JBP
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Mooloolaba Central Meeting Precinct

Coastal modelling

Updated Final Report

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Revision History

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Contract

This report describes work commissioned by Place Design Group (PDG), on behalf of the Sunshine Coast Council, by an email dated 30 March 2023. PDG's representative for the contract was Sasha Tieleman. Daniel Rodger, Clare Yang, Brian Lam and Alexandra Maskell 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 April 2023 to January 2024 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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Where field investigations are carried out, these have been restricted to a level of detail required to meet the stated objectives of the services. The results of any measurements taken may vary spatially or with time and further confirmatory measurements should be made after any significant delay in issuing this report.

Acknowledgements

JB Pacific acknowledges the traditional custodians of the lands and seas where we work. We pay our respects to Elders past, present, and emerging.

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Executive Summary

This study has been completed by JB Pacific (JBP) for Place Design Group (PDG) and Sunshine Coast Council (SCC) as part of the detailed design of the Mooloolaba Central Meeting Precinct (CMP). This study uses structural calculations and numerical modelling to support design inputs and quantify the impact of new coastal designs. This work consists of:

1. An extreme wave study
2. An extreme beach profile morphological study
3. An extreme wave overtopping study
4. A wave action study

Coastal designs have been provided to JBP for assessment. These designs seek to balance foreshore green space, beach area, and a renewed seawall with beach access. Coastal modelling is a challenging area of numerical (computer-based) modelling. There is no single model able to simulate the effects of tides, storm surges, waves, erosion and overtopping, and instead a suite of calculations and models have been used. These have limitations, including:

- Uncertainty in Input Data: Coastal models heavily rely on input data, such as bathymetry, topography, offshore wave conditions, tidal data, and sediment characteristics. These data may have uncertainties or inaccuracies, which can affect the results.
- Model Limitations: Coastal calculations and models are simplifications of complex natural processes. They may not fully capture all physical and environmental complexities, such as interactions between waves, currents and sediment transport.
- Assumptions: Calculations and modelling often require various assumptions, which might not always represent the actual conditions accurately. For instance, conditions before an event are not always known and have to be estimated prior to simulations.

Extreme wave study

The extreme wave study has estimated new nearshore wave conditions. The site experiences some protection by the Point Cartwright headland, although waves from a north-easterly direction are able to propagate into Mooloolaba Bay. Wave conditions have been transformed from offshore to nearshore using a spectral wave model and statistical emulator with nearshore waves conditions extracted at the -10m depth contour. These new extreme wave distributions have been compared to values published in the previous Coastal Processes Study for the Sunshine Coast (BMT, 2013), which shows that in general, there has been a decrease in extreme wave height.

Morphological study

Morphological modelling was undertaken to understand how the current beach changes after a storm, and if this will change significantly with the new designs. The modelling has targeted the most protruded sections of the proposed seawall to test three different typologies; a terrace, vertical and steeper design. This is therefore not a balanced assessment, instead focusing on the areas of greatest impacts (most protruding sea wall areas). For areas where the seawall retreats (moves landward) the impacts are expected to be less.

The morphological study has used an XBeach erosion model to simulate the changes to the beach under extreme storms. Under both the existing and proposed seawall design, the modelling shows there is the potential for over 200 m^3/m of sand to be lost from the beach in a significant storm (1% AEP), with a significant amount of the beach eroded above the 0mAHD contour.

The proposed design typically has a wider footprint than the existing seawall and reduces the amount of available backshore sand which would typically be eroded in a storm event. This results in reduced erosion volume, however a narrower post-storm beach with typically deeper scour levels occur adjacent to the proposed structure. A large erosion volume lost from the beach occurs under both the existing and proposed designs. If a wide usable beach was required, beach nourishment would be required for both the existing and proposed designs.

Wave overtopping study

Wave overtopping rates have been estimated using the EurOtop Neural Network (NN1) tool. This is a model that uses results from a world-wide dataset of physical overtopping studies to estimate the likely overtopping rate at a schematised cross section. As with all calculation approaches, the Neural Network tool has limitations - estimates are based on a limited dataset of physical model tests which vary in scale effects, accuracy of measurement equipment and wave generation techniques. Additionally, a wider range of results are available within the tool for standard structures, e.g. vertical seawalls or rock armour revetments, with limited data available for unique designs and non-standard structures such as the curving design with multiple terraces proposed at Mooloolaba. As a result, the results of the Neural Network tool should be used with a degree of caution and physical modelling is recommended to confirm overtopping rates if further certainty is required.

Six design profiles have been input into the Neural Network tool to estimate the mean overtopping rate. Several sections have calculated overtopping at multiple locations or design iterations, typically at the crest and a position further landward. The modelling includes three time horizons; present day, 2043 (within 20 years of construction) and 2073 (end of useful working life. All storms consider extreme waves and sea levels coinciding with a post-storm eroded beach level of -1m AHD. The mean overtopping rate can be compared against guidance thresholds from the EurOtop (v1) manual, with the following criteria used to describe the overtopping:

- Mean overtopping rates over 200 l/s/m: Damage to well protected embankment/seawalls
- Mean overtopping rates over 50 l/s/m: Damage to grassed areas and pavers/promenades
- Mean overtopping rates over 10 l/s/m: Unsafe for trained staff (e.g. SES)
- Mean overtopping rates over 1 l/s/m: Unsafe for pedestrians, but accessible for trained staff

Table E-1: Results of overtopping modelling (l/s/m), 2074 planning horizon

Cross section / design iteration	MHWS	Q20	Q50	Q100	Q500
Section 1 (crest at 5.5m AHD)	0.3	7.9	11.2	14.4	17.0
Section 1 (landward, 5.95m AHD)	0.1	1.8	2.6	3.4	4.0
Section 2 (crest at 6.4m AHD)	0.0	2.0	3.0	3.9	4.8
Section 2 (mid-crest at 5.5m AHD)	0.1	2.5	3.7	5.0	6.2
Section 3 (crest at 5.95m AHD)	0.3	5.0	6.7	8.3	9.7
Section 4 (step at 5.05m AHD)	2.3	27.5	34.7	41.4	46.8
Section 4 (step at 5.05, calculated landward at 6.0m AHD)	0.3	6.1	8.3	10.3	12.0
Section 4 (step at 5.5m AHD (+1 step)	1.2	15.3	19.5	23.4	26.5
Section 4 (step at 5.5m AHD (+1 step) calculated landward at 6.0m AHD	0.3	3.9	5.0	6.0	6.8
Section 4 (crest at 6.0m AHD (+2 step)	0.6	8.3	10.8	13.0	14.8
Section 4 (step at 6.0m AHD (+2 step) calculated landward at 6.0m AHD	0.2	2.1	2.8	3.3	3.8
Section 5 (at crest 5.5mAHD)	0.4	10.5	14.8	19.1	22.6
Section 5 (landward at 6m AHD)	0.1	2.7	4.0	5.3	6.4
Section 6 (crest at 5.05m AHD)	0.5	9.6	12.9	16.1	18.8

Wave action study

This assessment considers the wave actions that will be present on the proposed structure. Four proposed cross section designs have been provided for assessment of wave actions. This assessment considered wave suction, impact, and uplift forces, which have bene analysed using the PROVERB method and empirical equations. These should be considered as additional loads to other hydraulic and hydrostatic loads, live and superimposed surcharges.

The design procedure has used empirical equations established for vertical seawalls, and due to the architecturally non-uniform design which includes a curved planform and terraced revetments, should be used in conjunction with a suitable factor of safety

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Abbreviations

AEP .... Annual Exceedance Probability
AHD .... Australian Height Datum
ARI .... Average Return Interval
BSS .... Brier Skills Score
CMP .... Central Meeting Precinct
COPE .... Coastal Observation Program - Engineering
DEM .... Digital Elevation Model
EVA .... Extreme Value Analysis
HAT .... Highest Astronomical Tide
LiDAR .... Light Detection And Ranging
LwaV .... Loo with a View
MSQ .... Marine Safety Queensland
PDG .... Place Design Group
PSD .... Particle Size Distribution
SCC .... Sunshine Coast Council
SLSC .... Surf Life Saving Club

1 Introduction

This study has been completed by JB Pacific (JBP) for Place Design Group (PDG) and Sunshine Coast Council (SCC) as part of the detailed design of the Mooloolaba Central Meeting Precinct (CMP). This study uses structural calculations and numerical modelling to support design inputs and quantify the impact of new coastal designs. Coastal engineering, calculations and modelling is a challenging area, and there is no single approach used to simulate the effects of tides, storm surges, waves, erosion and overtopping. Instead this investigation has been undertaken in four parts:

1. An extreme wave study
2. An extreme beach profile morphological study
3. An extreme wave overtopping study
4. A wave action study

This report summarises the available data at the site and the results of each assessment. In addition to this introductory section, this report contains the following sections:

- **Section 2:** Data Review
- **Section 3:** Extreme wave modelling
- **Section 4:** Morphological modelling
- **Section 5:** Wave overtopping modelling
- **Section 6:** Wave action study

Three appendices are included:

- **Appendix A:** Extreme Wave Analysis
- **Appendix B:** Wave overtopping sheets for 90% detailed designs

2 Available data

A range of datasets are available at a regional scale as well as specific to the proposed site. These provide information on the proposed design, tides, extreme waves, and the underlying bathymetry.

2.1 Event frequencies

This report has adopted the preferred terminology for event frequency description outlined in Book 1, Chapter 2.2.5 of Australian Rainfall and Runoff (ARR)[1]. Very frequent events, occurring at least once per year, are referred to by exceedances per year (EY). Frequent to very rare events are referred to by average exceedance probability (% AEP). For ease of reading, AEP events are also referred to by their respective average recurrence interval (ARI) in the first instance, however the ARI frequency terminology is being phased out by industry.

2.2 Height datums

All height data is relative to the Australian Height Datum (AHD), unless otherwise specified.

2.3 Designs

Coastal designs have been provided to JBP for assessment. These designs seek to balance foreshore green space, beach area, and a renewed seawall with beach access. Modelling has been based on civil and structural drawings, discussions and markups provided by PDG. A general design layout and the location of cross-sections used in modelling is shown in Figure 2-1.

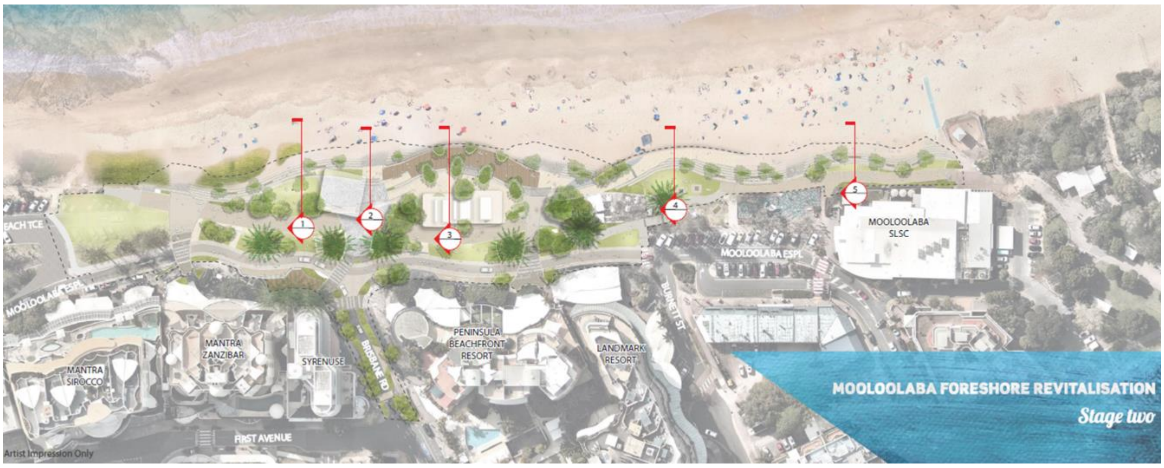

Figure 2-1: General design layout (PDG)

2.4 Offshore bathymetric data:

A range of data has been used to develop the numerical models.

- **GA 30m GBR30 bathymetric data 2018[2]:** A compilation of digital elevation models (DEM) at a regional scale. Data collation consists of deep-water multibeam surveying, airborne lidar bathymetry, and chart data.
- **SCC 5m bathymetric LiDAR 2013:** Supplied by Sunshine Coast Council. High resolution 5m bathymetric LiDAR from 0m to 30m depths in the lower estuarine reaches and offshore of the Maroochy and Noosa Rivers.

Survey data of the coastal zone:

1 Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia), 2019

2 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.

- **February 2013 post-storm beach contours**: This survey was undertaken by SCC to determine beach loss following TC Oswald in early 2013. This data has been used to calibrate the XBeach model.
- **The Coastal Observation Program - Engineering (COPE) data:** The COPE project was operational from 1971–1996 and aimed to collect local information in areas where extensive investigations were not viable and where little or no data existed. In doing this, COPE's broader aims were to assist in the understanding of coastal processes and their effects on the local coastline.
- **Periodic beach survey:** Provided by Sunshine Coast Council (SCC) for Mooloolaba beach. This data is available from 2016 to 2022 at regular intervals, resulting in a comprehensive set of contemporary cross shore profiles for the study location. Figure 2-2 shows an example of beach survey adjacent to Mooloolaba SLSC

Topographic data:

- **SCC 1m LiDAR 2014[3]:** Additional elevation data has been sourced from the 2014 LiDAR dataset. This data has an LGA-wide coverage and is available at a 1m resolution down to the waterline.
- **Design information** - all proposed design information has been provided to JBP for this assessment.

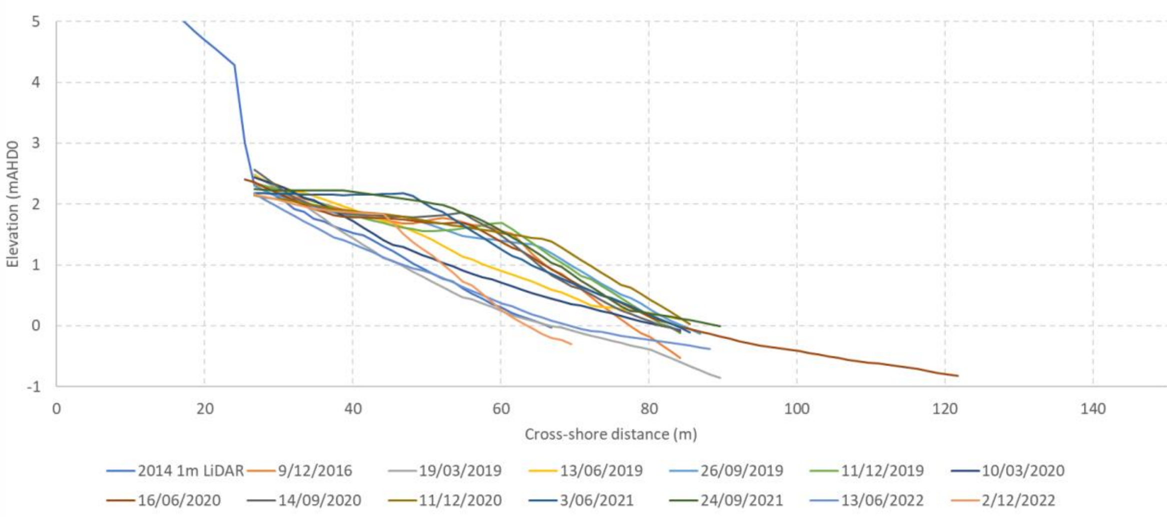

Figure 2-2: Cross-shore beach survey data adjacent to Mooloolaba SLSC

2.5 Tidal and extreme sea levels

Tidal plane information has been derived from the Queensland Tidal Planes (MSQ, 2023[4]) at Mooloolaba (PSM 37055). Modelling under typical conditions has used a Mean High Water Spring (MHWS). Input storm tide levels have been sourced from the Sunshine Coast Storm Tide Study (Aurecon 2013)[5] and are not inclusive of wave setup, which is implicitly included within wave overtopping model results. An end-of-design timeframe has been interpolated for a 2074 planning horizon.

3 QLD Government (2014), Queensland LiDAR Data - Sunshine Coast LGA 2014 Project
4 Available from: https://www.msq.qld.gov.au/tides/tidal-planes
5 Aurecon (2014) Sunshine Coast Storm Tide Study. Prepared for Sunshine Coast Council

Table 2-1: Coastal water levels used in this assessment

Tide or ESL	Sea level (mAHD)		
	Present day	2074 (interpolated)	2100
MHWS	0.7	1.2	1.5
5% AEP	1.4	2.4	3.0
2% AEP	1.5	2.7	3.3
1% AEP	1.6	2.9	3.5

2.6 Water level data

Recorded water level data has been sourced from the Mooloolaba storm tide gauge: ID 011008A, Sept. 1978 – 2022. Astronomical tide data has been derived from the Utide python-based tool, which constructs the principle tidal constituents from the recorded signal and hindcasts the astronomical series[6].

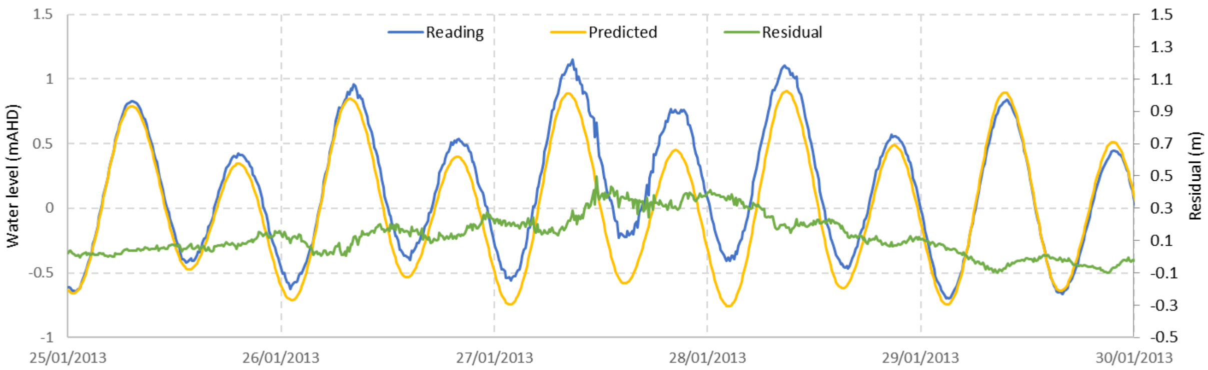

Figure 2-3: Water levels at Mooloolaba storm tide gauge during TC Oswald

2.7 Recorded wave data

Wave data used in model calibration has been sourced from recorded data at Mooloolaba wave rider buoy (WRB) which is located approximately 8km offshore of Yaroomba at a water depth of 32m. Data is available from 2000 to present day (directional from 2006). Figure 2-4 shows a wave rose for the Mooloolaba gauge.

6 Codiga, Daniel. (2011). Unified tidal analysis and prediction using the UTide Matlab functions. 10.13140/RG.2.1.3761.2008.

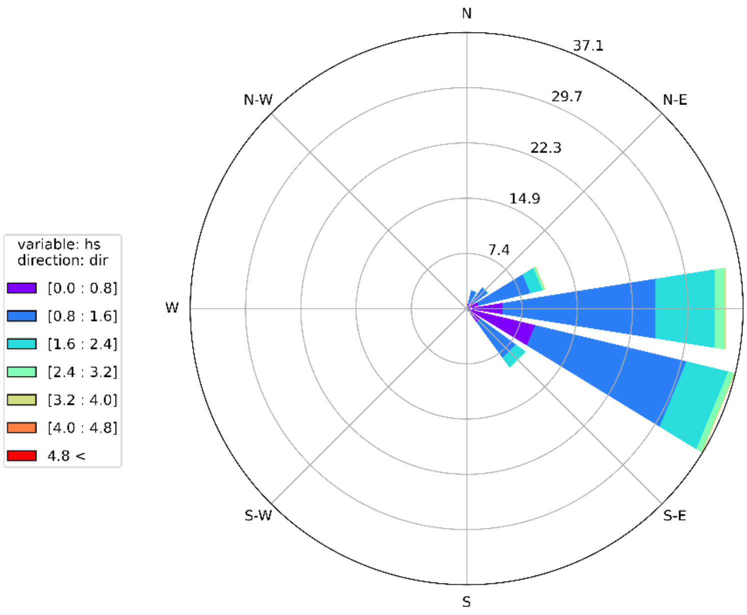

Figure 2-4: Wave rose for Mooloolaba WRB from 2016 to 2022

2.8 Sediment sampling

As part of this assessment, particle size distribution (PSD) testing has been completed on sand samples taken from the study site. This information is used to determine the median sand grain size (D50) used in morphological modelling. From sieve testing, the median grain size has been estimated at 0.2mm, a typical grain size of sandy, open-coast beaches in Queensland.

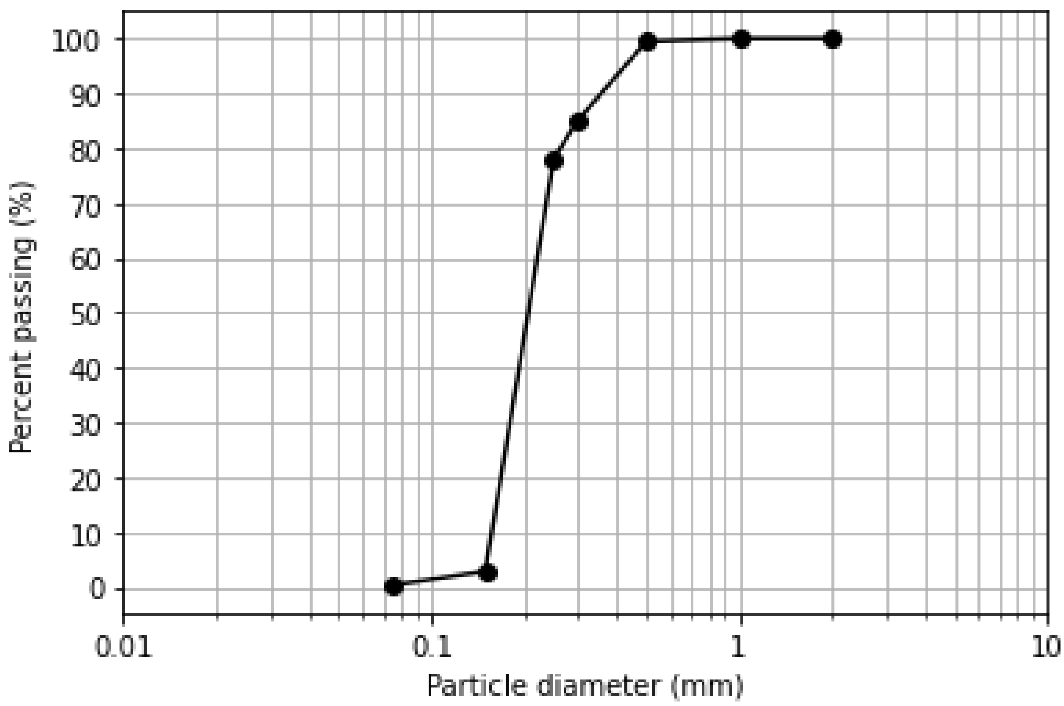

Figure 2-5: PSD testing of sand sample from Mooloolaba beach study site

2.9 Geotechnical survey data

Geotechnical survey conducted by Tectonic has been used in the assessment to estimate the depth of the underlying rock layer at each model location (included in Figure 2-6)[7]. This layer has been considered when schematising the wave overtopping models.

7 Tectonic (2023) Document: 23067-001 Draft Plan, Section and Logs

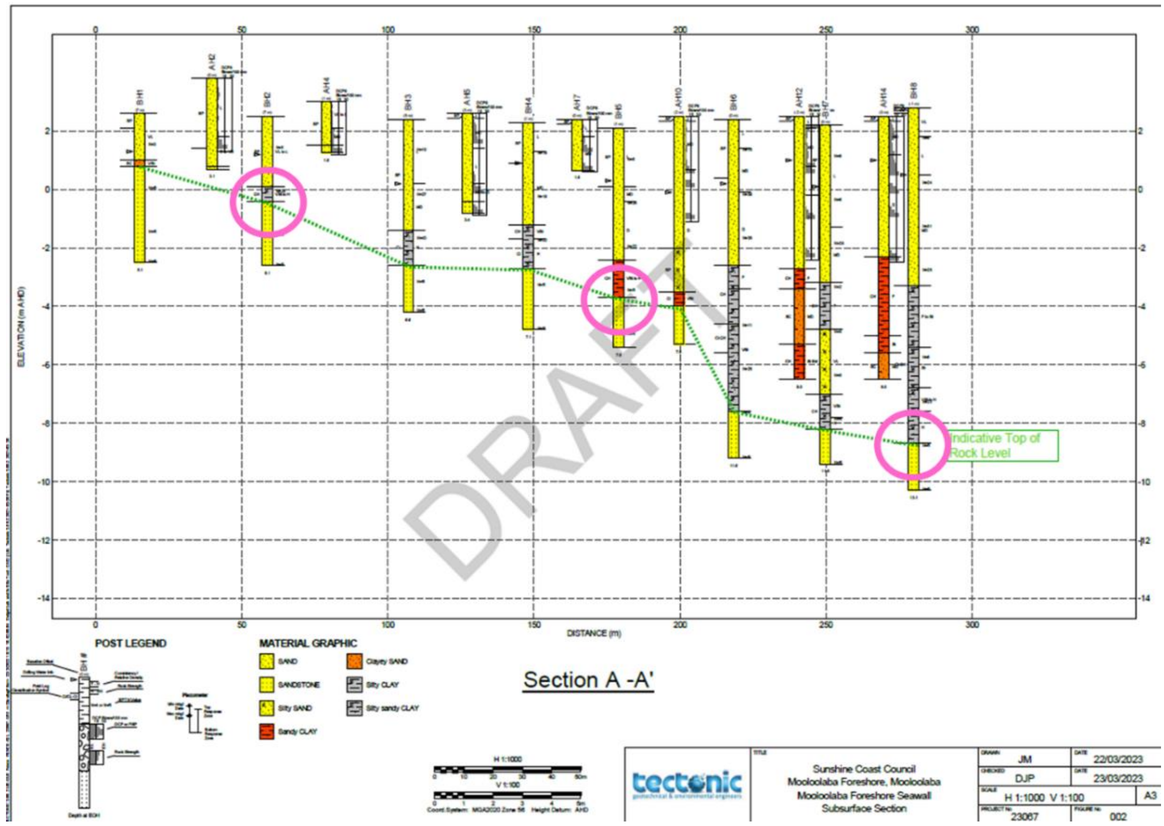

Figure 2-6: Depth of bedrock

3 Extreme wave study

3.1 Extreme offshore waves

Extreme value analysis (EVA) has been conducted for offshore wave data at the Brisbane Waverider Buoy (WRB) for use in wave modelling. A peak over threshold (POT) method has been used to isolate significant wave events and a generalised pareto distribution (GPD) fit to estimate the probability of extreme conditions. Figure 3-1 shows the fitting of the GPD function to wave data and estimation of extreme offshore wave heights at Brisbane WRB.

In the previous coastal processes study (BMT 2013)[8], a range of extreme offshore wave return periods were derived from the Brisbane WRB wave record. These conditions were applied to a numerical model and extracted at the -20m depth contour along the SCC. Since this study, an additional 10 years of wave data is available from the offshore buoy and the results of EVA on this updated dataset show that the conditions assessed in the previous study may underpredict extreme waves in the offshore, as shown in Table 3-1.

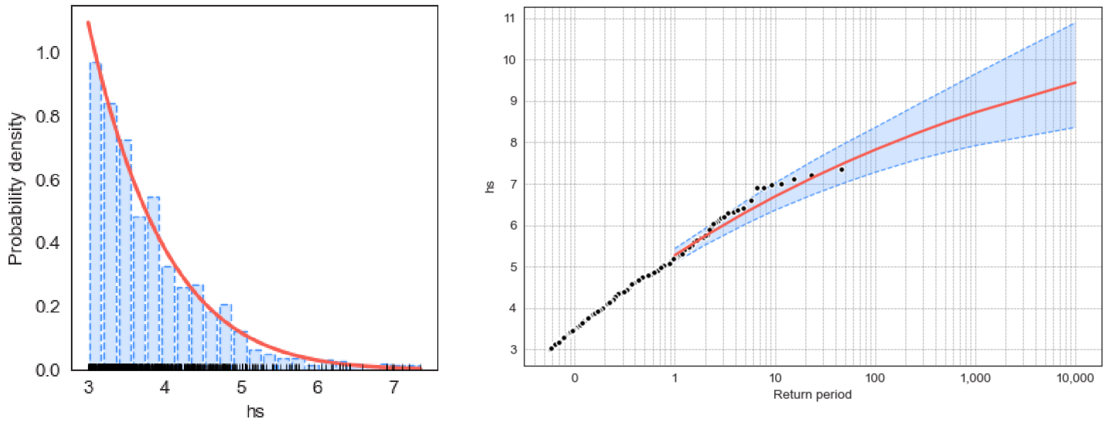

Figure 3-1 Left: GPD fit to wave heights above 3.0m and Right: GPD estimation of extreme wave height, for Brisbane WRB.

Table 3-1: Extreme wave height return periods for Brisbane WRB, compared with previous coastal assessment (BMT 2013)

ARI (yrs)	Hs (m) (JBP assessed)	Hs (m) (BMT (2013))	% increase
2	5.74	5.05	13.6%
5	6.30	5.85	7.7%
10	6.69	6.30	6.2%
20	7.10	6.70	6.0%
50	7.51	7.30	2.9%
100	7.82	7.80	0.3%

3.2 Extreme nearshore wave assessment

Whilst nearshore extreme wave estimates were evaluated in BMT (2013), their calculation approach followed a deterministic pathway where singular offshore extreme events were simulated through a wave model and extracted at the -20m contour. Updated nearshore extreme wave conditions have been assessed for the Mooloolaba study site using a probabilistic approach. This has developed a 10,000-year wave simulation, representing the full range of potential wave conditions, which were simulated into the nearshore region. The derived nearshore data preserves the marginal extremes for each variable and the dependency between the variables. These data can be used in the probabilistic design of structures. A full description of this methodology is presented in Appendix A.

8 BMT WBM (2013) Coastal Processes Study for the Sunshine Coast

Figure 3-2 shows a nearshore wave rose at the -10m AHD contour offshore of Mooloolaba Beach (O9). This wave rose displays the distribution of wave height and wave direction for the full large wave dataset. The wave rose displays a north-easterly wave climate at Mooloolaba Beach due to wave sheltering at the headland of Point Cartwright.

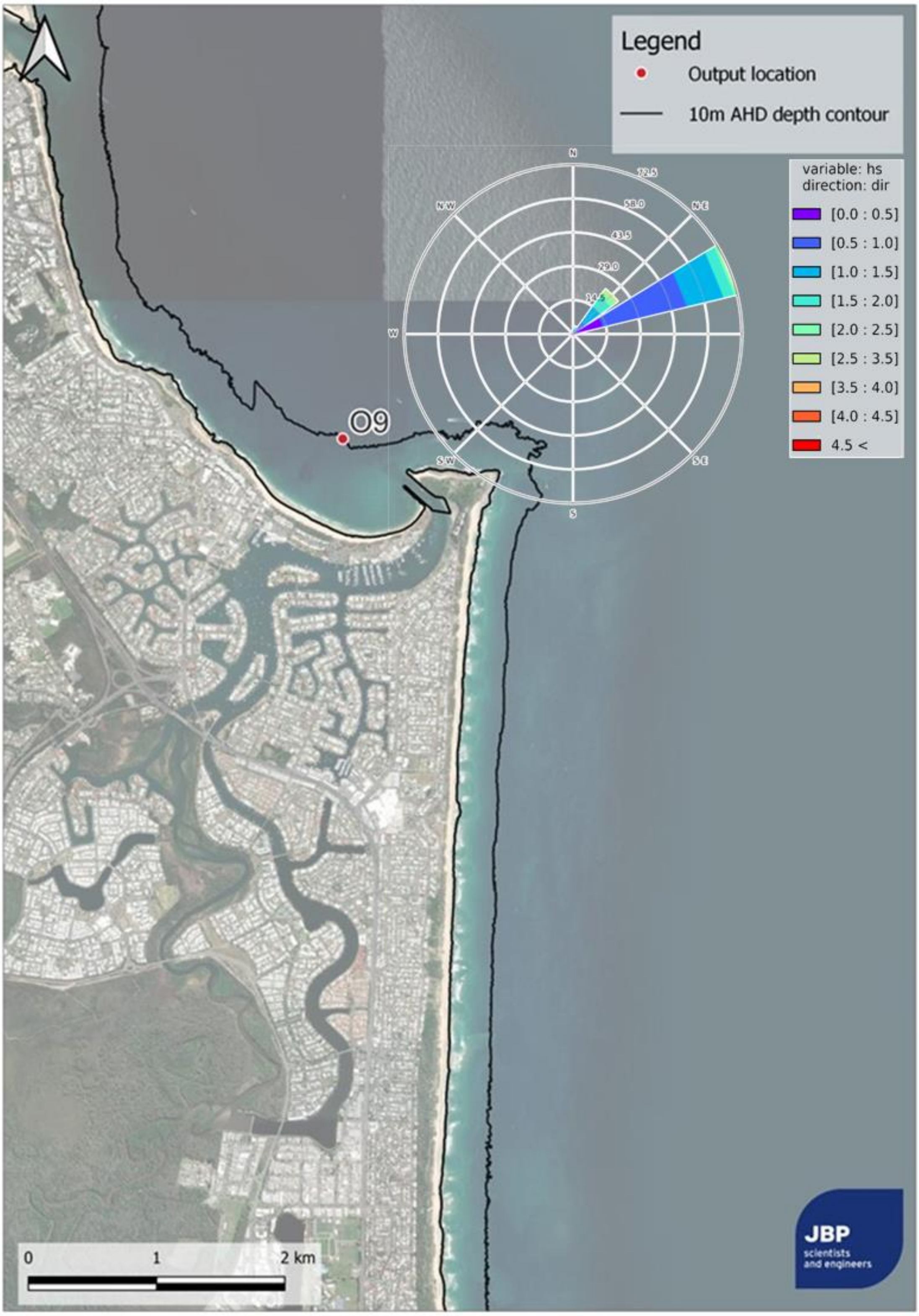

Figure 3-2: Present day emulated nearshore wave rose at Mooloolaba Beach

3.2.1 Present day nearshore extreme wave conditions

Extreme value analysis was conducted on the simulated data, with Table 3-2 showing extreme nearshore wave conditions for a range of return periods up to 0.1% AEP (1000-year ARI). The reassessed present day extreme nearshore wave conditions have been compared to values published in BMT (2013). In general the nearshore conditions show a decrease in extreme wave height.

Table 3-2: Present day nearshore extreme wave conditions at Mooloolaba Beach (O9).

	Hs (m), Tp (s), Dir (°N)					
Location	10%AEP	5%AEP	2%AEP	1%AEP	0.2%AEP	0.1%AEP
Mooloolaba Beach	3.5m 13.8s 44 deg° N	3.7m 14.1s 43 deg° N	4.0m 14.4s 42 deg° N	4.2m 14.9s 41 deg° N	4.6m 15.8s 39 deg° N	4.7m 16.1s 38 deg° N

4 Morphological modelling

4.1 Background

A beach is in a constant state of flux – changing in response to annual, seasonal and event-driven processes. The purpose of the morphological modelling was to understand how the current beach changes after a storm and if this will change significantly with the new designs. The modelling has targeted the most protruded sections of the proposed seawall to test three different typologies; a terrace, vertical and steeper design. This is therefore not a balanced assessment, instead focusing on the areas of greatest impacts (most protruding sea wall areas). For areas where the seawall retreats (moves landward) the impacts are expected to be less.

4.2 Method

The morphologic modelling has used XBeach, an open-source numerical model originally developed to simulate hydrodynamic and morphodynamic processes on sandy coasts and has been validated with a series of analytical, laboratory and field test cases using a standard set of parameter settings. The model includes:

- Short wave transformation (refraction, shoaling and breaking).
- Long wave (infragravity wave) transformation (generation, propagation and dissipation).
- Wave-induced setup and unsteady currents.
- Bed load and suspended sediment transport
- Non-erodible structure layers
- Dune face avalanching, bed update and breaching.

The model can be run in either 1D or 2D mode with either hydrostatic (phase-averaging) or non-hydrostatic (phase-resolving) wave simulation. For this assessment, the 3x cross-shore 1D models have been developed and simulated under hydrostatic wave conditions. This configuration requires significantly less computational time than a 2D model (although simulation times still remain in the order of 24-28 hours each), with the hydrostatic mode being better suited for modelling erosion and morphology.

Three sections along the extent of the proposed works have been modelled as 1D transects. Figure 4-1 shows the extent of the models which are located on the design profiles shown in Figure 4-2. These include:

- XBeach Model 1 (design profile 1): North of the existing Loo with a View (LwaV) building
- XBeach Model 2 (design profile 3): Midway between LwaV and Mooloolaba Surf Life Saving Club (SLSC)
- XBeach Model 3 (design profile 5): In front of Mooloolaba SLSC

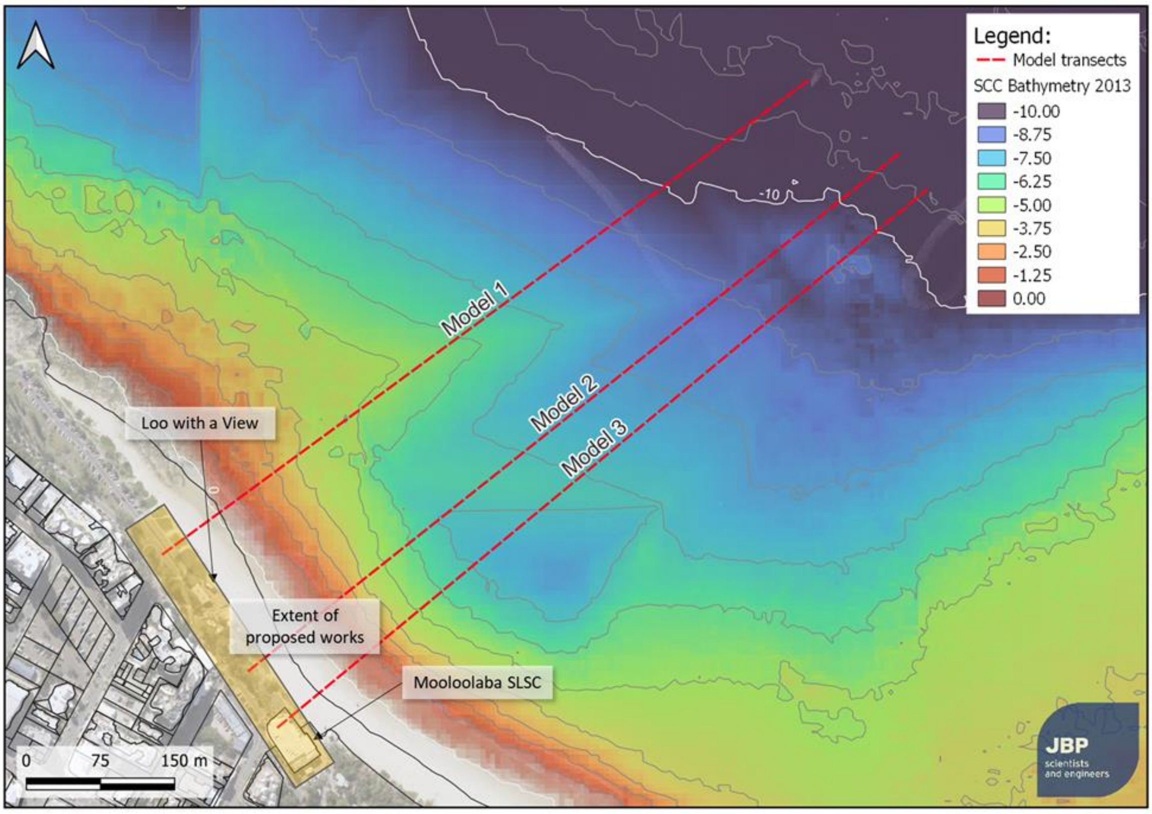

Figure 4-1: 1D cross-shore model locations for Mooloolaba morphological modelling

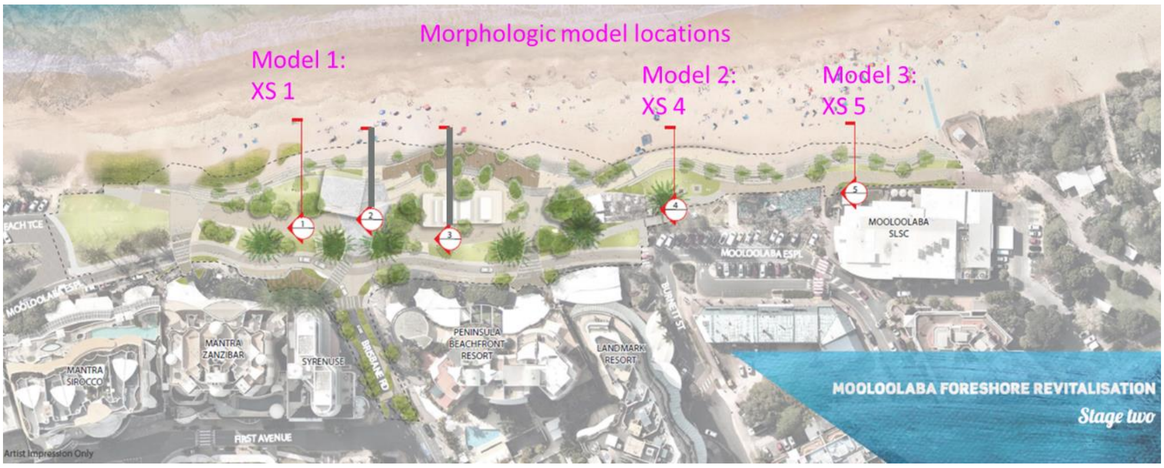

Figure 4-2: XBeach morphologic model locations, shown against design layout (PDG 3/7/2023)

4.2.1 Input structure configurations

Structural drawings for the existing and proposed seawall configurations at Mooloolaba CMP have been supplied by PDG. Morphological modelling was conducted at the 60% Detailed Design stage. For each model, the existing and proposed structure profile has been included in the model grid as a non-erodible layer.

4.2.1.1 Existing structures

For each modelled cross-section, the existing structure has been included in the model grid as a non-erodible layer. For Model 1 (north of LwaV), the existing structure is a partially buried rockwall comprised of sandstone boulders that is fronted by a well-vegetated dune. From construction drawings of the typical cross-section:

- Rockwall crest at 4.25mAHD
- Toe of buried rockwall at 0.45mAHD

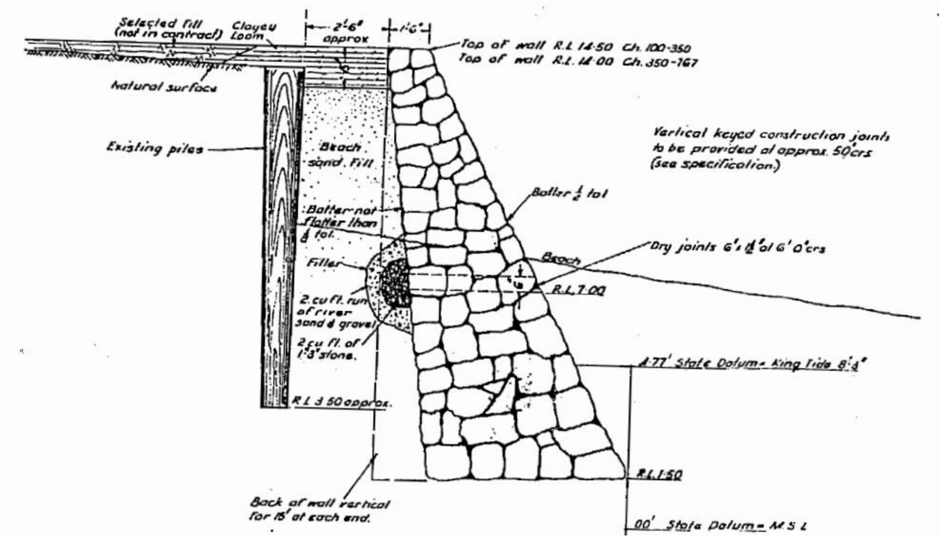

Figure 4-3: Top: Typical cross-section for existing structure in Model 1. Bottom: Facing south toward LwaV showing buried rockwall and fronting dune.

For Model 2 (middle transect) and Model 3 (southern transect at SLSC), the existing structure is an exposed rockwall. From construction drawings, the slope of this rockwall is variable along the alignment between 10V:1H and 2.1V:1H. Therefore, the structure slope for Models 2 and 3 has been derived from cross-sections of the 2013 post-storm beach survey conducted by SCC, as shown in Figure 4-4. From the typical cross-section:

- Rockwall crest at 4.5mAHD
- Toe of the exposed rockwall section at 1.21mAHD
- Toe of the buried rock revetment section is at -1.19mAHD

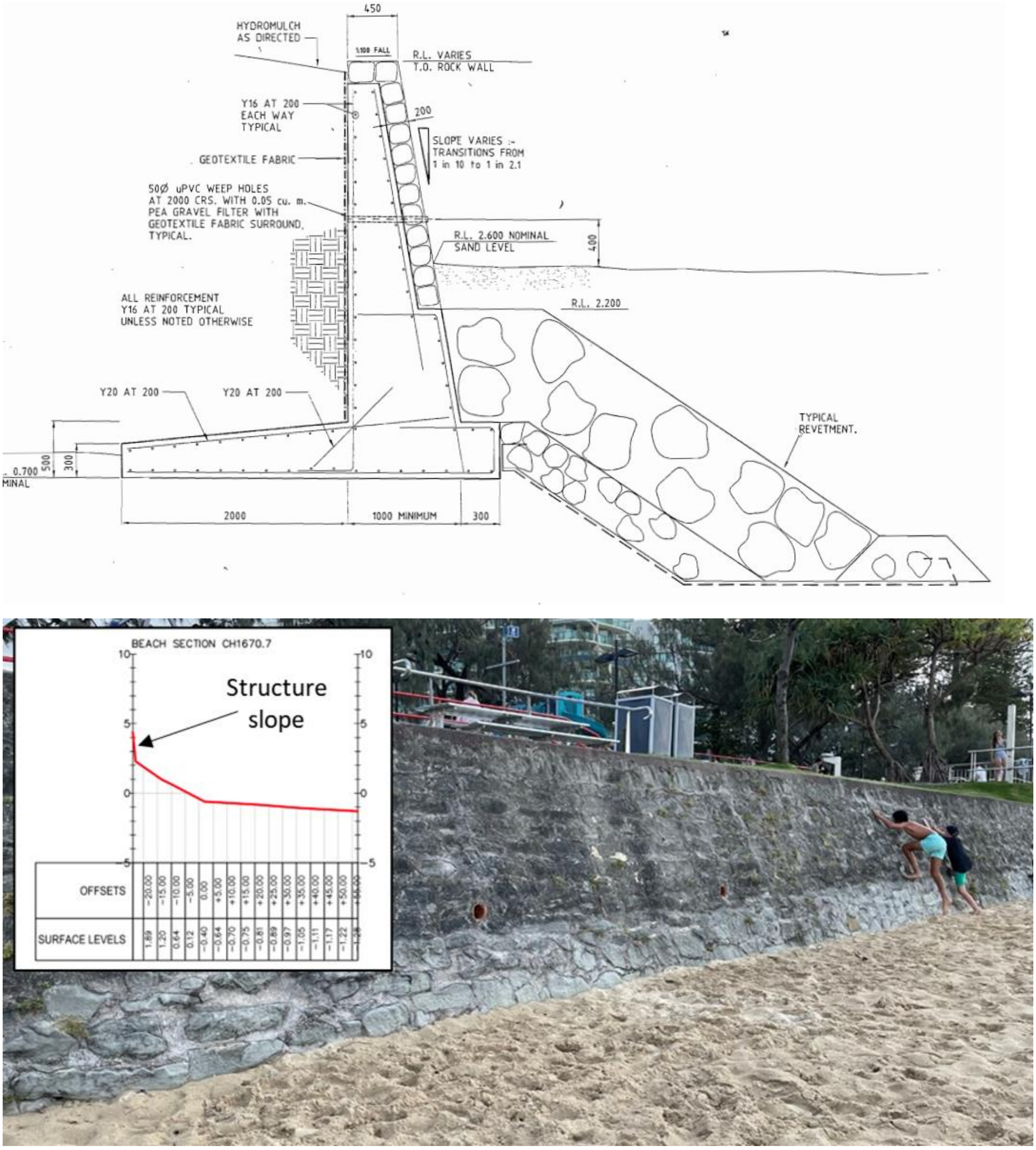

Figure 4-4: Top: Typical cross-section for existing structure in Models 2 and 3. Bottom: Facing north toward LwaV showing exposed, sloping rockwall structure (structure slope derived from 2013 surveyed profiles).

4.2.1.2 Proposed structures

For the 3x model transects the proposed structure has been included in the model as a non-erodible layer, and simulated during the 60% detailed design stage. Structure cross-sections have been derived from the Mooloolaba foreshore seawall layout concept drawings provided by PDG and Barlow Shelley[9]. As shown Figure 4-5, the models represent sections of stepped revetment and vertical walls.

To Note: Following the completion of the morphology modelling minor changes were made to these designs. This is discussed in Section 4.5

9 PDG (2023) Designs contained within document "2197-SK01.pdf". Issued 23/3/2023

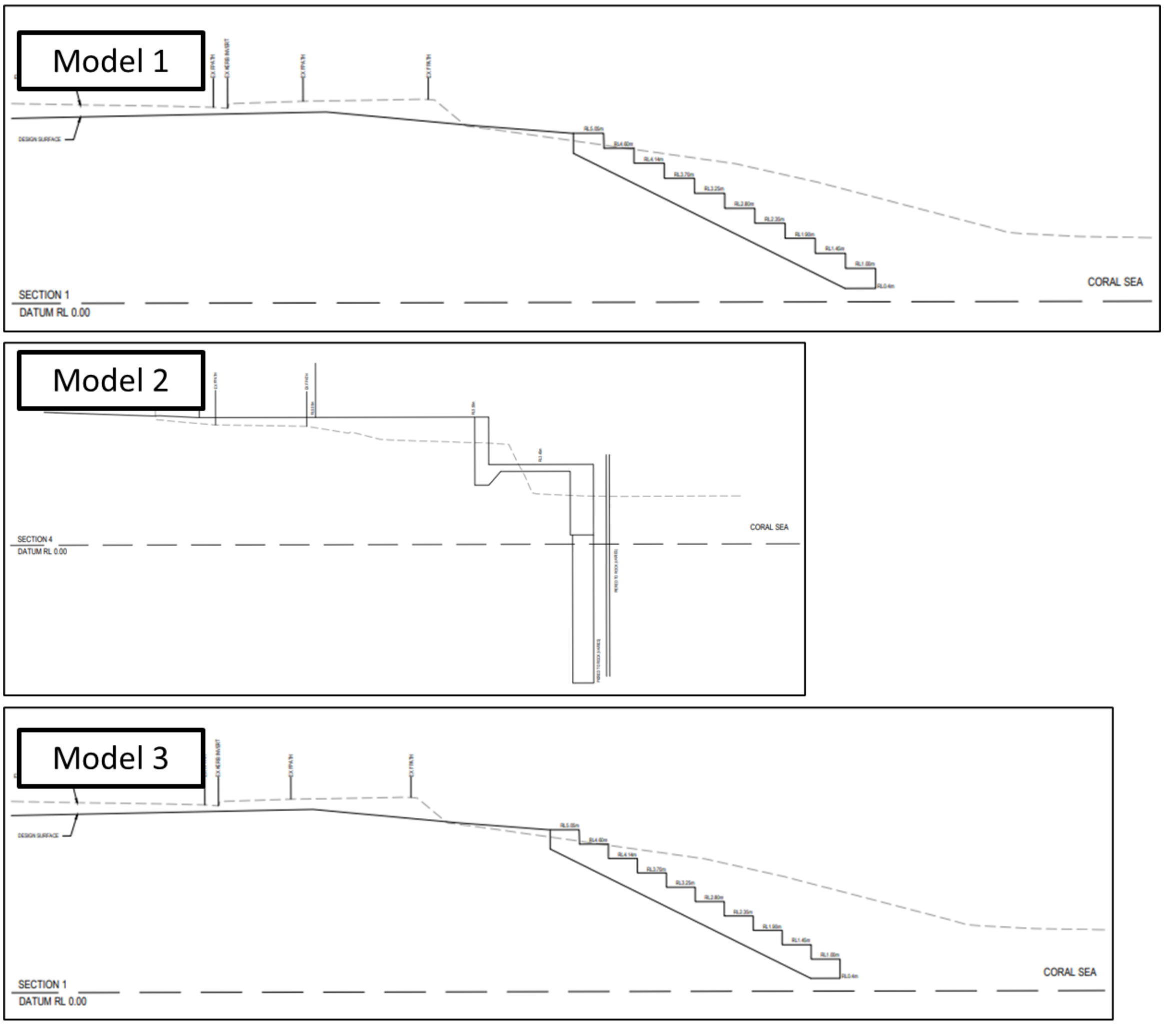

Figure 4-5: Proposed structure configurations for morphology models simulated at the 60% detailed design stage (Barlow Shelley 23/3/2023 "2197-SK01.pdf")

4.2.2 1D model grid

Each cross-shore model has been developed as a spatially varying 1D grid with a minimum grid cell size of 0.1m to observe small changes in erosion volume and beach width. Figure 4-6 shows the modelled cross-sections for both existing and proposed cases for each model. The following elevation data has been merged to establish the cross-shore grid for each model transect:

- 1m 2014 LiDAR in the foreshore
- The existing or proposed structure
- Most recent beach survey (December 2022)
- 5m 2013 SCC bathymetry LiDAR to the -10m contour

For each model, the structure profile and bed rock layer have been replicated in the model as non-erodible depth layer. From Mooloolaba CMP concept designs, it has been assumed that the existing beach profile will be re-established following construction of the proposed seawall upgrade. This is most significant at Model 1, where the existing structure is almost entirely buried by a wide, well vegetated dune (Figure 4-3). For both Model 2 and Model 3, the existing buried rock revetment has been retained in the model as toe protection for the proposed structure. For each model the unerodable rock level has been derived from geotechnical survey, as per Section 2.9.

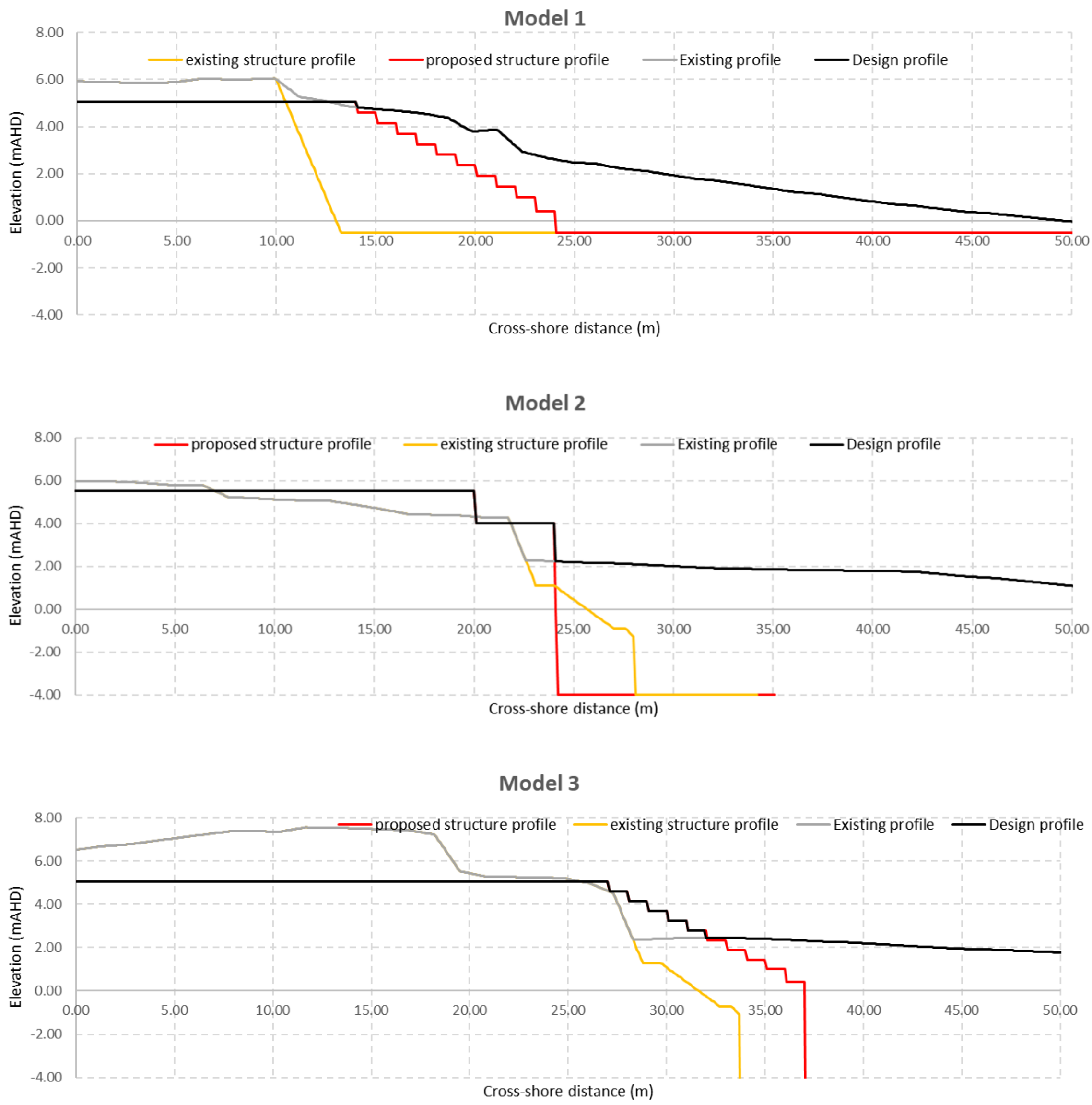

Figure 4-6: Existing and proposed structure configurations for Models 1, 2, and 3

4.3 Model calibration

The XBeach morphological models have been calibrated against an observed erosion event at Mooloolaba Beach. In January 2013 Tropical Cyclone (TC) Oswald formed in the Gulf of Carpentaria and crossed the northern peninsula before travelling south overland, following the east coast of Australia as a tropical low. Though this system tracked significantly inland from the Sunshine Coast, it produced north-easterly winds of up to 19 m/s (36 knotts) at Sunshine Coast Airport and a peak significant wave height at the Mooloolaba offshore buoy of 5.6m, resulting in substantial erosion along the Mooloolaba beach frontage. Following this event, SCC commissioned survey of the post-storm eroded beach between Urunga Esplanade and Brisbane Road. Profile data from this survey has been used to calibrate the morphological model (Figure 4-7).

Figure 4-7: Left: Erosion at the study area following TC Oswald (2013). Right: Post-storm surveyed profile

4.3.1 Input conditions

4.3.1.1 Wave and water level conditions

Figure 4-8 shows input conditions for model calibration, recorded wave data have been sourced from the Mooloolaba WRB for a period of 5 days to capture the duration of the 2013 event. Data gaps in the Mooloolaba WRB data during the event have been infilled with data from the nearby Moreton Bay North WRB, which shows good agreement to the Mooloolaba gauge. The model has been run with time-varying water levels extracted from the recorded data at Mooloolaba storm tide gauge.

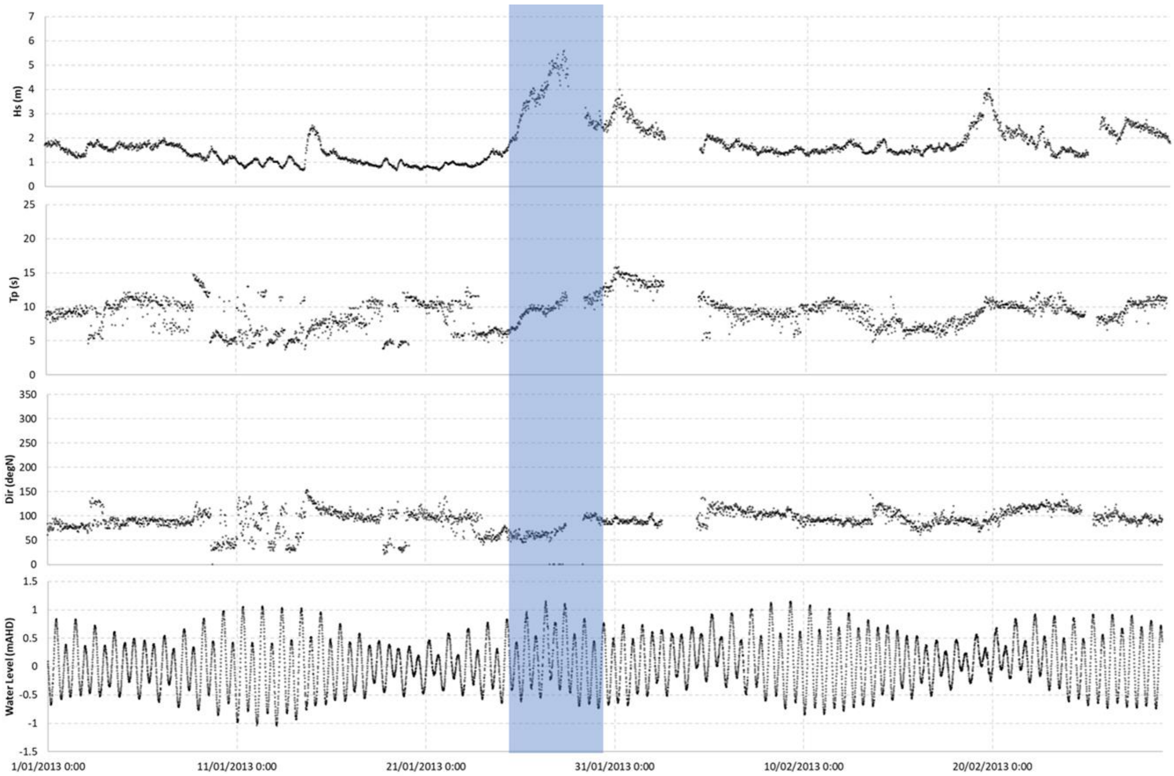

Figure 4-8: Input conditions for XBeach model validation, TC Oswald modelling period in blue.

4.3.1.2 Pre-storm profile

Survey data immediately predating the 2013 event is not available, therefore the pre-storm input profile has been developed as a long-term average of existing beach berm survey data at the Model 2 transect. The existing seawall slope has been derived from the Feb 2013 profile survey data. This data has been merged with nearshore bathymetric data sourced from the SCC 5m bathymetric LiDAR dataset. The use of this 'average' beach condition as the pre-storm profile is acknowledged to be a significant source of uncertainty within the model.

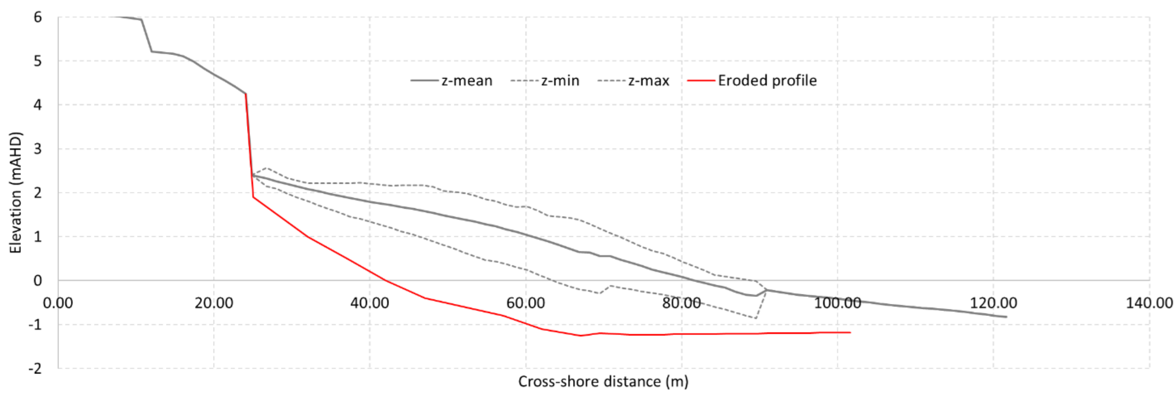

Figure 4-9: Input beach profile (solid), beach survey envelope (dashed), and surveyed eroded profile (red).

4.3.2 Results of calibration

The morphological model has been iteratively calibrated by modifying a range of model parameters including the morphological scale factor (*morfac*) and dune slumping angle (*dryslp*). Performance of the model at each stage of calibration has been assessed using the Brier Skills Score (BSS)

(Vousdoukas et al (2011)[10], van Rijn et al (2002)[11]) against the observed post-storm profile. The following morphological parameters produced the most accurate model result:

- *morfac* = 2
- *wetslp* = 0.2
- *dryslp* = 1.5
- *form* = vanthiel_vanrijn

The observed and best fit modelled profiles are shown in Figure 4-10. The modelled profile achieves a BSS rating of 0.4, considered a reasonable score (van Rijn (2002)). Discrepancies between the modelled and surveyed erosion profile are attributed to the lack of pre-storm beach profile data. Annual shorelines from the DEA Coastline database (Geoscience Australia, 2023[12]) show the 2012 shoreline position was further landward compared to future shorelines. This suggests that the pre-storm beach profile may have been more depleted than the long-term average profile used for the starting calibration profile. As such, the model with a BSS rating of 0.4 has been deemed to reproduce the expected outcome within a reasonable range and therefore suitable for use in design simulations.

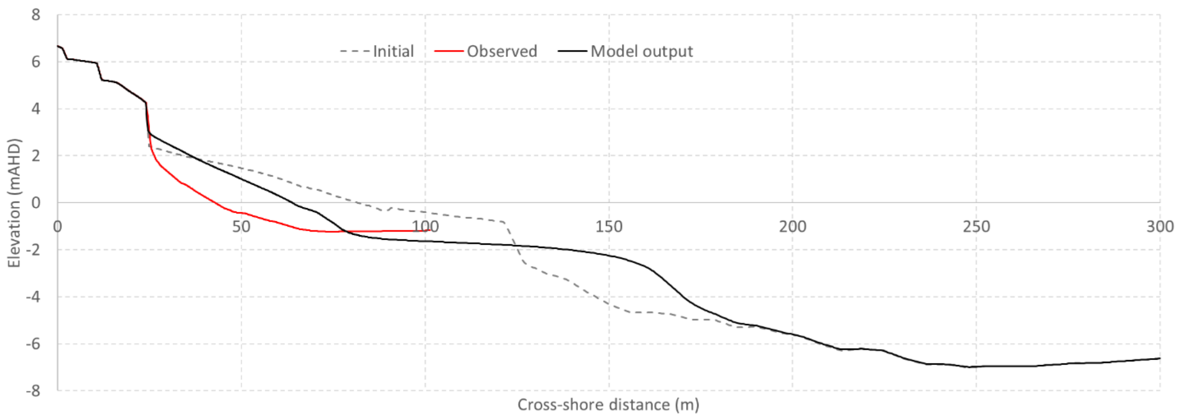

Figure 4-10: Input beach profile (dashed), output profile (black), and surveyed eroded profile (red).

4.4 Design storm simulations

Following calibration, the three 1D models have been simulated with design storm conditions. Two design events have been simulated:

- Event 1 - Inter-annual to decadal event: 50% AEP wave coinciding with a HAT water level
- Event 2 - Extreme design event: 1%AEP coincident wave and storm tide

Extreme wave conditions have been sourced from the updated extreme wave modelling (refer to Section 3). Input storm tide levels have been sourced from the Sunshine Coast Storm Tide Study (Aurecon 2013) and are not inclusive of wave setup. Water levels have been applied in the model as a time-varying water level, with the design level occurring at high tide. Design events have been applied for a period of 6 hours, representative of a high tide cycle.

10 Vousdoukas, Michalis & Ferreira, Óscar & Almeida, Luis Pedro & Pacheco, André. (2012). Toward reliable storm-hazard forecasts: XBeach calibration and its potential application in an operational early-warning system. Ocean Dynamics. 62. 1001-1015. 10.1007/s10236-012-0544-6.

11 Rijn, L.C & Walstra, Dirk-Jan & Grasmeijer, B.T. & Sutherland, James & Pan, Shunqi & Sierra, Joan. (2003). The predictability of cross-shore bed evolution of sandy beaches at the time scale of storms and seasons using process-based Profile models. Coastal Engineering. 47. 295-327. 10.1016/S0378-3839(02)00120-5.

12 Geoscience Australia, 2023. Available at: https://cmi.ga.gov.au/data-products/dea/581/dea-coastlines#about

4.4.1 Results of design storm modelling

Under the existing seawall design, the modelling shows there is the potential for over 200 m^3/m of sand to be lost from the beach in a significant storm (1% Annual Exceedance Probability, AEP), with almost all of the subaerial beach above 0mAHD lost during the design event. This occurs for both the existing and proposed structures. The proposed design typically has a wider footprint than the existing seawall and reduces the amount of available backshore sand which would typically be eroded in a storm event. This results in a reduced erosion volume, however typically deeper scour levels adjacent to the structure.

The comparison of remaining beach width is one metric which may be used to consider the changes between erosion under the existing and proposed structures. This has been measured along the 0m contour, as schematised in Figure 4-11. The post-storm profile under both seawall scenarios is substantially eroded. However, in terms of remaining beach width, the existing seawall scenario typically has a wider beach (at 0m AHD) than the proposed seawall design, as shown in Table 4-1 to Table 4-3. There is only one outlier, with model 3 showing an increased beach width for the proposed design under the inter-annual storm event. However for the extreme 1% AEP, the beach is again narrower than the existing design.

The large erosion volume lost from the beach occurs under both the existing and proposed designs. If a wide usable beach was required, beach nourishment would be required for both design options.

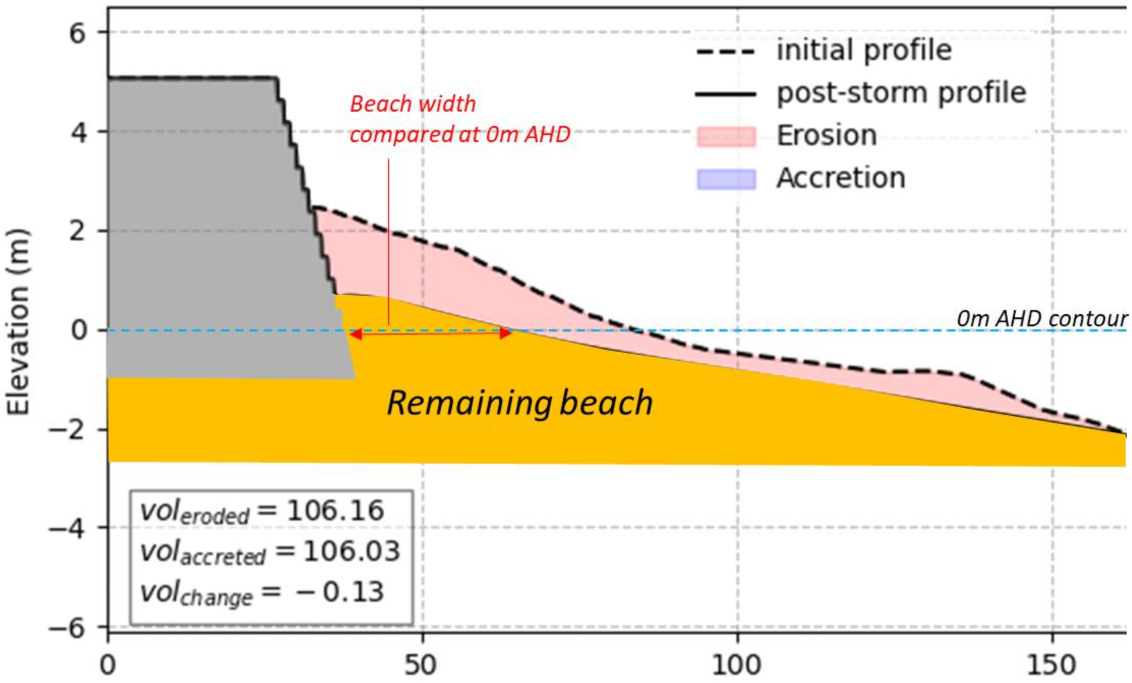

Figure 4-11: Post event beach for the proposed seawall, model 2 (middle section), Design Event 2 (1% AEP storm).

Table 4-1: Eroded volume and beach width at 0mAHD contour for Model 1:

	Volume			Beach width at 0mAHD		
Event	Existing seawall (m3/m)	Proposed seawall (m^3/m)	% change	Existing seawall (m)	Proposed seawall (m)	% change
Inter-annual to decadal	67.5	42.1	-38%	29.7	14.6	-51%
Extreme design event	79.3	48.0	-39%	23.3	12.7	-45%

Table 4-2: Eroded volume and beach width at 0mAHD contour for Model 2:

	Volume			Beach width at 0mAHD		
Event	Existing seawall (m3/m)	Proposed seawall (m^3/m)	% change	Existing seawall (m)	Proposed seawall (m)	% change
Inter-annual to decadal	149.4	155.0	4%	1.7	0	-100%
Extreme design event	214.7	223.4	4%	0	0	-

Table 4-3: Eroded volume and beach width at 0mAHD contour for Model 3:

	Volume			Beach width at 0mAHD		
Event	Existing seawall (m3/m)	Proposed seawall (m^3/m)	% change	Existing seawall (m)	Proposed seawall (m)	% change
Inter-annual to decadal	152.9	106.2	-31%	15.5	25.5	65%
Extreme design event	213.2	209.8	-2%	11.8	7.1	-40%

4.5 Structural design changes and likely influence on beach morphology

The XBeach morphology modelling was undertaken at the 60% detailed design stage. At the 90% detailed design stage (i.e. following the completion of the morphology modelling), minor changes were made to the designs which are shown in Figure 4-11 to Figure 4-13[13]. A summary of expected changes to the model results is presented below.

- Model 1/Cross section 1. The toe of the proposed wall has shifted landward and a change in grade of the lower terraces introduced, transitioning from a 1:2 to 1:1. This change is considered minor. Any landward shift of the proposed structure is expected to have less change from the existing structure, and consequently using the new design the proposed structure is expected to have a wider post-storm beach than the results published in Section 4.4.1.
- Model 2/Cross section 3. The toe of the proposed wall has shifted seaward. This change is likely to be insignificant on modelling results and within its margin of error. No changes are expected to those published in Section 4.4.1.
- Model 3/Cross section 5. The toe of the proposed wall has shifted seaward. This change is considered small, however any seaward shift of the proposed structure is expected to have more change from the existing structure. Using the new design, the proposed structure is expected to have narrower post-storm beach than the results published in Section 4.4.1.

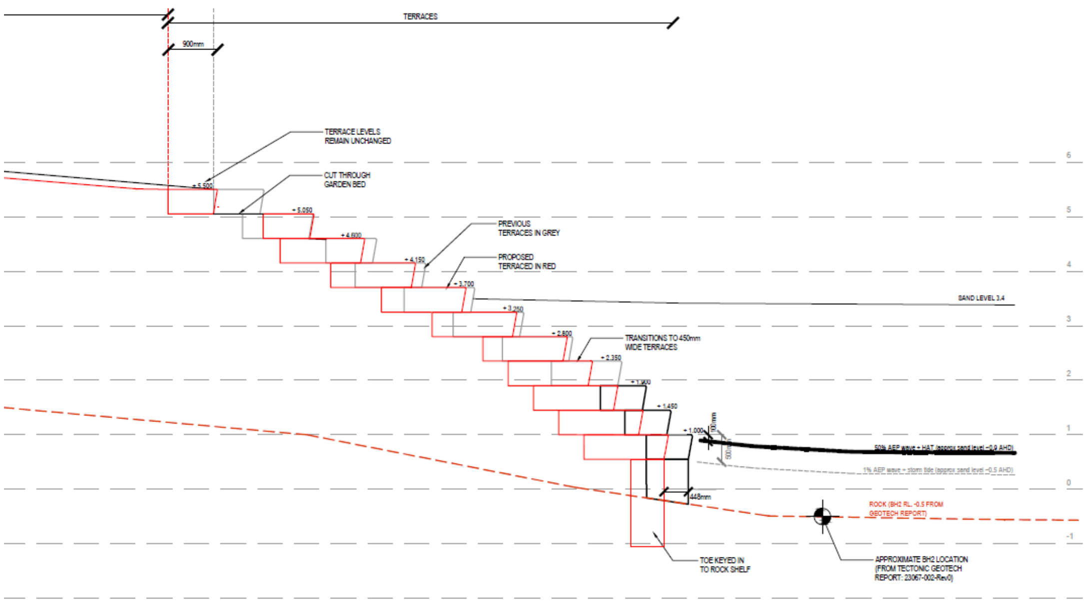

Figure 4-12: Model 1/cross section 1 at 60% detailed design (gray) - simulated in XBeach, and the revised cross section at 90% detailed design (red).

13 PGD (05 July 2023) 2022016_SurfClub seawall Section-SECTION UPDATES.pdf

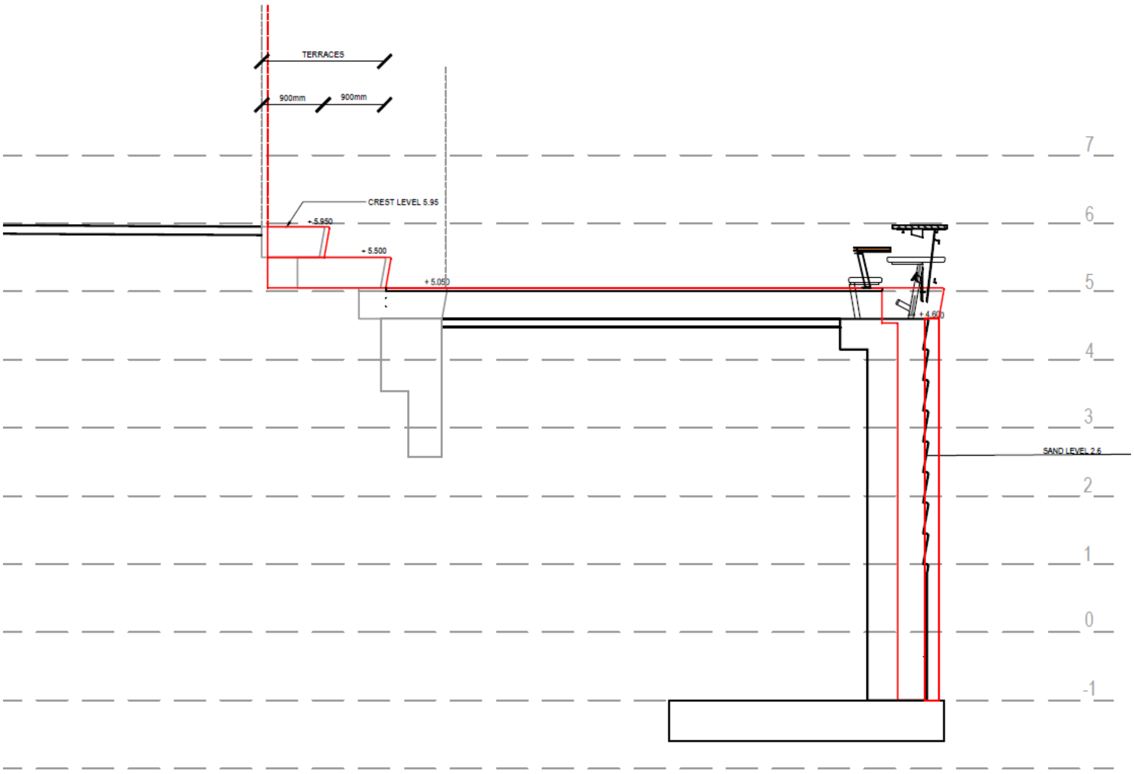

Figure 4-13: Model 2/cross section 3 at 60% detailed design (gray) - simulated in XBeach, and the revised cross section at 90% detailed design (red).

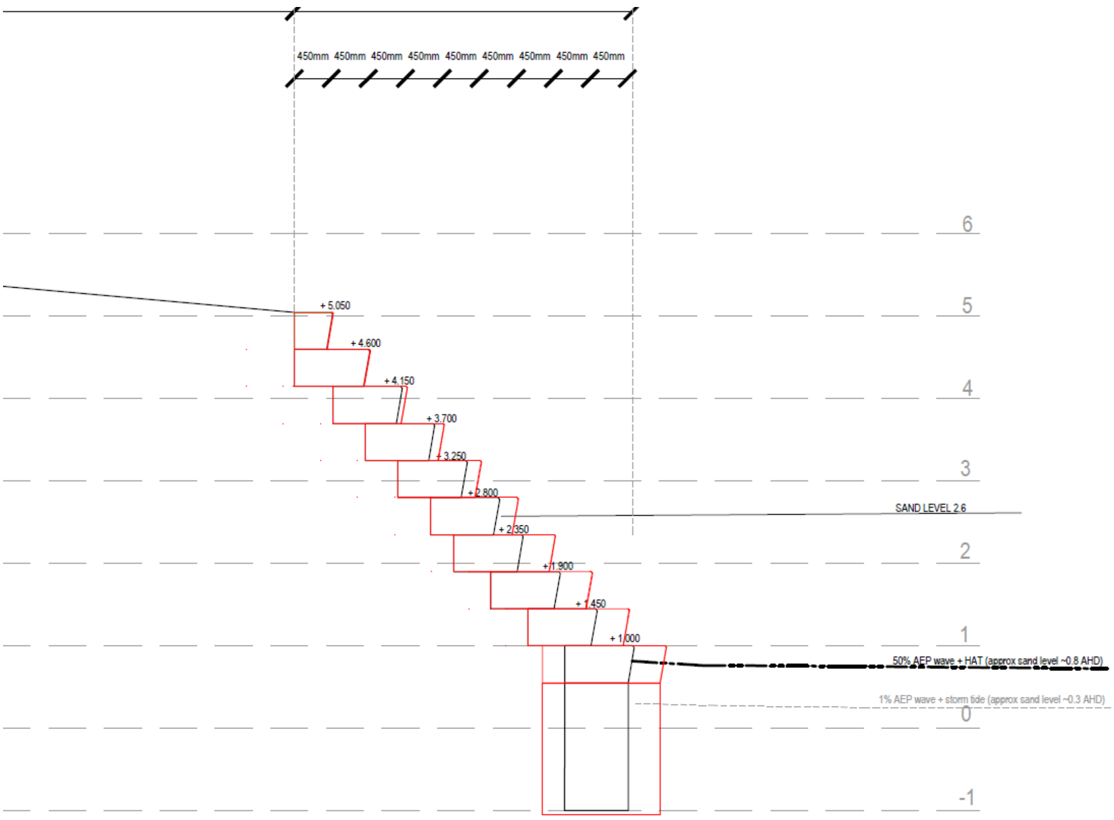

Figure 4-14: Model 3/cross section 5 at 60% detailed design (gray) - simulated in XBeach, and the revised cross section at 90% detailed design (red).

5 Overtopping modelling

5.1 Introduction

The expected wave overtopping of the proposed structure has been assessed. The complexity of the physical processes leading to wave overtopping introduces a high degree of uncertainty into its quantification. As a result, the overtopping caused by individual waves is not typically calculated; instead, the average overtopping rate for a sea-state is estimated using empirical equations, neural networks or physical models.

Wave overtopping modelling has been undertaken using the Neural Network (NN) calculation tool developed with the industry standard EurOtop Manual[14]. The NN tool provides the most suitable methodology for evaluating wave overtopping for composite defences such as seawall structures with armour. Even so, as with all calculation approaches, the Neural Network tool has limitations. Estimates are given based on a dataset of small-scale physical model tests which are affected by model and scale effects, and the accuracy of measurement equipment and wave generation techniques. There is also the potential for limited data for certain schematisations for example, overtopping across wide structures as few model tests are available within the database. As a result, it is important that the results of the Neural Network tool are used with a degree of engineering judgement and caution. The manual suggests the tool is only suitable for use in conceptual design and physical modelling should be used for detailed design - which is recommended by the asset owner if further certainty is required.

The Neural Network tool can be applied to different coastal designs. This requires the design to be 'schematised', a process which requires the structure to be split into 15 geometric parameters including: crest height (Rc); armour height (Ac); armour width (Gc); berm elevation (hb); berm width (B); upper slope (αu); lower slope (αd); and roughness (γf) (see Figure 5-1).

When schematising the cross-section designs, a mid-storm beach level of 0m AHD was used adjacent to the toe of the structure. This is based on the morphologic modelling, which shows the post-storm beach profile could erode to around -1m AHD, which reflects the sand levels after the event. Based on the full-storm erosion of -1.0m AHD, a mid-storm erosion level of 0.0m AHD has been adopted for wave overtopping calculations.

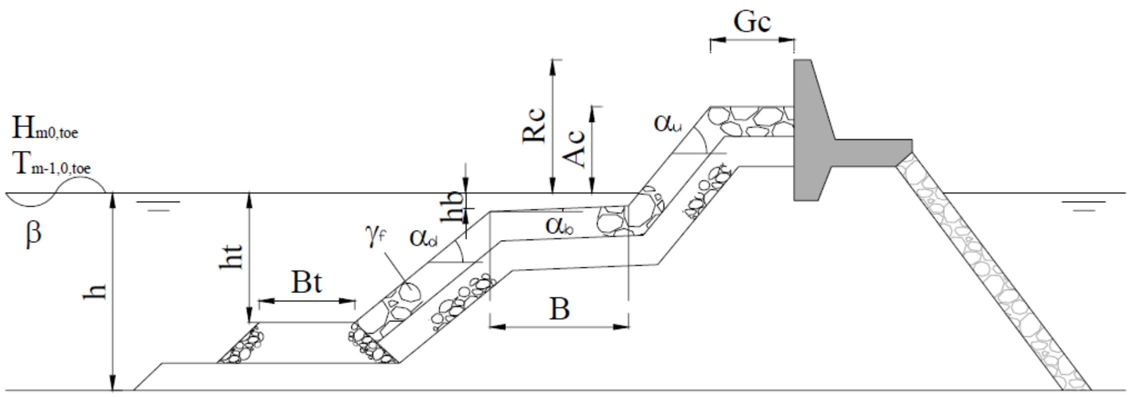

Figure 5-1 Neural Network structure schematisation of geometrical and hydraulic parameters

Using the Neural Network model, the average rate of overtopping can be calculated for a structure cross-section at the crest. The tool is unable to estimate overtopping at distances further inland of the coastal profile. If required, this is estimated through a 'rule of thumb' presented within the EurOtop 1 manual[15], where the hazardous effect of overtopping discharge at a landward position is expected to reduce by a factor of x. This is expressed as:

- Overtopping (landward) = Overtopping (crest) / distance (over a range of 5–25 m).

14 EurOtop. 2018. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. www.overtopping-manual.com

15 Pullen, Allsop, Bruce, Kortenhaus, Schüttrumpf, van der Meer (2007) EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual. Equation 3.1,pp 33

The overtopping rate at the crest, or a setback position, can then be related to guidance given in the EurOtop manual which relates hazardous situations to overtopping rates and volumes. The tolerable limits for pedestrians and vehicles are given in Table 5-1 and Table 5-2 respectively. The limits for damage to the defences by overtopping discharge is presented in Table 5-3.

Table 5-1: Limits for overtopping for pedestrians. Source: EurOtop

Hazard type and reason	Mean discharge Q (l/s/m)	Max volume Vmax (l/m)
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower level only, no falling jet, low danger of fall from walkway	1-10	500 at low level
Aware pedestrian, clear view of sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	0.1	20-50 at high level or velocity

Table 5-2: Limits for overtopping for vehicles. Source: EurOtop

Hazard type and reason	Mean discharge Q (l/s/m)	Max volume Vmax (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed	10 - 50[16]	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01 – 0.05[17]	5 – 50 at high level or velocity

Table 5-3: Limits for overtopping for property and damage. Source: EurOtop

Hazard type and reason	Mean discharge Q (l/s/m)
Damage to building structural elements	1[18]
Damage to equipment set back 5-10m	0.4[19]
No damage to embankment/seawalls if crest and rear slope are well protected	50-200
No damage to embankment / seawall crest and rear face of grass covered embankment of clay	1-10
Damage to paved or armoured promenade behind a seawall	200
Damage to grassed or lightly protected promenade	50

Based on the guideline values above, the following criteria has been used to describe the overtopping:

- Mean overtopping rates over 200 l/s/m: Damage to well protected embankment/seawalls
- Mean overtopping rates over 50 l/s/m: Damage to grassed areas and pavers/promenades
- Mean overtopping rates over 10 l/s/m: Unsafe for trained staff (e.g. SES)
- Mean overtopping rates over 1 l/s/m: Unsafe for pedestrians, but accessible for trained staff

The decision has been made to apply a 1 /s/m threshold for aware pedestrians. This is higher than the 0.1 l/s/m specified within Table 5-1 due to the limitations within the Neural Network tool that very low values are difficult to meet with results becoming asymptotic, nearing the threshold but not crossing it.

16 Note: These limits relate to overtopping defined at highways.
17 Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind
18 Note: This limit relates to the effective overtopping defined at the building
19 Note: This limit relates to overtopping defined at the defence

5.2 Overtopping for the proposed design (completed at 60% detailed design)

Overtopping modelling was initially undertaken at the 60% detailed design stage. This was completed for four cross sections only (sections 1 to 4 in Figure 2-1), with the 5th and 6th section only investigated at the latter stages of the design.

At this 60% detailed design stage, the proposed designs considered crest heights of both 5.5m AHD and 5.95m AHD, with multiple calculation points used along the cross section. The results are shown in Table 5-4, and generally show the following trends for a 1% AEP storm:

- There was no overtopping under a present day scenario.
- Overtopping rates were within the limits of trained staff (e.g. SES) at:
  - Section 4, with a crest level of 5.95m AHD, which remained below 10 l/s/m in a 1% AEP event.
- Overtopping rates were beyond the limits of trained staff but below the threshold for damage to grassed areas and pavers/promenades at:
  - Section 1, with a crest level of 5.95m AHD, experienced 21 l/s/m in a 1% AEP event.
  - Section 2, with a crest level of 5.95m AHD, experienced 35 l/s/m in a 1% AEP event.
  - Section 3, with a crest level of 5.95m AHD, experienced 11 l/s/m in a 1% AEP event.

Table 5-4: Results of overtopping modelling (l/s/m), future (2073) for 60% detailed design sections

	MHWS	1 in 20-year	1 in 50-year	1 in 100-year
Section 1 (crest @ 5.5m AHD)	0	7	20	39
Section 1 (crest @ 5.95m AHD)	0	3	10	21
Section 2 (crest @ 5.5m AHD)	0	13	34	60
Section 2 (crest @ 5.95m AHD)	0	6	18	35
Section 3 (crest @ 5.95m AHD)	0	1	5	11
Section 4 (crest @ 5.95m AHD)	0	1	4	10

5.3 Proposed design and assessment changes at 90% detailed design

Changes to the modelling approach storm scenarios, input wave conditions and cross section designs were made at the 90% detailed design stage.

5.3.1 Modelling approach

At the 90% detailed design stage the Neural Network (v1) was used to calculate overtopping. This differs from the modelling at the 60% detailed design stage, which used the Neural Network (v2) tool. Both tools are similar and use an empirically-based dataset of physical modelling tests to predict the likely overtopping for a new structure. However as a different tool it may produce different overtopping results. A switch was made from the Neural Network (v2) to the original (v1) tool at the 90% detailed design stage due to its greater ability to run large numbers of design iterations.

5.3.2 Professional review by the Water Research Laboratory

At the 90% detailed design stage a technical review of the coastal assessment was undertaken by the Water Research Laboratory (WRL). The review reported the coastal modelling is of a good professional standard. The main items for further discussion were:

1. The most extreme values for storm events presented is 1% AEP / 100 year ARI. Design standards (e.g. AS 4997-2005 Guidelines for the design of maritime structures) require more extreme events to be considered.

2. Sea level rise of 0.5 m (2074) and 0.8 m (2100) has been adopted, with many parameters calculated for the end of the design life (2074). Consideration of other sea level rise scenarios would have advantages.
3. The adoption of a breaker index of 0.55 may be applicable in very flat bathymetry (as per the cited paper), but may not be applicable for Mooloolaba.
4. The adoption of a design scour level of 0 m AHD may be non-conservative without additional analysis.
5. The wave force design tool is only to be used for concept design, and that physical modelling be used for detailed design

Following the WRL review, the following items were changed in the modelling methodology:

- Extreme storm events: Additional modelling was undertaken for a rarer storm with a 1 in 500-year return period.
- Additional planning horizons and sea level scenarios were modelled. This includes a 2043 planning horizon and sea level scenario.
- A more detailed breaker calculation was used within the modelling (see Section 5.3.3).
- The adopted score level was lowered to -2m AHD.
- The benefits of physical modelling was discussed with the client, however the decision was made to remain using the model-based tool only.

5.3.3 Wave conditions

Following the WRL review, the approach to calculate nearshore waves at the structure toe was altered. This now uses the depth limitation criteria proposed by Kamphuis (2000[20]) where:

- Hsb=0.56e^(3.5m)*d e.g 1.09m (9% inc)

Where Hsb = the shoaled broken wave height, m=foreshore slope and d=water depth. Foreshore slope has been extracted from the post-event morphological model:

- Post storm profile for northern section: 1:53 grade (1.9%)
- Post storm profile for middle section: 1:40 grade (2.5%)
- Post storm profile for SLSC section: 1:44 grade (2.2%)

The result was an increase in wave height at the structure toe of around 9%.

5.3.4 Designs

The seawall design was developed further by PDG and Barlow Shelley to respond to the 60% detailed design overtopping results (i.e. those in Section 5.2), and through an iterative design process to meet the requirements of the 90% design. This included:

- Deepening the storm scour level to -1m AHD or where it intersects with rock.
- Altering the toe design to minimise potential fall zone exposure.
- Iterations to the land immediately behind the seawall to consider changes in grade and small alterations to step designs to minimise wave overtopping exposure.

A full set of designs that have been modelled are:

- Section 1:
  - Section 1 (crest at 5.5m AHD)
  - Section 1 (landward, 5.95m AHD)
- Section 2:
  - Section 2 (crest at 6.4m AHD)
  - Section 2 (mid-crest at 5.5m AHD)
- Section 3:

20 Kamphuis, J. W. (2000). Introduction to Coastal Engineering and Management (World Scientific Publishing Co. Pte. Ltd).

    - Section 3 (crest at 5.95m AHD)
- Section 4:
    - Section 4 (step at 5.05m AHD)
    - Section 4 (step at 5.05, calculated landward at 6.0m AHD)
    - Section 4 (step at 5.5m AHD (+1 step)
    - Section 4 (step at 5.5m AHD (+1 step) calculated landward at 6.0m AHD
    - Section 4 (crest at 6.0m AHD (+2 step)
    - Section 4 (step at 6.0m AHD (+2 step) calculated landward at 6.0m AHD
- Section 5:
    - Section 5 (at crest 5.5mAHD)
    - Section 5 (landward at 6m AHD)
- Section 6 (a new section near Section 5)
    - Section 6 (crest at 5.05m AHD)

The results of all modelling is presented in the following Sections, which has been provided to SCC and the design team to inform the project design. The additional modelling for Section 4 has resulted in the decision to increase Section 4 wall height in the design by 1 step to 5.50 AHD.

5.4 Section 1 overtopping results in 2074 (during/post 90% detailed design)

The seawall design was developed further by PDG and Barlow Shelley to respond to the 60% detailed design overtopping results (i.e. those in Section 5.2), which shifted the toe to -1m AHD or where it intersects with rock. This was undertaken to minimise potential fall zone exposure and resulted in reduced terrace dimensions of 450 x 450mm below the existing sand level.

Wave overtopping estimates have been calculated at two locations - the seawall crest and the 5.95m AHD contour behind the structure, as shown in Figure 5-2. Figure 5-3 shows the schematisation in the Neural Network. The structure is represented as a stepped revetment, with a slight change of grade at 1.9m AHD.

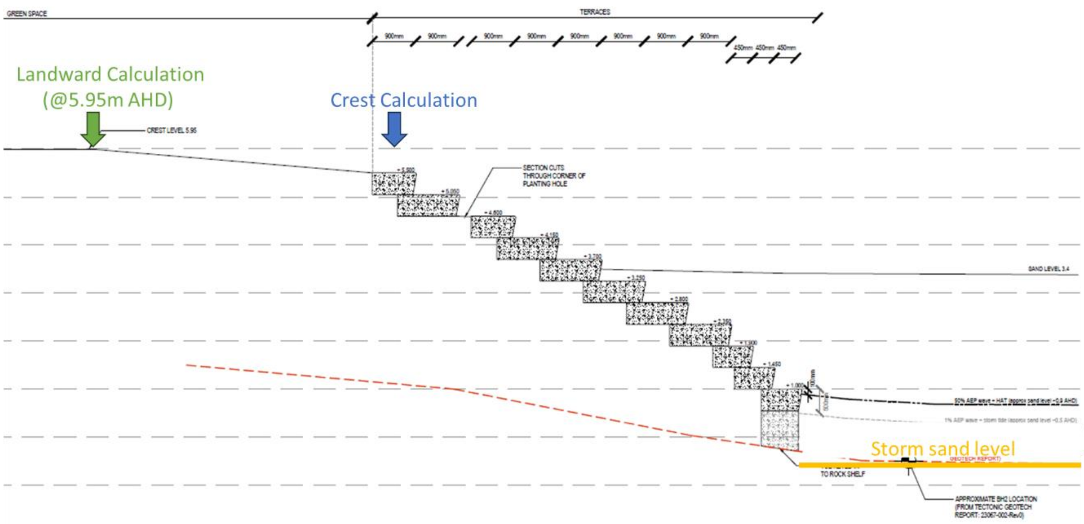

Figure 5-2 Section 1 design and overtopping calculation points

Predicted mean overtopping rates for all events are shown in Table 5-5. For the future 2074 1% AEP event:

- The overtopping rate is 14 l/m/s at the crest. This is beyond the safe limit for trained staff and SES.
- This reduces to 3 l/s/m at the 5.95m AHD contour behind the crest. This is within the tolerable limits of SES staff, but not pedestrians.
- Wave estimation further landward has used the EurOtop rule of thumb. This suggests the nature of overtopping is reduced to under 1 l/s/m a further 4m landward of the 5.95m contour. This is the adopted safe limit for aware pedestrians.

Table 5-5: Section 1: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Section 1 (crest at 5.5m AHD)					
Present day	0.0	0.7	1.1	1.4	1.9
2043	0.1	1.9	3.0	3.8	5.1
2173	0.3	7.9	11.2	14.4	17.0
Section 1 (landward, 5.95m AHD)					
Present day	0.0	0.2	0.3	0.4	0.5
2043	0.0	0.5	0.7	0.9	1.2
2173	0.1	1.8	2.6	3.4	4.0

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	Initials	Date
Designer:	CY	28/06/23
Checker:	AM	28/06/23
Approver:	DR	30/06/23
Office:	BRISBANE	

Input Data	
X	Y
-8	6
-8	6
0	5.5
7.2	1.9
9.6	0
9.6	-0.5
12	-0.5

Schematisation Inputs in Green - Automated Functions in Blue.		
TOE Data	***X***	***Y***
Front_of_toe_level_(mAOD)	9.6	-0.5
Toe_level_(mAOD)	9.6	-0.5
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	***X***	***Y***
Slope_downward_of_Berm_(cotangent), cot α_d	1.00	45.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00	26.57
Berm_level_(mAOD)	1.90	
BERM Calcs	***X***	***Y***
Start	7.2	1.9
End	7.2	1.9
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	0.90	
Berm level check	2.639	5.632
CREST Data	***X***	***Y***
Armour_crest_level_(mAOD)	0.0	5.50
Crest_level_(mAOD)	0.0	5.50
Width_of_Armour_crest_(m), Gc	0.00	
ROUGHNESS Data	***y***	
Upper Break in Roughness 1	5.5	
Lower Break in Roughness 2	1.9	
Roughness 1 - Grass Embankment	2.0	
Roughness 2 - Stepped revetment	0.8	
Roughness 3 - Stepped revetment	0.8	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-0.50
Toe_level_(mAOD)	-0.50
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	1.00
Berm_level_(mAOD)	1.90
Width_of_Berm_(m), B	0.90
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00
Armour_crest_level_(mAOD)	5.50
Width_of_Armour_crest_(m), Gc	0.00
Crest_level_(mAOD)	5.50
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.45
Break in Roughness 2	1.90
Roughness 1	2.00
Roughness 2	0.80
Roughness 3	0.80

Entered
Calculated
Review Notes

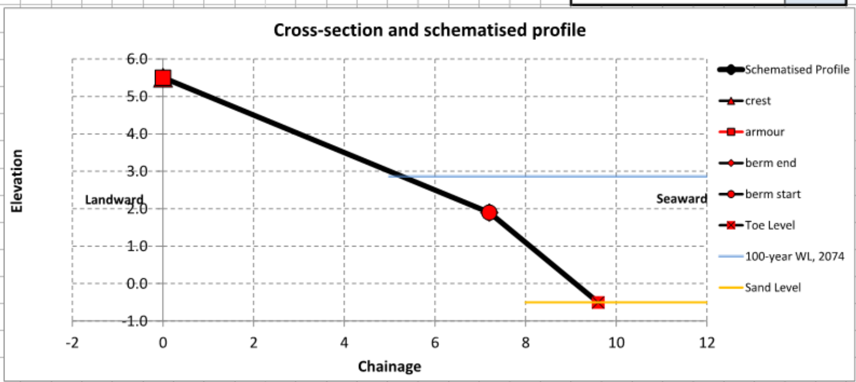

Figure 5-3 Section 1: example Neural Network inputs and schematisation (at crest)

5.5 Section 2 overtopping results in 2074 (during/post 90% detailed design)

The defence design and the overtopping calculation point has been changed to allow wave overtopping estimates along the middle promenade, as shown in Figure 5-4.

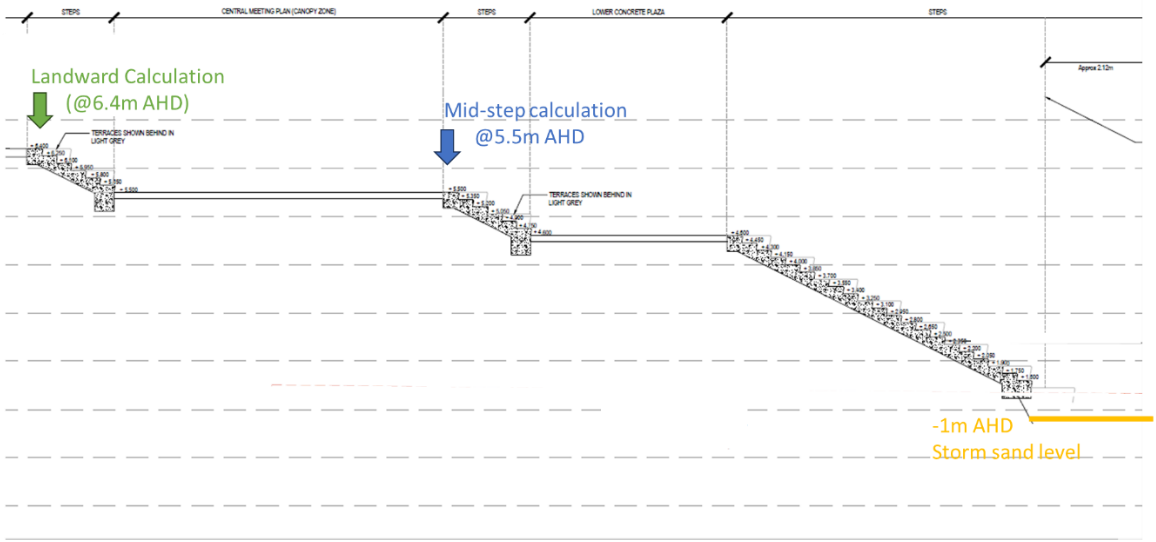

Figure 5-4 Section 2 design and overtopping calculation point

Figure 5-5 shows the schematisation in the Neural Network. The central/lower structure is schematised as a lower slope (stepped revetment) with the concrete plaza represented as a 4m wide berm at 5.05m AHD. Whilst the upper structure includes two sets of stairs this is unable to be schematised in detail in the Neural Network. Instead, an average 'upper slope' has been used to represent both upper stairs and their central 'canopy zone', reaching a crest level 6.4m AHD. This is based on the increase in elevation of 1.5m from the lower concrete plaza over a distance of 15m, giving an average upslope slope of 8.3 (cotangent).

The five storm scenarios were run and overtopping calculated at the mid-step and upper stair crest at 6.4m AHD. Predicted mean overtopping rates for all events are shown in Table 5-6. For the future 2074 1% AEP event:

- The overtopping rate is 5 l/m/s at the mid-stair crest. This is within the tolerable limits of SES staff, but not pedestrians.
- The overtopping rate is 4 l/m/s at the crest. This is within the tolerable limits of SES staff, but not pedestrians.
- Wave estimation further landward has used the EurOtop rule of thumb. This suggests the nature of overtopping is reduced to under 1 l/s/m a further 4m landward of the stair crest. This is the adopted safe limit for aware pedestrians.

Table 5-6: Section 2: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Section 2 (crest at 6.4m AHD)					
Present day	0.0	0.1	0.2	0.3	0.4
2043	0.0	0.5	0.7	1.0	1.3
2173	0.0	2.0	3.0	3.9	4.8
Section 2 (mid-crest at 5.5m AHD)					
Present day	0.0	0.2	0.3	0.5	0.6
2043	0.0	0.6	0.9	1.2	1.6
2173	0.1	2.5	3.7	5.0	6.2

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						Checker:	AM	28/06/23
Subject:	Overtopping Assessment - Section 2					***Approver:***	DR	30/06/23
						Office:	BRISBANE	

Input Data	
X	Y
-2	5.5
0	5.5
3.7	5.1
9.5	5.1
21.6	-1.0
21.6	-1.0
21.6	-0.5

Schematisation Inputs (Green) Automated Functions (Blue)		
TOE Data	***X***	***Y***
Front_of_toe_level_(mAOD)	21.6	-1.0
Toe_level_(mAOD)	21.6	-1.0
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	***X***	***Y***
Slope_downward_of_Berm_(cotangent), cot α_d	2.00	26.57
Slope_upward_of_Berm_(cotangent), cot α_u	8.30	6.87
Berm_level_(mAOD)	5.05	
BERM Calcs	***X***	***Y***
Start	9.5	5.1
End	3.7	5.1
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	5.80	
Berm level check	3.0515	6.0445
CREST Data	***X***	***Y***
Armour_crest_level_(mAOD)	0.0	5.5
Crest_level_(mAOD)	0.0	6.4
Width_of_Armour_crest_(m), Gc	0.00	
ROUGHNESS Data	***X***	***Y***
Upper Break in Roughness 1	5.1	
Lower Break in Roughness 2	6.4	
Roughness 1 - Earth Embankment	0.8	
Roughness 2 - Earth Embankment	0.8	
Roughness 3 - Concrete Revetment	0.8	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

8.3

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-1.00
Toe_level_(mAOD)	-1.00
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	2.00
Berm_level_(mAOD)	5.05
Width_of_Berm_(m), B	5.80
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	8.30
Armour_crest_level_(mAOD)	5.50
Width_of_Armour_crest_(m), Gc	0.00
Crest_level_(mAOD)	6.40
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.05
Break in Roughness 2	6.40
Roughness 1	0.80
Roughness 2	0.80
Roughness 3	0.80

Entered
Calculated
Review Notes

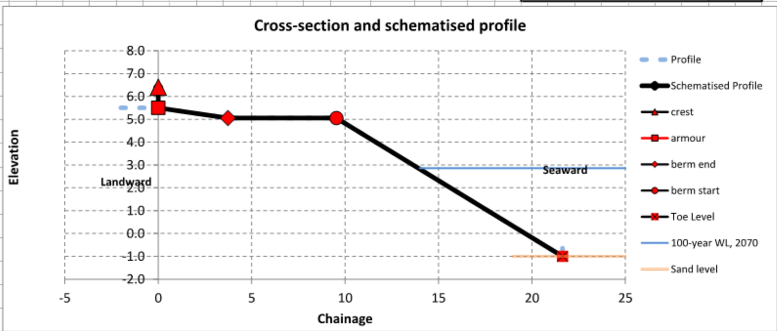

Figure 5-5 Section 2: Example Neural Network inputs and schematisation

5.6 Section 3 overtopping results in 2074 (during/post 90% detailed design)

Overtopping estimates have been calculated at one location at the 5.95m AHD crest behind the structure. The defence design and location of the single overtopping calculation point is shown in Figure 5-6. Figure 5-7 shows the schematisation in the Neural Network. The structure is represented as a vertical wall, with a 4.5m wide crest at 5.05m AHD, and a rear stairway reaching to 5.95m AHD.

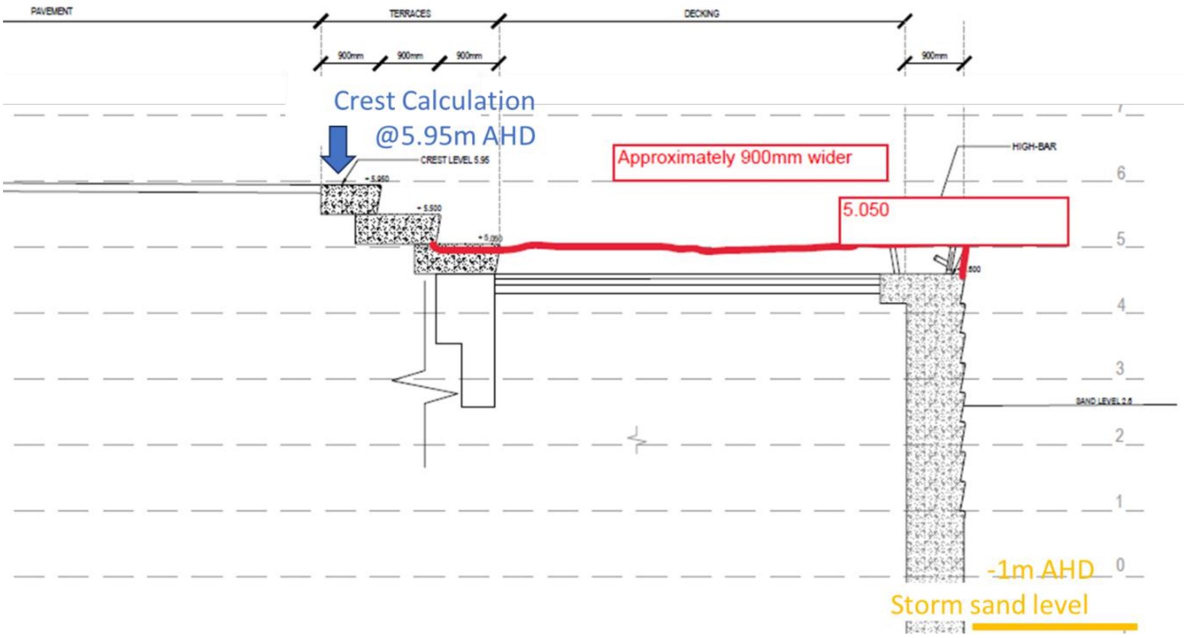

Figure 5-6 Section 3 design and overtopping calculation points

Predicted mean overtopping rates for all events are shown in Table 5-7. For the future 2074 1% AEP event:

- The overtopping rate is 8 l/m/s at the crest. This is within the tolerable limits of SES staff, but not pedestrians.
- Wave estimation further landward has used the EurOtop rule of thumb. This suggests the hazardous nature of overtopping is reduced to under 1 l/s/m a further 8m landward of the 5.95m contour. This is the adopted safe limit for aware pedestrians.

Table 5-7: Section 3: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Section 3 (crest at 5.95m AHD)					
Present day	0.0	0.7	1.0	1.2	1.5
2043	0.1	1.6	2.2	2.8	3.5
2173	0.3	5.0	6.7	8.3	9.7

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Checker:	AM	28/06/23					
Approver:	DR	04/07/23					
Office:	BRISBANE						

Project Title:	Mooloolaba foreshore
Subject:	Overtopping Assessment - Section 3

Input Data	
X	Y
-2	6.0
0	6.0
1.8	5.1
6.3	5.1
6.3	-1.0
6.3	-1.0
6.3	-0.5

Schematisation Inputs in Green - Automated Functions in Blue.		
TOE Data	X	Y
Front_of_toe_level_(mAOD)	6.3	-1.0
Toe_level_(mAOD)	6.3	-1.0
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	X	Y
Slope_downward_of_Berm_(cotangent), cot α_d	0.00	#DIV/0!
Slope_upward_of_Berm_(cotangent), cot α_u	2.00	26.57
Berm_level_(mAOD)	5.05	
BERM Calcs	X	Y
Start	6.3	5.1
End	1.8	5.1
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	4.54	
Berm level check	2.2265	5.2195
CREST Data	X	Y
Armour_crest_level_(mAOD)	0.0	6.0
Crest_level_(mAOD)	0.0	6.0
Width_of_Armour_crest_(m), Gc	0.00	
ROUGHNESS Data	X	Y
Upper Break in Roughness 1	6.0	
Lower Break in Roughness 2	5.1	
Roughness 1 - Vertical wall	1.0	
Roughness 2 - Steps	0.8	
Roughness 3 - Rear area - imperveous	1.0	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-1.00
Toe_level_(mAOD)	-1.00
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	0.00
Berm_level_(mAOD)	5.05
Width_of_Berm_(m), B	4.54
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00
Armour_crest_level_(mAOD)	5.95
Width_of_Armour_crest_(m), Gc	0.00
Crest_level_(mAOD)	5.95
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.95
Break in Roughness 2	5.05
Roughness 1	1.00
Roughness 2	0.80
Roughness 3	1.00

Entered
Calculated
Review Notes

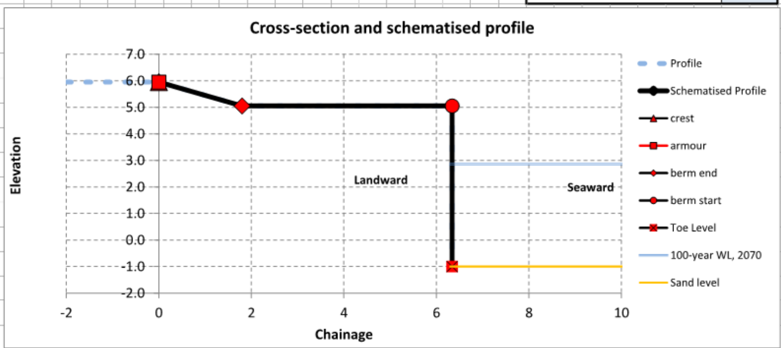

Figure 5-7 Section 3: Example Neural Network inputs and schematisation

5.7 Section 4 overtopping results in 2074 (during/post 90% detailed design)

Section 4 has been subject to a range of scenarios to inform the PDG design. These are shown in Figure 5-8 and include:

- Scenario 1: A seawall with a low crest (4.4m AHD), with two calculation points at the rear steps (top step at 5.05m AHD) and a landward position at 5.95m AHD.
- Scenario 2: A seawall with a low crest (4.4m AHD), with the rear steps raised by one step to 5.5m AHD, and a landward calculation point at 5.95m AHD.
- Scenario 3: A seawall with a low crest (4.4m AHD), with the rear steps raised by two steps to 5.95m AHD, and a landward calculation point at 6.0m AHD.

Figure 5-9 shows an example schematisation in the Neural Network. Whilst the seaward wall is a typical structure within the Neural Network, the rear terraces, stairs and green space is not considered a standard design. The entire structure has been schematised initially as a vertical wall with an armour crest at 4.4m AHD, with the model then extending to include the ramp and upper stairs (spanning 7.2m wide) with a crest of either 5.05m, 5.5m or 5.95m AHD, which is the first calculation point. Further calculations (modelling and analytical adjustments) were undertaken to extent the crest width wider and higher to represent landward areas that approach the 5.95m AHD contour.

Overtopping rates have been calculated at two locations for the three scenarios. Predicted mean overtopping rates for all events are shown in Table 5-8. For the future 2074 1% AEP event:

- For Scenario 1, with the top step at 5.05m AHD, the overtopping rate is 41 l/s/m. This is beyond the tolerable limits of SES staff, and at approximately the threshold for damage to grassed areas and pavers/promenades. This reduces to around 10 l/s/m at the landward calculation point at 5.95m, which is the safe limit for SES staff.
- For Scenario 2, with the top step at 5.5m AHD, the overtopping rate is 23 l/s/m. This is beyond the tolerable limits of SES staff. This reduces to 6 l/s/m at the landward calculation point at 5.95m, which is within the safe limit for SES staff but not pedestrians.
- For Scenario 2, with the top step at 5.95m AHD, the overtopping rate is 13 l/s/m. This is beyond the tolerable limits of SES staff. This reduces to 3 l/s/m at the landward calculation point at 5.95m, which is within the safe limit for SES staff but not pedestrians.

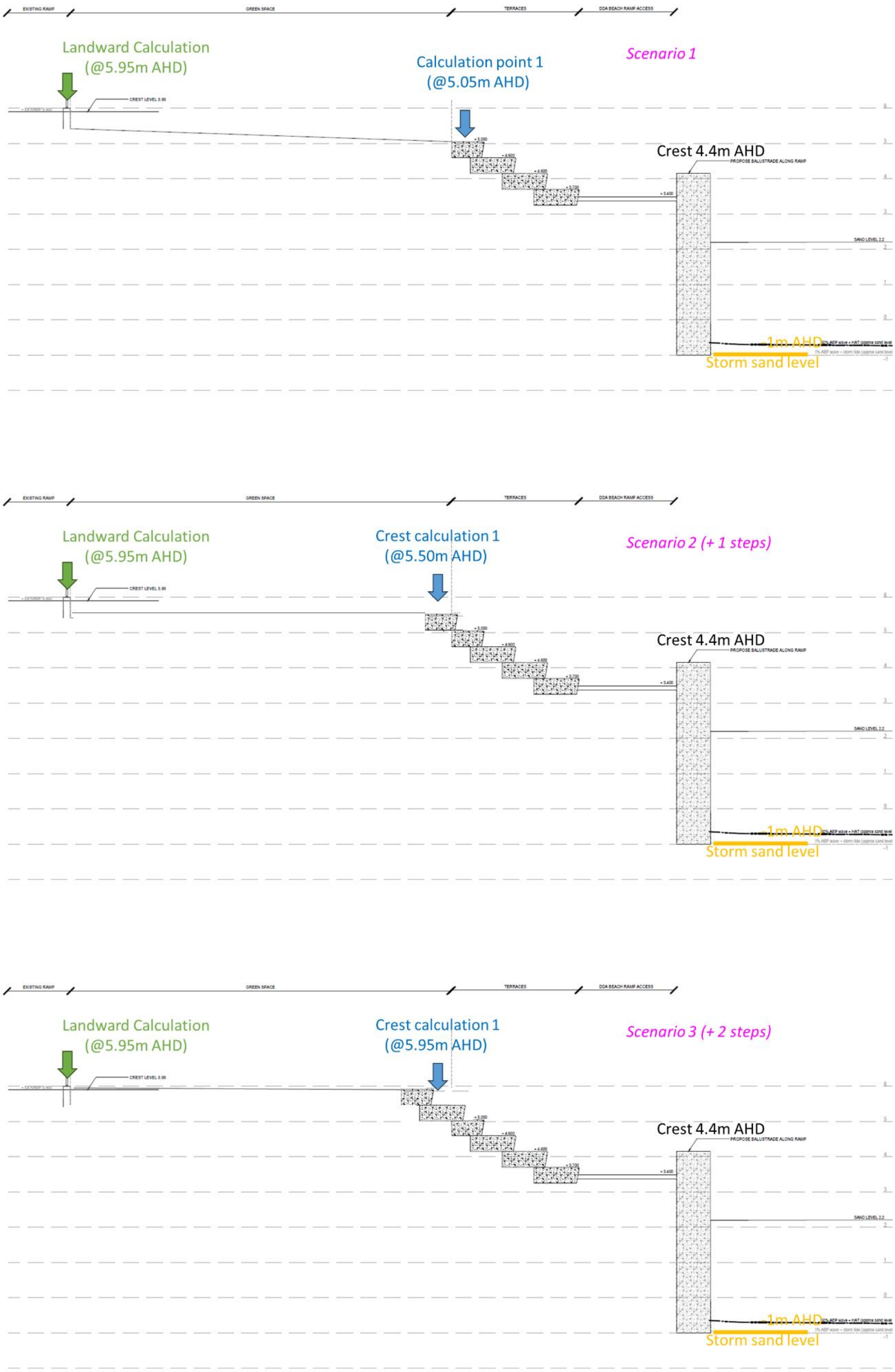

Figure 5-8   Section 4 design (three scenarios) and overtopping calculation points

Table 5-8: Section 4: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Step calculation point					
Section 4 (step at 5.05m AHD)					
Present day	0.1	4.8	6.8	8.2	10.5
2043	0.5	10.8	14.5	17.3	20.7
2173	2.3	27.5	34.7	41.4	46.8
Section 4 (step at 5.5m AHD (+1 step)					
Present day	0.1	2.4	3.5	4.2	5.5
2043	0.3	5.7	7.8	9.3	11.4
2173	1.2	15.3	19.5	23.4	26.5
Section 4 (crest at 6.0m AHD (+2 step)					
Present day	0.0	1.3	1.8	2.2	2.8
2043	0.2	2.9	4.0	4.9	6.1
2173	0.6	8.3	10.8	13.0	14.8
Landward calculation point					
Section 4 (step at 5.05, calculated landward at 6.0m AHD)					
Present day	0.0	0.7	1.0	1.2	1.7
2043	0.1	1.8	2.6	3.2	4.2
2173	0.3	6.1	8.3	10.3	12.0
Section 4 (crest at 6.0m AHD (+2 step) calculated landward at 6.0m AHD					
Present day	0.0	1.3	1.8	2.2	2.8
2043	0.2	2.9	4.0	4.9	6.1
2173	0.6	8.3	10.8	13.0	14.8
Section 4 (step at 6.0m AHD (+2 step) calculated landward at 6.0m AHD					
Present day	0.0	0.2	0.3	0.3	0.5
2043	0.0	0.6	0.8	1.0	1.2
2173	0.2	2.1	2.8	3.3	3.8

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	Initials	*Date*
Designer:	CY	28/06/23
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Office:	BRISBANE	

Input Data	
X	Y
-2	5.1
2	3.7
7.1	3.7
7.1	4.6
7.3	4.6
7.3	-1.0
7.2	-0.5

Schematisation Inputs in Green - Automated Functions in Blue.		
TOE Data	***X***	***Y***
Front_of_toe_level_(mAOD)	7.2	-1.0
Toe_level_(mAOD)	7.2	0.0
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	***X***	***Y***
Slope_downward_of_Berm_(cotangent), cot α_d	0.00	#DIV/0!
Slope_upward_of_Berm_(cotangent), cot α_u	0.00	#DIV/0!
Berm_level_(mAOD)	2.86	
BERM Calcs	***X***	***Y***
Start	7.2	2.9
End	7.2	2.9
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	0.00	
Berm level check	3.05	6.04
CREST Data	***X***	***Y***
Armour_crest_level_(mAOD)	7.2	4.4
Crest_level_(mAOD)	0.0	5.1
Width_of_Armour_crest_(m), Gc	7.20	
ROUGHNESS Data	***X***	***Y***
Upper Break in Roughness 1	5.1	
Lower Break in Roughness 2	3.4	
Roughness 1 - Concrete Revetment	1.0	
Roughness 2 - Concrete Revetment	0.8	
Roughness 3 - Concrete Revetment	1.0	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-1.00
Toe_level_(mAOD)	0.00
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	0.00
Berm_level_(mAOD)	2.86
Width_of_Berm_(m), B	0.00
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	0.00
Armour_crest_level_(mAOD)	4.40
Width_of_Armour_crest_(m), Gc	7.20
Crest_level_(mAOD)	5.05
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.05
Break in Roughness 2	3.40
Roughness 1	1.00
Roughness 2	0.80
Roughness 3	1.00

Entered
Calculated
Review Notes

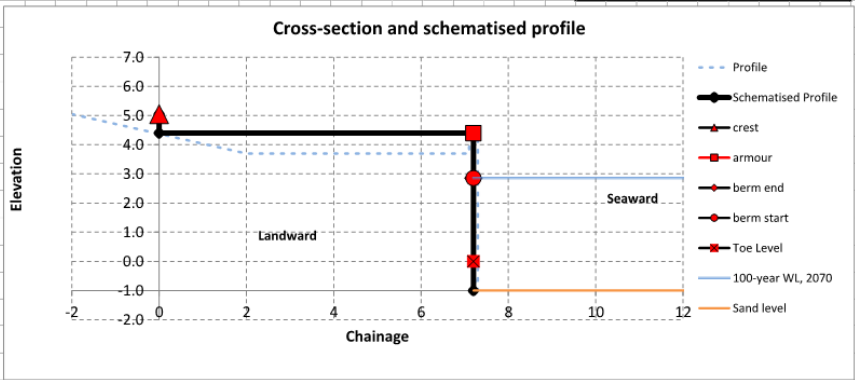

Figure 5-9 Section 4: Example Neural Network inputs and schematisation

5.8 Section 5 overtopping results in 2074 (during/post 90% detailed design)

The Section 5 defence design and overtopping calculation points are shown in Figure 5-12. Wave overtopping rates have been estimated at two locations - the crest at 5.5m AHD and the landward crest at 6.0m AHD.

Figure 5-13 shows the schematisation in the Neural Network. The structure has been schematised as a sloping stepped revetment with a crest level at 5.5m AHD. A second model was developed to represent the rear areas, with a 4.6m wide armour crest used to represent the turfed area that rises from the top step to 6.0m AHD.

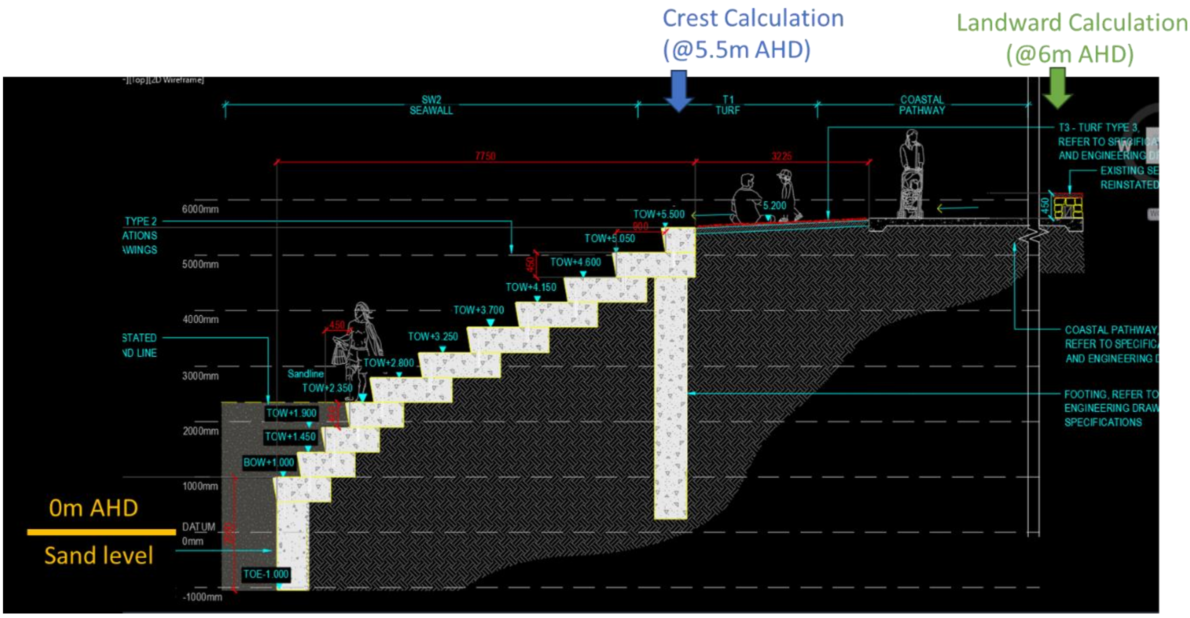

Figure 5-10 Section 5: design and overtopping calculation points

Predicted mean overtopping rates for all events are shown in Table 5-10. For the future 2074 1% AEP event:

- The overtopping rate is 19 l/m/s at the seaward crest. This is beyond the tolerable limits for SES staff.
- The overtopping rate behind the structure at 6.0m AHD reduces to 5 l/s/m. This is within the tolerable limits of SES staff, however not pedestrians.

Given these relatively high overtopping rates, it is recommended to review the seawall crest and associated path levels when the SLSC site is redeveloped in the future. If the seawall design is not altered, under a 2073 planning horizon the section of foreshore in front of the SLSC is recommended to be closed prior to a Q20 storm to limit access by the public and emergency services.

Table 5-9: Section 5: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Section 5 (at crest 5.5mAHD)					
Present day	0.0	1.0	1.4	1.9	2.5
2043	0.1	2.6	4.0	5.2	6.9
2173	0.4	10.5	14.8	19.1	22.6
Section 5 (landward at 6m AHD)					
Present day	0.0	0.3	0.4	0.5	0.7
2043	0.0	0.7	1.0	1.4	1.7
2173	0.1	2.7	4.0	5.3	6.4

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Subject:	Overtopping Assessment - Section 5				

	Initials	Date
Designer:	CY	28/06/23
Checker:	AM	28/06/23
Approver:	DR	04/07/23
Office:	BRISBANE	

Input Data	
X	Y
-2	5.5
0	5.5
0.0	5.1
5.4	2.8
9.2	0.0
9.2	-0.5

Schematisation Inputs in Green - Automated Functions in Blue.		
TOE Data	*X*	*Y*
Front_of_toe_level_(mAOD)	9.2	-1.0
Toe_level_(mAOD)	9.2	-1.0
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	*X*	*Y*
Slope_downward_of_Berm_(cotangent), cot α_d	1.00	45.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00	26.57
Berm_level_(mAOD)	2.80	
BERM Calcs	*X*	*Y*
Start	5.4	2.8
End	5.4	2.8
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	0.00	
Berm level check	3.0515	6.0445
CREST Data	*X*	*Y*
Armour_crest_level_(mAOD)	0.0	5.5
Crest_level_(mAOD)	0.0	5.5
Width_of_Armour_crest_(m), Gc	0.00	
ROUGHNESS Data		
Upper Break in Roughness 1	5.5	
Lower Break in Roughness 2	5.5	
Roughness 1 - Impermeable	1.0	
Roughness 2 - Stepped Revetment	0.8	
Roughness 3 - Stepped Revetment	0.8	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-1.00
Toe_level_(mAOD)	-1.00
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	1.00
Berm_level_(mAOD)	2.80
Width_of_Berm_(m), B	0.00
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00
Armour_crest_level_(mAOD)	5.50
Width_of_Armour_crest_(m), Gc	0.00
Crest_level_(mAOD)	5.50
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.50
Break in Roughness 2	5.50
Roughness 1	1.00
Roughness 2	0.80
Roughness 3	0.80

Entered
Calculated
Review Notes

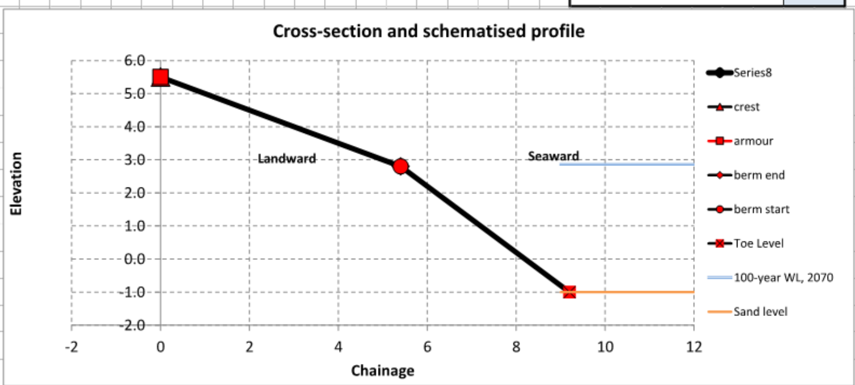

Figure 5-11 Section 5: Example Neural Network inputs and schematisation

5.9 Section 6 overtopping results in 2074 (during/post 90% detailed design)

A new cross section was analysed to the north of Section 5 (north of the SLSC). The defence design and overtopping calculation point is shown in Figure 5-12. Wave overtopping estimates have been calculated at the crest.

Figure 5-13 shows the schematisation in the Neural Network. The structure has been schematised as a sloping stepped revetment with a crest level at 5.05m AHD. A second model was developed to represent the rear areas, with a 4.6m wide armour crest used to represent the turfed area that rises from the top step to 5.4m AHD. These design levels are considered low in relation to the future sea level, however are limited by the existing footpath to the southern end of SLSC.

The modelling does not include a wall positioned to the rear of the coastal path, which is adjacent to the SLSC. This wall will provide some protection against any overtopped water passing the pathway, although it is not a continuous barrier and has not been designed to withstand extreme coastal forces.

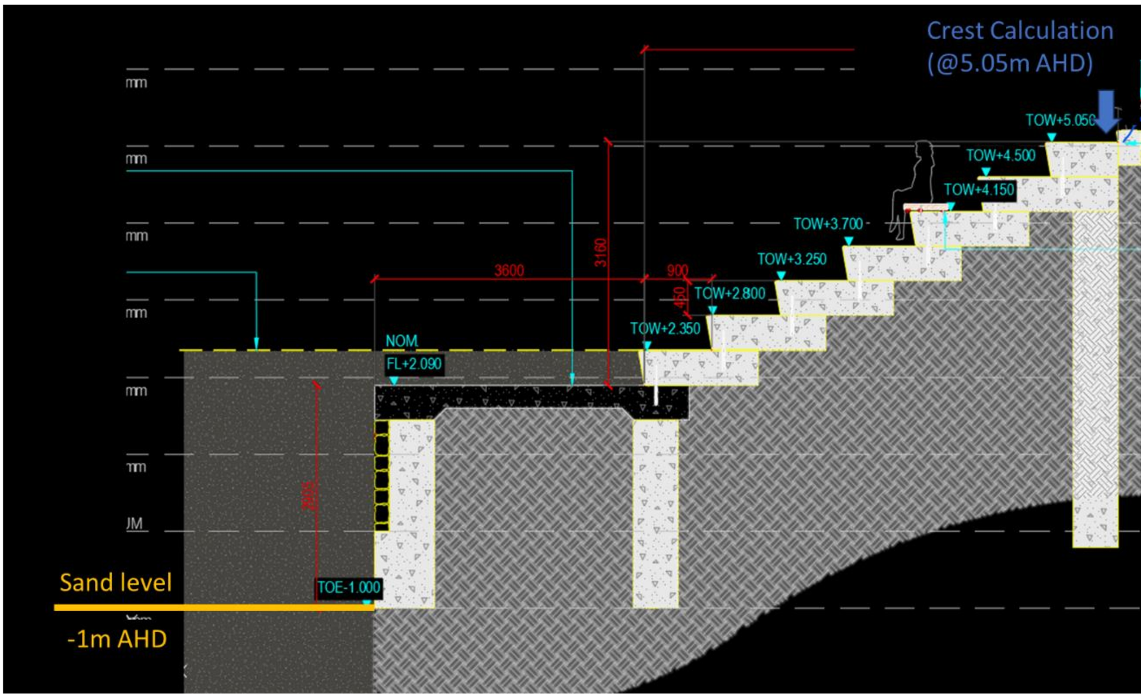

Figure 5-12 Section 6: design and overtopping calculation points

Predicted mean overtopping rates for all events are shown in Table 5-10. For the future 2074 1% AEP event:

- The overtopping rate is 16 l/m/s at the crest. This is beyond the tolerable limits for SES staff.

Table 5-10: Section 6: Results of overtopping modelling (l/s/m)

Year	MHWS	Q20	Q50	Q100	Q500
Section 6 (at crest 5.05mAHD)					
Present day	0.0	1.1	1.6	2.1	2.8
2043	0.1	2.9	4.2	5.2	6.7
2173	0.5	9.6	12.9	16.1	18.8

CALCULATION RECORD

Project Code:	2023s0366	**Page**	1	**of**	24	
Project Title:	Mooloolaba foreshore					
Subject:	Overtopping Assessment - Section L					

	Initials	*Date*
Designer:	CY	28/06/23
Checker:	AM	28/06/23
Approver:	DR	04/07/23
Office:	BRISBANE	

Input Data	
X	Y
-2	5.1
0	5.1
0.0	5.1
9.9	1.9
9.9	0.0
9.9	-0.5

Schematisation Inputs in Green - Automated Functions in Blue.		
TOE Data	***X***	***Y***
Front_of_toe_level_(mAOD)	9.9	-1.0
Toe_level_(mAOD)	9.9	-1.0
Width_of_toe_(m), B_t	0.00	
BERM Slope Data (Angle Check)	***X***	***Y***
Slope_downward_of_Berm_(cotangent), cot α_d	0.00	#DIV/0!
Slope_upward_of_Berm_(cotangent), cot α_u	2.00	26.57
Berm_level_(mAOD)	1.90	
BERM Calcs	***X***	***Y***
Start	9.9	1.9
End	6.3	1.9
Slope_of_Berm_(tangent), tan aB	0.00	
Width_of_Berm_(m), B	3.60	
Berm level check	3.0515	6.0445
CREST Data	***X***	***Y***
Armour_crest_level_(mAOD)	0.0	5.1
Crest_level_(mAOD)	0.0	5.1
Width_of_Armour_crest_(m), Gc	0.00	
ROUGHNESS Data		
Upper Break in Roughness 1	5.1	
Lower Break in Roughness 2	2.1	
Roughness 1 - Concrete	1.0	
Roughness 2 - Stepped Revetment	0.8	
Roughness 3 - Concrete	1.0	
OTHER Data		
Wave return wall (If at crest then; yes=1, No=0)	0.0	
Normal_angle_of_defence_(degrees from North)	0.0	

Neural Network Inputs	Value
Front_of_toe_level_(mAOD)	-1.00
Toe_level_(mAOD)	-1.00
Width_of_toe_(m), B_t	0.00
Slope_downward_of_Berm_(cotangent), cot α_d	0.00
Berm_level_(mAOD)	1.90
Width_of_Berm_(m), B	3.60
Slope_of_Berm_(tangent), tan α_B	0.00
Slope_upward_of_Berm_(cotangent), cot α_u	2.00
Armour_crest_level_(mAOD)	5.05
Width_of_Armour_crest_(m), Gc	0.00
Crest_level_(mAOD)	5.05
Normal_angle_of_defence_(degrees from North)	0.00
Wave return wall (yes=1, No=0)	0.00
Break in Roughness 1	5.05
Break in Roughness 2	2.09
Roughness 1	1.00
Roughness 2	0.80
Roughness 3	1.00

Entered
Calculated
Review Notes

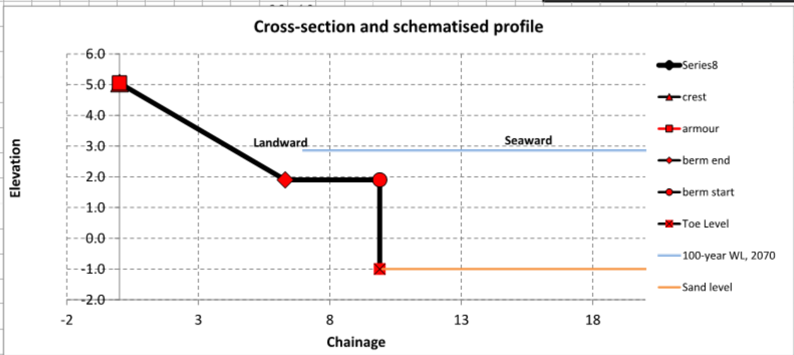

Figure 5-13 Section 6: Example Neural Network inputs and schematisation

5.10 Overtopping summary during/post 90% detailed design

Mean overtopping rates has been estimated for six proposed design profiles, for 5 design conditions for waves. Results are summarised in Table 5-11. Several sections have calculated overtopping at multiple locations, typically at the crest and a position further landward. The modelling includes three time horizons; present day, 2043 (within 10 years of construction) and 2073 (end of useful working life. All storms consider extreme waves and sea levels coinciding with a post-storm eroded beach level of -1m AHD. The mean overtopping rate can be compared against guidance thresholds from the EurOtop (v1) manual, with the following criteria used to describe the overtopping:

- Mean overtopping rates over 200 l/s/m: Damage to well protected embankment/seawalls
- Mean overtopping rates over 50 l/s/m: Damage to grassed areas and pavers/promenades
- Mean overtopping rates over 10 l/s/m: Unsafe for trained staff (e.g. SES)
- Mean overtopping rates over 1 l/s/m: Unsafe for pedestrians, but accessible for trained staff

Table 5-11: All results of overtopping modelling (l/s/m), 2073 planning horizon

Cross section / design iteration	MHWS	Q20	Q50	Q100	Q500
Section 1 (crest at 5.5m AHD)	0.3	7.9	11.2	14.4	17.0
Section 1 (landward, 5.95m AHD)	0.1	1.8	2.6	3.4	4.0
Section 2 (crest at 6.4m AHD)	0.0	2.0	3.0	3.9	4.8
Section 2 (mid-crest at 5.5m AHD)	0.1	2.5	3.7	5.0	6.2
Section 3 (crest at 5.95m AHD)	0.3	5.0	6.7	8.3	9.7
Section 4 (step at 5.05m AHD)	2.3	27.5	34.7	41.4	46.8
Section 4 (step at 5.05, calculated landward at 6.0m AHD)	0.3	6.1	8.3	10.3	12.0
Section 4 (step at 5.5m AHD (+1 step)	1.2	15.3	19.5	23.4	26.5
Section 4 (step at 5.5m AHD (+1 step) calculated landward at 6.0m AHD	0.3	3.9	5.0	6.0	6.8
Section 4 (crest at 6.0m AHD (+2 step)	0.6	8.3	10.8	13.0	14.8
Section 4 (step at 6.0m AHD (+2 step) calculated landward at 6.0m AHD	0.2	2.1	2.8	3.3	3.8
Section 5 (at crest 5.5mAHD)	0.4	10.5	14.8	19.1	22.6
Section 5 (landward at 6m AHD)	0.1	2.7	4.0	5.3	6.4
Section 6 (crest at 5.05m AHD)	0.5	9.6	12.9	16.1	18.8

6 Wave action study

6.1 Timing and available data

The wave action study was completed at the 90% detailed design stage, and used available designed from the previous stage (60% detailed design).

The data available during the wave action study include the following:

- Four proposed seawall designs have been provided to JBP based on the 60% detailed design stage (cross sections are shown in Figure 6-4)
- The scour depth has used a mid-storm erosion profile, scoured down to 0.0m AHD.
- Tidal plane information has been derived from the Queensland Tidal Planes (MSQ, 2023[21]) at Mooloolaba (PSM 37055). Modelling under typical conditions has used a Mean High Water Spring (MHWS).
- Storm tide levels have been sourced from the Sunshine Coast Storm Tide Study (Aurecon 2013)[22] and are not inclusive of wave setup, which is implicitly included within the assessment methods. An end-of-design timeframe has been interpolated for a 2074 planning horizon.
- Extreme nearshore waves are based on new coastal modelling (See Section 3). Nearshore waves at the toe of the proposed structure have been calculated using the mid-storm eroded beach level (0m AHD) by applying a depth limitation based on the peak water levels. This has used a height/depth relationship of 0.55 following Nelson (1994)[23,24]. This has been developed based on a range of literature (both laboratory and field measurements) that supports a wave height to water depth ratio of 0.55 to represent stable, shallow water oscillatory waves propagating in water of constant depth (i.e a horizontal bed representative of a flat eroded profile).

6.2 Methodology

6.2.1 Typical actions in seawall design

Wave actions can be a governing factor in stability design for a seawall. Figure 6-1 shows all the typical actions to consider in a seawall design. Wave impact (both landward or seaward horizontal force to face of wall) and uplift forces have been investigated for the Mooloolaba seawall, which should be considered as additional loads to other hydraulic and hydrostatic loads, live and superimposed surcharges etc. Note: whist shown in the figure, mooring and berthing actions are not applicable to the proposed design.

21 Available from: https://www.msq.qld.gov.au/tides/tidal-planes

22 Aurecon (2014) Sunshine Coast Storm Tide Study. Prepared for Sunshine Coast Council

23 Nelson, R.C. (1994a), Depth limited design wave heights in very flat regions, Coast. Eng., 23, 43-59

24 Nelson, R. (1997). Height limits in top down and bottom up wave environments. Coastal Engineering, 32(2-3), 247-254.

Figure 5.1 Typical actions to consider in quay and sea wall design

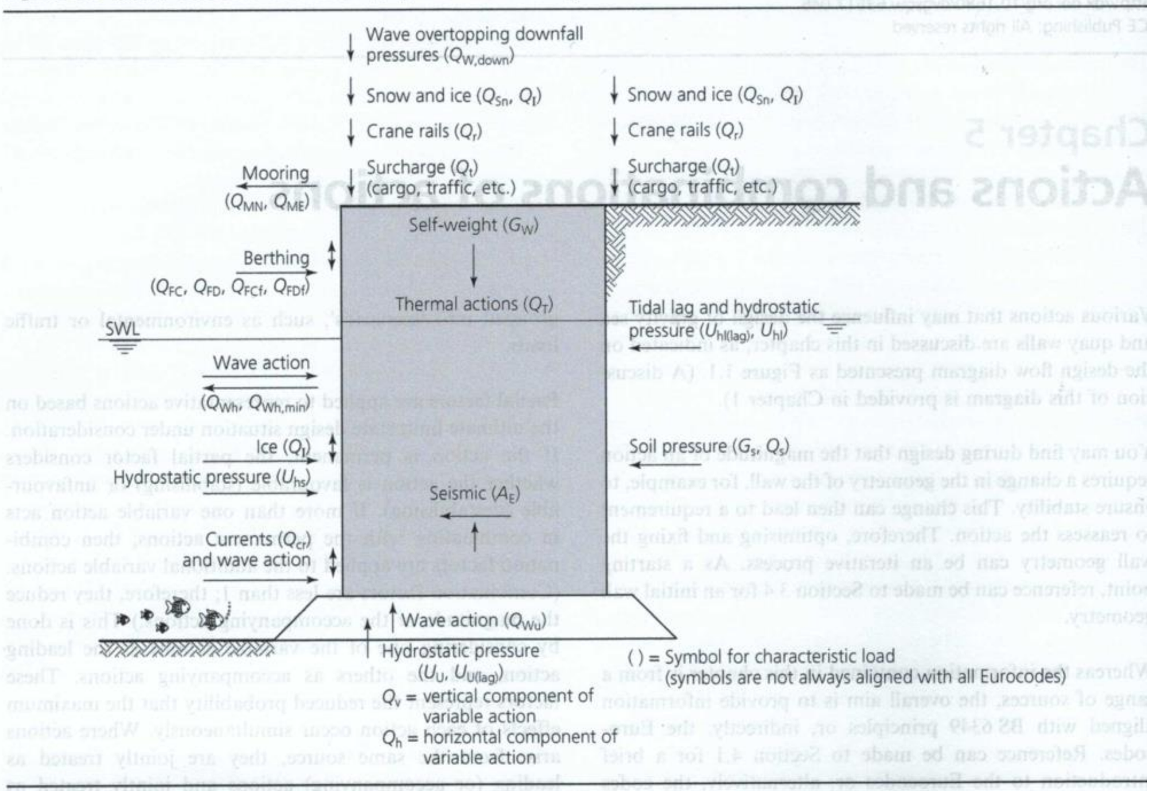

Figure 6-1: Typical actions to consider in quay and sea wall design (Design of Vertical Gravity Sea and Quay Walls, ICE, 2020)

6.2.2 Vertical wall

Vertical seawalls can experience both pulsating and impacting wave actions, each requiring different calculation approaches. PROVERBS (European Commission, 1999) is a parameterised tool capable of predicting the effects of wave actions on vertical seawalls, as shown in Figure 6-2. This process includes a range of empirical methods to determine actions required for wall design for each wave condition. It is possible that the same structure experiences different wave actions due to when or how waves break against the wall in different design storms, due to different combinations of water level and wave heights. Consequently the largest storm may not necessarily result in the greatest impact loads. Also due to limitations in understanding of wave loading, loading may be provided in different formats, such as pressure, point force, or moment depending on the setup of the empirical equations.

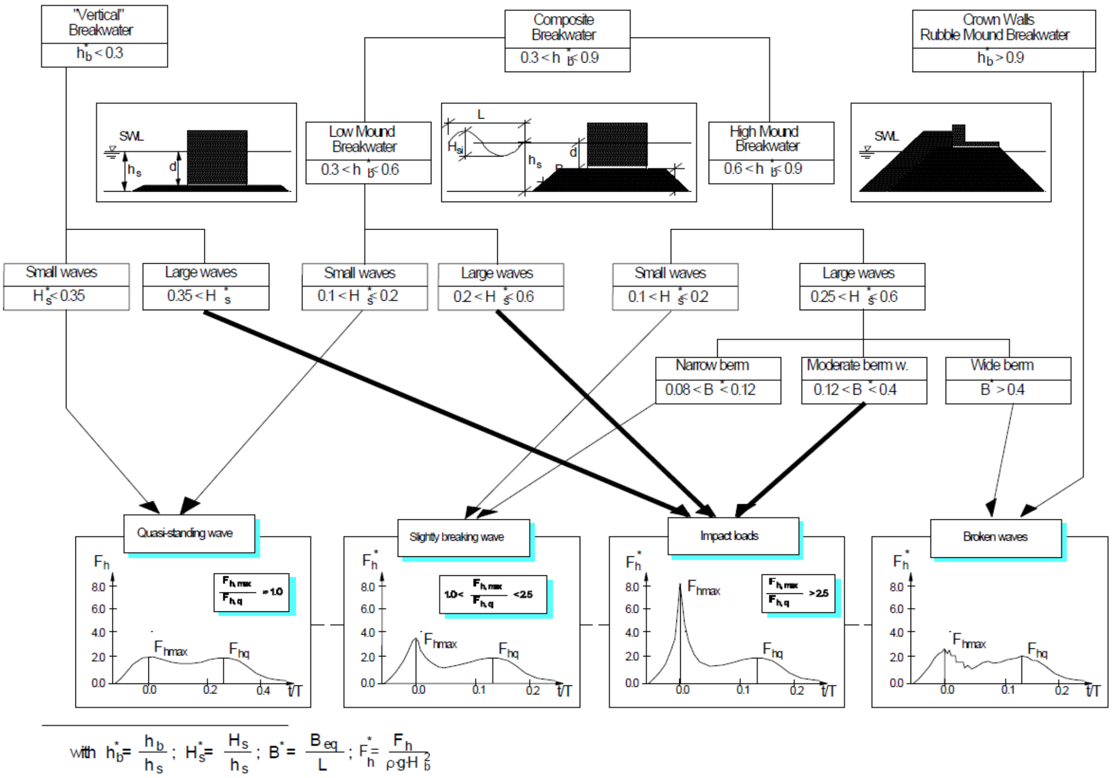

Figure 6-2: PROVERBS configuration

Assessment methods have followed the Institute of Civil Engineers (ICE) publication 'Design of Vertical Gravity Sea and Quay Walls' (Ackhurst 2020). The most critical assessment has considered seaward actions (i.e. suction on wall face due to reflected waves) which can trigger overturning and sliding failure modes. Whilst still important, shoreward actions are typically needed for joint designs or to confirm wall stability during construction (i.e. before backfilling). Other hydraulic actions such as pore water pressure, live and surcharge loads should be considered on top of the wave actions.

The Sainflou and Goda methods (referenced in Table 6-1) can be used to consider pressure diagrams along the vertical face and foundation width of the proposed seawall to identify a theoretical lever arm to transform forces to momentum. However, at the time of study, foundation widths and points of rotation for sections were not available, and so it is recommended further calculations by a structural engineer should test the force being applied at critical points, such as at the water level.

Table 6-1: Empirical methods to determine wave responses

Wave condition	Shoreward wave action	Seaward wave action	Uplift
Pulsating/non-breaking/reflecting wave	Goda (2000)[25]	Sainflou modified by McConnell et al. (1999)[26]	Goda (2000)
Impulsive/plunging/breaking/impact	Allsop and Vicinanza modified by McConnell (2003)[27]		
Broken wave	Blackmore and Hewson (1984)[28]		

25 Goda, Y., & Takagi, H. (2000). A reliability design method of caisson breakwaters with optimal wave heights. Coastal Engineering Journal, 42(4), 357-387.

26 McConnell, K. J., Allsop, N. W. H., & Flohr, H. (1999). Seaward wave loading on vertical coastal structures. In Proceedings of the International Conference on Coastal Structure'99 (Vol. 1).

27 Cuomo, G., Allsop, W., & McConnell, K. (2003). Dynamic wave loads on coastal structures: Analysis of impulsive and pulsating wave loads. In Coastal structures 2003 (pp. 356-368)

28 Blackmore, P.A.& Heewson, P.J. (1984) Experiments on full-scale wave impact pressures. Coastal Engineering 89, 331-346

6.2.3 Terraced/ stepped seawall

The assessment of wave loads has used a schematised design sections to allow the use of the PROVERBS model for both the terraced and vertical seawall sections.

Note: Identification of wave loads on a terraced revetment lacks the research base of standard seawalls, and therefore has more uncertainty. However, it is recognised that wave loads are not as critical as for vertical seawalls due to the more stable geometry. It is a practical approach to compare the wave forces against the concrete strength being used for the terraced sections as wave forces would have much less impact to overall stability than for a vertical seawall. It is assumed the steps are cast homogenously and fixed into the ground and the standard strength of the concrete (N40) is sufficient to resist the wave pressure/force.

6.3 Study scenarios

The assessment uses the PROVERBS model to assess wave loads from design storm events.

Coastal inputs included extreme sea levels and depth-limited waves for a 2074 planning horizon. The designs are based on the 60% detailed design cross sections shown in Figure 6-4.

The results are shown in Table 6-3 to Table 6-5. This indicates that for a 100-year ARI storm event in 2073, the seaward wave action can range from 19 to 114 kN, shoreward wave action between 52 to 280kN, and uplift action between 9 to 94 kN.

These results are from empirical formulae, however are similar to an article published by Water Research Laboratory NSW on physical testing of wave forces on stepped seawalls[29] which have a similar profile as Section 1 and 2. The WRL physical tests indicate wave forces were approximately 300kN/m.

Note that the lever arm from toe was assumed as dimensions of the wall sections were not provided in full at the time of the assessment. It is recommended where lever arm was not mentioned, the force can be applied at the worst location possible by the structure engineers.

Table 6-2: Scenarios for wave force study at year 2074

ARI, 1 in x years	20 years	50 years	100 years	500 years	1000 years
STL (mAHD)	2.73	3.03	3.27	3.46	3.54
Nearshore significant wave (m)*	2.05	2.22	2.35	2.45	2.50
Peak periods (s)**	14.5	15.0	15.1	16.0	16.0
*: Depth limited with a future beach level at -1mAHD. **: JBP wave modelling results.					

29 B Modra, I Coghlan, J Carley, G Blumberg, W Boyd. (2016). Wave Forces and Overtopping on Stepped Seawalls. Retrieved from: https://www.wrl.unsw.edu.au/sites/wrl/files/uploads/PDF/Ben%20Modra%20Full%20Paper%202016.pdf

Figure 5.2 Goda's method: Wave pressure distribution diagram with parameters

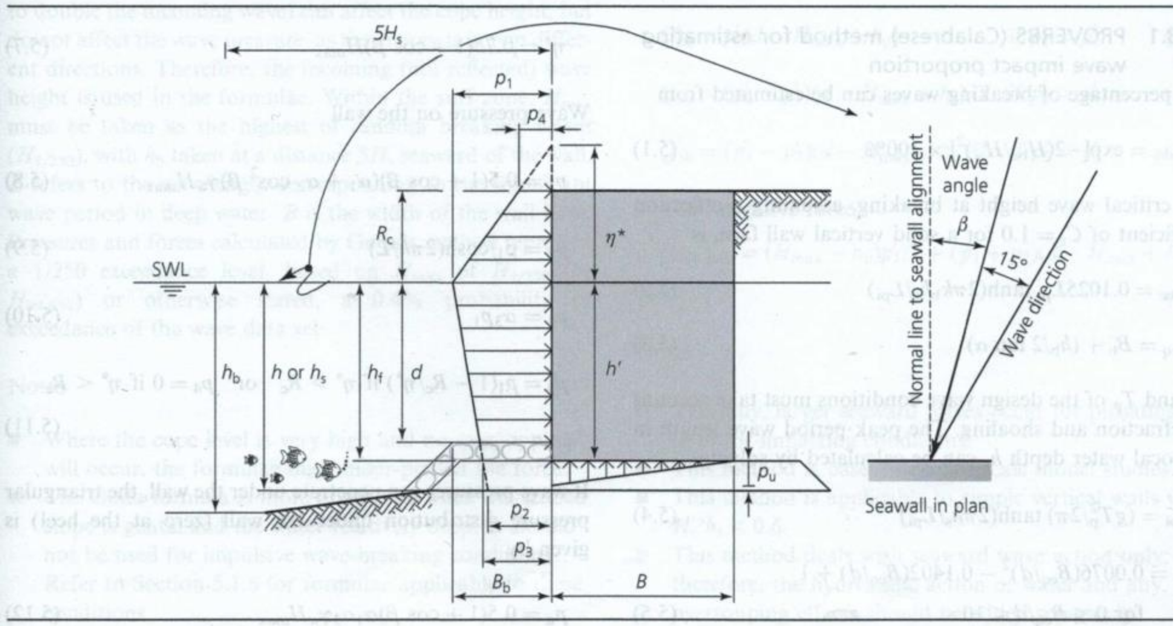

Figure 6-3: Goda's method: Wave pressure distribution diagram with parameters (Design of Vertical Gravity Sea and Quay Walls, ICE, 2020)

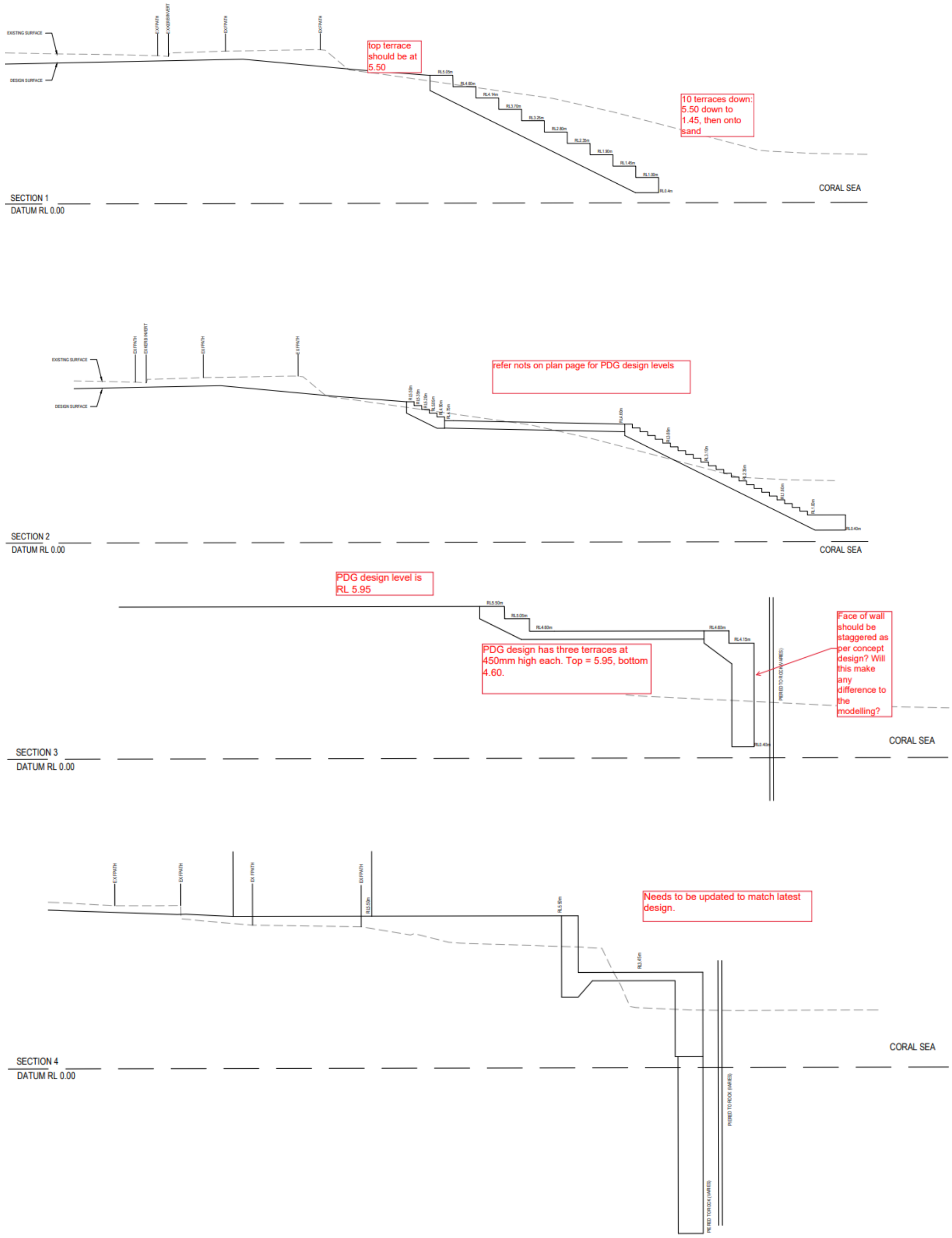

Figure 6-4: Assessed sections, provided by PDG and Barlow Shelley (60% DD).

Table 6-3: Seaward wave action* (kN) for 2073

Section/ ARI	20 yrs	50 yrs	100 yrs	500yrs	1000yrs
1	43.73	50.65	57.37	63.40	66.10
2	8.79	13.85	18.68	23.03	24.81
3	86.69	101.26	113.58	124.08	128.53
4	82.68	96.14	107.34	116.67	120.54
* Sainflou's method modified by McConnell (1999)					

Table 6-4: Shoreward wave action for 2073

Section	Shoreward action	20 yrs	50 yrs	100 yrs	500yrs	1000yrs	Method
Section 1	Wave pressure (kPa)	231.19	260.12	279.56	312.03	318.03	Blackmore and Hewson (1984)
Section 2	Wave impact force (kN)	28.28	40.58	51.99	62.04	66.55	Allsop and Vicinanza modifiled by McConnel (2000)
Section 3	Wave impact force (kN)	174.15	203.5	228.45	249.23	258.24	
Section 4	Wave impact force (kN)	174.15	203.5	228.45	249.23	258.24	

Table 6-5: Uplift action* (kN) for 2073

Section/ ARI	20 yrs	50 yrs	100 yrs	500yrs	1000yrs
1	82.27	88.68	93.59	98.08	99.71
2	24.82	30.33	34.64	38.18	38.16
3	9.44	10.19	10.76	11.30	11.49
4	7.85	8.34	8.66	8.93	9.00
* Goda's method (2000)					

A Appendix A - wave modelling

A.1 Extreme offshore waves

Extreme value analysis (EVA) has been conducted for offshore wave data at the Brisbane Waverider Buoy (WRB). A peak over threshold (POT) method has been used to isolate significant wave events and a generalised pareto distribution (GPD) fit to estimate the probability of extreme conditions. Figure 3-1 shows the fitting of the GPD function to wave data and estimation of extreme wave heights at Brisbane WRB.

In the previous coastal processes study (BMT 2013)[30], a range of extreme offshore wave return periods were derived from the Brisbane WRB wave record. These conditions were applied to a numerical model and extracted at the -20m depth contour along the SCC. Since this study, an additional 10 years of wave data is available from the offshore buoy and the results of EVA on this updated dataset show that the conditions assessed in the previous study may underpredict extreme waves in the offshore, as shown in Table 3-1.

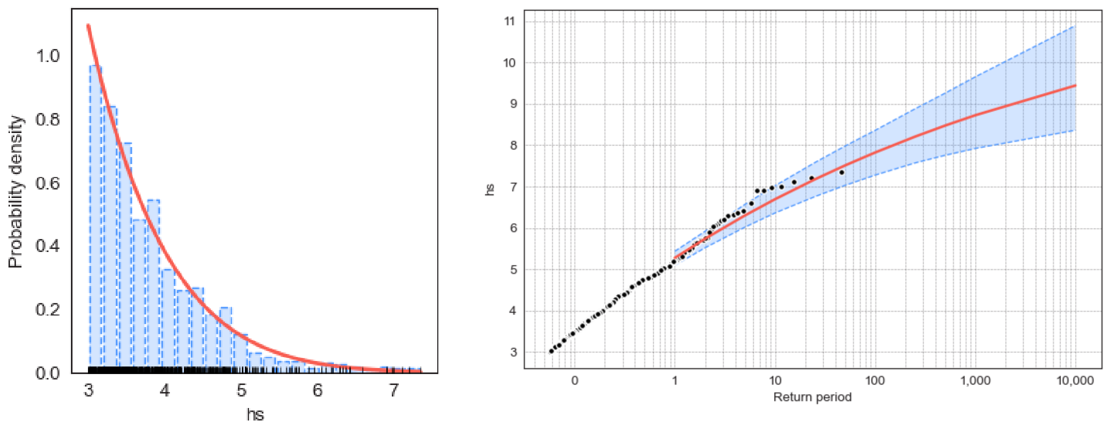

Figure 6-5 Left: GPD fit to wave heights above 3.0m and Right: GPD estimation of extreme wave height, for Brisbane WRB.

Table 6-6: Extreme wave height return periods for Brisbane WRB, compared with previous coastal assessment (BMT 2013)

ARI (yrs)	Hs (m) (JBP assessed)	Hs (m) (BMT (2013))	% increase
2	5.74	5.05	13.6%
5	6.30	5.85	7.7%
10	6.69	6.30	6.2%
20	7.10	6.70	6.0%
50	7.51	7.30	2.9%
100	7.82	7.80	0.3%

A.2 Extreme nearshore wave assessment

Whilst nearshore extreme wave estimates were evaluated in BMT (2013), their calculation approach followed a deterministic pathway where offshore extremes were simulated through a wave model and extracted at the -20m contour. This approach assumes that the wave height exceedances at the offshore WRB are the same as in the nearshore (i.e., a 1%AEP wave at the offshore location, when applied to the model, produces a 1%AEP wave at the nearshore location). This approach is conservative and does not consider the directionality of extreme conditions, which may be critical

30 BMT WBM (2013) Coastal Processes Study for the Sunshine Coast

when assessing extremes for locations sheltered within bays or behind headlands, for example Mooloolaba Beach.

Updated nearshore extreme wave conditions have been assessed for the Mooloolaba study site. A probabilistic approach has been used to establish a 10,000-year wave simulation, representing the full range of potential wave conditions in the nearshore. The following methodology has been used to derive these conditions, which is documented in Appendix A.

1. **Metocean data collation:** Historical offshore wave data is collated for the Brisbane WRB.
2. **Data declustering:** The historical data series is declustered into discrete events.
3. **Data simulation:** The declustered data is used to produce a large 10,000-year set of potential offshore conditions.
4. **Data sampling:** A subset of 200 representative events is sampled from the large dataset.
5. **Wave modelling:** The 200 representative events are applied as wave model boundary conditions, with results extracted in the nearshore at the study site
6. **Nearshore wave emulation:** An emulator is used to translate the remainder of the large set of wave conditions to the nearshore

A.3 Metocean data collation

Historic offshore wave data has been sourced from the Brisbane WRB, spanning from 1976 to 2022. Wave data from the Brisbane buoy before 1996 is non-directional. In order to capture the directional spread of wave conditions for the full Brisbane record, direction data from the ERA5 global wave hindcast model has been used to supplement the wave record prior to 1996. The hindcast model provides hourly estimates from 1979 to present day for a range of atmospheric conditions and has a spatial resolution of 0.5° for wave extraction. Figure 6-6 compares the ERA5 hindcast data with recorded wave conditions at Brisbane WRB

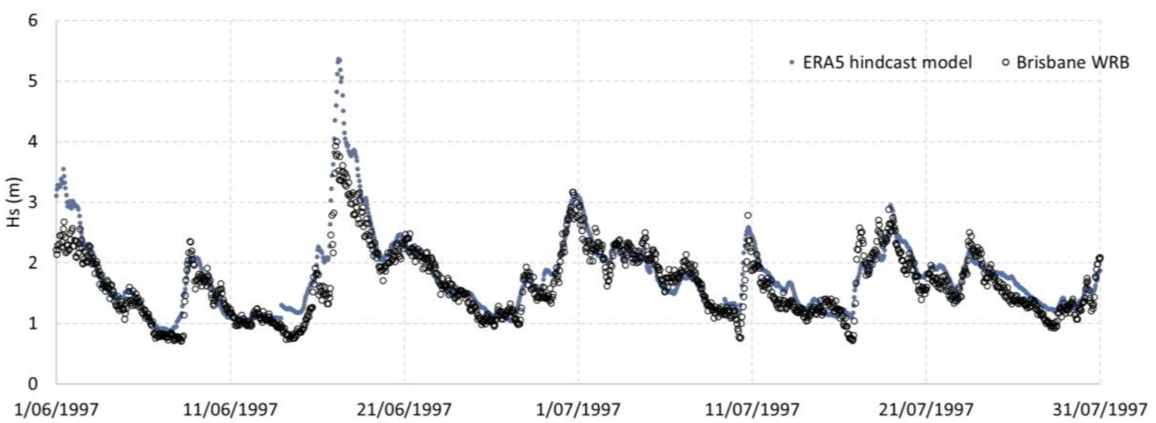

Figure 6-6: Sample comparison of wave data from Brisbane WRB and ERA5 hindcast model

A.4 Data declustering:

Peak analysis has been conducted on the Brisbane WRB data to isolate discrete weather events. For the purposes of this study, a discrete weather event is defined as a peak in the wave height (Hs) record with a minimum duration of 3 days, and minimum prominence of 0.3m (i.e. wave heights above 0.3m to their nearest neighbour in the record). From the 47-year recorded data series, approximately 3045 weather events have been discretised. Figure 6-7 shows an example of declustered events of peak wave height and corresponding peak wave period within the record.

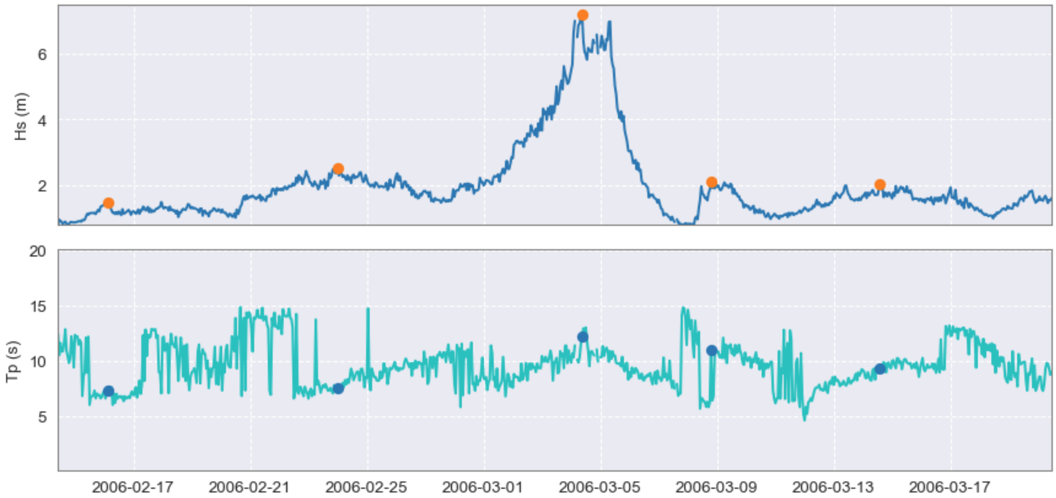

Figure 6-7: Declustering of discrete weather events in wave record, identified peak events as points showing significant wave height in meters (top) and peak wave period in seconds (bottom)

A.5 Data simulation

A probabilistic approach has been used to establish 10,000-years of potential offshore wave conditions. Conventionally, this would be accomplished by creating a set of conditions where all possible combinations for wave height, period and direction are favoured equally. However, a more robust method has been used which relies on multivariate analysis to simulate a full set of possible conditions, based on the recorded wave data. This method favours a more realistic distribution of wave conditions, as the characteristics of the historical data are used directly to simulate a much larger set of conditions.

First, the distribution of each of the declustered event parameters (Hs, Tp and Dir) is determined, as well as the correlation of each parameter to every other. Next, a Gaussian copula method is applied to the data. This method fits a univariate distribution to each parameter and creates a set of 10,000 years' worth of simulated conditions. Figure 6-8 shows a comparison of historical events to the larger simulated set for significant wave height, wave direction and wave period.

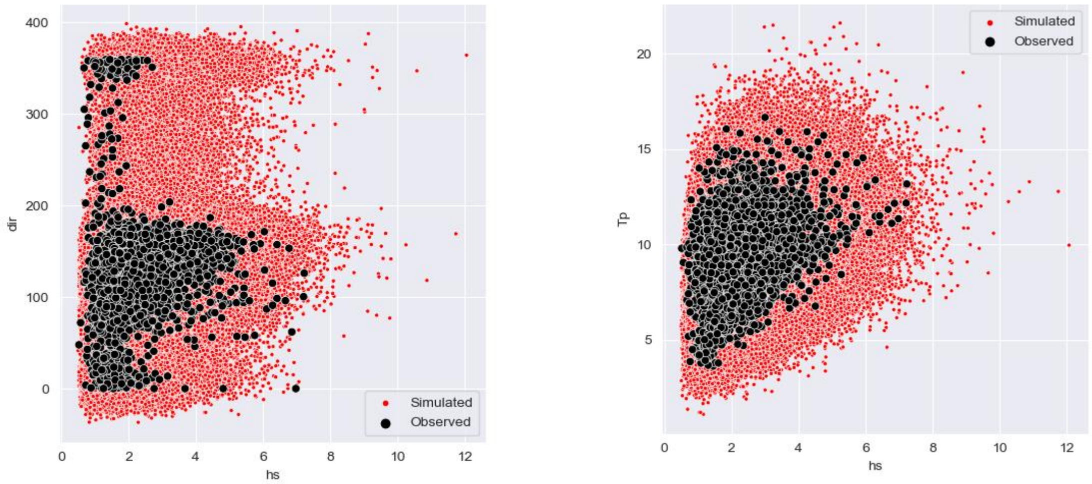

Figure 6-8 Historical and simulated offshore data of wave height (Hs) and wave direction (dir) (left), and significant wave height (Hs) and peak wave period (Tp) (right).

A.6 Data sampling

The large 10,000-year set of offshore data is required to be translated into nearshore wave conditions for each coastal unit. Numerical modelling will be used to simulate conditions into the nearshore, however it is not computationally efficient to model the full large dataset. Therefore, a subset of 200 events have been sampled from the large set to be used in numerical modelling. A Maximum Dissimilarity Algorithm (MDA) has been used for sampling. This method ensures that the full distribution and extremes of the larger dataset are retained in numerical modelling. Figure 6-9 compares the sampled events and the larger 10,000-year dataset.

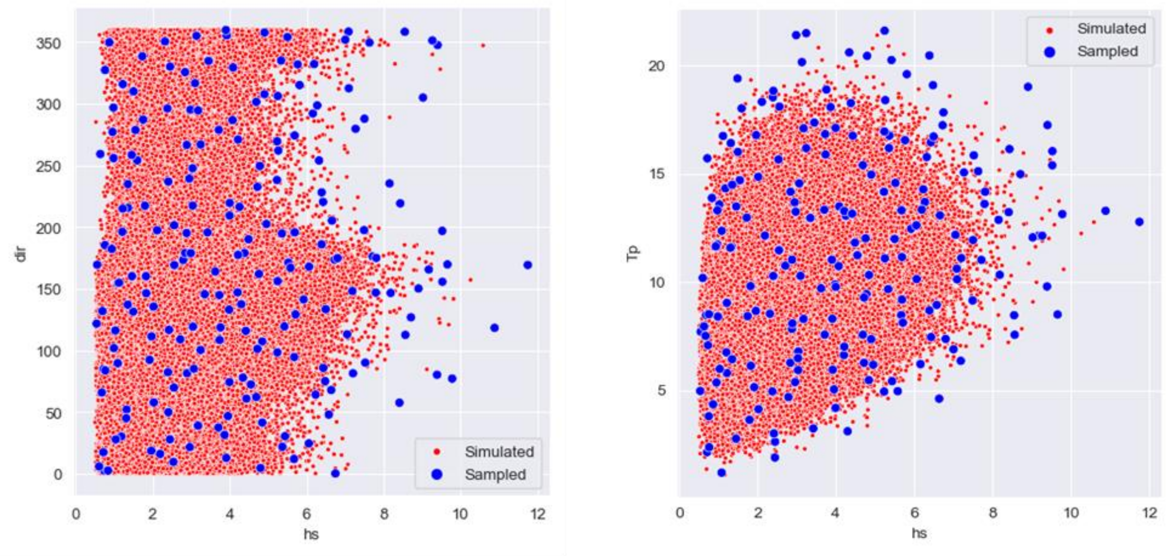

Figure 6-9: Simulated and MDA-sampled offshore data for Hs and Dir (left), and Hs and Tp (right).

A.7 Wave Modelling

A spectral wave model has been developed to model the subset of events. The SWAN wave model has been used. SWAN is an open source third-generation wave model, which is freely available, that simulates wave propagation in coastal and inland areas. It 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.

A.7.1 Modelling domain

A flexible mesh has been developed for wave modelling which allows for regions of high grid resolution at targeted sites along the coastline and around islands, with varying spatial resolution throughout the model domain. This approach optimises computational cost whilst resolving the wave interaction and complex geometry of the study area. The wave model extends offshore approximately 40 km towards the 70m depth contour aligned with the Brisbane WRB. The southern region of the domain extends from the northern tip of Moreton Island to the headlands at Noosa as shown in Figure 6-10. A minimum grid resolution of 30m spans the entirety of the SCC coastline, with coarser resolution ranging from 30m to 3.5km out to the model boundary.

A.7.2 Model Bathymetry

Model bathymetry data has been sourced from the following datasets as visualised in Figure 6-10.

- **5m Sunshine Coast LiDAR Topo-Bathy 2011:** This data has been derived by remotely sensed topographic (elevation) and bathymetric (depth) information, spanning the Maroochydore offshore area and Noosa offshore area using Airborne LiDAR Bathymetry during October – November 2011. Along the surveyed area from Noosa to Maroochydore, this dataset extends down to around -30m AHD and consequently has been used in model bathymetry to the extent of this dataset.
- **30m Geoscience Australia Bathymetry 2018[31]:** A compilation of digital elevation models (DEM) and bathymetric data at a regional scale. Data collation consists of deep-water multibeam surveying, airborne lidar bathymetry, and chart data. This data set resolves features to a resolution of 30m and has been used for the overall model and offshore regions. The present-day bathymetry of the Pumicestone Passage channel system has been derived from this set.

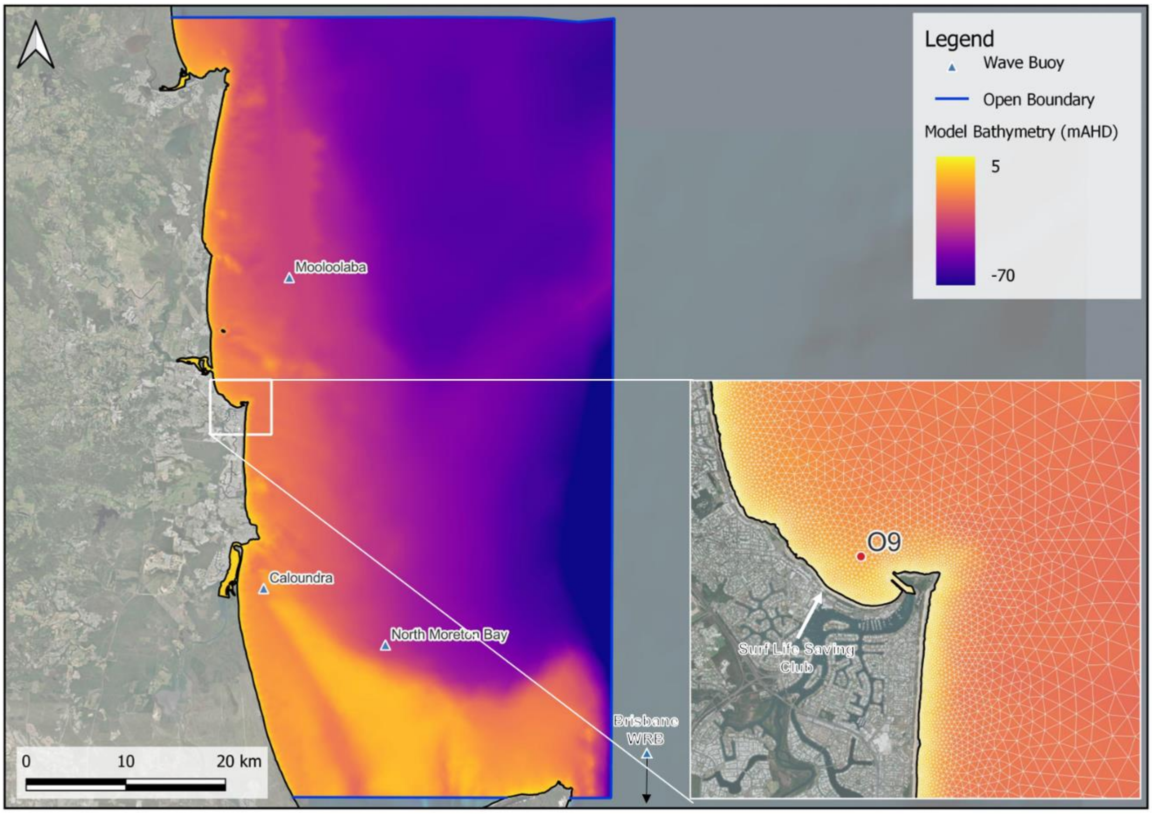

Figure 6-10: Wave model domain and inset detail of Mooloolaba study site showing nearshore output location for Mooloolaba study site

A.7.3 Model Calibration

The wave model has been calibrated against significant wave events observed at the Mooloolaba WRB. For each event, offshore conditions have been derived from the Brisbane WRB and applied to the wave model as a continuous timeseries. The calibration periods for each event is listed below:

- Event 01: 05/03/2004 to 6/03/2004, peak wave height at Brisbane WRB: 6.98m, peak wave height at Mooloolaba WRB: 5.84m
- Event 02: 27/01/2013 to 28/01/2013, peak wave height at Brisbane WRB: 7.1m, peak wave height at Mooloolaba WRB: 5.59m
- Event 03: 01/05/2015 to 02/05/2015, peak wave height at Brisbane WRB: 5.75m, peak wave height at Mooloolaba WRB: 5.20m
- Event 04: 21/08/2007 to 22/08/2007, peak wave height at Brisbane WRB: 5.47m, peak wave height at Mooloolaba WRB: 4.42m

31 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.

Each event has been subject to sensitivity analysis to determine suitable calibration parameters. Physics parametrisation schemes for wave energy dissipation due to bottom friction has been calculated through the JONSWAP, Madsen et al. and Collins constant parameterisation, where dissipation is based on a constant coefficient applied throughout the modelling domain.

A.7.4 Results of calibration

Figure 6-11 shows a comparison of recorded (Mooloolaba WRB) and modelled wave height data for Event 01. Table 6-7 displays the mean square error and peak error statistics for wave height when adopting the various physics parametrisations for bottom friction. Under the Collins parameterisation the model produced the best agreement to observed peak wave height, with an average error of 0.3% across the four events. Under this schematisation, the model was deemed to satisfactorily reproduce a range of large historic wave events and has been used for the extreme wave assessment.

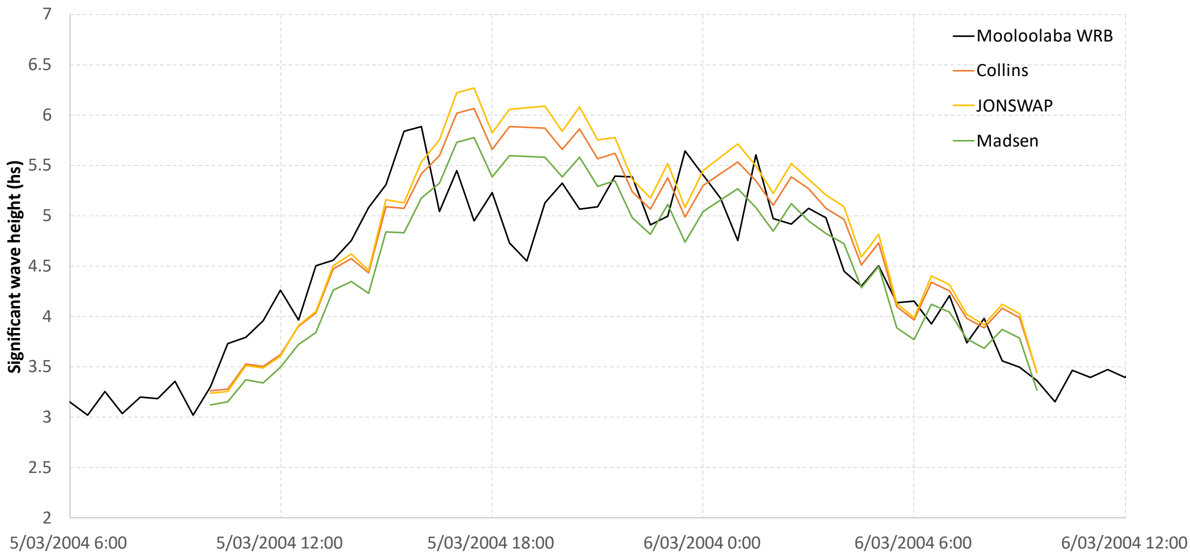

Figure 6-11: Modelled and recorded wave height at Mooloolaba WRB for Event 01, including each friction model applied for 5/03/2004 10:00 am to 6/03/2004 9:30 am.

Table 6-7: Comparison of recorded and modelled peak wave conditions for calibration events.

	Model period	Peak recorded Hs (m)	Peak modelled Hs (m)	Peak Error (m)	Peak Error (%)
Event 01	05/03/2004 to 6/03/2004	5.9	6.1	0.2	3.0%
Event 02	27/01/2013 to 28/01/2013	5.6	6.1	0.5	8.7%
Event 03	01/05/2015 to 02/05/2015	5.2	5.1	-0.1	-1.5%
Event 04	21/08/2007 to 22/08/2007	4.4	4.0	-0.4	-9.1%
Average				**0.05**	**0.3%**

A.7.5 Nearshore wave modelling

The calibrated model has been used in numerical wave modelling to extract present day nearshore wave conditions. The 200 sampled offshore wave events have been applied to the model and extracted at the study site at the -10m offshore depth contour, approximating the depth of closure as calculated from recorded wave data.

A.8 Nearshore wave emulation

Following the modelling of the 200 sampled wave conditions, the full 10,000-year set can be translated to the nearshore. This is accomplished using an emulation approach. The 200 nearshore modelled wave results are paired with their respective offshore input conditions. These offshore and nearshore pairs are used to a train a radial basis function (RBF) machine learning algorithm. An

RBF is a type of artificial neural network comprised of an input layer, a hidden layer, and an output layer. This method allows for universal approximation and faster learning speed than more complex neural networks. The trained RBF model has been used to emulate the full set of 10,000 years of offshore conditions to the nearshore.

A.9 Present day nearshore wave results

Figure 3-2 shows emulated nearshore results as a wave rose at Mooloolaba Beach (O9). This wave rose displays the distribution of wave height and wave direction for the full large wave dataset. The wave rose displays a north-easterly wave climate at Mooloolaba Beach due to wave sheltering at the headland of Point Cartwright.

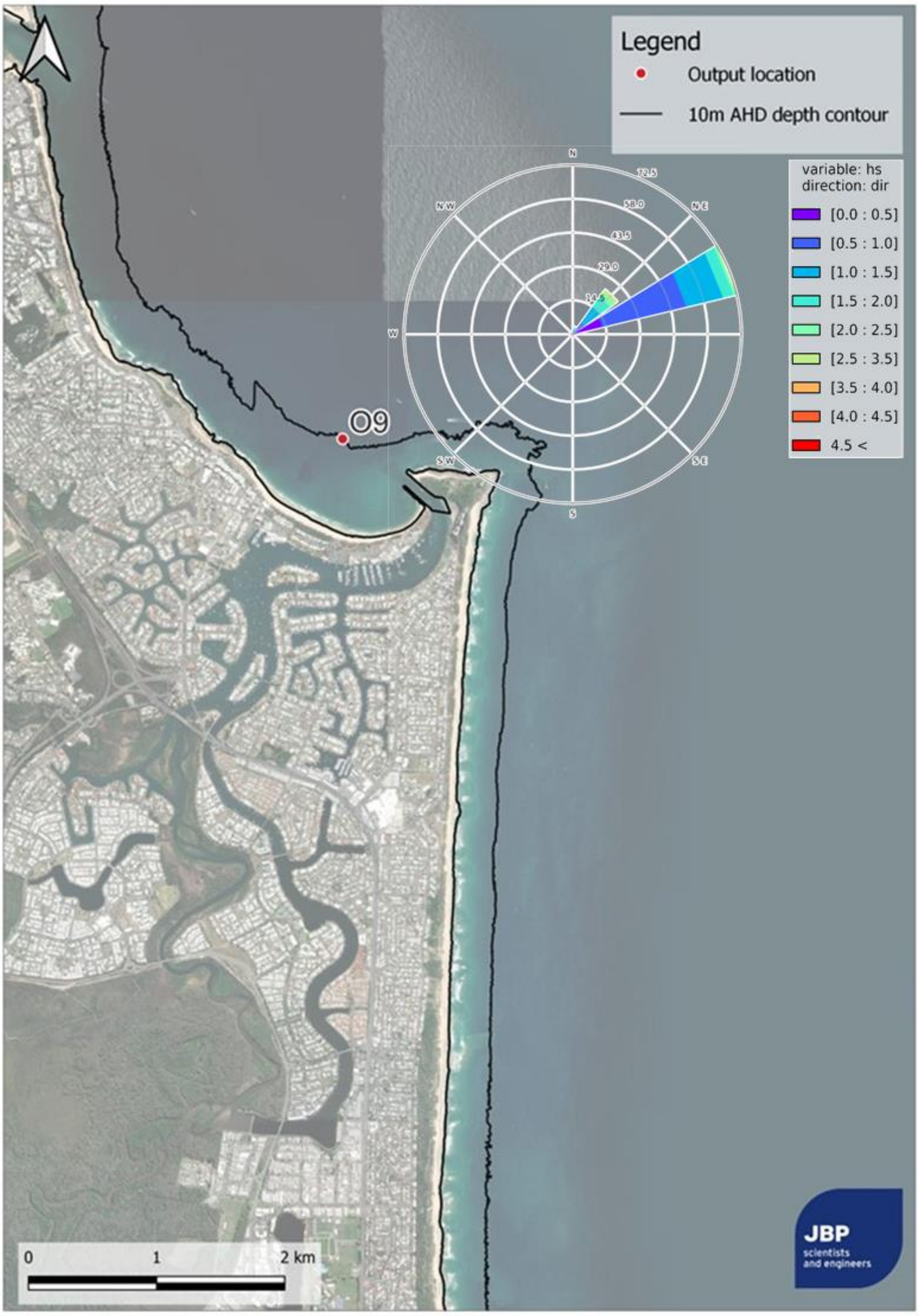

Figure 6-12: Present day emulated nearshore wave rose at Mooloolaba Beach

A.9.6 Present day nearshore extreme wave conditions

Following the translation of 10,000-years of wave conditions, extreme value analysis can be conducted for nearshore conditions. Table 3-2 shows extreme nearshore wave heights for Mooloolaba Beach. Extreme wave conditions have been estimated for a range of return periods up to 0.1% AEP (1000-year ARI).

Table 6-8: Present day nearshore extreme wave conditions at Mooloolaba Beach (O9).

	Hs (m), Tp (s), Dir (°N)					
Location	10%AEP	5%AEP	2%AEP	1%AEP	0.2%AEP	0.1%AEP
Mooloolaba Beach	3.5, 13.8, 44	3.7, 14.1, 43	4, 14.4, 42	4.2, 14.9, 41	4.6, 15.8, 39	4.7, 16.1, 38

A.9.7 Comparison to previous study

The reassessed present day extreme nearshore wave conditions have been compared to values published in BMT (2013). In the previous study, extreme wave conditions were assessed at the offshore Brisbane WRB. These were applied to a wave model and extracted at the -20m contour. As a result, these conditions are deemed conservative (i.e., a 1%AEP wave at Brisbane may not coincide with a 1% AEP wave at Mooloolaba) and do not necessarily account for local bathymetric effects including sheltering from headlands.

Table 6-9 compares reassessed extreme nearshore waves at Mooloolaba with the previous study. This table shows a decrease in extreme wave height, this is attributed to the differences in methodology described above, as well as the difference in depth of results between the previous study and the current study, -20m and -10m respectively.

Table 6-9: Comparison of BMT (2013) and JBP reassessed 1%AEP wave heights

Location (BMT 2013)	BMT Hs (m)	Coastal Unit (JBP)	JBP Hs (m)	% change
Mooloolaba Surf Club	5.9	O9	4.2	**-29%**

B Appendix B - 90% Detailed Design Wave Overtopping Calculation Sheets

Attached separately

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