PLANS AND DOCUMENTS referred to in the PDA DEVELOPMENT APPROVAL

Approval no: DEV2021/1187

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Queensland Government

APPENDIX D

Coastal Processes Assessment

Redland Bay Barge Terminal Coastal Processes Assessment

Final Report

August 2021

Projex Partners

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Revision History

Revision Ref / Date Issued	Amendments	Issued to
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1.1 (DRAFT) / February 2020	Current afflux and reporting points	DB, JM
1.2 (DRAFT) / February 2020	Tidal datum	DB, JM, RK
2.0 / March 2020	Comments addressed	DB
2.1 / August 2021	RPEQ	DB

Contract

This report describes work commissioned by Daniel Berry of Projex Partners by email dated 17 December 2020. Michael Thomson and Daniel Rodger of JBP carried out this work.

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The methodology adopted and the sources of information used by JBP in providing its services are outlined in this report. The work described in this report was undertaken between December 2020 and January 2021 and is based on the conditions encountered and the information available during this period of time. The scope of this report and the services are accordingly factually limited by these circumstances.

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Acknowledgements

JBP would like to acknowledge the bathymetry data provided by Projex Partners, and the use of publicly available tide records from Department of Environment and Science for this project.

Executive Summary

This report has been prepared by JBPacific (JBP) on behalf of Projex Partners, to undertake a coastal modelling assessment at Redland Bay, Queensland. The study will support the detailed design of a proposed ferry terminal expansion at the Redland Barge Terminal.

A numerical model was used to simulate the tide characteristics within Moreton Bay, which was calibrated against tide data collected within Shorncliffe and Scarborough. It was used to simulate a double spring-neap tidal cycle, which included individual tides reaching a Mean High Water Spring level. In an existing scenario, during the flood (incoming) tide, water flowed southward towards the site and was deflected around the northern rockwall jetty, before forming a slow-moving eddy current at the seaward end of the ramp. During an ebb (receding) tide, the patterns are reversed, with the tide draining to the north, with a reversed eddy current forming in the nearshore.

A numerical model was used to simulate the tide characteristics within Moreton Bay. Two design scenarios were considered: conditions during a Mean High Water Springs (MHWS) tide, and peak conditions over a 14-day spring-neap tidal cycle.

- A Mean High Water Spring (MHWS) tide was first simulated to understand potential changes around the proposed upgrade. There was no measured change to water levels, however minor changes to current speeds, ranging from -0.04 to 0.01 m/s. These changes occurred during very low current speeds. The greatest reduction in current speed (-0.04 m/s) occurred during a current of 0.0 m/s. The greatest increase in current speed (-0.01 m/s) occurred during a current of 0.06 m/s.
- Further assessment of the magnitude and extent of these changes has been tested during a full 14-day spring neap tidal simulation. No changes to water levels were predicted within the tidal simulation. Minor changes to nearshore tidal currents were observed, which ranged between -0.1 m/s up to +0.04m/s adjacent to the proposed upgrade. Beyond 50m of the proposed terminal this impact reduces to between -0.02 to +0.02 m/s. At the seaward end of the public jetty there is no significant change in peak current speed. At this magnitude, these changes are not believed to have a significant change to coastal hydrodynamics, coastal sediment transport or scour and sedimentation patterns.

Nearshore wave conditions have been estimated using Delft3D. Only wind-driven wave effects have been considered, due to the protected location of the study area within the lower Moreton Bay channel system. For a 1% AEP event the nearshore wave conditions ranged between 1.4 to 1.7m, with peak periods between 3.5-5.0s. A range of scenarios have been considered to estimate stable rock sizes, using varied return periods, planning horizons and Safety Factors. Rock sizes range from a Dn_{50} of 0.8m for a present day, 2% AEP, 1.5 FoS, to a Dn_{50} of 1.1m for 2100 planning horizon, 0.5% AEP event with a 2.0 FoS. These can be considered for incorporation into the detailed design, which will need to consider relevant legislation, codes, standards and risks.

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Abbreviations

AEP .... Average Exceedance Probability
AHD .... Australian Height Datum
CEM .... Coastal Engineering Manual
CH .... Chainage
D50 .... Medium required rock diameter
DES .... Department of Environment and Science
DTMR .... Department of Transport and Main Roads
GBR .... Great Barrier Reef
HAT .... Highest Astronomical Tide
JBP .... Jeremy Benn Pacific (JBPacific)
LAT .... Lowest Astronomical Tide
LGA .... Local Government Area
M50 .... Medium required rock mass
MHWN .... Mean High Water Neap
MHWS .... Mean High Water Spring
MLWN .... Mean Low Water Neap
MLWS .... Mean Low Water Spring
MSL .... Mean Sea Level
MSLP .... Mean Sea Level Pressure
TC .... Tropical Cyclone
TIN .... Triangular Irregular Network
USACE .... United States Army Corps of Engineers
VDM .... Van der Meer equations

1 Introduction

This report has been prepared by JBPacific (JBP) on behalf of Projex Partners, to undertake a coastal modelling and engineering assessment at the Redland Barge Terminal, Queensland. The study will support the detailed design of a proposed expansion of facilities at 254 Esplanade, Redland Bay QLD 4165. The location of the sites is shown in Figure 1-1.

The study area is within Moreton Bay. At a regional scale it is offered protection from ocean swell by Moreton Island and North Stradbroke Island. However, the area is prone to extreme coastal processes, including strong currents within Moreton Bay, storm surges and wind-generated waves.

The proposed expansion includes an extension of the existing landing ramp, the addition of a second landing ramp, and upgraded rock walls on the eastern and northern side. This report supports the detailed design of the project by providing new coastal modelling on tides, hydrodynamics, extreme wave conditions, and stable rock size estimates.

This report contains the following sections:

- **Section 2: Background to coastal processes and available data**
- **Section 3: Tidal assessment**
- **Section 4: Extreme wave assessment**
- **Section 5: Rock size estimation**

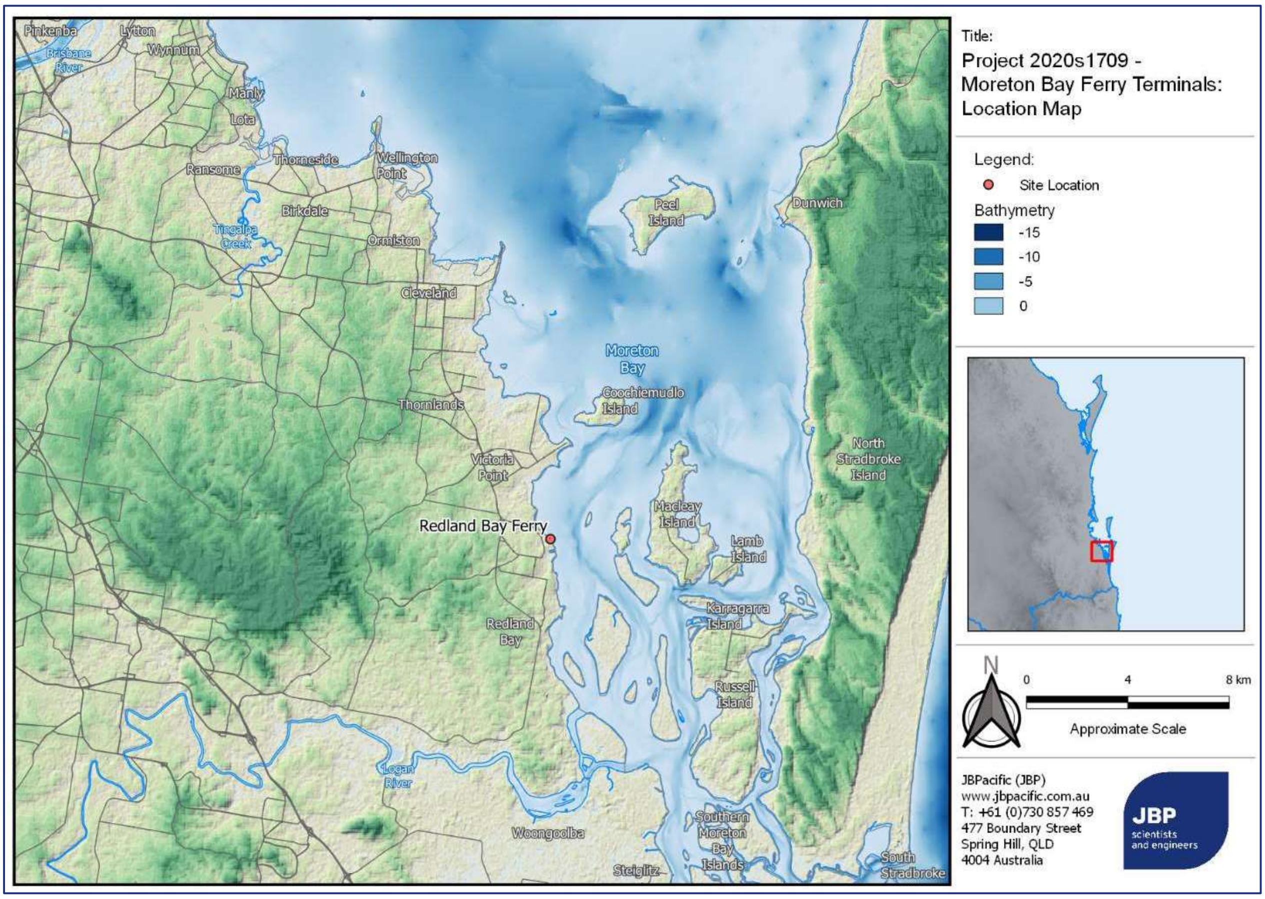


Figure 1-1: Redland Barge Terminal study site

2 Coastal processes and available data

Before undertaking any studies involving coastal modelling, it is first important to consider the underlying coastal processes affecting the site. Moreton Bay experiences a range of hydrodynamic, waves, and morphologic processes that are linked through dependant and independent variables. This includes the underlying astronomical tide, the passage of local storms and cyclones, the interaction of storm surges along the open coastline and within the bay, the local wave climate and any sheltering provided by inner bay islands such as Coochiemudlo, Macleay and Garden Island. A range of these coastal processes are shown in Figure 2-1.

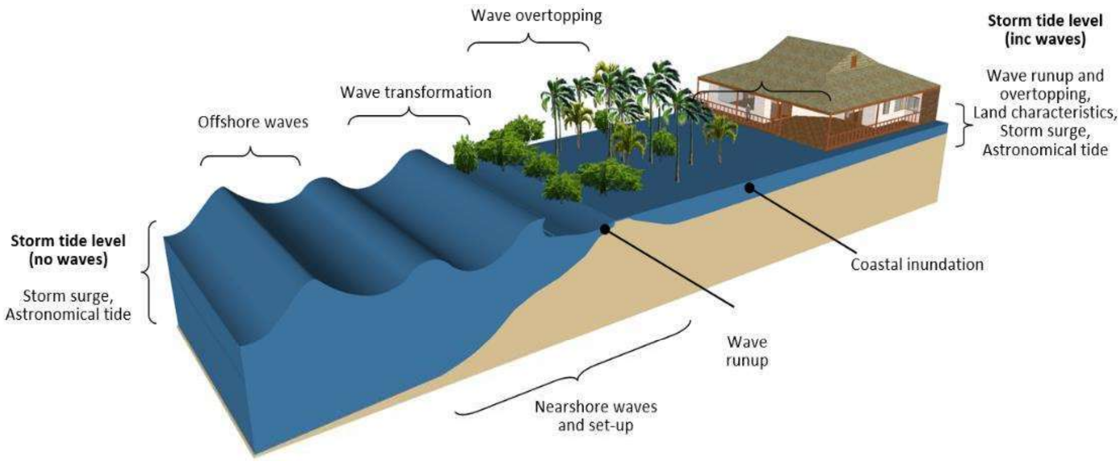


Figure 2-1: Drivers of coastal risk

The way in which the different coastal processes interact will determine the tidal and wave conditions experienced at any location. As shown in Figure 2-1, these may include the following:

- Astronomical tide: This is the regular periodic variation in water levels due to the gravitational effects of the moon and sun, which can be predicted with generally very high accuracy at any point in time (past and present) if sufficient measurements are available. The highest expected tide level at any location is termed the Highest Astronomical Tide (HAT) and occurs once each 18.6 year period, although, at some sites, high tide levels similar to HAT may occur several times per year and the level of HAT is often exceeded by the combination of a high tide and a non-astronomical weather-related event.
- Storm surge: This is the combined result of the severe atmospheric pressure gradients and wind shear stress of the storm acting on the underlying ocean. The storm surge is a long period "wave" capable of sustaining above-normal water levels over several hours or even days. The wave travels with and ahead of the storm and may be amplified as it progresses into shallow waters or is confined by coastal features. The magnitude of the surge is affected by several factors such as storm intensity, size, speed and angle of approach to the coast and the coastal bathymetry.
- Wave setup: As waves break, they create a localised effect to increase the sea level, known as breaking wave setup. It predominately occurs at a sloping beach or structure and becomes less significant within river mouths or protected low-lying mangrove or swampy lands.
- Nearshore waves and wave runup: If broken waves reach the shoreline any residual energy may intermittently run up and down the beach face, known as wave runup. This may cause localised impacts as waves can reach elevations higher than the underlying storm tide level. The vertical elevation the waves may reach will be dependent on the slope of the shoreline, the porosity, vegetation and the coastal (wave and sea) conditions.

The scope of this project considers tidal processes and extreme wave conditions only.

2.1 Available data

A range of studies and datasets are available at a regional scale throughout the Moreton Bay region. These provide information on tides, storm tides, waves and the underlying bathymetry.

2.1.1 Offshore Tidal Planes

Offshore tidal data is available through the global TPXO tidal model[1], which has a typical spatial resolution of around 4km along the Queensland coastline. The TXPO v8 model was used to extract tidal harmonics for eight primaries (M2, S2, N2, K2, K1, O1, P1, Q1), two long periods (Mf, Mm) and three non-linear (M4, MS4, MN4) constituents. These were extracted immediately offshore of Moreton and North Stradbroke Islands (to the north of the site) and between North Stradbroke and South Stradbroke Islands (to the south of the site).

2.1.2 Nearshore Tidal planes

Tidal planes are published in the 2021 Queensland Tide Tables[2] for Redland Bay (-27°37', 153°18') and are presented in Table 2-1, including an allowance for sea level rise under a 2070 and 2100 planning horizon of + 0.5m and 0.8m respectively.

Table 2-1: Tide levels from QLD Tide Tables, including future sea level rise estimates

Tidal Planes at Redland Bay				
Tidal Plane	2020 (mLAT)	2020 (mAHD)	+0.5m (circa 2070)	+0.8m (circa 2100)
HAT	3.0	1.6	2.1	2.4
MHWS	2.4	1.0	1.5	1.8
MHWN	1.9	0.6	1.1	1.4
MSL	1.4	0.0	0.5	0.8
MLWN	0.8	-0.5	0.0	0.3
MLWS	0.4	-0.9	-0.4	-0.1
LAT	0.0	-1.3	-0.8	-0.5
AHD	1.3			
Note: AHD height above survey level*	2.2			
Note: Survey reference level	3.5			
AHD quoted based on Permanent Mark 42645, referenced from MSQ tide tables.				

2.1.3 Storm tide levels and extreme wave conditions

Storm tide levels for a range of return periods and planning horizons are believed to be estimated within the Redlands Storm Tide Inundation Study (Cardno 2011), however this could not be provided by Council. Instead, extreme storm tide level are based on review of Council planning information and new analysis of wave estimates.

Future (2100), 1% Annual Exceedance Probability (AEP) storm tide levels have been accessed through Councils Red-e-map and PD Online system for the Stradbroke Ferries Barge Ramp Redland Bay (260 Esplanade Redland Bay). The 2100 1% AEP storm tide level is 3.23 mAHD. This is the maximum storm tide level predicted to be reached on the subject property during a 1% AEP Storm Tide event in the year 2100. It considers the effects of climate change through the inclusion of a sea level rise of 0.8m and an increase in cyclone maximum potential intensity of 10%.

The present day 1% AEP storm tide has been estimated based on the difference of the future 1% AEP conditions above the future Highest Astronomical Tide (HAT). Applying the same storm surge to present day HAT levels will result in a slightly conservative estimate, due to the future conditions

1 OSU (2020) TPXO Global Tidal Models, Oregon State University, USA.

2 Maritime Safety Queensland 2020, Queensland Tide Tables. Published at: https://www.msq.qld.gov.au/-/media/MSQInternet/MSQFiles/Home/Tides/Online-tide-tables/2020/2020queenslandtidetables.pdf?

including the effects of increased cyclone intensity. Finally, a 2070 storm tide level was interpolation from the two planning horizons. The three storm tide estimates are therefore:

- 2100, 1% AEP storm tide: 3.23 mAHD (from Council)
- 2070, 1% AEP storm tide: 2.96 mAHD (interpolated)
- Present day, 1% AEP storm tide: approximately 2.51 mAHD (approximate estimation)

2.1.4 Recorded tide levels

Recorded water level information is available for two locations within Moreton Bay, through the storm tide monitoring network run by the Department of Environment and Science (DES). This data was used for the calibration of a tidal model.

Data is available at two locations:

- Scarborough monitoring site
  - Date of installation: 29 April 2015
  - Location: Scarborough boat harbour
  - Coordinates: Latitude: 27° 11.616' S, Longitude: 153° 6.557' E
- Shorncliffe monitoring site
  - Date of installation: 25 May 2016
  - Location: Shorncliffe Pier
  - Coordinates: Latitude: 27 ° 19.2589 E, Longitude: 153 ° 05.2311 S

2.2 Topography and Bathymetry

Four elevation datasets have been used within the study.

- Bathymetry within Moreton Bay is based on the DeepReef 30m dataset[3]. The GBR30 bathymetric dataset was developed in collaboration between James Cook University, Geoscience Australia, and the Australian Hydrographic Office to compile all available digital bathymetry data to develop regional-scale, 30m resolution grids. This contains deep-water multibeam surveys, airborne lidar bathymetry and chart data, all edited as point clouds to remove noise, and merged into a consistent WGS84 horizontal datum, and an approximate mean sea level vertical datum.
- Nearshore bathymetry was provided by Projex Partners for the project[4]. High detail contour data was provided, which was used to develop a nearshore bathymetric elevation model.
- An elevation model of the proposed ferry expansion was provided by Projex Partners for the project. This includes a ramp extension, batters and dredged area.
- Topographic data is based on 1m LiDAR elevation dataset sourced from the Queensland Department of Natural Resources, Mines and Energy. This data was captured between June and October of 2014 using airborne laser scanning.

3 Beaman, R.J. (2018) "100/30 m-resolution bathymetry grids for the Great Barrier Reef", SSSI Hydrography Commission Seminar, March 2018. Surveying and Spatial Sciences Institute (SSSI), Canberra, Australia.

4 Supplied by Project Partners (Daniel Berry) on 17 December 2020. " [#497-001] Redland Barge Terminal - Coastal - Go Ahead"

3 Tidal estimation and afflux

Tidal processes throughout the study area have been estimated through numerical modelling, which was calibrated at the two storm tide monitoring stations within Moreton Bay.

3.1 Model development

Numerical modelling has been undertaken using Delft3D, an integrated model capable of estimating tides, extreme water levels, currents, cyclones and wave conditions. It is an open-source model[5]. As schematised in Figure 3-1, several modules of Delft3D can be used within modelling scenarios. For this tidal assessment the Delft3D-FLOW module was used to simulate hydrodynamics. The Delft3D-Wave module was used separately to estimate wave conditions (see Section 4).

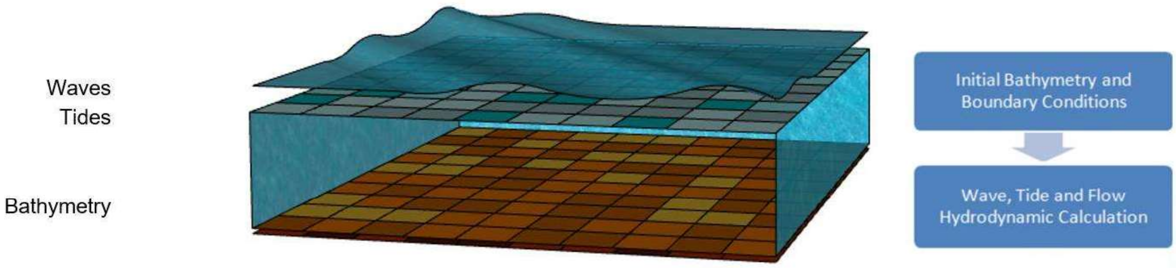


Figure 3-1: Delft3D hydrodynamic and wave calculations

3.2 Modelling extent

The model spans approximately 80km, covering Moreton Bay (approximately 1500km^2). Three tidal boundaries have been used, positioned along the following channels:

- The northern channel adjacent to Bribie Island
- The eastern channel between Moreton and North Stradbroke Islands
- The southern channels between North and South Stradbroke Islands.

A nested approach was used, where the Delft3D tidal model was established to use external boundary conditions from the TXPO v8 tidal harmonics model. A high detail sub-model was then established at the Redlands Ferry Terminal site.

The Moreton Bay model was constructed using a computational grid with a regular spatial resolution of 100m. The high-resolution nested model at the study site has a spatially-varying grid with a minimum cell size of 1.25m. A bathymetry grid was constructed for the model domain based on several sources of data.

- Offshore data is based on the DeepReef 30m bathymetry dataset.
- Nearshore bathymetric data is based on survey provided by Projex Partners for this project.
- Above-ground features are based on 1m LiDAR data accessed through ELVIS.

This data was processed and merged over the Delft3D grid. Once merged, the grid was inspected to ensure that the locations where datasets intersected did not contain abnormal changes in bathymetry, which could distort coastal processes. Any gaps in the bathymetry were smoothed and averaged with the adjacent grid cell.

5 Website: http://oss.deltares.nl/web/delft3d/download

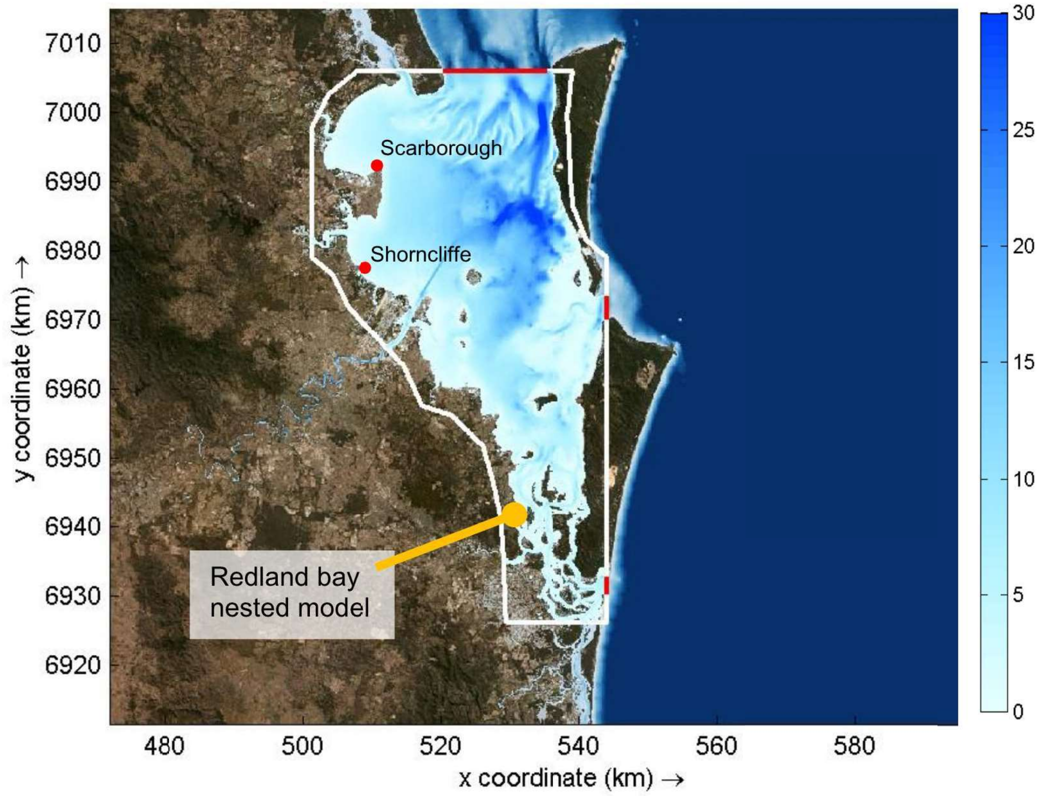


Figure 3-2: Computational grid extent and bathymetry for overall Delft3D model (top) and nested Redland Bay grid (bottom), open boundaries shown in red.

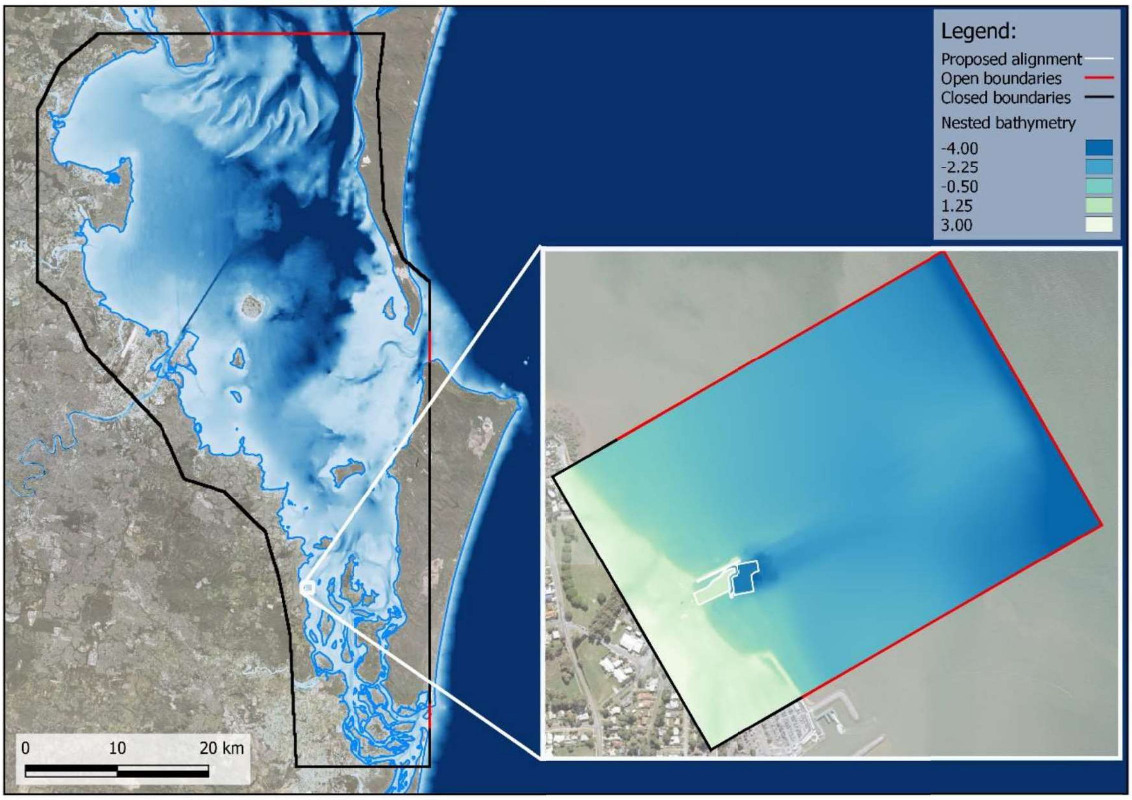


Figure 3-3: Nested model domain and locations of tidal current extraction points

3.3 Boundary conditions

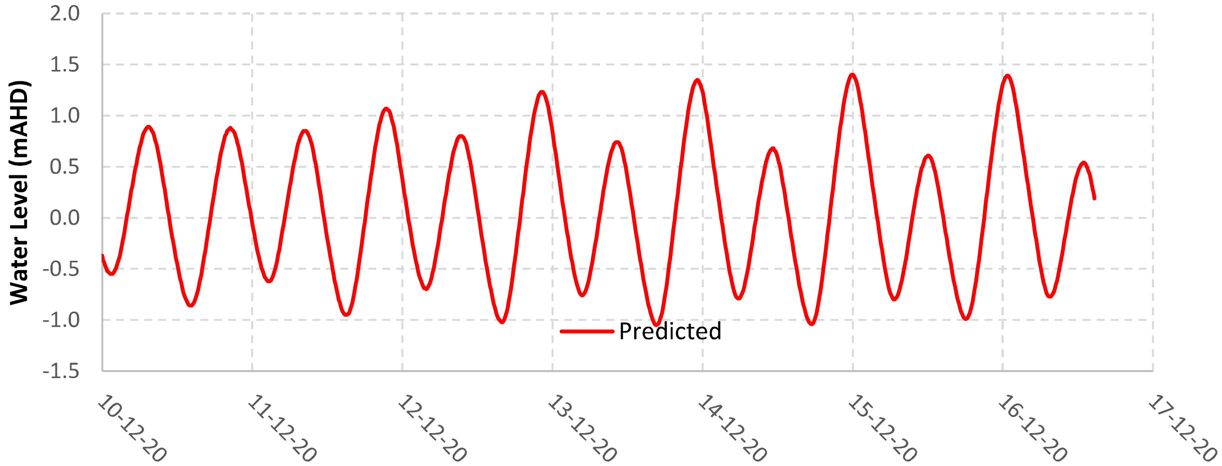
Offshore tidal conditions throughout the model have been based on tidal harmonics extracted from TXPO v8 global tidal harmonic model.

Three boundary conditions were used across the model domain, spanning the channels to the north, east and south of the bay. Tidal constituents were applied at the start and end of each boundary based on the TXPO model, as described in Section 2.1.1. The bottom depths within the model exceed 30m along much of the model ocean boundary.

From these tidal constituents a tide time series has been generated and applied to the overall model boundaries and modelled with the Moreton Bay and lower channel region. The modelled flow velocities and water level fluctuations induced by the astronomical tide have been extracted throughout Redland Bay as a time series. The extracted tide cycle is then applied in the nested sub-model as an offshore input timeseries. The nested offshore boundary is located approximately 700m offshore, at a depth of 5m.

3.4 Calibration and validation data

The overall model was calibrated against 7 days of near real-time astronomical tide data at the Shorncliffe and Scarborough gauge locations[6]. This period spanned 10 to 17 December 2020. Two datasets were provided by QLD State Government; the astronomical tide predictions and the observed (recorded) water levels, relative to Lowest Astronomical Tide (LAT). An adjustment from LAT to AHD of 1.31m for Shorncliffe and 1.17m for Scarborough was applied based on QLD Tidal Predictions[7]. Figure 3-4 shows the predicted astronomical tide levels for each gauge location during the model period. As the TXPO harmonic boundary conditions are purely astronomical, the Delft3D model has been compared to this data for calibration.



6 QLD Gov. (2020) Storm tide monitoring | Environment, land and water | Queensland Government (www.qld.gov.au)

7 MSQ (2020) Queensland Tide Tables Standard Port Tide Times 2020. The State of Queensland (Department of Transport and Main Roads).

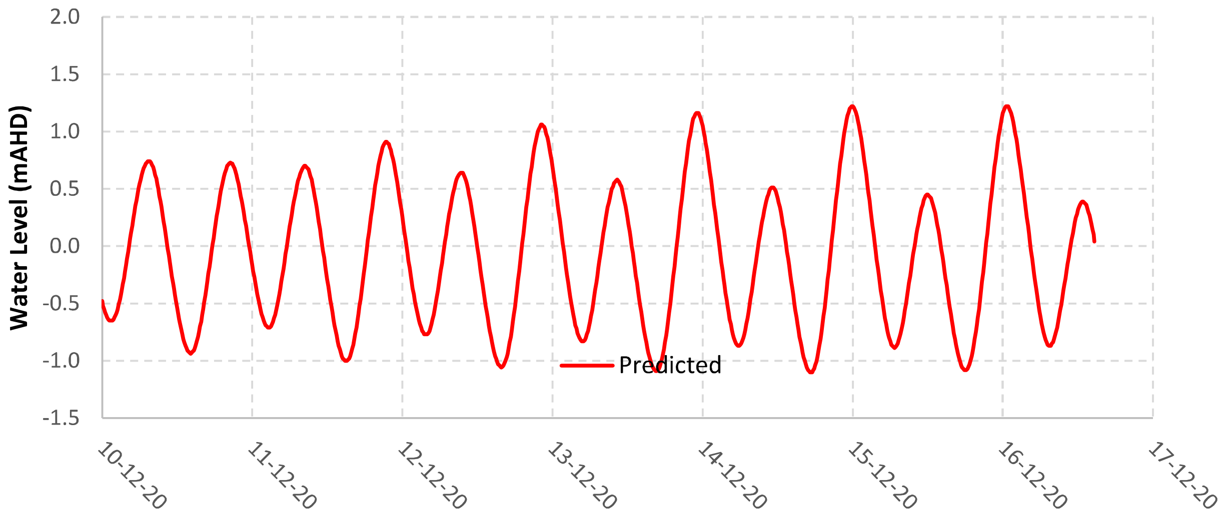
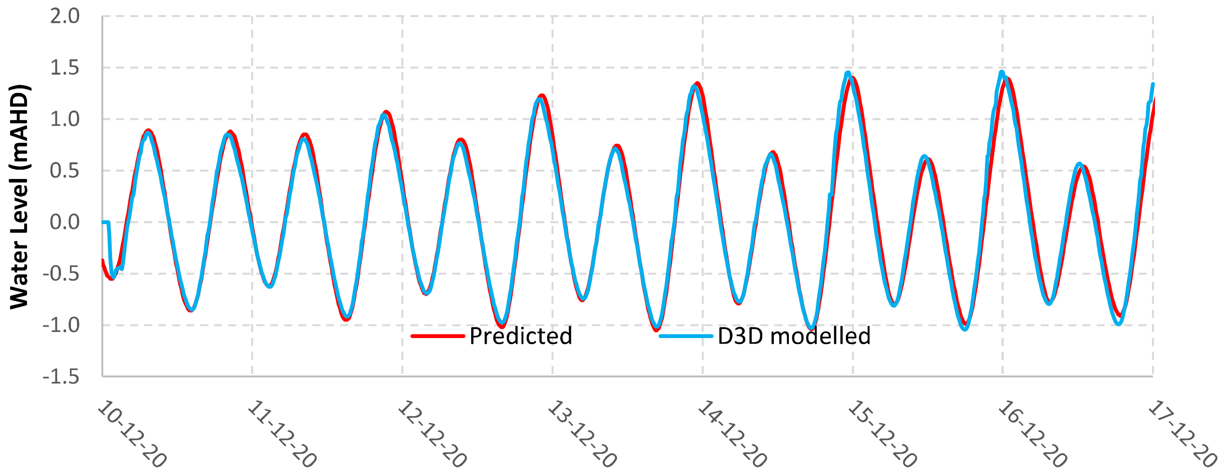


Figure 3-4: Astronomical tide series for Shorncliffe (top) and Scarborough (bottom) during December 2020, as sourced from QLD State Government.

The overall model has been run with the boundary TXPO tide harmonics for the period of 10th to 17th of December and results extracted at the Shorncliffe and Scarborough locations. Figure 3-5 and Figure 3-6 show a comparison of the D3D model results to the predicted tide level at each site. The model results show:

- Over the timeseries the absolute error is 8cm and 12cm for Shorncliffe and Scarborough respectively. This represents a difference of 4% and 6% respectively over the 2m spring tide range.
- There is an average difference between peak (maximum) modelled and astronomical tides of 0.04m and 0.1m at Shorncliffe and Scarborough respectively. This results in an absolute peak difference of 2% and 5% respectively.
- A comparison of all modelled and astronomical data shows an R^2 value of over 0.97, indicating the model is reproducing the tidal signal extremely well.

Any minor discrepancies between the model and astronomical data may be attributed to nonlinear tide-surge interactions (including contributions from frictional and shallow water effects within the Delft3D model), and minor differences in amplitude and timing within the TXPO harmonic predictions. These errors are considered within an acceptable range to proceed with the design stage of modelling.



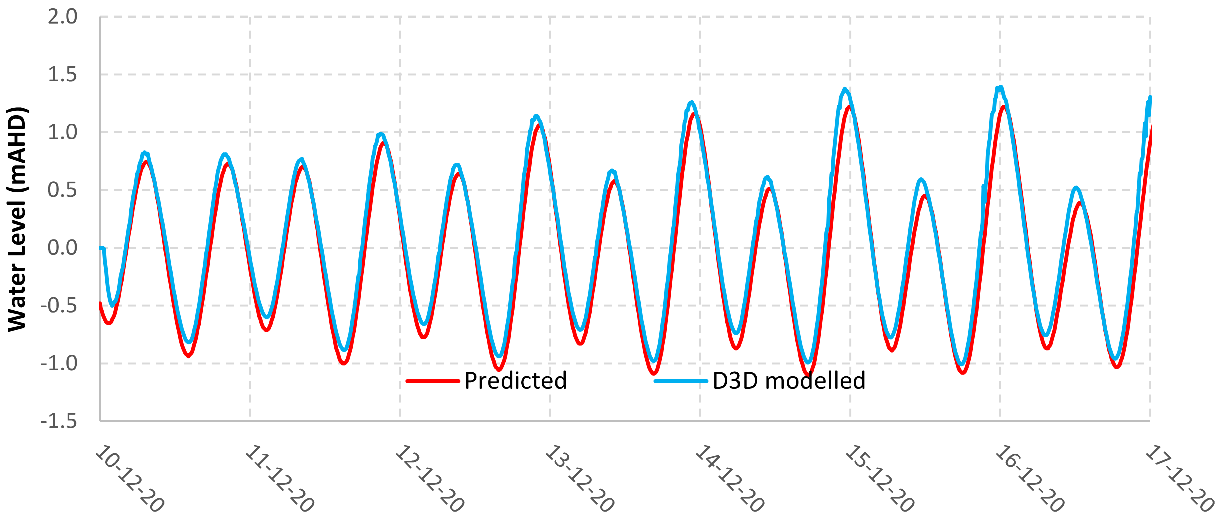


Figure 3-5: Astronomical and modelled tide series for Shorncliffe (top) and Scarborough (bottom) during the December 2020 model period.

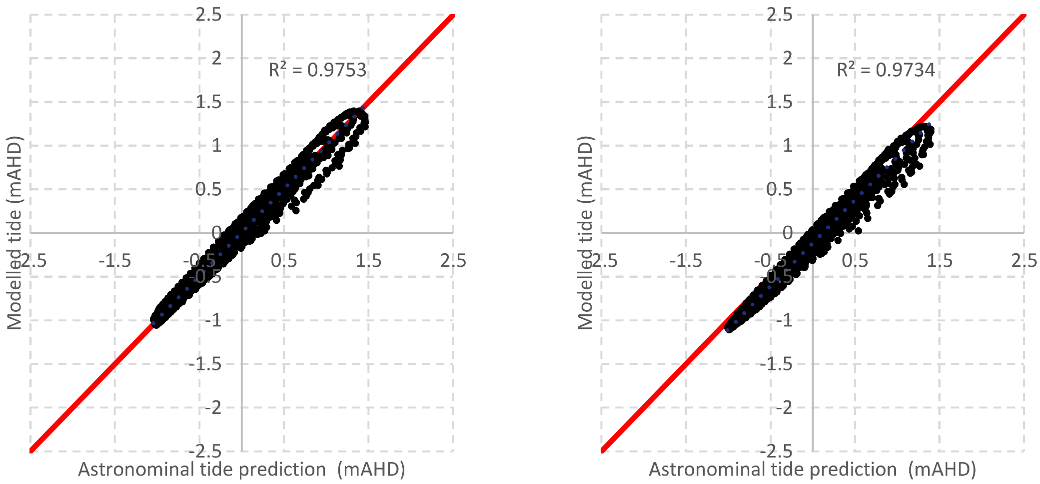


Figure 3-6: Astronomical and modelled tide series correlation for Shorncliffe (left) and Scarborough (right) during December 2020 model period.

3.5 Design simulations

The period from 3rd to 18th of January 2020 period was adopted as the design simulation. This includes individual tides matching a MHWS level (1.04m AHD) at Redland Bay, and up to approximately 1.4m AHD. Figure 3-7 shows the TXPO-generated spring-neap tide series simulated in the overall model. Two design scenarios have been considered:

1. Mean High Water Springs (MHWS) tide, where currents and water levels were extracted for tide level approximating MHWS
2. Peak tide level and current speed over a 14-day Spring-Neap tidal cycle

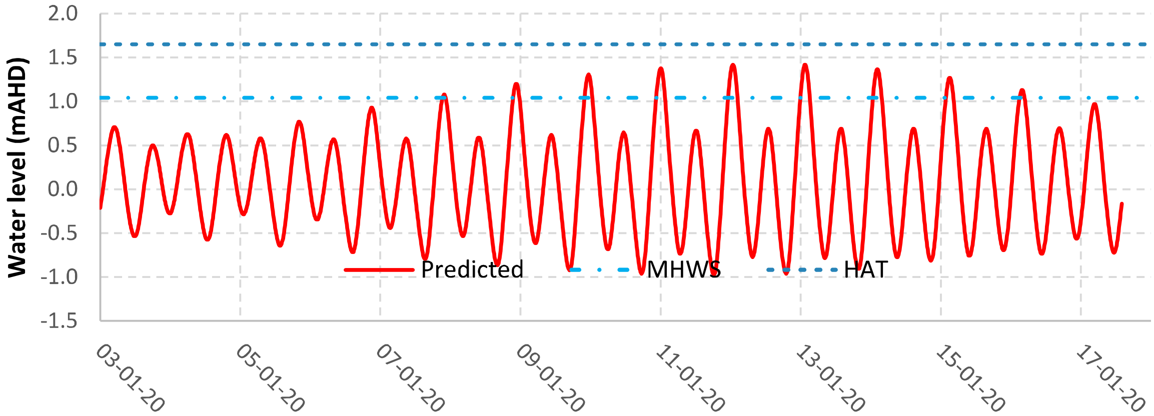
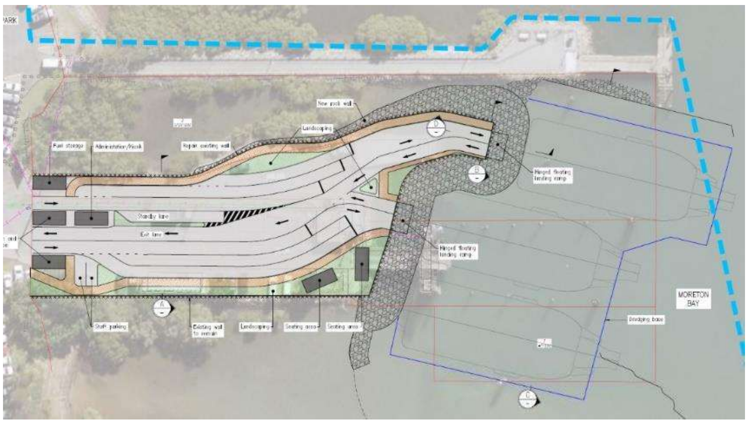


Figure 3-7: Design TXPO spring-neap tide series applied to overall model

Each design scenario considered existing conditions and the proposed terminal upgrade, which was incorporated into the Delft3D model based on the layout shown in Figure 3-8, and bathymetry data supplied by Projex Partners[8]. Figure 3-9 shows the changes in the model bathymetry from the existing terminal layout. Changes have been applied directly into the model bathymetry, which include:

- Extension of the existing landing ramp
- Addition of a second landing ramp
- Sloping rockwall of 1:1.5 along the eastern and northern sides
- Formalised dredged base with uniform depth = -3.55mAHD
- Formalised deck surface at 1.8mAHD.



8 Development layout and survey bathymetry supplied by Project Partners (Daniel Berry) on 17 December 2020. " [#497-001] Redland Barge Terminal - Coastal - Go Ahead"

Figure 3-8: Proposed ferry terminal layout plan (Source: Projex Partners).

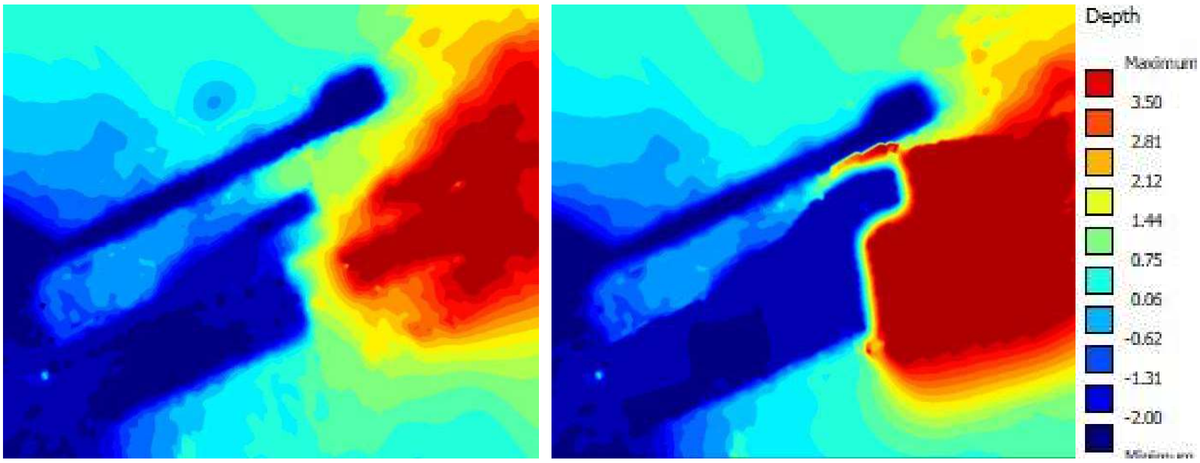


Figure 3-9: Model bathymetry for existing (left) and upgraded (right) ferry terminal configuration.

3.5.1 MHWS tide scenario (initial testing of nearfield changes)

This simulation has been used to screen potential changes to water levels and currents during a MHWS tide, which occurred on 8 January 2021 and reached a level of 1.04mAHD. The model was run for the existing structure and the proposed upgraded structure.

A number of nearshore reporting points have been used to estimate changes to tidal hydrodynamics and currents around the proposed extension. These are shown in Figure 3-10 and include the following:

1. Within the mangrove area between the ramp and rockwall jetty (depth = -0.5mAHD)
2. The end of the proposed ramp (depth = -1.5mAHD)
3. Within the proposed dredged base (depth = -3.4mAHD)
4. 100m offshore (depth = -3.5mAHD)
5. 200m offshore within the main channel (depth = -2.5mAHD).

3.5.1.1 Existing structure

During the MHWS, the flood tide (incoming) flowed southward towards the site before being deflected around the northern rockwall jetty. A slow-moving eddy current was formed at the seaward end of the ramp, with the tide also flowing between the rockwall jetty and ferry terminal to fill the mangrove area. During an ebb tide (receding), the mangrove area is drained and the dominant flow direction in the channel is northward, with a reversed eddy current formed within the ferry area. The modelled tide and current speeds adjacent to the ramp are shown in Table 3-1 and Table 3-2. At the reporting points, there was no measured change to water levels. There were minor changes to current speeds, ranging from -0.04 to 0.01 m/s. These changes occurred during very low current speeds. The greatest reduction in current speed (-0.04 m/s) occurred during a current of 0.08 m/s. The greatest increase in current speed (-0.01 m/s) occurred during a current of 0.06 m/s. Further assessment of the extent of these changes has been tested during a full 14-day spring neap tidal simulation.

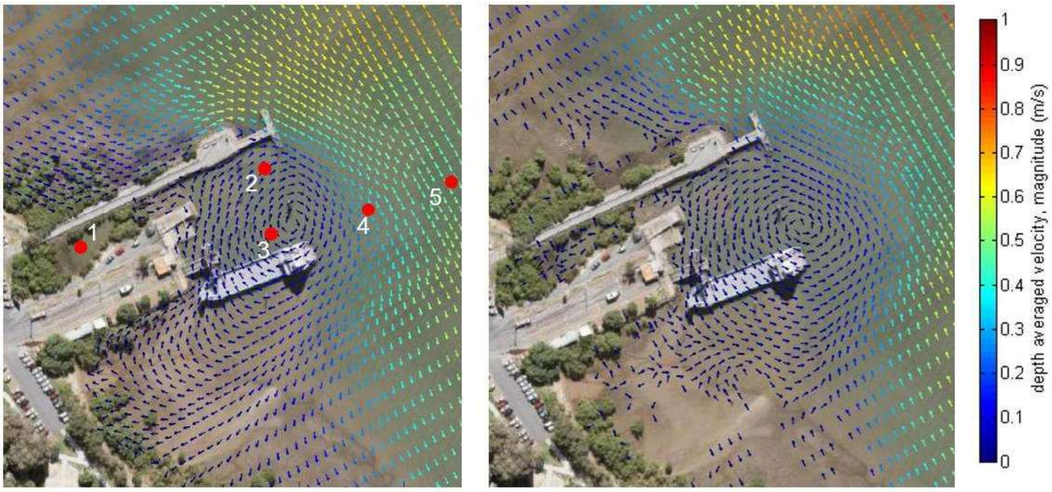


Figure 3-10: Tidal currents around existing ferry structure for incoming (left) and receding (right) MHWS high tide, recording points shown as red dots.

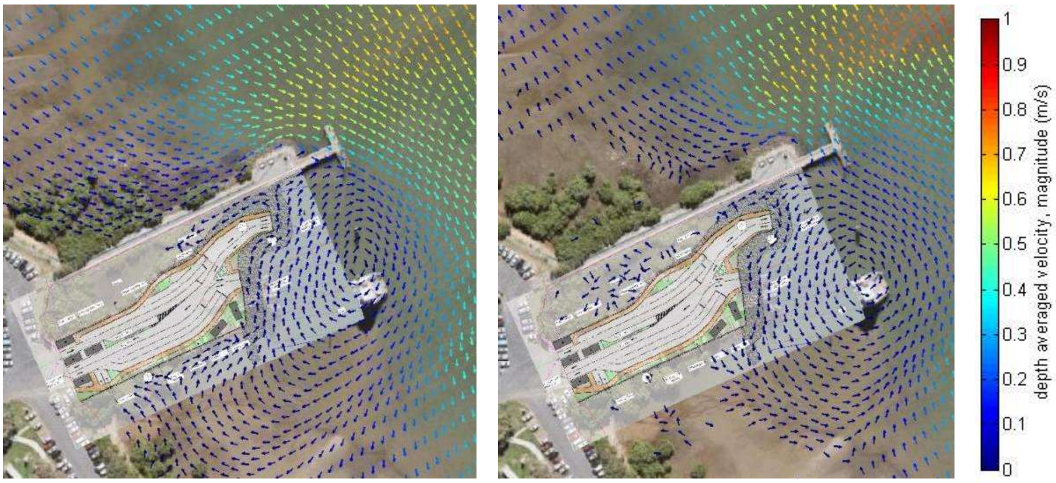


Figure 3-11: Tidal currents around proposed ferry structure for incoming (left) and receding (right) during a MHWS tide

Table 3-1: Peak tide conditions around the proposed terminal for MHWS

Location	Existing-case MHWS[9] water level (mAHD)	Proposed-case MHWS water level (mAHD)	Change (m)	Change (%)
1. Mangrove area	1.08	1.08	0.00	0.0%
2. End of proposed ramp	1.08	1.08	0.00	0.0%
3. Dredging base area	1.08	1.08	0.00	0.0%
4. 100m offshore	1.08	1.08	0.00	0.0%
5. 200m offshore	1.08	1.08	0.00	0.0%

9 Based on the astronomic tide occurring on 18 January 2021, which approximated the MHWS level of 1.04mAHD

Table 3-2: Peak currents around the proposed terminal for MHWS tide

Location	Proposed-case MHWS current (m/s)	Proposed-case MHWS current (m/s)	Change (m/s)	Change (%)
1. Mangrove area	0.02	0.01	-0.01	-50.0%
2. End of proposed ramp	0.08	0.04	-0.04	-50.6%
3. Dredging base area	0.06	0.07	0.01	14.8%
4. 100m offshore	0.14	0.13	-0.01	-6.5%
5. 200m offshore	0.50	0.50	0.00	0.0%

3.5.2 Peak Spring-Neap scenario

A full spring-neap tidal cycle was run to understand the magnitude and extent of changes during the largest conditions during the simulation. Reporting points were extended to allow detailed analysis of current changes (see Figure 3-12).

The peak changes to tide height and current speed were extracted from the model for the entire spring-neap tide cycle for the existing structure and proposed structure. Water level and current speed differences are presented in Table 3-3 and Table 3-4 respectively.

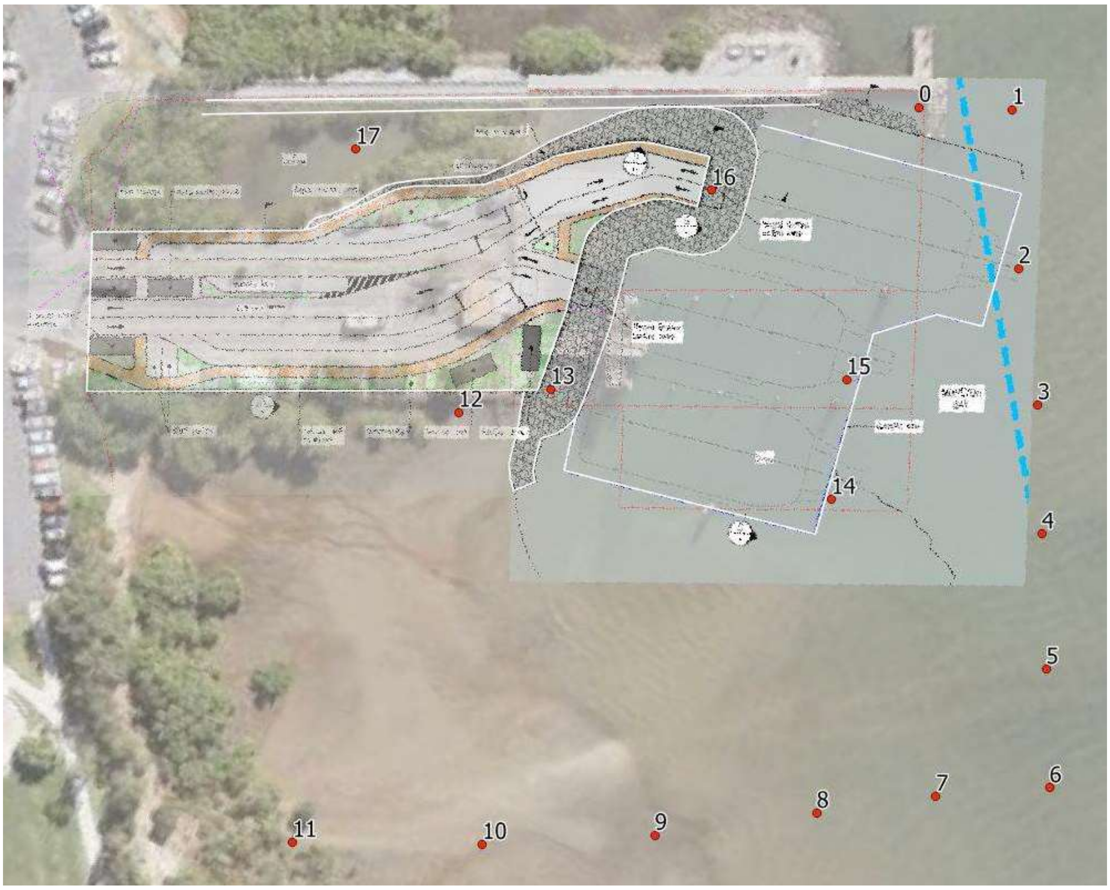


Figure 3-12: Proposed ferry terminal layout plan view showing Delft3D model recording points for Spring-Neap period (Projex Partners).

Table 3-3: Peak tide conditions around the proposed terminal during simulated tidal period

Location	Existing-case peak water level (mAHD)	Proposed-case peak water level (mAHD)	Change (m)	Change (%)
Reporting points around 50m perimeter				
0 - jetty end	1.59	1.59	0.00	0.0%
1 - 50m perimeter	1.59	1.59	0.00	0.0%
2 - 50m perimeter	1.59	1.59	0.00	0.0%
3 - 50m perimeter	1.59	1.59	0.00	0.0%
4 - 50m perimeter	1.59	1.59	0.00	0.0%
5 - 50m perimeter	1.59	1.59	0.00	0.0%
6 - 50m perimeter	1.59	1.59	0.00	0.0%
7 - 50m perimeter	1.59	1.59	0.00	0.0%
8 - 50m perimeter	1.59	1.59	0.00	0.0%
9 - 50m perimeter	1.59	1.59	0.00	0.0%
10 - 50m perimeter	1.59	1.59	0.00	0.0%
11 - 50m perimeter	1.59	1.59	0.00	0.0%
Reporting points adjacent to proposed structure				
12 - south wall	1.59	1.59	0.00	0.0%
13 - south wall	1.59	1.59	0.00	0.0%
14 - dredge area	1.59	1.59	0.00	0.0%
15 - dredge area	1.59	1.59	0.00	0.0%
16 - ramp end	1.59	1.59	0.00	0.0%
17 - mangroves	1.59	1.59	0.00	0.0%

Table 3-4: Changes to peak currents around the upgraded terminal during simulated tidal period

Location	Existing-case peak current (m/s)	Proposed-case peak current (m/s)	Change (m/s)	Change %
Reporting points around 50m perimeter				
0 - jetty end	0.40	0.40	0.00	-0.6%
1 - 50m perimeter	0.78	0.78	0.00	0.0%
2 - 50m perimeter	0.41	0.40	0.00	-1.0%
3 - 50m perimeter	0.41	0.41	0.00	-0.3%
4 - 50m perimeter	0.47	0.48	0.01	2.7%
5 - 50m perimeter	0.64	0.63	-0.02	-2.3%
6 - 50m perimeter	0.67	0.67	0.00	-0.4%
7 - 50m perimeter	0.50	0.50	0.00	-0.4%
8 - 50m perimeter	0.34	0.34	0.00	-0.2%
9 - 50m perimeter	0.28	0.29	0.02	5.8%
10 - 50m perimeter	0.25	0.27	0.02	8.0%
11 - 50m perimeter	0.17	0.18	0.01	8.7%
Reporting points adjacent to proposed structure				
12 - south wall	0.14	0.18	0.04	27.7%
13 - south wall	0.16	0.07	-0.10	-59.5%
14 - dredge area	0.14	0.10	-0.04	-30.2%
15 - dredge area	0.15	0.16	0.02	11.1%
16 - ramp end	0.09	0.04	-0.06	-60.6%
17 - mangroves	0.03	0.04	0.00	6.7%

3.5.3 Changes to water levels

No significant change to water levels was predicted within the tidal simulation. The tidal levels within the post-development scenario correspond with the existing scenario.

3.5.4 Changes to currents

The extended ramp and rock wall will cause a minor deflection to tidal currents, causing a variation in the slow-moving eddy currents in the nearshore. Figure 3-13 shows this change in flood and ebb tidal currents.

Within the simulation, in close proximity to the proposed terminal (Reporting points 12-17) the change in current speed could change between -0.1 m/s up to +0.04m/s. These patterns align with the MHWS simulation, which also showed a tendency for a greater reduction in currents.

Beyond 50m of the proposed terminal, recording points 0-11 show a reduced impact, with changes to current speed between -0.02 to +0.02 m/s. At the seaward end of the public jetty there is no significant change in peak current speed. At this magnitude, these changes are not believed to have a significant change to coastal hydrodynamics, coastal sediment transport or scour and sedimentation patterns.

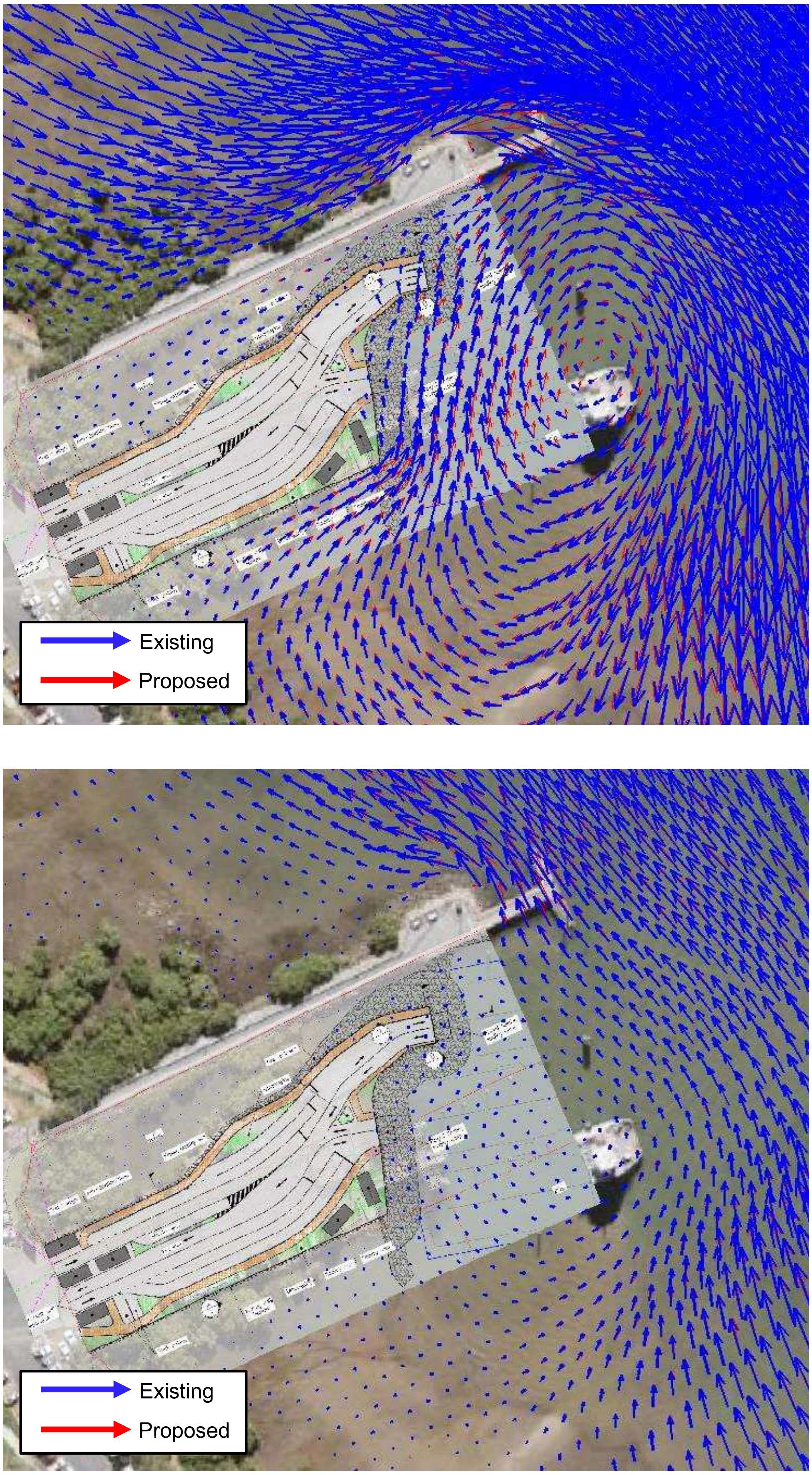


Figure 3-13: Change in current around structure for an incoming (top) and receding (bottom) tide during peak modelled tide.

4 Wave estimation

Wave conditions have been estimated using the D-Wave standalone module within Delft3D. Only wind-driven wave effects have been considered, due to the protected location of the study area within the lower Moreton Bay channel system. Wind conditions for a 0.5%, 1% and 2% Annual Exceedance Probability (50-year, 100-year and 200-year) event have been modelled during a Present Day, 2070 and 2100[10] water level scenario.

4.1 Approach

The Delft3D model uses the SWAN spectral wave formulae, which is a third-generation wave model that simulates wave propagation in coastal and inland areas. The model accounts for the following physics:

- Wind-wave interactions, which is the transfer of wind energy into wave energy, leading to the growth of waves.
- Shoaling, which is the build-up of energy as a wave enters shallow water, causing an increase in wave height.
- Refraction, which is the change in wave speed as waves propagate through areas of changing depth, causing a change in wave direction.
- Wave breaking, which is the destabilisation of a wave as it enters shallow water, causing broken waves with the characteristic whitewash or foam on the crest.
- Wave dissipation, which limits the size of waves through white-capping, bottom friction and depth-induced breaking.
- Diffraction, which is the spreading of wave energy behind structures, headlands and islands, which causes waves to change direction.

4.2 Model domain

The spatial domain of the wave model has been constructed using a computational grid with a spatially-varying resolution. Offshore areas used a grid size of around 50m, with the nearshore minimum grid resolution 12.5m at the Redland ferry terminal site. Spatial elevation and bathymetry data used in the model has been sourced from topographical and bathymetry survey data from the Deep Reef 30m GBR dataset, 1m LiDAR and bathymetric survey provided by Projex Partners.

4.3 Input conditions

4.3.1 Input wind conditions

Extreme wave conditions are a function of wind speed, direction, fetch, and water level. Within the lower Moreton Bay channel system wave conditions are assumed to fetch-limited, i.e. restricted by the distance over which a wind field can apply energy to the water's surface. The longest fetch applicable to the Redland ferry terminal site is approximately 13.5km, from the west coast of North Stradbroke Island.

A detailed assessment of climatology and probabilistic wind modelling was not within the scope of this project, and no wave calibration data exists for the Redland area. Instead, extreme wind speeds have been calculated from the Australian Standard AS1170.2-2011 (Wind Actions) and methods described in the USACE Coastal Engineering Manual (CEM)[11]. This approach has been validated by comparing against recorded wind data at nearby gauges by determining the maximum recorded wind speed within a given interval.

Extreme wind speeds for the 0.5%, 1% and 2% AEP have been applied as a time and spatially-constant input within the wave model. A range of wind directions were tested between 0°N to 135°N in increments of 7.5° to identify the worst-wase wind direction producing the maximum wave heights. Figure 4-1 shows the location of observation points within the D-WAVE model domain.

10 Redland City Council - Property Detail Online, pdonline.redland.qld.gov.au

11 USACE, EM 1110-2-1100 (Part II) 30 Sept 2015: Ch. 2, Sec. 3, pp II-2-3, Temporal variability of wind speeds

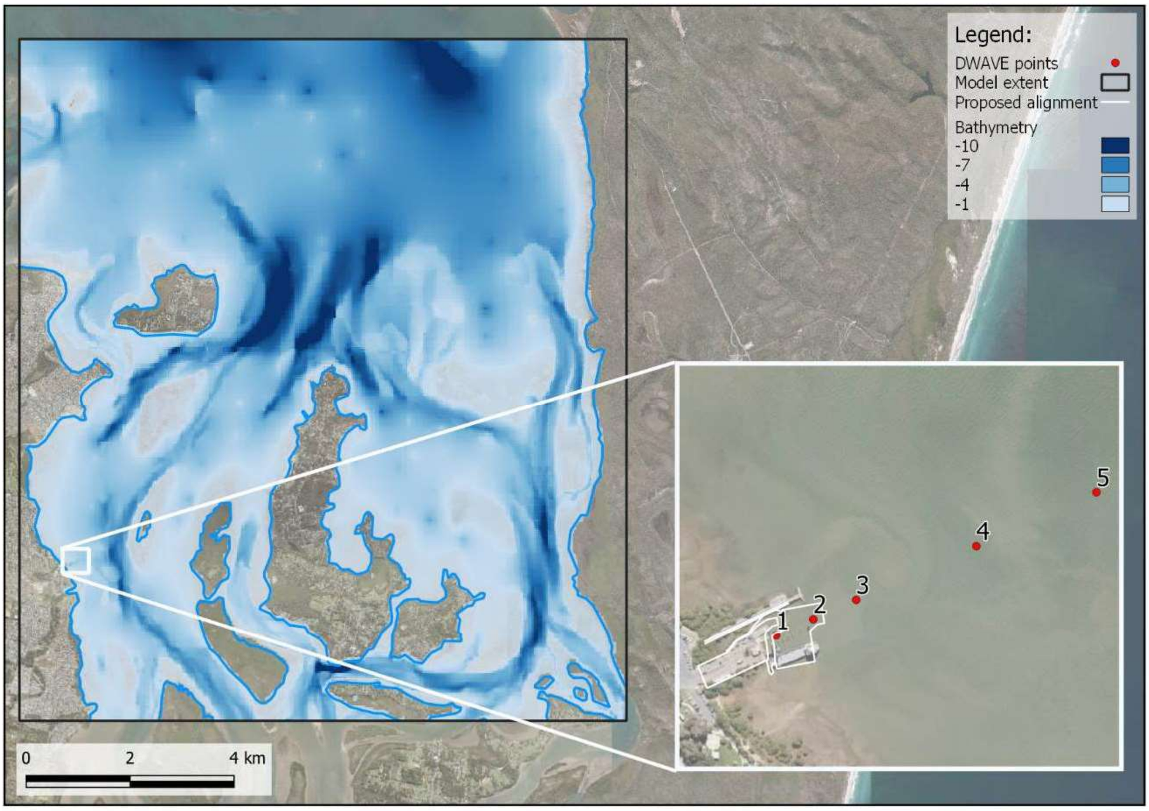


Figure 4-1: D-WAVE modelling domain and location of output points at Redland ferry terminal

4.3.2 Input storm tide conditions

The D-WAVE model has been run for three planning horizons: Present Day, 2070, and 2100.

Each planning horizon has used a static 1% AEP storm tide level. This is based on the 2100 1% AEP storm tide level provided by Redland City Council, and the approaches defined in Section 2.1.3.

Table 4-1: Peak tide conditions around the proposed terminal during simulated tidal period

	Present day	2070	2100
1% AEP Storm tide level	2.51 mAHD	2.96 mAHD	3.23 mAHD

4.4 Nearshore wave conditions at Redland Ferry Terminal

The model was used to simulate a 0.5%, 1% and 2% AEP wind-driven wave conditions, under Present Day, 2070 and 2100 planning horizons at the proposed boat ramp. Wave characteristics were extracted at locations adjacent to the Redland ferry terminal. The significant wave height (Hs) and peak periods (Tp) are shown in Table 4-2 and Table 4-3 for present day, 2070 and 2100 planning horizons.

Table 4-2: Design conditions at Ferry terminal for 1% AEP events.

		Present Day 1% AEP		2070 1% AEP		2100 1% AEP	
Output location	Depth (mAHD)	Hs (m)	Tp (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
1	-1.16	1.42	3.48	1.52	4.47	1.58	4.47
2	-3.55	1.50	3.48	1.59	3.95	1.64	4.47
3	-2.66	1.51	3.95	1.59	4.47	1.65	4.47
4	-2.35	1.54	4.47	1.62	4.47	1.67	5.07
5	-2.25	1.59	4.47	1.67	5.07	1.72	5.07

Table 4-3: Design conditions at Ferry terminal for 0.5% AEP events.

		Present Day 0.5% AEP		2070 0.5% AEP		2100 0.5% AEP	
Output location	Depth (mAHD)	Hs (m)	Tp (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
1	-1.16	1.50	3.95	1.61	4.47	1.67	4.47
2	-3.55	1.59	4.47	1.69	4.47	1.75	4.47
3	-2.66	1.60	4.47	1.71	4.47	1.76	4.47
4	-2.35	1.64	4.47	1.74	4.47	1.80	5.07
5	-2.25	1.70	5.07	1.79	5.07	1.84	5.07

5 Rock armour design

Rock armour sizing has been undertaken using the nearshore wave modelling results. Rock armour is used to dissipate wave energy and protect the structure fill from wave impact. To calculate rock armour stability, the Van der Meer (VDM) deep and shallow water equations have been adopted[12]. The formulas are used to predict the stability of armour units on a uniform slope. The method includes the effect of storm duration, wave period, structure permeability, and damage level. The main benefit of this method is its distinction between plunging waves and surging waves, which is typically a result of wave period. The following equations are shown as an example and are used when waves are plunging, therefore must satisfy ($\xi_{s-1,0} < \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = C_{pl} P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) \xi_{s-1,0}^{-0.5}$$

In which:

$$N = \frac{\text{storm duration}}{\text{wave period}} <= 3000$$

$$H_{2\%} = 1.4Hs$$

$$\xi_{s-1,0} = \frac{\tan\alpha}{\sqrt{\frac{\frac{2\pi}{g} h_s}{T_{m-1,0}^2}}}$$

$$\xi_{cr} = \left(\frac{C_{pl}}{C_s} P^{0.31} \sqrt{\tan\alpha}\right)^{\frac{1}{P+0.5}}$$

Where Dn = rock diameter, Sd = damage factor, Hs = significant wave height, $H_{2\%}$ = Wave height exceeded by 2% of waves, Tm = mean period, N = number of waves, ξ = surf similarity parameter.

5.1 Ramp cross section

Projex Partners are undertaking the detailed design of the ramp, and have provided a section for the eastern exposed slope as shown in Figure 5-1. The cross-section identifies the rock armour slope, with storm tides and the direction of primary wave attack added to the plot for reference. For this cross section, the toe of the rock armour is at -3.55m AHD, with a slope of 1V:1.5H, to a crest level of 1.8m AHD.

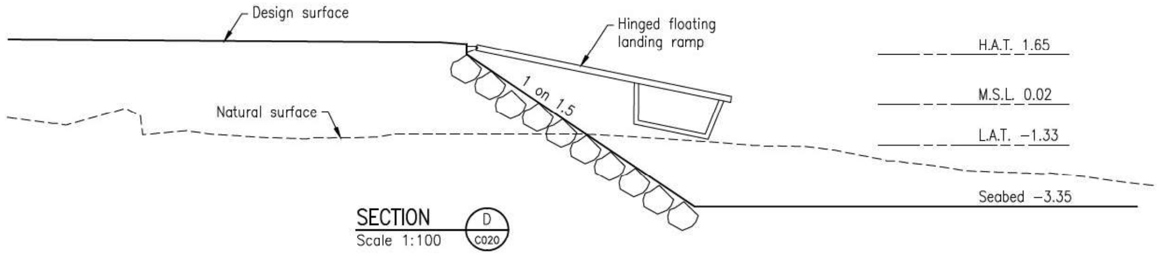


Figure 5-1: Cross section for eastern seaward rockwall slope[13]

5.2 Design scenarios

Three planning horizons have been considered: Present day (2020), 2070, and 2100. For each planning horizon three wave return periods have been considered, a 0.5%, 1% and 2% event. Wave conditions have been applied directly from the results of wave modelling discussed in Section 4.

12 Van der Meer, J W (1988). "Rock slopes and gravel beaches under wave attack". PhD thesis. Delta University of Technology, Delft.
13 Supplied by Project Partners (Daniel Berry) on 17 December 2020. " [#497-001] Redland Barge Terminal - Coastal - Go Ahead"

5.3 Storm duration

Storm duration is a critical factor when determining rock armour size as it influences the number of design waves acting on the exposed slope. A longer duration inherits more wave impacts and an increase in armour rock mass. A worst-case design storm has been assumed wherein each storm event occurs across a 6-hour high tide period.

5.4 Assumptions

- Rock sizing has been estimated based on wind-driven wave attack only
- The effect of scour caused by propelled wash from vessels has not been considered
- The VDM method assumes a double layer (primary and secondary) of rock armour
- Each layer is a double standard layer with an overlapping coefficient of 0.91 and blockiness of 0.65
- Crest effects and overtopping have not been considered
- For rock armour sizing the toe is assumed to be at a depth of -3.55m AHD, with a slope of 1V:1.5H.

Figure 5-2 shows a schematisation of the rock armour layers with wind-driven waves and storm tide level.

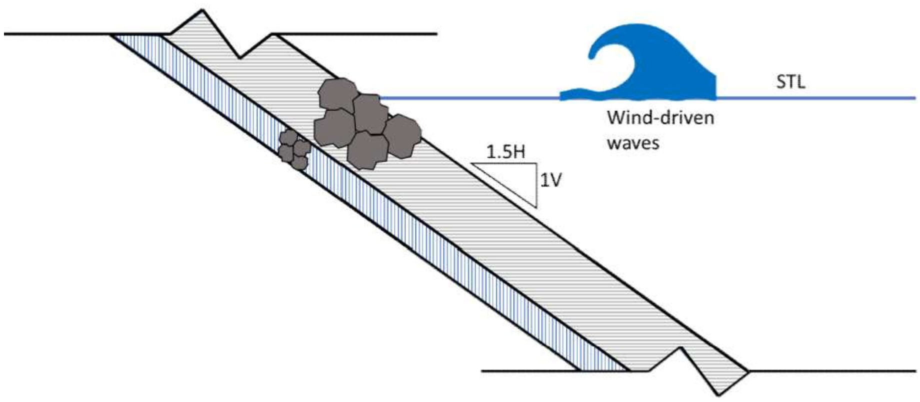


Figure 5-2: Schematisation of rock armour layers

5.5 Rock armour design

Table 5-1 shows the additional parameters that are used to determine the median required rock mass (M50) to achieve stability during the design event. Using the VDM method for rock armour stability and the described parameters, the required median rock armour is shown in Table 5-2.

Table 5-1: Armour design parameters

Parameter	Value
Armour rock density	2.65 t/m³ providing a practicable minimum that is achievable from quarry sources.
Seawater density	1.025 t/m³
Storm duration	6.0 hours, duration of high tide.
Storm damage	Sd = 2 correlates with maximum 5% of armour units becoming displaced during the design event.
Notional permeability	0.1 has been selected as the structure utilises a double interlocking armour layer on a filter layer that increases the structure permeability, therefore dissipating wave energy. This is considered conservative building in safety to the design.
Slope	1V:1.5H
Mass Factor of Safety	1.5 and 2.0

A range of scenarios have been considered when estimating stable rock sizes, with varied return periods, planning horizons and Safety Factors. Rock sizes range from a Dn_{50} of 0.8m for a present day, 0.5% AEP, 1.5 FoS, to a Dn_{50} of 1.1m for 2100 planning horizon, 2% AEP event with a 2.0 FoS. These can be considered for incorporation into the detailed design, which will need to consider relevant legislation, codes, standards and risks.

Table 5-2: Summary of armour design for present day, 2070, 2100

			Primary armour		Secondary armour	
Year	RP	FoS	D50 (m)	M50 (kg)	D50 (m)	M50 (kg)
2020	2.0%	1.5	0.80	1372.1	0.36	128.9
	1.0%	1.5	0.80	1380.7	0.37	129.7
	0.5%	1.5	0.93	2125.7	0.42	199.6
	2.0%	**2.0**	**0.88**	**1829.4**	**0.40**	**171.8**
	1.0%	**2.0**	**0.89**	**1840.9**	**0.40**	**172.9**
	0.5%	**2.0**	**1.02**	**2834.3**	**0.46**	**266.2**
2070	2.0%	1.5	0.88	1781.7	0.40	167.3
	1.0%	1.5	0.88	1832.5	0.40	172.1
	0.5%	1.5	0.97	2438.4	0.44	229.0
	2.0%	**2.0**	**0.96**	**2375.5**	**0.44**	**223.1**
	1.0%	**2.0**	**0.97**	**2443.4**	**0.44**	**229.5**
	0.5%	**2.0**	**1.07**	**3251.2**	**0.49**	**305.3**
2100	2.0%	1.5	0.90	1920.9	0.41	180.4
	1.0%	1.5	0.95	2279.1	0.43	214.0
	0.5%	1.5	1.00	2637.5	0.45	247.7
	2.0%	**2.0**	**0.99**	**2561.2**	**0.45**	**240.5**
	1.0%	**2.0**	**1.05**	**3038.8**	**0.48**	**285.4**
	0.5%	**2.0**	**1.10**	**3516.7**	**0.50**	**330.3**

6 Summary

This report has been prepared by JBPacific (JBP) on behalf of Projex Partners, to undertake a coastal modelling assessment at Redland Bay, Queensland. The study will support the detailed design of a proposed ferry terminal expansion at the Redland Barge Terminal.

A numerical model was used to simulate the tide characteristics within Moreton Bay. Two design scenarios were considered: conditions during a Mean High Water Springs (MHWS) tide, and peak conditions over a 14-day spring-neap tidal cycle.

- A Mean High Water Spring (MHWS) tide was first simulated to understand potential changes around the proposed upgrade. There was no measured change to water levels, however minor changes to current speeds, ranging from -0.04 to 0.01 m/s. These changes occurred during very low current speeds. The greatest reduction in current speed (-0.04 m/s) occurred during a current of 0.0 m/s. The greatest increase in current speed (-0.01 m/s) occurred during a current of 0.06 m/s.
- Further assessment of the magnitude and extent of these changes has been tested during a full 14-day spring neap tidal simulation. No changes to water levels were predicted within the tidal simulation. Minor changes to nearshore tidal currents were observed, which ranged between -0.1 m/s up to +0.04m/s adjacent to the proposed upgrade. Beyond 50m of the proposed terminal this impact reduces to between -0.02 to +0.02 m/s. At the seaward end of the public jetty there is no significant change in peak current speed. At this magnitude, these changes are not believed to have a significant change to coastal hydrodynamics, coastal sediment transport or scour and sedimentation patterns.

Nearshore wave conditions have been estimated using Delft3D. Only wind-driven wave effects have been considered, due to the protected location of the study area within the lower Moreton Bay channel system. For a 1% AEP event the nearshore wave conditions ranged between 1.4 to 1.7m, with peak periods between 3.5-5.0s. A range of scenarios have been considered to estimate stable rock sizes, using varied return periods, planning horizons and Safety Factors. Rock sizes range from a Dn_{50} of 0.8m for a present day, 2% AEP, 1.5 FoS, to a Dn_{50} of 1.1m for 2100 planning horizon, 0.5% AEP event with a 2.0 FoS. These can be considered for incorporation into the detailed design, which will need to consider relevant legislation, codes, standards and risks.

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