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CONSULTING ENGINEERS

Byron Shire Council

Main Beach Shoreline Project

Numerical modelling and geomorphic assessment of concept options

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Document Summary

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Disclaimer

Bluecoast has prepared this document for Byron Shire Council with financial assistance from the NSW Government through its Coastal Management Program. This document does not necessarily represent the opinions of the NSW Government or the Department of Planning and Environment.

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Executive Summary

The coastal protection works on Main Beach between the Byron Bay SLSC and First Sun Holiday Park are referred to as the Jonson Street Protection Works (JSPW). Their function is to protect the town centre from coastal erosion. The works are degraded and do not provide suitable public amenity, aesthetics, public safety outcomes or beach access. The Main Beach Shoreline Project (MBSP) looks at how the JSPW can be updated to improve the coastal protection of Byron Bay's town centre.

This report supports the MBSP by providing a technical assessment of shortlisted concept designs identified to modify the JSPW. The technical assessment consists of two interrelated lines of investigation:

- a geomorphic assessment which uses a largely data-driven approach to summarise relevant coastal processes and infer the relative effects of the shortlisted designs on long term coastal processes
- application of numerical modelling tools to predict the response of the coastal environment to each shortlisted design relative to the basecase (i.e., the existing situation)

The shortlisted design options considered in this report are:

- Option 2 – berm rock revetment and pathway
- Option 5 – protective structure moved landward by 10m
- Option 6 – protective structure moved landward by 30m
- Option 7 – existing structure upgraded to contemporary standards.

A baseline geomorphological assessment was completed to explain the most relevant coastal processes occurring in the Byron embayment that influence the response to the JSPW. Adopting a data-driven approach an analysis of the study areas' sand budget was undertaken, which maps historical sand volume changes in 41 coastal sand cells. The most likely drivers for the observed coastal changes are described based on observational data, previous literature, state-of-the-art numerical modelling and/or coastal processes knowledge. Key outcomes are:

- Headland bypassing around Cape Byron results in a highly variable sand supply to the southern embayment with the annual range estimated to be from around 150,000 to over 900,000m^3/year. When coupled with the wave propagation characteristics of the embayment, the variable sand supply leads to a highly variable shoreline in the southern embayment.
- Sand movement pathways within the embayment follow two pathways: a littoral pathway (4m water depth) and a cross-embayment pathway. Based on sand volume changes determined from repeat surveys the relative split between the two pathways, when averaged across the embayment, has been calculated to be 70 : 30 (littoral : cross embayment). This is revised from previous assessments that assumed a 50 : 50 split between the pathways.
- The embayment geomorphic structure, including bedrock and coffee rock reefs and outcrops influence wave propagation, sand movements, shoreline dynamics and surfzone morphology in the embayment. The embayment's hard substrate reduces the volume of sand that can be stored in the southern embayment.
- The JSPW interacts with the embayment's natural sand movements, with the level of interaction (over the medium to long-term) controlled by the amount of sand in the Main Beach compartment, which in turn is a function of headland bypassing and wave climate.

A detailed wave and flow model capable of reproducing wave breaking and wave-generated currents along Main Beach has been developed using the SWASH model. The SWASH model results provide detailed information on the transformation of waves over the Byron embayment and its shallow reefs. The model identifies wave energy hot spots and shadows emanating from the shallow reefs and the effect this wave pre-conditioning has on nearshore hydrodynamics. Previous studies and field observations demonstrate that alongshore surfzone currents, driven by wave radiation stresses caused by wave breaking, go westward at the project site. The SWASH model confirms this clearly showing the main current flows west north-west parallel to the coast. The significant wave focusing areas over the reefs (rock outcrops) affect this alongshore current by causing alongshore accelerations/decelerations which also influence the location and behaviour of rip currents.

Comparison of the SWASH modelling results allow the effects of the shortlisted JSPW design options on the nearshore wave and hydrodynamics to be predicted, with key outcomes being:

- All options have minimal and largely localised changes to nearshore wave conditions, however, for higher tides and/or lower beach levels alongshore surfzone currents are changed from all options except Option 7.
- Outputs from the SWASH simulations at The Wreck surf spot during typical surfing conditions shows that the project cases do not significantly affect the wave heights or currents in this high value recreational area. Similar results, with no significant change in wave heights, current or wave breaking pattern, are observed in the SWASH simulations for the area immediately seaward of the JSPW. This is an area known to provide good surf from time to time. As demonstrated in the geomorphic assessment, good surfing conditions are believed to be related to the wave pre-conditioning (owing to Middle Reef), the distribution of surfzone coffee rock and headland bypassing which results in a 'bulge' morphology when the southern embayment is full of sand.
- While the project cases lead to a localised increase in current speeds these are in line with adjacent speeds. The removal of the groyne which acts as an obstacle, as is the case for all but Option 7, would see a minor but positive improvement in swimmer and surfer safety. It is suggested that the swimmer safety implications of these results be discussed with local NSW Surf Life Saving representatives in the next evaluation stages.
- Results for Option 5 and Option 5 show little discernible difference, with similar predicted outcomes for changes to local wave patterns, surfzone currents, cross-shore flow profile, surfing amenity and swimmer safer.

XBeach modelling focused on wave overtopping. It demonstrated that overtopping of the current JSPW far exceeds the safe limits for people on the seawall crest for the present-day 100-year average recurrence interval (ARI) water level and wave conditions. Damage to assets may also occur under this condition. The design options all significantly reduce overtopping to safer levels under present-day conditions. For future sea level rise scenarios, Option 2, Option 5 and Option 6 perform well but Option 7 does not and would require adaptation to maintain safe overtopping.

The bespoke shoreline modelling provides information on the expected response of the coastal environment at Main Beach (to the east) and Belongil Beach (to the west) following construction of each of the shortlisted design options. Based on the amount of sand bypassing the JSPW under basecase and project cases, the model quantifies the relative amount of beach volume and shoreline change expected at the adjacent beaches. Key outcomes are:

- All options that substantially realign the JSPW landward (i.e., Option 2, Option 5 and Option 6) result in a net reduction in beach volume (i.e., shoreline recession) at Main Beach with a corresponding advance in the beach volume/shoreline at Belongil Beach. The realignment of the shoreline in response to these options is not consistent but rather depends on the condition of the

Main Beach shoreline. On average the estimated shoreline change for these options are in the range of -5 to -12m at Main Beach and +5m to +10m at Belongil Beach.

- Of the two options that realign the rock revetment landward, there is not a substantial difference in the shoreline response between Option 5 (10m realignment) and Option 6 (30m realignment).
- Option 7, which upgrades the structure to contemporary standards while largely retaining the existing footprint, results in only minor shoreline changes.
- The model demonstrates that headland bypassing and the variability it causes to Main Beach's sand supply is the principal factor controlling shoreline dynamics along Main Beach and that is likely to remain the case irrespective of the option implemented.

The information presented in this technical report provides the basis for further development and evaluation of the shortlisted options at selecting a preferred option to carry forward. The evaluation and determination of the preferred option though CMP preparation in Stage 3 is the recommended pathway This is likely to include further engineering design development, cost estimates, economic appraisal, community consultation and multi-criteria analysis (MCA) workshops. It is recommended that the final design be subject to further detailed technical assessment to confirm the findings presented herein.

Based on the results of the detailed technical assessment, Option 5 and Option 6 do not appear to be sufficiently different from a technical performance perspective to warrant further evaluation of both options. In considering which options to carry forward from this technical assessment, it is recommended Council consider the likely outcome of further economic appraisal and/or multi-criteria assessment of Option 6. The lower benefits associated with loss of public and private assets/revenue and foreshore amenity coupled with the higher construction cost would mean that Option 6 will almost certainly compare poorly against Option 5 in any further evaluation.

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1. Introduction

1.1 About this report

This report supports the Main Beach Shoreline Project (MBSP). The MBSP is a Byron Shire Council initiative to improve coastal management of Byron Bay's town centre in line with the project objectives outline in Section 1.4. The purpose of the report is to provide a technical assessment of shortlisted concept designs identified to modify the Jonson Street protection works. The technical assessment consists of two interrelated lines of investigation:

- application of numerical modelling tools to predict the response of the physical coastal environment to each shortlisted design relative to the basecase (i.e., the existing situation)
- a geomorphic assessment which used a largely data-driven approach to summarise relevant coastal processes and infer the relative effects of the shortlisted designs on long term coastal processes.

In describing the outcomes to the above, the relative performance of the shortlisted design options regarding changes to nearshore waves and hydrodynamics, wave overtopping, and adjacent shorelines is provided. While the purpose of this report is not to arrive at a preferred option, where the relative performance of shortlisted design options is not sufficiently different to justify further evaluation recommendations are made on the potential to reduce the number of options carried forward. Further evaluation of the shortlisted options against social, environmental and economic factors is planned for subsequent work phases and will be aimed at arriving at a preferred option.

The report also provides information on the establishment of the models including the field data used in calibration and validation to demonstrate the models are suitable for application on the MBSP.

1.2 Project location

The project is located within the Byron Bay embayment on the far north coast of NSW. A site plan of the project is shown in Figure 1. The Byron embayment stretches from Cape Byron in the south to the Brunswick River entrance in the north. At the project site the gently curving shoreline is orientated in a south southeast/north northwest direction.

The embayment is exposed to waves from the east to north-east sector, with the predominant offshore waves from the south-east sector refracting and diffracting around Cape Byron and into the embayment. Long-term recession of the shoreline at Byron Bay has been identified in previous studies (PWD, 1978; WBM, 2000 and BMT, 2013). The strong wave refraction and diffraction processes drive a predominant east to west sediment transport along the foreshore at Byron Bay, with sand being supplied to the embayment from a nearshore sand lobe, and from sand bypassing Cape Byron and being driven by wave-generated surf-zone currents into Wategos Beach.

The project area is Arakwal Country. The Bundjalung of Byron Bay – Arakwal Bumberlin people are the recognised Aboriginal Traditional Custodians of the Byron Bay district. The project area is located within the Byron Shire Council's Local Government Area, the State Electorates of Ballina and the Federal Electorate of Richmond.

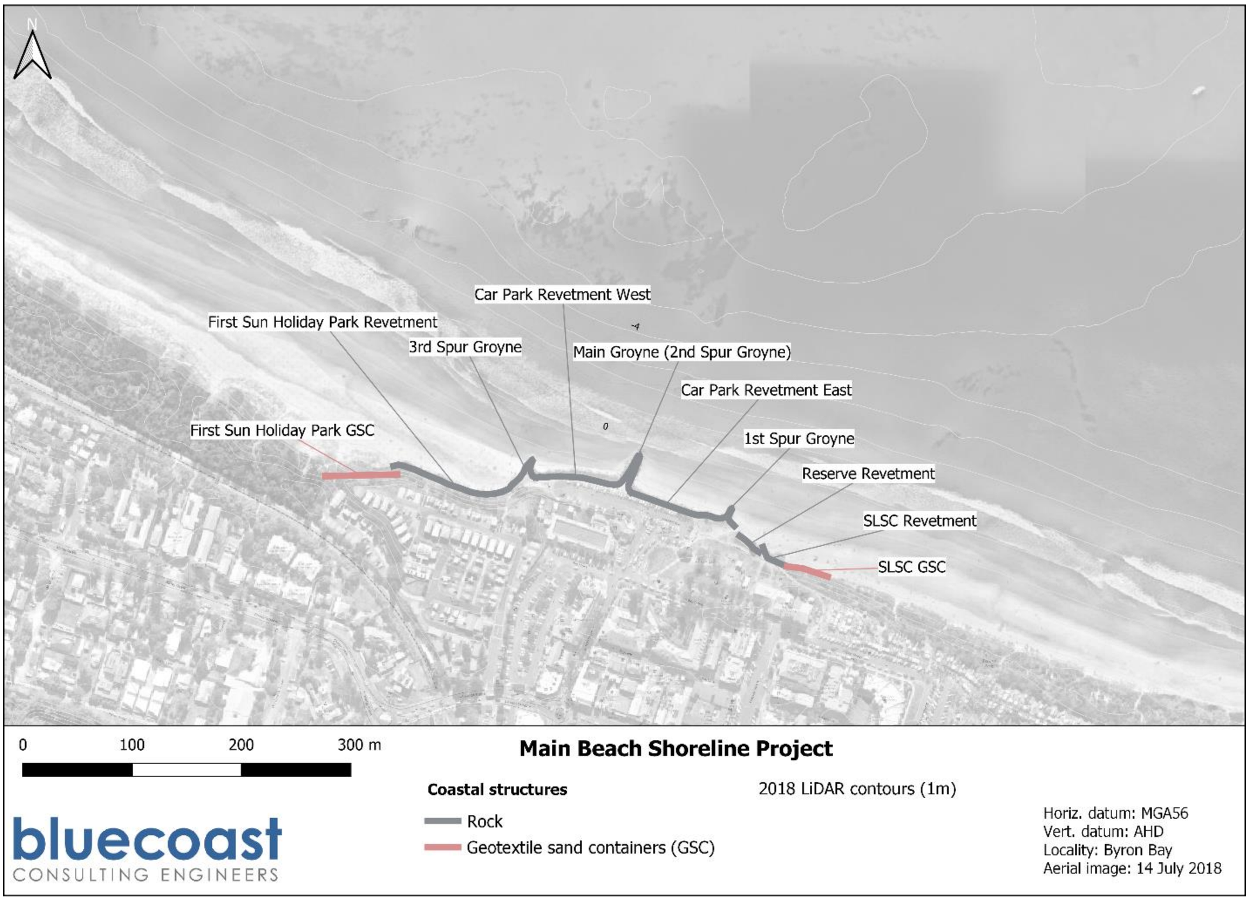

Figure 1. Map of project area.

1.3 Project background

Various coastal protection structures have been constructed by public authorities and individual residents along the Byron embayment's shoreline. On Main Beach, the coastal protection works between the First Sun Holiday Park and the Byron Bay SLSC are referred to as the Jonson Street Protection Works (JSPW), see Figure 1. Their function is to protect the town centre from the threat of coastal erosion.

Coastal protection works at this site date back to the original timber jetty that stood here from 1888 to 1930's (Bluecoast, 2021). The main rock revetments, which later became known as the JSPW, were repaired and extended in the early 1960's. In 1975 upgrades to the JSPW included the construction of three groynes, the main central groyne and two smaller spur groynes. With limited maintenance since the 1990's, the works are in poor condition (Bluecoast, 2020a) and have been previously identified as being degraded and not compliant with contemporary coastal engineering standards (WRL, 2009 and WorleyParsons, 2014). The works don't provide suitable public amenity, aesthetics, public safety outcomes or beach access. Modification works are required to bring the JSPW up to contemporary engineering standards.

Several investigations into the modification of the JSPW have been undertaken to date. A concept design for the upgrade of the JSPW is presented in WorleyParsons (2014). However, a Council meeting (22 February 2018; Res 18-104) resolved that further modification options shall be canvassed, evaluated and costed. The investigations of further options shall reconsider the available options, undertake a contemporary assessment of the options and refine a preferred concept design that as best as possible meets the project objectives in consultation with the community.

1.4 Project objectives

The following objectives for the MBSP were resolved by Council (18-839):

1. To provide adequate protection to the Byron Bay town centre against current and future coastal hazards.
2. To mitigate adverse current and future risks from coastal hazards, taking into account current and future coastal hazards.
3. To reduce the adverse impacts on coastal processes (e.g., downdrift effects) through reduction of the project footprint.
4. To improve the structural integrity and public safety of the JSPW.
5. To improve public safety around the JSPW.
6. To enhance recreational amenity of the foreshore around the JSPW.

1.5 Context of this report

Byron Shire Council (Council) have engaged Bluecoast Consulting Engineers (Bluecoast) to deliver the first stage of the Main Beach Shoreline Project (MBSP). The MBSP is a design investigation using multiple lines of evidence to investigate options and solutions for modification of the coastal protection works at Main Beach, Byron Bay. The project's first stage is focused on finding the solution for modification of the works that will give the best possible outcomes for Main Beach, Byron Bay and adjacent areas. This report is the last in a series of three reports for the MBSP, with the earlier supporting documents outlined in Table 1.

Table 1. Project documents that support this report.

Supporting document	Status	Document ID	Revision date	Refer to this document for:
Condition Assessment Report	FINAL	E2020/12114	20 February 2020	• Coastal engineering condition assessment undertaken on the JSPW
Baseline Understanding Report	FINAL	E2020/30756	30 July 2021	• Assessment of existing situation • Coastal processes summary • Identification of opportunities associated with MBSP
Concept Design Development Report	FINAL	E2020/30777	3 November 2020	• A summary of the project's critical factors • Preliminary design of seven concept options for the MBSP

The JSPW are a public asset that provides a significant role in protecting the Byron Bay town centre from the First Sun Holiday Park to the Byron Bay Surf Life Saving Club (SLSC) from coastal erosion and inundation. The MBSP is an important project for the community of Byron Shire, with the intent to improve the current situation. Through modification of the works, significant public benefit will be gained through improved coastal protection of the town centre, enhancing recreational amenity, improving public safety, improving public access and use of the foreshore and beach.

In recognition of the importance of a thorough design process for this project, Council have engaged experts to provide technical review and advice on key deliverables as the project progresses. In around May 2020, the following experts in the field of coastal engineering and science reviewed a draft Concept Design Development Report:

- Dr Phil Watson, Principal Coastal Specialist at Department of Planning and Environment (DPE)
- Greg Britton, Technical Director at Royal HaskoningDHV
- James Carley, Principal Coastal Engineer at the Water Research Laboratory.

The experts review and advice was reflected in the November 2020 version of the Concept Design Development Report. Greg Britton and DPE also reviewed the draft version of this report. The feedback from which has been incorporated into this revised version.

1.6 Objectives, scope and structure of this report

The objective of the modelling and geomorphological assessment presented in this report is to develop an understanding of the likely response (i.e., physical response of the coastal environment) to each shortlisted designs with consideration of:

- changes to the way sand moves around the JSPW and the effect of this on adjacent shorelines and coastal profile including Main Beach and Clarkes Beach to the east, and Belongil Beach to the west (see Figure 2).
- surfing amenity noting the highly regarded surf quality from time to time in the surf zone adjacent the JSPW.

The scope of the technical assessment encompasses both modelling and geomorphic assessment and is set out within the following report structure:

- A summary of the data used in the report is provided in Section 2.
- Section 3 details a baseline geomorphological assessment focused on explaining the most relevant coastal processes occurring in the Byron embayment that influence the response to the JSPW.
- Section 4 outlines the modelling approach including the adopted modelling tools and calibration standards.
- Section 6 provides information on the 40-year nearshore wave hindcast model for study area.
- Section 7 sets out the SWASH modelling application which is a detailed wave and flow model used to predict the effects of the shortlisted designs of the JSPW under various conditions.
- Section 8 outlines XBeach modelling used to inform overtopping of the JSPW and the shortlisted designs.
- Section 9 provides a quantified coastal processes model and the predicted long-term coastal response of the shortlisted design options.

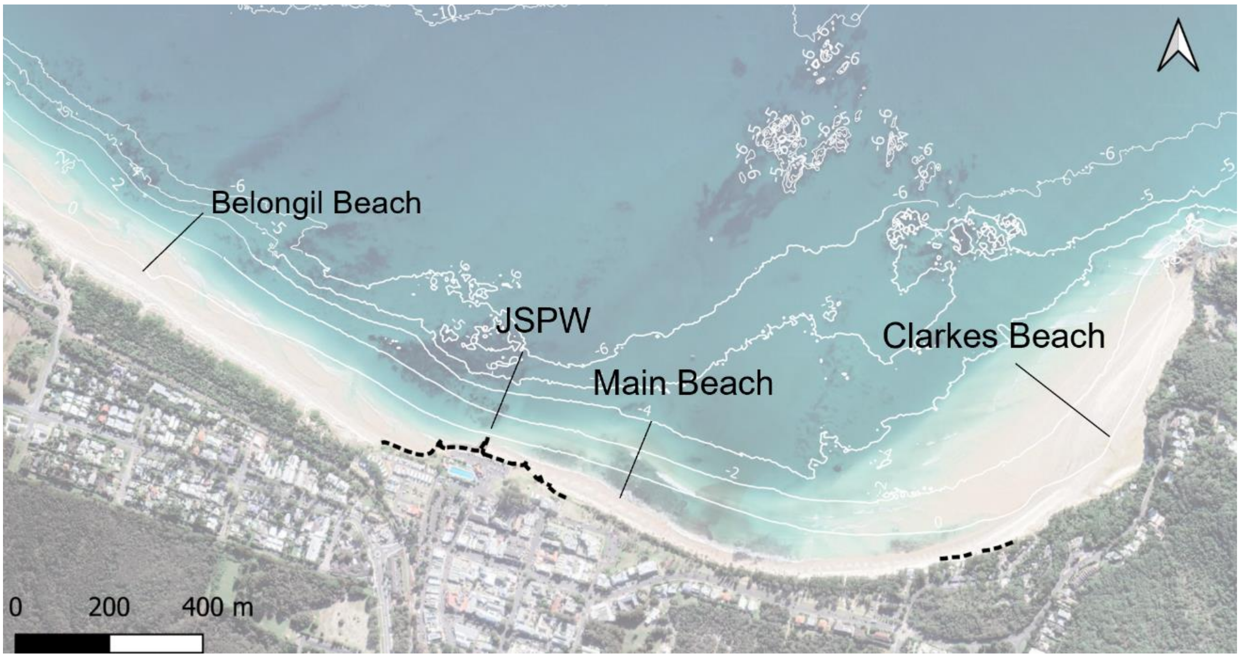

Figure 2. The JSPW and adjacent beaches.

2. Data used

2.1 Introduction

This section provides an overview of the data used in this assessment. The data was derived from historical records that were kindly made available for use as well as a project specific data collection which was undertaken to address key data gaps. The data has been used for the establishment and application of numerical models and for the data-driven geomorphic coastal response assessment.

A description of the existing coastal and estuarine environment, including key environmental drivers like waves, water level variation and tidal and fluvial flows is provided in Bluecoast (2021) and BMT (2013).

2.2 Existing data

Byron Bay's coastline is reasonably well observed. Both long term monitoring site and historical surveys are undertaken in the region. A summary of the datasets used is presented in Table 2, with associated monitoring sites (where applicable) displayed in Figure 3.

Table 2. Overview of existing observational data used in this study.

ID	Description	Source	Dates
Water Levels	Water levels from: • Tweed Entrance South (1 min) • Tweed Heads offshore (1 hour) • Brunswick Heads (1- and 15-minutes)	MHL	 May 2014 – May 2016 December 1982 – July 2019 Feb 1986 – Jul 1999
Waves	Measured wave heights, directions and periods at Byron Bay WRB at 1-hour sampling	MHL	Oct 1976 – Oct 2021 (directional since 1999)
	CAWCR hindcast of modelled wave heights, directions and periods offshore of Byron Bay at an hourly sampling	CSIRO	1976 – Nov 2021
Winds	Byron Bay AWS at a 1-minute sampling period	BOM	Sep 2010 – Oct 2021
Topography and bathymetry	Digital Earth Australia (DEA) shorelines	Geoscience Australia	From 1988 to 2019
	Single beam bathymetry and coastal topography	OEH	2002
	Drone surveys	Bluecoast	Jul, Oct 2019 Feb, Jul, Oct, Dec 2020
	Coastal lidar data at 5-meter resolution	DPE	2011 and 2018
	High resolution, rectified aerial imagery	Nearmap	2012, 2013, 2014, 2015, 2016, 2017, 2018, 2019, 2021
Aerial imagery	Nearmaps aerial imagery	Nearmaps.com	Various dates from 2012

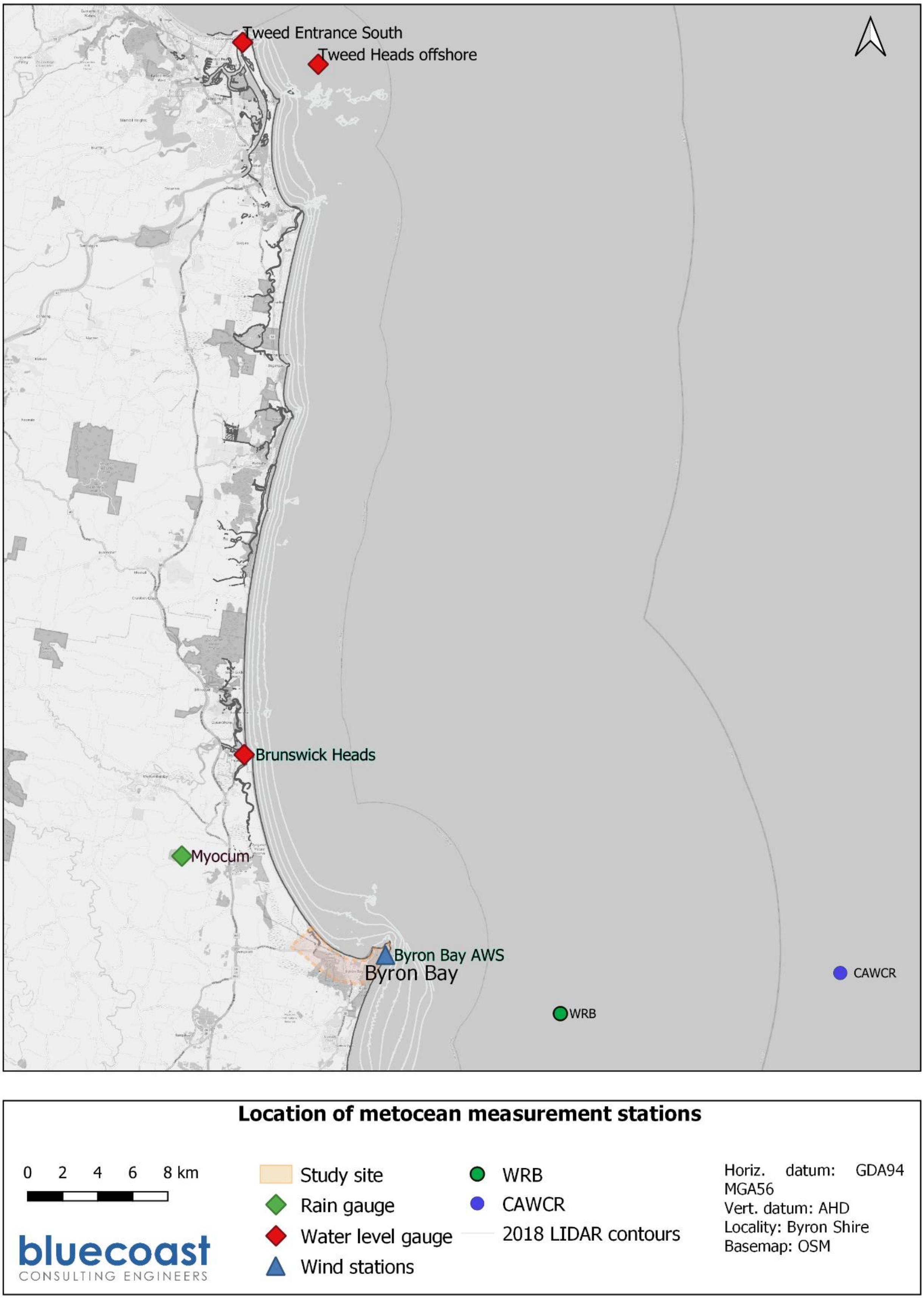

Figure 3. Map of historical and long-term water level, wind and wave monitoring sites.

2.3 Project specific data collection

A project specific data collection exercise was completed between July 2019 and December 2020 and used to inform the design investigations. The monitoring provided contemporary beach morphology and nearshore wave and current data to allow adequate validation of the numerical models. A summary of the monitored datasets used is presented in Table 3, with associated monitoring sites displayed in Figure 4.

The oceanographic monitoring consisted of two sites monitored for a period of just over two months. A Nortek ADCP[1] (Acoustic Doppler Current Profiler) was deployed in the nearshore area at a location referred to as MB01 which is approximately 500m from the shoreline in approximately 6m water depth (relative to AHD). The ADCP measured waves, currents and water level variation. A RBR pressure transducer[2] was deployed within the Jonson Street structure at the seaward end of the main groyne in approximately 0.3m water depth (MB02). This instrument measured water levels during high tides and/or large wave events. For more information on metocean monitoring data see **Appendix A**.

An unmanned aerial vehicle (UAV), or drone, was used to undertake six beach surveys along the Byron Bay shoreline. By using the high-resolution images and the GCPs, ortho-mosaic aerial images, a digital surface model (DSM) and a high-density point cloud were produced. For more information on the drone beach surveys see **Appendix B**. A summary of four drone surveys undertaken for the MBSP is shown in Figure 5.

Table 3. Overview of project specific data used in this study.

ID	Instrument	Description	Dates
Nearshore waves and currents (MB01)	ADCP	Measured wave heights, directions, periods, currents and water levels at Byron Bay (lat. -28.6369 long. 153.6152) approx. 5.8 m water depth	15 December 2019 to 27 February 2020
Waves and water levels (MB02)	RBR	Measured water level at Byron Bay (lat. -28.6400 long. 153.6131) approx. 0.3 m water depth	15 December 2019 to 27 February 2020
Beach morphological change	UAV/Drone	Each mission was flown at an altitude of 70m resulting in an image resolution of 2.1cm/pixel.	29 July 2019 28 October 2019 27 February 2020 31 July 2020 2 October 2020 15 December 2020

[1] An ADCP is a device that uses sound to measure the velocity of a fluid flowing past it. They are commonly used in oceanography and coastal engineering to measure currents and waves.

[2] RBR (Rougier & Bureau RBR) pressure transducer is a type of sensor that converts a pressure measurement into an electrical signal. RBR pressure transducers can record at a high frequency and are commonly used in coastal engineering to measure dynamic pressure fields.

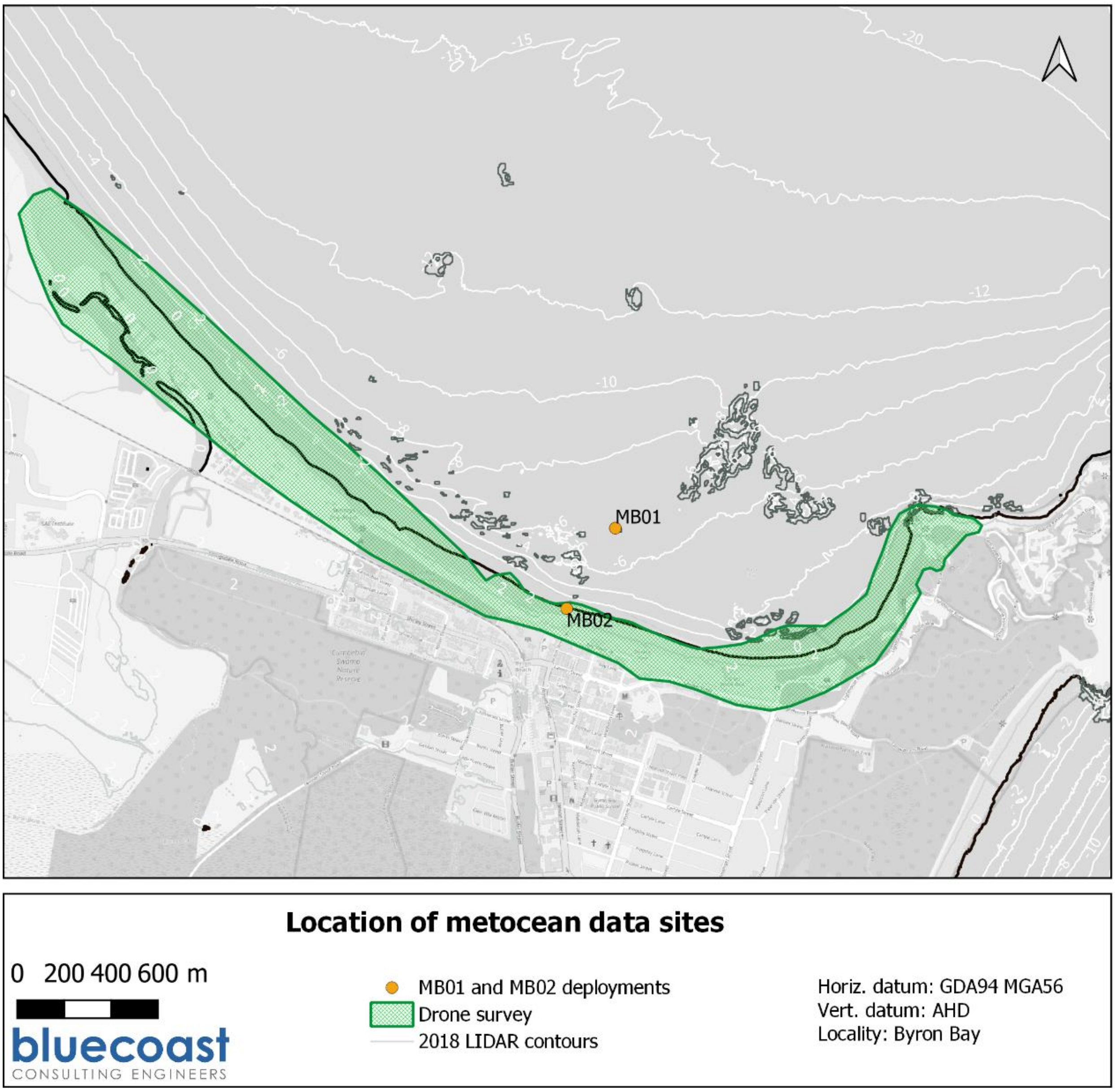

Figure 4. Location of metocean data sites.

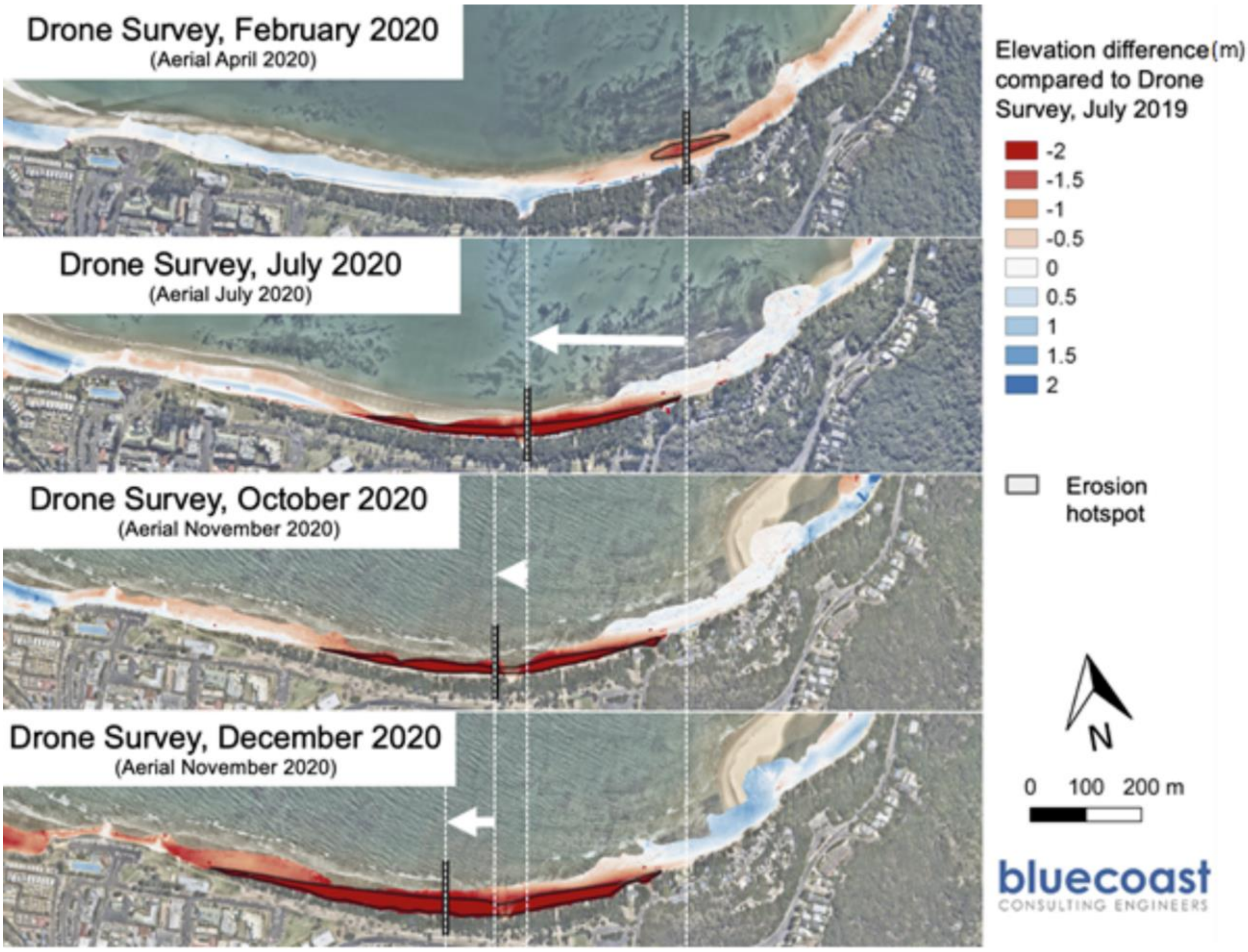

Figure 5: Summary of four drone surveys relative to the July 2019 drone survey undertaken for the MBSP.

3. Baseline geomorphic assessment

3.1 Introduction

A baseline geomorphological assessment was completed to explain the most relevant coastal processes occurring in the Byron embayment that influence the response to the JSPW. The baseline assessment adopts a data-driven approach to infer the rates and pathways of sand movements. At its centre is an analysis of the study areas' sand budget, which maps historical sand volume changes in 41 coastal sand cells (see Section 3.3). The most likely drivers for the observed coastal changes are described based on observational data, previous literature, state-of-the-art numerical modelling and/or coastal processes knowledge. Wherever possible, multiple lines of evidence have been used to cross-check, validate and provide greater confidence in the findings. Limitations are stated and uncertainty has been quantified for some of the findings.

The baseline geomorphic assessment considers the existing (basecase) conditions[2] of the JSPW and the embayment more broadly. Following the application of numerical modelling, as set out in Sections 4 to

[2] The basecase is intended to represent contemporary conditions of the study area, as they existed at the time of this report. The selection of what represents the basecase is somewhat dependent on what data is being analysed. For example, the 2018 Coastal LiDAR survey (captured in July -August 2018) is used to represent the basecase morphology for survey analysis.

Section 8, the geomorphic assessment will be expanded to also infer the predicted response of the coastal environment to each of the shortlisted designs relative to the basecase. This second predictive part of the geomorphic assessment, focused on JSPW, is provided in Section 9.

3.2 Overview on coastal processes

The Byron embayment is contained within the *'Tweed'* coastal sediment compartment which, as defined in the *NSW Coastal Management Act 2016,* extends from Cape Byron to Point Danger. Adjacent to the *'Tweed'* compartment is the '*Cape Byron to Ballina*' sediment compartment with extents from the Richmond River to Cape Byron (see Figure 6). The JSPW, a key focus of the MBSP, are located on the Byron embayment shoreline about 2.8km west of the Cape and about 3.0km south-east of the Belongil Creek entrance.

As outlined in the MBSP's Baseline Understanding Report (Bluecoast, 2021) there has been numerous studies examining the coastal processes within the Byron embayment including a number that focused on the JSPW. Key studies from which the MBSP has drawn include:

- 1978 *Byron Bay – Hastings Point Erosion Study* (NSW Department of Public Works, 1978).
- 2013 *Byron Shire Coastline Hazard Study Update* (BMT, 2013). Being the most recent comprehensive investigation into coastal processes and hazards undertaken in the Byron Shire.
- Dean Patterson's 2013 PhD thesis titled *Modelling as an aid to understand the evolution of Australia's central east coast in response to late Pleistocene-Holocene and future sea level change*
- 2013 Goodwin et. al. paper titled *An insight into headland sand bypassing and wave climate variability from shoreface bathymetric change at Byron Bay, New South Wales, Australia*
- 2014 WorleyParsons' *Investigating the Re-design of the Jonson Street Protection Works*
- 2021 Ribo et. al. paper entitled *Shelf sand supply determined by glacial-age sea-level modes, submerged coastlines and wave climate*

Other reference studies including Bluecoast's current investigations for Stage 2 of Byron Shire's Open Coast Coastal Management Program (CMP) are listed in the reference list.

Based on a review and synthesis of all relevant previous investigations, site observations and data analysis, the following elements are key to the baseline geomorphic assessment, with each described in turn below:

- The Byron embayment's sand budget including observed sand volume changes across the full coastal profile in the long and medium terms.
- Rate of net alongshore sand transport (LST) and gradients in longshore transport.
- Variable embayment sand supply via headland bypassing around Cape Byron and its effect of the quantities of sand in the southern embayment (from Little Wategos Beach to JSPW), the northern embayment (from JSPW to Belongil Creek) and the embayment's shorelines.
- Sand movement pathways within the embayment including the proportion that moves via the littoral pathway and the proportion that follows the cross-embayment pathway.
- The embayment geomorphic structure, including bedrock and coffee rock reefs and outcrops which influence wave propagation, sand movements, shoreline dynamics and surfzone morphology in the embayment.
- The JSPW and its interactions with the embayment's natural sand movements.

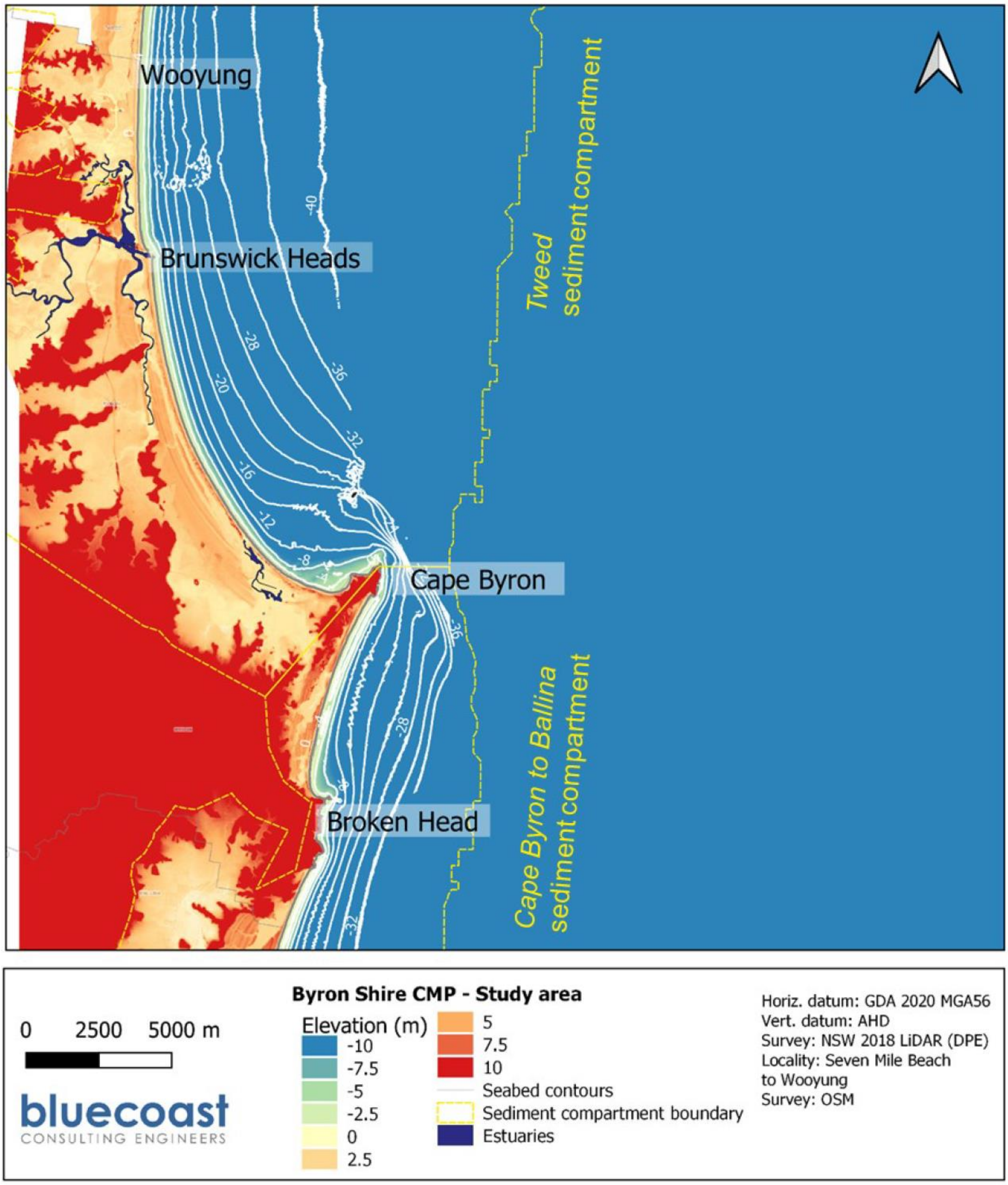

Figure 6. Coastal sediment compartments.

3.3 Byron embayment sand budget

An assessment of the change in the sand volumes within the project region, from Broken Head to Belongil Creek, was undertaken adopting the 41 analysis cells shown in Figure 8. The extents and division of the cells were defined in consideration of previous assessments, survey extents, observed processes as well as the cross-shore divisions of the coastal profile (see Figure 7). Cells were given a unique ID following *XX-A-B*, where *XX* is the beach cell, *A* is longshore beach sub-cell and *B* is cross-shore cell explained below:

- Subaerial beach (*1*): which extends from approximately shoreline (or approximate zero-meter AHD contour) to back beach area.
- Upper shoreface[3]: is the zone where under average conditions waves break and most wave energy is dissipated. Water level gradients, currents and sand movement are highest in this zone with the strong morphodynamic activity manifested in profile change and shoreline advance or retreat. On the upper shoreface time scales of profile change are in the order of days to years. Along Tallow Beach the upper shoreface was given *2* as the cross-shore identifier and is all depths less than around 12m. Within the Byron embayment the upper shoreface was divided into two sub-cells with *2* denoting depths less than around 4m AHD) and *3* denoting the sub-cell with depths between around 4m and 10m.
- Lower shoreface: is the zone of the profile were waves shoal. The seaward extent is marked by the closure depth. Sand transport rates on the lower shoreface are typically small with the profile responding to longer-term, decade-millennium time scale changes in wave climate and sea level. The lower shoreface was denoted by *LS*.

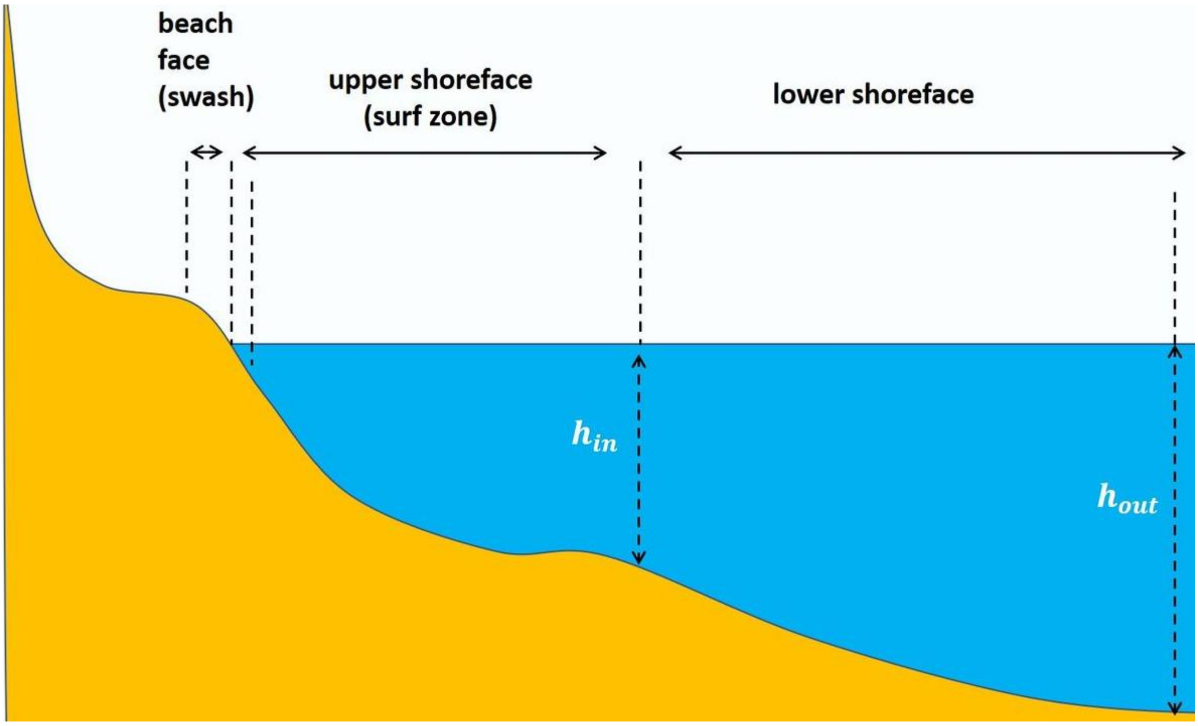

Figure 7. Representation of the coastal profile showing beach face and lower and upper shoreface (source: Coastal Wiki).

[3] The shoreface is the zone seaward of the shoreline where offshore generated waves interact with the upward sloping seabed. It extends seaward to where the influence of wave action on cross-shore sediment transport is on average minor compared to other influences.

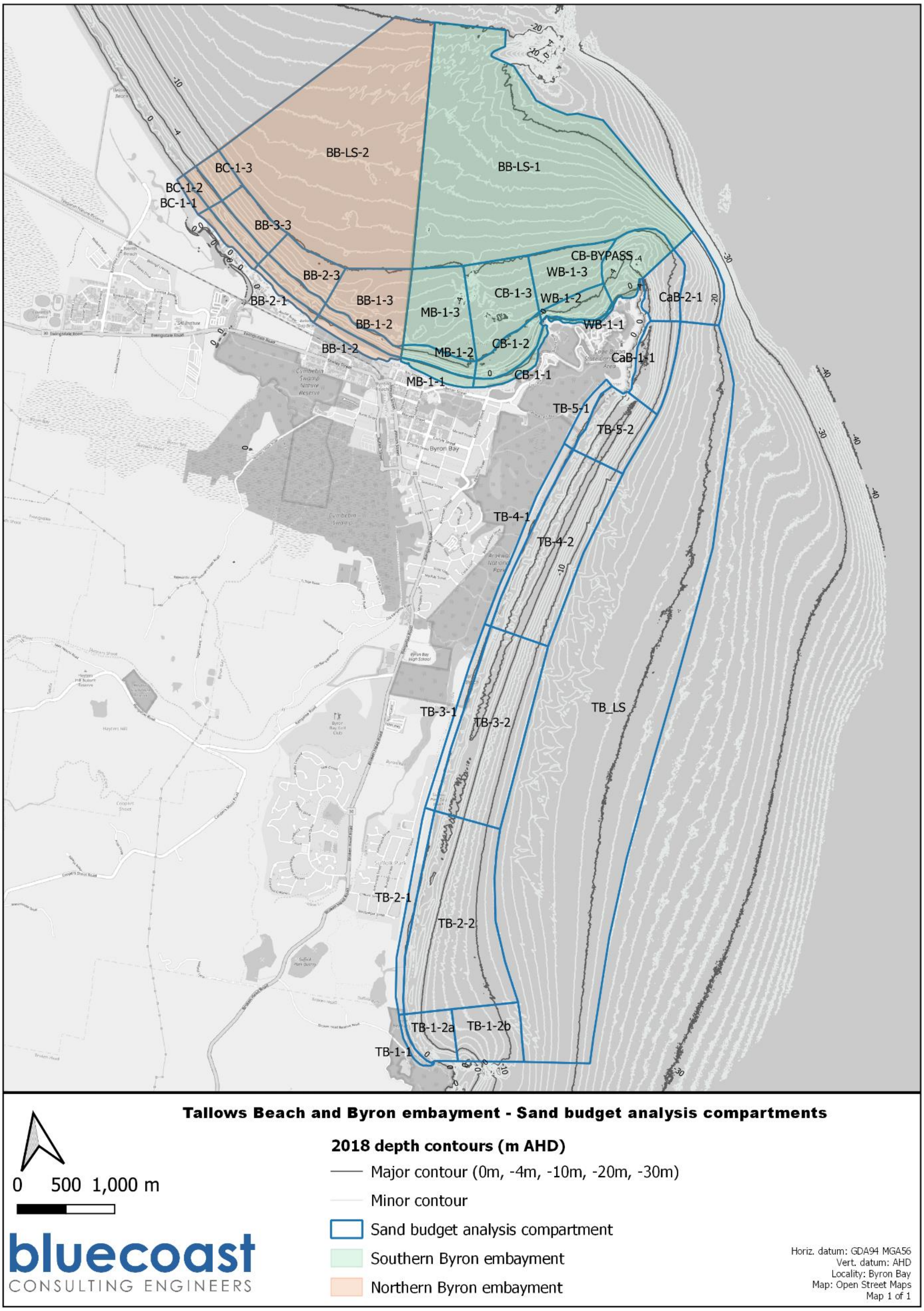

Figure 8. Sand budget analysis cells used for MBSP.

Changes in sand volumes relative to the 2018 survey was calculated, where survey extents allow, for each cell and for all available surveys. The sand volume changes for all cells are provided in **Appendix C**. Example maps of the two recent high resolution 2011 and 2018 surveys are shown in Figure 9 and Figure 10, respectively. Changes in surveyed levels relative to 2018 for the selected 1883, 2002 and 2011 surveys are shown in Figure 11, Figure 12 and Figure 13. Additional sequential survey difference maps for 2011 less 2002 and 2002 less 1883 are provided in Figure 14 and Figure 15.

All surveys were adjusted to be relative to AHD. The 1883 survey was conducted by Staff Commander Frederick Howard RN and comprised 7090 lead line sounding reduced to Low Water Ordinary Spring (LWOS) and plotted with 1 foot accuracy. This survey was reconciled to AHD by adding 0.852 m to all soundings following the procedure set out in Goodwin et al 2013.

An error or uncertainty analysis specific to the volume analysis reported herein has not been completed. Goodwin (2013), who used the same 1883, 2002 and 2011 surveys reported volume uncertainties of between 20% and 120% and generally adopted ±20% for transport rates. To capture the uncertainty in the derived volumes from survey analysis this typical uncertainty range of ±20% for transport rates has been adopted herein.

Table 4 provides a summary of the sand volume changes observed in the available surveys. Table 4 also provides cross-profile ratios comparing the relative change across the upper profile (subaerial beach : surfzone : lower surfzone) only and the entire profile (upper profile : lower shoreface).

Table 4. Summary of surveyed sand volume changes in Byron Bay region.

Zone	Volume (m^3) change relative to 2018 baseline			
	1883	2002	2011	2018
Tallows Beach				
Subaerial beach and upper shoreface	-	130,000	1,050,000	0
Lower shoreface (to 22m water depth)	-	248,000	-1,970,000	0
Cape Byron				
Subaerial beach and upper shoreface	-	-340,000	-980,000	0
Lower shoreface (to 22m water depth)	-	-90,000	-115,000	0
Southern embayment (Little Wategos to JSPW)				
Subaerial beach and upper shoreface	1,640,000	-13,500	700,000	0
Ratio (beach : surfzone : lower surfzone)	*4 : 18 : 78*	-	*13 : 67 : 19*	-
Lower shoreface (to 15m water depth)	2,690,000	41,500	120,000	0
Ratio (upper profile : lower shoreface) net change	*61 : 49*	-	*85 : 15*	-
Northern embayment (JSPW to Belongil Creek)				
Subaerial beach and upper shoreface	2,380,000	-330,000	-385,000	0
Ratio (beach : upper surfzone : lower surfzone)	*23 : 41 : 36*	-	*53 : 18 : 29*	-
Lower shoreface (to 15 water depth)	1,350,000	100,000	75,000	0
Ratio (upper profile : lower shoreface) net change	*64 : 36*	-	*84 : 16*	-

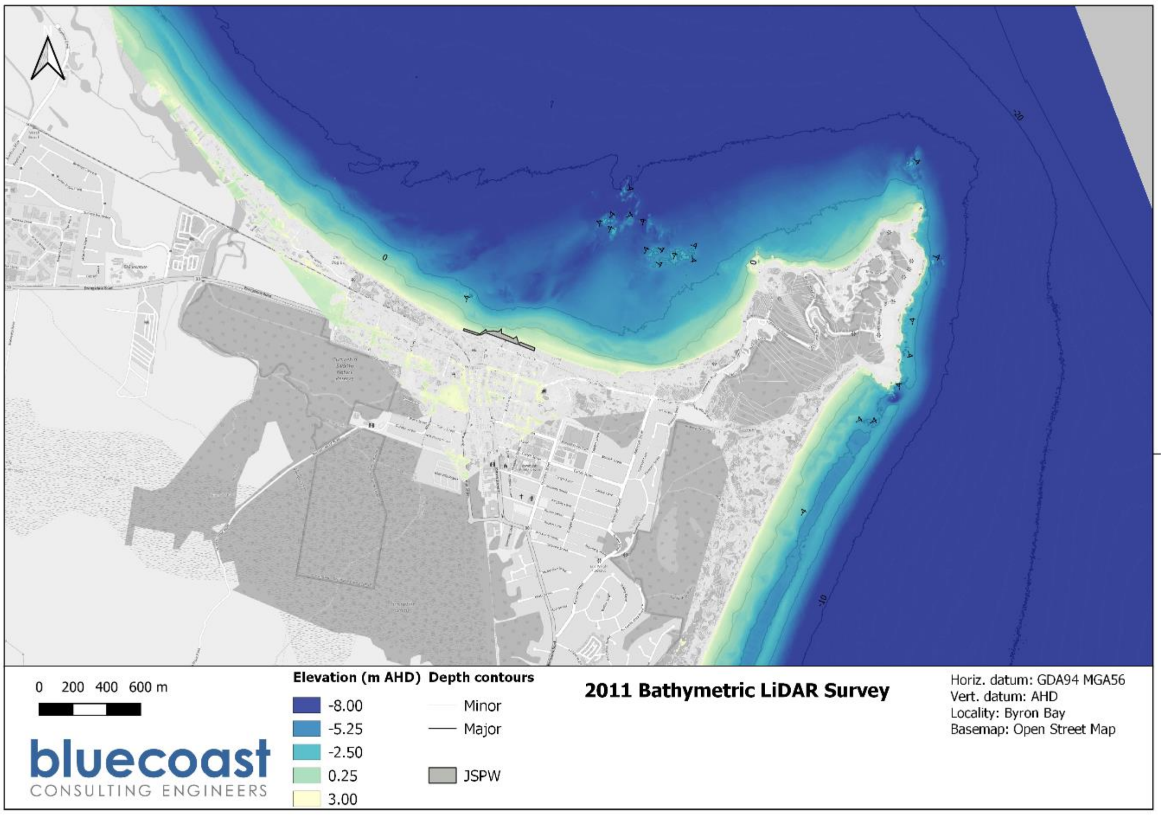

Figure 9. 2011 bathymetric and topographic LiDAR survey.

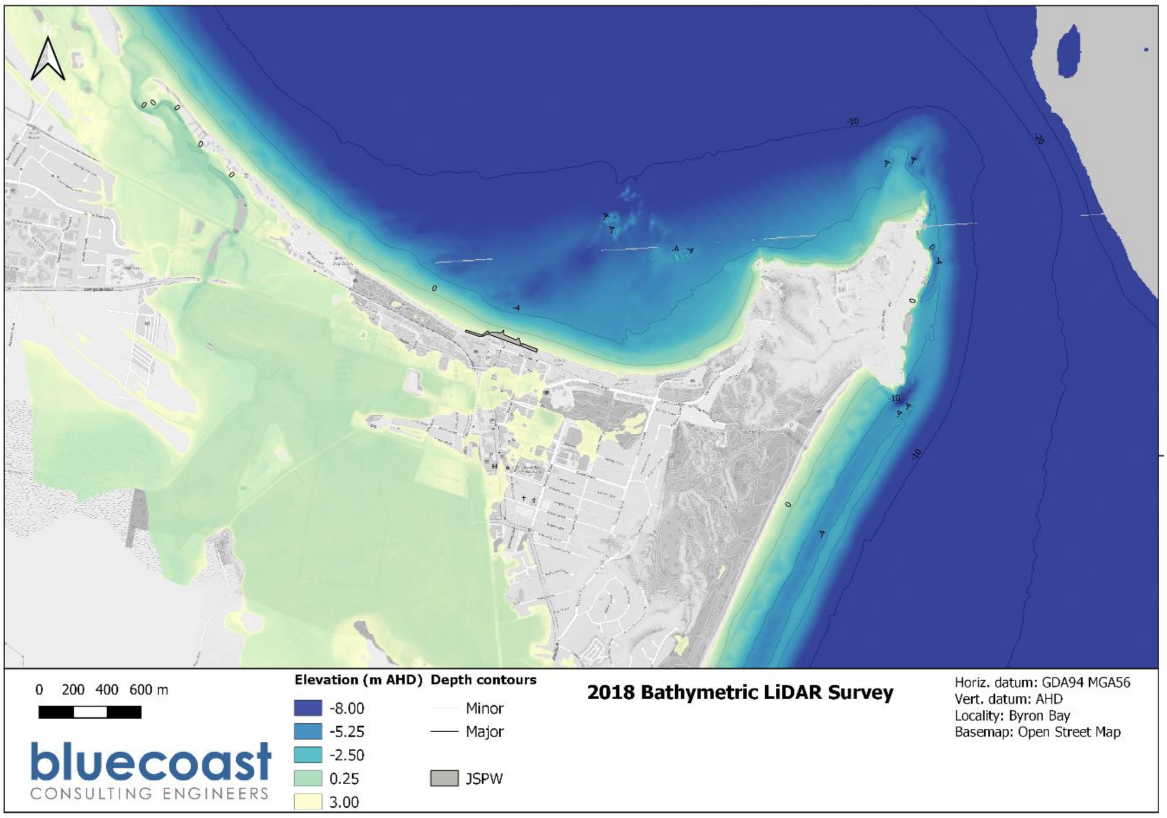

Figure 10. 2018 bathymetric and topographic LiDAR survey.

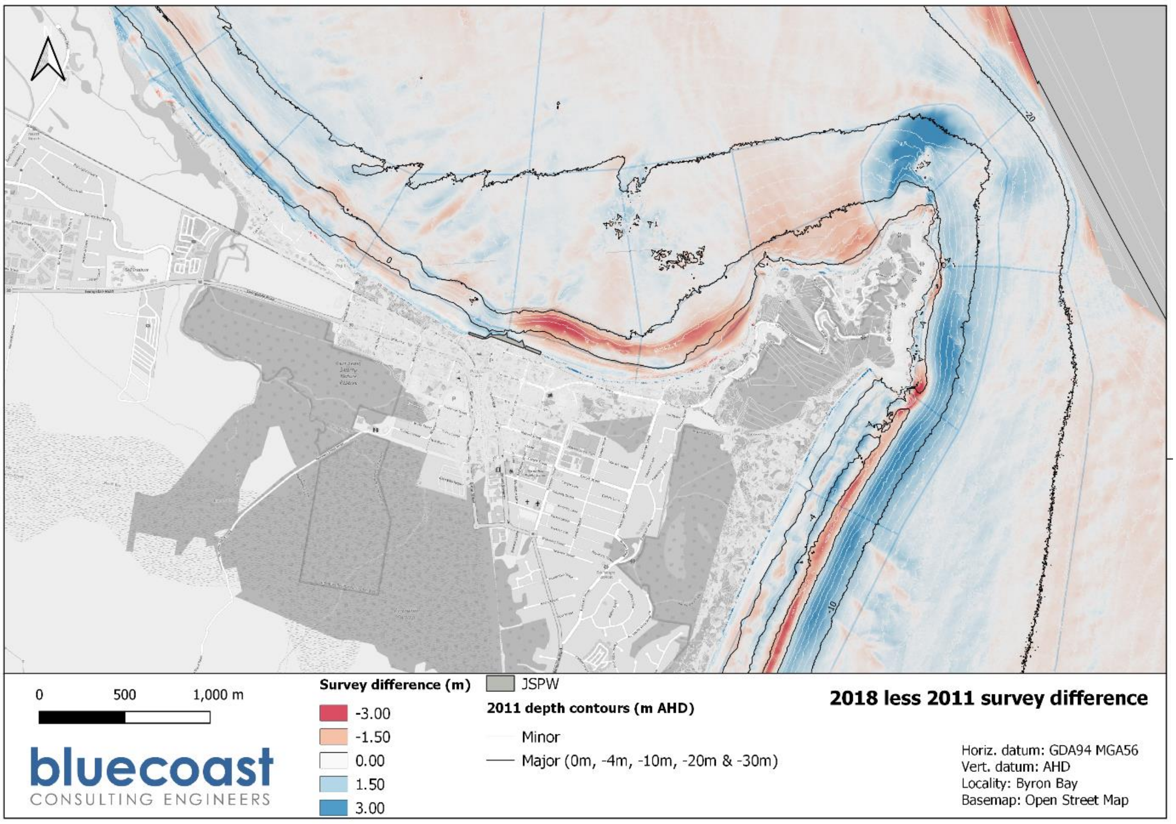

Figure 11. Survey difference 2018 less 2011.

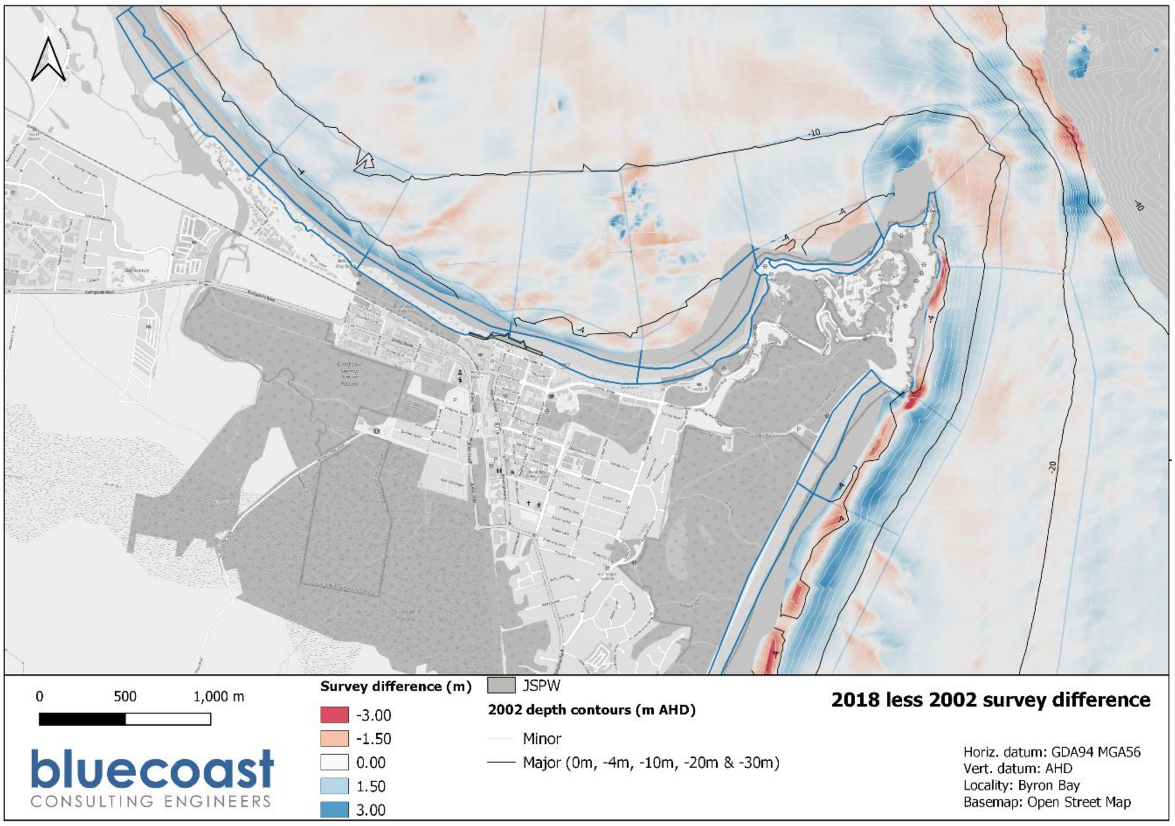

Figure 12. Survey difference 2018 less 2002.

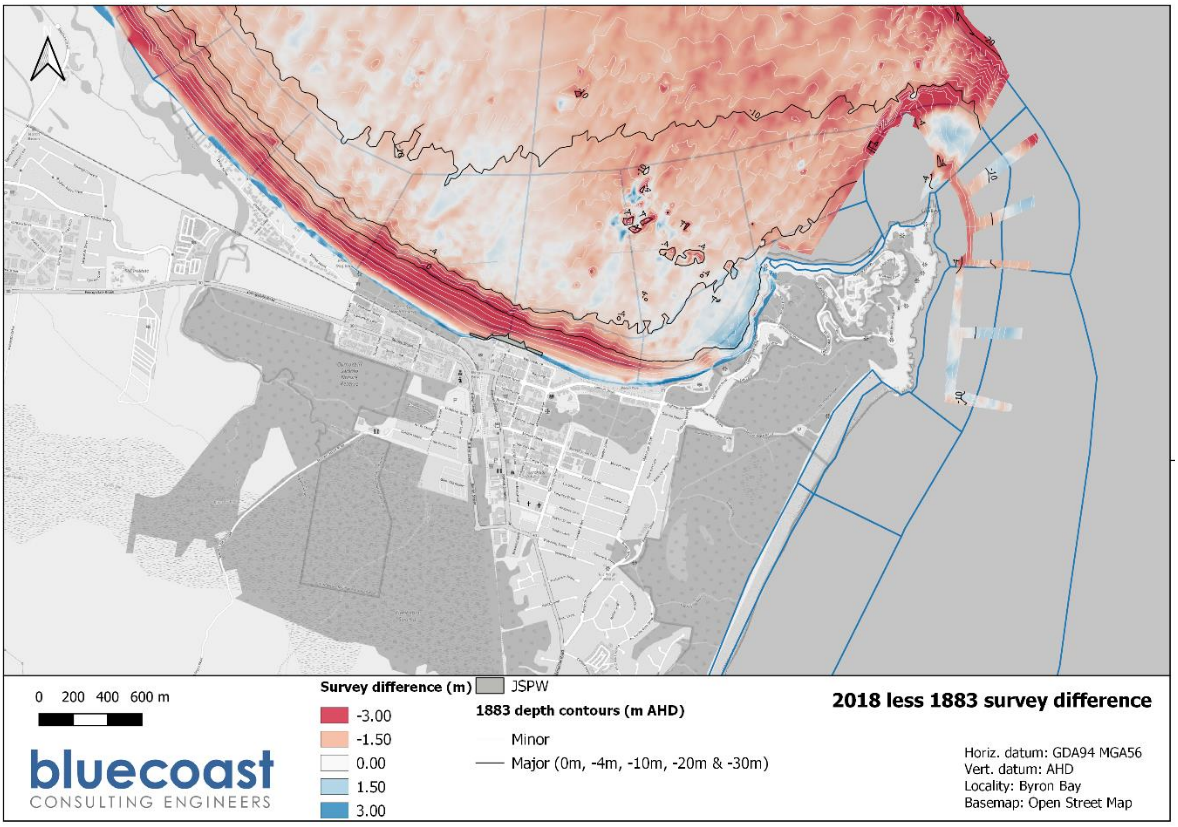

Figure 13. Survey difference 2018 less 1883.

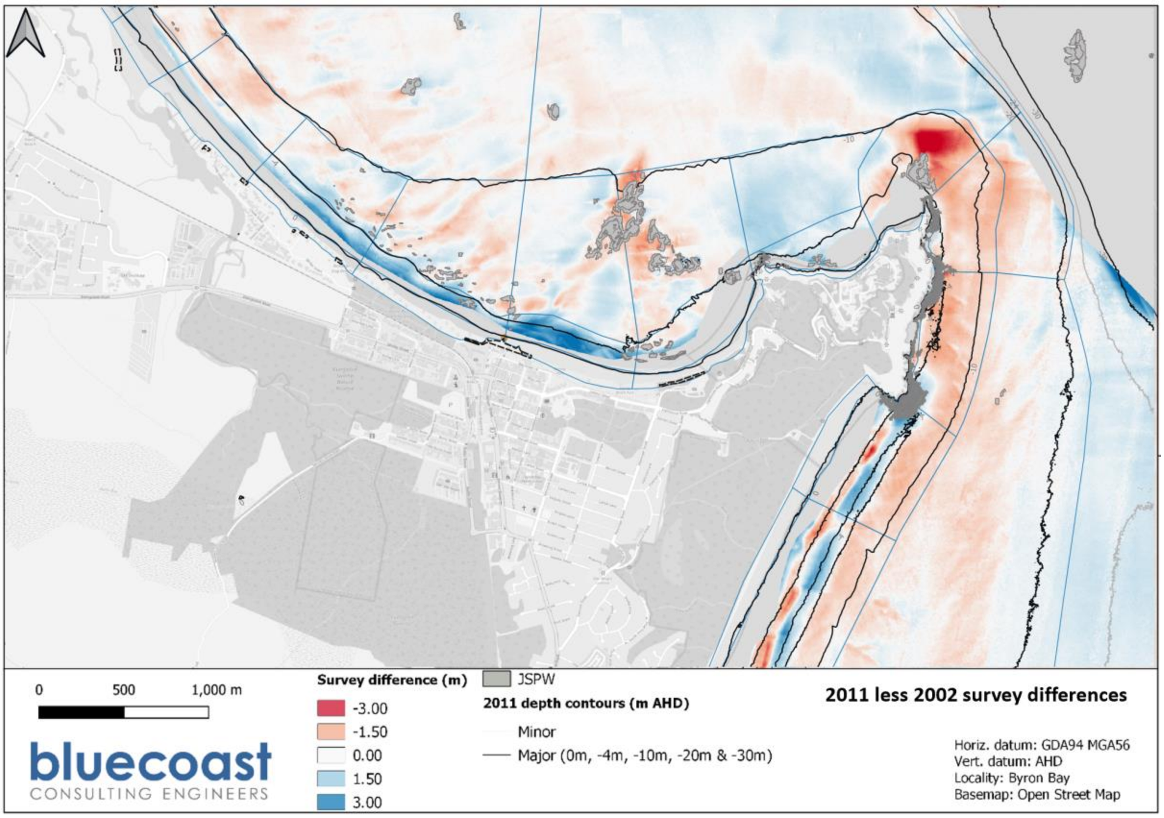

Figure 14. Survey difference 2011 less 2002.

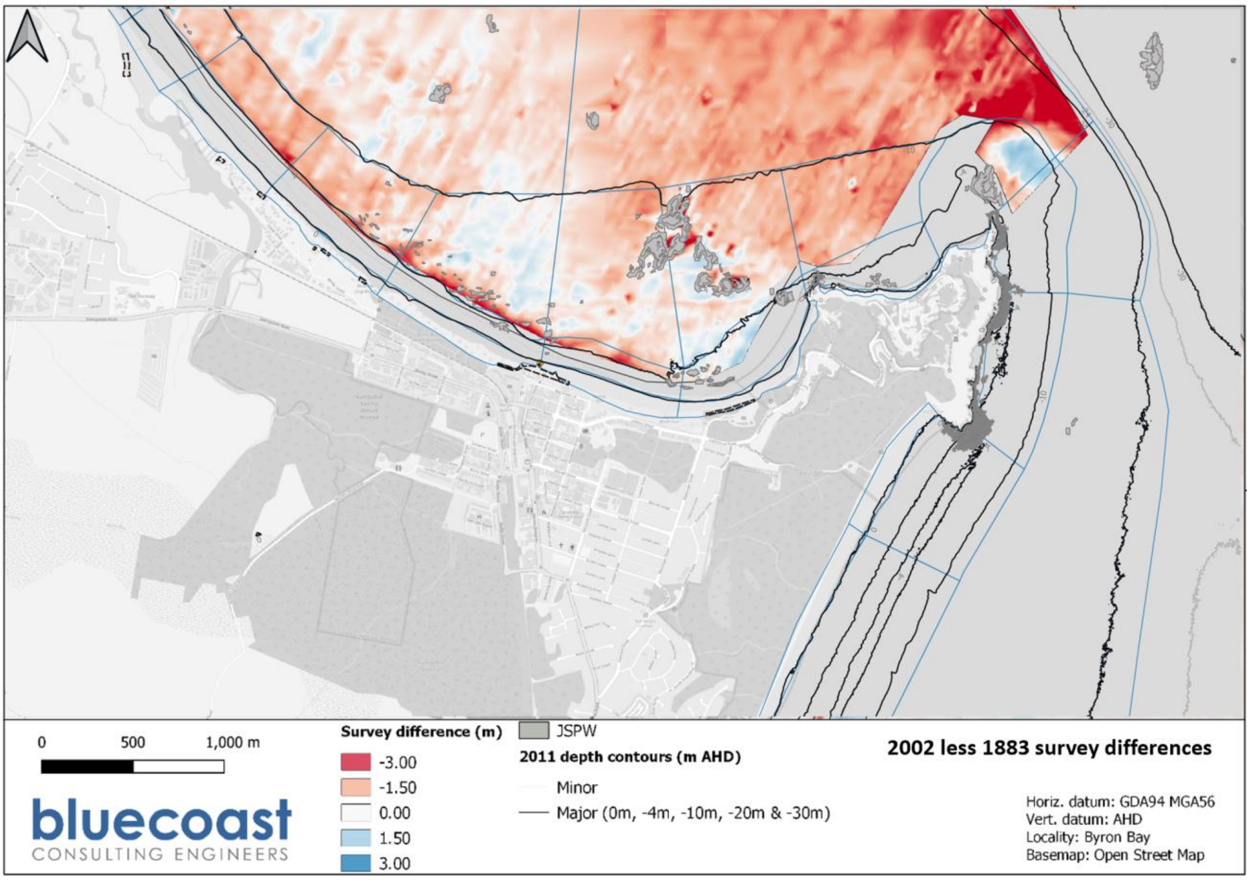

Figure 15. Survey difference 2002 less 1883.

3.3.1 Time scales for change

The beaches along the study region experience change over various time scales as illustrated in Figure 16 and described as:

- Long term changes occur over decade to centuries and are driven by persistent changes to sand budgets (e.g., reducing/increasing sand supply) and sea level rise.
- Medium term changes occur over years to decades and are driven by climatic cycles like ENSO and IPO and link to shifts in the wave climate.
- Short term changes can occur over days, weeks, months or years and are linked to storms, seasonal variations and ENSO fluctuation.

In the context of the sand budget analysis, it is important to understand these fluctuations. Surveys are undertaken at a point in time with the morphology captured reflecting the preceding conditions. Short to medium term influence may thus mask longer-term trends and care must be taken in interpreting the sand volume changes. Key outcomes are described below for long term and medium-term observations.

Figure 15 and Figure 18 show the four surveys against time histories of the Interdecadal Pacific Oscillation (IPO) and Southern Oscillation Index (SOI used to track ENSO):

- 1883 survey was captured in a neutral SOI year within a multidecadal IPO La Niña like phase
- 2002 survey was captured in a neutral SOI neutral year while the 2011 survey was captured in an extreme La Niña SOI year both within an IPO El Niño like phase
- 2018 survey was neutral SOI year with the IPO transitioned to a La Niña like phase

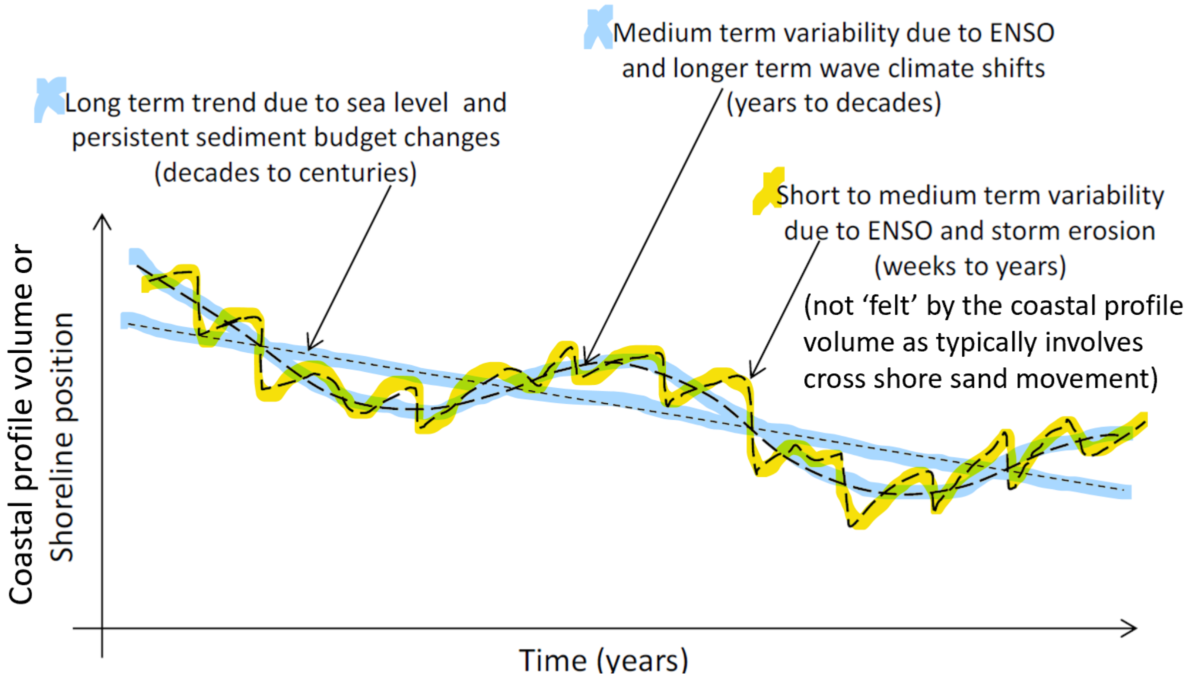

Figure 16. Conceptual illustration of time scales for beach changes (adapted from BMT, 2013).

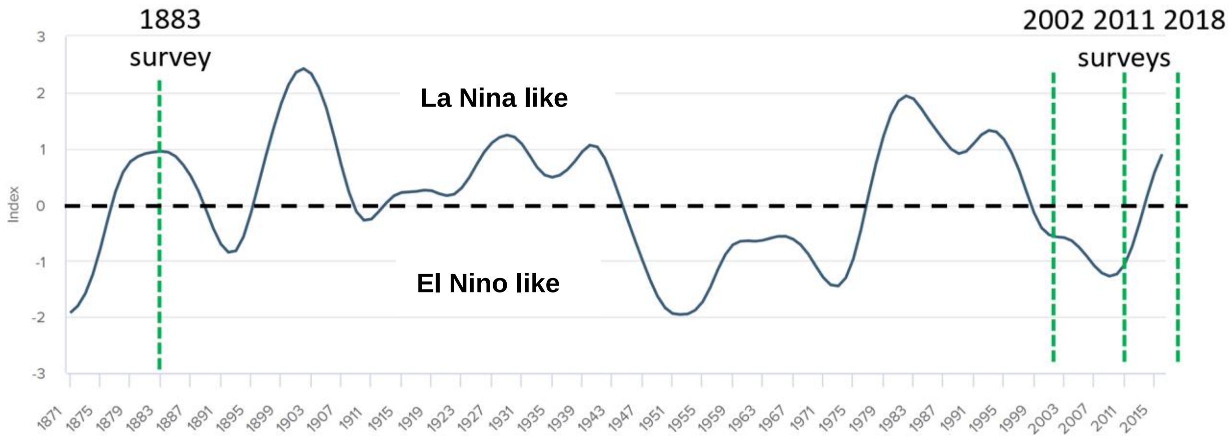

Figure 17. Annual average Interdecadal Pacific Oscillation index, 1871 to 2016 (adapted from Met Office).

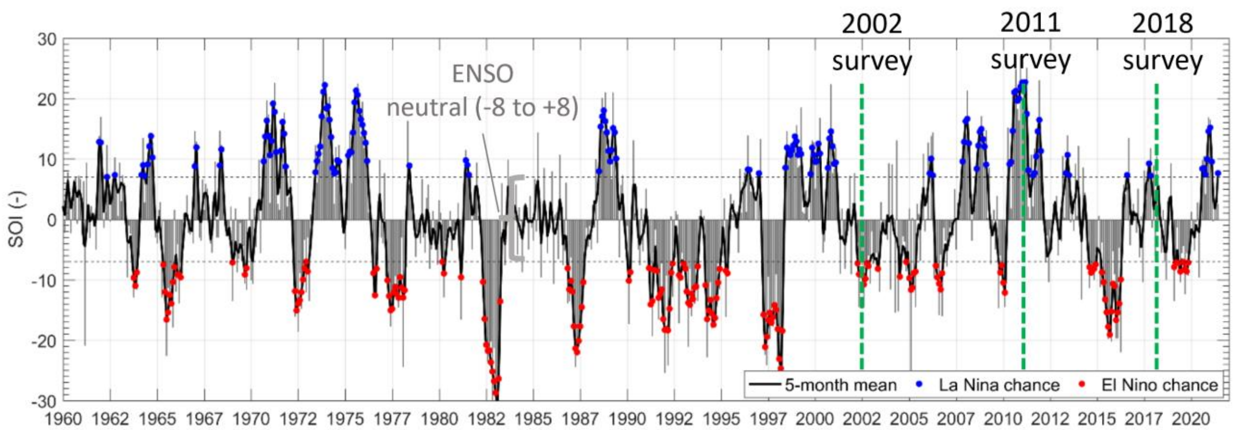

Figure 18. Monthly and annual Southern Oscillation Index (ENSO), 1960 to 2020.

3.3.2 Long term change

Over the 127 years from 1883 to 2018 a **net loss of sand at an approximate rate of 62,000m³/yr** ±20% was observed from the Byron embayment (Wategos Beach to Belongil Creek). This sand loss rate considers the coastal profile (from the top of the dune to -15m AHD) and is calculated using the volume changes between the 1883 survey and average of the 2002, 2011 and 2018 surveys using a 127-year period (i.e., around 8 million cubic metres in total surveyed sand volume change).

A long-term net loss of sand from the embayment agrees with the previous studies. The reported magnitudes of the loss rates are:

- PWD (1978) estimated an embayment sand loss rate of 65,000m³/yr based on gradients in their calculated longshore transport rates (i.e., 15,000m³/yr into the embayment by headland bypassing and 80,000m³/yr out to the north as longshore transport). The PWD (1978) estimates did not specifically consider the sand losses from the lower shoreface.
- BMT (2013) used photogrammetry data (i.e., subaerial beach above 0m AHD) to estimate an embayment sand loss rate of approximately 50,000m³/yr. A factor of 2.2 was assumed to extend subaerial volume changes to cover the full coastal profile. The scatter in observed ratios of subaerial, surfzone and lower surfzone profile change presented in Table 4 highlights the difficulty in determining coastal sand volume changes using subaerial beach data. However, the observed average factor considering the upper profile only would be 4.3 but larger if the lower shoreface was also considered.
- Goodwin (2013) used a similar survey analysis method as reported herein and obtained a long-term sand loss rate of 30,000m³/y from the embayment. Despite the similar methods employed the difference in the magnitude is likely due to (i) different analysis extents (Goodwin did not include the subaerial beach in the 1883 survey, had slightly different longshore extents) and (ii) Goodwin excluded any seabed changes less than or equal to the vertical survey uncertainty.

About half (49%) of the embayment's long-term sand loss has occurred from the lower shoreface (above 15m water depth). There are significant further sand losses that appear below the 15m. Considering the area between the 15m and 20m depth contours a further 4.9 M m³ of sand loss is observed since 1883, or an additional 39,000m³/yr (see volume changes **Appendix C**). These volumes are not considered in the loss rate of 62,000m³/yr.

Most of this lower shoreface sand loss comes from the southern part of the embayment (BB-LS-1). In the 1883 survey this area was shallower than in contemporary surveys. It is reasoned that in the 1883 era the lower shoreface of the Byron embayment was more generously supplied with littoral sand bypassing the Cape under SOI neutral wave direction and significantly higher mean wave heights (Goodwin, 2013). Since 1883 reduced supply and a continuation of the slow onshore migration of sand across the embayment has seen this area deepen by an average of 1.5m. A further 1.3 M m3 (or 33%) of the sand lost from embayment's lower shoreface has occurred from the northern area (BB-LS-2).

Table 5 presents annualised sand loss rates from various zones in the Byron embayment. It is reasoned that the losses from the lower shoreface, particularly in the southern embayment, have been supplying the embayment's upper shoreface and beach face. It is therefore difficult to relate the sand loss rates to observed shoreline recession rates. Other factors that influence any useful comparison are the JSPW and other coastal structures along Belongil Beach. However, in the most recent analysis shoreline recession appears to be present along the northern embayment (Bluecoast, 2022b).

The sand loss rate from the upper profile (above -10m AHD) is 31,500m³/yr ±20%. That is about half the long-term sand loss occurred above -15m AHD.

Table 5. Annual sand loss rates from various zones in the Byron embayment.

Profile zone	Annualised sand loss rates (1883 to contemporary) [m^3/yr]	
	Southern embayment (% of sand loss rate)	Northern embayment (% of sand loss rate)
Upper profile, above -10m AHD (beach, surfzone and lower surf zone)	11,000 (18%*)	20,500 (33%*)
Lower shoreface (-10m AHD to -15m AHD)	20,500 (33%*)	10,000 (16%*)

Note: * The percentage contribution from each zone is calculated as the proportion of the embayment's total sand loss rate or 62,500m^3/yr ±20%.

3.3.3 Contemporary (2002 to 2018) change

Between 2002 and 2018, the observed amount of sand within the upper shoreface and subaerial beach of the Byron embayment shows considerable variation (see Figure 19 and Table 4). Sand volume changes were most pronounced in the southern embayment's upper profile, for example in:

- 2002 the southern embayment contained similar amounts of sand than in 2018 (slightly less, -13,500m^3 when the beach face volume missing from the 2002 survey extents is estimated from photogrammetry)
- 2011 the southern embayment contained approximately 700,000m^3 **more** sand than in 2018.

The surveys represent snapshots in time, they indicate that amount of sand stored in the southern embayment's upper profile (above the 10m depth contour) can vary by at least 0.7 million m^3. As discussed further below this is a result of the higher variability in the headland bypassing supply.

As shown in Figure 19, the subaerial beach and upper shoreface sand volumes along the northern embayment are less variable, ranging by up to 400,000m^3 across the contemporary surveys. This lower observed variability may demonstrate the southern embayment's ability to provide a more consistent supply of sand to this northern zone (i.e., the southern embayment acts to buffer the northern parts of the embayment from the variable Cape bypassing supply). This effect can be further inferred by the increase in sand volume in the northern embayment between 2011 and 2018 with a corresponding decrease in the sand volume over the same period in the southern embayment. Longshore transport rates out of the southern embayment are higher when the compartment is full of sand covering the various nearshore reefs and the shoreline is accreted (see Figure 19) allowing uninterrupted bypassing of the JSPW.

Despite the considerable variation the contemporary sand volume changes show a stabilisation or volume increase in the case of the northern embayment.

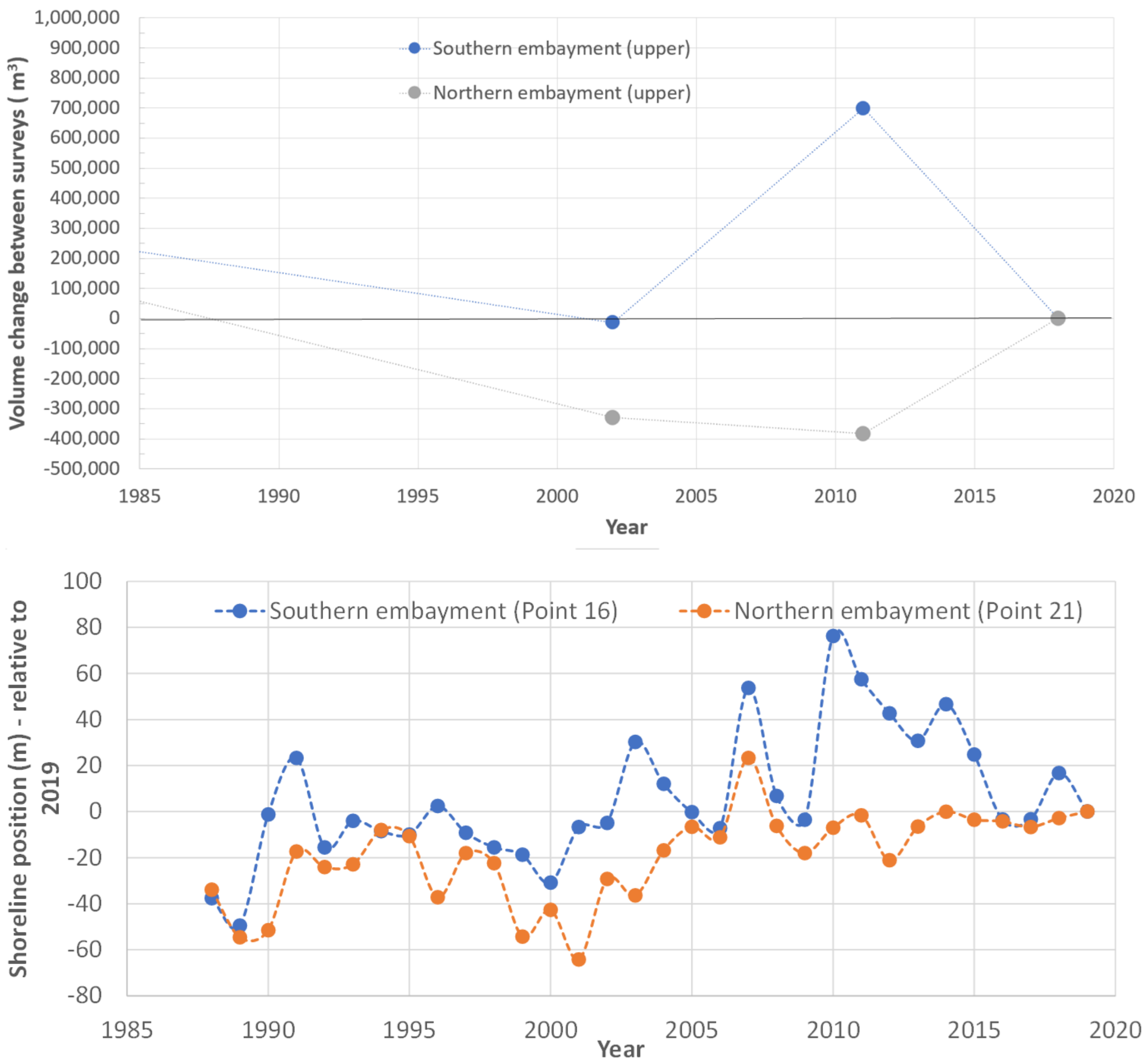

Figure 19. Contemporary Byron embayment sand volumes (top) and relative shoreline positions (bottom).

3.4 Net longshore sediment transport rates

Driven by wave action, longshore sediment transport (LST) occurs predominately in the mid- to outer surf zone and normally inshore of the -4m depth contour. The dominant south-easterly offshore wave climate is oblique to the north-south coastline orientation driving a net longshore movement of sand to the north along the 'Cape Byron to Ballina' and 'Tweed' sediment compartments. While the alongshore sediment transport may be directed either north or south depending on the prevailing wave direction, in the Byron region the net sediment transport direction is to the north.

Longshore transport gradients are the dominant factor in the sand budget and shoreline changes in the region (BMT, 2013). However, there are no known measurements of LST rates in the region and previous studies present a wide divergence of estimates. The analysis of Patterson (2007; 2010; 2013) are considered the most recent and comprehensive undertaken in the region. Patterson's used directional wave records, wave transformation modelling and longshore sand transport calculations to determine a gradient in the net longshore sand transport rate from about 150,000-200,000m^3/yr at the Clarence River to about 550,000m^3/yr at the Gold Coast. Other data-driven studies provided rates that were in general agreement with Patterson's calculated rates:

- Goodwin et al. 2013 used survey analysis estimate a net sand transport bypassing Cape Byron into the downdrift Byron embayment to be at least 350,000m³/yr (±20%) between 2002 and 2011 surveys.
- Bluecoast (2022a) used sand pumping and dredging volumes and 11 full coastal profile surveys between 1972 to 2021 to calculate a net sand transport rate of approximately 560,000m³/yr near the Tweed Sand Bypassing pumping jetty at the northern end of Letitia Beach.

While the longshore transport rates from Patterson 2013 are considered the most reliable and largely adopted herein, they are also noted as being many times greater than those presented in the PWD (1978) study. PWD (1978) used field measurements of progradation following construction at the Brunswick River training walls to estimate that the net longshore transport rate at this Byron region location was 110,000 to 120,000m³/year. WRL (2011) considered the substantially higher rates adopted by Patterson (2010) warrants clarification and/or additional studies. BMT 2013 superseded WRL 2011 review and as part of Stage 2 of the CMP, LST rates are currently being reassessed using a sand budget approach.

PWD (1978) reported a 49,000m³/yr component of the longshore sand transport is lost at Cape Byron because of bifurcation of the headland bypassing caused by the shelf current. The component transport downslope by the southward flowing East Australian Current (EAC) was calculated as the volume of sand in excess to the normal offshore coastal profile in the lobe region and dividing this by the number of years (6,000) which it has been depositing (Roy and Stephens, 1978). BMT 2013 adopted this value of 50,000m³/yr and included it as a loss rate in their alongshore sand movement and shoreline change modelling.

The rates of longshore sand transport and along coast gradients adopted for this study are provided in Table 6. Of key interest to this study is the rate of LST bypassing Cape Byron, which is highlighted above.

Table 6. Adopted net longshore transport rates.

Location	Net longshore transport rate (m³/yr) and uncertainty	Source
Clarence River	150,000	Patterson (2013)
Northern end of Tallows Beach (updrift of Cape Byron)	400,000 to 470,000 (±20%)	Patterson (2013) and Goodwin et. al (2013)
Bypassing Cape Byron	**350,000 to 390,000 (±20%)**	Patterson (2013) and Goodwin et. al (2013)
Belongil Beach	330,000 to 370,000 (±20%)	Goodwin et al 2013
Tweed River sand pumping jetty	560,000 (±25%)	Bluecoast (2022)

LST rates are highly variable responding to variation in the direction and energy in the offshore wave climate, which is sensitive to ENSO and other climate cycles of years, decades and longer timescales.

Typically, during La Niña events waves along northern NSW are bi-directional with southeast and easterly wave conditions. El Niño events are associated with a unidirectional south easterly wave climate (Mortlock and Goodwin, 2016). This wave climate variability, particularly the wave obliquity but also wave energy, largely controls the magnitude and direction of longshore sand transport along the study areas coast and headland bypassing around Cape Byron (da Silva, 2021a). The alignment of the beach, if therefore important when considering LST rates and how ENSO effects these. For example, along Tallows Beach high rates of LST would be expected in El Niño events being driven by a more southern wave climate. Whereas in the southern embayment the higher energy and more eastern waves during La Niña events would be expected to drive higher LST rates.

Climate change is also likely to influence LST rates and their variability. The expansion of the tropics with warming climate is expected to lead to a poleward shift in storm type, with more tropical origin storms than extra-tropical storms with a southern origin. The anticipated outcomes of these changes on the Eastern Australia wave climate would be an anti-clockwise rotation of the mean wave direction and associated changes to sand movement (Silva et al., 2021). The mean wave height offshore of the Gold Coast, just north of the study area, is expected to decrease as well as an anticlockwise rotation of around 5° in the mean wave direction (GCCM, 2020). Such a shift would be expected to reduce net northern LST along Tallows Beach but could increase potential net northerly LST rates in the southern embayment.

3.5 Headland bypassing, embayment sand volumes and shoreline behaviour

Cape Byron is the most easterly point on mainland Australia and the most prominent headland in the two adjacent sediment compartments. The Cape has a significant influence on net northward littoral sand movements. Sand moving around Cape Byron, a process referred to headland bypassing, influences the supply of sand to the embayment as well as the way sand moves through the embayment.

Recent insights into headland bypassing are provided by the work of Silva et al. (2021a) who undertook a detailed assessment of sand movements around Fingal Head. Fingal Head is 50 kilometres to the north of Cape Byron and within the 'Tweed' sediment compartment. Using repeat hydrographic surveys and aerial imagers, the study identified two distinct headland bypassing processes:

- Sandbar-driven bypassing related to high-energy wave events. Between June 2018 and January 2020 hundreds of thousand cubic metres of sand was observed moving around Fingal Head by sandbar-driven bypassing during Tropical Cyclone Oma.
- Sand leaking around the headland following persistent low energy wave conditions and widening of the updrift beach (i.e., pre-loading of the apex) eventually resulted in sand leaking around the headland.

Sand supply to the Byron embayment is controlled by variations in the offshore wave climate which results in intermittent headland bypassing of slugs of sand around Cape Byron. Cape Byron is a more prominent headland in comparison to Fingal Head, and while the sand leakage process/pathway may also occur here the sandbar-driven process observed by Silva et al. (2021a) can also be observed at Cape Byron.

This is best demonstrated by comparing two recent high resolutions surveys:

- 2011 LiDAR survey (see Figure 9) conducted over Byron Bay region in June and July of 2011. The 2011 survey was conducted during an extreme La Niña year.
- 2018 LiDAR survey (see Figure 10) which captured the entire NSW coastline in around August and September 2018, an ENSO neutral year. The survey was captured around 6 months after the passage of Tropical Cyclone Oma.
- The 2002 single beam hydrographic survey (no very shallow water or subaerial beach) was captured under similar wave climate conditions to the 2018 survey.

The difference between these two surveys is shown in Figure 11. The morphology captured in the 2018 survey shows a post-storm or wave energy condition. A prominent storm bar is observed along the northern end of Tallows Beach which extends north around Cape Byron. A large sand deposit is observed off Little Wategos Beach having bypassed Cape Byron. The outer bar at Tallow Beach is much less pronounced in 2011 and there is less sand on the shoreface along Cape Byron.

Headland bypassing is highly variable with the annual range of sand supply around the Cape estimated to be from around 150,000 to over 900,000m^3/year. Table 7 provides a summary of the sand volume changes, relative to 2011, around Cape Byron and northern Tallow Beach. These areas, as noted in Table 7, define the headland bypassing pathway around the Cape (see Figure 11 and Figure 12). Relative to 2011, the 2002 and 2018 surveys captured 684,000 to 1.38M m^3 more sand along this bypassing pathway. This provides insight into the quantity of sand in the slugs that bypass the Cape.

Large scale climatic variability linked to ENSO, PDO and IPO has been shown to result in interannual to decadal differences to the frequency and magnitude of headland bypassing. Most pronounced, extended periods of La Niña dominance (several years) would be expected to result in upper beach erosion at Tallows Beach, reducing the sand availability for sand bypassing around Cape Byron. At the same time, high energy wave events during extreme La Niña periods also arrive from a more easterly wave direction, reducing/increasing the northward longshore transport potential due a reduced wave obliqueness in respect to the coastline orientation and exposure. Conversely, extended El Niño dominance results in the opposite effect.

While a reduction in headland bypassing may be triggered by a La Niña event, or series of events, the erosive effect on the embayments shorelines may not be apparent for two to five years' time. Similarly, it takes time for sand slugs in a large bypassing event to move into the embayment and supply the beaches (e.g., Clarkes Beach and Main Beach and later still Belongil Beach) with sand.

Table 7. Sand volume changes along the Cape Byron sand bypassing pathway.

Alongshore area	Surfzone cell	Sand volume change (m^3)		
		2002 relative to 2011	2011	2018 relative to 2011
Northern Tallow Beach surfzone	TB-5-2	48,000	0	404,000
Cape Byron surfzone	CaB-1	223,000	0	414,000
	CaB-2	175,000	0	114,000
	CB-BYPASS	238,000	0	452,000
Total bypassing slug potential volume (m^3)		**684,000**	na	**1,382,000**

In both the 2018 and 2002 survey, the southern embayment from Wategos Beach to JSPW has lower sand levels and is generally lacking in nearshore morphological features (refer to Figure 10 for 2018 survey). Conversely, the 2011 survey captures a period when the southern embayment contains significant quantities of sand within the littoral transport pathway or surf zone in water depths of 4m or less. As described above in 2011 the southern embayment contained approximately 700,000m^3 **more** sand than in 2018 and 2002 surveys.

When coupled with the wave propagation characteristics of the embayment, the variable sand supply leads to a highly variable shoreline in the southern embayment. This is illustrated in Figure 20 which shows DEA Coastlines in the embayment. These are annual mean sea level shorelines since 1988 shown alongside ‘boxplots’ illustrating the higher degree of variability observed in the southern embayment shorelines (east of JSPW) compared to the northern embayment.

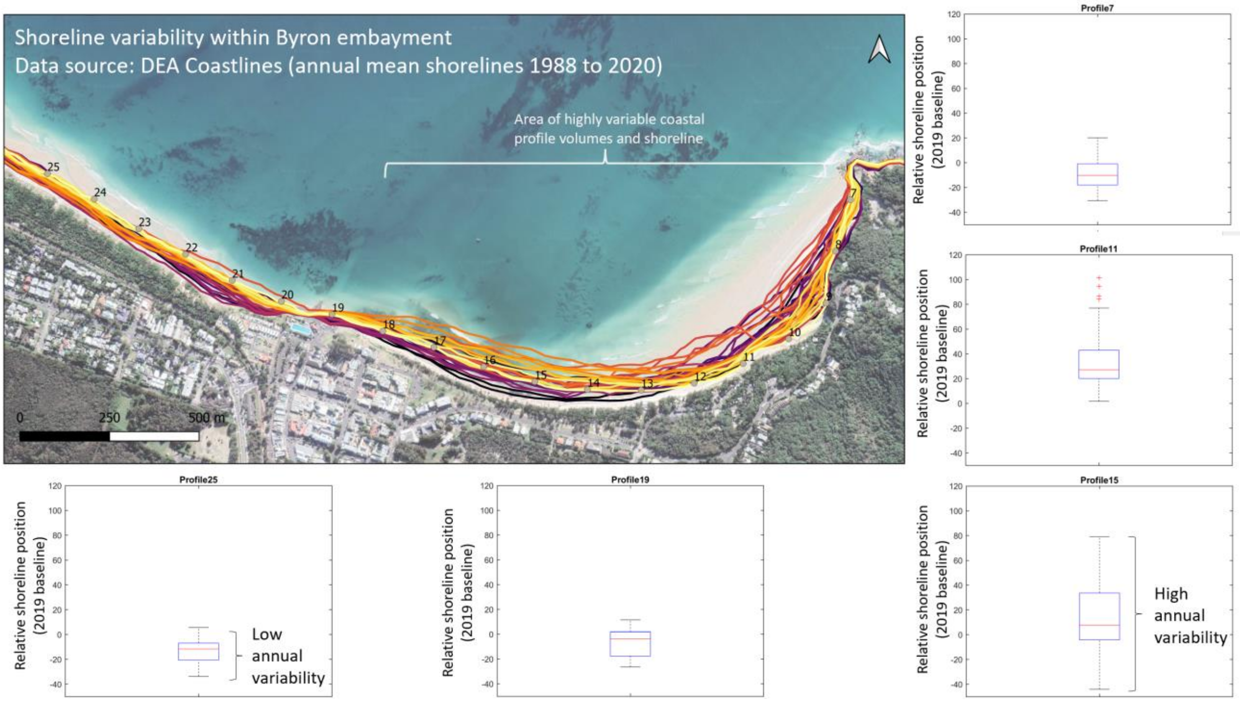

Figure 20. Byron embayment shorelines and box plots showing variability along embayment.

At times of reduced bypassing and embayment sand supply, the northward movement of sand out of the embayment persists resulting in a deficit of sand at the southern end (i.e., more sand moving north out of the southern embayment than is being supplied from the headland bypassing around the Cape). This situation is more likely to occur in more easterly high energy wave conditions typical of La Niña, which reduce headland bypassing but increase littoral transport along the embayment shoreline, thus causing erosion of sand and shoreline recession in the southern embayment. Conversely, pulses of high sand supply from persistent south to southeast sector or higher than average mean wave heights associated with ENSO neutral or El Niño phases, are likely to result in increased headland bypassing. However, more southerly waves are strongly reduced in height in refracting around Cape Byron and are associated with reduced littoral zone alongshore transport along the southern embayment shoreline. This leads to a tendency for shoreline accretion there due to the surplus supply relative to the losses to the north.

The NSW Beach Profile Database provides ‘snapshots’ of the beach profile above 0m AHD since about the 1940s. These snapshots are derived from photogrammetry, LiDAR and other surveying techniques and allow insight into the shoreline and subaerial beach behaviour. A recent review of this data was undertaken as part of the Byron Open Coast CMP investigations (Bluecoast, 2022b). In agreeance with BMT (2013), the review suggests that data quality issues and the influence of sand mining (mostly stopped around the late 1960s) is evident but only in the pre-1970s beach profiles. Similarly, large profile variations due to storms in the late 1960, 1970’s and 1999 occurred and may take years to recover. Therefore the identified changes can sometimes be misleading and may not be representative of shoreline recession processes. Linear regression analysis was undertaken to derive long-term rates of subaerial beach volume change for three analysis periods, i.e.:

- 1940 to 2021 (81 years) – full data period
- 1970 to 2021 (51 years) – post sand mining and 1960 storms
- 1980 to 2021 (41 years) – post construction of the Jonson Street Protection Works (JSPW)

Figure 21 shows the rates of subaerial beach volume change along the Byron embayment for the three analysis periods. The outcomes most relevant to the MBSP are:

- In agreement with the sand loss trend identified in the sand budget analysis, the embayment shows shoreline recession.
- Sand mining influences aside, the 1940 – 2021 analysis suggests that recession rates have reduced in the southern embayment but generally increased in the northern embayment.
- Comparing 1970 to 2021 and 1980 to 2021 this same trend is seen (i.e., shoreline stabilising to the west of JSPW, reducing recession rates along Belongil Beach (Block 6) and increasing recession rates downdrift of the coastal structures that terminate at the northern end of Childe Street at around chainage 5,000m).

The influence of the JSPW on coastal processes and shoreline behaviour is discussed further in Section 3.8.

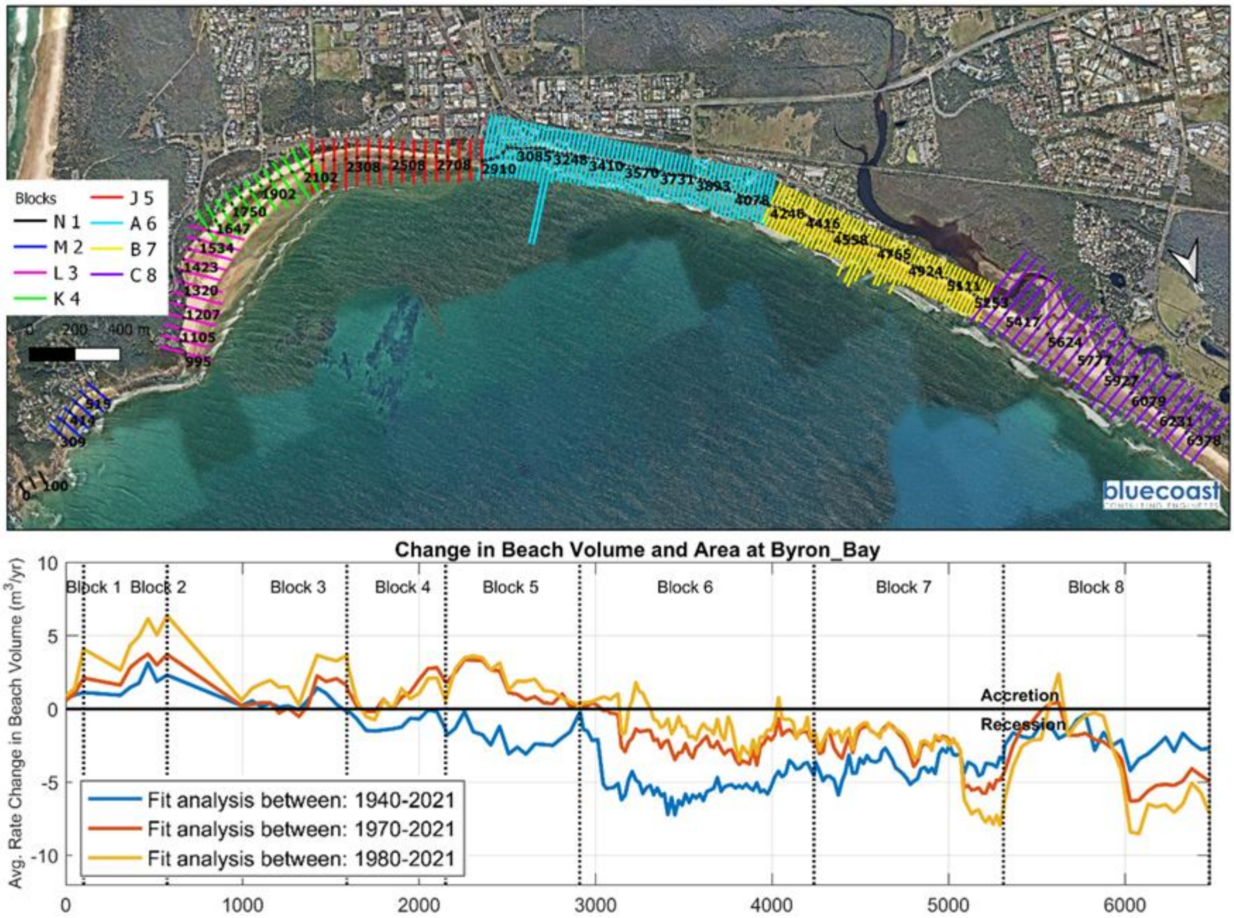

Figure 21. Subaerial beach volume changes showing areas recession/accretion rates for Byron embayment.

3.6 Embayment sand movement pathways

Two distinct Byron embayment sand movement pathways have been identified in previous studies (BMT, 2013 and Goodwin, 2013). These two distinct, but related pathways are defined by the different mechanisms that drive the sand transport as:

- A littoral pathway limited to the surfzone where sand movements are forced by obliquely breaking waves and the longshore currents they drive. The littoral processes are confined to depths not much greater than the wave breakpoint depths, or about 4 to 5m water depths within the embayment. Herein the littoral pathway has been defined based on the toe of steeper beach faces found in the embayment at around 4m water depth, see Figure 22.

- The cross-embayment transport which extends beyond wave breaking depth to up 15m water depth and reported to be driven by wave asymmetry, wind and wave radiation stresses (BMT, 2013). As indicated in Figure 22, the cross-embayment pathway extends from the 4m water depth. Sand movements in these water depths are typically much slower.

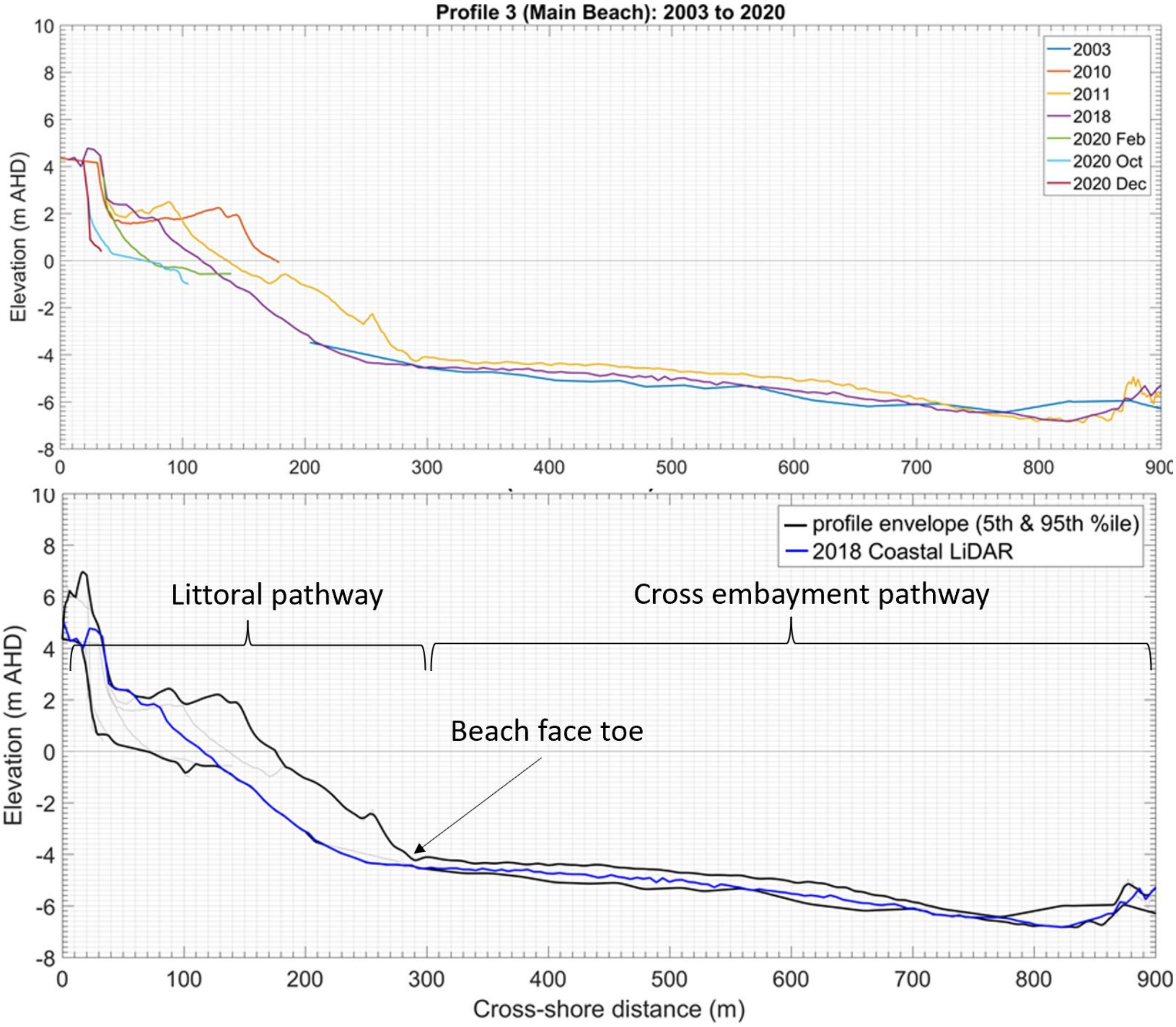

Figure 22. Coastal profile at Main Beach (top) with envelope of profile variation between 2003 and 2020 (bottom).

The proportion of sand movements that follows each pathway, as well as where along the northern shoreline the cross-embayment pathway re-joins the littoral pathway, is a relevant consideration for the MBSP as it influences:

- the net longshore sediment transport rate bypassing the JSPW
- the nature of the sand supply to areas downdrift of the JSPW.

BMT (2013) and Goodwin (2013) state that the relative proportions have not been quantified reliably. In BMT (2013), a 50 : 50 split was based on Goodwin's results and used as an input to their shoreline model, where this assumption was found to produce good results. Based on sand volume changes determined from repeat surveys and presented in Section 3.3 as well as profile changes (e.g., Figure 22) the relative split between the two pathways, when averaged across the embayment, has been calculated to be 70 : 30 (littoral : cross embayment). As discussed below this split is not uniform across the embayment with sand progressively moving onshore and re-joining the littoral pathway.

Qualitatively, the dominance of the littoral pathway, is reinforced by a review of recent aerial photography, see Figure 23. Inspection of these images in the context of other data described herein revels:

- The July 2018 aerial was captured around the same time as the 2018 Coastal LiDAR survey. It shows the headland bypassing pathway loaded with sand but very little sand on Wategos Beach and a well below average amount of sand along Clarkes and Main beaches.
- By June 2019 sand bypassing the Cape appears to have moved along the cross-embayment pathway and onshore and is starting to weld to the shore to fill Wategos Beach and The Pass. Clarkes Beach has eroded as evident by the narrowing beach and nearshore reefs becoming exposed.
- By July 2020 the slug of bypass sand appears to have fully welded to the littoral pathway with Little Wategos and Wategos Beaches very wide and almost continuous having formed in front of rocky areas. A large sand spit has formed out from The Pass. However, further erosion of Clarkes Beach is evident with more reef exposed as the sand level lowers.
- By July 2021 the bulk of the bypassed sand appears to have reached Clarkes Beach with some sand in the surfzone moving all the way to JSPW. The ‘bulge’ shape of the surfzone pathway is seen around the western end of Main Beach.

The aerials of this bypass event indicate it took around 3-years from the time when the sand slug bypassed the Cape for the sand to start filling in the eastern end of Main Beach. More recent aerials show it took another 12-months for the sand to infill Main Beach all the way to the JSPW. While these timeframes were indicative over this period, it is important to note that rate of sand movements through the embayment are influenced by the wave climate encountered over a given period. Large waves events, such as can be generated by tropical cyclones, and their associated storm wave direction can also result in strong pulses of alongshore sediment transport.

Further evidence in support of a higher proportion of sand movement along the littoral transport comes from current speed data collected in the Byron embayment:

- PWD 1979 reports current speeds and alongshore direction of surf zone currents (assume to be less than 4m water depth relative to AHD). Over a 4-month period in 1977 a location in the southern embayment was monitored with an average surf zone current speeds of 0.3m/s (predominately, ~90% of the time, northward flowing) and a peak speed of 0.9m/s.
- High-quality current speed measurements in approximately 6m water depth relative to AHD were made as part of the MBSP (see **Appendix A**). These deeper measurements, located outside the surf zone (littoral pathway) and instead within the cross-embayment pathway, show slower currents. An average speed of less than 0.1m/s and 90th percentile speed of less than 0.2m/s were recorded. It is noted that the maximum near bottom speed was 0.8m/s but that this was recorded during an extreme tropical cyclone event.

The cross-embayment pathway is likely to have a higher proportion east of The Pass (i.e., Wategos and Little Wategos Beach), but within the inner embayment the evidence suggests the littoral pathway is dominant. Fisherman’s Lookout at The Pass acts as a second inner embayment headland, inside of which incoming waves approach also at right angles to the shoreline with wave crests bending by refraction and diffraction before breaking along the sand bank. This high angle of wave obliquity drives relatively high LST rates despite the lower wave heights and it is common to see recurved spits form, stretching out towards Clarkes and Main Beach with lagoons forming on the inner beach berm. Sediment transport equations like the type used in the BMT (2013) modelling do not adequately resolve the transport rate on these high angle coastlines.

From the bathymetry evidence, BMT (2013) concluded that the alongshore sand transport becomes exclusively ‘littoral’ somewhere at or north of Belongil Creek.

Figure 23. Aerial imagery showing the movement of sand through the southern embayment via the littoral pathway (data source: Nearmap).

3.7 Geomorphology, reefs and wave transformation

Geological and seabed characterisation mapping for the Byron embayment is shown in Figure 24. This map as well as historical and recent observations confirm the embayment has extensive indurated sand (or coffee rock) lenses and bedrock outcrops including Julian Rocks and Middle Reef. These hard features affect wave transformation and the movement of sand through the embayment as well as influencing shoreline dynamics. An example is provided in Figure 26 showing the 2018 less 2011 survey differences alongside a wave height map from a SWASH model simulation (see Section 7). A wave shadow between two ‘streaks’ of higher waves emanating from Middle Reef is seen to coincide with an area of nearshore seabed change in the surfzone just east of the JSPW (as seen as an accumulation of sand in the 2011 survey which had move on (eroded) by the time of the 2018 survey). This suggests that the nearshore reefs influence the surfzone morphology when the southern embayment cells are full of sand.

Although not quantitative, a review of historical aerial photography reveals this same pattern (i.e., ‘bulge’ nearby JSPW when southern embayment full of sand), including aerials taken prior to the introduction of the groynes (see Figure 27). The August 1971 image shows a peeling wave breaking in this area, which when considering the other evidence, indicates that the intermittent good surfing conditions that are known to occur here are likely to be largely related to the wave pre-conditioning (owing to Middle Reef), the distribution of surfzone coffee rock and this resulting ‘bulge’ morphology. The JSPW and the central groyne are considered to have a lesser effect on the intermittent good surfing conditions.

Seismic data from the 1970s suggests that bedrock lies at shallow depths beneath the seabed surface (PWD, 1978), which is validated somewhat by recent aerial images that show reefs within the embayment intermittently exposed and then covered with sand (see Figure 25). Hard substrate also reduces the volume of sand that can be stored in the southern embayment.

From the Cape to Clarkes Beach the shape of the embayment’s shoreline is controlled by the greywacke bedrock that forms Cape Byron. Little Wategos, Wategos, The Pass and Clarkes Beach (east) are underlaid by the bedrock and boulders, cobbles and gravels that have been weathered off. Further to the west, the embayment’s backbeach area becomes a Holocene beach barrier system comprised of marine sand deposits. At various locations along Main Beach and Belongil Beach coffee rock in the dunes and swash zone can be exposed at times of erosion and low sand levels.

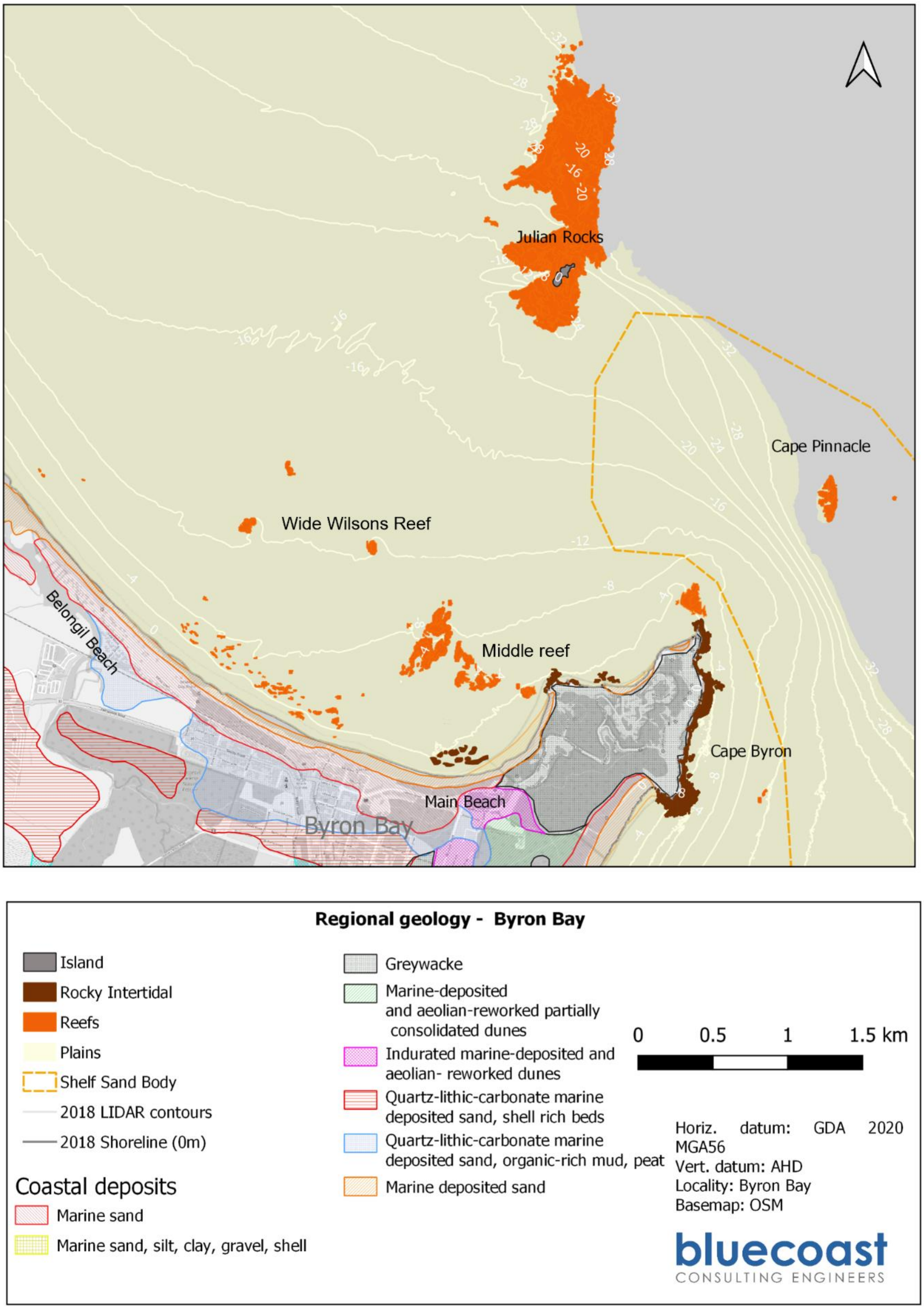

Figure 24: Byron Bay regional coastal Quaternary geology and seabed characterisation map.

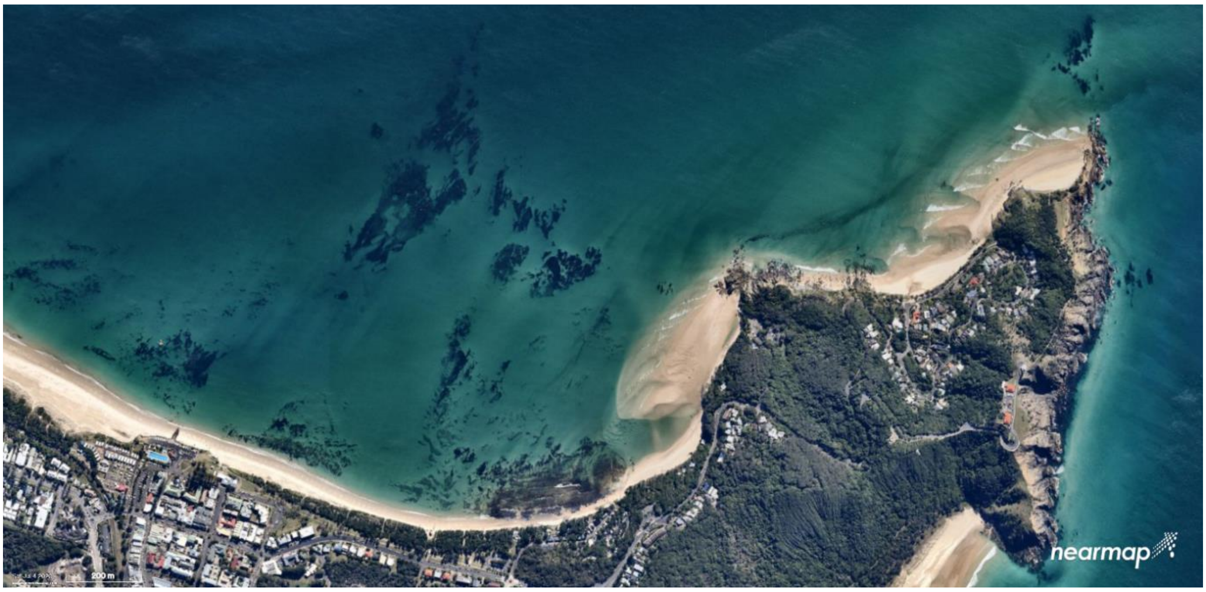

Figure 25. July 2020 aerial photography showing reefs and coffee rock outcropping (source: Nearmap).

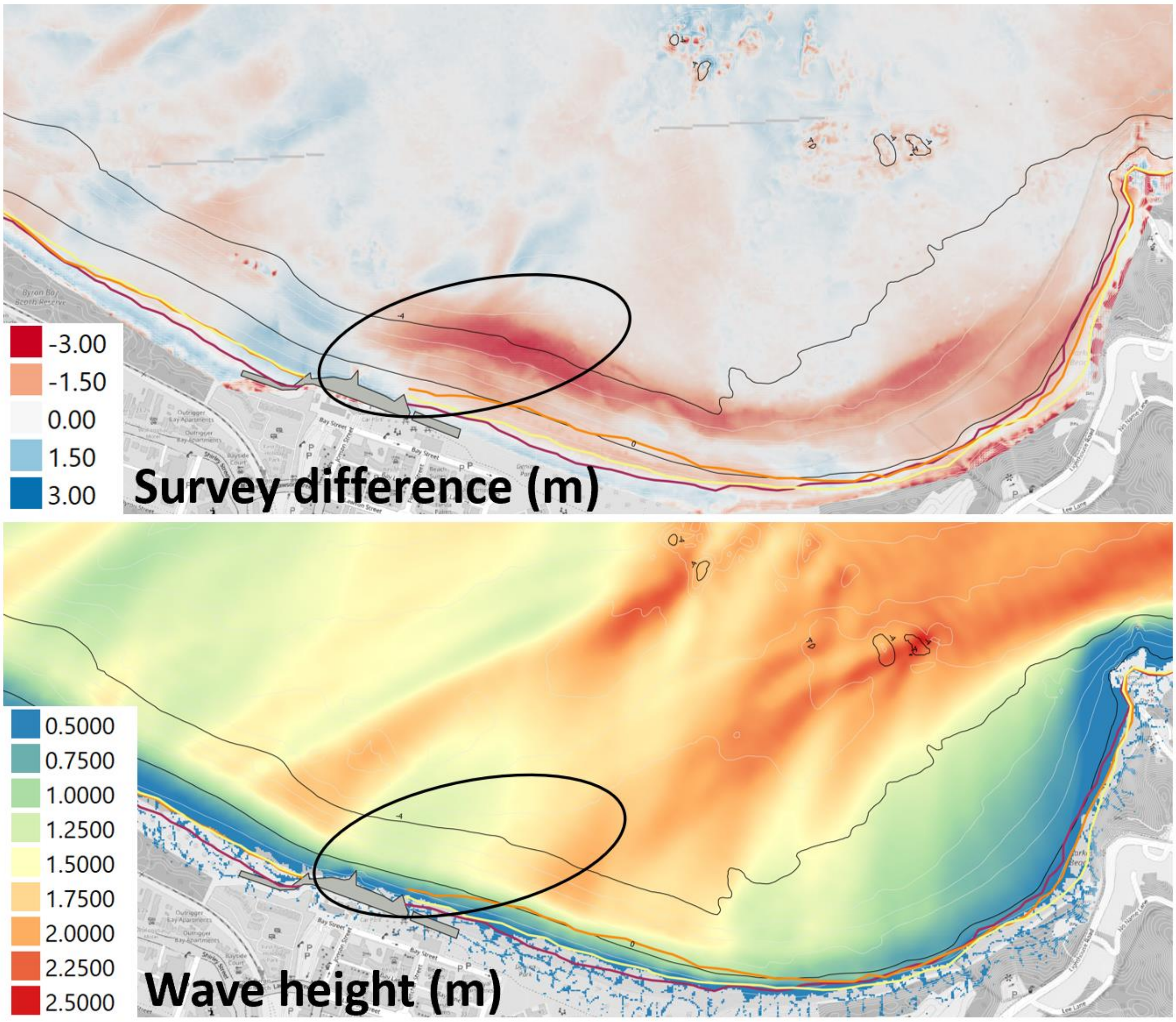

Figure 26. Survey difference 2018 less 2011 (top) and SWASH wave height map (bottom).

Figure 27. Two historical aerial images highlighting the bulge of sand in the lee of Middle Reef wave shadow and nearby JSPW.

3.8 The JSPW and their interactions with the embayment's shoreline

A comprehensive history of the Jonson Street Protection Works (JSPW), inclusive of photos, drawings and other information, is provided in the MBSP's Baseline Understanding Report (Bluecoast, 2021). For this report it is useful to provide a summary as laid out in Table 8.

Table 8. Brief history of the JSPW and other Belongil coastal protection works.

Dates	Timeline event
1888	The Public Works Department (PWD) built a 402-metre-long timber jetty at the end of Jonson Street at the site of the JSPW (the JSPW central groyne conceals some of the old timber piles).

Dates	Timeline event
1925	Wing abutments are constructed on the old jetty including the placement of '4-inch sheathing' or rock protection (using 10cm diameter rocks).
1928	The old jetty is removed and replaced with a new jetty at Belongil Beach.
1963 - 1964	Around 1963 and in response to heavy seas and erosion, temporary coastal protection works (rock placements) were undertaken at the end of Jonson Street. In 1964, in response to severe erosion caused by tropical cyclone Audrey the first engineering drawings of what later become known as the JSPW were produced. The construction of the design saw the main section of the JSPW rock revetment built on its current alignment.
1976 - 1977	The JSPW were upgraded with the construction of three groynes and restoration of sections of the pre-existing rock revetment. A series of ad-hoc rock seawalls were built between the former Walkers Meat Works (Border and Kendell Street) and Manfred Street. These works were initially piecemeal in nature, protecting individual houses along Belongil Spit (PWD, 1978). They have been progressively infilled over the years with geotextile container (geobag) and rock seawalls (around the 1990s). The rock seawalls now extend to the end of the private properties along Belongil Spit at the northern end of Childe Street.
1990s	Extension of the JSPW to include emergency protection works at Cavanbah Beach comprising of two rows of boulders laid at the toe of the embankment at First Sun Holiday Park, believed to be installed in the late 1990's. Geobag coastal protection structure in front of the (western portion) of First Sun Holiday Park were constructed shortly thereafter.
2002	As an interim coastal protection measure, a geobag revetment was constructed in front of the present-day Byron Surf Life Saving Club (SLSC). Belongil interim stabilisation works (geobag revetments) were constructed along sections of Belongil Beach backed by public land including at Manfred, Don and Border Street.
2015	Replacement of Manfred Street geotextile container revetment with interim rock wall.

In the context of the contemporary coastal environment the dimensions and alignment of the JSPW are an important consideration. Figure 28 provides an elevation map of the structure from a December 2020 drone survey when beach levels were low on the eastern side of the structure. This is overlaid on the 2018 Coastal LiDAR contours. Also provided in this figure is an aerial photomosaic, combining December 2020 and July 2022 images to show the structure in an uncovered state (i.e., not buried under beach sand/vegetation).

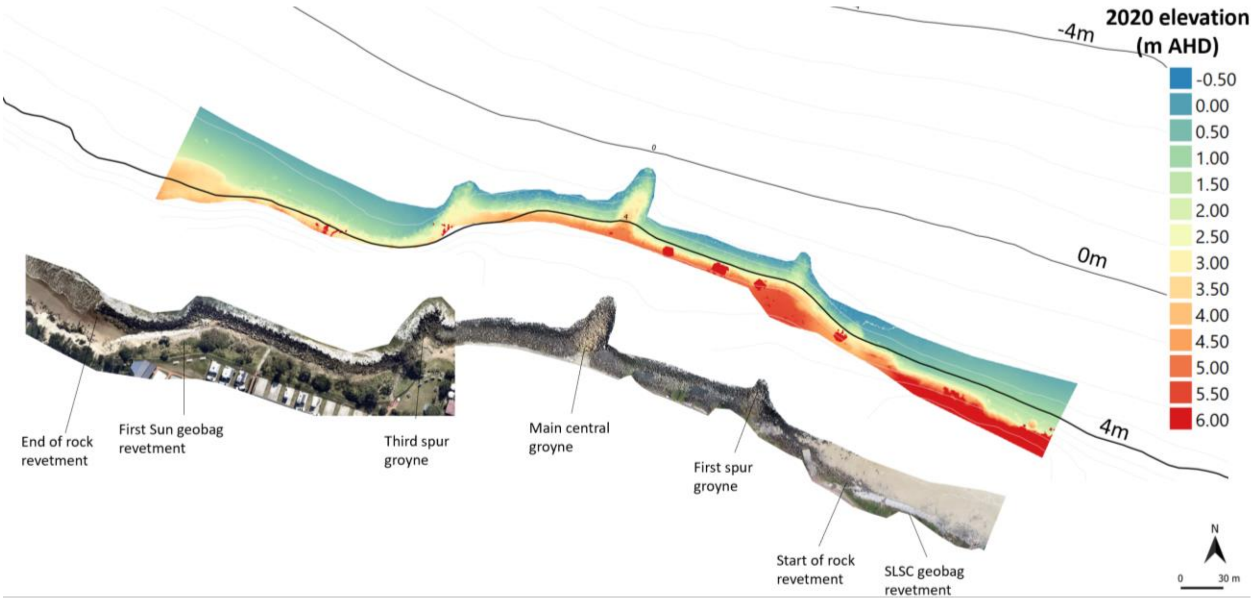

Figure 28. Elevation model (top) of JSPW taken in December 2020 along with 2018 contours and aerial mosaic combining December 2020 and July 2022 images.

Table 9 provides a summary of the key geometrical features of the JSPW deemed significant in influencing shoreline processes. The main central groyne is the most seaward part of the structure. The main parameters that influence the response of the shoreline to the groyne is its permeability and the bypassing ratio, which is the ratio between the water depth at the head of the structure D_s and the water depth of the active longshore transport D_{LT}. At the JSPW, the groyne tip's toe (D_s) is generally above mean sea level (as approximated by 0m AHD) meaning sand bypassing will vary substantially with tidal conditions (i.e., fully bypassing at low tide when the groyne toe is effectively dry, and pedestrians are able to walk around its seaward end). However, there are times, during high tides, large wave conditions and/or period of low beach levels, when the JSPW interrupt longshore sand transport and reduces the sand bypassing ratio below full bypassing. While not discussed further herein, it is noted that the condition of the structure, including the main groyne is poor (Bluecoast, 2021). Given the main groyne is in poor condition, has a loose rock core and tapers in width it would be expected to have a relatively high permeability.

Table 9. Summary of the main geometries of the JSPW effecting shoreline interactions.

Geometry	Dimensions
Alongshore length	Overall length: 506m Rock revetment: 452m Geobag revetments: 54m (SLSC) 47m (First Sun) – located behind rock revetement and therefore not counted in overall length.
Seaward projection of the rock revetment from the adjacent shoreline	Approximately 30m when measured from an assumed 'natural' shoreline which in the absence of the structure is estimated to adjoin the 4m contour on either side of the structure.
Length of main central groyne	Approximately 30m seaward of the alignment of the main rock revetment.

Geometry	Dimensions
Beach level at the tip of the central groyne (D_s)	Varies but generally around 0m AHD or just above. The beach level at the toe of the tip of the groyne, as taken from surveys was: • 2011: 0.29m AHD • 2018: 0.22m AHD • 2020: -0.06m AHD

The effect of the JSPW on the adjacent beach compartments has been studied extensively with much written on the subject (PWD, 1978; NSW Government, 1990; WBM Oceanics Australia, 2000; WBM, 2004; BMT, 2010; BMT, 2013; WRL, 2011 and WorleyParsons, 2014). There is consensus in the literature that:

- the JSPW[4] do influence the planform of the adjacent beaches being an initial response of accretion/stabilisation of the updrift shoreline (i.e., Main Beach to the east) and additional erosion of the downdrift shoreline (i.e., Belongil Beach to the west)
- the wider Byron embayment was experiencing 'natural' erosion as shorelines receded due to net sand loss over the observed period.

There is no consensus in the literature on JSPW relative influence on planform changes against those of background recession nor on the cross-shore and alongshore extents of the JSPW induced erosion along Belongil Beach nor on the time scales of change. BMT WBM (2013) importantly notes that the embayment's shoreline and beach widths are largely controlled by the natural sand movement processes of headland bypassing, onshore/offshore transport, longshore transport combined with the effects of the influence of bedrock and indurated sands.

To further examine the influence of the JSPW on the embayment's shoreline, analysis of the relevant observational data is presented in Figure 29 to Figure 32. This includes recent data that was not available at the time of previous literature. Based on this further analysis the following key points are noted:

- During the early 20th century through to the 1950s beaches in the southern embayment were wide with a surplus of sand and expansive areas devoid of dune vegetation. Prior to the JSPW being present the area at the end of Jonson Street was occupied by the original clubhouse of the Byron Bay SLSC and a boat shed. Photo records show a wide expanse of beach and a sparsely vegetated dune seaward of the original clubhouse (see Figure 29).

  Beach profile (or photogrammetry) data at Main Beach (see Figure 30, Block 5, profile 10 as representative profile) shows a sharp decline in sub-aerial beach volume observed between 1940's and 1970. While it is noted that sand mining may well have influenced these observations, the sharp decline in beach widths/volumes concords with the photo records from the former clubhouse.

  The alignment of the main rock revetment of the JSPW was based on the desire to protect these two public buildings (Bluecoast, 2021). The original clubhouse was present in 1946 and from the

[4] Note that coastal structures do not result in a change to the regional sand budget (i.e., they do not introduce or remove sand from the system). Structures, particularly those that interrupt longshore transport, can redistribute sand with corresponding amounts of accretion and erosion adjacent to the structure. On coastlines suffering net sand loss, seawalls with significant longshore lengths can 'lock in' sand in the dunes that would have otherwise been released to supply downdrift shorelines.

1947 photogrammetry data it can be estimated that the clubhouse was then around 10-20m landward of the then +4m AHD contour. The main rock revetment is now some 30m seaward of an assumed 2018 ‘natural’ +4m AHD contour. Meaning there has been some 40-50m of shoreline recession at the site of JSPW that is unrelated to the structure itself.

In accordance with the findings in Section 3.3.2, this indicates that the southern embayment has experienced a naturally occurring net sand loss since at least the 1880’s. The process driving this natural sand loss is unknown. Goodwin et al 2013 reasoned the observed morphological change was evidence of a change in the regional wave climate indicating a shift in dominate deepwater wave direction from 120°-140° (1883) to 140°-160° (2002/2011), with a corresponding change in headland bypassing and embayment pathways. It is unclear if this sand loss trend is part of a longer-term cycle of naturally varying sand supply, which may be reversed in the future. Superimposed on the net loss is significant shorter-term cyclical variations in the embayment’s sand supply. These are driven by ENSO over time scales of 2 to 7 years and IPO over longer decadal scales.

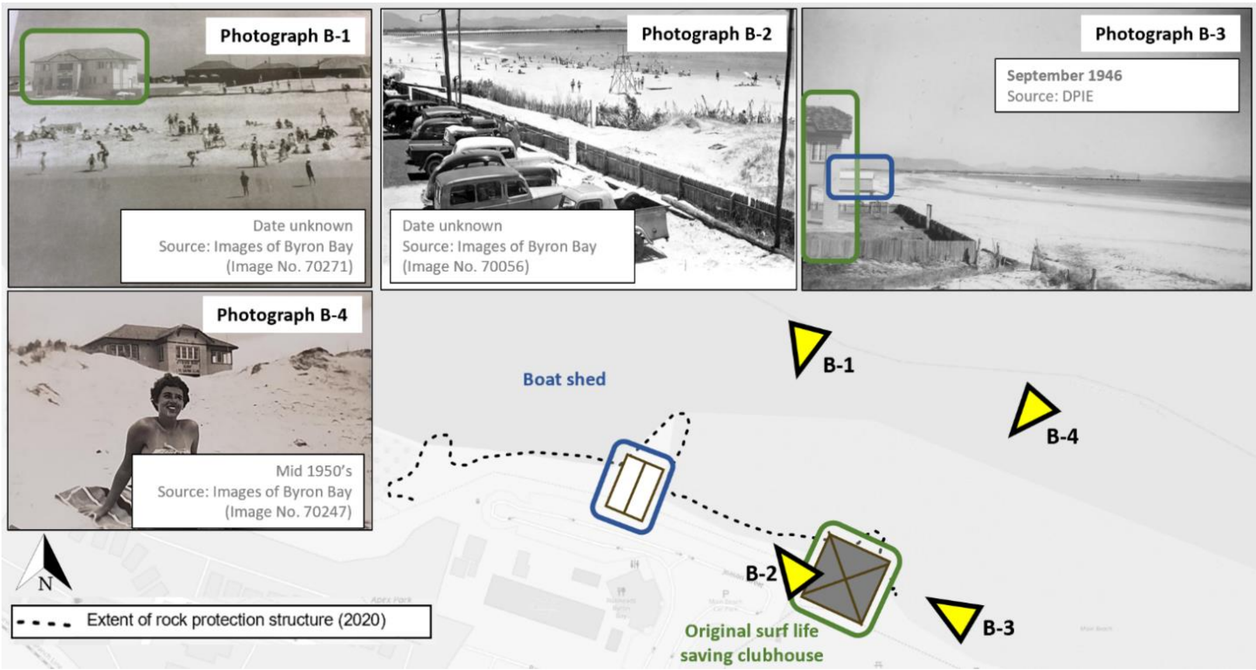

Figure 29. Photomap of the end of Jonson Street area prior to the JSPW taking shape at the site.

- Following the construction of the JSPW a relatively short period of planform realignment occurred. With reference to the photogrammetry data (see Figure 30) this manifests as:
  - A sharp stabilisation (or reversal of the rapid erosion trend) updrift of JSPW (see Block 5, profile 10) along Main Beach transitioning to an accretionary trend between 1980’s and 2021. The recent accretion trend observed in updrift beach profile agrees with the trend observed in DEA shorelines along Main Beach, see Figure 31.
  - Continuation of a persistent erosion trend, albeit reducing in rate, along Belongil Beach immediately downdrift of the JSPW (see Block 6, Profile 14). More recently this area appears to have stabilised undergoing a slightly accreting trend between 1980 and 2021. Again, the recent accretion trend agrees with DEA shorelines trends, see Figure 31.
  - Between 1km and 2km downdrift of the JSPW (see Block 6, Profile 50 and Block 7, Profile 35) the erosion trend has occurred as a more consistent rate when viewed over the 1970-2021 and 1980-2021 periods.

○ North of the Belongil seawalls (see Block 7, Profile 53) the erosion trend has accelerated.

This pattern indicates that the planform adjustment to the JSPW occurred relatively quickly (i.e., within 20-years of the original construction). Downdrift erosion induced by the JSPW appears to have been reduced by the anchoring effect of the updrift Belongil seawalls. This is evidenced by the recent accretionary trend at Block 6, Profile 14 (i.e., a similar pattern to that observed earlier updrift of the JSPW). The combined effect of the JSPW and the Belongil seawalls is to translate the net sand loss to the north of the private properties along Belongil Spit (i.e., to the very northern tip of the spit, Belongil Creek and the beach north of the creek.)

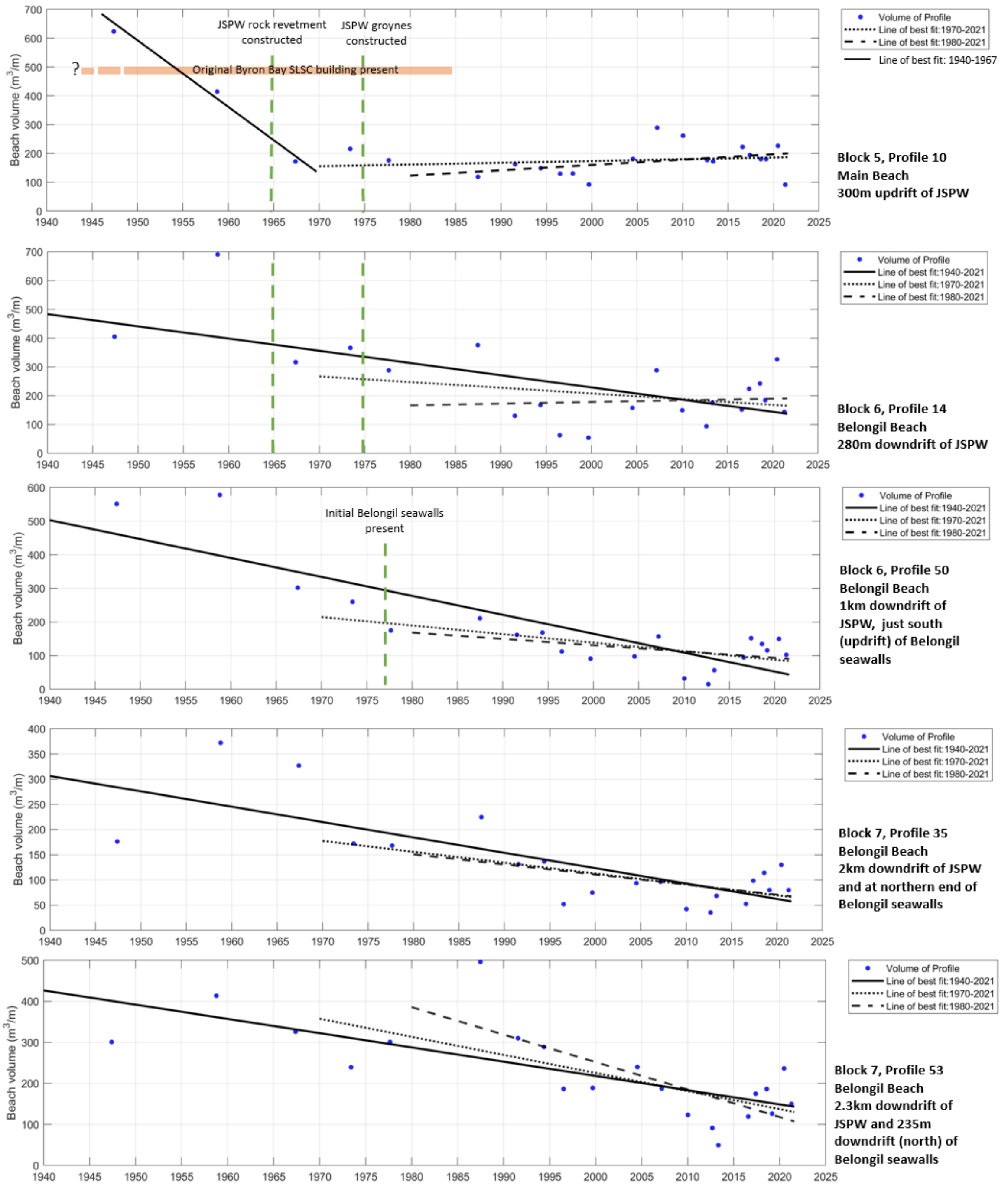

Figure 30. Timeseries of sub-aerial beach volumes (blue dots) from representative photogrammetry profiles at Main Beach (top) and Belongil Beach (bottom 4 panels) including lines of best fit.

Note: Lines of best fit are regression lines that show the trend in sub-aerial beach volume at each profile over a defined period. The steeper the line the more rapid the erosion (downward sloping) or accretion (upward sloping) observed.

- Headland bypassing and the supply of sand to the southern embayment has a controlling influence on the embayment's sand volume and shoreline. Coastal profile volumes calculated from surveys in 2002, 2011 and 2018 as shown in Figure 32 provide strong evidence of this. In 2011, and because of a surplus of supply, there was just under 300,000m³ more sand along the Main Beach compartment than in 2002. By 2018, this surplus had reduced by at least 200,000m³ with corresponding increases in sand volumes along Belongil Beach (i.e., more sand was bypassing the JSPW then was being supplied from the south).

  The fluctuations in the Main Beach shoreline since full bypassing was reached in the 1980s provides further evidence (see Figure 29). Following bypassing events, slugs of sand infill Main Beach moving the shoreline seaward to a position where sand can bypass the JSPW at all tides. This results in the structure having little to no effect on shoreline processes at these times. Between bypassing events when sand supply is low the reverse is true and structure-shoreline interactions increase.

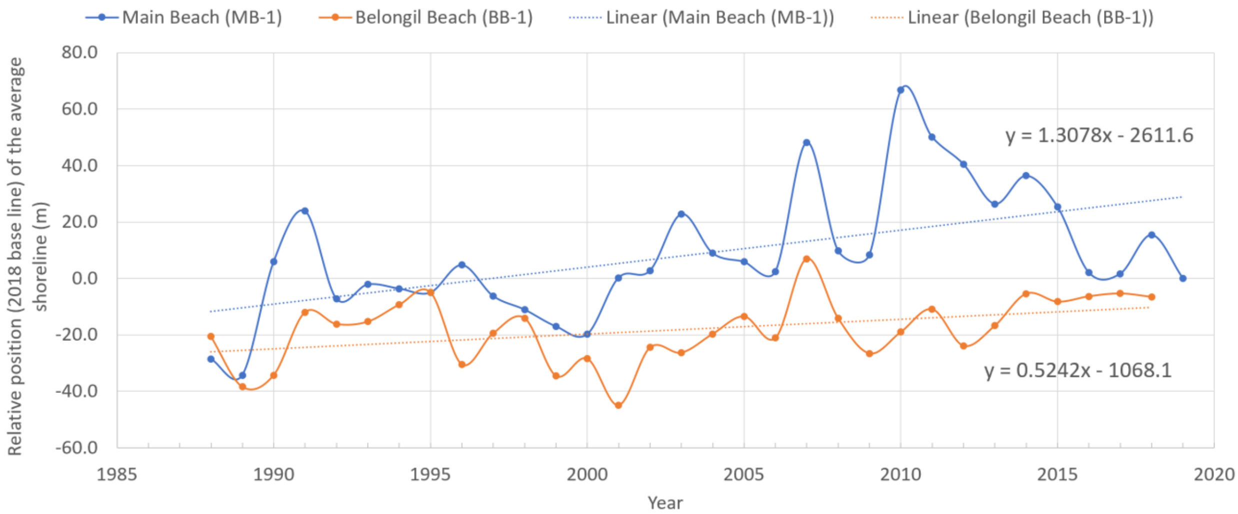

Figure 31. Average relative shoreline positions over 1km updrift (Main Beach) and 1km downdrift (Belongil Beach).

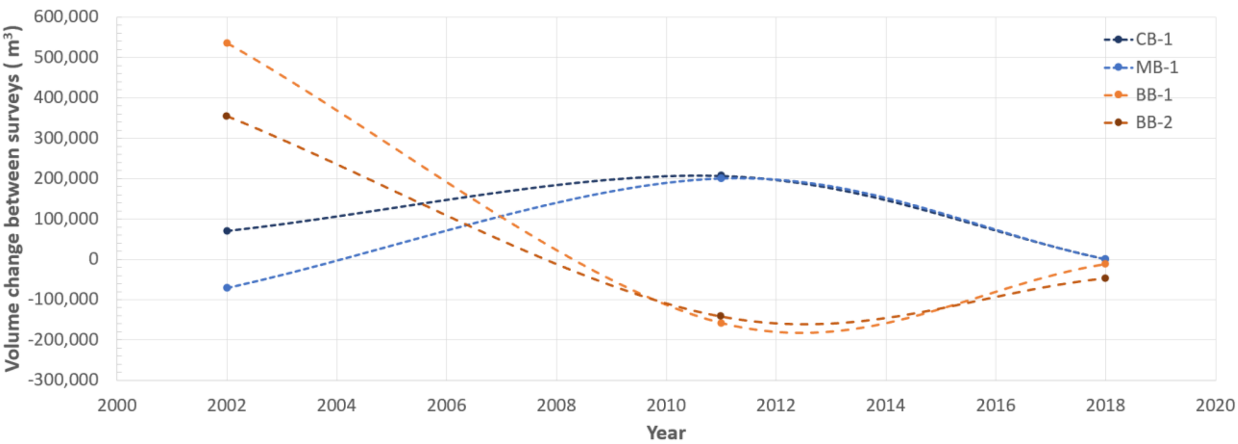

Figure 32. Full coastal profile volume change along Clarkes Beach (CB-1), Main Beach (MB-1), southern Belongil Beach (BB-1) and northern Belongil Beach (BB-2).

4. Shortlisted design options

4.1 Design development and initial assessment

The Concept Design Development Report outlines an appraisal of a longlist of options with a preliminary suite of the seven most suitable designs selected for further consideration. This included consideration of key design elements: i.e., the alignment of the protection works, range of material/structure type and range of treatments of the existing groynes. At the 27 August 2020 Council meeting, Council resolved (Res 20-435) to endorse the top seven concept options for key stakeholder and broader community engagement, being:

- Option 1 – rock revetment and stepped concrete seawall
- Option 2 – berm rock revetment and pathway
- Option 3 – detached groyne
- Option 4 – artificial headland with sand bypassing
- Option 5 – protective structure moved landward by 10m
- Option 6 – protective structure moved landward by 30m
- Option 7 – existing structure upgraded to contemporary standards.

Res 20-436 also called for an alternative Option 8 to be assessed in accordance with the overall project objectives. This option consisted of the realignment of the shoreline to a more natural position (as per Option 6) whilst compensate some of the reduced area for recreational space through the construction of an elevated platform that extends over the beach. However, the findings of the assessment concluded that the alternative Option 8 was not considered feasible and as such it was not added to the top seven options for broader engagement.

Community engagement was undertaken along with key stakeholder consultation during December 2020 and January 2021. The aim of the community consultation was to gain an appreciation of what the community value most about Main Beach and to inform the selection / development of the top three (3) preferred discrete options to progress to the next phase of the project, being the detailed technical investigation (this report).

The recommended three shortlisted design options to take forward and progress to detailed investigation were:

- Option 2 – berm rock revetment and pathway
- Option 5 – protective structure moved landward by 10m
- Option 7 – existing structure upgraded to contemporary standards.

A report was tabled at the Council Meeting of 28 October 2021, outlining the results of community consultation and feedback received during the engagement period. During the discussion of concept options to progress there was slight misunderstanding and misinterpretation of the landward alignment options as to how far the options were proposed to be realigned. As such, Council endorsed the following three options to take forward and progress to detailed investigation:

- Option 2 – berm rock revetment and pathway
- Option 6 - protective structure moved landward by 30m
- Option 7 - existing structure upgraded to contemporary standards.

The difference in the Council endorsed options is the substitution of Option 5 with Option 6.

The objective of the MBSP is to find a solution for the modification of the works that will give the best possible outcomes for Main Beach, Byron Bay and adjacent areas. Due to the time taken to set up numerical modelling tools for a data driven approach it was decided to undertake an assessment of both Option 5 and Option 6 (i.e., shortlisted of four – Option 2, Option 5, Option 6 and Option 7).

4.2 Shortlisted design options

Following baseline investigations to determine the design context (Bluecoast, 2021), a long list of seven potential concept design options were established for the JSPW. The preliminary development of the long list concept design options is documented in the MBSP's Concept Design Development Report (Bluecoast, 2020b). This included consideration of key design elements: i.e., the alignment of the protection works, range of material/structure type and range of treatments of the existing groynes.

Informed by community engagement and stakeholder consultation, reporting to Council and project team discussion a shortlist of four options were selected for further technical assessment. The four shortlisted options are summarised in Table 10 and Figure 33, including the treatment of key design parameters for each option to be evaluated. The Concept Design Development Report provides a more detailed description of each option.

Table 10: Summary of the key design elements in each option.

Project case	Alignment	Structure type	Treatment of groynes
Option 2 – berm rock revetment and pathway	Current alignment	Predominately rock revetment with inclusion of shared path on lower level (berm)	All groynes removed
Option 5 – protective structure moved landward by 10m	Landward alignment (10m)	Predominately rock revetment	All groynes removed
Option 6 – protective structure moved landward by 30m	Landward alignment (30m)	Predominately rock revetment	All groynes removed
Option 7 - existing structure upgraded to contemporary standards	Current alignment[1]	Rock revetment	All groynes retained

Note: [1] minor change in structure footprint (i.e., up to 3m extension of footprint to allow for rock armour) may be required.

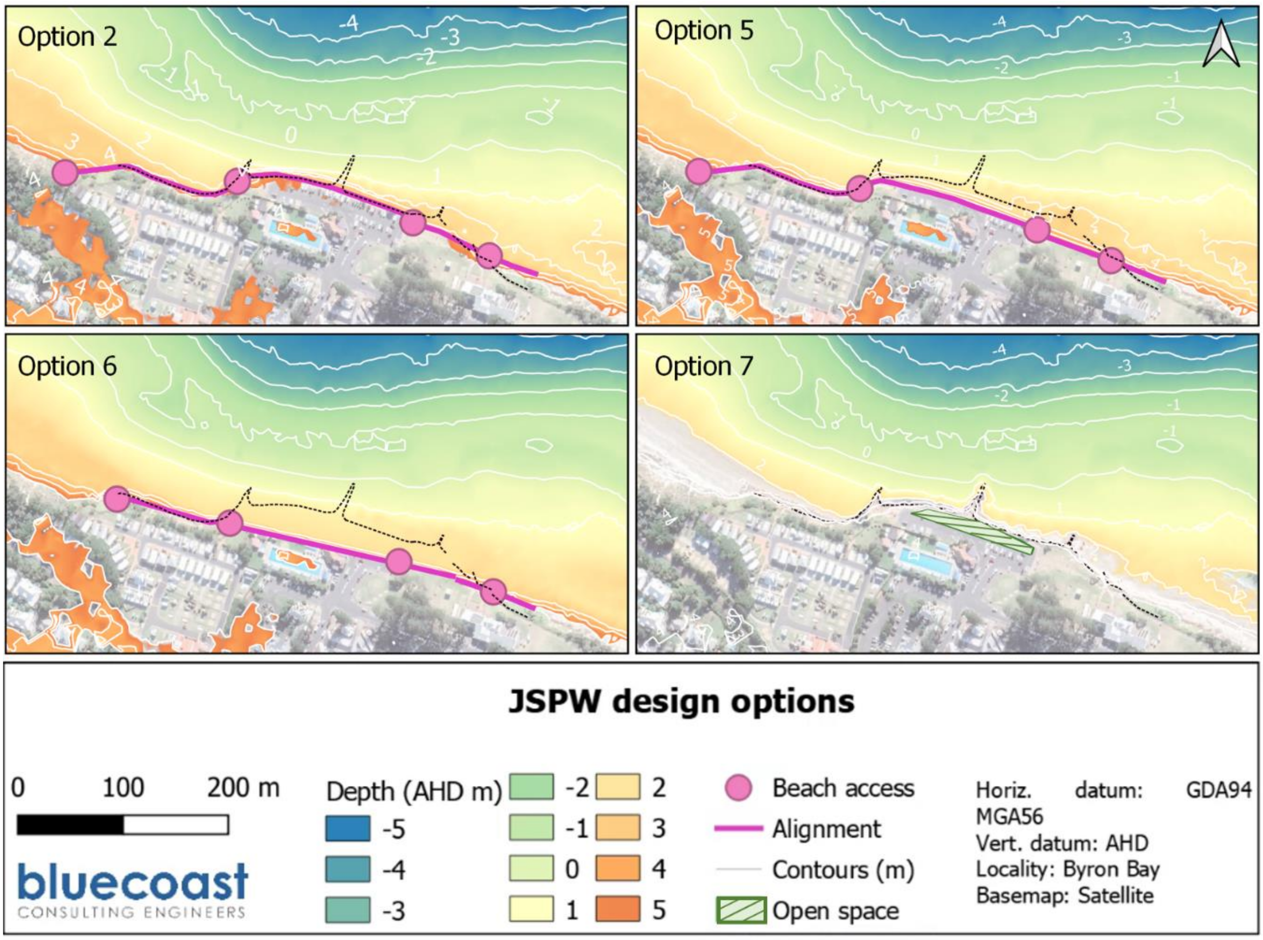

Figure 33. Summary of the concept design options.

5. Modelling approach

5.1 Modelling objectives

This report describes the application of a suite of numerical modelling tools that will help assessing:

- Nearshore wave climate within the Byron embayment to determine wave and other design conditions at the JSPW.
- Local wave and flow processes around the JSPW and the impact of the design modifications on these processes.
- The impact of the JSPW on the surrounding surf breaks and swimming areas.
- Any changes to sediment transport and coastal response during different wave condition (i.e., ambient condition, storm condition and long-term impacts)

5.2 Modelling approach

A suite of three numerical modelling tools were adopted for the MBSP design investigations. Each was selected as an appropriate software to investigate specific elements of the required assessment. The models are:

- **SWAN:** a 40-year wave hindcast was completed using SWAN, a spectral wave model. The SWAN model was used to transform offshore waves to the nearshore enabling a long-term wave climate at the project site based on offshore wave records. In addition to providing the nearshore wave climate, the hindcast wave model outputs have been used as boundary conditions for SWASH and XBeach models.
- **SWASH:** is a phase resolving, high-resolution, three-dimensional surf-zone wave and hydrodynamic model allowing simulation of non-linear wave and hydrodynamic processes within 1-2km of the site (e.g., wave transmission and nearfield hydrodynamics). SWASH is considered as being the most suited to model wave breaking and wave generated currents over structures but is not capable of modelling sediment transport or the response of the coastal profile, where XBeach is. Computationally demanding, this model is limited to event-based simulations (i.e., single wave and water level conditions).
- **XBeach:** A nearshore coastal response 1D model allowing simulation of detailed wave and flow conditions was used to assess wave overtopping discharges over JSPW structure and design options.

To enable the best possible accuracy of the results SWAN, SWASH and XBeach models were calibrated using local wave measurements collected specifically for this project.

5.3 Model calibration

5.3.1 Calibration standards

Model calibration is the process of setting physically realistic values for model parameters so that the model reproduces observed values to the desired level of accuracy. The process provides confidence in the model results and is essential for the accurate representation of coastal hydrodynamics and wave processes. The calibration standards presented in Table 11 have been adopted for this study based on the recommendation from Williams and Esteves (2017). These standards have been used to demonstrate that the models are capable of accurately representing the natural processes observed in the measured data.

The statistical standards provided in Table 11 are a good basis for assessing model performance, but experience has shown that sometimes they can be too prescriptive. It is also necessary for visual checks to be undertaken. Under certain conditions, models can meet statistical calibration standards but appear to perform poorly. Conversely, seemingly accurate models can fall short of the guidelines. Accordingly, a combination of both statistical calibration standards and visual checks has been used to ensure that the model is reliably representing the natural processes.

Table 11. Calibration standards for minimum level of performance of hydrodynamic and wave models.

Model predictions	Calibration standard
Water level	±10% of measured level (spring tide), ±15% of measured level (neap tide)
Average current speed	±20% of measured speed
Peak current speed	Within <0.05 m/s (very good), <0.1 m/s (good), <0.2 m/s (moderate) or <0.3 m/s (poor) of the measured peak speed
Wave height	±10% of height

Model predictions	Calibration standard
Wave period	±20% of period
Wave direction	5° of peak wave direction
Wave set up	±15% of measured wave set up (taken as 1% wave set up)

5.3.2 Calibration statistics

Model performance has been analysed by comparing the model predictions against the measured data using the statistical descriptions defined below along with average and difference comparisons:

- Model Skill (Murphy's Skill Score)
- Bias
- RMS Error
- Scatter Index
- R^2

The derivation of the parameters is outlined in brief below as defined in Willmott et. al. (1985). The **model skill** at simulating the measured conditions is given by Equation 1 below. This produces zero in cases of no agreement and one for perfect agreement between the modelled and measured data and is based off the Murphy's Skill Score:

$$Model\ Skill = 1 - \frac{\sum_{i=1}^{N}[M_i - O_i]^2}{\sum_{i=1}^{N}([M_i - \bar{O}_i] - [O_i - \bar{O}_i])^2} \qquad \text{Eqn, 1}$$

where:

- Oi is the observed or measured data (Ō is the mean of Oi)
- Mi is the modelled data
- N is the number of samples

The **bias** is a measure of the difference between the expected value and the true value. It is calculated using Equation 2. An unbiased model has a zero bias. Otherwise, the model is said to be positively or negatively biased, an indication as to whether the model is persistently over or underpredicting the physical conditions, respectively:

$$Bias = \frac{1}{N}\sum_{i=1}^{N} M_i - O_i \qquad \text{Eqn. 2}$$

The **RMS Error (RMSE),** Equation 3, is also a measure of the difference between the expected value and the true value of a parameter. It provides a measure of the magnitude of the difference between the modelled and measured values:

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}[M_i - O_i]^2} \qquad \text{Eqn. 3}$$

The **scatter index** is the RMSE normalised by the mean of the observations, see Equation 4. Generally, if SI is less than one, representation is acceptable as it provides an indication of the scatter of the data about the mean.

$$SI = \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}([M_i - M] - [O_i - \bar{O}])^2}}{\bar{O}} \qquad \text{Eqn. 4}$$

The **r squared (R2)** parameter, see Equation 5, is the coefficient of determination which describes the proportion of variance in a linear regression model between observed and modelled data. This parameter does not consider bias and hence needs to be assessed in combination with other statistics. It should be considered that a good agreement between modelled and observed data can be achieved despite a low R2 value, the formula is as follows

$$R^2 = 1 - \frac{\sum_{i=1}^{N}(O_i - M_i)^2}{\sum_{i=1}^{N}(O_i - \bar{O})^2} \qquad \text{Eqn. 5}$$

6. 40-year wave hindcast

6.1 Introduction

This section sets out the details of a wave hindcast model established for Byron Bay and used to transform a 40-year offshore wave record to the nearshore of Main Beach. Information from the 40-year wave hindcast will be used to provide a long-term description of the wave climate at the project site, including informing design conditions. It will also be used as boundary conditions for a series of more detailed models that cover the nearshore area of the Byron Bay embayment. The modelling methodology, model calibration and validation and results are described in this section along with a brief discussion of key findings.

6.2 Model configuration

6.2.1 Model description

The SWAN spectral wave model was adopted. This model includes a new generation spectral wind-wave model based on a rectangular grid. The model simulates the growth, decay and transformation of wind generated waves and swells in offshore and coastal areas. The SWAN model includes diffraction, refraction, shoaling, bottom friction, depth induced breaking, white capping, wind growth and non-linear interactions that affect waves propagating from offshore to nearshore. Therefore, SWAN is considered appropriate for this application (i.e., for the transformation of offshore waves to a nearshore location).

6.2.2 Model domain

The model extent, computational grid as well as the adopted bathymetry for the SWAN hindcast model are shown in Figure 34. The domain covers an alongshore (north-south) extent of approximately 43km and extends 32km offshore of Byron Bay. The model's eastern boundary was placed sufficiently far offshore so to be in deep water where the CAWCR wave hindcast outputs could be used as a boundary condition. The Byron Bay wave rider buoy (WRB) was included within the model domain.

As shown in Figure 34, two grids were used:

- larger offshore grid with a x-y resolution of 250m
- a nested 50m resolution grid provided enough spatial representation of the nearshore bathymetry and islands to capture spatial changes in wave conditions.

The model definition allowed for high resolution wave propagation into Byron Bay (i.e., downscaling of the global hindcast wave model).

6.2.3 Model bathymetry

The following survey data was used to inform the bathymetric description of the SWAN model:

- GEBCO 2009 data was used for the offshore region. This data is a global terrain model for ocean and land at 15 arc-second intervals. It was sourced from the International Hydrographic Organization.
- 2018 Coastal LiDAR dataset collected by the Department of Planning and Environment (DPE). This is a combined topographic and bathymetric dataset and used to describe the model's bathymetry for water depths less than 40m and the coastal topography models land boundaries and the nearshore areas.

All elevation and depth data have been corrected and applied relative to Australian Height Datum (AHD) and MGA zone 56.

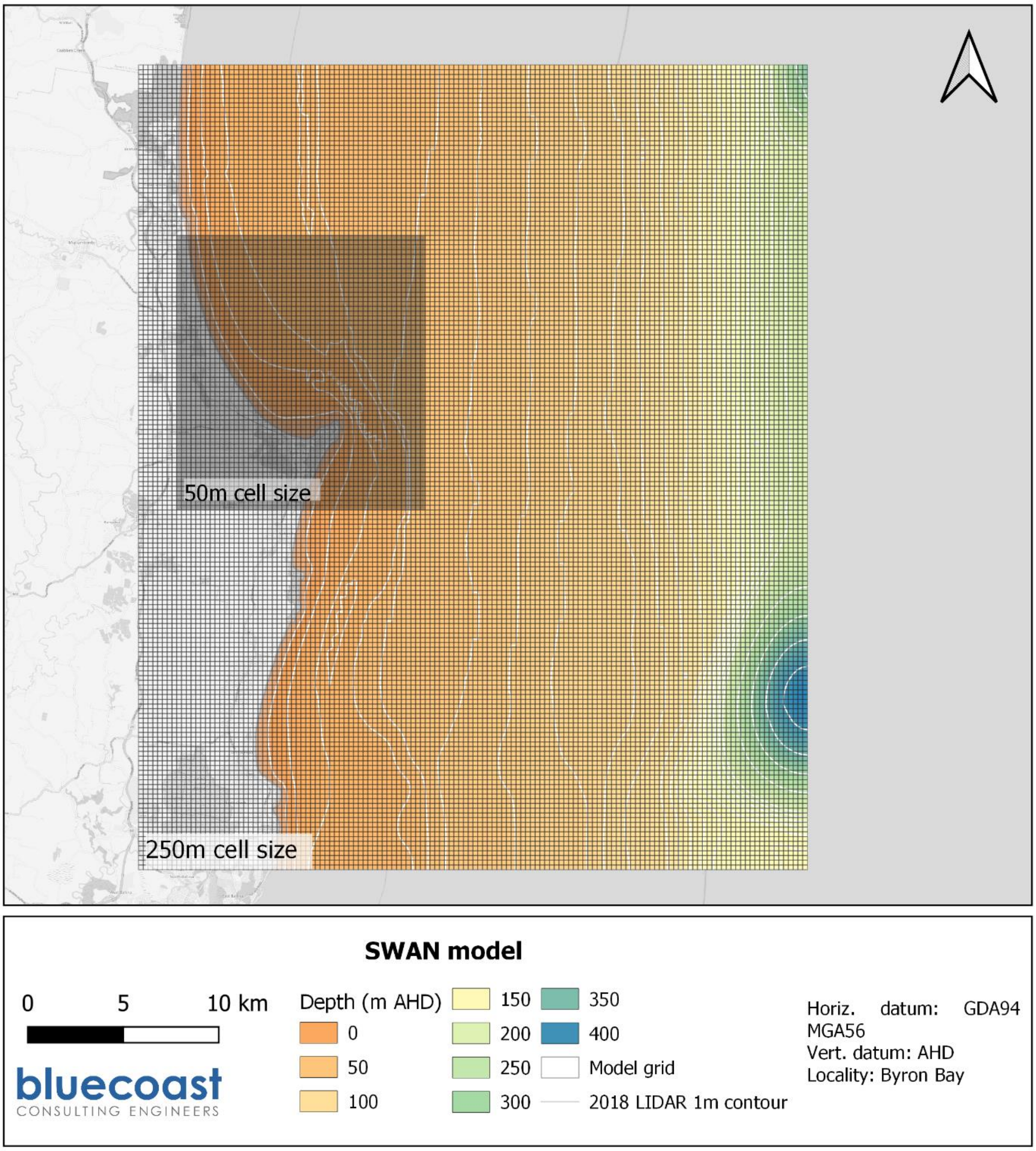

Figure 34. SWAN 40-year hindcast model bathymetry and grids.

6.2.4 Model setup

SWAN represents the wave field on a regular grid using the spectral density at discrete frequencies and directions. SWAN was setup to use the third-generation physics used in stationary mode. All relevant shallow water processes for coastal areas were considered: non-linear wave-wave interactions (quadruplets and triads), bottom friction, wind growth, depth induced breaking and whitecapping. The bottom friction was defined using the empirical JONSWAP model (coefficient of 0.038). The depth induced wave breaking gamma coefficient of 0.73 was adopted for the simulations.

For the wave computation, the direction resolution was set to 5°. Lower and upper limits for wave frequencies were specified as 0.033Hz and 0. 625Hz (or 1.6 to 33 second wave period range). The default value of 24 frequencies participants was adopted. Outputs were provided in 1-hour increments.

6.2.5 Model boundary conditions

The offshore boundary condition was defined using 40-years (1979-2019) of the CAWCR wave hindcast reanalysis by the CSIRO and Australian Bureau of Meteorology (BoM and CSIRO, 2014). This regional hindcast was developed using the WaveWatch III v4.08 wave model forced with NCEP CFSR hourly winds and contains directional spectral output at 0.3° resolution over the Australian region. A single spectral output location (see Figure 4) was used to define the various sea states along the SWAN model boundaries.

Wind data was obtained for the Cape Byron Lighthouse weather station from the Australian Bureau of Meteorology (BoM). Table 12 provides the co-ordinates for the CAWCR and wind data extraction locations.

Table 12. BoM/CSIRO model extraction locations.

Station	Longitude	Latitude
CAWCR wave data	-28.60 E	153.93 S
BoM wind data (station 58216)	-28.64 E	153.64 S

6.2.6 Hindcast methodology

Transformation of each individual wave record in the 40-year CAWCR wave dataset to the nearshore of Main Beach would be computationally expensive and time consuming. Instead, the nearshore wave transformation was achieved using a hybrid downscaling method presented by Camus et al. (2011). This method combines statistical techniques and numerical modelling.

A reduced number of representative offshore conditions were selected using a Maximum Dissimilarity algorithm. This algorithm distributes the selected wave conditions evenly throughout the range of observed conditions, with some points selected along the borders of the data space, therefore guaranteeing the most representative subset in comparison with the original sample. The SWAN model is then applied to propagate those selected offshore conditions to Main Beach. Finally, a radial basis function technique was applied to reconstruct the 40-year time series of wave height, wave direction and wave period on the coast at the model extract locations described below. The radial basis function approximation is very convenient for scattered and multivariate data, and it has been applied successfully in many previous studies.

6.2.7 Data extract locations

Model results including wave parameter timeseries were extracted from the 40-year hindcast model as the following locations (Figure 35):

- MB01 deployment location where the nearshore ADCP measured waves and currents offshore of Jonson Street
- Boundary to the SWASH embayment models (3 output locations)
- 11 locations around the Byron embayment approximate along the 4m depth contour.

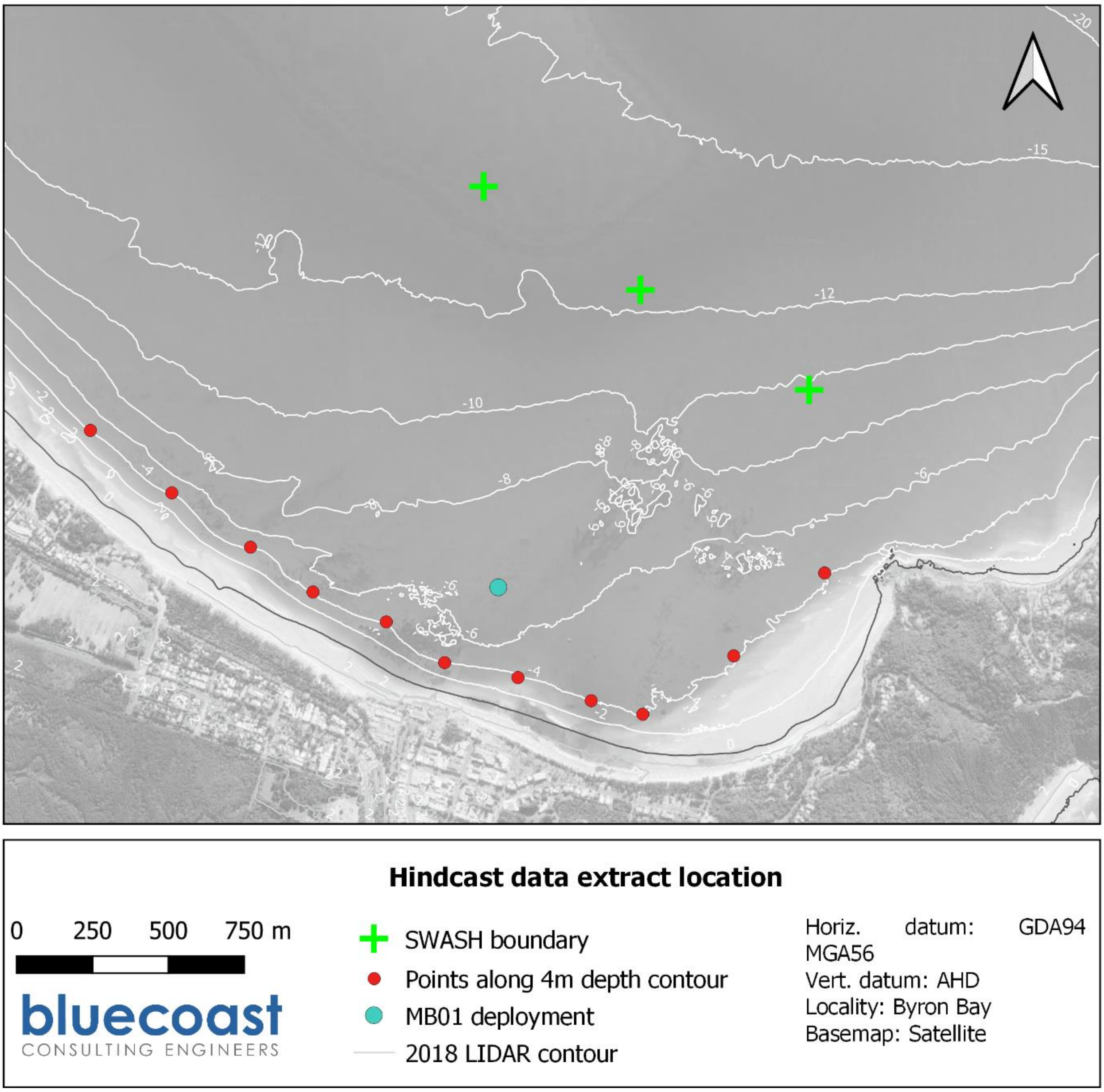

Figure 35. 40-year hindcast model extract locations.

6.3 Model calibration and validation

The calibration and validation of the wave hindcast model involved:

- A detailed model calibration using a 10-day period from the 20 December to 30 December 2019. This period was selected as it included waves from a range of offshore directions and one event with larger wave heights.
- Validation of the model results using an alternative two-month period comparing model results with measured data at the nearshore location (MB01).

Since the aim of the 40-year wave hindcast is to develop a long-term record of wave conditions, the model calibration and validation focused on the following wave parameters: significant wave heights, peak wave periods and peak wave directions.

6.3.1 Model calibration

Following several iterations of boundary configuration, wind forcing and model parameterisation the model was considered calibrated as the agreement with observed data was within the calibration standard set out in Section 5.3, see Table 13.

The modelled and measured wave height, wave period and wave direction at the Byron Bay WRB and MB01 are shown in Figure 36 and Figure 37, respectively. One discrepancy occurred around 23 and 24 December 2019, where a negative bias can be observed for modelled wave heights. The CAWCR reanalysis significant wave height was lower than the measured at WRB for those days. During 22 December modelled and measured wave heights increase reaching the ~3m wave height. After that, modelled wave height decreases steeply, while measured wave height decreases steadily. Model skill and statistics at the WRB and MB01 output locations are presented for the calibration period in Table 13 calculated using the method outlined in Section 5.3.2 using the data described in Section 2.

The MB01 site is within the Byron Bay embayment and allows the calibration of wave transformation processes such as wave refraction and wave dissipation due to bottom friction. Even though modelled wave height and wave period follow the trend of measured wave height and wave period, there are some exceptions during 22, 23 and 29 December. These exceptions coincided with northerly wind direction. In line with the calibration standards in Section 5.3.1, the agreement between the measured and modelled data, in particular wave heights, are acceptable to inform the subsequent detailed modelling.

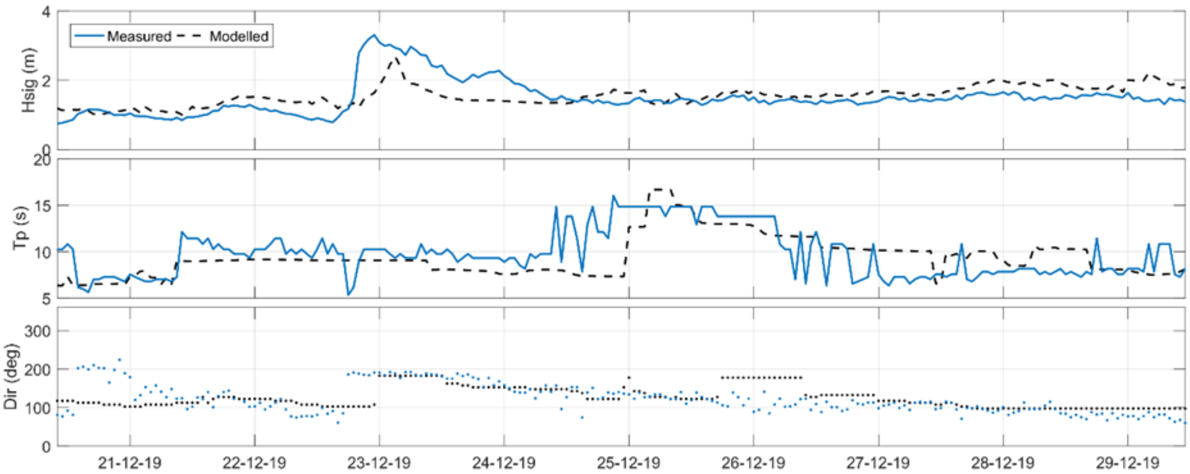

Figure 36. Comparison of measured and modelled wave height, period and direction at the WRB location during the calibration period.

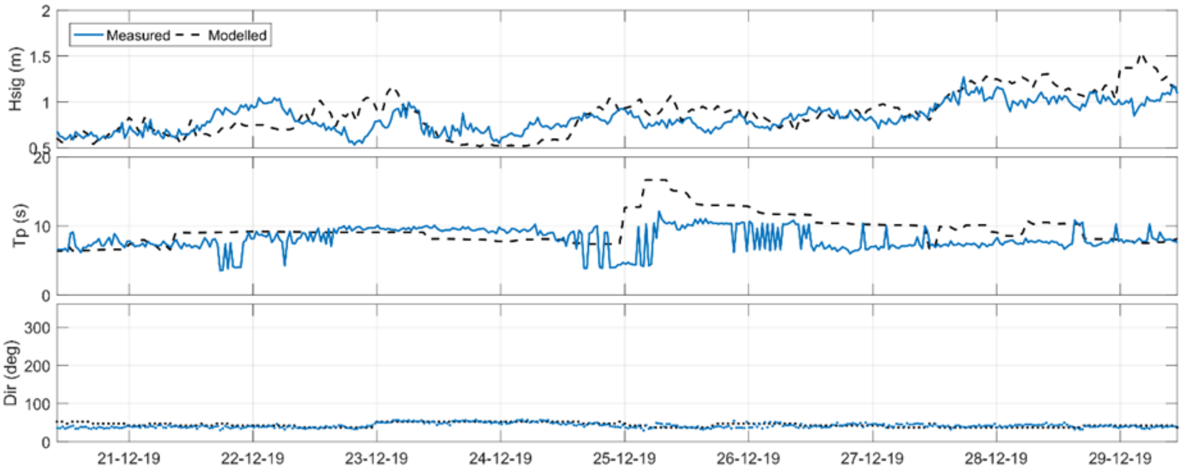

Figure 37. Comparison of measured and modelled wave height, period and direction at the MB01 location during the calibration period.

Table 13. Model skill and statistics of wave conditions at WRB and MB01 during the calibration period.

Location	Parameter	Meas. Mean	Mod. Mean	R^2	RMSE	Mean difference
WRB	Sig. wave height (Hs)	1.30m	1.42m	0.62	0.20	7%
	Peak wave period (Tp)	8.4s	10.0s	0.7	2.1	16%
	Peak wave direction (Dp)	101°N	120°N	0.4	25.9	9°**
MB01	Sig. wave height (Hs)	0.82m	0.87m	0.56	0.16	5%
	Peak wave period (Tp)	7.7s	9.2s	0.2	3.2	19%
	Peak wave direction (Dp)	40°N	43°N	0.4	6.2	3°

Note: ** The offshore wave climate at the WRB site is influenced by range of sea and swell sources, each with vary peak wave directions, and often occurring at the same time (i.e., bimodal sea states). The skill of our wave hindcast model at this location is largely depended on the CAWCR global model was used a boundary condition. Given that peak wave direction was within calibration standards at MB01 (at the study site) the model was able to reproduce refraction and diffraction around Cape Byron and considered suitable for our purposes.

6.3.2 Validation of hindcast

To validate the model and verify the hindcast methodology the calibrated SWAN model was then used to compare the timeseries of modelled and measured wave parameters over the full two-month deployment period of MB01. Figure 38 shows the comparison of modelled and measured wave heights over the entire measured record at the MB01. The model skill and statistics presented in Table 14 indicate that the model performance at MB01 is acceptable over the validation period (i.e., the mean differences are within the ranges of the calibration standards). The model agreement is within acceptable standards.

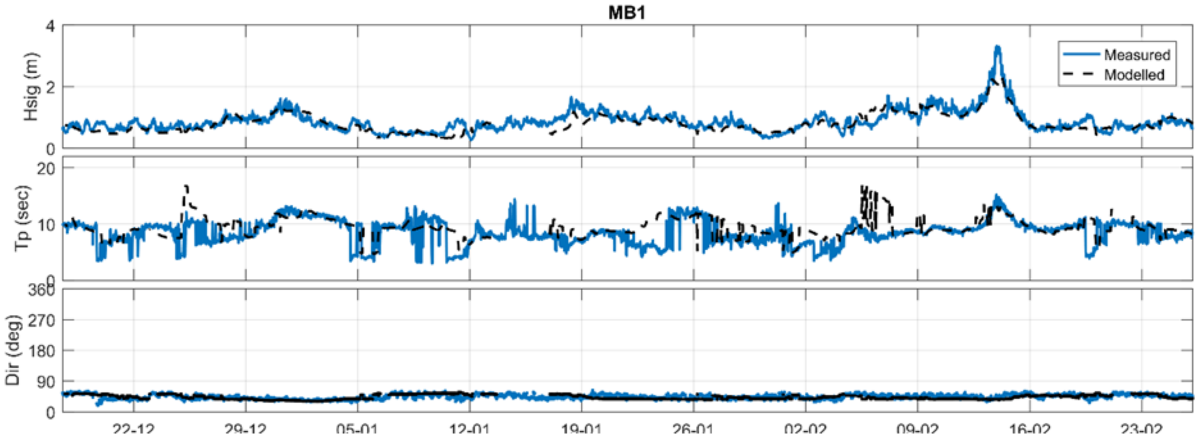

Figure 38. Comparison of measured and modelled wave height, period and direction at MB01 over the entire 2-month data record.

Table 14. Model skill and statistics of wave conditions at MB01 for the entire data record.

Location	Parameter	Meas. mean	Mod. mean	R^2	RMSE	Mean difference
MB01 **(2 months of data)**	Sig. wave height (Hs)	0.91m	0.85m	0.72	0.2	6%
	Peak wave period (Tp)	8.4s	9.4s	0.2	2.4	10%
	Peak wave direction (Dp)	43°N	42°N	0.1	7.3	1°

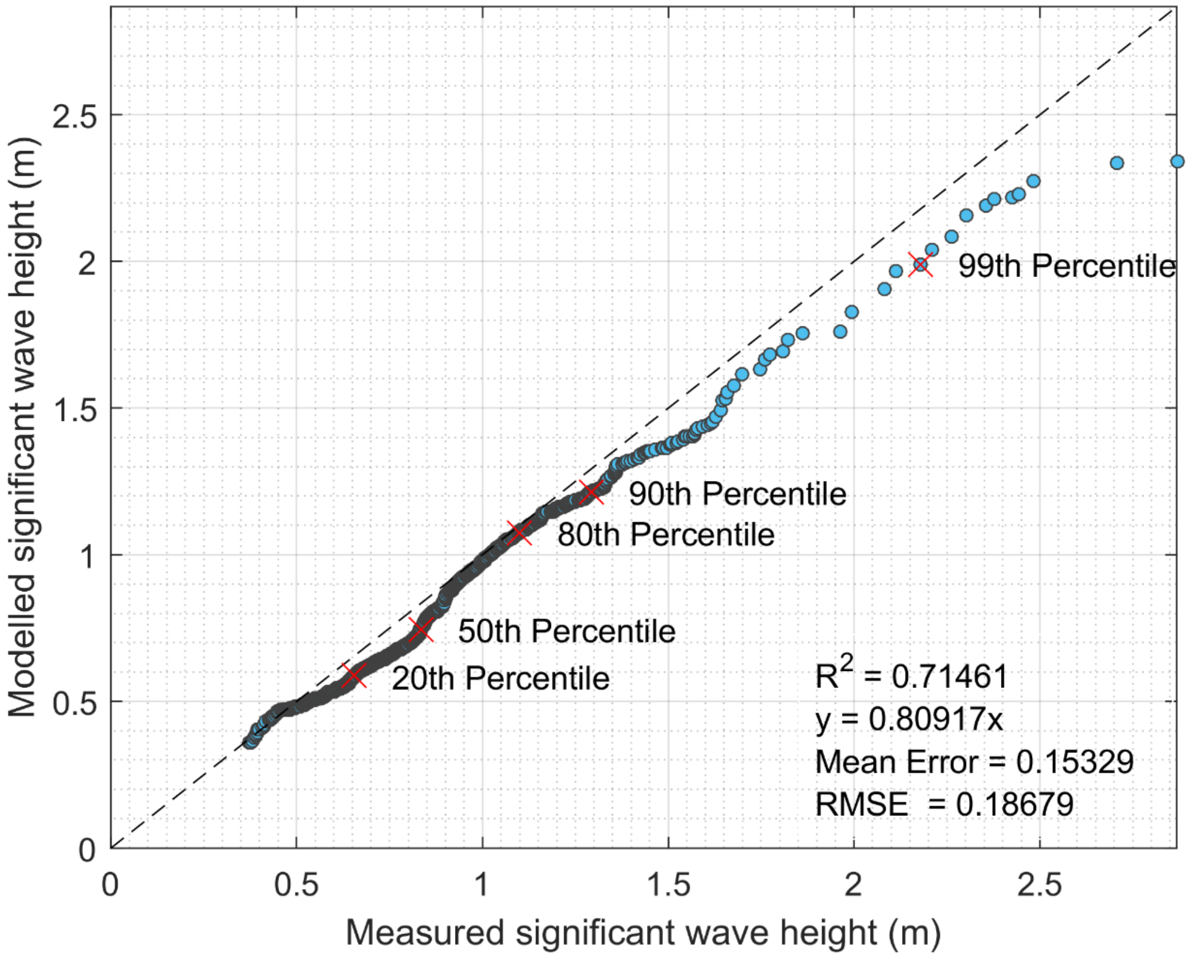

Figure 39. Quantile-quantile plot of measured and modelled wave heights at MB01 over the entire record of measured data (2-months).

6.4 Results

6.4.1 Example wave height maps

Maps of significant wave height and direction for a median energy wave condition (approximately 50th percentile on 30 November 2010) and a higher energy wave condition (maximum on 28 January 2013) are shown in Figure 40 and Figure 41, respectively.

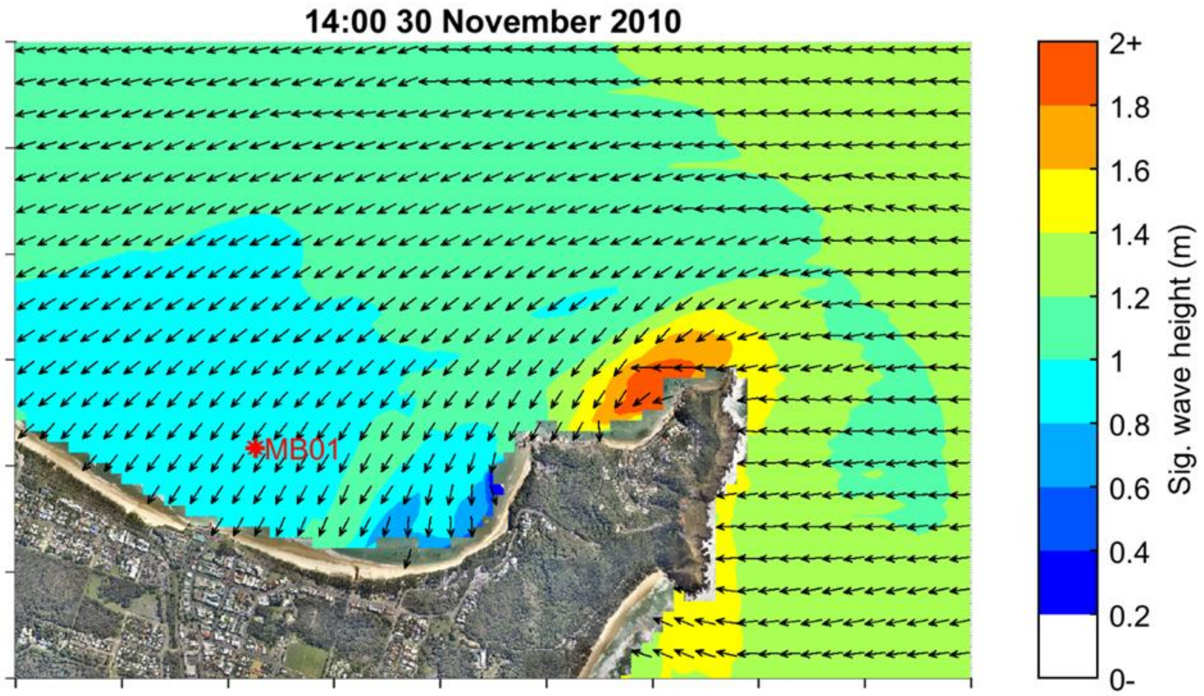

Figure 40. Map showing spatial pattern of the 50th percentile significant wave heights from the 40-year wave hindcast.

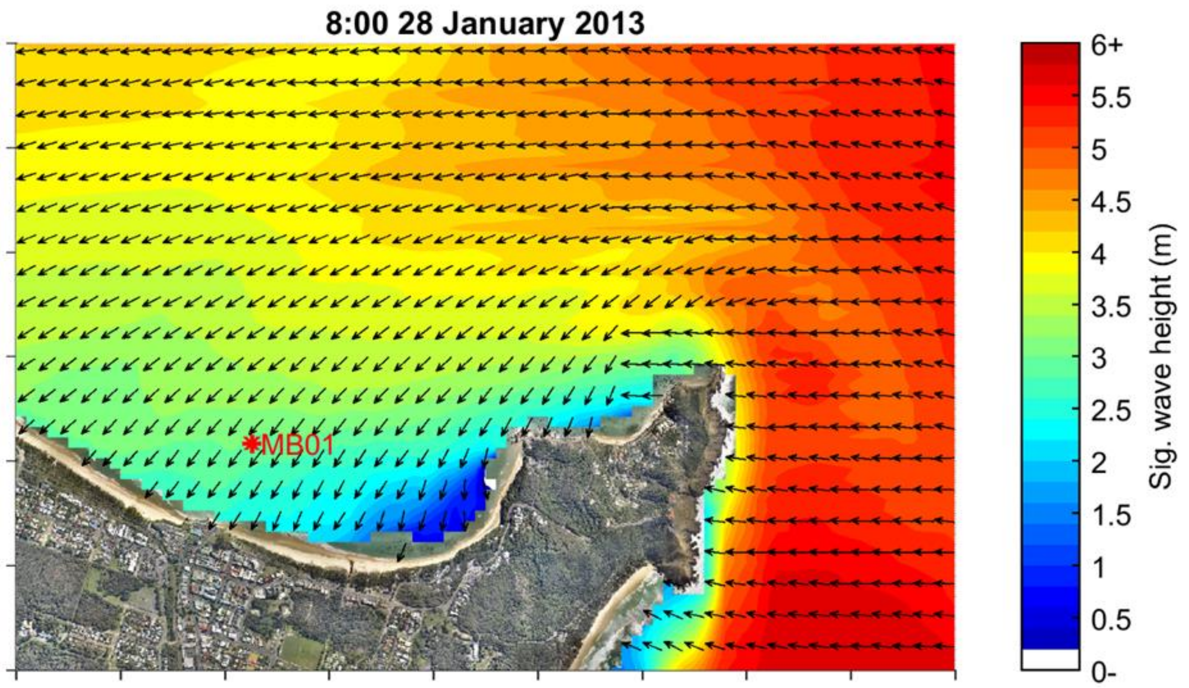

Figure 41. Map showing spatial pattern of the maximum significant wave heights from the 40-year wave hindcast.

6.4.2 Descriptive wave statistics

The long-term average and seasonal wave statistics for the 40-year hindcast at the MB01 site are summarised in Table 15. The wave climate at MB01 site is dominated by low energy, mid-period swell waves with a seasonal increase in the percentage of sea waves in the spring months.

Table 15. Modelled wave climate statistics between January 1979 and June 2019 and measured wave statistics over 2 months at MB01 site.

Parameter	Statistic	Long term averaged (40 years) MB01 modelled data (water depth 5.8m)					MB01 measured data (2 months)
		Annual	Winter	Spring	Summer	Autumn	
Sig. wave height (Hs) [m]	Mean	0.80	0.76	0.70	0.84	0.87	0.93
	20%ile	0.52	0.48	0.48	0.57	0.58	0.68
	50%ile	0.74	0.69	0.66	0.79	0.82	0.87
	75%ile	0.96	0.92	0.84	1.01	1.04	1.07
	90%ile	1.21	1.19	1.04	1.25	1.30	1.34
	99%ile	1.97	1.95	1.64	1.98	2.23	2.28
	99.5%ile	2.26	2.17	1.79	2.26	2.53	2.75
	Max	3.14	3.00	2.64	3.07	3.15	3.21
Peak wave period (Tp) [s]	Mean	9.3	9.5	8.8	9.3	9.7	8.5
	20%ile	7.8	7.9	7.4	7.8	8.1	6.9
	50%ile	9.1	9.2	8.8	9.1	9.4	8.7
	75%ile	10.4	10.5	9.7	10.3	10.7	9.9
	90%ile	11.7	11.9	10.9	11.7	12.0	11.3
	99%ile	16.5	16.6	15.9	15.7	16.6	13.1
	% of time sea (Tp < 8s)	20%	22%	31%	22%	16%	39%
	% of time swell (Tp > 8s)	80%	78%	69%	78%	84%	61%
Mean wave dir (Dp) [°N]	Weighted mean	39	40	40	39	39	43
	Mean	44	45	46	43	43	44
	Standard deviation	6.5	6.5	6.7	6.4	6.2	6.7

6.4.3 Wave roses and joint frequency tables

Wave rose plots for SWAN model results along the Byron embayment's 4m water depth contour are provided in Figure 42. A scatter plot of significant wave height and wave direction from the MB01 location is provided in Figure 43. The measurements show a narrow band of incoming wave directions, consistent with the wave hindcast.

Additionally joint frequency tables for wave parameters, including wave height versus direction, wave height versus period, and wave period versus direction, at this site are provided in **Appendix D**.

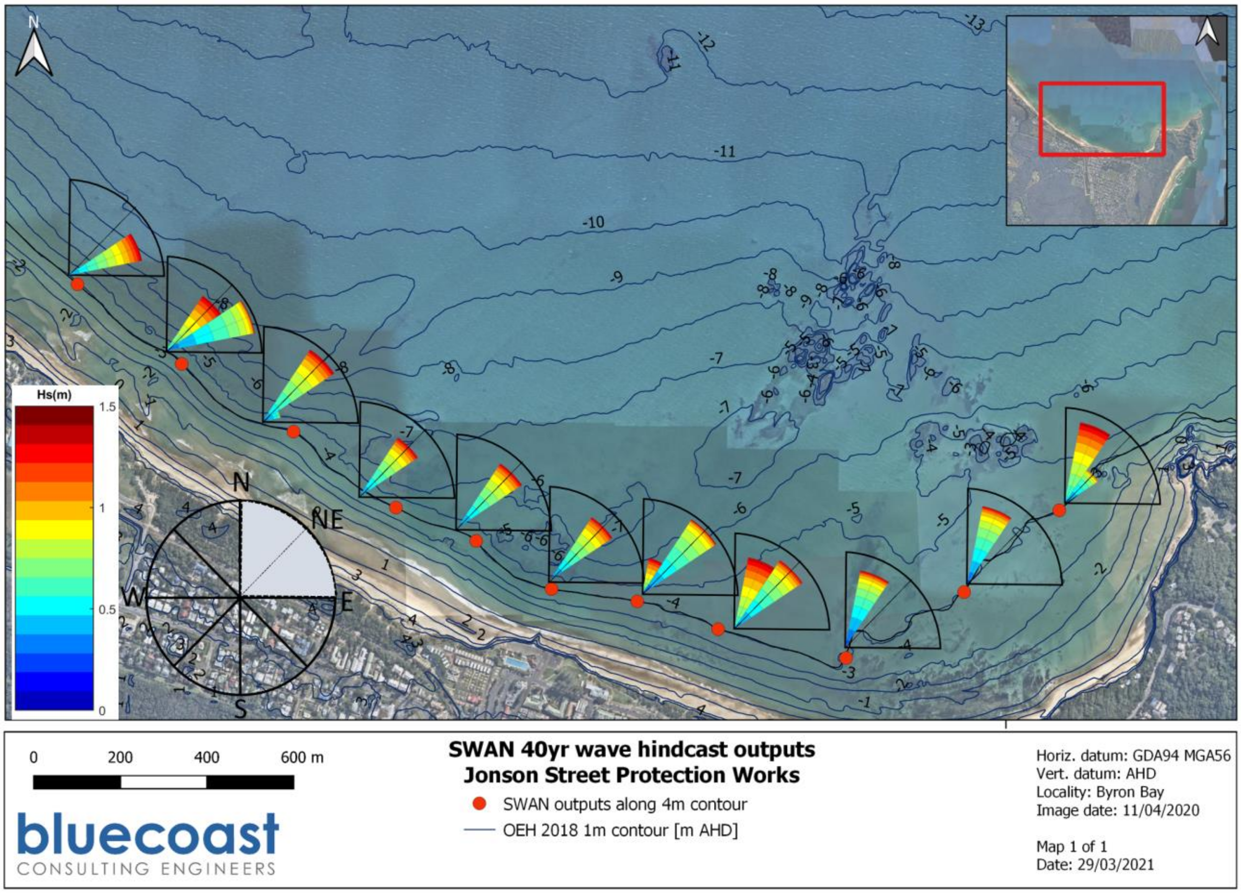

Figure 42. Nearshore wave roses along the 4m depth contour extracted from the 40-year wave hindcast results.

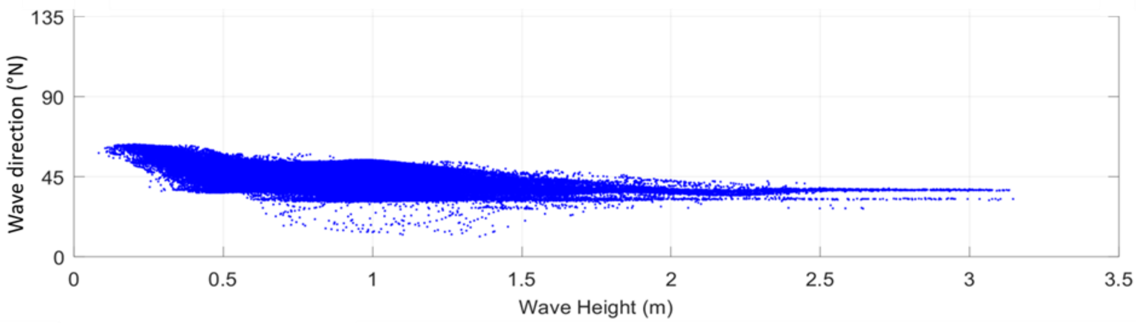

Figure 43. Scatter plot of significant wave height versus wave direction for the 40-year wave hindcast at the MB01 location.

6.4.4 Extreme wave climate

An extreme value analysis (EVA) was undertaken on the 40-year hindcast at the MB01 model extraction location. The resulting design average recurrence interval (ARI) wave conditions are presented in Table 16. Note that the maximum wave heights are the same for all ARI cases since wave heights are depth limited at MB01 (water depth approximately 6m).

Figure 44 shows the extreme value distribution of significant wave heights and associated peak wave periods at MB01. The 50-year and 100-year ARI significant wave heights at MB01 are 3.37m and 3.52m, respectively for a 1-hour duration. Their associated peak wave periods are 14.0s and 13.7s, respectively.

Table 16. Extreme value analysis results derived from the 40-year wave hindcast at MB01.

ARI (years)	Sig. wave height (m)	Lower confidence limit 98% (m)	Upper confidence limit 98% (m)	Associated maximum wave height (m)	Associated peak wave period (m)
1	2.44	2.39	2.49	4.5	15.0
5	2.87	2.76	2.98	4.5	11.6
10	3.03	2.89	3.16	4.5	12.5
50	3.37	3.18	3.57	4.5	14.0
100	3.52	3.30	3.74	4.5	13.7

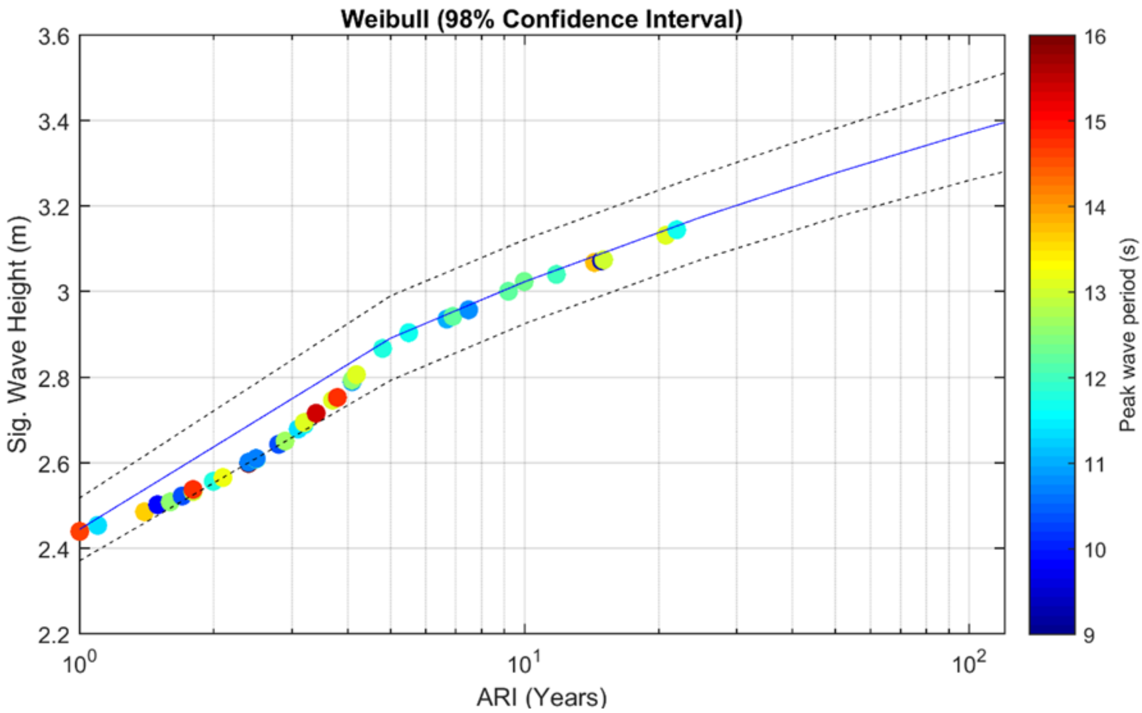

Figure 44. Design significant wave height curve including extreme wave height data (coloured by associated wave period) based on 40-year wave hindcast at the MB01 location.

6.4.5 Summary

The deep-water wave climate comprises a highly variable wind wave climate superimposed on a persistent low to moderate energy swells predominantly from the southeast to east directional sectors. Wind waves come predominantly from the east to southeast sectors and range from small, short period to large storm and cyclone waves

The embayment is exposed to waves from the east to north-east sector, with the predominant offshore waves from the south-east sector refracting and diffracting around Cape Byron and into the embayment. The wave climate at MB01 site is dominated by low energy, swell waves with a seasonal increase in the percentage of sea waves in the spring months. The waves roses show a narrow band of incoming wave directions at the nearshore locations.

The extreme wave climate at MB01 is characterised by a 50-year and 100-year ARI significant wave height of 3.37m and 3.52m, respectively for a 1-hour duration. The maximum wave heights during large storms observed in the hindcast at MB01 are depth limited.

7. SWASH modelling

7.1 Introduction

This section sets out a detailed wave and flow model, SWASH, established for MBSP and used to provide wave-by-wave transformation across the Byron embayment and wave-structure interactions at the JSPW for four selected wave and water level conditions. The modelling methodology, model calibration and validation and results are described along with a brief discussion of key findings.

The SWASH wave and flow modelling aims to provide:

- A detailed transformation of waves over the Byron embayment and its shallow reefs, to identify wave energy hot spots and shadows and their effect on nearshore hydrodynamics.
- Predictions of the effect of the shortlisted JSPW design options on nearshore waves and hydrodynamics.
- Prediction of the effect of the shortlisted JSPW design options on surfing and swimming amenity opportunities.

7.2 Modelled options

The four shortlisted options (i.e., project cases) presented in Section 4 have been compared to the basecase, which consists of the existing JSPW in its current form. As Option 7 retains the existing structure alignment but upgrades it to contemporary standards. This would be expected to incur only minor changes to the current structure footprint (i.e., 3m seaward extension to allow for additional rock armouring), with the scale less than that which could be meaningfully discerned by the SWASH modelling. Therefore, the SWASH modelling results of the existing condition (basecase) and Option 7 (project case) results are considered the same.

7.3 Model configuration

7.3.1 Model description

A three-dimensional non-hydrostatic wave-flow model called SWASH (an acronym of Simulating WAves till SHore) has been established. Unlike spectral wave models SWASH resolves individual waves. SWASH is a nonlinear shallow water wave and hydrodynamic model which accounts for wave-breaking, non-linear wave transformation, interaction with structures and estimates wave-induced water level setup.

The SWASH model can be used to predict the transformation of dispersive surface waves from offshore to the beach and is capable of accurately representing the following physical phenomena:

- Wave propagation, frequency dispersion, shoaling, refraction and diffraction
- Nonlinear wave-wave interactions (including surf beat and triads)
- Wave breaking
- Wave runup and rundown
- Partial reflection and transmission
- Wave interaction with structures
- Wave-current interaction
- Wave-induced currents.

7.3.2 Model domain

A high-resolution SWASH model was setup to cover the Byron Bay shoreline. The model extent adopted was 1km by 3km extended from Wategos to Don Street at Belongil Beach. A regular grid resolution of 3.0m was used with two vertical layers. The model extent, computational grid as well as the adopted bathymetry for the SWASH model is shown in Figure 45.

7.3.3 Model bathymetry

The basecase model adopts the Laser Airborne Depth Sounder (LADS) hydrographic and topographic survey carried out by the Department of Planning and Environment (then the NSW Office of Environment and Heritage) over three days in June and July 2011. The dataset covers the area from 200m behind the high-water line to a depth offshore of around 30-meters and with a 5-meter resolution. The accuracy of soundings to 20 m depth is not expected to exceed 0.50 m within a 95% confidence level, meeting the International Hydrographic Organisation (IHO) Order-1b minimum requirements for depth accuracy (Goodwin, 2013).

The bathymetry of the Byron embayment is highly variable with the seabed and shoreline changes influencing the nearshore waves and currents. Sensitivity testing using the SWASH model was carried out comparing 2018 coastal LIDAR surveyed bathymetry and topography and the 2011 survey with a focus on the JSPW and immediately adjacent shorelines. The 2011 bathymetry was selected because it is more representative of the average coastal profile condition and was observed to have more wave-structure interaction when compared to the 2018 survey, which captured an accreted coastal profile.

The JSPW do not change between surveys and were noted to be better represented using 2018 coastal LIDAR survey. Therefore, the basecase bathymetry (i.e., the existing condition) is formed by merging the 2011 LADS survey for the embayment and the 2018 coastal LiDAR for the JSPW.

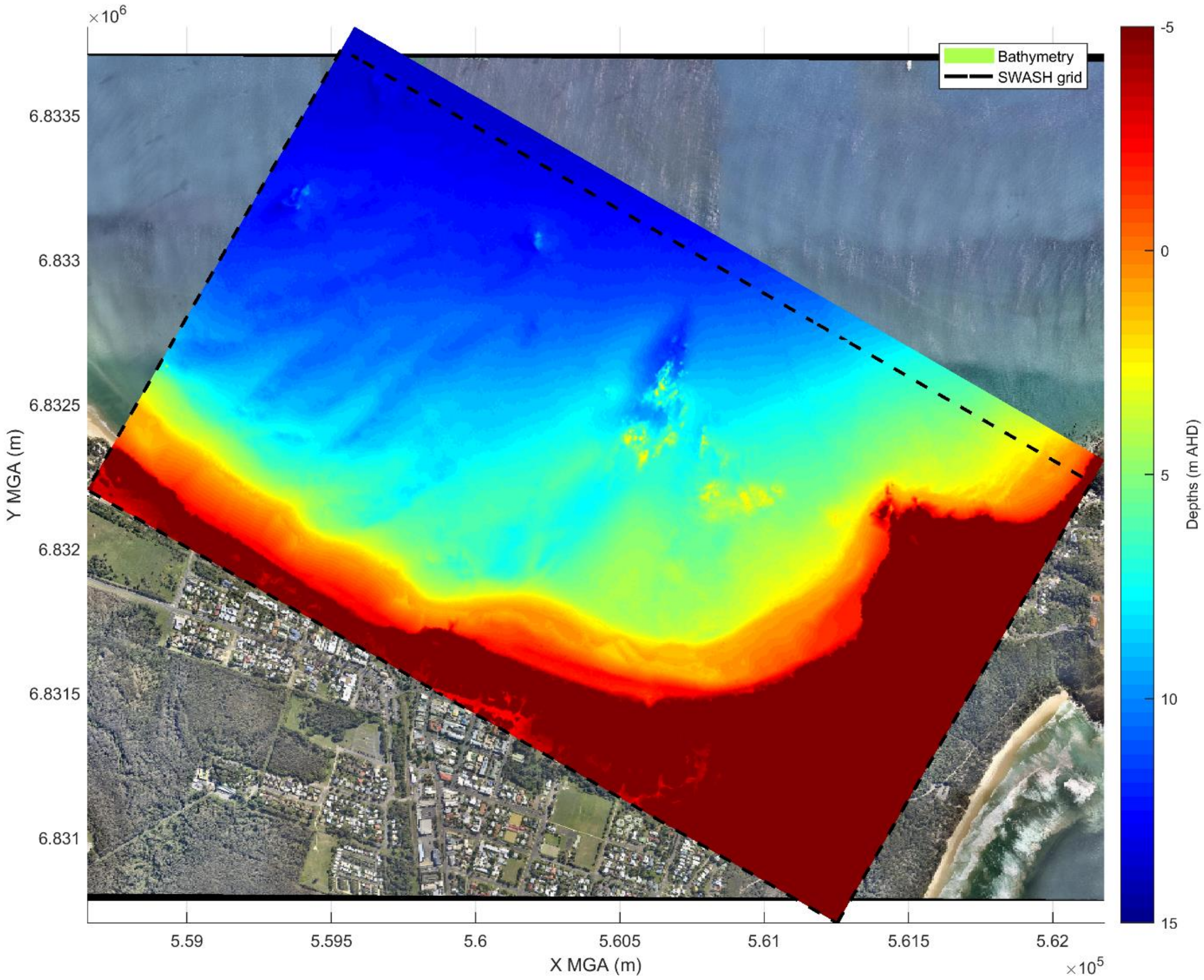

Figure 45. Model extent, computational grid and bathymetry.

7.3.4 Model setup

The model was setup to simulate waves over a 25-minute period for a single wave condition. Waves at the boundary are generated by a numerical wavemaker to provide a spectral representation of the waves. The results from the model are averaged over the two layers in the model (adopted to represent any differences in wave and current conditions between the surface and bed) and over the last 18 minutes of the simulation (the first 7 minutes are considered the 'warm-up' period). The simulation and analysis duration were selected after testing showed that the hydrodynamics stabilised within 4-5 minutes of the simulations starting. The time step was 0.01 seconds. Point outputs were stored every 0.2 second (or 5Hz), which is more than enough to robustly analyse time-series parameters.

Wave breaking was parametrized using Hydrostatic Front Approximation (HFA) (Kennedy et al., 2000; Tonnelli and Petti, 2012) using default parameters of α=0.6 and β=0.3, as recommended by Smit et al. (2013). Manning formula with a constant friction coefficient of 0.016 was used. Porosity layers along the left and right model boundaries, over the rocky zone in The Pass and over existing Jonson Street structure were added to make the model stable.

7.3.5 Model outputs

The following results were extracted from the model simulations:

- Map outputs of the waves, currents and water level conditions across the model domain.

- Timeseries outputs at the two observed locations (MB01 and MB02) as well as the six locations fronting the JSPW.
- Three shore normal profiles adjacent to the JPSW.

The locations of the point and profile extracts as displayed below in Figure 46 and in Table 17.

Figure 46. Location of SWASH model extraction points and profiles.

Table 17. SWASH model extraction points and profiles.

Station	X (MGA56)	Y (MGA56)
MB01	560,129	6,832,086
MB02	559,922	6,831,744
P2	560,013	6,831,708
P3	559,929	6,831,751
P4	559,830	6,831,749
P5	559,778	6,831,771
P6	559,681	6,831,808
P7	559,716	6,831,945
C1	559,678 to 559,774	6,831,727 to 6,831,918
C2	559,890 to 559,938	6,831,698 to 6,831,880
C3	560,056 to 560,107	6,831,637 to 6,831,866

7.4 Model calibration

7.4.1 Selection of calibration scenarios

Two representative wave (mid- and high-energy) conditions for the SWASH model calibration were selected from the two-month measurement period. These conditions are provided in Table 18 along with the long-term (40-year) statistics they represent, the time they occurred and the measured wave and water parameters at MB01.

Table 18. SWASH model calibration conditions.

Scenario	Long term (40-year) wave statistic	Measured conditions at MB01				
		Time	Hs [m]	Tp [s]	Dp [°N]	WL (m AHD)
Mid-energy	50th percentile Hs	23/12/2019 18:00	0.75	10.0	52	0.28
High-energy	90th percentile Hs	12/02/2020 10:00	1.58	10.0	45	0.80

The offshore wave boundary wave conditions for the SWASH model are derived from the 40-year wave hindcast at three output locations spaced at 600m intervals along the boundary (Figure 47). The two selected calibration events in Table 18 were extracted for the equivalent SWAN hindcast results and applied to SWASH as a spatially varying boundary. A JONSWAP wave spectra was generated with a peak enhancement parameter of 3.7 which were selected to match the respective measured wave spectra.

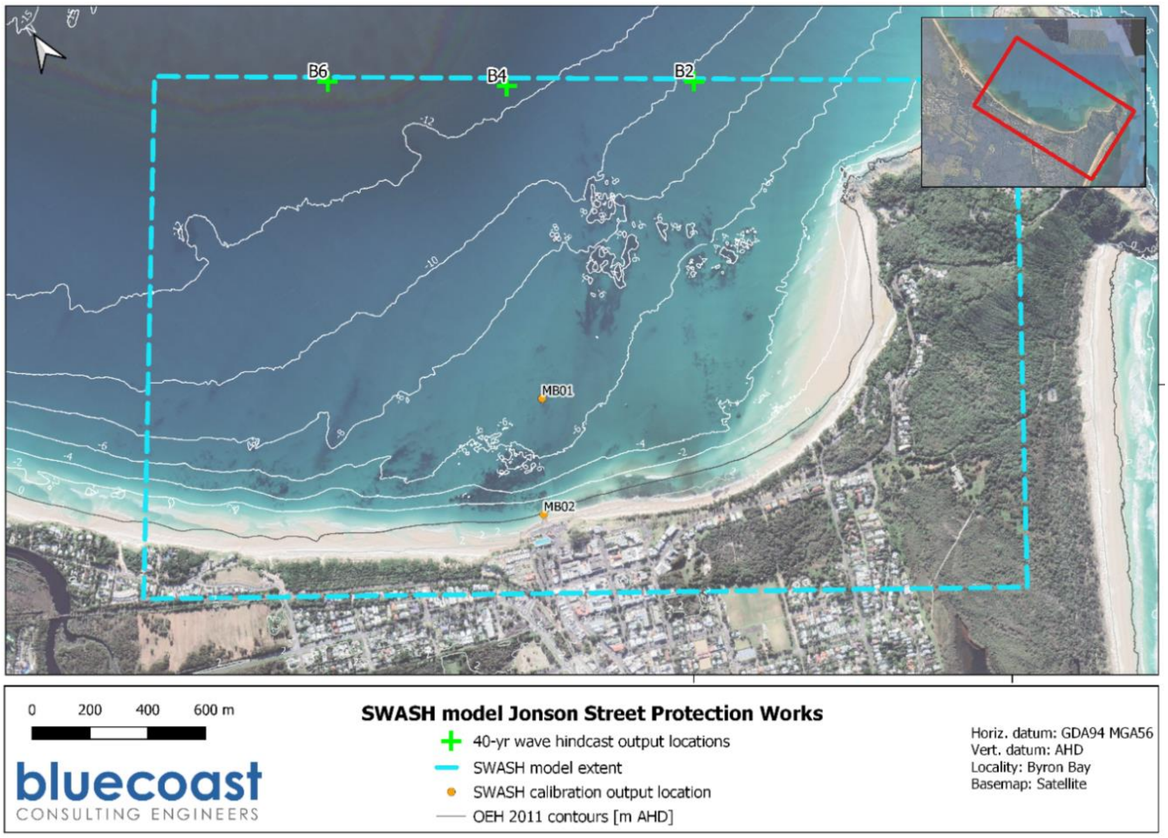

Figure 47. SWASH offshore boundary configuration using SWAN 40-year hindcast results.

7.4.2 Calibration results

Table 19 provides a comparison of the measured and modelled wave and current conditions for the two selected calibration conditions at MB01 and MB02. The results show that for both events the modelled and measured wave and currents conditions show good agreement. At MB02, wave setup was the primary measurement aim and this parameter is therefore presented in the below comparisons. Figure 48 shows a comparison of the 1D spectra between the measured data and model results at MB01 for both conditions. The results demonstrate that the model can represent the processes which occur at the Byron embayment and provides additional confidence in the modelling results.

Table 19. SWASH calibration results.

Parameter	Mid wave energy			High wave energy		
	Measured	Modelled	Difference	Measured	Modelled	Difference
Significant wave height (m) at MB01	0.75	0.75	0%	1.58	1.61	2%
Peak wave period (s) at MB01	9.5	9.3	-2%	10.0	9.5	5%
Peak wave direction at MB01	52	50	2°	45	45	0°
Current magnitude at MB01	0.03	0.03	0%	0.05	0.05	0%
Wave set up (m) at MB02	0.07	0.07	0%	0.19	0.18	5%

Note: * Water level input was increased (+0.2m) to run high wave height.

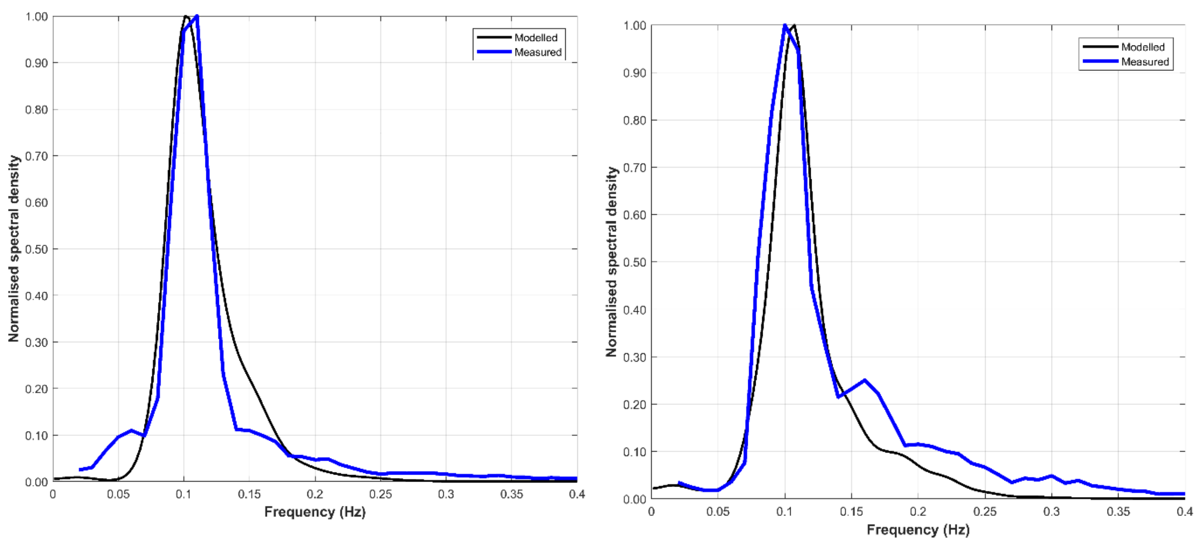

Figure 48. Measured and modelled 1D spectra for mid-energy (left) and high-energy condition (right).

A further model verification specific to the nearshore hydrodynamics was noted under the S2 wave condition (see Section 7.5.1 for the definition of the S2 wave condition). Under these wave and water conditions a rip current is predicted by the model on the western side of JSPW. To understand if this rip current occurs, a review of aerial photography was undertaken, and it was observed that from time to time this rip current is present (see Figure 49). This qualitative comparison provides a level of confidence that the model can simulate some of the finer scale surfzone hydrodynamics in the study area.

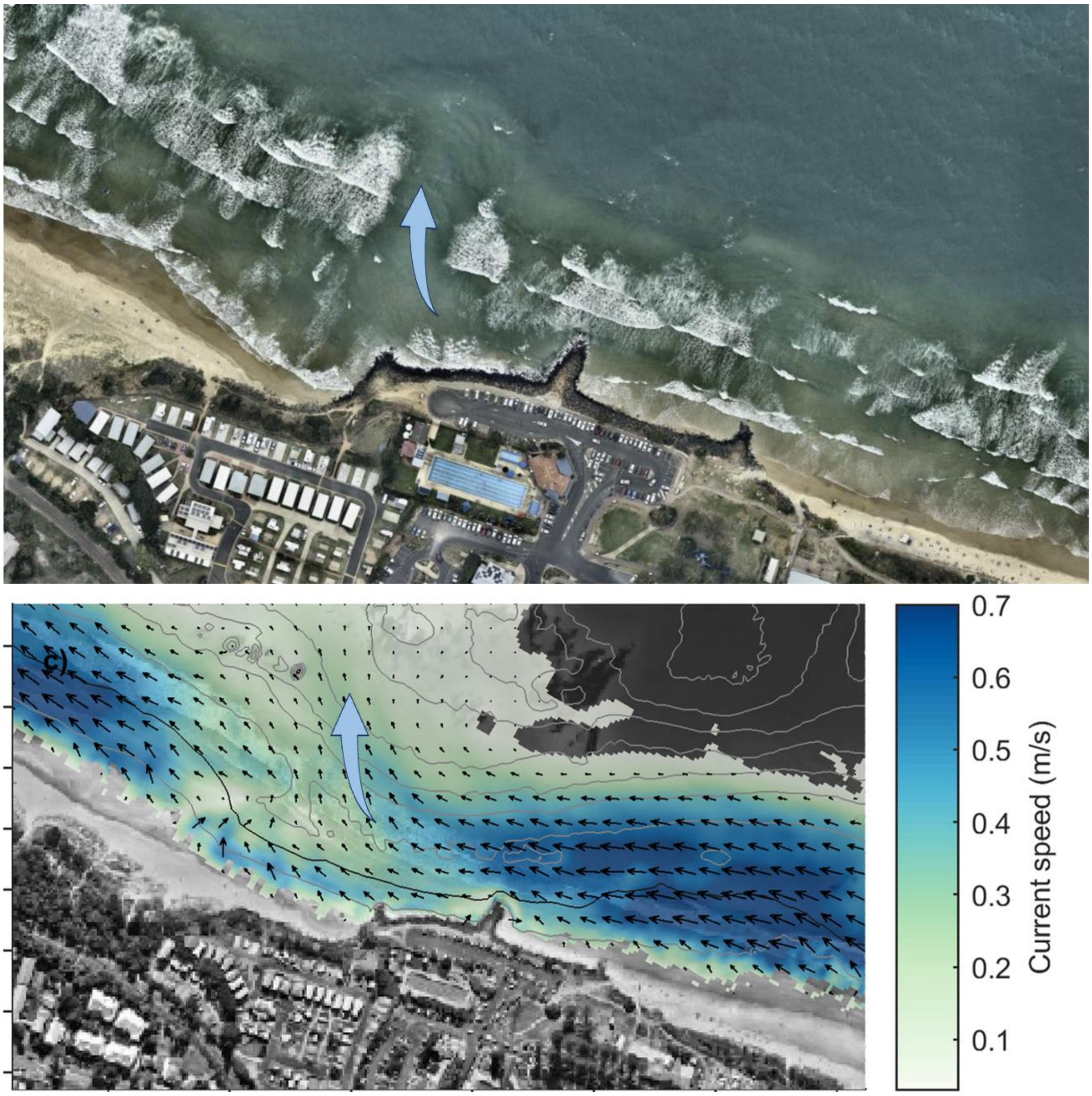

Figure 49. Nearmaps aerial photograph from 11 March 2019 (top) alongside SWASH current field for S2 conditions (bottom).
Note: The rip current is highlighted in both images with the light blue arrow.

Map outputs for the mid- and high-wave energy conditions, respectively, are provided in Figure 50 and Figure 51 and show from top to bottom:

- model bathymetry
- significant wave height
- average wave driven currents

- sea surface elevation or *eta* which is the vertical displacement of the surface referenced to MWL (i.e., a ‘snapshot’ of wave crests and troughs as they sweep across the embayment)

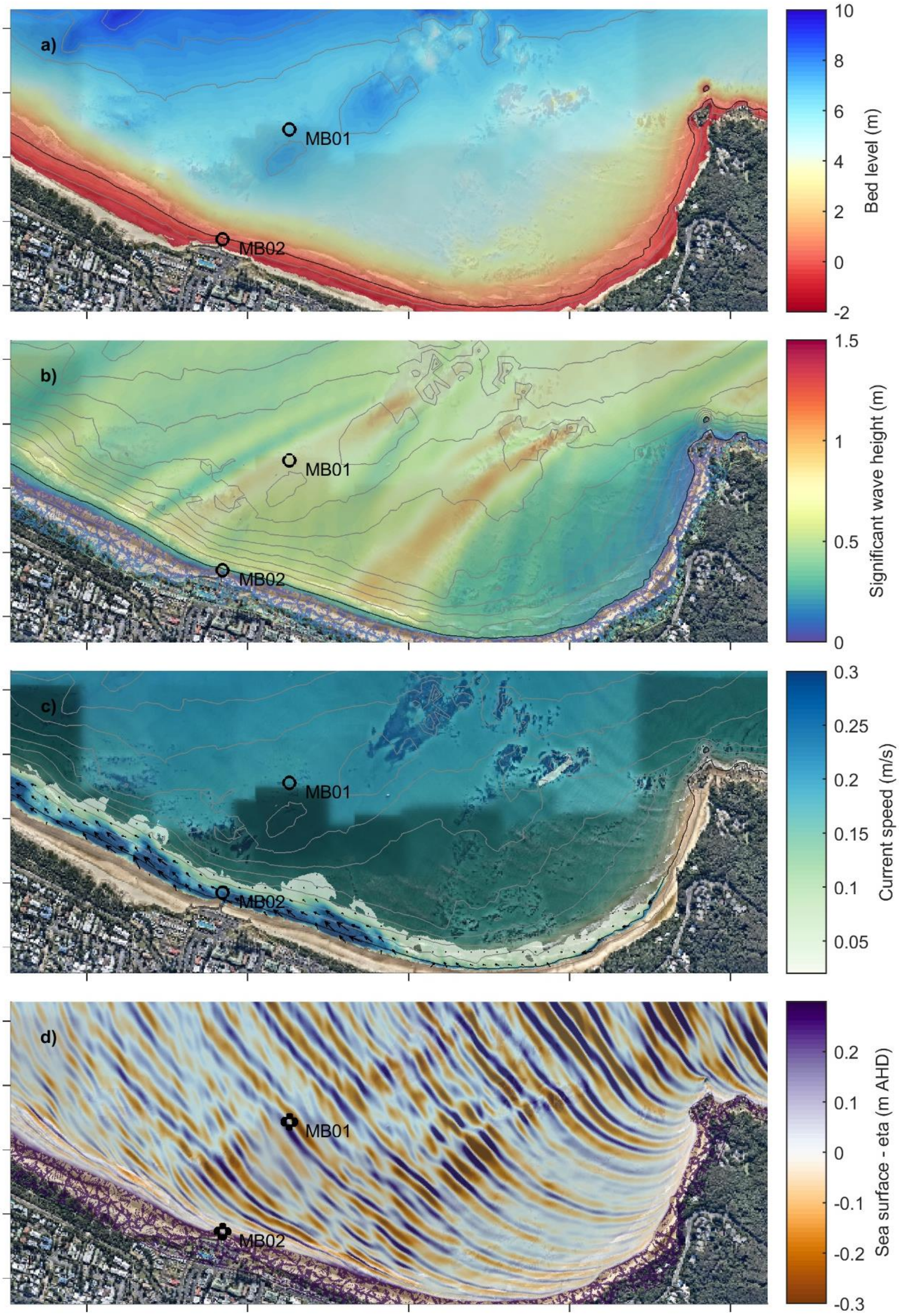

Figure 50. Mid-wave energy result maps (a. bathymetry, b. significant wave height, c. currents and d. surface elevations).

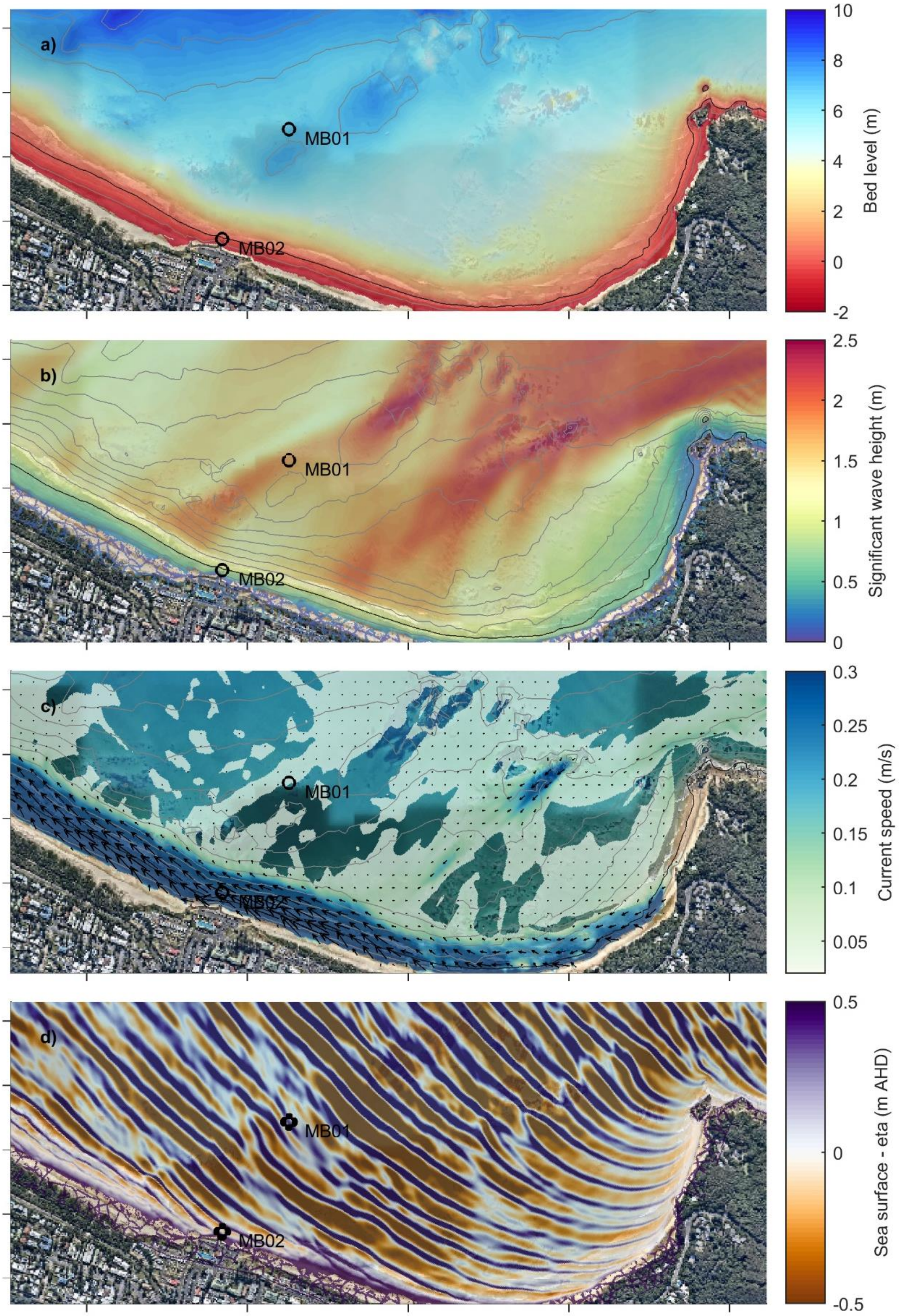

Figure 51. High-wave energy condition result maps (a. bathymetry, b. significant wave height, c. currents and d. surface elevations).

7.5 Modelling results

7.5.1 Modelled wave and water level conditions

For the purposes of evaluating the complex interaction between the JSPW and nearshore wave and hydrodynamic processes the basecase and JSPW design options were simulated for the three selected conditions in Table 20. These adopted wave and water level conditions are justified for their use as follows:

- Two morphology conditions with a higher high tide water level (High High-Water Solstice Springs (HHWSS)) and waves selected as:
  - 50th percentile wave condition as the median or day-to-day wave conditions
  - 90th percentile wave conditions were selected as representative of higher wave energy but regularly occurring event.
- Storm event and sea level rise
  - 50-year ARI wave height and 100-year ARI water level (+1.46m AHD) were selected to analyse the nearshore hydrodynamics in an extreme event. The 2100 sea level rise projection was adopted from IPCC AR6 (for Yamba, NSW) relative to 1995 - 2014 baseline (Garner et al., 2021). The adopted sea level rise of 0.78m by 2100 is a mid-value between 50th percentile SSP1 (RPC2.6) and 83rd percentile SSP5 (RCP8.5) projections.
- An additional wave and water level condition to evaluate the relative surfing and swim amenity:
  - 75th percentile wave condition as medium-high wave energy and mean sea level.

Table 20: Select wave and water level conditions used for the SWASH modelling.

Representative wave condition	Wave type	Significant wave height (m)	Peak wave period (s)	Wave direction	Water level (m AHD)
S1 50th percentile wave height at MB01	Spectral wave	0.87	10	NE (SSE offshore)	1.1
S2 90th percentile wave height at MB01	Spectral wave	2.28	11	NE (SSE offshore)	1.1
S3 50-year ARI and SLR for 2100 scenario using a SLR value of 0.78m	Spectral wave	2.37	13	NE (SSE offshore)	2.2
S4 75th percentile wave height at MB01 and MSL	Spectral wave	1.5	11	NE (SSE offshore)	0.0

7.5.2 Model results

Simulated wave and hydrodynamics maps and statistics for a series of design scenarios are provided herein. Discussion and comparison of the results is provided in Section 6.6.

Modelled nearshore hydrodynamics are presented for the basecase and project cases in Figure 52, Figure 53, Figure 54 and Figure 55 for S1, S2, S3 and S4 wave and water level conditions, respectively. Like the calibration figures these layouts also show the following panels from top to bottom:

- model bathymetry
- significant wave height
- average wave driven currents
- sea surface elevation (i.e., a 'snapshot' of wave crests and troughs as they sweep across the embayment) - this output is only shown for S2, S3 and S4 conditions.

Summary statistics were calculated at the model extraction locations shown in Figure 46 and used in the assessment of the nearshore hydrodynamics, surfing and swim amenity between the basecase and project cases. The wave height and current speed statistics for each observation point are presented in Table 21, Table 22 and Table 23, for the S1, S2 and S3 wave and water level conditions, respectively. The location of each observation point is presented in Figure 46 and Table 17. P2, P3 and P4 are located in front of each JSPW groyne, in order from east to west and along the 1-meter contour. P5 is in front of First Sun Holiday Park and P6 is 100m to the west of P5 along the 0m contour. P7 is located at The Wreck surf spot.

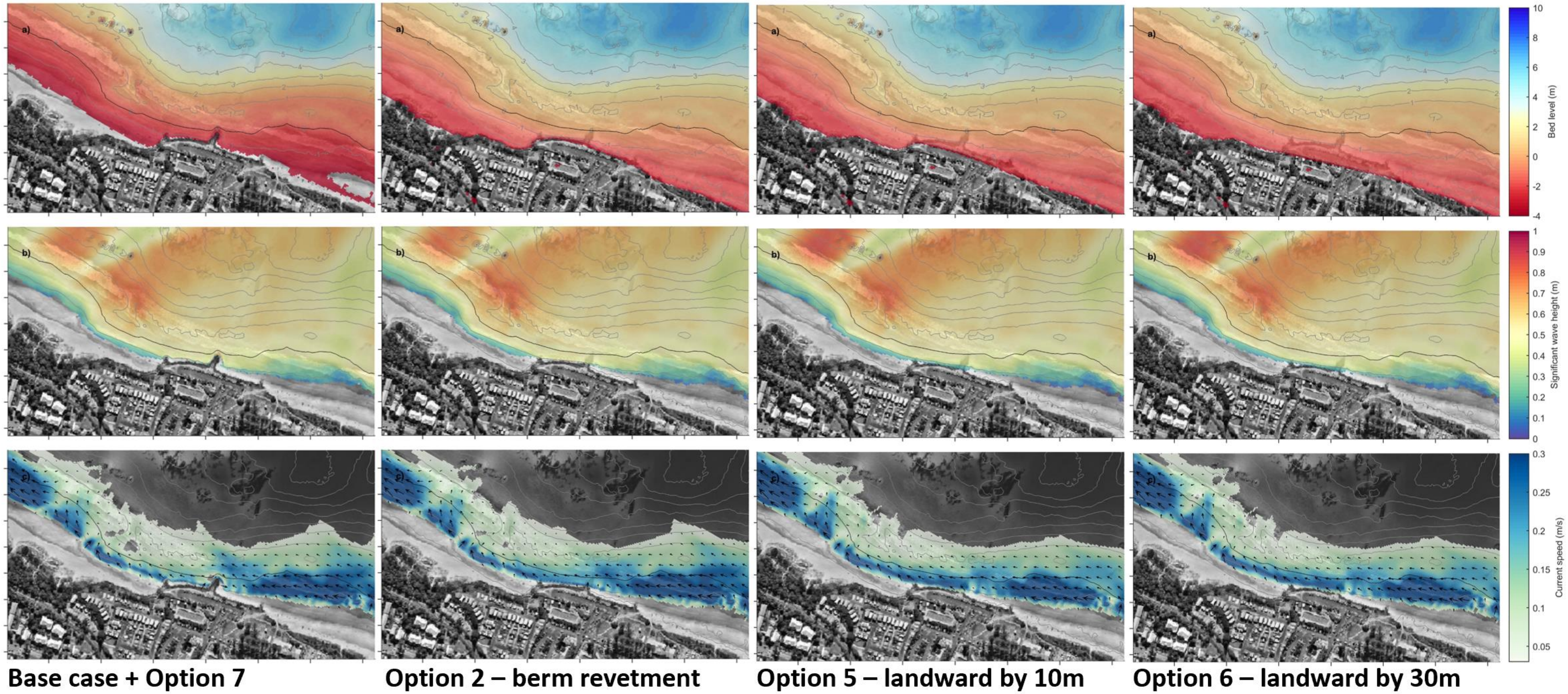

Figure 52. SWASH modelling results for basecase and project cases for S1 wave condition.

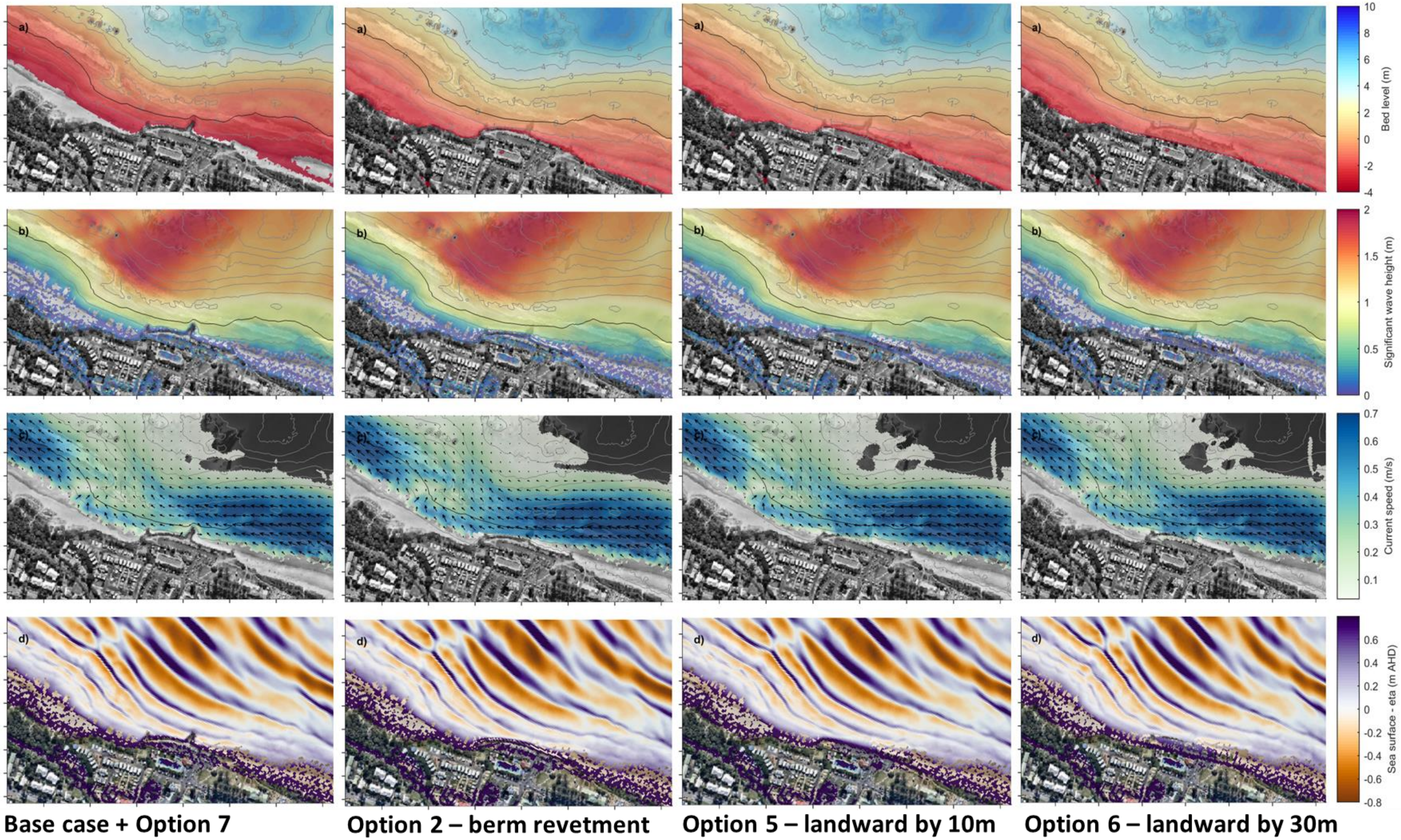

Figure 53. SWASH modelling results for basecase and project cases for S2 wave condition.

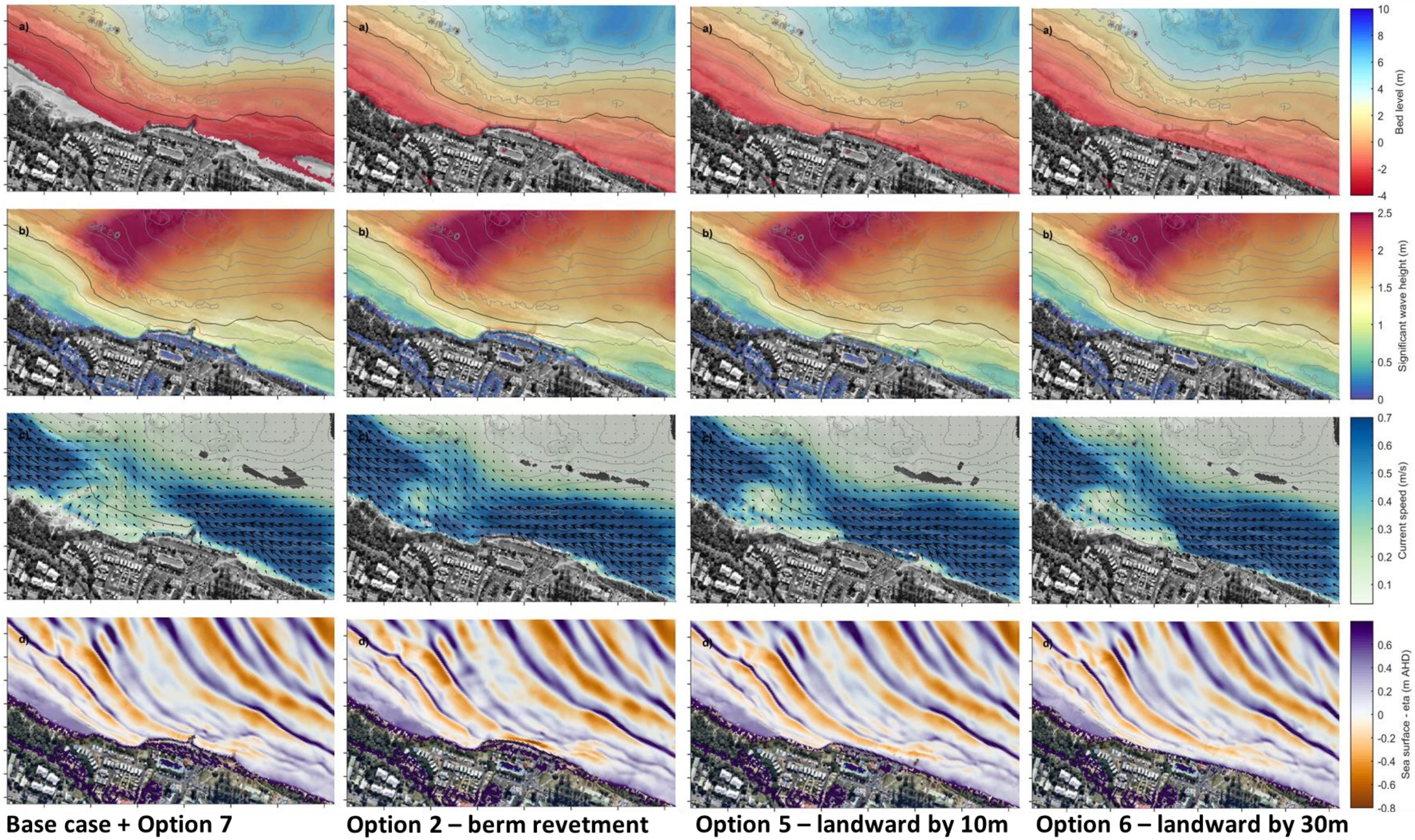

Figure 54. SWASH modelling results for basecase and project cases for S3 wave condition.

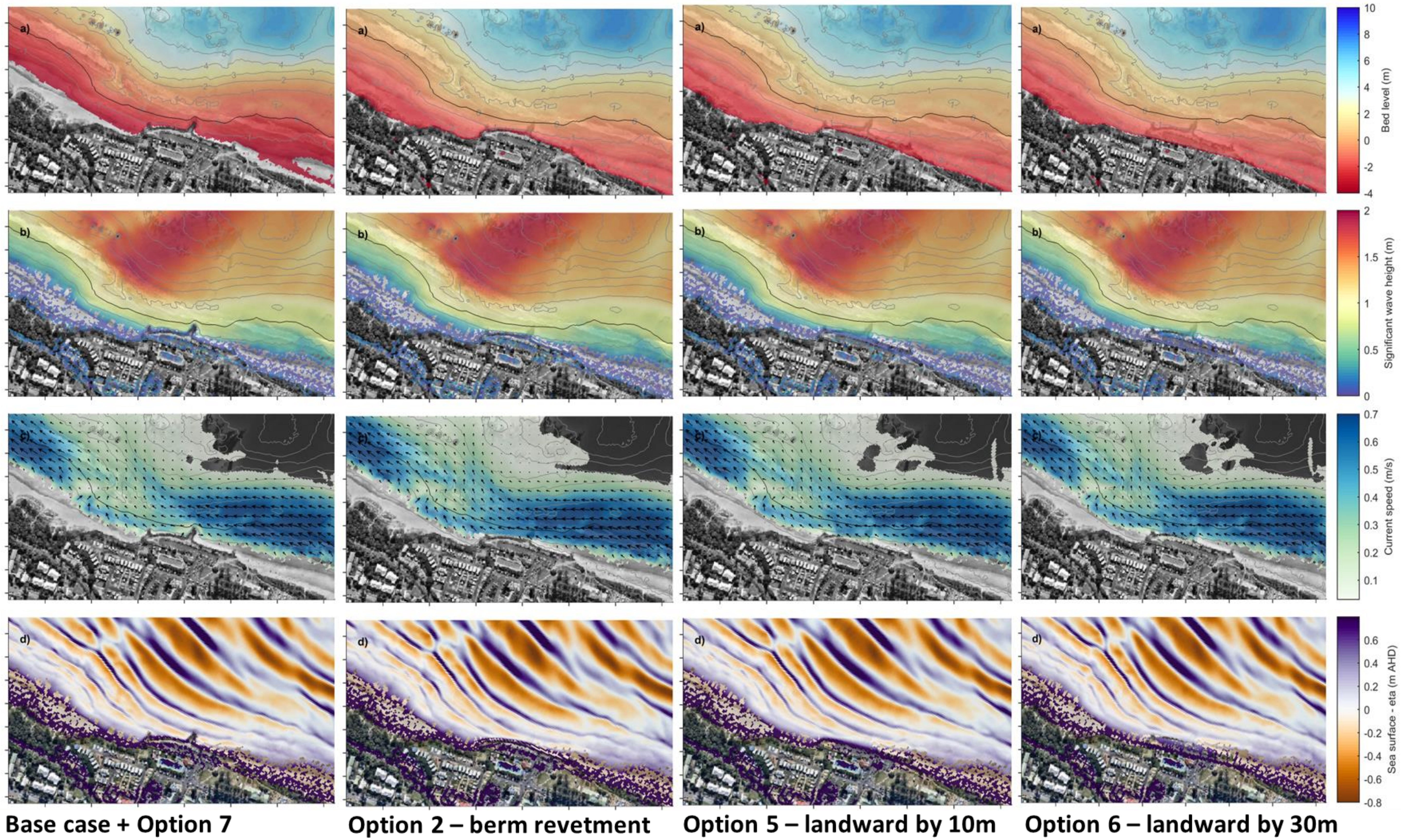

Figure 55: SWASH modelling results for basecase and project cases for S4 wave condition.

Table 21. Modelled wave height and flow velocity statistics at the observation points for S1 condition.

Location	Case	Wave height (m)		Flow velocity (m/s)			
		Hs	Hmax	Mean	Max	50^{th} %ile	98^{th} %ile
P2	Basecase & Option 7	0.18	0.29	0.80	1.97	0.77	1.58
	Option 2	0.19	0.24	0.96	2.18	0.95	1.80
	Option 5	0.20	0.27	0.84	2.32	0.80	1.67
	Option 6	0.20	0.27	0.84	2.32	0.80	1.67
P3	Basecase & Option 7	0.53	0.64	0.49	1.24	0.46	0.99
	Option 2	0.51	0.61	0.46	1.37	0.43	1.06
	Option 5	0.50	0.57	0.46	1.27	0.42	1.08
	Option 6	0.49	0.57	0.46	1.27	0.42	1.08
P4	Basecase & Option 7	0.49	0.57	0.52	1.83	0.39	1.48
	Option 2	0.47	0.58	0.47	1.71	0.37	1.39
	Option 5	0.49	0.58	0.52	1.51	0.43	1.26
	Option 6	0.49	0.58	0.51	1.52	0.44	1.25
P5	Basecase & Option 7	0.53	0.60	0.42	1.47	0.36	1.23
	Option 2	0.52	0.65	0.46	1.50	0.40	1.24
	Option 5	0.53	0.59	0.44	1.42	0.39	1.11
	Option 6	0.53	0.59	0.44	1.43	0.39	1.11
P7	Basecase & Option 7	0.48	0.89	0.17	0.80	0.15	0.52
	Option 2	0.46	0.85	0.17	0.80	0.15	0.50
	Option 5	0.47	0.87	0.19	0.90	0.17	0.52
	Option 6	0.47	0.87	0.20	0.90	0.16	0.52

Table 22. Modelled wave height and flow velocity statistics at the observation points for S2 wave conditions.

Location	Case	Wave height (m)		Flow velocity (m/s)			
		Hs	Hmax	Mean	Max	50th%ile	98th%ile
P2	Basecase & Option 7	0.31	0.42	0.86	2.26	0.79	2.11
	Option 2	0.30	0.43	0.93	2.46	0.91	2.04
	Option 5	0.31	0.41	0.97	2.88	0.97	2.10
	Option 6	0.29	0.41	0.88	2.63	0.86	1.97
P3	Basecase & Option 7	0.65	0.79	0.99	2.56	0.94	1.94
	Option 2	0.64	0.76	1.24	2.71	1.20	2.05
	Option 5	0.62	0.80	1.20	2.88	1.17	2.15
	Option 6	0.64	0.81	1.18	2.57	1.12	2.16
P4	Basecase & Option 7	0.55	0.69	1.13	3.33	1.02	2.62
	Option 2	0.63	0.78	1.15	3.24	1.05	2.68
	Option 5	0.56	0.85	1.18	3.22	1.10	2.57
	Option 6	0.53	0.75	1.31	3.27	1.23	2.80
P5	Basecase & Option 7	0.67	0.80	0.93	2.98	0.81	2.31
	Option 2	0.67	0.89	0.95	2.62	0.90	2.27
	Option 5	0.68	0.88	0.96	3.50	0.84	2.43
	Option 6	0.64	0.78	1.05	2.78	0.96	2.38
P7	Basecase & Option 7	1.35	1.95	0.58	2.37	0.53	1.38
	Option 2	1.37	1.95	0.59	2.41	0.53	1.43
	Option 5	1.37	1.98	0.61	2.48	0.56	1.43
	Option 6	1.38	2.00	0.61	2.44	0.57	1.41

Table 23. Modelled wave height and flow velocity statistics at the observation points for S4 wave conditions.

Location	Case	Wave height (m)		Flow velocity (m/s)			
		Hs	Hmax	Mean	Max	50th%ile	98th%ile
P2	Basecase & Option 7						
	Option 2						
	Option 5						
	Option 6						
P3	Basecase & Option 7						
	Option 2						
	Option 5						
	Option 6						
P4	Basecase & Option 7						
	Option 2						
	Option 5						
	Option 6						
P5	Basecase & Option 7						
	Option 2						
	Option 5						
	Option 6						
P7	Basecase & Option 7						
	Option 2						
	Option 5						
	Option 6						

7.6 Discussion

The SWASH model results provide detailed information on the transformation of waves over the Byron embayment and its shallow reefs. The model identifies wave energy hot spots and shadows emanating from the shallow reefs and the effect this wave pre-conditioning has on nearshore hydrodynamics. Previous studies and field observations demonstrate that alongshore surfzone currents, driven by wave radiation stresses caused by wave breaking, go westward at the project site. The SWASH model confirms this clearly showing the main current flows west north-west parallel to the coast. The significant wave focusing areas over the reefs (rock outcrops) affect this alongshore current by causing alongshore accelerations/decelerations which also influence the location and behaviour of rip currents.

Comparison of the SWASH modelling results allows the effects of the shortlisted JSPW design options on the nearshore wave and hydrodynamics to be predicted. The key outcomes are:

- All options have minimal and largely localised changes to nearshore wave conditions. Figure 56 shows wave height difference maps for the modelled project cases relative to the basecase. Most areas show no discernible change. The largest changes occur nearby the central groyne. For Option 5 and Option 6 less wave reflection is predicted due to the realignment of the revetment landward resulting in a reduction in wave heights (by up to 0.2m) just northeast of the groyne and along the western part of the JSPW. When waves encounter JSPW (basecase and Option 2) a portion of the wave energy is reflected into the water, increasing the wave heights in front of the structure. For option 5 and Option 6 wave-structure interactions are reduced as are wave reflections (i.e. wave energy is absorbed), reducing wave heights in front of the structure.
- Option 2 results in increases in wave heights along the western revetment as removal of the groyne without the landward shift of the revetment slightly increases wave reflections and heights here. Option 7 is not shown as no discernible changes in wave heights compared to the basecase would be expected.

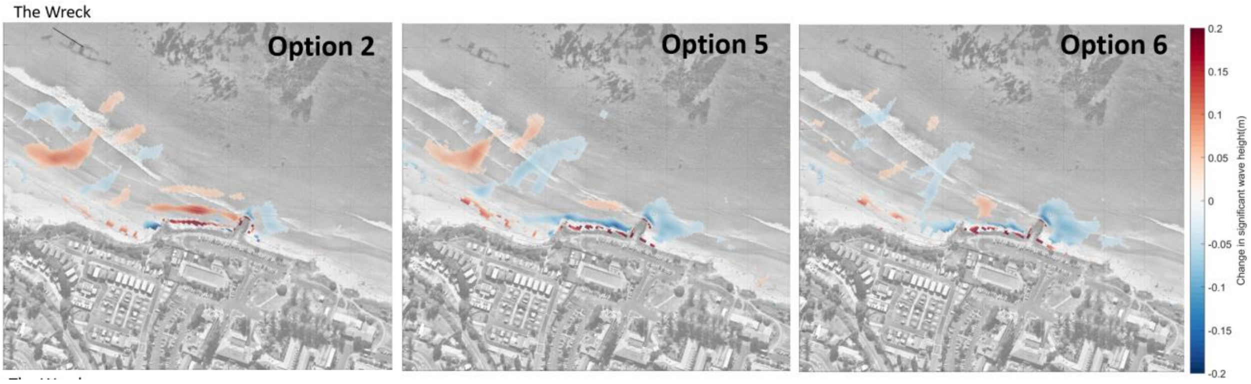

Figure 56. Wave height difference maps for wave condition S2 comparing each project case to the basecase.

- Alongshore surfzone currents are changed significantly from all options except Option 7. At higher tides or during low sand levels on the beach, the groynes of the basecase and/or Option 7 act as a barrier decreasing current speeds downdrift of the JSPW. Figure 57 shows current speed difference maps for the modelled project cases relative to the basecase. The groynes are removed in Option 2, Option 5 and Option 6 and current speed increases by ~10% comparing to basecase at P3 and P4, with higher flow speeds closer to the revetment alignments in each project case. Current speed increases for these options are more evident with high water level and high wave height conditions (S2 and S3 wave conditions). As discussed further below, changes to surfzone currents are a key factor in the expected increase in sand bypassing for Option 2, Option 5 and Option 6.

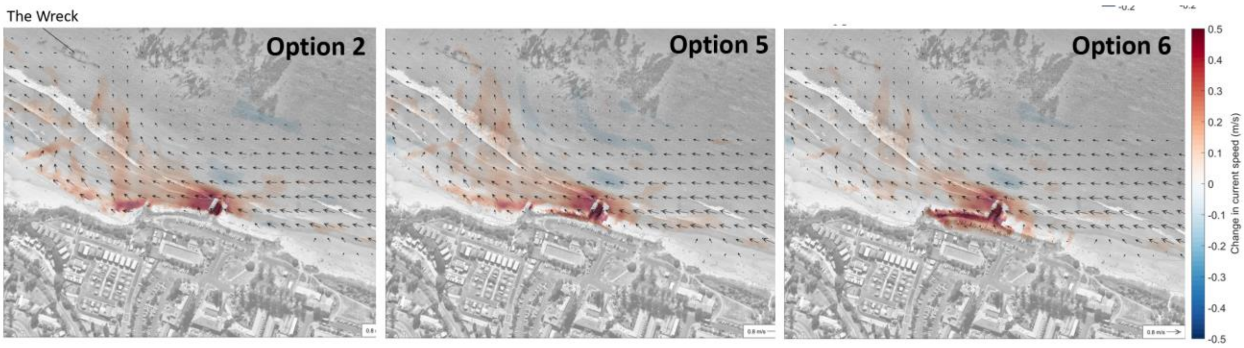

Figure 57. Current speed difference maps for the S2 condition for each project case to the basecase.

- Further examining the nearshore currents at three cross-shore profiles, as shown in Figure 58 for the S2 condition, reveals more details about the specific nearshore hydrodynamic changes expected for the project cases. In alongshore direction nearby the JSPW:
  - Updrift (C1): the modelled current speed profiles are essentially the same for the basecase and each of the project cases
  - Central groyne (C2): is where most of the differences are observed. The peak current speeds of just under 0.7m/s are largely unchanged. However, in all simulated project cases flow speeds in the shallow subtidal and swash zone behind the groyne (chainage 40m) (i.e., noting that the groyne has been removed in each of the project cases) are substantially increased with a corresponding minor decrease in speeds observed in the deeper parts of the profile (chainage 100m or more). In terms of the overall net discharge (m^3/s) across the profile there is very little change between the basecase and the project cases when the depths are considered (i.e., the project cases only increase the discharge by 3%, 5% and 6% respectively for Option 2, 5 and 6). In the basecase the central groyne forces flow around the 30m structure length out into deeper water which does not occur in the project cases.
  - Downdrift (C3): Small changes in current speeds can be observed at the downdrift end where these follow a similar redistribution of the flow field. In each of the modelled project cases the seaward flowing rip current is moderately increased under the S2 conditions.
- The SWASH modelling results for Option 5 and Option 5 show little discernible difference across nearshore wave and hydrodynamics interactions. As explained in the points above these two options have similar predicted outcomes for changes to local wave patterns, surfzone currents and the cross-shore profile of flow around the structure.

Comparison of the SWASH modelling results allows the effects of the shortlisted JSPW design options on surf amenity and swimmer safety to be predicted. The key outcomes are:

- Statistical outputs from the SWASH simulations at The Wreck surf spot during typical high tide surfing conditions (e.g., S1 and S2, see Point 7 in Table 21 and Table 22) shows that the project cases do not significantly affect the wave heights or currents in this high value recreational area. To further examine the simulated wave breaking patterns snapshots of the instantaneous sea surface showing individual waves for the S2 conditions are shown in Figure 59 . The figures and other similar snapshots do not show a significant change in the sea surface patterns around The Wreck. In terms of the amount of wave breaking around The Wreck surf spot the instances of wave breaking were summed for each simulation and compared to the basecase, which indicated:
  - Option 2 retained 92% of the wave breaking simulated for the basecase (i.e., 8% reduction in wave breaking).

- Option 5 retained 82% of the wave breaking simulated for the basecase (i.e., 18% reduction in wave breaking).
- Option 6 retained 85% of the wave breaking simulated for the basecase (i.e. 15% reduction in wave breaking).

- Similar results, with no significant change in wave heights, current or wave breaking pattern, are observed in the SWASH simulations for areas immediately seaward of the JSPW and to the east of JSPW. This area is known to provide good surf from time to time. But as discussed in Section 3.7, good surfing conditions here are likely to be more related to the wave pre-conditioning (owing to Middle Reef), the distribution of surfzone coffee rock and headland bypassing which results in this 'bulge' morphology.
- It is acknowledged that the S2 wave condition combined with a water level of 1.1m is a rare combination of conditions. The higher water level was selected to promote more wave-structure interactions with the JSPW (and the design options) which is considered conservation from a surf amenity impact perspective. To ensure a broader range of conditions were tested, an additional scenario using the S4 conditions shown in Table 20 (i.e., Hs = 1.5m, Tp = 11 sec, Dp = NE and tide = 0m AHD) was simulated. Results are shown in **Figure** 60. The interaction between waves and currents with JSPW and the options is also negligible with also negligible differences in wave breaking patterns and currents around The Wreck or to the east of the JSPW.
- As seen in Figure 57, the project cases all show increased nearshore current speeds over a localised area nearby the central groyne. These are due to the removal of the groyne and are not greater than nearshore current speeds seen along other adjacent areas. In the basecase the nearshore current flow swiftly towards and around the groyne, meaning the removal of the groyne which acts as an obstacle, as is the case for all but Option 7, would be seen as a minor but positive improvement in swimmer and surfer safety. It is suggested that these results be discussed with local NSW Surf Life Saving representatives in the next evaluation stages to understand their opinion on the swimmer safety implications.
- The SWASH modelling results for Option 5 and Option 5 show little discernible difference in regard to the surf amenity and swimmer safety outcomes predicted.

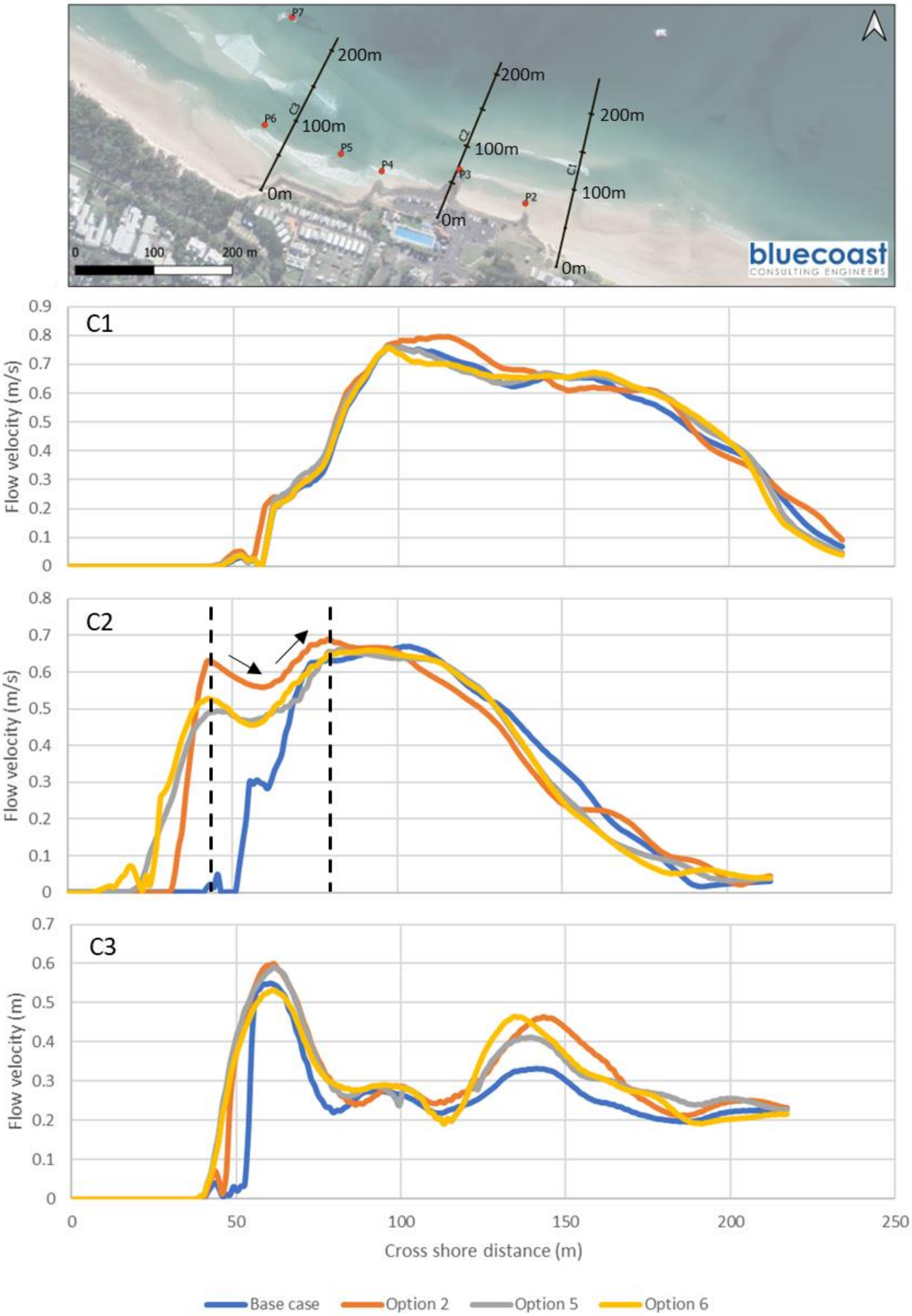

Figure 58. Cross-shore velocity profiles (C1, C2 and C3) for the basecase and project cases for S2 wave and water level conditions.

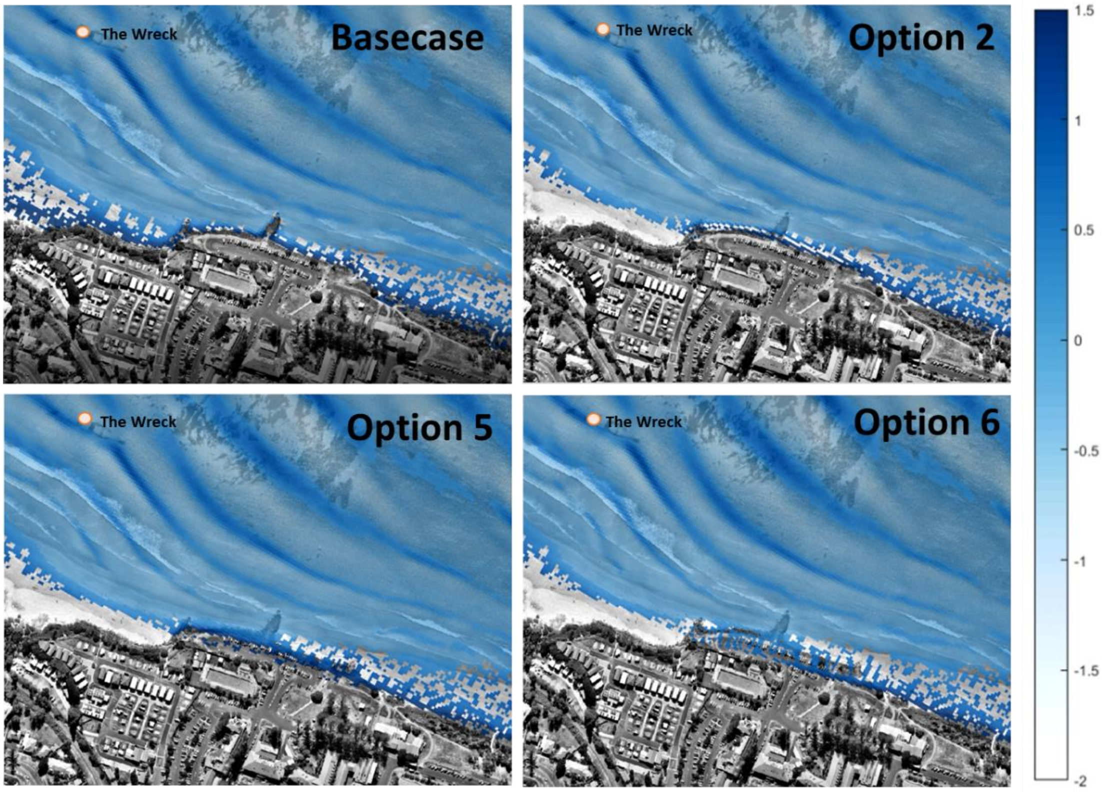

Figure 59. Snapshot of modelled sea surface for the basecase and project cases under S2 wave condition.

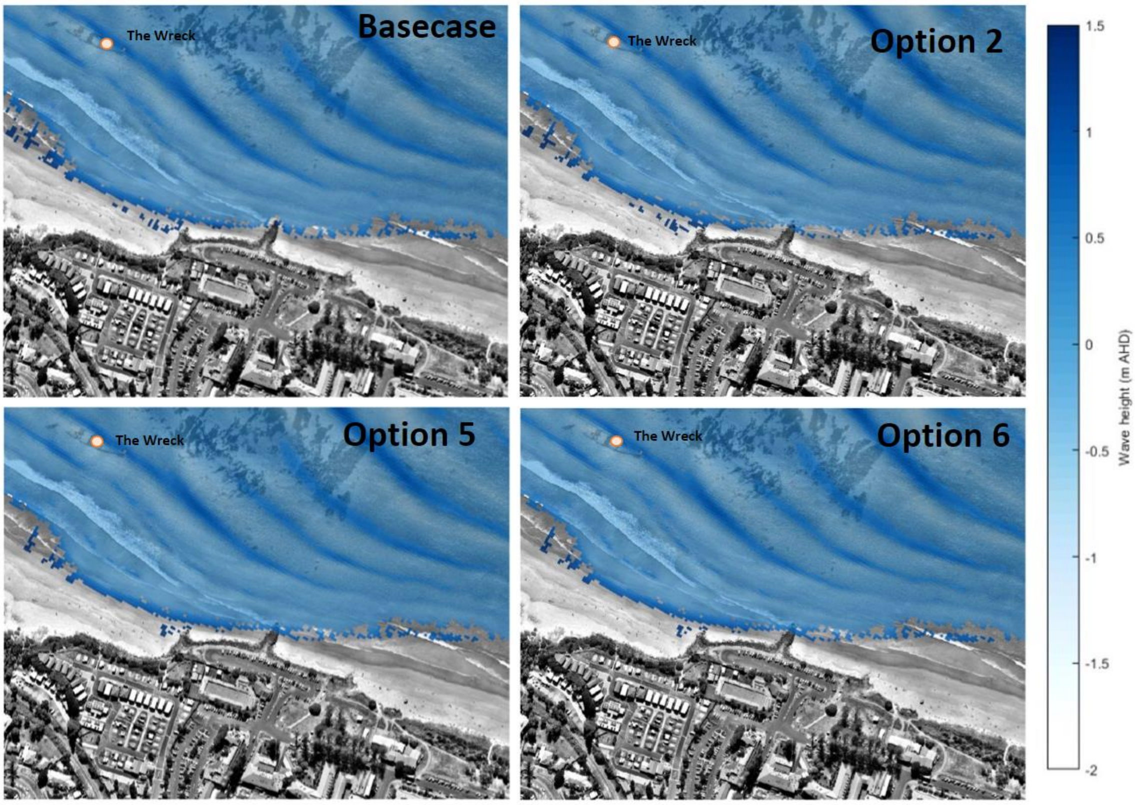

Figure 60. Snapshot of modelled sea surface for the basecase and project cases under S4 wave condition.

8. XBeach modelling

8.1 Introduction

High-resolution, non-linear nearshore wave simulations were undertaken using the XBeach model (Roelvink et al. 2009). XBeach model was applied to estimate wave overtopping along the JSPW for both the basecase and project cases.

8.2 Model setup

A one-dimensional XBeach model was adopted for this study. The 2018 Coastal LiDAR survey was used as the basis from which to create a representative cross-shore profile. Figure 61 shows the location of the cross-shore profile, the elevation profile and the grid. The cross-shore resolution ranges from 2m at the offshore end of the profile to 1m at the shoreward end.

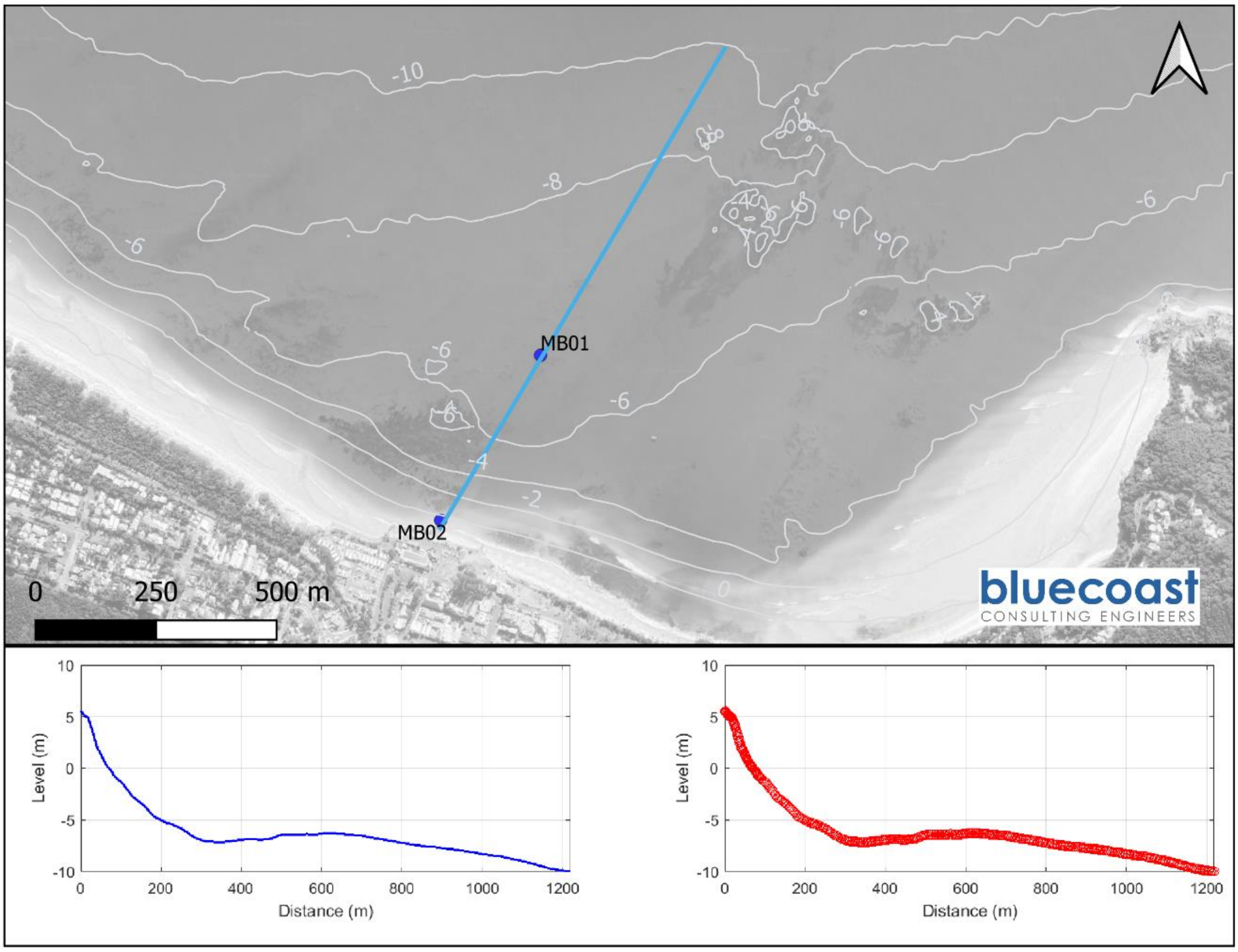

Figure 61. Adopted coastal profile location, XBeach model profile elevation (blue) and 1D grid (red).

The model was set up in surf beat mode which estimates short-wave motion by solving the wave action equation. Surf beat mode is used for coastal morphological response modelling with long waves, run-up and run-down of long waves important to coastal erosion during storms included. Sensitivity tests were carried out to assess the impact of model mode over a 2-day period during the peak of a wave event on 12 February 2020. Both surf beat and non-hydrostatic (NH) mode of the XBeach model was undertaken. In the non-hydrostatic mode, depth-averaged flow due to waves and currents are computed using the non-linear shallow water equations, including a non-hydrostatic pressure. In this computationally demanding mode, incident-band (short wave) run-up and over wash are fully resolved and have been compared to the results from the surf beat mode. As expected, it was found that the surf beat mode provides accurate results for this investigation.

8.3 Model calibration

A model calibration exercise was undertaken to confirm XBeach can reasonably reproduce the observed wave, water level and coastal response observations during storm conditions. The XBeach model calibration focused on the passage of Tropical Cyclone Uesi (Category 3) which developed in the South-West Pacific Ocean and travelled towards the East coast of Australia. It brought large swells to the Queensland and NSW coasts. This event was well monitored by project specific measurements including the MB01 and MB02 sites being deployed, and pre- and post-storm drone surveys collected (see Section 2.3).

Firstly, the model was calibrated to observed wave height and water level measurements at MB01 and MB02, respectively. This involved several simulations to test and tune key hydrodynamic model parameters until a good agreement between modelled and observed data was achieved. Figure 62 shows the timeseries comparison of the modelled and measured wave heights at MB01 and water level at MB02 for the TC Uesi event. The plot shows generally good agreements, with:

- a peak significant wave height of 3.1m and 15s of peak wave period at MB01 recorded compared to a modelled significant wave height of 2.8m (within 9%) and 15s peak period
- modelled wave setup was 0.8m, which is in good agreement with the measured wave setup (see **Appendix A**).

The calibrated model adopted the wave breaking model of Roelvink (1993) with a gamma factor of 0.8. Wave dissipation by bottom friction was modelled using a friction coefficient of 0.01.

Figure 62. Comparison between measured and modelled significant wave height at MB01 .

Water level was validated for the peak of the TC Uesi event using photographs captured during the event by residents. Table 24 shows XBeach model results for mean overtopping discharge, maximum overtopping discharge and maximum water level. Model results represent well enough what the overwash in the photograph shows (see Figure 59).

Table 24. Modelled mean overtopping discharge, maximum overtopping volume and maximum water level for TC Uesi peak event.

Condition	Mean Qx (l/s/m)	Max Q_{max} (l/m)	Max water level (m AHD)
TC Uesi	0.34	2,003	4.9

Figure 63. Photograph from TC Uesi showing wave overtopping at JSPW.

8.4 Adopted wave and water level scenarios

When assessing wave overtopping at the JSPW it is important to consider the joint probability of nearshore wave conditions and ocean water levels. As such, a joint probability analysis was completed using the 40-year wave hindcast at MB01 and measured water levels from MHL's Tweed Heads offshore water level gauge. The analysis results are presented in Figure 64 with a summary of the adopted 100-year ARI joint wave conditions and water levels adopted for the overtopping discharge assessment presented in Table 25. A one-hour duration has been assumed for the peak of the joint water level and wave event.

Predicted sea level rise was considered based on IPCC AR6 sea level rise projections (for Yamba, NSW). The mid value between the SSP2.6 (50th percentile) and SSP8.5 83rd percentile was adopted for each time horizon: sea level rises of +0.26m and +0.78m for the 2050 and 2100 planning periods, respectively.

Table 25. Overview of adopted 100-year ARI joint wave and water level scenarios for the overtopping assessment.

Planning period	Still water level (m AHD)	Significant wave height (m)	Peak wave period (s)	Duration (hours)
Present day	1.15	3.1	13	1
2050	1.41	3.1	13	1
2100	1.93	3.1	13	1

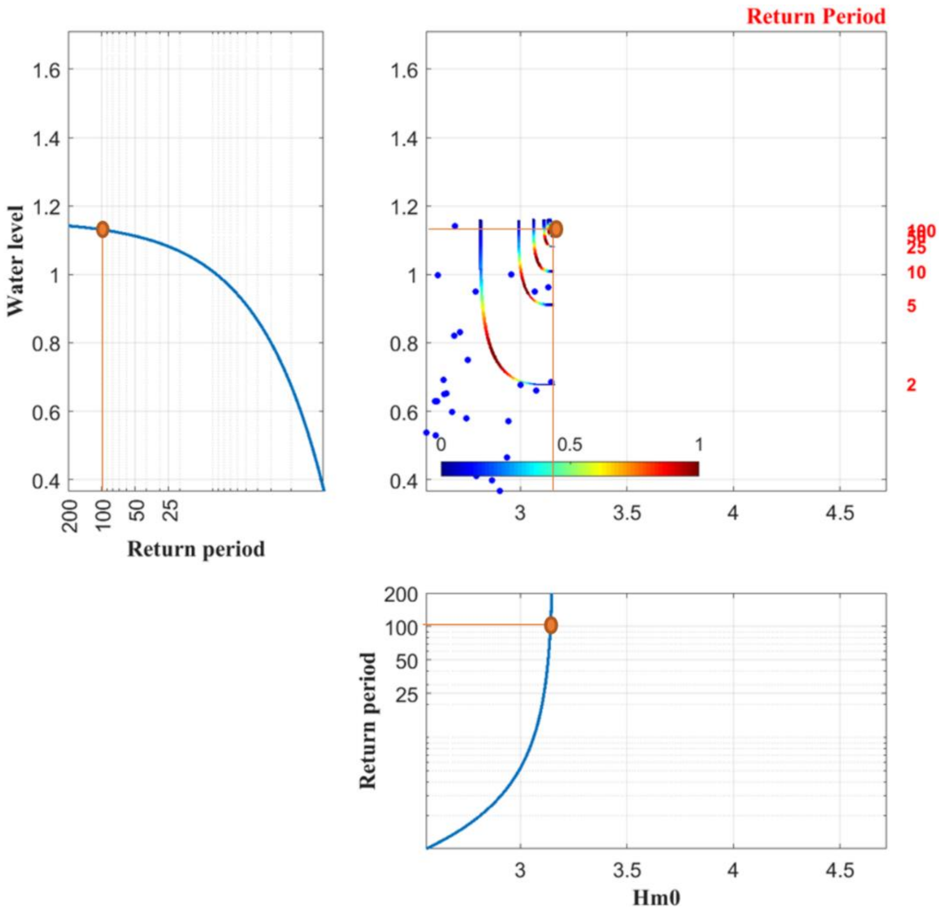

Figure 64. Joint probability of nearshore waves at MB01 and water levels offshore Tweed Heads. Orange point shows 100-year ARI scenario.

8.5 Overtopping results

Wave overtopping of the basecase, and project cases has been assessed for the adopted 100-year ARI joint wave and water level probability. A summary of the overtopping discharge volumes for each scenario and project case, as well as basecase is provided in Table 26. Mean overtopping volume (Qx) in litres per seconds per metre (l/s/m), maximum volumes (Q_{max}) in litres per metre (l/m) and the maximum water level height during the one-hour simulations are provided. When the water level exceeds the height of the structure's crest, which is 4.9m AHD, wave overtopping takes place and water discharge (Qx) flowing landward is expected, posing potential threat to human safety and infrastructure. For a given mean overtopping discharge (Mean Qx)the smaller waves give only a minor overtopping volume, whereas the larger waves give many cubic metres of overtopping water in one wave and their influence thus better described by the maximum volumes (Max Qx)

Eurotop (2018) provide guidance on safe mean and maximum overtopping volumes in consideration of impacts to the structure as well as people and infrastructure in the lee, see Table 27.

Table 26. Overtopping discharges (Qx) and peak water level from the XBeach modelling.

Planning period	Case	Mean Qx (l/s/m)[1]	Max Qx (l/m)[1]	Max water level (m AHD)
Present day	Basecase	**0.33**	**2,816**	4.9
	Option 2	0.01	**1,794**	4.9
	Option 5	0.00	0	4.4
	Option 6	0.00	0	4.1
	Option 7	0.29	**2,653**	4.9
2050	Basecase	**0.35**	**2,984**	4.9
	Option 2	0.03	**1,815**	4.9
	Option 5	0.01	260	4.9
	Option 6	0.00	0	4.4
	Option 7	**0.32**	**2,786**	5.1
2100	Basecase	**0.40**	**3,386**	5.1
	Option 2	**0.32**	**2,219**	5.1
	Option 5	0.01	489	5.1
	Option 6	0.00	300	5.0
	Option 7	**0.39**	**3,080**	5.1

[1] in **bold** when Max Qx and/or Mean Qx exceed the ‘safe’ levels provided in Eurotop 2018

Table 27. Overview of safe overtopping volumes provided in EurOtop (2018).

Hazard type and reason	Offshore significant wave height (m)	Mean discharge Qx (l/s per m)	Max volume V_{max} (l per m)
Rubble mound structure (no damage)	>5	1	2,000 to 3,000
Rubble mound structure (rear side designed for wave overtopping)	>5	5-10	10,000 to 20,000
People at seawall (clear view of the sea)	3	0.3	600
Cars on seawall (close behind crest)	3	<5	2,000

8.6 Discussion

The XBeach results and Eurotop (2018) guidance indicate the following key considerations regarding wave overtopping at JSPW:

- Overtopping of the basecase during the present day 100-ARI wave and water level conditions far exceed the safe volumes for people at seawall. These exceedances of safe conditions are expected to get worst with sea level rise. Damage to the structure and to landward infrastructure may also occur under present day sea levels during these rare events.
- Option 2, Option 5 and Option 6 significantly reduce overtopping of the modified structure crest improving public safety outcomes to arguably acceptable levels. Being set back further from the shoreline, Option 5 and Option 6 perform the best for overtopping. While there are some difference between Option 6 and Option 5, both satisfy the EurOtop (2018) criteria for mean overtopping discharge and maximum overtopping volume relating to structural damage, safety to people and safety to cars (i.e. overtopping is not a particular differentiator for these options).
- Option 7 reduces overtopping of the structure to safe values for the present situation but surpasses the limit of safety for sea level rise scenarios. Removing or restricting car parking along the front row of parking spaces would reduce the likelihood of people and cars being at risk from overtopping.
- As sea levels rise, waves have a greater chance of overtopping the structure. Option 2 and Option 7 provide public safety for current situation but, without future adaptation, overtopping would exceed safe levels by 2050 for Option 7 and by 2100 for Option 2. Option 5 and Option 6, however, are within safe levels out to 2100 based on the sea level rise scenarios adopted .

9. Quantified conceptual sand movement model

9.1 Introduction

A quantified conceptual model (quantified model) of sand movement was developed to draw conclusions regarding the expected long-term coastal response (30 years or greater) of the adjacent beaches in response to the shortlisted design options for the JSPW. The model brings together the various investigations and lines of evidence for an overall understanding of coastal processes within the project study area. The model estimates the extent of the expected response of the coastal environment at Main Beach (to the east) and Belongil Beach (to the west) following construction of each of the shortlisted design options including:

- the amount of sand bypassing the JSPW
- compartment volume and shoreline changes to adjacent beaches.

The model is focused on medium to long-term changes (i.e., timescales of change greater than 1-year) and does not specifically consider seasonal and storm-induced coastal responses. This is considered appropriate in assessing expected long-term coastal response (30 years or greater) of the adjacent beaches in response to the shortlisted design options for the JSPW.

9.2 Byron embayment conceptual sand movement model

Figure 65 provides a graphical overview of the quantified conceptual model of sand movements (quantified model) in the Byron region. The model is based on the Byron embayment sand budget as well as the assessment of each of the sand movement pathways, sources and sinks presented in the baseline geomorphic assessment (see Section 3). Table 28 provides an overview of the annual average sand transport rates.

In developing the sand budget and movement model, a control volume defined in profile from the crest of the dune down to:

- -20m AHD along Tallows Beach and north of Belongil Creek
- -10m AHD within the Byron embayment, was adopted.

The net onshore supply of 30,000m^3/yr in the Byron embayment is assumed to occur shoreward of the -10m AHD control volume and therefore accounts for the long-term net sand loss of 30,000m^3/yr observed in surveys and discussed in Section3.3.2. The model does not account for the sand loss seaward on the -10m AHD depth contour. If the sand losses to the -15m AHD are considered, the total long-term sand loss rate is approximately 60,000m^3/yr.

Table 28. Summary of adopted annual transport rates in quantified conceptual sand movement model.

Zone [sand cell codes, see Figure 8]	Annual net longshore sand transport rates (m^3/yr)				Onshore/ offshore transport rates (m^3/yr)
	Littoral pathway	Cross-embayment pathway	TOTAL	Degree of annual variability	
Tallows (north) [TB]	450,000[1]	na	450,000	Moderate	
Cape Byron [CaB]	400,000	na	400,000	Extremely high	-50,000[1]
Wategos Beach [WB]	160,000[2]	240,000[2]	400,000	Very high	5,000[3]
Clarkes Beach [CB]	285,000[2]	120,000[2]	405,000	High	10,000[3]
Main Beach [MB]	330,000[2]	85,000[2]	415,000	High	5,000[3]
Belongil Beach (1) [BB-1 & BB-2]	380,000[2]	40,000[2]	420,000	Moderate	5,000[3]
Belongil Beach (2) [BB-3 & BB-4]	425,000	na	425,000	Moderate	5,000[3]
North of Belongil Creek	430,000	na	430,000	Moderate	-

Note: 1. Derived from literature, BMT (2013) and PWD (1978), respectively.

2. Split between littoral pathway derived from contemporary sand budget analysis and explained in Section 3.6.

3. Derived from long term sand budget analysis.

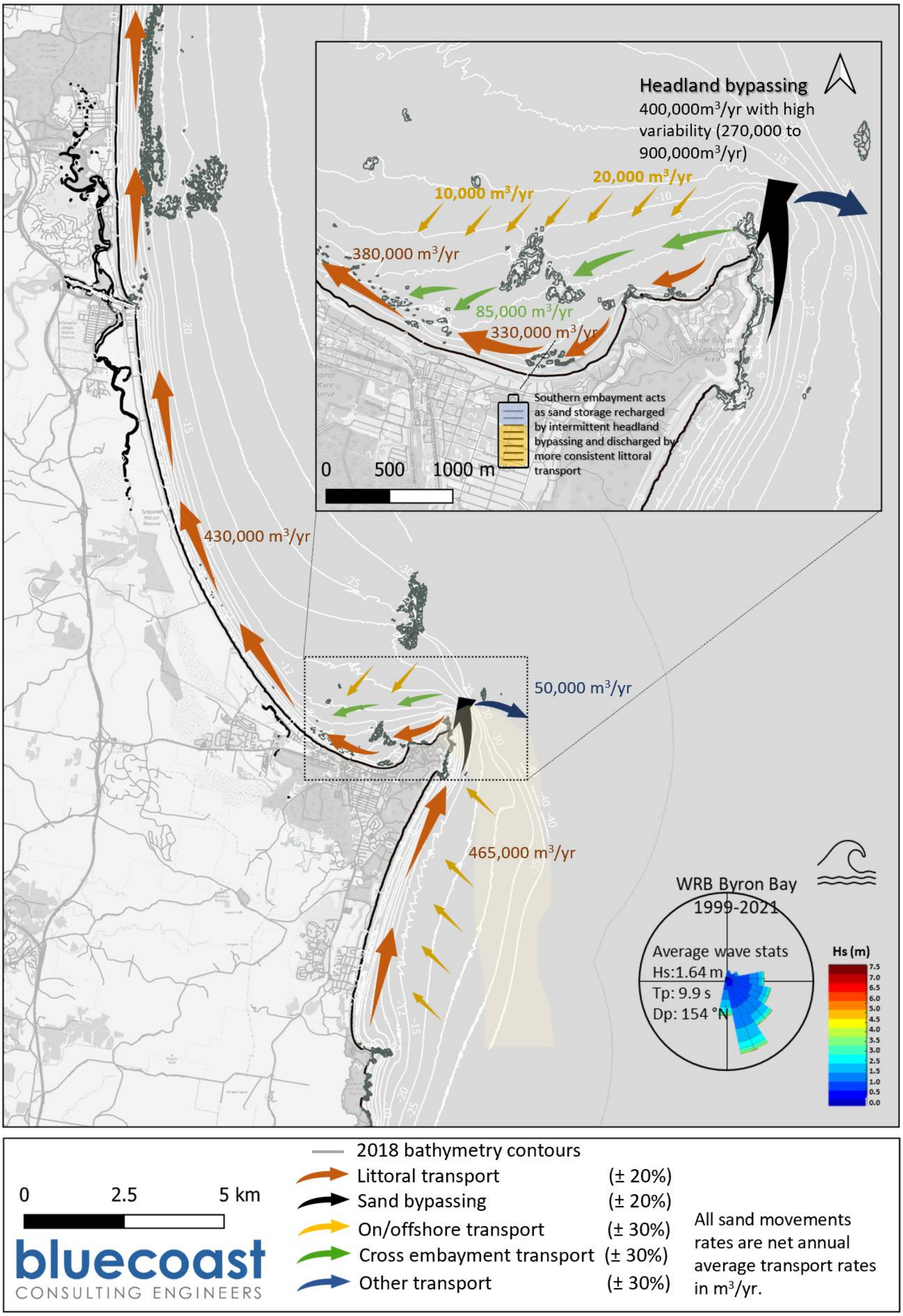

Figure 65. Quantified conceptual model of sand movements through the Byron embayment.

9.3 Coastal response to shortlisted design option for the JSPW

9.3.1 Methodology

The MBSP involves the construction of one of the shortlisted design options for the JSPW (project cases). By drawing on the geomorphic and numerical modelling investigations, a shoreline model specific to the Byron embayment, has been developed. This bespoke shoreline model was then used to predict the response of the coastal environment to each project case.

The project cases are described in Section 4.2 and further documented in the MBSP's Concept Design Development Report (Bluecoast, 2020b). Option 2, Option 5 and Option 6 involve the removal of the groynes, while Option 5 and Option 6 also realign the rock revetment landward by 10m and 30m, respectively. Option 7 would result in the least changes to the existing footprint of the structure as it is intended as an upgrade to bring the structure in line with contemporary engineering standards. For the shoreline modelling Option 7 has been assumed to result in a 3m seaward[5] extension of the structure to account for the additional rock armouring required to achieve a more robust structure.

Due to its north facing location within the Byron embayment and being on the downdrift edge of the main zone of influence of headland bypassing standard one-dimensional shoreline modelling techniques[6] were tested and found to be unreliable. The main reasons being that:

- Most shoreline models (e.g., GENESIS, LITPACK, UNIBEST and EVO-MOD) are based around established equations or longshore sand transport (e.g., CERC, 1984 and Kamphuis, 1991). The transport curves are a sine curve function of the wave angle, with a maximum at roughly 45°. For relative angles beyond this critical angle, longshore transport decreases for increasing angles and the morphological behaviour of the coastline becomes fundamentally unstable. Within Byron's southern embayment (e.g., The Pass) wave crests approach the shoreline almost perpendicular and therefore far exceed this high-angle instability. There are two recently developed shoreline models (i.e., ShorelineS and Coastal Evolution Model) that apply a special treatment to high-angle instability, but these models do not address the second limitation below.
- Sand supply to the southern embayment is highly variable being controlled by headland bypassing around Cape Byron. This is a complex process for which there are no known standard one-dimensional shoreline models able to confidently simulate the variable flow of sand and sand slugs that go around these features.

To overcome these limitations, a bespoke shoreline modelling approach was developed. The project specific model combines observational data and longshore sand transport equations. The following steps describe the model's establishment including the underlying assumptions used:

1. The CERC3 longshore sand transport (LST) equation was used at Belongil Beach, around 1,000m downdrift of the JSPW (CERC, 1984). At this northern location, the shoreline conforms with the underlying assumptions required for the longshore transport equation to be applied and, when calibrated, sufficiently describes rates of alongshore sand movement. The calculation used wave heights and directions extracted from the 40-year wave hindcast at the 4m depth contour. The annual average rate was calibrated to agree with 378,000m^3/yr being the littoral transport rate (i.e., LST) at this location from the quantified conceptual model. The annual LST at Belongil Beach is the littoral outflow of sand from the model and is referred to as Q_{out}.

[5] Footprint changes subject to detailed design.
[6] It is noted that shoreline modelling can also be made with complex two-dimensional horizontal (2DH) process-based morphological models, as was shown for the recent case of the 'Sand Engine' in Holland (Luijendijk et al., 2017) as well as for other complex coastal forms. However, this comes at great computational expense, requires detailed field data and is beyond the scope of this project.

2. An annual timeseries of volume change in the Main Beach (ΔV_{MB}) and Belongil Beach (ΔV_{BB}) compartments was generated by converting the DEA shorelines to compartment volume change. This was done by multiplying the annual mean shoreline position by:

    a. the active profile height (i.e., 7m, being 3m height on the sub-aerial beach and -4m being the toe of the surfzone slope)

    b. length of the beach compartment.

    The annual volume change time series was validated against volume changes determined from the analysis of the 2002, 2011 and 2018 surveys. Missing volumes in the 2002 survey were filled using volumes taken from the photogrammetry differences. Good agreement between the calculated and surveyed volumes was achieved, as shown in Figure 66.

    By adopting the DEA shorelines, the model adopts a 31-year analysis period from 1988 to 2019 and a timestep of one-year.

3. The annual sand bypassing rate (Q_{bypass}) and longshore transport inflow rate into the model domain (i.e., Q_{in} the LST ~800m updrift of the JSPW) were simulated by working in an updrift direction starting from Q_{out} and using the DEA derived annual volume changes. This resulted in a completed description of the volume changes and sand flows between the two beach compartments for the basecase or existing JSPW condition. The modelled beach volume changes and LST rates for the basecase are presented in Figure 66 and Figure 67, respectively.

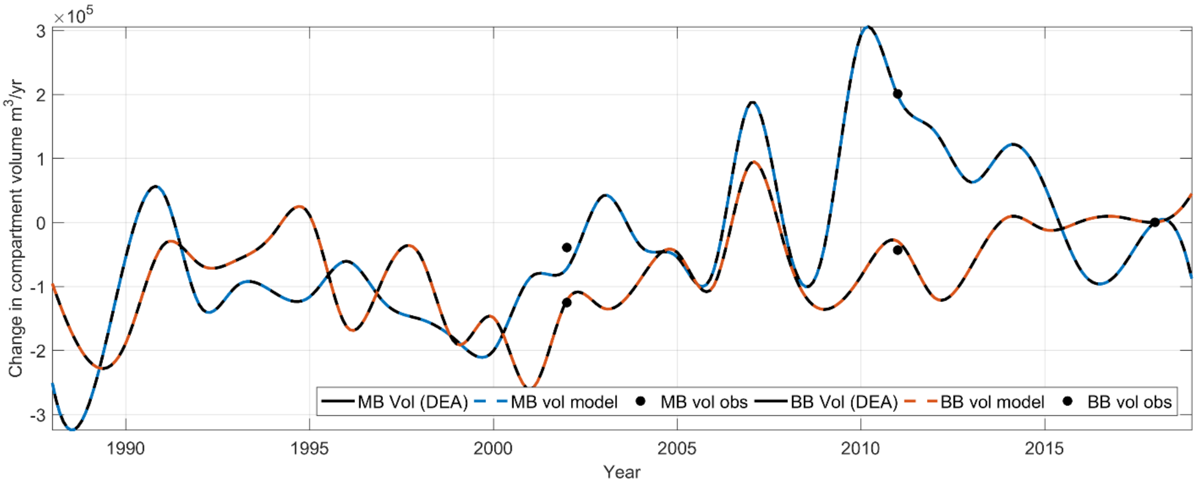

Figure 66. Calculated beach volume change (m^3/yr) for the Main Beach (MB) and Belongil Beach (BB) compartments.

Note: Observed beach volume changes derived from survey shown as black dots.

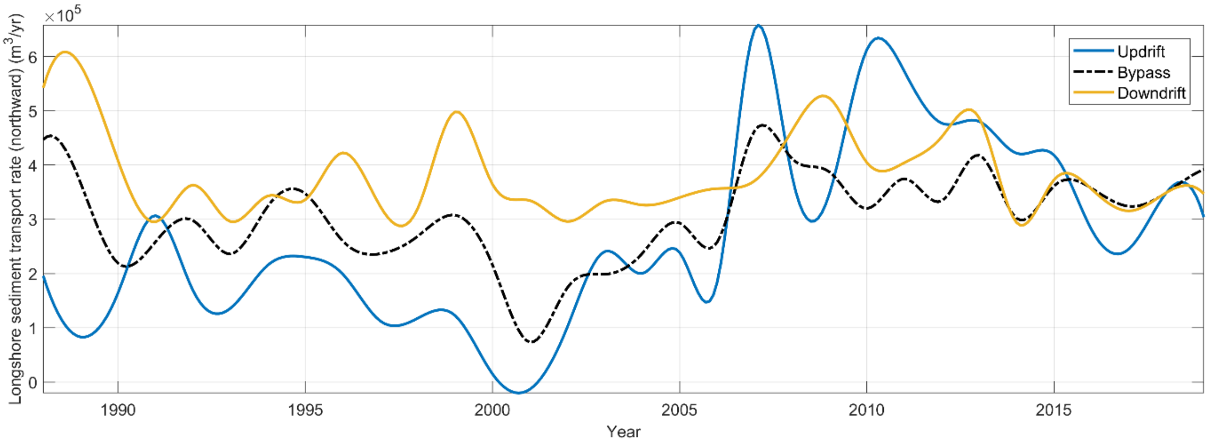

Figure 67. Modelled longshore sand transport rates (m³/yr) for Belongil Beach (downdrift), Main Beach (updrift) and bypassing the JSPW.

The higher degree of variability observed on the updrift LST rates compared to the downdrift LST rates (see Figure 67) provides further insight into the need to adopt the approach presented herein.

4. The relative degree of sand bypassing for each of the project cases was then modelled by applying a factor to Q_{bypass}. The bypass factor for each shortlisted design option was developed from:

   - The results of the SWASH simulations presented in Section 7 whereby the average relative discharge across each JSPW design simulation (see example in Figure 68) was calculated.
   - The geometry of the shortlisted design option relative to the position of the updrift (Main Beach) shoreline and the basecase as defined by the tip of the central groyne. When the Main Beach shoreline was fully accreted the model assumed equal bypassing for basecase and project cases (i.e., bypass factor = 1). When the modelled Main Beach shoreline was landward of the groyne tip the relative shoreline and option geometry was used to calculate the bypass factor for each project case. The calculated bypass factors considered the context of the local wave and water level climate to arrive at the annual bypass factor for each timestep.

The bypass factor was then applied to sand flows in the model to calculate the updrift and downdrift beach compartment volumes for each year from 1988 to 2019. Cumulative changes were considered by calculating the bypass factor at each timestep based on the previously calculated annual Main Beach shoreline. The simulated annual timeseries of project case beach volumes was then converted back to shoreline change on a shoreline-by-shoreline basis. From these results the effect of the options on the adjacent beaches could be estimated.

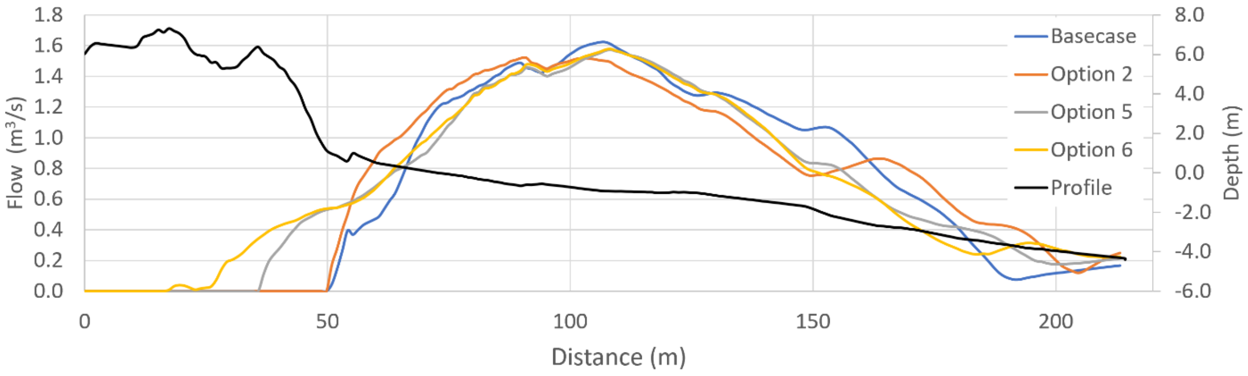

Figure 68. SWASH results showing nearshore flow (m^3/s) across the beach profile (black line) adjacent to the central groyne of the JSPW for basecase and project cases.

9.3.2 Results

Figure 70 to Figure 73 present the modelled envelope of shoreline change for Option 2, Option 5, Option 6 and Option 7, respectively. In each of these figures:

- the project case and basecase shorelines are compared for the mean, 5th and 95th percentiles with the statistics derived from the modelled annual shorelines from 1988 to 2019
- the alignment of the design options is shown alongside the basecase structure alignment and material type.

Figure 69 presents a timeseries of the modelled shoreline changes for the Main Beach compartment. This plot shows the modelled updrift shoreline dynamics. Table 29 and Table 30 provide descriptive statistics of shoreline and volume change, respectively, based on the modelling results for the basecase and project cases.

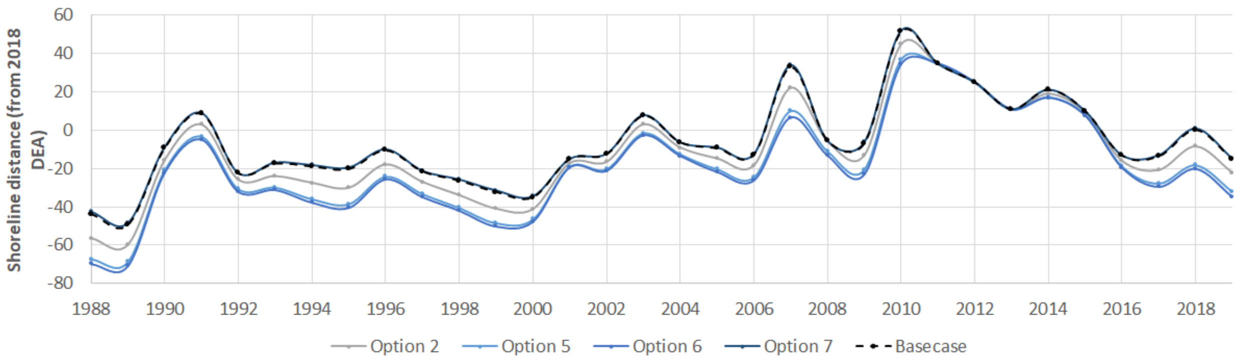

Figure 69. Timeseries of shoreline change for updrift (Main Beach) compartment.

Table 29. Shoreline change descriptive statistics from the modelling results of the 31-year simulation.

Compartment	Statistic	Option 2	Option 5	Option 6	Option 7
		Shoreline change relative to basecase (m)			
Updrift Main Beach	**Mean** of shoreline envelope	-5.4	-10.9	-12.5	0.6
	95th percentile Seaward side of shoreline envelope	-0.8	-0.8	-1.1	0.1
	5th percentile Landward side of shoreline envelope	-11.9	-22.2	-24.7	1.5
Downdrift Belongil Beach	**Mean**	4.5	9.2	10.4	-0.5
	95th percentile Seaward side of shoreline envelope	6.0	14.0	16.3	-0.5
	5th percentile Landward side of shoreline envelope	8.3	13.3	14.1	-1.1

Note: The shoreline envelope refers to the envelope containing all mean sea level shorelines over the 31-year record. The 5th percentile statistic refers to the landward side of the envelope. Annual mean shorelines would only be expected to be landward of this 5% of the time. The 95th percentile statistic refers to the seaward side of the envelope. Annual mean shorelines would only be expected to be landward of this 95% of the time.

Table 30. Beach compartment volume change descriptive statistics from the 31-year simulation results.

Compartment	Statistic	Option 2	Option 5	Option 6	Option 7
		% Change in volume			
Updrift Main Beach	**5th percentile**	-28%	-51%	-57%	3%
	95th percentile	-2%	-2%	-3%	0%
	Range (95th less 5th)	14%	28%	31%	-2%
Downdrift Belongil Beach	**5th percentile**	26%	42%	45%	-3%
	95th percentile	98%	231%	269%	-8%
	Range (95th less 5th)	-6%	2%	6%	1%

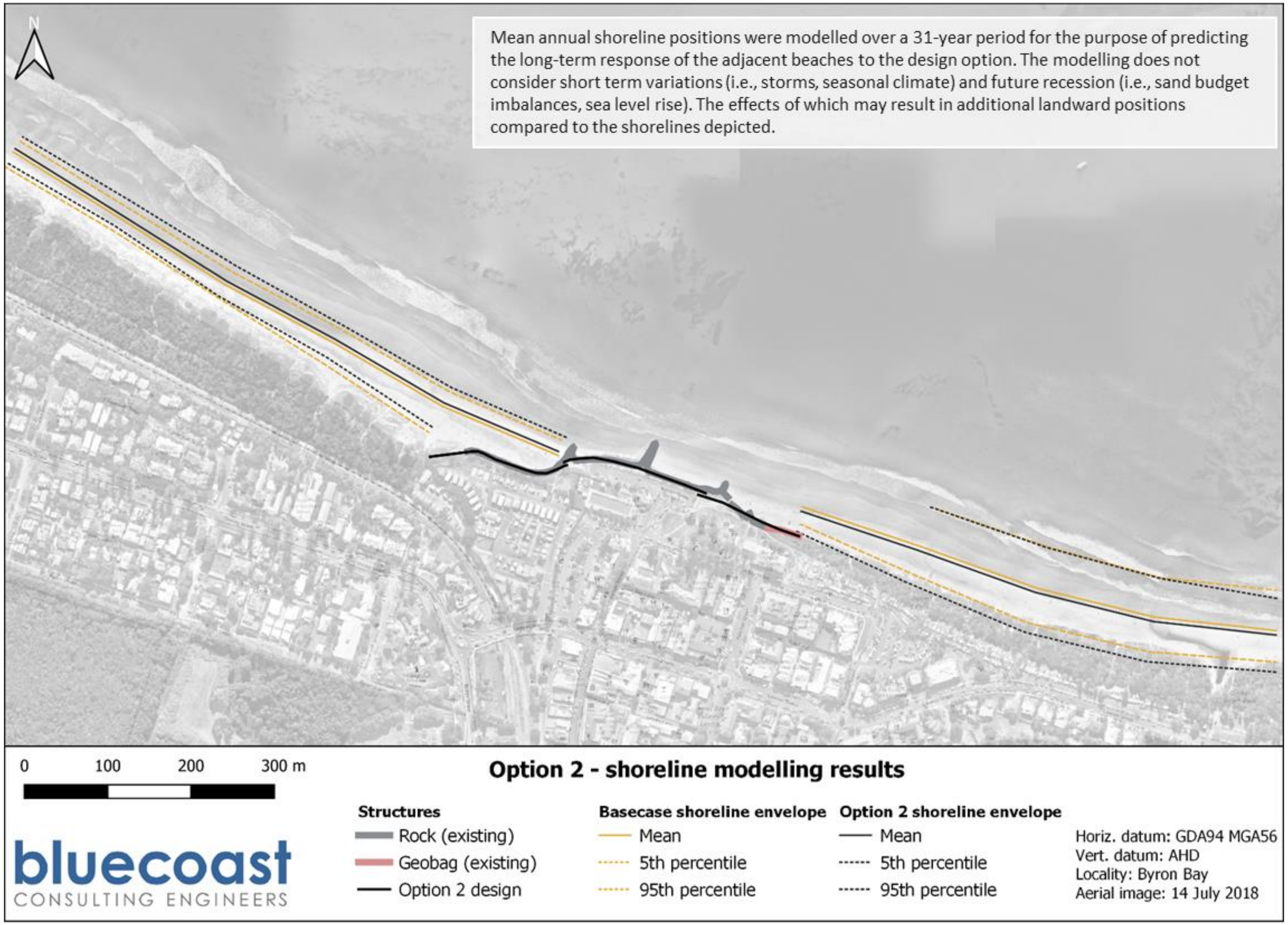

Figure 70. Shoreline modelling results showing the predicted change in shoreline position adjacent to JSPW for the Option 2 design.

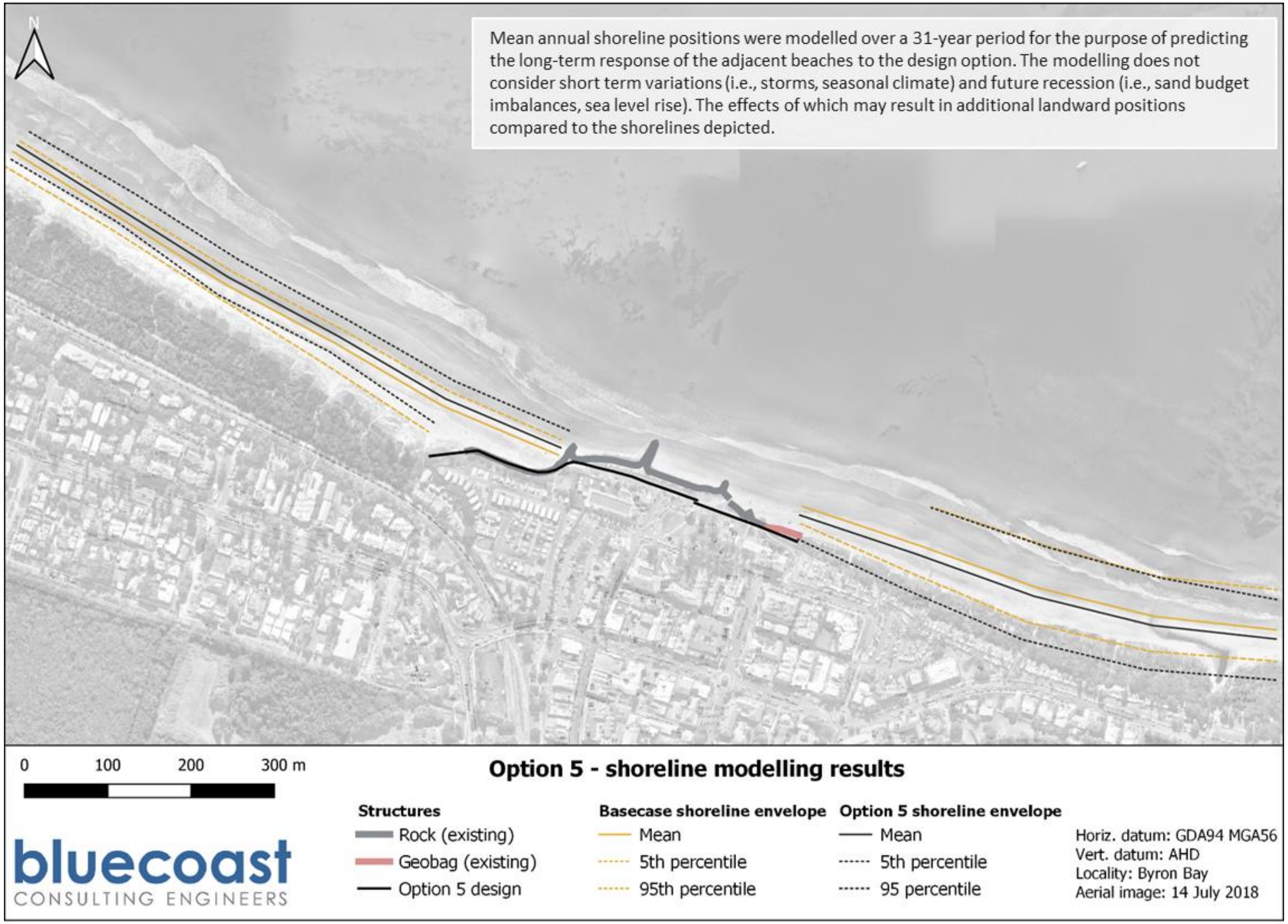

Figure 71. Shoreline modelling results showing the predicted change in shoreline position adjacent to JSPW for the Option 5 design.

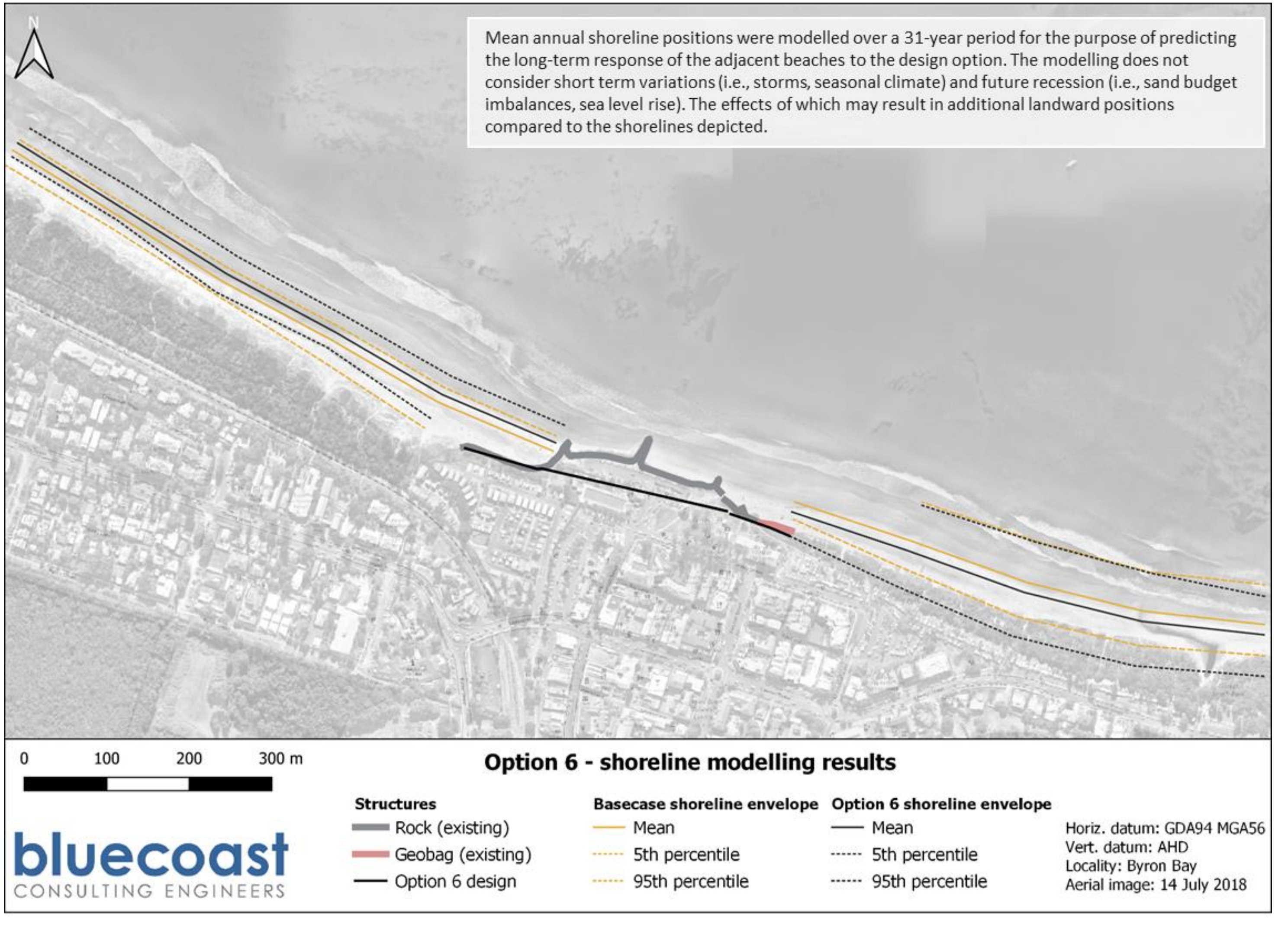

Figure 72. Shoreline modelling results showing the predicted change in shoreline position adjacent to JSPW for the Option 6 design.

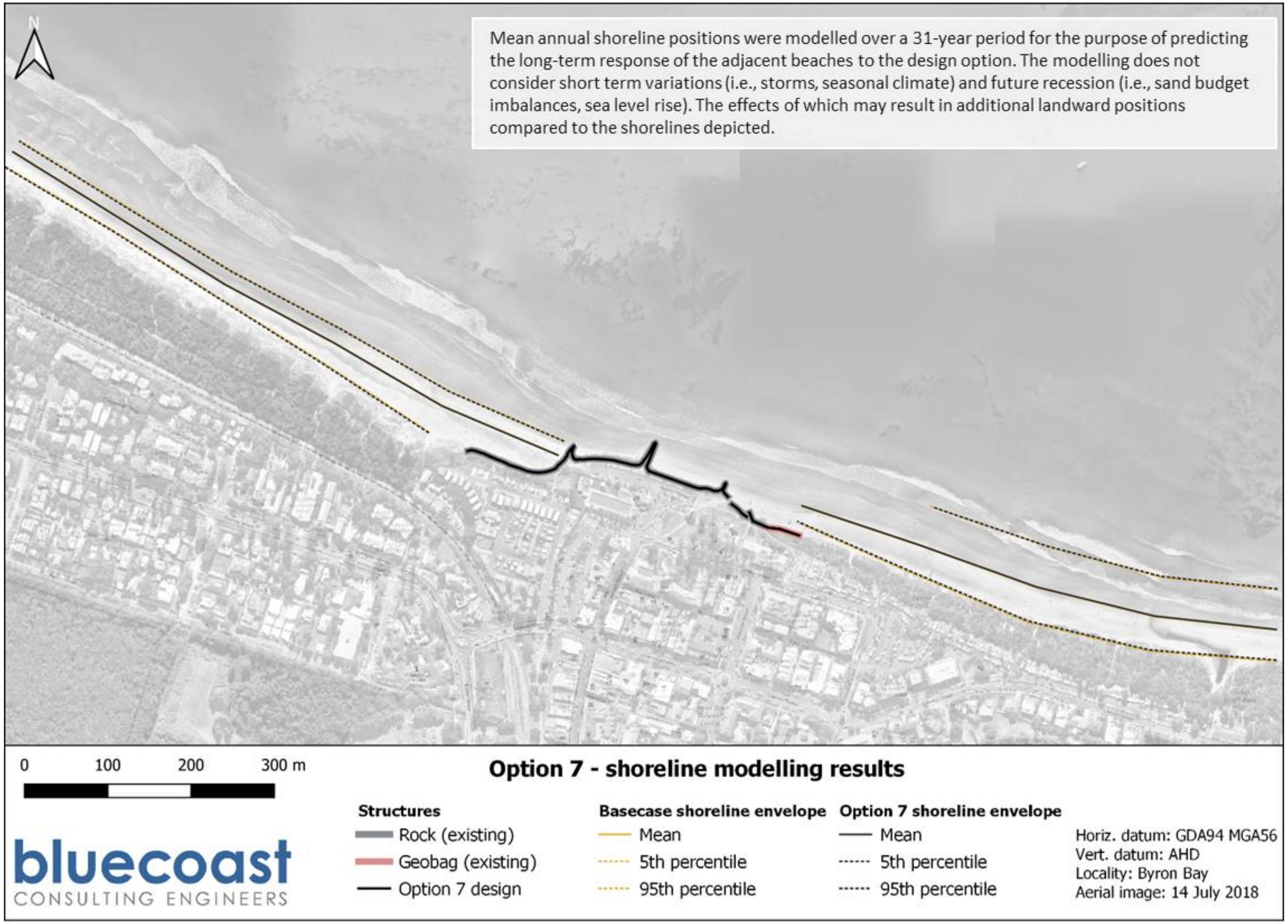

Figure 73. Shoreline modelling results showing the predicted change in shoreline position adjacent to JSPW for the Option 7 design.

9.3.3 Discussion

The bespoke shoreline modelling provides information on the expected response of the coastal environment at Main Beach (to the east) and Belongil Beach (to the west) following construction of each of the shortlisted design options. Based on the amount of sand bypassing the JSPW under basecase and project cases, the model quantifies the relative amount of beach volume and shoreline change expected at the adjacent beaches.

Key outcomes are:

- All options that substantially realign the JSPW landward (i.e., Option 2, Option 5 and Option 6) result in a net reduction in beach volume (i.e., shoreline recession) at Main Beach with a corresponding advance in the beach volume/shoreline at Belongil Beach. The realignment of the shoreline in response to these options is not consistent but rather depends on the condition of the Main Beach shoreline. The shoreline change is most prominent when the Main Beach shoreline is in a more 'eroded' (or landward) condition and negligible when Main Beach is in a naturally accreted state. On average the estimated shoreline change for these options are in the range of -5 to -12m at Main Beach and +5m to +10m at Belongil Beach. Refer to Table 29 for more detail shoreline change information for each option.
- Of the two options that realign the rock revetment landward, there is not a substantial difference in the shoreline response between Option 5 (10m realignment) and Option 6 (30m realignment). This is a result of the diminishing additional sand bypassing that would be expected for Option 6. Given this outcome it is difficult to see how the additional construction costs, loss of assets and loss of foreshore amenity associated with this option could be justified. It is recommended that Option 6 does not progress any further.
- Option 7, which upgrades the structure to contemporary standards while largely retaining the existing footprint, results in only minor shoreline changes. This is as expected.
- The model demonstrates that headland bypassing and the variability it causes to Main Beach's sand supply is the principal factor controlling shoreline dynamics along Main Beach and that is likely to remain the case irrespective of the option implemented. Along Belongil Beach the influence of headland bypassing is less but still a controlling factor.

It is important to consider the following limitations of the shoreline modelling:

- By adopting the 31-year period between 1988 to 2019 the modelled shoreline dynamics are aligned to the conditions encountered in that period. Adopting a different period may lead to different results particularly with respect to headland bypassing.
- Climate change projections, including sea level rise, may impact the relative performance of the options. The principal effects of climate change are expected to be:
  - cross profile shoreline recession response in sea level rise which would see adjacent beaches move landward (i.e., relative distance between the tip of the groyne and the shoreline would be greater) with more interaction with the JSPW
  - changes in the regional wave climate which would then affect headland bypassing and sand supply.
- Being focused on medium to long term changes (i.e., adopting annual timesteps) the shoreline model does not include the shorter-term cross profile effect of storms. Regarding cross-shore sand movements the role of storms is to erode the upper (sub-aerial) beach and deposit this material on storms bar in the surfzone. This is referred to as 'storm bite' and would effectively occur in addition to the medium- and longer-term shoreline changes predicted by the shoreline modelling.

Considering only the cross-shore component of storm derived sand movements there would be very little difference in the coastal response between options. While not on a storm event time scale, longshore transport is included in the shoreline model.

Byron Shire Council is currently undertaking Stage 2 of the open coast CMP and these issues are being considered within a regional context in the Coastal Hazard Assessment Study. As part of Stage 3 of the CMP, the information and outcomes in this report should be incorporated with the Stage 2 CMP hazard assessment work to allow the evaluation of the MBSP options to overcome these limitations.

10. Summary and next steps

10.1 Summary

The coastal protection works on Main Beach between the Byron Bay SLSC and First Sun Holiday Park are referred to as the Jonson Street Protection Works (JSPW). Their function is to protect the town centre from coastal erosion. The works are degraded and do not provide suitable public amenity, aesthetics, public safety outcomes or beach access. The Main Beach Shoreline Project (MBSP) looks at how the JSPW can be updated to improve the coastal protection of Byron Bay's town centre.

This report supports the MBSP by providing a technical assessment of shortlisted concept designs identified to modify the JSPW. The technical assessment consists of two interrelated lines of investigation:

- a geomorphic assessment which uses a largely data-driven approach to summarise relevant coastal processes and infer the relative effects of the shortlisted designs on long term coastal processes
- application of numerical modelling tools to predict the response of the coastal environment to each shortlisted design relative to the basecase (i.e., the existing situation)

A baseline geomorphological assessment was completed to explain the most relevant coastal processes occurring in the Byron embayment that influence the response to the JSPW. Adopting a data-driven approach an analysis of the study areas' sand budget was undertaken, which maps historical sand volume changes in 41 coastal sand cells. The most likely drivers for the observed coastal changes are described based on observational data, previous literature, state-of-the-art numerical modelling and/or coastal processes knowledge. Key outcomes are:

- Headland bypassing around Cape Byron results in a highly variable sand supply to the southern embayment with the annual range estimated to be from around 150,000 to over 900,000m^3/year. When coupled with the wave propagation characteristics of the embayment, the variable sand supply leads to a highly variable shoreline in the southern embayment.
- Sand movement pathways within the embayment follow two pathways: a littoral pathway (4m water depth) and a cross-embayment pathway. Based on sand volume changes determined from repeat surveys the relative split between the two pathways, when averaged across the embayment, has been calculated to be 70 : 30 (littoral : cross embayment). This is revised from previous assessments that assumed a 50 : 50 split between the pathways.
- The embayment geomorphic structure, including bedrock and coffee rock reefs and outcrops influence wave propagation, sand movements, shoreline dynamics and surfzone morphology in the embayment. The embayment's hard substrate reduces the volume of sand that can be stored in the southern embayment.

- The JSPW interacts with the embayment's natural sand movements, with the level of interaction (over the medium to long-term) controlled by the amount of sand in the Main Beach compartment, which in turn is a function of headland bypassing and wave climate.

A detailed wave and flow model capable of reproducing wave breaking and wave-generated currents along Main Beach has been developed using the SWASH model. The SWASH model results provide detailed information on the transformation of waves over the Byron embayment and its shallow reefs. The model identifies wave energy hot spots and shadows emanating from the shallow reefs and the effect this wave pre-conditioning has on nearshore hydrodynamics. Previous studies and field observations demonstrate that alongshore surfzone currents, driven by wave radiation stresses caused by wave breaking, go westward at the project site. The SWASH model confirms this clearly showing the main current flows west north-west parallel to the coast. The significant wave focusing areas over the reefs (rock outcrops) affect this alongshore current by causing alongshore accelerations/decelerations which also influence the location and behaviour of rip currents.

Comparison of the SWASH modelling results allow the effects of the shortlisted JSPW design options on the nearshore wave and hydrodynamics to be predicted, with key outcomes being:

- All options have minimal and largely localised changes to nearshore wave conditions, however, for higher tides and/or lower beach levels alongshore surfzone currents are changed from all options except Option 7.
- Outputs from the SWASH simulations at The Wreck surf spot during a range of surfing conditions shows that the project cases do not significantly affect the wave heights or currents in this high value recreational area. Similar results, with no significant change in wave heights, current or wave breaking pattern, are observed in the SWASH simulations for the area immediately seaward of the JSPW. This is an area known to provide good surf from time to time. As demonstrated in the geomorphic assessment, good surfing conditions are believed to be related to the wave pre-conditioning (owing to Middle Reef), the distribution of surfzone coffee rock and headland bypassing which results in a 'bulge' morphology when the southern embayment is full of sand.
- While the project cases lead to a localised increase in current speeds these are in line with adjacent speeds. The removal of the groyne which acts as an obstacle, as is the case for all but Option 7, would see a minor but positive improvement in swimmer and surfer safety. It is suggested that the swimmer safety implications of these results be discussed with local NSW Surf Life Saving representatives in the next evaluation stages.

XBeach modelling focused on wave overtopping. It demonstrated that overtopping of the current JSPW far exceeds the safe limits for people on the seawall crest for the present-day 100-year average recurrence interval (ARI) water level and wave conditions. Damage to assets may also occur under this condition. The design options all significantly reduce overtopping to safer levels under present-day conditions. For future sea level rise scenarios, Option 2, Option 5 and Option 6 perform well but Option 7 does not and would require adaptation to maintain safe overtopping.

The bespoke shoreline modelling provides information on the expected response of the coastal environment at Main Beach (to the east) and Belongil Beach (to the west) following construction of each of the shortlisted design options. Based on the amount of sand bypassing the JSPW under basecase and project cases, the model quantifies the relative amount of beach volume and shoreline change expected at the adjacent beaches. Key outcomes are:

- All options that substantially realign the JSPW landward (i.e., Option 2, Option 5 and Option 6) result in a net reduction in beach volume (i.e., shoreline recession) at Main Beach with a corresponding advance in the beach volume/shoreline at Belongil Beach. The realignment of the shoreline in response to these options is not consistent but rather depends on the condition of the

Main Beach shoreline. On average the estimated shoreline change for these options are in the range of -5 to -12m at Main Beach and +5m to +10m at Belongil Beach.

- Of the two options that realign the rock revetment landward, there is not a substantial difference in the shoreline response between Option 5 (10m realignment) and Option 6 (30m realignment).
- Option 7, which upgrades the structure to contemporary standards while largely retaining the existing footprint, results in only minor shoreline changes.
- The model demonstrates that headland bypassing and the variability it causes to Main Beach's sand supply is the principal factor controlling shoreline dynamics along Main Beach and that is likely to remain the case irrespective of the option implemented.

10.2 Implications for options to carry forward

The purpose of this report is not to arrive at a preferred design option for the modification of the JSPW. However, the technical assessment of the four shortlisted design options has highlighted similar performance outcomes for the two landward realignment options, Option 5 (10m realignment) and Option 6 (30m realignment).

This is principally observed in the similar predicted shoreline response of these two options. Along Main Beach, the mean shoreline change relative to the basecase has only a 1.6m difference between these options (i.e., -10.9m for Option 5 and -12.5m for Option 6, see Table 29). This outcome is due to the diminishing additional sand bypassing that would be expected for Option 6. Similarly, surf amenity and swimmer safety outcomes from the SWASH modelling and the amount of wave overtopping predicted from the XBeach modelling showed only marginally differences between these two options. Based on these results, Option 5 and Option 6 do not appear to be sufficiently different from a technical performance perspective to warrant further evaluation of both options.

The Concept Design Development report (Bluecoast, 2020b) provided a first-pass multi-criteria assessment (MCA) of seven longlist options identified for the MBSP. Based on information available at the time, the MCA considered anticipated performance across coastal protection, shoreline impacts, amenity (beach, foreshore and surfing), public safety and cost. Regarding Option 5 and Option 6, the same broad level of performance was anticipated for coastal protection, shoreline impacts, beach amenity, surfing amenity and public safety (see Table 21 in Bluecoast 2020b). The detailed quantitative performance assessment completed herein supports the anticipated performance for these criteria from Bluecoast, 2020b. Any further evaluation (i.e., economic appraisal or a second-pass MCA) would therefore be expected to yield similar results for Option 5 and Option 6 against these criteria.

Where Option 5 and Option 6 are materially different is in the level of foreshore amenity provided and in the cost of construction. The difference in foreshore amenity, was largely due to the permanent change to the size and character of the iconic Main Beach foreshore area under the more significant 30m landward alignment (Option 6). As outlined in Table 31, the car park, Fishheads café as well as a proportion of Apex Park would be lost or relocated under Option 6.

Table 31. Comparison of foreshore assets effected by the two landward realignment options

	Option 5 (10m realignment)	Option 6 (30m realignment)
Car park	• 88% of paved area retained • 55 of 95 car parks retained • Footpath relocated	• 31% of paved area retained • 10 of 95 car spaces retained • Footpath relocated

	Option 5 (10m realignment)	Option 6 (30m realignment)
Apex Park	• 82% of grassed area retained	• 80% of grassed area retained
Council building (Fishheads Café)	• Retained	• Removed or relocated
Memorial Swimming Pool	• Fully retained	• Pool footprint is retained but there would be a partial removal of the pool complex.
First Sun Holiday Park	• Retained	• Retained

Regarding anticipated costs, both Option 5 and Option 6 reconstructed landward of the existing revetment would offer some benefits in the ease of excavation to the required toe depths (i.e., protection from tides and waves). However, the excavation and removal of the existing structures, carparks, footpaths, services, and foreshore park would be costly. It would be more disruptive to traffic, beach and foreshore access and patrons of the pool and Fishheads Café when compared to other options. Given the extra 20m or realignment required for Option 6, the extra excavation, material removal and asset relocation/realignment costs and the level of disruption would be significantly greater than Option 5.

In considering which options to carry forward from this technical assessment, it is recommended Council consider the likely outcome of further economic appraisal and/or multi-criteria assessment of Option 6. Economically, the lower benefits associated with loss of public and private assets/revenue and foreshore amenity coupled with the higher construction cost would mean that Option 6 will almost certainly compare poorly against Option 5. Similarly, the permanent change to the character of the location would be unlikely to perceived positively by all sectors of the community.

10.3 Next steps

The information presented in this technical report provides the basis for further development and evaluation of the shortlisted options. This would be aimed at selecting a preferred option to carry forward for detailed design, seeking approvals and implementation. This was originally intended to be covered by the next phase of the MBSP, however, there is merit in incorporating the selection of the preferred option into Stage 3 of the LGA-wide open coast CMP. The evaluation and determination of the preferred option though CMP preparation in Stage 3 is the recommended pathway. Next steps would include:

- Any further engineering design development of the shortlisted options (i.e., Option 2, Option 5 and option 7) taken forward sufficient to inform cost estimates for each design.
- An economic appraisal (e.g., cost benefit analysis (CBA)) to examine the relative costs and economic benefits of each of the shortlisted options. In addition to the technical aspects considered in this report the economic appraisal would explore benefits/costs associated with:
    - beach and foreshore amenity
    - coastal erosion risk to assets in Byron Bay town centre, Main Beach along Apex Park, Clarkes Beach and Belongil Beach
    - public safety
    - access and pedestrian movements across the project area.

- A multi-criteria assessment (MCA) would be informed by further community consultation and explore the non-economic factors including the social and environmental aspects of the shortlisted design options.

Once the preferred design is selected by Byron Shire Council and the design further developed, it is recommended that the final alignment, footprint, levels and planform of the preferred design be subject to further detailed technical assessment to confirm the findings presented herein.

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Appendix A: Metocean monitoring

Appendix B: Drone surveys

Appendix C: Sand budget volumetric analysis results

Longshore zone	ID	Volume (m^3) change (2018 base line) 1883	2002	2011	2018
Tallows Beach	TB-1-1		-	49,119	-
	TB-1-2a		116,325	111,284	-
	TB-1-2b		- 16,981	- 355,799	-
	TB-2-1		-	125,763	-
	TB-2-2		470,973	966,121	-
	TB-3-1		-	78,987	-
	TB-3-2		347,345	691,965	-
	TB-4-1		-	29,264	-
	TB-4-2		- 432,172	- 201,141	-
	TB-5-1		-	- 39,618	-
	TB-5-2		- 355,884	- 403,998	-
	TB_LS		247,498	- 1,969,648	-
Cape Byron	CB-1-1		- 190,993	- 414,107	-
	CB-2-1		62,517	- 112,635	-
	CB-BYPASS		- 213,771	- 451,643	-
	CB-LS		- 89,667	- 113,074	-
Southern Byron embayment	WB-1-1		-	10,409	-
	WB-1-2		50,591	145,271	-
	WB-1-3	545,339	- 49,943	144,432	-
	CB-1-1	- 23,861	-	60,984	-
	CB-1-2	- 2,091	70,320	145,892	-
	CB-1-3	433,116	27,562	- 14,681	-
	MB-1-1	89,665	- 31,707	22,209	-
	MB-1-2	294,031	- 39,506	178,608	-
	MB-1-3	304,427	- 41,123	5,412	-
	BB-LS-1A (15m)	2,687,996	41,507	120,828	
	BB-LS-1B (15-20m)	4,249,167	- 546,284	272,416	-
Nouthern Byron embayment	BB-1-1	173,631	-	- 18,513	-
	BB-1-2	361,079	- 158,144	6,097	-
	BB-1-3	284,555	32,700	- 31,066	-
	BB-2-1	109,063	-	- 51,043	-
	BB-2-2	244,720	- 142,113	3,593	-
	BB-2-3	175,684	- 58,968	- 33,464	-
	BB-3-1	138,853	-	- 108,286	-
	BB-3-2	241,749	- 51,968	- 49,266	-
	BB-3-3	231,029	- 8,307	- 62,985	-
	BC-1-1	120,282	-	- 24,871	-
	BC-1-2	134,619	8,731	- 31,281	-
	BC-1-3	160,971	48,314	17,784	-
	BB-LS-2A (15m)	1,348,208	102,408	73,589	
	BB-LS-2B (15-20)	644,956	- 166,021	230,278	-

Longshore/cross-shore zone	1883	2002	2011	2018
Tallows Beach				
Beach and upper shoreface	-	129,606	1,051,947	0
Lower shoreface	-	247,498	- 1,969,648	0
Cape Byron				
Beach and upper shoreface	-	- 342,247	- 978,385	0
Lower shoreface	-	- 89,667	- 113,074	0
Southern embayment				
Beach and upper shoreface	1,640,625	- 13,805	698,536	0
Beach (1)	*4%*		*13%*	
Surf zone (2)	*18%*		*67%*	-
Lower surf zone (3)	*78%*		*19%*	-
Lower shoreface (to -15m)	2,687,996	41,507	120,828	0
Upper : lower profile	*38%*	*-50%*	*85%*	
Northern embayment				
Beach and upper shoreface	2,376,235	- 329,753	- 383,303	0
Beach (1)	*23%*		*53%*	
Surf zone (2)	*41%*		*18%*	
Lower surf zone (3)	*36%*		*29%*	
Lower shoreface (to -15m)	1,348,208	102,408	73,589	0
Upper : lower profile	*64%*	*-76%*	*-84%*	
Sub-total (northern and southern embayment to -15m)	**8,053,064**	**- 199,643**	**509,650**	
Total (northern and southern embayment to -20m)	**12,947,187**	**- 911,949**	**1,012,345**	**-**

Appendix D: 40-year hindcast complete results

Joint Frequency Table (%) Showing Hs Against Direction for the Period 02-Jan-1979 08:00:00 to 01-Mar-2020 10:00:00

N=360819	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	Cumul.
0.0-0.5	-	-	14.67	1.68	-	-	-	-	-	-	-	-	-	-	-	-	16.35	16.35
0.5-1	-	0.53	60.65	0.02	-	-	-	-	-	-	-	-	-	-	-	-	61.21	77.56
1.0-1.5	-	0.94	17.65	-	-	-	-	-	-	-	-	-	-	-	-	-	18.59	96.15
1.5-2	-	0.29	2.62	-	-	-	-	-	-	-	-	-	-	-	-	-	2.91	99.06
2.0-2.5	-	0.07	0.61	-	-	-	-	-	-	-	-	-	-	-	-	-	0.67	99.74
2.5-3	-	0.03	0.21	-	-	-	-	-	-	-	-	-	-	-	-	-	0.24	99.98
3.0-3.5	-	*	0.02	-	-	-	-	-	-	-	-	-	-	-	-	-	0.02	100.00
Total	-	1.86	96.43	1.70	-	-	-	-	-	-	-	-	-	-	-	-		
Cumul.	-	1.86	98.30	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00		

* denotes values less than 0.01%   - denotes no records in bin

Figure 74: Joint frequency table for wave height versus wave direction at location MB01.

Joint Frequency Table (%) Showing Hs Against Tp for the Period 02-Jan-1979 08:00:00 to 01-Mar-2020 10:00:00

N=360819	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	Total	Cumul.
0.0-0.5	1.93	6.37	6.65	1.05	0.24	0.04	0.06	-	16.35	16.35
0.5-1	0.44	12.65	32.42	12.38	2.06	0.55	0.71	-	61.21	77.56
1.0-1.5	-	0.59	7.99	6.79	2.12	0.60	0.50	-	18.59	96.15
1.5-2	-	*	0.61	1.33	0.58	0.26	0.12	-	2.91	99.06
2.0-2.5	-	*	0.04	0.36	0.16	0.06	0.06	-	0.67	99.74
2.5-3	-	-	*	0.12	0.08	0.02	*	-	0.24	99.98
3.0-3.5	-	-	-	*	0.01	*	*	-	0.02	100.00
Total	2.37	19.61	47.72	22.05	5.25	1.54	1.45	-		
Cumul.	2.37	21.99	69.70	91.75	97.01	98.55	100.00	100.00		

* denotes values less than 0.01%   - denotes no records in bin

Figure 75: Joint frequency table for wave height versus wave period at location MB01.

Joint Frequency Table (%) Showing Tp Against Direction for the Period 02-Jan-1979 08:00:00 to 01-Mar-2020 10:00:00

N=360819	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	Cumul.
4-6	-	0.02	1.32	1.04	-	-	-	-	-	-	-	-	-	-	-	-	2.37	2.37
6-8	-	0.26	18.68	0.67	-	-	-	-	-	-	-	-	-	-	-	-	19.61	21.99
8-10	-	1.02	46.70	-	-	-	-	-	-	-	-	-	-	-	-	-	47.72	69.70
10-12	-	0.36	21.69	-	-	-	-	-	-	-	-	-	-	-	-	-	22.05	91.75
12-14	-	0.10	5.15	-	-	-	-	-	-	-	-	-	-	-	-	-	5.25	97.01
14-16	-	0.04	1.50	-	-	-	-	-	-	-	-	-	-	-	-	-	1.54	98.55
16-18	-	0.06	1.40	-	-	-	-	-	-	-	-	-	-	-	-	-	1.45	100.00
18-20	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	100.00
Total	-	1.86	96.43	1.70	-	-	-	-	-	-	-	-	-	-	-	-		
Cumul.	-	1.86	98.30	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00		

* denotes values less than 0.01%   - denotes no records in bin
Metadata:
Project: Jonson Street Protection Works
Location: MB01 [560129.27000 , 6832086.95000]
Data period: 02-Jan-1979 08:00:00 to 01-Mar-2020 10:00:00
Data source: reconstruction_MB1.mat
Data summary: All Records
Number of Records: 360819
% Calm:

Figure 76: Joint frequency table for wave period versus wave direction at location MB01.