	Event wave height (m) at Leeward 2 (L2) ($H_{\text{event, L2}}$)	Leeward 1 transmission ($H_{\text{trans, L1}} / H_{\text{event, ST}}$)	Leeward 2 transmission ($H_{\text{trans, L2}} / H_{\text{event, ST}}$)
4.9	0.4	0.2	7%	4%
5.2	0.5	0.3	9%	6%
5.8	0.4	0.2	6%	4%
6.3	0.6	0.4	9%	7%
6.8	0.4	0.5	5%	7%

Table 6.4 Wave transmission for the five largest overtopping events of Test Condition 5 (Section M5A)

Event wave height (m) at the Seaward Toe (ST) ($H_{\text{event, ST}}$)	Event wave height (m) at Leeward 1 (L1) ($H_{\text{event, L1}}$)	Event wave height (m) at Leeward 2 (L2) ($H_{\text{event, L2}}$)	Leeward 1 transmission ($H_{\text{trans, L1}} / H_{\text{event, ST}}$)	Leeward 2 transmission ($H_{\text{trans, L2}} / H_{\text{event, ST}}$)
5.4	1.3	0.6	25%	10%
6.1	0.6	0.2	9%	4%
6.2	0.6	0.4	9%	7%
6.4	0.8	0.2	13%	3%
6.9	0.6	0.4	8%	5%

Table 6.5 Average wave transmission for the five largest overtopping events during Test Conditions 3, 4 and 5 (Section M5A)

Wave Condition	Average event wave height (m) at the Seaward Toe	Average event wave height (m) at Leeward 1	Average event wave height (m) at Leeward 2	Average Leeward 1 transmission	Average Leeward 2 transmission
WC 3	6.6	0.5	0.4	8%	6%
WC 4	5.8	0.4	0.3	8%	5%
WC 5	6.2	0.8	0.4	13%	6%

6.1.4 Rock armour damage assessment for test section M5A

Breakwater armour damage was recorded for each of the eight test conditions. Armour damage was recorded cumulatively for Test Conditions 1 to 5 (high water level tests). After a minor rebuild, damage was then recorded cumulatively for Test Conditions 6 to 8 (low water level tests). A summary of the damage assessment results is provided in Table 6.6. The associated types of overtopping (spray, white water jets, or complete submergence of the crest sheet) observed for each test are also provided in Table 6.6. A more extensive damage assessment for Test Conditions 1 to 8 follows. The reported armour damage numbers are expressed relative to the total number of rocks within each respective zone of the test section, as shown in Figure 6.2.

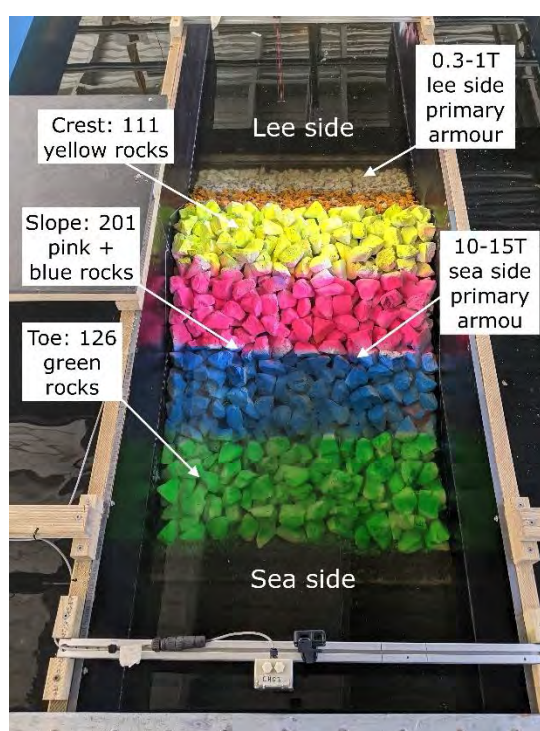


Figure 6.2 Completed M5A cross-section model, including armour stone counts

Table 6.6 Test section M5A summary of damage assessment and overtopping type

Test Condition	Cumulative # rock movement > D ₅₀ (Seaside)	Cumulative # rock movement > D ₅₀ (Leeside)	Percentage of rock movement > D ₅₀ (%) * (Seaside)	Percentage of rock movement > D ₅₀ (%) ** (Leeside)	Over topping (type)
1	0	0	0%	0%	spray
2	0	3	0%	< 1%	jets
3	1	8	< 0.1%	1%	jets / sheet
4	1	23	< 0.1%	3%	jets / sheet
5	8	60	2%	7%	jets / sheet
<i>Minor rebuild of structure</i>					
6	0	0	0%	0%	spray
7	0	0	0%	0%	jets
8	1	6	< 0.1%	< 1%	jets / sheet

Notes:

*The percentage of rock movement on the seaside primary armour was calculated as the cumulated number of rocks that moved greater than D₅₀, divided by the total number (438) of primary armour rocks (see Figure 6.2).

**The percentage of rock movement on the leeside primary armour was calculated as the cumulated number of rocks that moved greater than D₅₀, divided by the estimated total number (800) of primary armour rocks exposed (above 3 m AHD) during testing.

Test Condition 1 (1 year ARI waves / 1 year ARI + SLR water level)

No damage was observed to the M5A test structure during the 1 year ARI 'shakedown' test. Overtopping of the structure was limited to occasional spray, reaching the leeward row of the 10 to 15 T rocks on the crest.

Test Condition 2 (50 year ARI waves / 10 year ARI + SLR water level)

No damage was observed to the seaside primary armour during Test Condition 2. However, a few seaside primary armour rocks were observed to rock around the line of the still water level (SWL). Negligible damage to the 0.3 to 1 T leeside armour was observed with less than 10 rocks along the SWL becoming displaced. This damage was caused by white-water jets that occasionally reached the leeside armour. Figure 6.3 shows images of the test structure before and after this test.

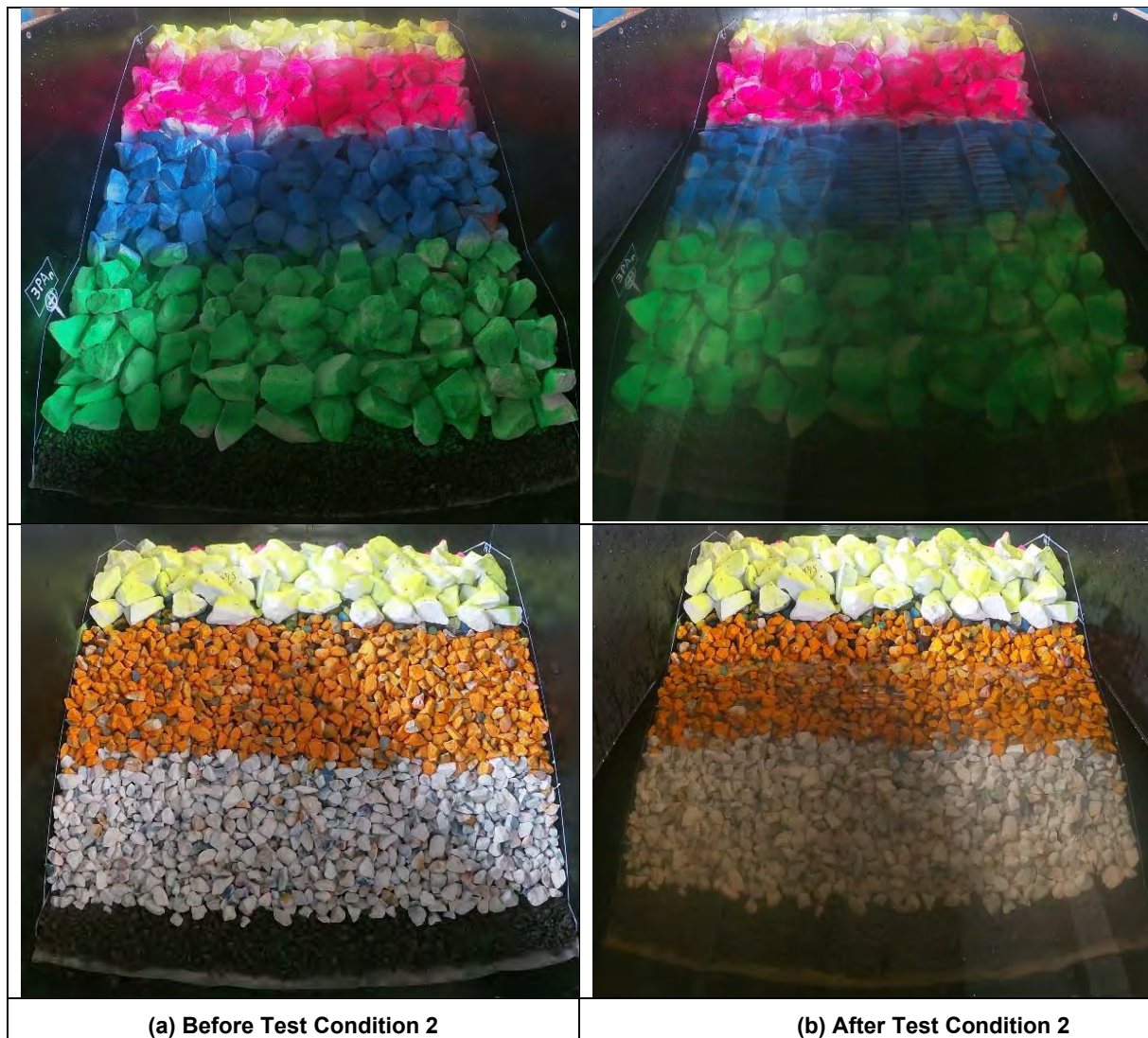


Figure 6.3 Images from Test Condition 2 showing seaside (top) and leeside (bottom) armour

Test Condition 3 (100 year ARI waves / 25 year ARI + SLR water level)

Negligible damage to the seaside primary armour was observed during Test Condition 3, with one rock that had previously been rocking eventually becoming displaced during a larger set of waves (< 0.1% damage). An increased number of 0.3 to 1 T leeside armour rocks were displaced along the SWL. This increased displacement appeared to be primarily due to the direct overtopping jet impact and partially the result of water level drawdown during bigger sets of waves.

Overtopping of the structure notably increased in intensity during this test, with jets observed to reach at least 20 m leeward of the crest and the crest becoming fully submerged (white water) at least 10 to 15 times during the 3-hour (prototype) time series.

Overall, the structure was stable, with no damage to the seaside toe and crest and minimal rocking of the seaside primary armour. Continued displacement of the leeside armour occurred, though was restricted to the upper layer only and no exposure of geotextile/core resulted. Figure 6.4 shows images of the test structure before and after this test.

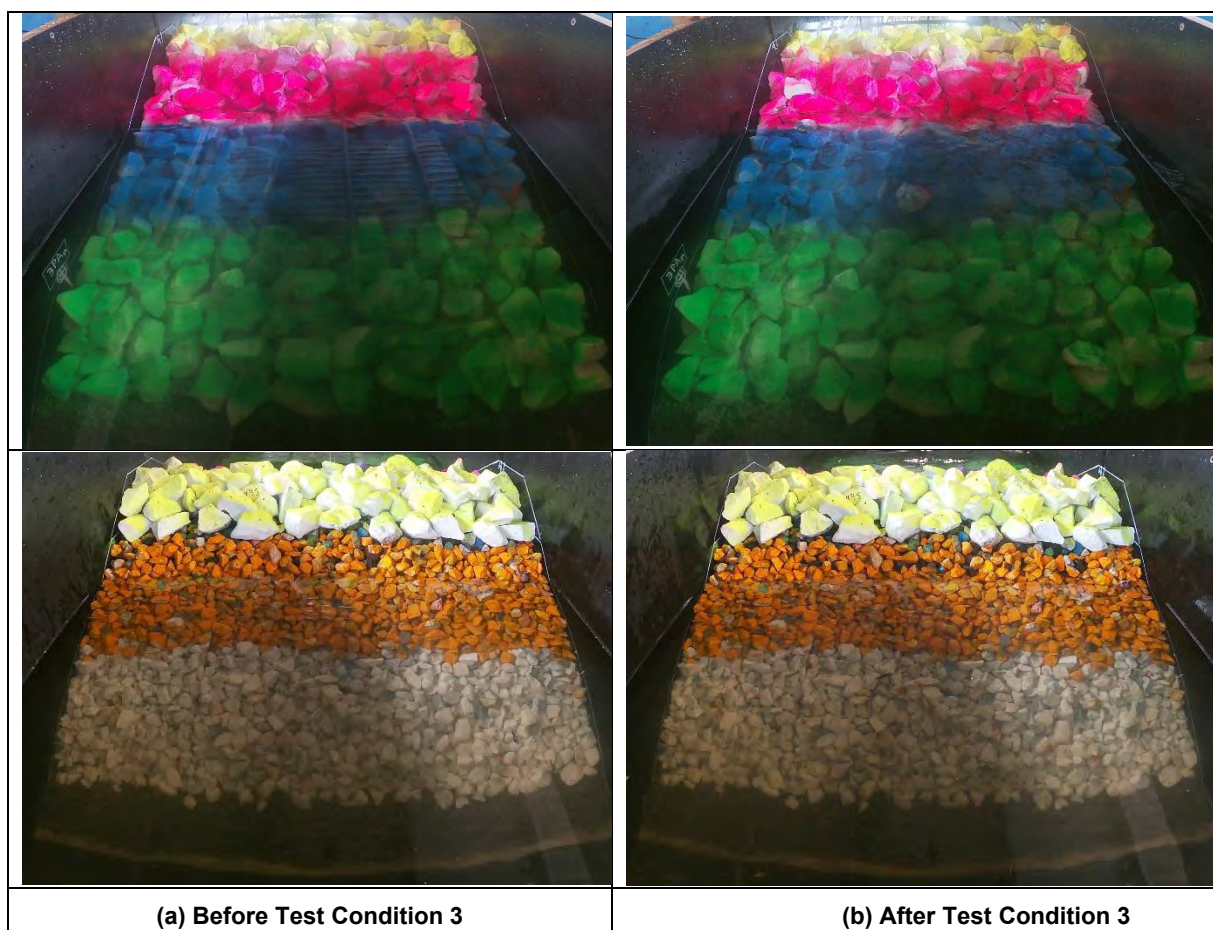


Figure 6.4 Images from Test Condition 3 showing seaside (top) and leeside (bottom) armour

Test Condition 4 (200 year ARI waves / 50 year ARI + SLR water level)

No additional damage to the seaside primary armour was observed during this test, although a small number (approximately 5) of the 10 to 15 T armour stones on the crest were observed to rock during the larger overtopping events. An increased number of 0.3 to 1 T leeside armour stones were observed to be displaced, due to the direct overtopping jet impact. This increased damage to the leeside armour resulted in exposed geotextile (i.e. breakwater core) in two different locations around the SWL.

An increased frequency and intensity of overtopping events was observed, with overtopping jets reaching at least 30 m leeward of the crest and full submergence of the crest occurring with most of the larger wave sets.

Figure 6.5 shows images of the test structure before and after Test Condition 4.

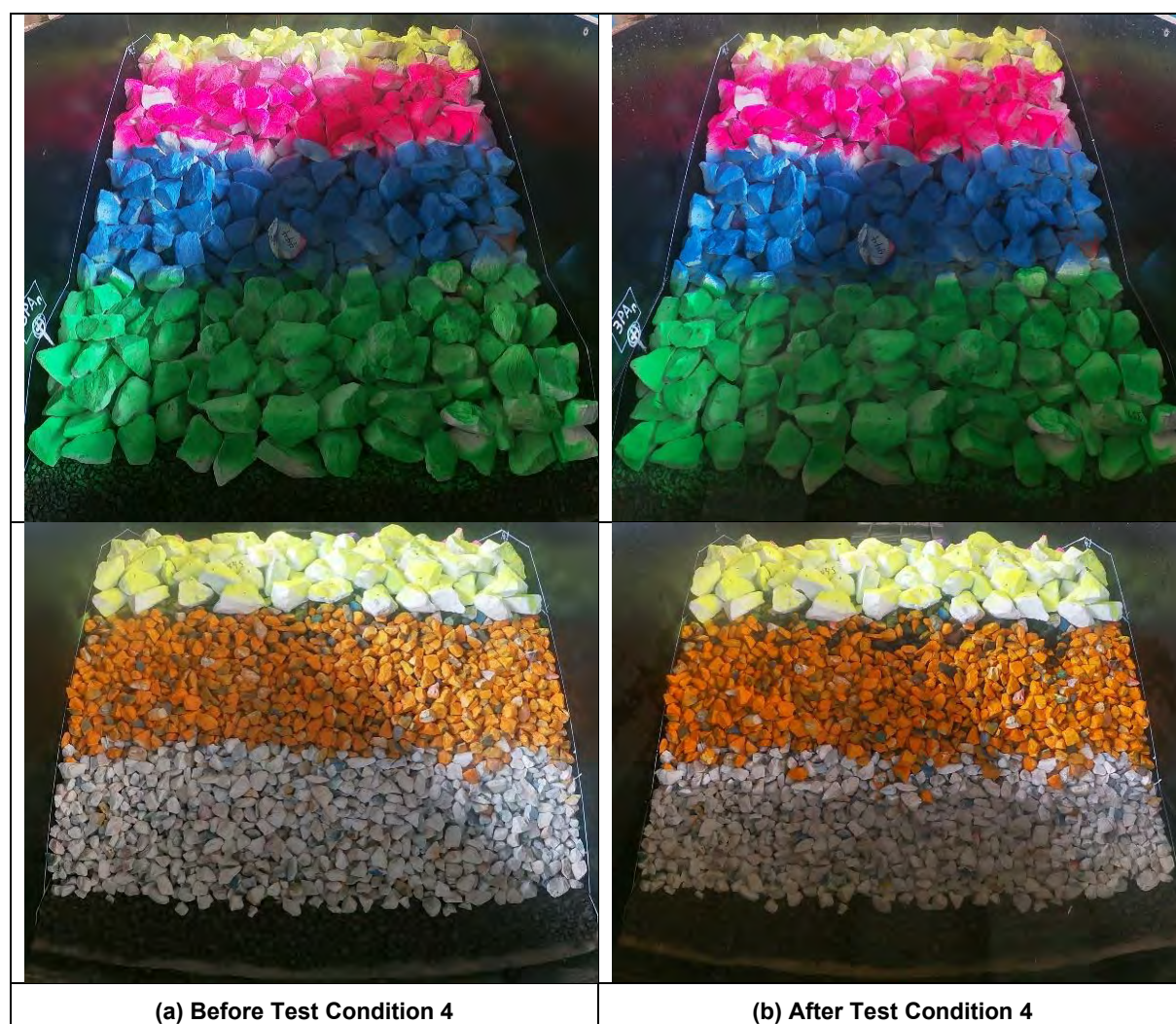


Figure 6.5 Images from Test Condition 4 showing seaside (top) and leeside (bottom) armour

Test Condition 5 (500 year ARI waves / 100 year ARI + SLR water level)

A rapid progression of damage to the leeward side of the 10 to 15 T crest armour (leeward row of rocks) was observed, with six rocks becoming fully displaced from the crest and one rock falling down the leeside slope, towards the end of the 3-hour (prototype) test sequence. This damage accounted for 1.6% of all the 10 to 15 T primary armour rock, and 6% of the 10 to 15 T rock on the crest. Damage of the crest appeared to be exacerbated by the lack of structural support from the leeside primary armour, which had become progressively more damaged throughout the test program. The remainder of the leeside 10 to 15 T rocks of the crest were observed to progressively shift backwards resulting in an overall slumping of the crest. The 10 to 15 T toe and seaward slope armour was stable with no damage and only minimal rocking of armour stones.

Increased damage to the leeside armour was also observed, resulting in greater exposure of the geotextile/core in three different locations around the SWL.

Overtopping events were more intense and more frequent than previously observed during Test Condition 4, with complete submergence of the crest occurring during all of the larger wave sets.

Figure 6.6 shows images of the test structure before and after Test Condition 5, while Figure 6.7 shows a post-test image of the structure with annotation of damage.

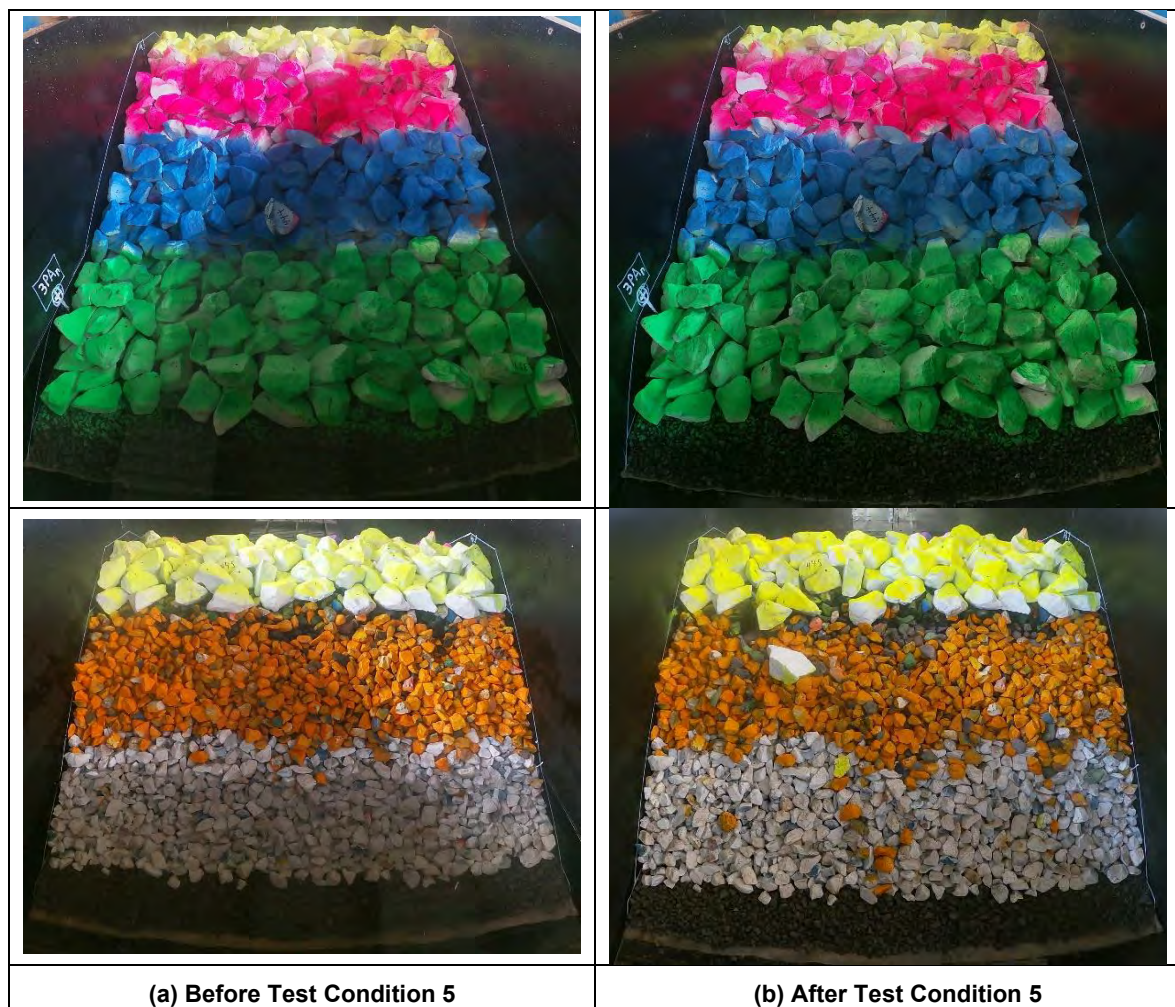


Figure 6.6 Images from Test Condition 5 showing seaside (top) and leeside (bottom) armour



Figure 6.7 Annotated plan view image following completion of Test Condition 5

Test Condition 6 (100 year ARI waves / LWL water level)

No damage to the seaside or the leeside primary armour was observed during Test Condition 6. A few of the 10 to 15 T rocks (less than 5) were observed to rock around the SWL. Overtopping events were limited to occasional spray reaching the leeward side of the crest.

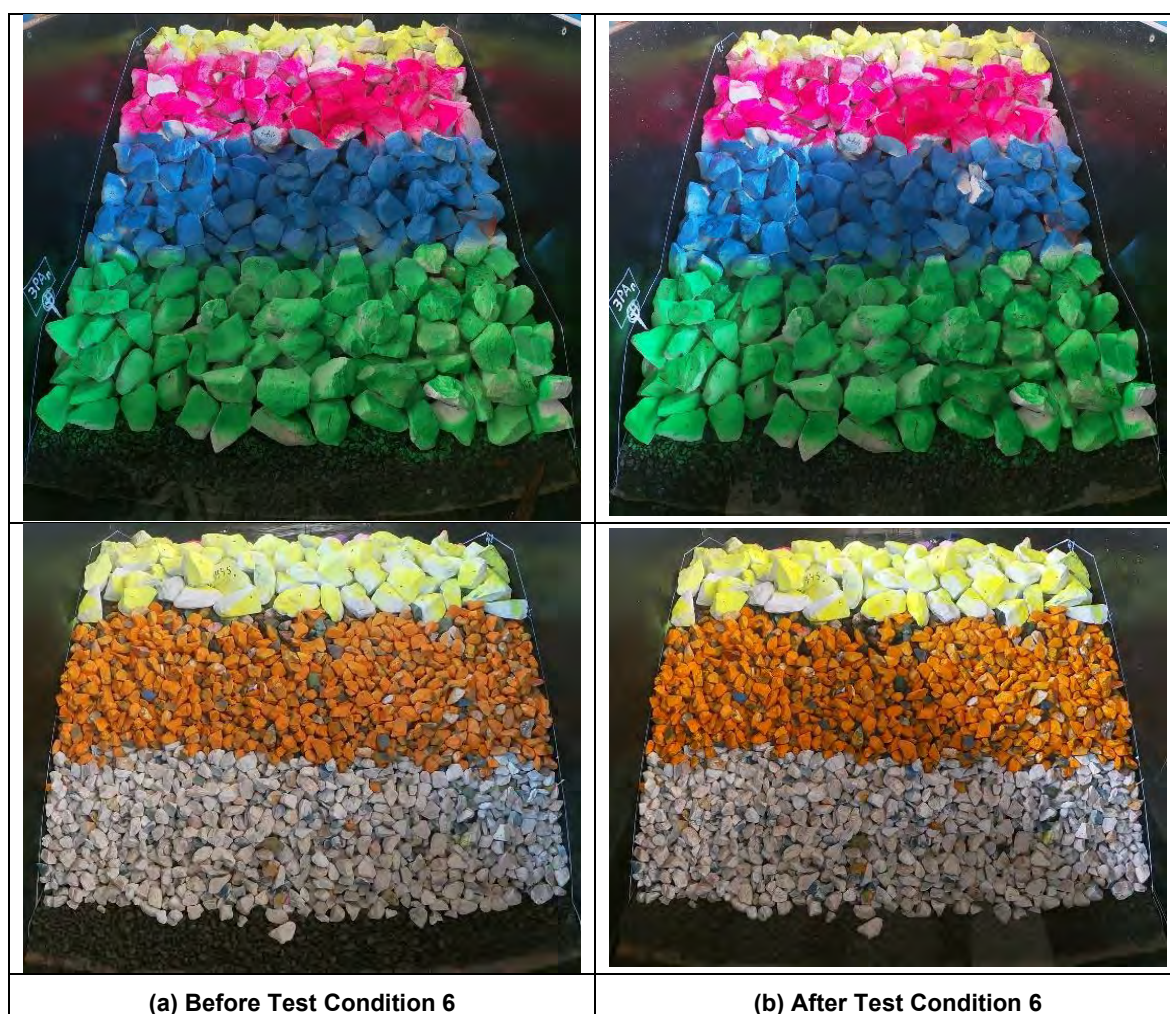


Figure 6.8 Images from Test Condition 6 showing seaside (top) and leeside (bottom) armour

Test Condition 7 (200 year ARI waves / LWL water level)

No damage to seaside or leeside primary armour was observed during Test Condition 7. Again, a few rocks (less than five) were observed to be rocking around the SWL, but the seaside toe remained stable. Overtopping was limited to small, aerated jets during the largest 5% of waves. Figure 6.9 shows images of the test section before and after Test Condition 7.

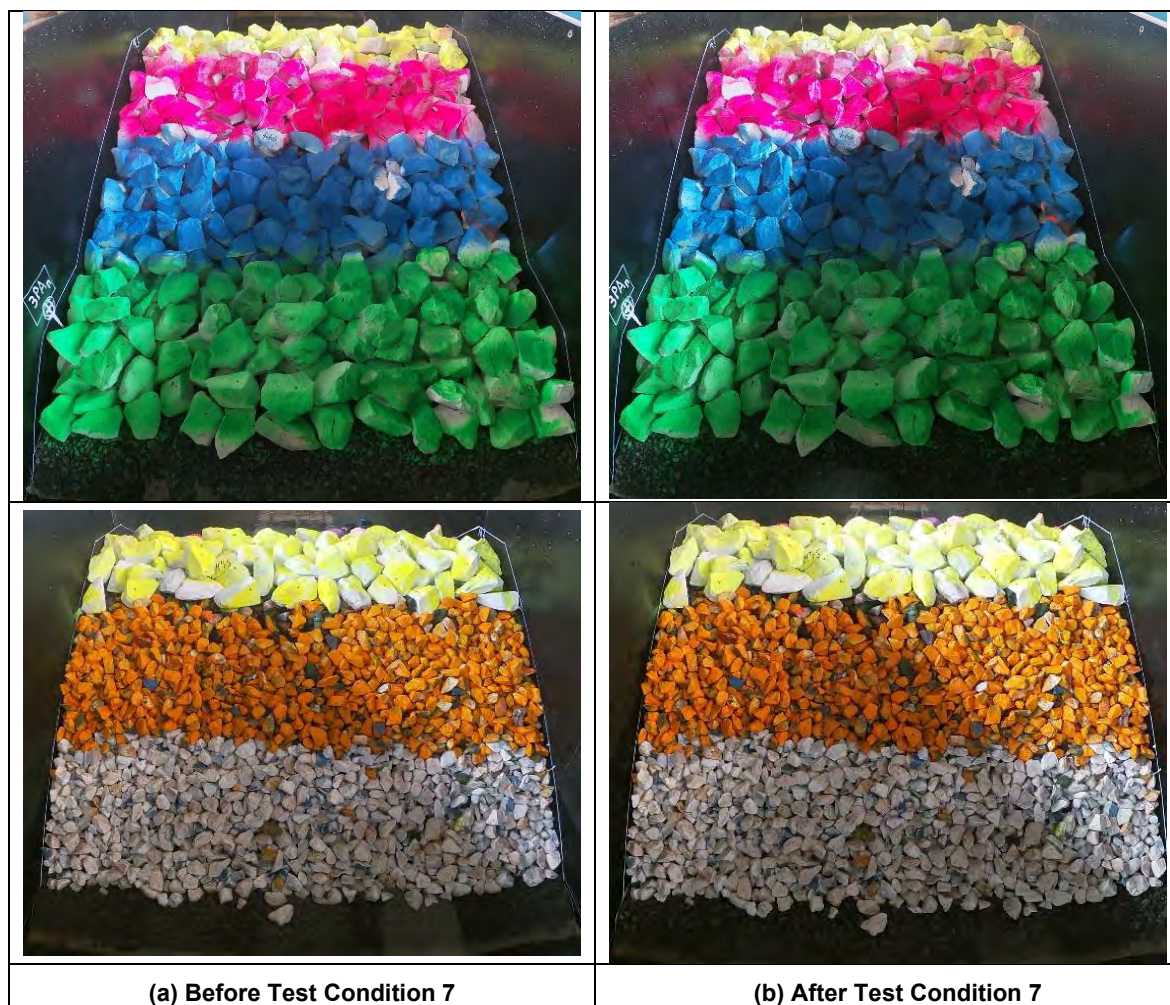


Figure 6.9 Images from Test Condition 7 showing seaside (top) and leeside (bottom) armour

Test Condition 8 (500 year ARI waves / LWL water level)

Minor/negligible damage to the seaside primary armour at the toe was observed during Test Condition 8, with one rock falling a distance greater than D_{50} and five rocks settling a distance of less than D_{50} . Damage to the seaside toe primary armour was due to both the wave breaking forces and the drawdown of the larger waves. Minor damage (<5 rocks) to the exposed leeside primary armour was observed during overtopping events.

The largest ~1% of waves appeared to break at the bathymetry transition approximately 70 m seaward of the breakwater toe. These waves appeared to cause less damage than the sets of slightly smaller waves (~5% of waves), that would break on or nearer to the breakwater structure. These sets of slightly smaller waves occasionally resulted in complete submergence of the crest during overtopping.

Figure 6.10 shows images of the test section before and after Test Condition 8.

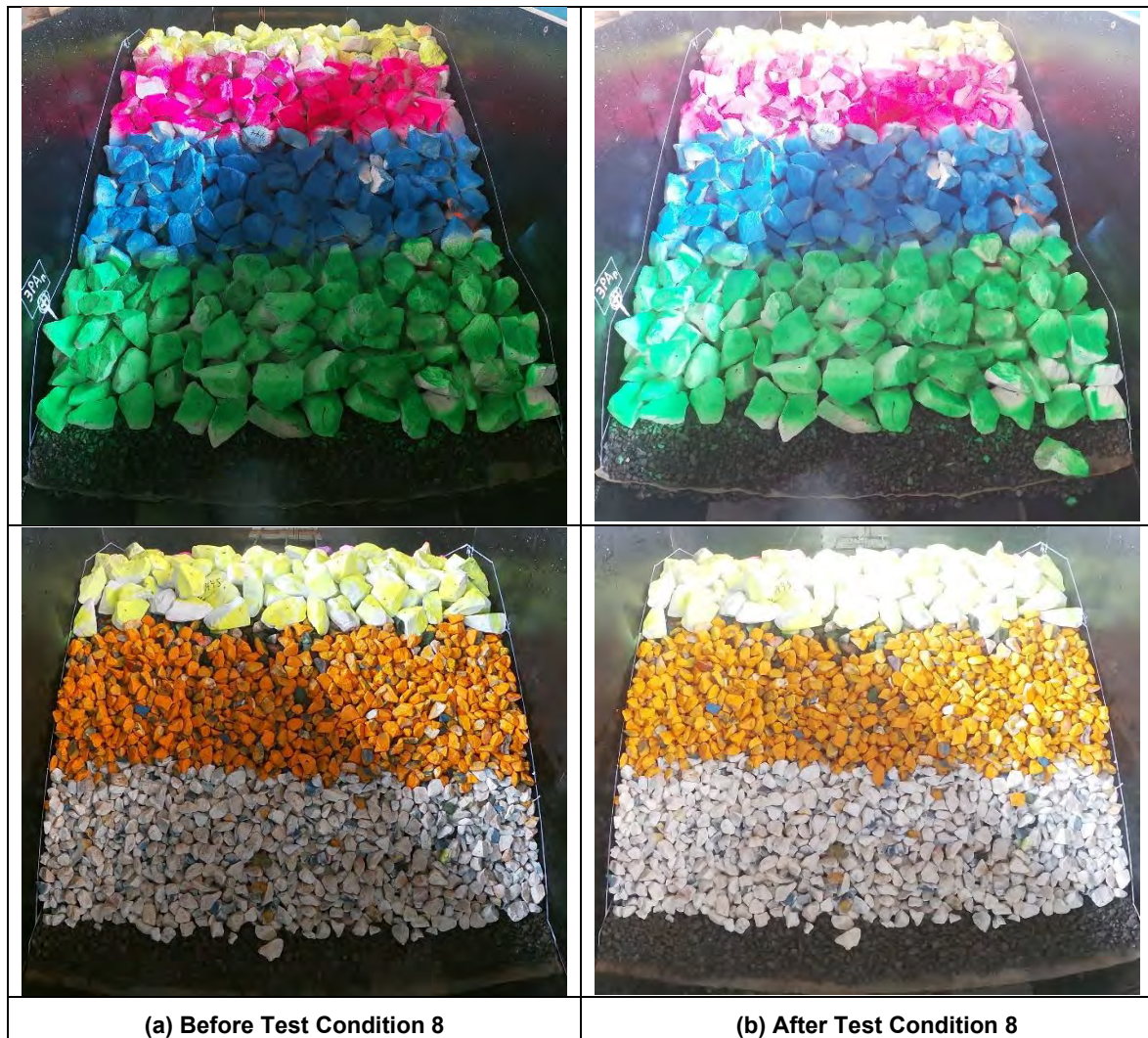


Figure 6.10 Images from Test Condition 8 showing seaside (top) and leeside (bottom) armour

Additional test: Test Condition 6 with 6 to 10 T armour stones on the seaward side toe

Given the observed stability of the 10 to 15 T rocks on the seaside toe of the structure, WRL fronted the seaside primary armour with two rows/layers of 6 to 10 T rocks and re-ran Test Condition 6 (100 yr ARI / LWL). The test was run for reduced duration of approximately 1,000 waves, and negligible damage was observed (one rock moved a distance of $\sim 1 \times D_{50}$). This test demonstrated the potential for a reduced armour size (6 to 10 T) to be stable on the seaward slope of the breakwater, though would require additional testing with proper model construction and full test duration.

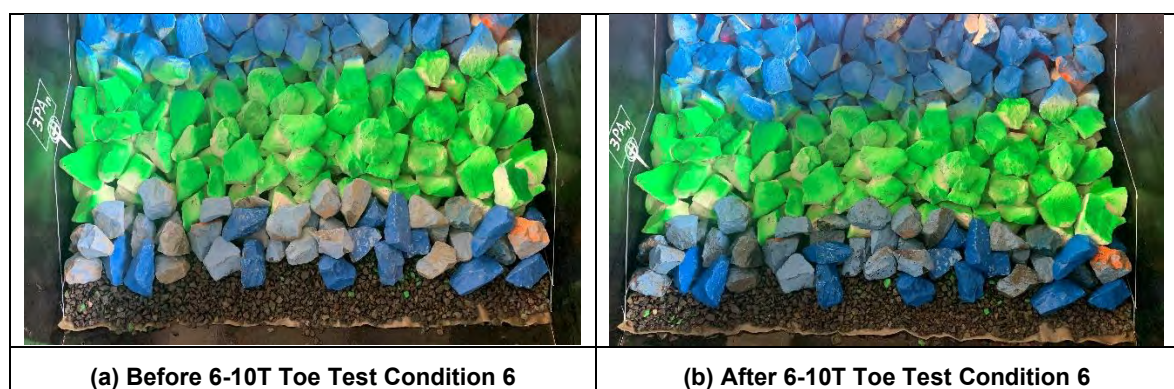


Figure 6.11 Images from additional test with 6 to 10 T primary armour on seaward toe

6.2 M5B 2D breakwater section testing

The 2D Main Breakwater test section M5B was constructed within the flume and subjected to a reduced sequence of test conditions as requested by Cardno, with each test running for 3 hours (prototype) duration. Table 6.7 shows a summary of the test conditions that were conducted for section M5B. Following completion of the high water level tests (Test Conditions 1, 3, 4, and 5), a minor rebuild of the 6 to 10 T crest armour and the upper leeside 1 to 3 T armour was conducted to ensure the overall stability of the upper sections of the breakwater when testing subsequent low water level conditions (Test Conditions 6 and 8).

Table 6.7 Summary of test conditions run for section M5B

Test Condition	Wave Condition	WL Condition	H _{m0} (m)	T _p (s)	WL (m AHD)
1	1 yr ARI	1yr ARI + SLR	1.86	6.5	3.44
3	100 yr ARI	25 yr ARI + SLR	2.98	8.8	4.46
4	200 yr ARI	50 yr ARI + SLR	3.15	9.2	4.75
5	500 yr ARI	100 yr ARI + SLR	3.36	9.6	5.05
6	100 yr ARI	LWL no SLR	2.98	8.8	2.5
8	500 yr ARI	LWL no SLR	3.36	9.6	3

The breakwater structure changes between section M5A and M5B included:

- Modification to the crest and seaside primary armour
 - o Primary armour rock grading decreased from 10 to 15 T to 6 to 10 T
 - o Primary armour crest level lowered by 1.1 m (8.8 to 7.7 m AHD)
 - o Primary armour crest width increased by 2 m (4.8 to 6.8 m)
 - o Toe crest width decreased from by 1.2 m (5.1 to 3.9 m)
 - o Leeside crest armour extended down to the level of the core
- Modification to the leeside primary armour and addition of a secondary armour
 - o Primary armour rock grading increased from 0.3 to 1 T to 1 to 3 T
 - o Addition of 60 to 300 kg secondary armour layer
- Modification to the core
 - o Core crest width decreased by 1 m (8 to 7 m)
 - o Core crest level lowered by 0.2 m (4.1 to 3.9 m AHD)

Figure 4.7 shows a comparison of the two 2D test sections.

6.2.1 Wave measurements for M5B testing

A summary of the wave statistics obtained at each of the wave probes for all tests on section M5B, is provided in Table 6.8.

Table 6.8 Measured wave statistics for testing of section M5B

Test Condition	Location ¹	H _{m0} (m)	T _p (s)	H _{1/3} (m)	T _z (s)	H _{mean} (m)	H _{1%} (m)	H _{max} (m)
1	Leeward 2	0.18	198.03	0.13	9.34	0.08	0.21	0.24
	Leeward 1	0.22	89.33	0.16	10.09	0.10	0.27	0.31
	Seaward Toe	1.82	6.77	1.79	5.36	1.13	3.07	3.54
	Offshore 3PA	1.93	6.50	1.86	5.35	1.18	3.14	3.68
3	Leeward 2	0.52	121.89	0.42	9.48	0.26	0.65	0.74
	Leeward 1	0.49	128.86	0.36	10.90	0.23	0.63	0.91
	Seaward Toe	3.64	8.74	3.76	7.56	2.36	6.05	7.34
	Offshore 3PA	3.08	8.67	2.98	7.00	1.88	4.92	7.19
4	Leeward 2	0.58	112.01	0.47	8.02	0.29	0.76	0.92
	Leeward 1	0.50	169.18	0.38	7.63	0.22	0.84	1.08
	Seaward Toe	3.78	9.31	3.93	7.94	2.47	6.13	6.91
	Offshore 3PA	3.18	9.29	3.08	7.34	1.98	4.83	5.53
5	Leeward 2	0.64	67.92	0.53	6.57	0.32	0.92	1.50
	Leeward 1	0.54	174.12	0.41	5.84	0.24	0.97	2.17
	Seaward Toe	3.83	9.76	3.95	8.08	2.49	6.47	7.78
	Offshore 3PA	3.25	9.55	3.12	7.44	1.99	5.09	7.00
6	Leeward 2	0.57	62.72	0.42	13.50	0.30	0.68	0.82
	Leeward 1	0.57	46.96	0.48	19.36	0.29	0.74	0.79
	Seaward Toe	3.52	8.99	3.73	7.50	2.38	5.09	5.71
	Offshore 3PA	3.21	8.61	3.12	7.04	1.98	5.28	7.30
8	Leeward 2	0.65	110.12	0.49	13.82	0.32	0.84	0.98
	Leeward 1	0.67	79.21	0.52	16.73	0.29	0.88	0.92
	Seaward Toe	4.32	11.25	4.39	8.04	2.93	5.91	6.39
	Offshore 3PA	4.15	9.45	4.01	7.46	2.54	6.46	7.86

Notes:

1. Only the offshore wave measurement location (*Offshore 3PA*) allowed for the separation of the incident and reflected waves. The reflected wave signal could not be removed from the three additional single wave probes located closer to the tested structure and were used to provide information on wave transmission, namely *Leeward 1*, *Leeward 2* and *Seaward Toe*. As such, the *Seaward Toe* measurements which contain both incident and reflected wave energy, are slightly larger than the *Offshore_3PA* values which have the component of reflected wave energy removed.
2. Wave period values for the Leeward probes 1 and 2 are on average overestimated due to the intermittent nature of overtopping. Observed H_{max} are likely to be overestimated due to the high level of white water spray and splashing hitting the leeside wave probes.

6.2.2 Wave transmission analysis for test section M5B

Wave conditions on the leeside of the M5B breakwater cross-section were assessed to provide preliminary information regarding transmitted wave energy. During testing it was observed that transmitted waves in the lee of the breakwater were caused by a combination of wave overtopping over the crest of the structure and wave transmission through the porous layers of rock armour. The transmitted wave conditions were assessed by analysing the recorded timeseries of the water surface on the leeward side of the model compared with the seaward side of the model.

Wave transmission data obtained from the wave probes for each of the tests was analysed in two ways. Firstly wave statistics were produced via zero-crossing and wave spectral analysis (shown previously in Table 6.8), and secondly, wave transmission through the structure was assessed by analysing the time-series for the five largest individual overtopping events (refer to the previous discussion in Section 6.1.3 on Test Section M5A for further explanation). This latter analysis was completed for Test Condition 5 only (500-year ARI waves and 100-year ARI + SLR water level condition), as this test condition presented the highest amount of wave transmission. By comparing wave transmission results between M5A and M5B for the same design wave conditions, the difference in wave transmission through/over the structure due to differences in cross-section design, were also determined.

For Test Condition 5 (500-year ARI waves and 100-year ARI + SLR), analysis of the five largest overtopping events showed that wave transmission increased between M5A and M5B (see Table 6.9), with the average transmission at *Leeward 1 WP* (8 m from the leeward toe) increasing from 10 to 23% and at *Leeward 2 WP* (25 m from the leeward toe) increasing from 6 to 16%. The increase in wave transmission through/over the structure between M5A and M5B was most likely due to the 1.1 m lowering of the structure crest level.

Table 6.9 Wave transmission for the five largest overtopping events of Test Condition 5 (Section M5B)

OT Event #	Structure	Event wave height at the Seaward Toe (ST) ($H_{\text{event, ST}}$)	Event wave height at Leeward 1 (L1) ($H_{\text{event, L1}}$)	Event wave height at Leeward 2 (L2) ($H_{\text{event, L2}}$)	Leeward 1 transmission ($H_{\text{trans, L1}} / H_{\text{event, ST}}$)	Leeward 2 transmission ($H_{\text{trans, L2}} / H_{\text{event, ST}}$)
1	M5B	6.08	2.17	1.50	36%	25%
	M5A	5.43	1.34	0.56	25%	10%
2	M5B	6.11	1.00	0.55	16%	9%
	M5A	6.14	0.57	0.24	9%	4%
3	M5B	6.70	0.80	0.76	12%	11%
	M5A	6.23	0.57	0.41	9%	7%
4	M5B	6.63	1.17	0.77	18%	12%
	M5A	6.43	0.84	0.21	13%	3%
5	M5B	6.95	2.17	1.50	31%	22%
	M5A	6.93	0.57	0.38	8%	5%

These wave transmission results were further analysed by a comparison between the zero-crossing wave statistics at *Leeward 1 WP*, which showed the difference in overtopping between M5A and M5B breakwater sections increased in proportion to wave height (see Figure 6.12).

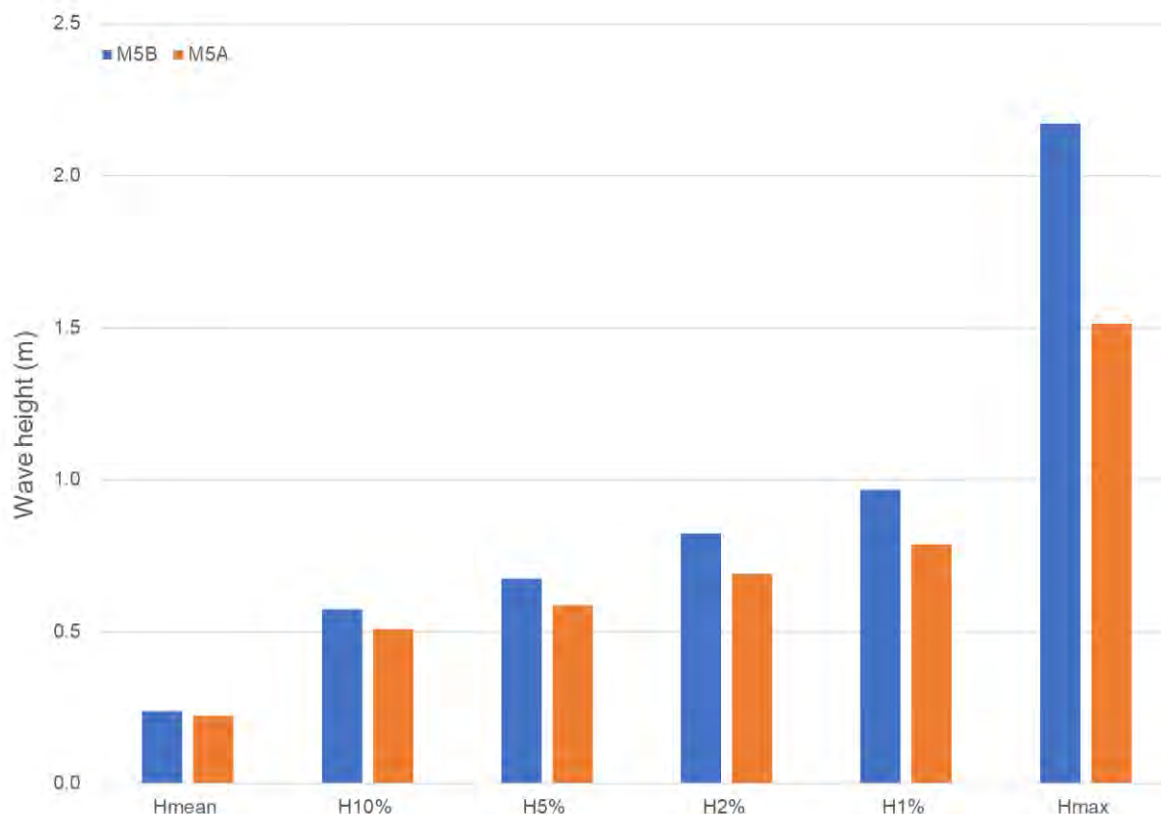


Figure 6.12 Comparison of transmitted wave conditions between breakwater M5A and M5B based on Test Condition 5, 500 year ARI waves and 100 year ARI + SLR water level

6.2.3 Rock armour damage assessment for test section M5B

Breakwater armour damage was recorded for each of the six test runs. Armour damage was recorded cumulatively for the high water level test conditions (Test Conditions 1, 3, 4, and 5), followed by a minor rebuild of the 6 to 10 T crest armour and the upper leeside 1 to 3 T armour. Subsequent low water level conditions (Test Conditions 6 and 8) were then conducted and damage again reported cumulatively. A summary of the damage assessment results is provided in Table 6.10. The associated types of overtopping (spray, white water jets, or complete submergence of the crest sheet) observed for each test are also provided in Table 6.10. A more extensive damage assessment for each test follows. The reported armour damage numbers are expressed relative to the total number of rocks within each respective zone of the test section, as shown in Figure 6.13.

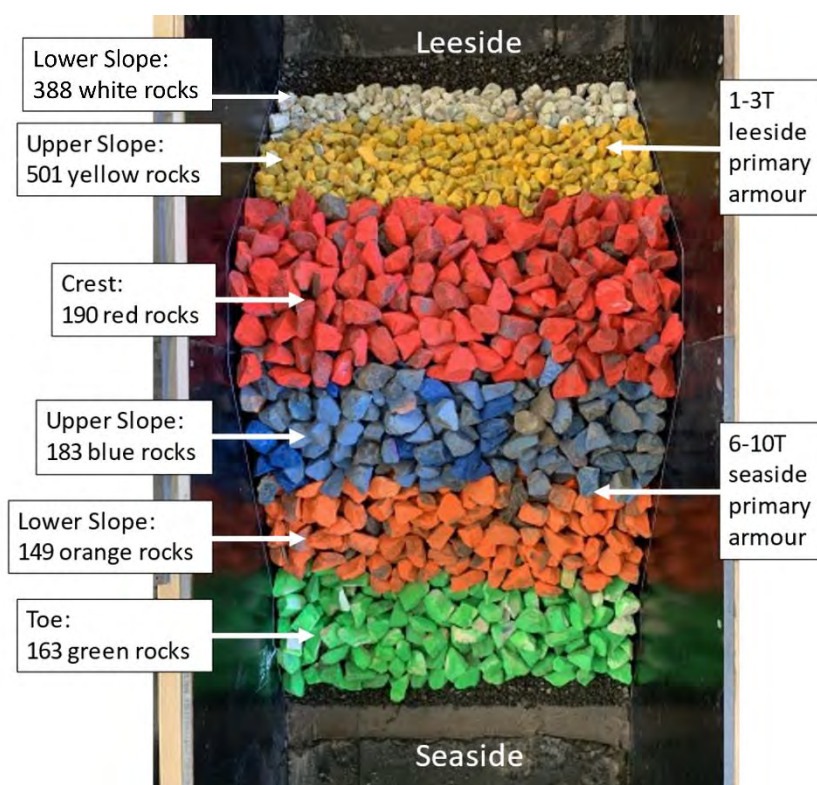


Figure 6.13 Completed M5B cross-section model, including armour stone counts

Table 6.10 Test Section M5B summary of damage assessment and overtopping type

Test Condition	Cumulative # rock movement > D ₅₀ (Seaside)	Cumulative # rock movement > D ₅₀ (Leeside)	Percentage of rock movement > D ₅₀ (%) (Seaside)	Percentage of rock movement > D ₅₀ (%) (Leeside)	Over topping (Type)
1	0	0	0%	0%	spray jets / partial sheet
3	3	5	0.4%	0.6%	jets / sheet
4	5	9	0.7%	1.0%	jets / sheet
5	17	22	2.5%	2.5%	jets / sheet
<i>Minor rebuild of structure</i>					
6	0	0	0%	0%	spray jets / sheet
8	1	0	0.1%	0%	

Notes:

*The percentage of rock movement on the seaside primary armour was calculated as the cumulated number of rocks that moved greater than D₅₀, divided by the total number (685) of primary armour rocks (see Figure 6.13).

**The percentage of rock movement on the leeside primary armour was calculated as the cumulated number of rocks that moved greater than D₅₀, divided by the estimated total number (889) of primary armour rocks exposed (above 3 m AHD) during testing.

Test Condition 1 (1 year ARI waves / 1 year ARI + SLR water level)

No damage was observed to the M5B structure during the 1 year ARI 'shakedown' test. Overtopping was limited to light spray landing on the leeward side of the 6 to 10 T crest armour during the largest 10% of waves. The still water level (SWL) was positioned 2 cm (model scale) above the orange/blue seaside primary armour transition (refer to Figure 6.13).

Test Condition 3 (100 year ARI waves / 25 year ARI + SLR water level)

The seaward 6 to 10 T primary armour experienced negligible damage (0.4%) during Test Condition 3, with only three rocks around the SWL becoming displaced and mobilised throughout the test. The weights of the three displaced rocks were 11.8 T, 7.1 T, and 7.4 T. In addition, two of the 6 to 10 T rocks on the crest were observed to be rocking. The leeward 1 to 3 T primary armour experienced limited damage (1.0%), with five rocks moving a distance greater than D_{50} during impact from overtopping jets. Impact from the overtopping jets was localised to the top two rows of exposed leeside primary armour. Another 15 rocks were observed to slightly shift and settle (moving a distance less than D_{50}) during the test. Figure 6.14 shows images of the test structure before and after this test.

Overtopping of the structure occurred during the largest 5% of waves and predominantly comprised white-water jets that landed on, or between, the leeside crest and leeside toe. The largest sets of waves resulted in partial submergence of the crest.

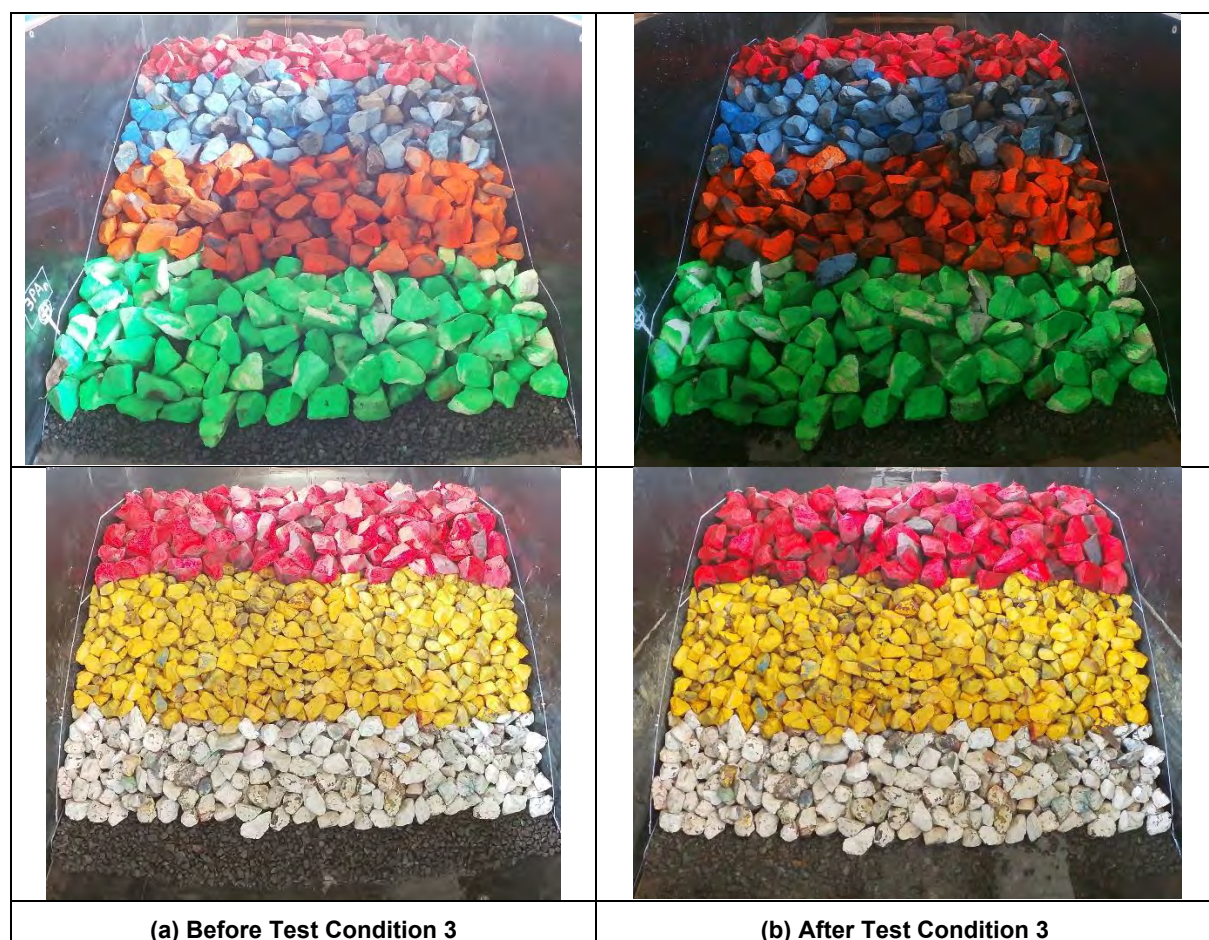


Figure 6.14 Images from Test Condition 3 showing seaside (top) and leeside (bottom) armour

Test Condition 4 (200 year ARI waves / 50 year ARI + SLR water level)

The 6 to 10 T seaside primary armour experienced negligible damage (0.7%) during Test Condition 4, as a further two rocks around the SWL became displaced and rolled down the seaside slope into more stable positions. A total of seven rocks (three around the SWL and four on the crest) were observed to be rocking. The 1 to 3 T leeside primary armour saw minimal damage (2.5%) with four rocks near the SWL becoming mobilised during overtopping jets and relocated a distance greater than D_{50} . Another 10 rocks around the impact zone of the overtopping jets were shifted a distance less than D_{50} . The top row of the leeside primary armour was partially exposed during overtopping events. Figure 6.15 shows images of the test structure before and after this test.

An increase in overtopping comprised of white-water jets both jumping and fanning through the crest, and complete submergence of the crest during the largest waves. Overtopping jets were observed to reach just beyond the leeward toe of the structure.

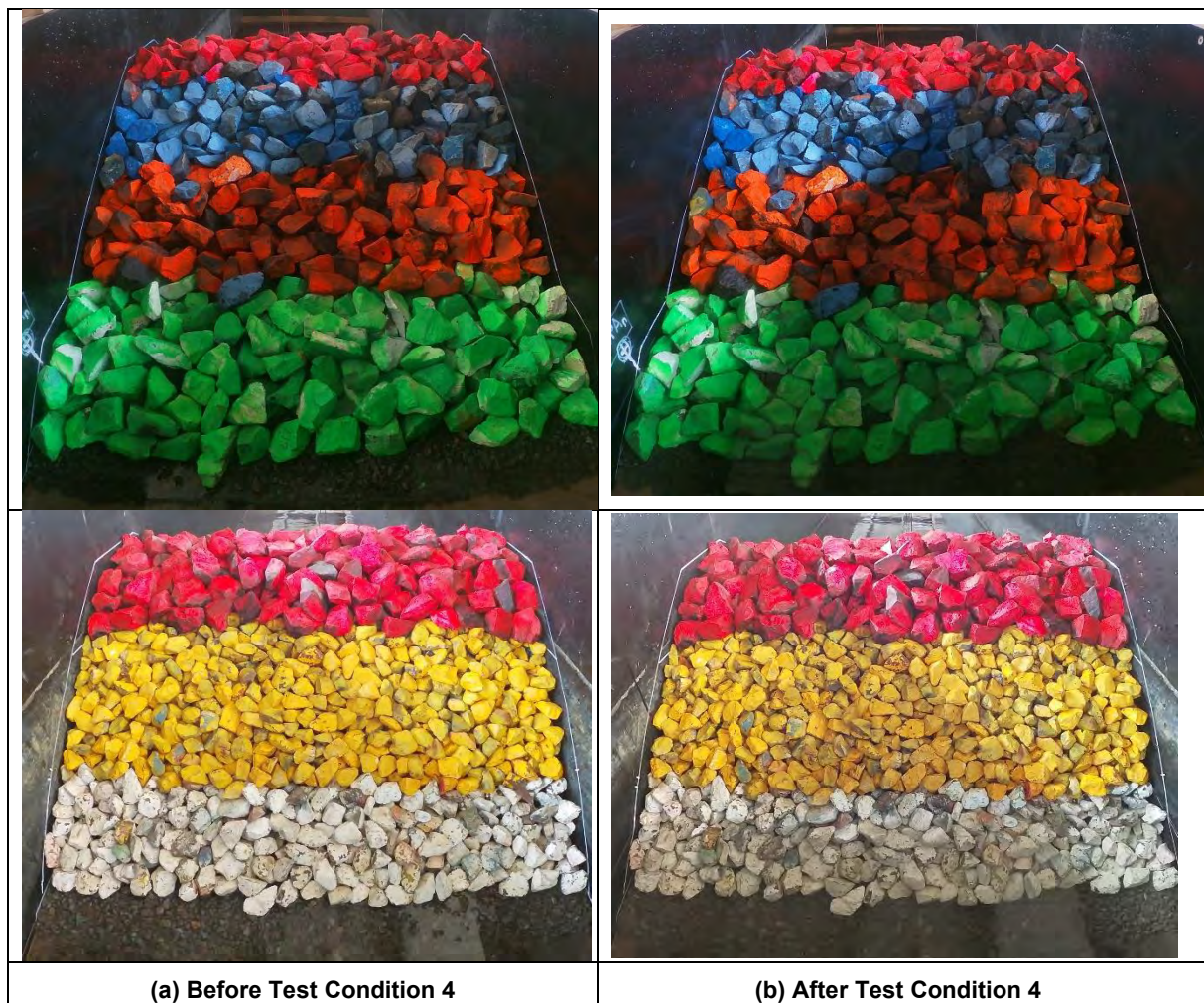


Figure 6.15 Images from Test Condition 4 showing seaside (top) and leeside (bottom) armour

Test Condition 5 (500 year ARI waves / 100 year ARI + SLR water level)

Minimal damage (2.5%) to the 6 to 10 T seaside primary armour occurred during Test Condition 5, comprised of a total of 12 rocks becoming displaced, 11 of which were positioned on the leeward crest and at either edge/boundary of the test structure (less stability from interlocking with adjacent armour stones), and one that was on the seaside slope. Once armour rocks were displaced, the impacted areas of armour were observed to be stable and did not continue to unravel. There was less armour stone rocking observed than for the 200 yr ARI test condition (three stones on the crest and one at SWL). An increase in damage (4.4%) to the 1 to 3 T leeside primary armour coincided with an increase in overtopping intensity and frequency, with 13 rocks moving a distance greater than D_{50} . Complete submergence of the crest occurred during the 5% largest waves. Figure 6.16 shows images of the test structure before and after this test.

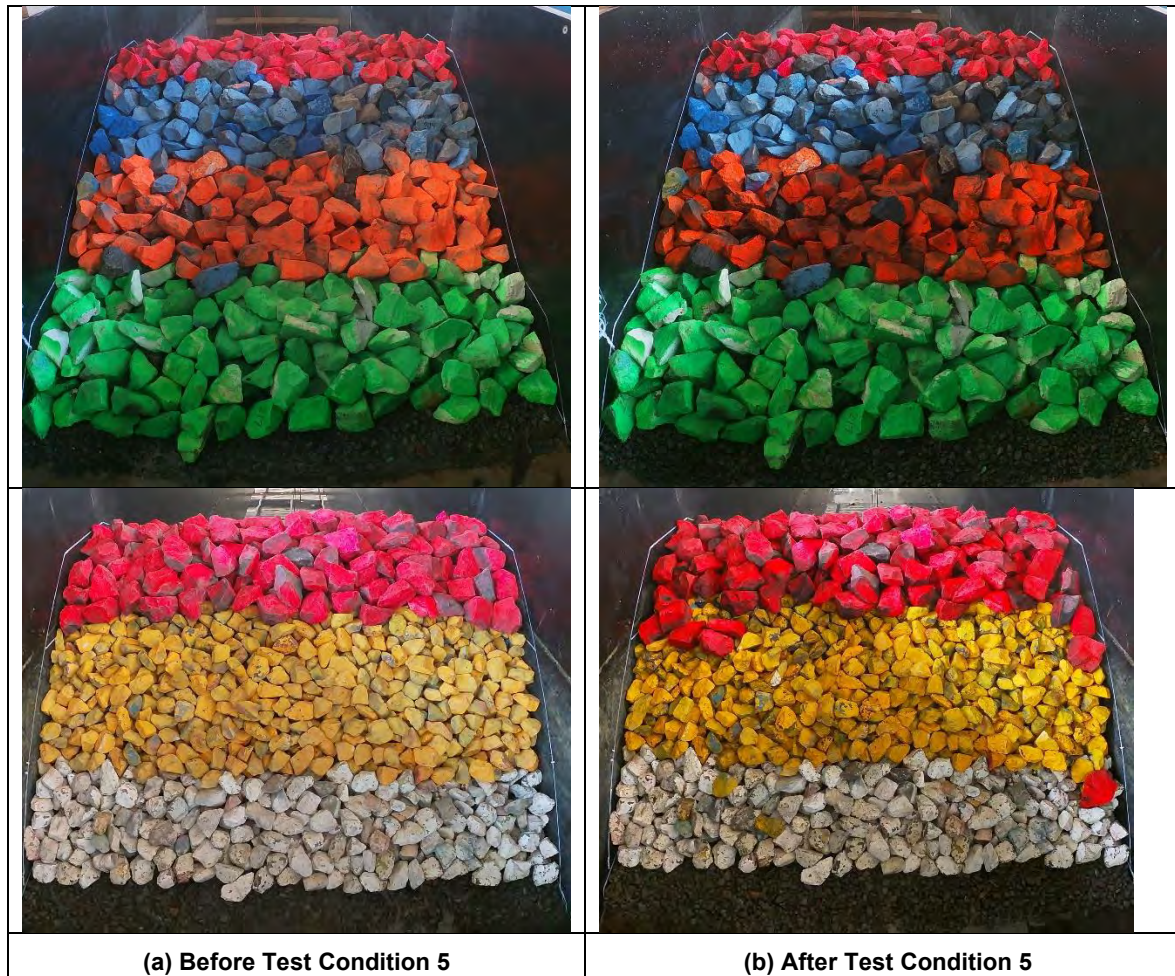


Figure 6.16 Images from Test Condition 5 showing seaside (top) and leeside (bottom) armour

Test Condition 6 (100 year ARI waves / LWL water level)

No damage or rocking of the 6 to 10 T seaside primary armour and no damage to the 1 to 3 T leeside primary armour occurred during Test Condition 6. The waves were breaking predominantly at the mid slope of the test structure (blue/orange transition) and overtopping was limited to spray reaching the leeward toe. Figure 6.17 shows images of the test structure before and after this test.

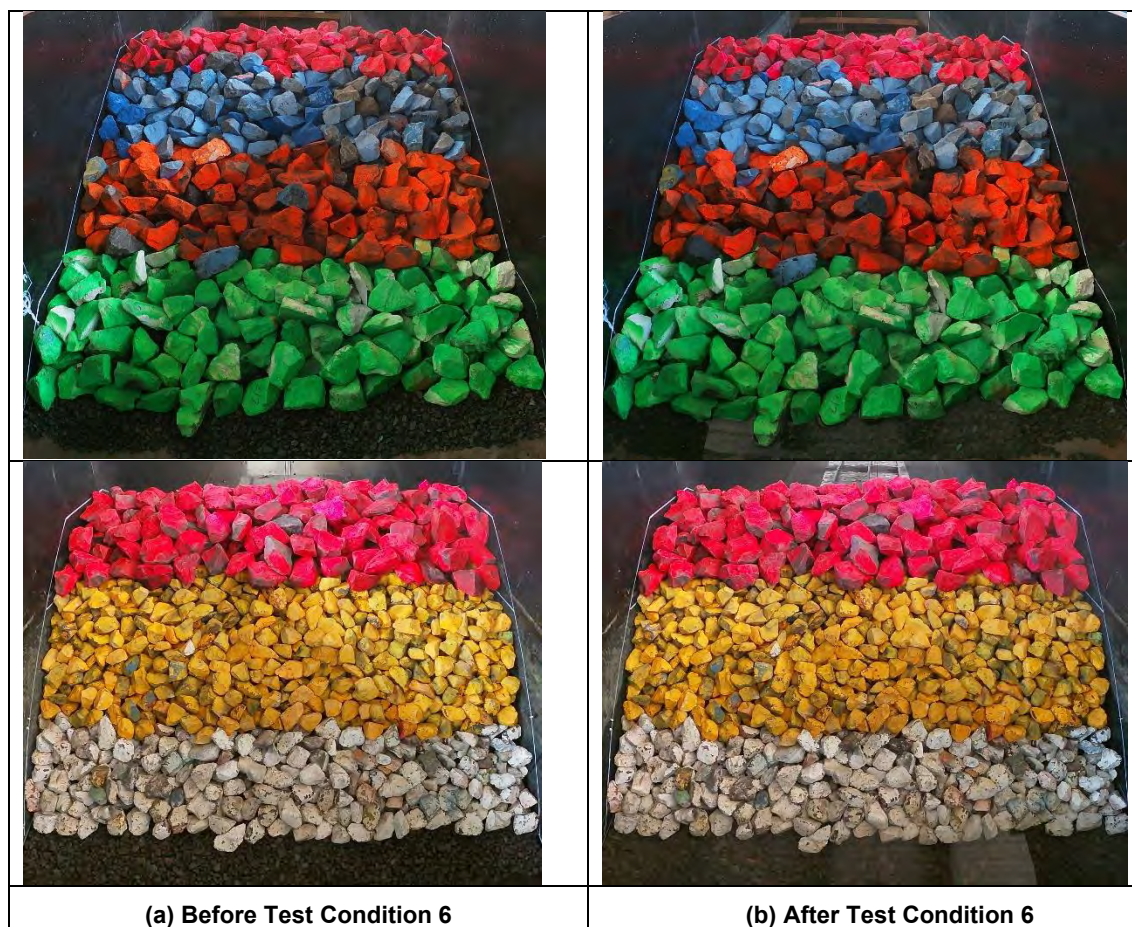


Figure 6.17 Images from Test Condition 6 showing seaside (top) and leeside (bottom) armour

Test Condition 8 (500 year ARI waves / LWL water level)

Negligible damage (0.1%) to the 6 to 10 T seaside primary armour was observed during Test Condition 8, with no damage to the toe and one rock at the SWL becoming displaced and proceeding to roll down the seaside slope. The same three loosened rocks as previous tests were observed to rock at the SWL.

No damage to the 1 to 3 T leeside primary armour was observed despite an increase in overtopping, with the largest 3% of waves resulting in complete submergence of the breakwater crest. The largest 1% of waves were observed to break seaward of the test section, prior to hitting the structure. Figure 6.18 shows images of the test structure before and after this test.

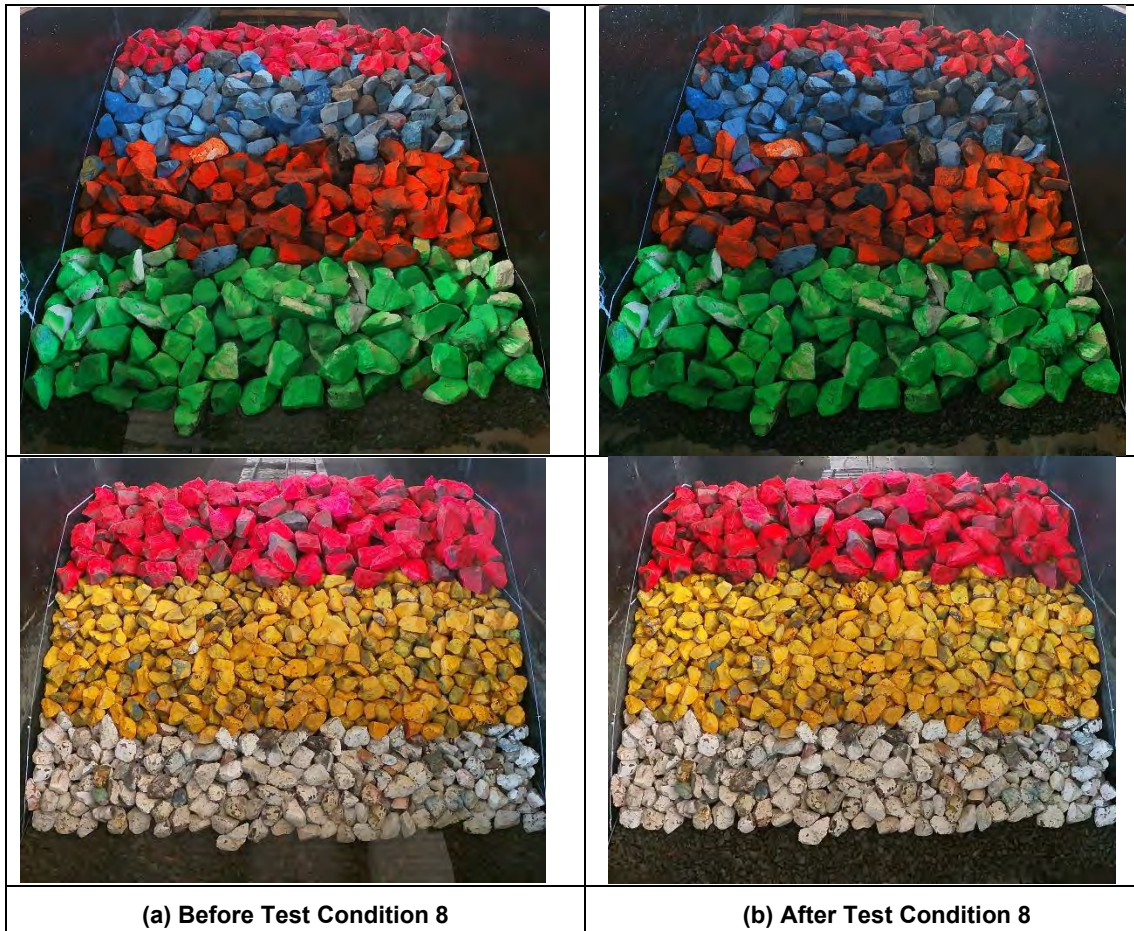


Figure 6.18 Images from Test Condition 8 showing seaside (top) and leeside (bottom) armour

7 Stage 2 quasi 3D testing

7.1 Overview of quasi 3D testing

Measurements collected during the quasi 3D Main Breakwater testing in Stage 2 of the program focussed on:

- Stability of rock armour around the roundhead of the breakwater
- Breakwater armour and slope transition design
- Wave transmission around the roundhead of the breakwater

The arrangement of wave probes during this Stage 2 of the program was similar to the prior Stage 1 2D testing, with some slight adjustments as shown in Figure 7.1. The locations of the wave probes comprising the *Offshore 3PA*, were kept unchanged from the calibration phase to allow direct comparison between wave conditions (with test structure in place) and the calibrated wave conditions (no structure). A single wave probe “*Seaward Toe WP*” was positioned 5 m (prototype) offshore from the seaward toe of the structure, to provide information on the wave conditions directly seaward of the breakwater. A single wave probe “*Head Toe WP*” was positioned 5 m (prototype) offshore from the toe at the tip of the roundhead, to provide information on the wave conditions passing the head of the breakwater. A third wave probe, “*Leeward Toe WP*” was positioned 15 m from the leeward toe of the structure, to assess wave transmission around the roundhead and in the lee of the breakwater.



Figure 7.1 Layout of wave probes for Stage 2 testing (quasi 3D breakwater roundhead)

The quasi 3D Main Breakwater roundhead test section was constructed within the flume and subjected to a reduced sequence of test conditions as requested by Cardno, with each test running for 3 hours (prototype) duration. Table 7.1 shows a summary of the test conditions that were conducted for the quasi 3D model. All four of the test sequences were conducted progressively/cumulatively, with no changes or rebuilding of the test structure between test runs.

Table 7.1 Summary of test conditions ran for section M5B

Test Condition	Wave Condition	WL Condition	H _{m0} (m)	T _p (s)	WL (m AHD)
1	1yr ARI	1yr ARI + SLR	1.86	6.5	3.44
3	100 yr ARI	25 yr ARI + SLR	2.98	8.8	4.46
5	500 yr ARI	100 yr ARI + SLR	3.36	9.6	5.05
6	100 yr ARI	LWL no SLR	2.98	8.8	2.5

7.2 Wave measurements and transmission for quasi 3D testing

A summary of the wave statistics obtained at each of the wave probes for all quasi 3D tests, is provided in Table 7.2. A clear trend of reducing wave height as waves passed around the head of the breakwater can be observed from the measured wave data, with wave heights at the leeward toe typically being only 45 to 60% of wave heights at the seaward toe. Wave transmission increased for tests with higher water level and higher wave overtopping rates.

Table 7.2 Measured wave statistics for quasi 3D testing

Test Condition	Offshore 3PA			Seaward Toe WP			Head Toe WP			Leeward Toe WP		
	H _s (m)	T _p (s)	H _{max} (m)	H _s (m)	T _p (s)	H _{max} (m)	H _s (m)	T _p (s)	H _{max} (m)	H _s (m)	T _p (s)	H _{max} (m)
1	1.9	6.6	3.6	1.8	6.7	3.9	1.7	6.4	3.3	0.8	6.3	1.3
3	3.0	8.7	6.8	3.4	8.9	6.4	2.9	8.7	5.4	1.8	9.5	3.9
5	3.1	9.4	6.8	3.6	9.2	6.9	3.2	9.5	5.8	2.1	9.5	4.2
6	3.2	8.7	6.6	3.3	8.8	5.4	2.9	8.8	4.8	1.5	9.8	3.0

Notes:

- Only the offshore wave measurement location (*Offshore 3PA*) allowed for the separation of the incident and reflected waves. The reflected wave signal could not be removed from the three additional single wave probes located closer to the tested structure and used to provide information on wave transmission, namely *Seaward Toe WP*, *Head Toe WP* and *Leeward Toe WP*. As such, the *Seaward Toe* measurements which contain both incident and reflected wave energy, are slightly larger than the *Offshore 3PA* values which have the component of reflected wave energy removed.

7.3 Rock armour damage assessment

Breakwater armour damage was recorded in a cumulative manner (i.e. the breakwater was not reconstructed between each test). The criteria for counting/quantifying rock damage was any rock that displaced a distance greater than the D_{50} of that armour size over the course of the test, consistent with the method previously used in the Stage 1 2D testing program. Subsequently, rocks that had moved a distance of less than D_{50} due to rocking or settling were not counted as damage. Damage as a percentage (%) was calculated as the cumulative total number of rocks that moved a distance greater than D_{50} , divided by the total number of rocks for the respective armour zone/class.

High resolution laser scanning was also used to assist in locating localised armour damage zones for the Stage 2 quasi 3D testing. Scans of the breakwater model were completed before each test run, and the measured point cloud data used to comparatively map differences in the breakwater armour surface between tests.

Photos before and after each test along with a heatmap highlighting rock movement areas are shown in Figure 7.2 to Figure 7.8. A summary of measured armour damage for all tests is presented in Table 7.3.

Most damage was observed to occur on the seaward breakwater trunk (with 4.1% of rock movement at the end of all four test runs for the pink area), as well as the leeward side of the roundhead in the area around the water level (with 4.8% of rock movement at the end of all four test runs for the purple area). The heat maps from the laser scanning (Figure 7.4 for example) provide a clear image of the localised areas of the structure that experienced the most armour movement.

While the measured armour damage, even within localised areas, remained within typically acceptable ranges for rock armoured rubble mound breakwaters, the damage should be considered in conjunction with accessibility constraints for future maintenance. It is also suggested that the leeward corner of the roundhead (purple zone within the model) be targeted for any larger armour stones that are won from the quarry during rock armour production.

Table 7.3 Summary of armour damage for quasi 3D testing

Breakwater zone	Total number of rocks in zone	<u>Test Condition 1</u>		<u>Test Condition 3</u>		<u>Test Condition 5</u>		<u>Test Condition 6</u>	
		Wave Condition: 1 YR ARI		Wave Condition: 100 YR ARI		Wave Condition: 500 YR ARI		Wave Condition: 100 YR ARI	
		WL Condition: 1 YR ARI + SLR		WL Condition: 25 YR ARI + SLR		WL Condition: 100 YR ARI + SLR		WL Condition: LWL no SLR	
		Cumulative Count	% Damage	Cumulative Count	% Damage	Cumulative Count	% Damage	Cumulative Count	% Damage
Seaward Slope (pink)	246	1	0.4	5	2.0	9	3.7	10	4.1
Roundhead 1 Slope (blue)	201	0	0.0	1	0.5	1	0.5	1	0.5
Roundhead 2 Slope (green)	177	0	0.0	2	1.1	2	1.1	2	1.1
Roundhead 3 Slope (purple)	207	0	0.0	4	1.9	8	3.9	10	4.8
Roundhead Total	585	0	0	7	1.2	11	1.9	13	2.2
Leeside Slope (blue)	303	0	0.0	2	0.7	3	1.0	3	1.0
Seaside Toe (orange)	87	0	0.0	0	0.0	0	0.0	0	0.0
Roundhead Toe (white)	452	1	0.2	1	0.2	1	0.2	1	0.2
Leeside Toe (white)	76	0	0.0	0	0.0	0	0.0	0	0.0



Figure 7.2 Quasi 3D model before (top) and after (bottom) Test Condition 1



Figure 7.3 Quasi 3D model before (top) and after (bottom) Test Condition 3

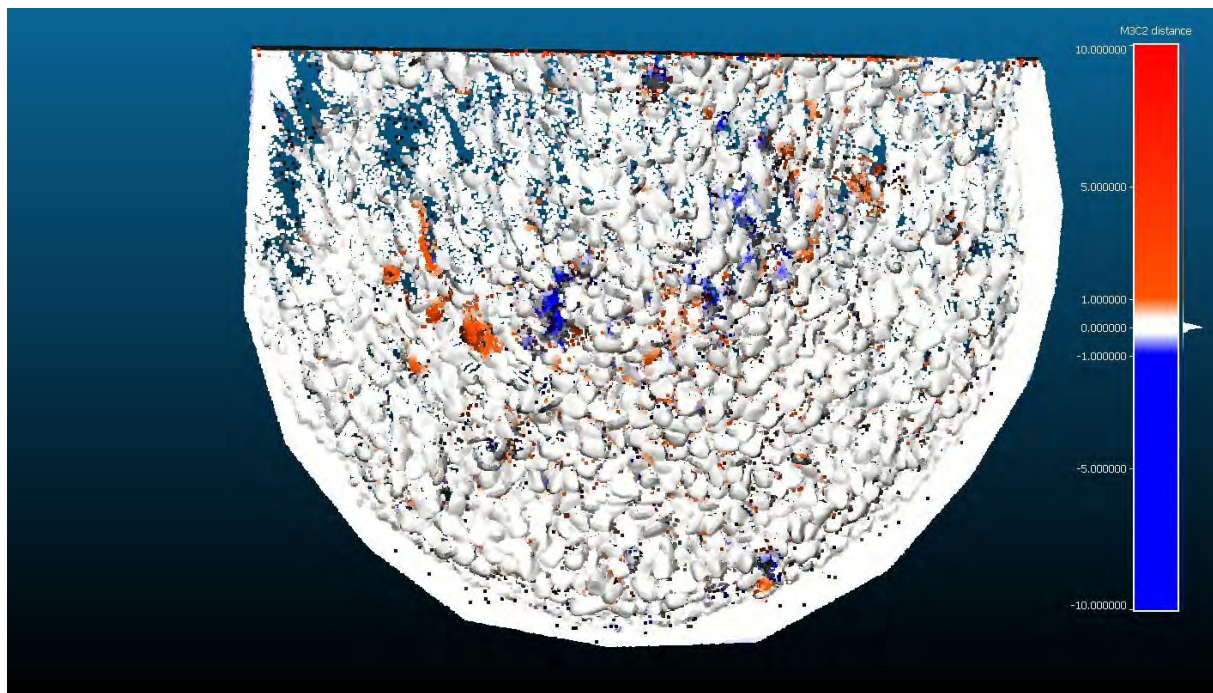


**Figure 7.4 Quasi 3D model armour displacement during Test Condition 3
(Red = accretion; Blue = erosion)**

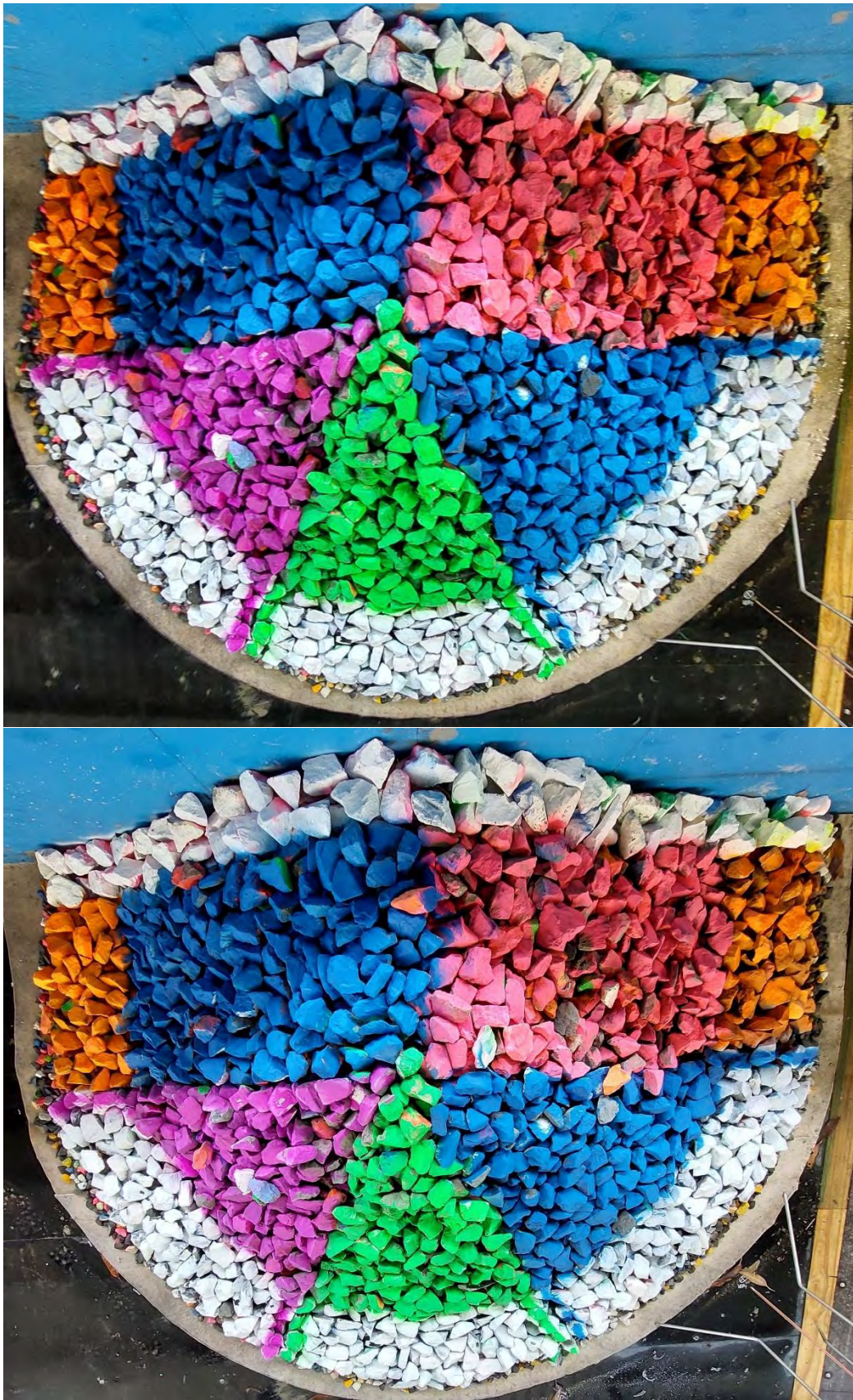


Figure 7.5 Quasi 3D model before (top) and after (bottom) Test Condition 5

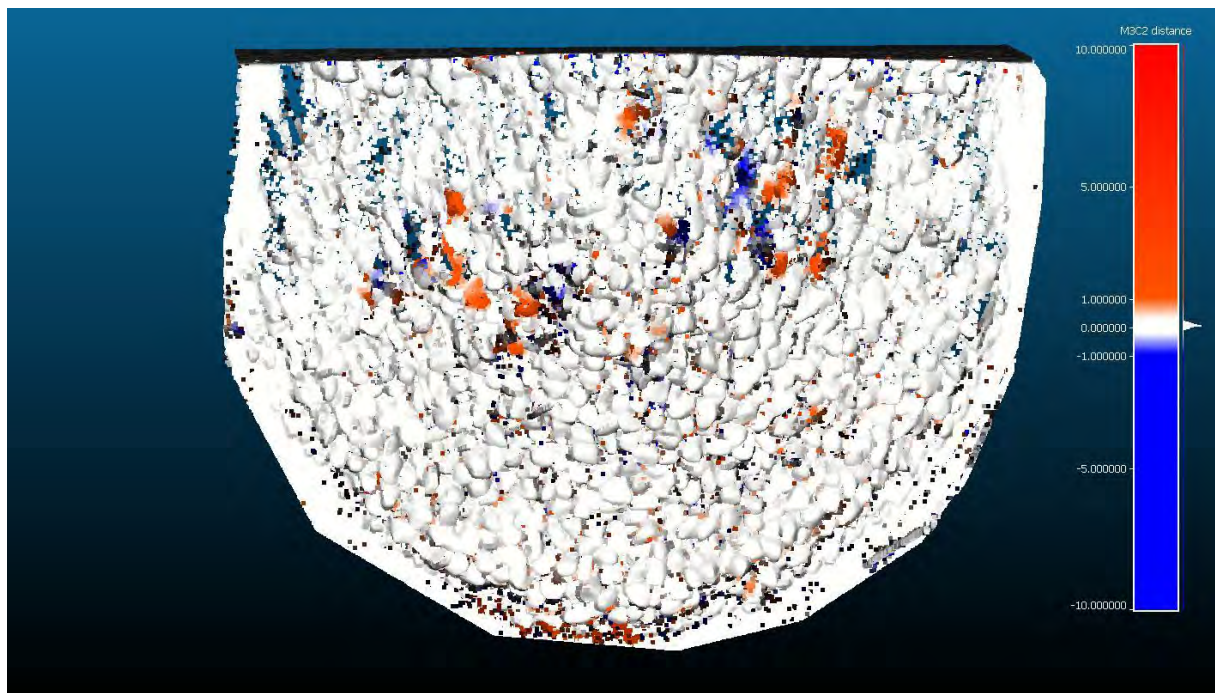


**Figure 7.6 Quasi 3D model armour displacement during Test Condition 5
(Red = accretion; Blue = erosion)**



Figure 7.7 Quasi 3D model before (top) and after (bottom) Test Condition 6

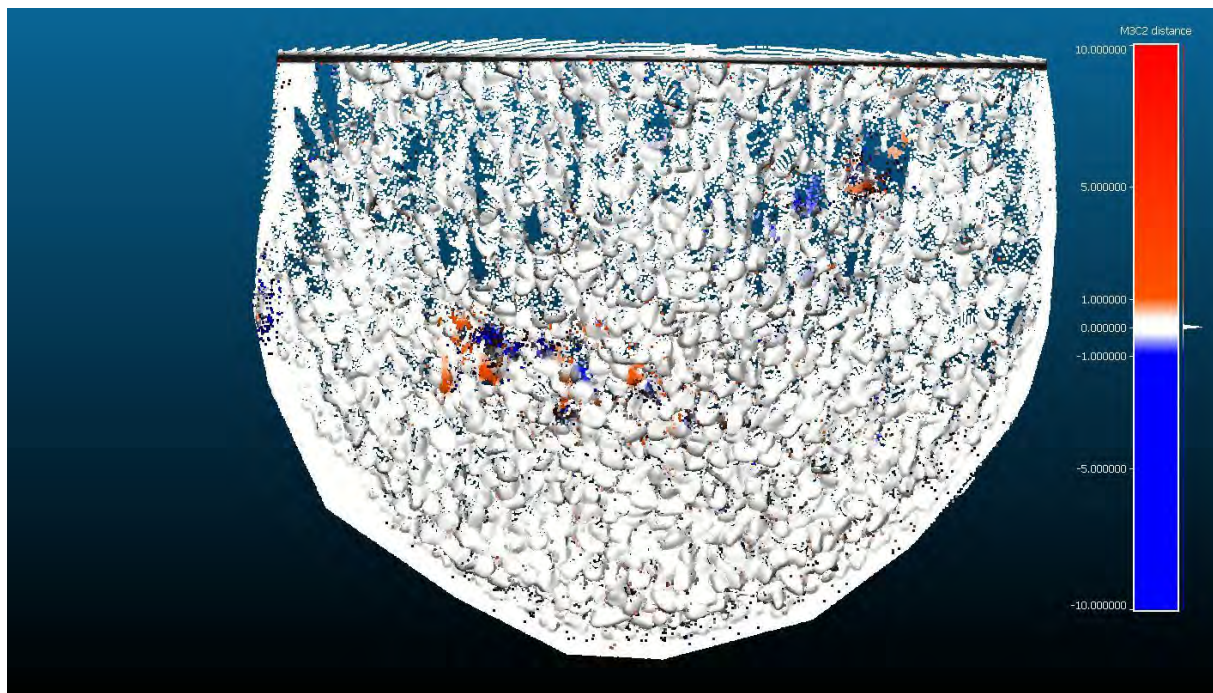


**Figure 7.8 Quasi 3D model armour displacement during Test Condition 6
(Red = accretion; Blue = erosion)**

8 Conclusions

The Water Research Laboratory was engaged by Cardno to complete a physical modelling investigation to support their design of the Main Breakwater, for a new harbour to be constructed at Mandorah, Northern Territory. Results from the physical modelling were provided to Cardno progressively throughout the modelling program, with this technical report presenting the overall modelling study.

The physical modelling program was conducted as 2D (Stage 1) and quasi-3D (Stage 2) flume modelling, and investigated a range of parameters for the new marine facilities development including:

- Nearshore wave processes
- Armour stability and cross section design for the Main Breakwater, including both trunk and roundhead sections
- Wave transmission through overtopping and diffraction around the Main Breakwater head

All modelling was conducted at a length scale of 1:28, and met the best-practice physical modelling guidelines provided by Hydralab III (2007a; 2007b).

Eight different test scenarios were identified by Cardno for testing within the physical model, and comprised various wave and water level conditions. The scenarios included extreme conditions (up to 500 year ARI), allowance for sea level rise, as well as consideration of large waves at low water level conditions for specific sections of the breakwater such as the toe armour.

The modelling program was conducted in two distinct stages:

- Stage 1: 2D modelling of a typical outer trunk section of the breakwater, which explored two potential designs for cross-section 5 of the breakwater (M5A and M5B)
- Stage 2: Quasi 3D modelling of the roundhead of the breakwater, which explored a single, optimised design

The initial series of 2D breakwater trunk testing explored the cross section design M5A, which had the following key characteristics:

- Crest level 8.8 m AHD
- Seaward slope armoured with 10 to 15 T armour stones
- Leeward slope armoured with 0.3 to 1.0 T armour stones

The results of the testing of M5A identified very low damage levels to the 10 to 15 T armoured seaward slope, however, the upper sections of 0.3 to 1.0 T armoured leeward slope experienced considerable damage through wave overtopping. This damage reduced the support to the crest armour and resulted in general slumping of the leeward side of the crest.

Cardno subsequently adjusted the design to M5B through the following cross-section changes:

- Modification to the crest and seaside primary armour
 - o Primary armour rock grading decreased from 10 to 15 T to 6 to 10 T
 - o Primary armour crest level lowered by 1.1 m (8.8 to 7.7 m AHD)
 - o Primary armour crest width increased by 2 m (4.8 to 6.8 m)

- o Toe crest width decreased by 1.2 m (5.1 to 3.9 m)
 - o Leese side crest armour extended down to the level of the core
- Modification to the leese side primary armour and addition of a secondary armour
 - o Primary armour rock grading increased from 0.3 to 1 T to 1 to 3 T
 - o Addition of 60 to 300 kg secondary armour layer
- Modification to the core
 - o Core crest width decreased by 1 m (8 to 7 m)
 - o Core crest level lowered by 0.2 m (4.1 to 3.9 m AHD)

Testing of the M5B cross-section design showed significantly improved performance, with only minimal damage observed even during extreme wave and water level conditions.

Stage 2 of the modelling program investigated the roundhead of the breakwater, and included three-dimensional wave-structure interactions through a quasi-3D modelling approach (2D bathymetry and waves; 3D model breakwater structure). The breakwater design was based on M5B detailing, such that the complete roundhead was armoured with 6 to 10 T stones and had a crest level of 7.7 m AHD. The roundhead was subjected to four different test scenarios, and the design generally performed well. Cumulative armour damage at the end of all four tests remained at less than 5%, even in localised areas. The most vulnerable section of the roundhead was identified as the upper leeward corner, and consideration should be given to targeting this zone with any larger “over-size” armour stones that are won during the quarrying of armour material.

9 References

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Appendix A Rock gradings

A1 2D Model - M5A

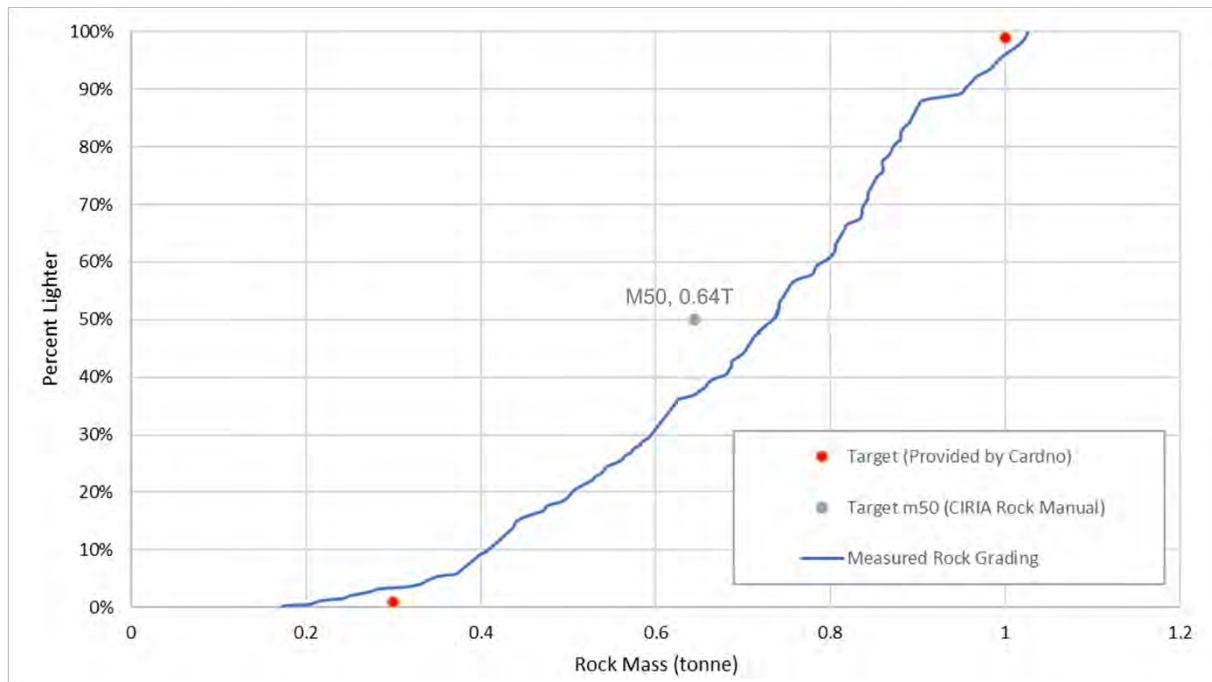


Figure A-1 Rock mass distribution and targets for the 0.3 – 1 t rock armour

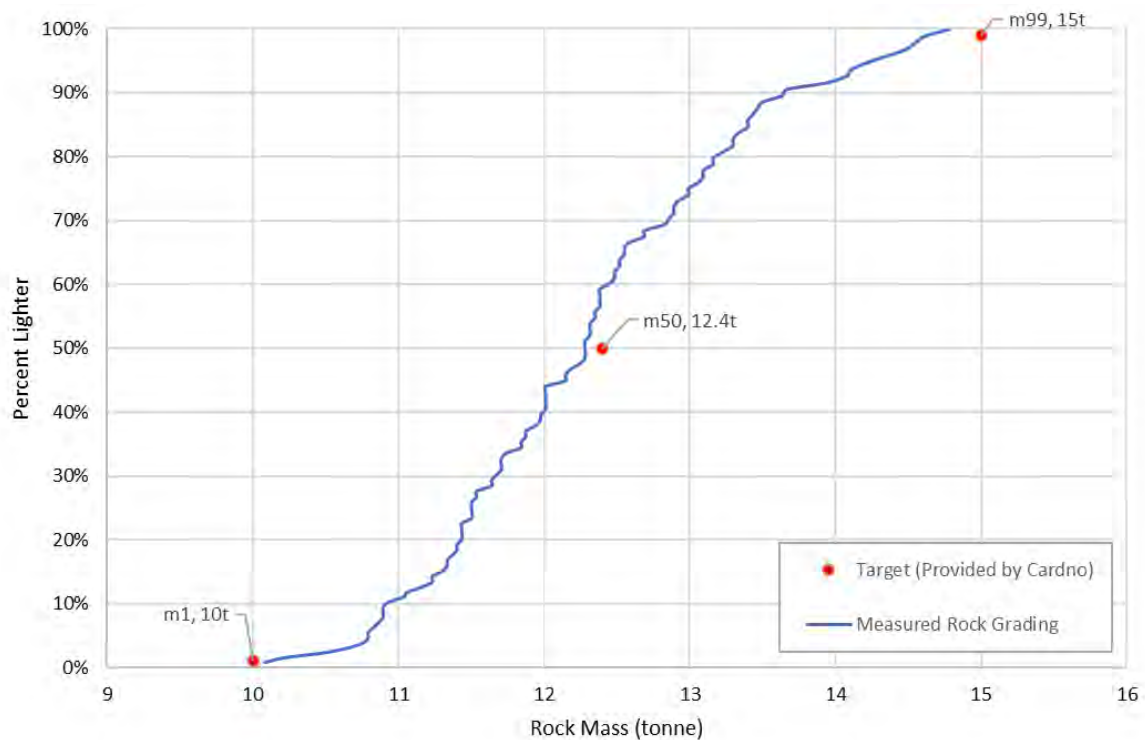


Figure A-2 Rock distribution and targets for the 10 - 15 t rock armour

A2 2D Model - M5B

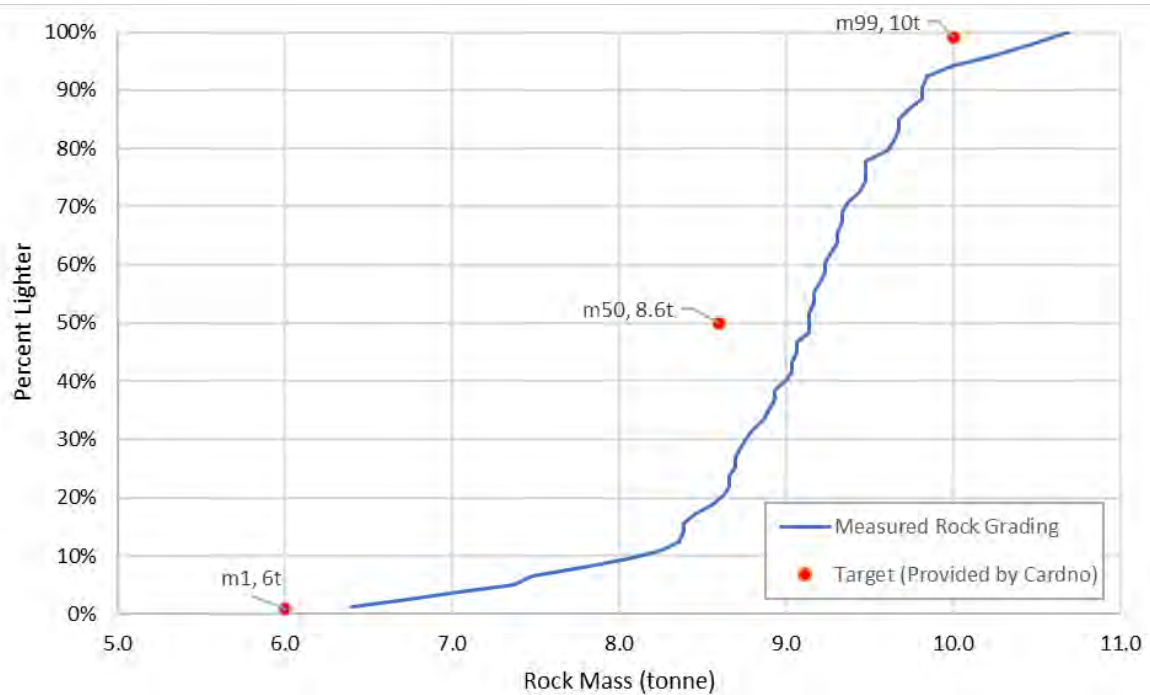


Figure A-3 Rock mass distribution and targets for the 6 – 10 t rock armour

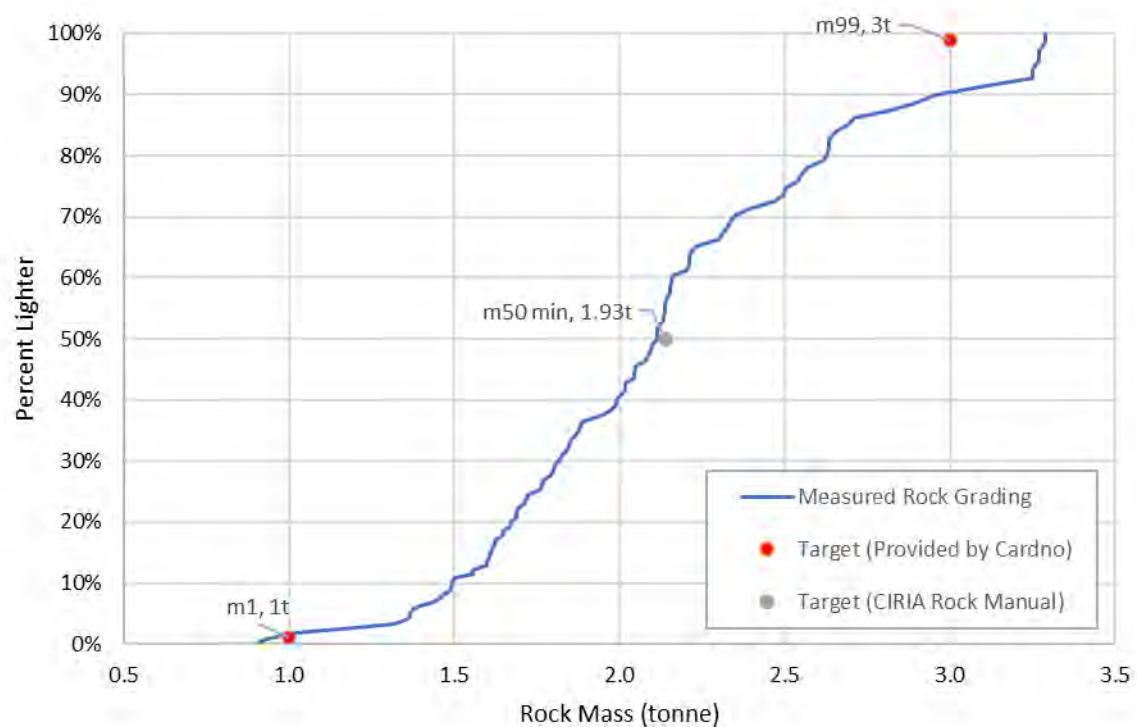


Figure A-4 Rock mass distribution and targets for the 1 – 3 t rock armour

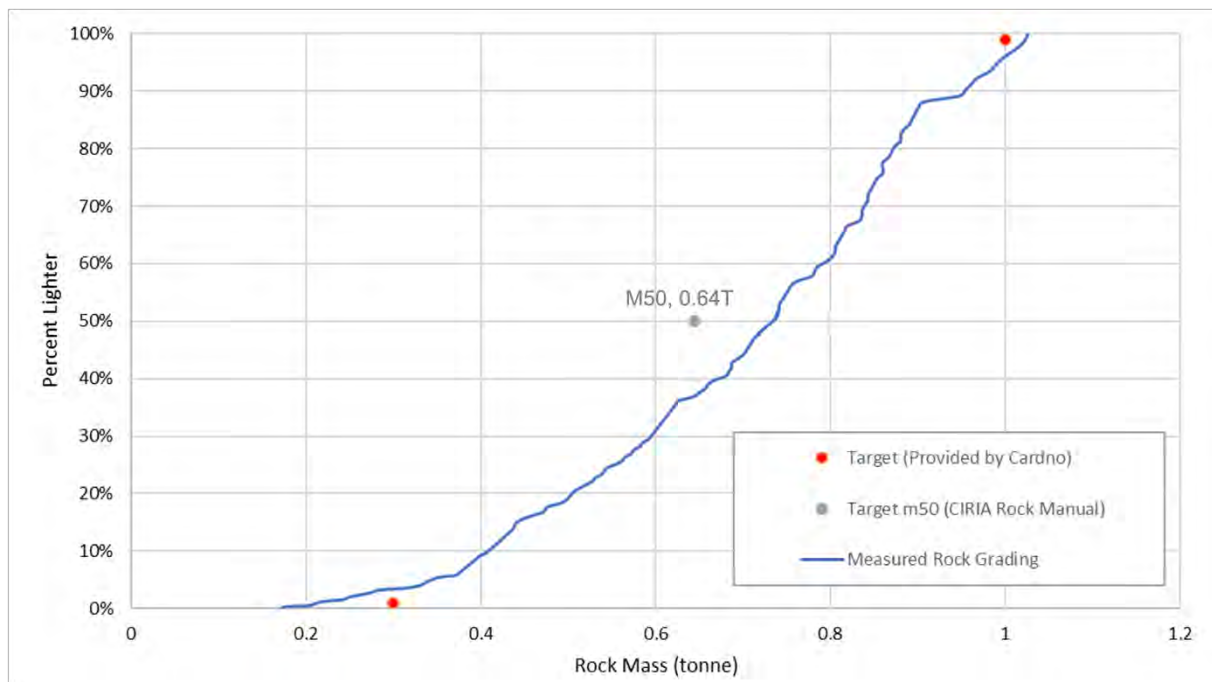


Figure A-5 Rock mass distribution and targets for the 0.3 – 1 t rock armour

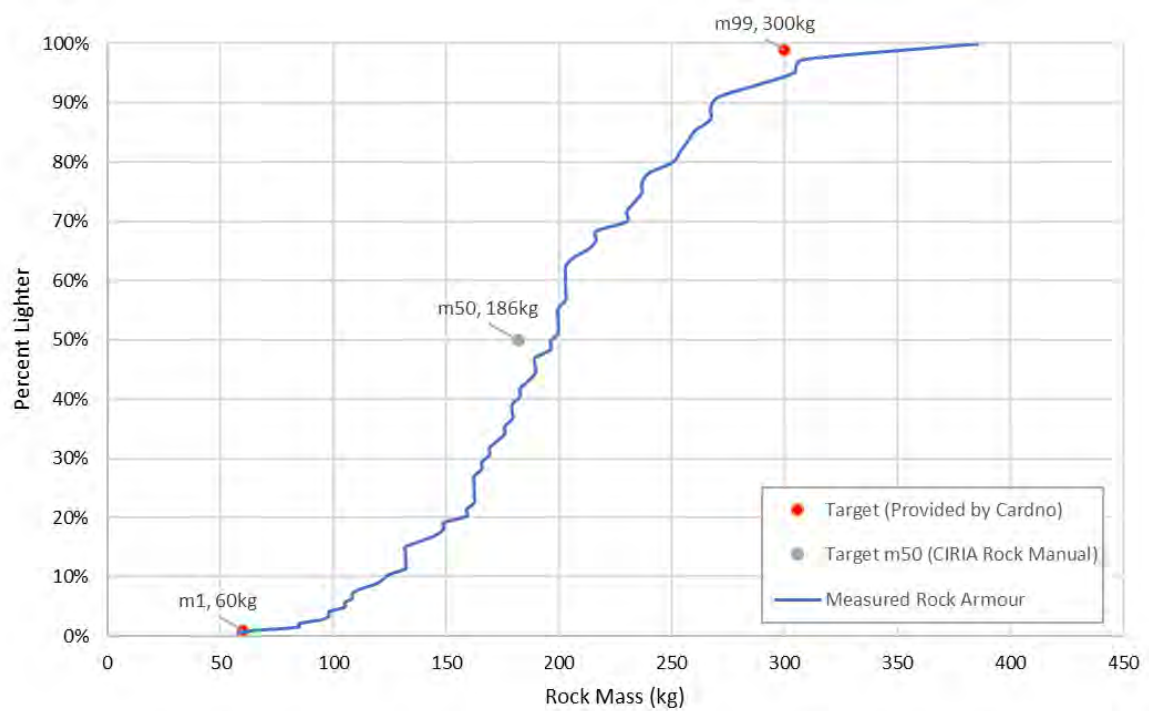


Figure A-6 Rock mass distribution and targets for the 60 – 300 kg rock armour

A3 Quasi 3D model

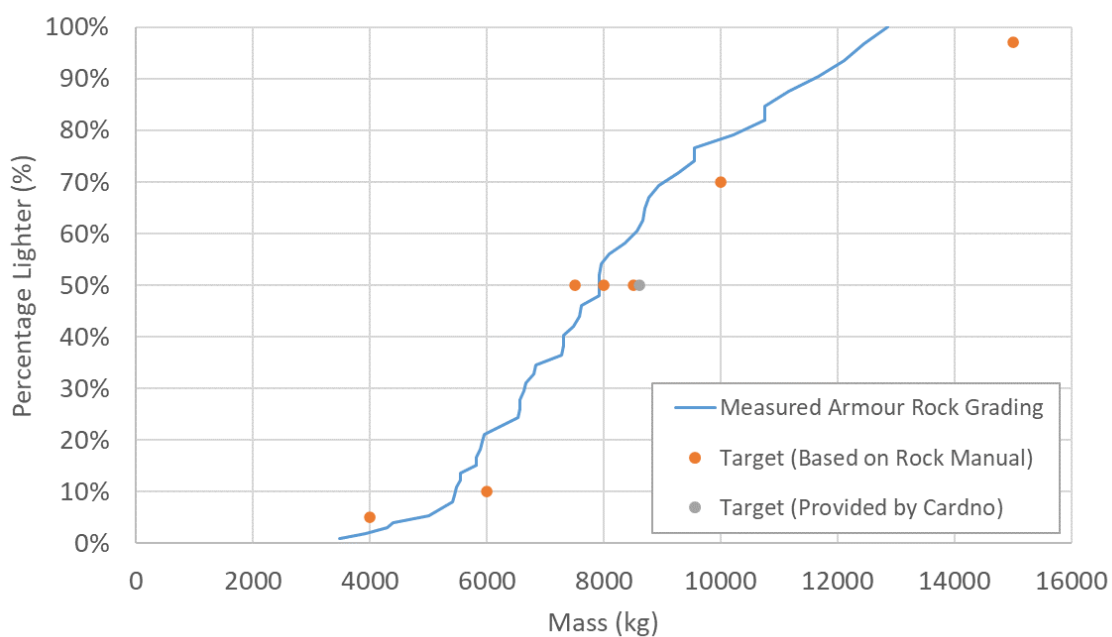


Figure A-7 Armour Layer 6 - 10 t Rocks Grading Curve

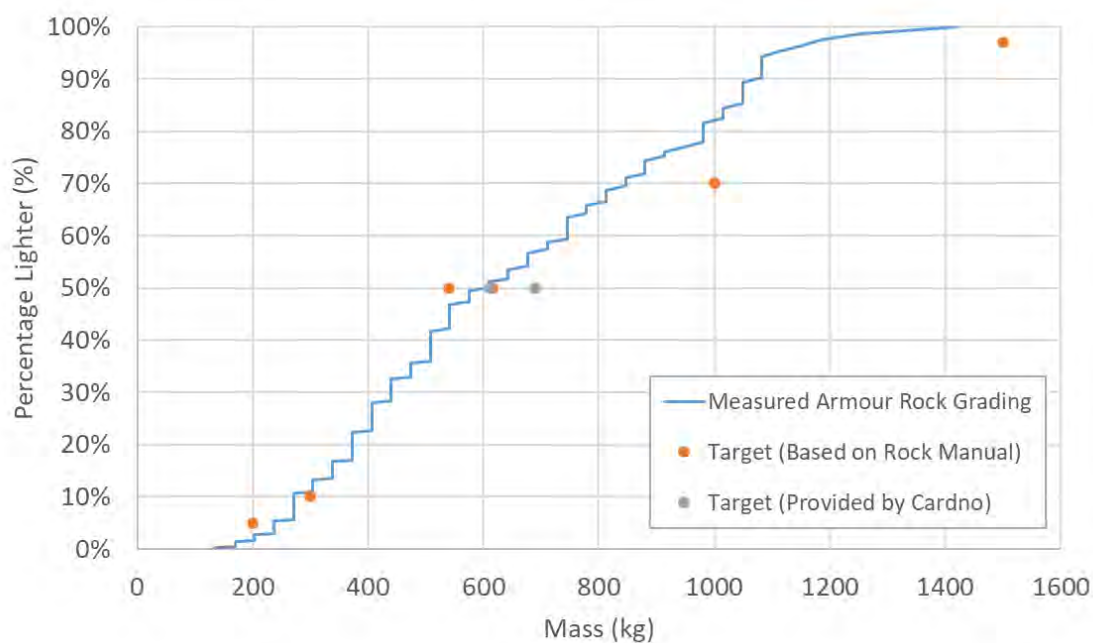


Figure A-8 Armour Layer 0.3 to 1 t Rocks Grading Curve