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Eurobodalla Coastal Hazard Assessment

WRL Technical Report 2017/09
October 2017

By I R Coghlan, J T Carley, A J Harrison, D Howe, A D Short,
J E Ruprecht, F Flocard and P F Rahman

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1. Introduction

1.1 Background

The Eurobodalla Shire Council (ESC) coastline is approximately 110 km long extending from South Durras to Mystery Bay and includes Batemans Bay east of the Tollgate Bridge. ESC is preparing a Coastal Management Program (CMP) which will apply to its open coast areas, including 83 beaches and adjoining headlands. Stage 1 of the CMP comprised a scoping study for the entire Eurobodalla coastline prepared for ESC by Umwelt Australia (Umwelt, 2017). The scoping study discussed the primary and secondary sediment compartments within the whole local government area and, building on earlier work by SMEC (2010), recommended targeted, detailed coastal hazard assessments be undertaken only at those beaches with public and private assets potentially at high risk from coastal hazards.

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Umwelt, to prepare a Coastal Hazard Assessment for the highest priority beaches identified in the Stage 1 scoping study. This report forms Stage 2 of the CMP for ESC.

The Stage 2 study area extends southward from Durras Beach (south) to Broulee Beach as shown in Figure 1-1 and includes a selection of only 17 beaches. WRL examined **sandy beaches and seawalls** for which ESC has at least some management responsibility within the study area for extreme events between 2017 and 2100. That is, the examination of beaches managed by ESC which are fronted by rock platforms/reefs and backed by cliffed regions was outside of the scope of this study. The examination of beaches managed by other authorities such as the NSW National Parks and Wildlife Service (NPWS) and seawalls managed by NSW Crown Lands were also outside of the scope of this study.

The study was originally commissioned in 2011 to examine beaches within Batemans Bay only. In 2012, the scope of the study was extended to the wider local government area. In 2013, the study was put on hold while a sea level rise policy and planning framework were prepared, additional photogrammetric, topographic and bathymetric datasets were collected and the NSW Government undertook coastal reforms. The study was re-commissioned with a modified scope and alternative methodologies in December 2016.

The methodology applied in this report for the Eurobodalla Coastal Hazard Assessment was developed in consultation with ESC and the NSW Office of Environment and Heritage (OEH) and considers the following documents:

- NSW Coastal Management Act (2016);
- *Draft* NSW Coastal Management Manual (OEH, 2016);
- Coastal Risk Management Guide (DECCW, 2010);
- ESC sea level rise policy and planning framework (ESC, 2014;Whitehead & Associates, 2014);
- NSW Coastline Management Manual (NSW PWD, 1990).

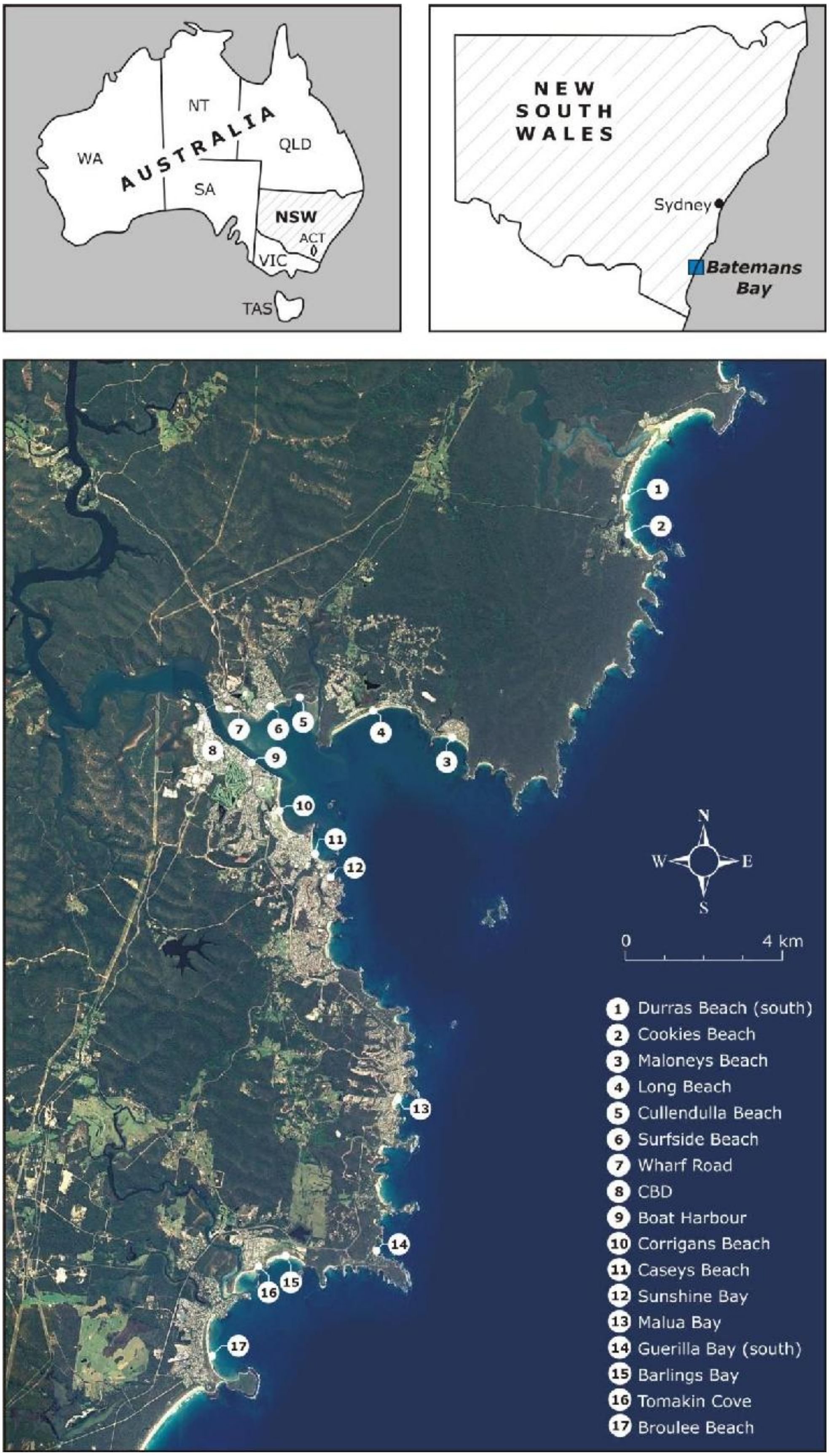


Figure 1-1: Location and Study Area

1.2 Principal Tasks

While the study has many components, the principal deliverables are:

- Conceptual sediment transport models and erosion/recession hazard maps (10 beaches);
- Tidal (excluding wave effects) and coastal inundation hazard maps (17 beaches).

The key deliverables for each beach are summarised in Table 1-1

Table 1-1: Breakdown of Principal Coastal Hazard Assessment Tasks

Beach	Conceptual Sediment Transport Models	Erosion Mapping		Inundation Mapping	
		Deterministic Method	**Probabilistic Method** (5% and 1% Encounter Probability)	**Tidal** (HHWSS and 63% AEP)	**Coastal** (63%, 5% and 1% AEP)
Durras Beach (South)				✔	✔
Cookies Beach				✔	✔
Maloneys Beach	✔	✔		✔	✔
Long Beach	✔		✔	✔	✔
Cullendulla Beach				✔	✔
Surfside Beach (East)	✔		✔	✔	✔
Surfside Beach (West)	✔		✔	✔	✔
Wharf Road				✔	✔
Central Business District				✔	✔
Boat Harbour				✔	✔
Corrigans Beach				✔	✔
Caseys Beach				✔	✔
Sunshine Bay	✔	✔		✔	✔
Malua Bay	✔		✔	✔	✔
Guerrilla Bay (South)	✔	✔		✔	✔
Barlings Beach	✔	✔		✔	✔
Tomakin Cove	✔		✔	✔	✔
Broulee Beach	✔		✔	✔	✔

Note: AEP - annual exceedance probability
HHWSS – High High Water Solstices Springs tidal level

Assessment of the coastal cliff instability hazard was outside the scope of works of this WRL study. Targeted, detailed geotechnical slope instability risk assessments for three (3) priority headlands within Batemans Bay (between Maloneys Beach and Long Beach, between Corrigans Beach and Caseys Beach and between Caseys Beach and Sunshine Bay) were previously prepared by ACT Geotechnical Engineers (2012).

1.3 Coastal Hazard Assessment Workflow

While some iterations occurred, in broad terms, the following sequence was followed in the preparation of the principal tasks for the coastal hazard assessment:

1. Site inspections at 17 beaches were undertaken and available literature collated and reviewed;
2. The governing physical processes were assessed including assessment of photogrammetry, numerical modelling of waves and erosion, and estimation of closure depth;
3. Consensus input values for erosion/recession modelling at 10 beaches were established with an expert panel;
4. Conceptual sediment compartment models were prepared for 10 beaches;
5. Erosion/recession modelling was undertaken and associated maps prepared for 10 beaches;
6. Tidal inundation maps were prepared for 17 beaches; and
7. Coastal inundation modelling was undertaken and associated maps prepared for 17 beaches.

1.4 Overview of the Report

- **Section 2** summarises coastal site inspections and sand sample analysis completed along the Eurobodalla study area;
- **Section 3** describes how conceptual sediment compartment models were developed for each beach focusing on its sediments, their sources and sinks, and linkages, if any, to adjoining beach compartments;
- **Section 4** describes and assesses the influence of relevant coastal processes with respect to coastal hazards;
- **Section 5** presents the processes by which consensus values for storm demand, Bruun factor and underlying shoreline movement rate were established at each beach where erosion/recession maps were prepared;
- **Section 6** outlines the probabilistic and deterministic erosion/recession hazard methodology;
- **Section 7** describes the tidal inundation (excludes wave effects) hazard methodology;
- **Section 8** describes the coastal inundation (includes wave effects) hazard methodology;
- **Section 9** provides a qualitative review of secondary coastal hazards within the Eurobodalla study area;
- **Section 10** describes the assumptions and limitations of the study; and
- **Section 11** summarises a number of further investigations recommended to be undertaken.

This report has been structured to highlight and summarise the key findings of the study. A significant amount of more detailed background information, reporting and mapping has been documented in appendices, rather than in the main body of the report. Appendices to this report include:

- **Appendix A** reviews all available literature relevant to coastal hazards in the area;
- **Appendix B** describes site inspections undertaken and analysis of collected sand samples;
- **Appendix C** documents the analysis of photogrammetric data for erosion and recession;
- **Appendix D** provides background information for the SWAN numerical wave modelling;
- **Appendix E** discusses the methodology and results of SBEACH numerical erosion modelling;
- **Appendix F** outlines the range of methods used to estimate closure depth;
- **Appendix G** discusses the dune stability schema used for erosion/recession mapping;
- **Appendix H** reviews the connectivity of the salient/tombolo at Broulee Island;
- **Appendix I** comprises the deterministic and probabilistic erosion/recession maps;
- **Appendix J** tabulates the width of the zone of reduced foundation capacity at each profile;
- **Appendix K** comprises the HHWSS and 1 year ARI tidal inundation maps;
- **Appendix L** comprises the coastal inundation maps (including wave effects); and
- **Appendix M** provides boundary (tailwater) conditions for a future Durras Lake flood study.

2. Site Inspections

2.1 Overview

WRL formally inspected 20 sections of the Eurobodalla coastline at the following times (Table 2-1 - with the NSW sub-section class, coastline type, length and the general direction of orientation as per Short, 2007):

- Campaign 1: 31 October – 1 November 2011 (Batemans Bay beaches);
- Campaign 2: 4-8 December 2012 (beaches outside Batemans Bay); and
- Campaign 3: 22-23 February 2017 (ten beaches requiring erosion/recession maps).

WRL's coastal engineers have also conducted informal inspections dating back to 1979 of many of the beaches in the study area outside of the formal inspection times. For Campaign 1, site inspections were performed by Mr Ian Coghlan and Mr James Carley of WRL in the company of Mr Norman Lenehan (ESC) and Mr Daniel Wiecek (OEH). Campaign 2 was undertaken by Mr Ian Coghlan, Mr James Carley and Jamie Ruprecht of WRL in the company of Mr Norman Lenehan (ESC) and Mr Mohammed Ullah (OEH). Campaign 3 was undertaken by Mr Ian Coghlan in the company of Prof. Andrew Short (University of Sydney). Note that Cullendulla Beach, Tomakin Beach and Bengello Beach have been included in this section because they are adjacent to, but not included in, the erosion/recession hazard assessment.

Table 2-1: Coastline Sub-Sections Considered for the Study (Short, 2007)

Name	Reference ID	Type	Length (m)	Drn *
Durras Beach	NSW 512	transverse bar and rip / rhythmic bar and beach	2,300	ESE
Cookies Beach	NSW 513	low tide terrace	800	ENE
Maloneys Beach	NSW 526	reflective / low tide terrace	810	S
Long Beach	NSW 529	low tide terrace / transverse bar and rip + seawall	2,150	SE
Cullendulla Beach	NSW 530	beach + sand flats	660	S
Surfside Beach (East)	NSW 531	low tide terrace	850	SE
Surfside Beach (West)	NSW 532	beach + sand flats	270	SW
Wharf Road	N/A	reflective + tidal sand flats + seawall	900	SW
Central Business District	N/A	seawall	680	NE
Boat Harbour	N/A	seawall	2,070	NE
Corrigans Beach	NSW 533	low tide terrace + seawall	1,800	NE
Caseys Beach	NSW 534	low tide terrace + seawall	850	E
Sunshine Bay	NSW 535	reflective	520	ENE
Malua Bay	NSW 543	transverse bar and rip +seawall	510	E
Guerrilla Bay (south)	NSW 552	low tide terrace	290	E
Barlings Beach	NSW 557	low tide terrace / transverse bar and rip	1,110	S
Tomakin Cove	NSW 558	low tide terrace	270	SE
Tomakin Beach	NSW 559	low tide terrace	900	SE
Broulee Beach	NSW 560	transverse bar and rip / low tide terrace /reflective	1,740	ENE
Bengello Beach	NSW 562	transverse bar and rip / rhythmic bar and beach	6,000	SE

Note: Drn: approximate direction that the beach faces

Comprehensive field notes and photographs are documented for the 20 coastline sub-sections in Appendix B. These notes consider the beaches and coastal infrastructure within each coastline sub-section. The site inspections focused on the present condition of coastal protection works maintained by ESC (where present) and on the inter-relation of such protection works, other infrastructure (amenities blocks, roads, cycle paths, car parks, stormwater outlets, utilities) and public and private property with the coastal processes acting on each beach. WRL was advised that ESC is responsible for maintenance of the seawalls at the CBD/Boat Harbour (western half) and Caseys Beach. The condition of coastal protection works not maintained by ESC was assessed at a cursory level.

In Section 3 of this report, the geomorphology and sediment transport of the 10 beaches requiring erosion/recession hazard mapping is discussed in greater detail.

2.2 Sand Samples for Particle Size Analysis

Sediment samples were collected from each of the ten beaches requiring erosion/recession maps. Additional samples were also collected at Durras Beach, Cookies Beach, Cullendulla Beach, Tomakin Beach and Bengello Beach. For beaches outside of Batemans Bay, the location of each sediment sample (collected in 2012) is illustrated on the site details figure for each coastline sub-section referred to in Appendix B (exact sand sample locations were not recorded for the Batemans Bay beaches in 2011). The dried sediment samples were treated according to Australian Standard 1289.3.6.1 (2009) to determine the particle size distributions by mechanical sieving. A photograph of each dried sample and its associated particle size distribution is also shown in Appendix B. The median particle size (d_{50}) for the sand fraction of sediment (60 µm to 2 mm) varies between 180 and 1,240 µm as shown in Table 2-2. Particle size standard deviations (i.e. "sorting") of these samples are shown in Table 3-3.

Table 2-2: Median Sand Fraction Particle Sizes (60 µm to 2 mm)

Name	Sample	d_{50} (µm)	d_{50} (mm)
Durras & Cookies Beaches	1	430	0.43
	2	320	0.32
	3	350	0.35
Maloneys Beach	1	210	0.21
Long Beach	1	240	0.24
Cullendulla Beach	1	180	0.18
Surfside Beach (east)	1	250	0.25
Surfside Beach (west)	1	210	0.21
Sunshine Bay	1	1,010	1.01
	2	210	0.21
Malua Bay	1	400	0.40
	2	290	0.29
Guerrilla Bay	1	280	0.28
	2	300	0.30
Barlings Beach	1	320	0.32
	2	280	0.28
Tomakin Cove & Beach	1	350	0.35
	2	190	0.19
Broulee Beach	1	210	0.21
	2	220	0.22
Bengello Beach	1	220	0.22
	2	320	0.32
	3	340	0.34
	4	330	0.33
	5	350	0.35
	6	1,240	1.24

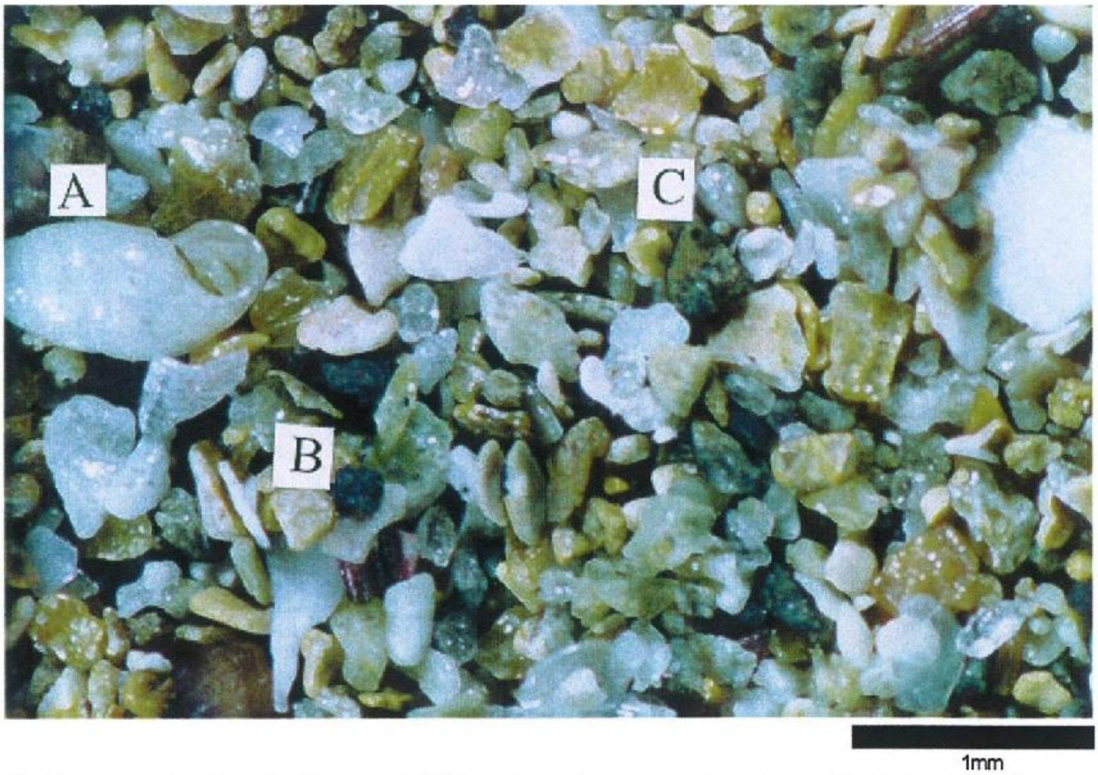
Generally, the sediment from each of the beaches is characterised as medium grained sand. However, it also important to note exceptions to this within the coastline sub-sections. The sediment from Cullendulla Beach, Sunshine Bay, Tomakin Cove & Beach and Broulee Beach has a relatively high fraction of fine sand (60 µm to 200 µm). Small amounts of silt were also visible in the samples from Cullendulla Beach, Surfside Beach (east) and Surfside Beach (west). Sediment Sample 6 from Bengello Beach (taken immediately north of the northern Moruya River training wall) has a relatively high fraction of coarse sand (600 µm to 2 mm), although this is not considered representative of the full length of the beach. Sediment from Sunshine Bay has a relatively high fraction of fine gravel (2 to 6 mm) within the sample. Moderate shell content amounts were also visible in the samples from Durras and Cookies Beaches, Barlings Beach and Broulee Beach.

2.3 Sand Samples for Carbonate Content Analysis

During field inspection Campaign 3, WRL collected additional sand samples to test for carbonate content. The dried sediment samples were treated with hydrochloric acid to determine the percentage carbonate content (Table 2-3). These values generally compared well with previous analysis from the Australian Beach Safety And Management Program database (ABSAMP, 2009). This work was undertaken to inform the development of the conceptual sediment transport models (Section) and particularly to identify the exact location of the significant sediment change between Bengello Beach (marine quartz) and Broulee Beach (carbonate sand), and beaches to the north. Carbonate sand, which is generally fragments of shell material, is derived from the rocks and sea floor immediately adjacent to a beach and supplied onto it in an ongoing fashion. The lithic-quartz sand is derived from both the Clyde and Moruya River fluvial sands, as well as inner shelf sands transported landwards during the sea level transgression. An example photograph, taken using the aid of a microscope, of a sand sample from the western end of Long Beach by WBM Oceanics (2000) clearly shows a mix of carbonate sand and marine quartz (Figure 2-1).

Table 2-3: Carbonate Content of Sand Samples

Beach	Section/Comment	Carbonate Content (%)	
		WRL Analysis	ABSAMP (2009)
Maloneys Beach	Eastern end	76.0	78.2
	Western end	69.2	
Long Beach	Eastern end	78.3	41.7
	Western end	63.8	
Cullendulla Beach	Western end	62.0	
Surfside Beach (East)	Central	19.5	
Surfside Beach (West)	Central	20.1	
Sunshine Bay	Central (Sand Fraction)	62.3	9.6
	Central (Gravel Fraction)	0.9	
Malua Bay	Central	78.4	77.2
Guerilla Bay	Central	44.8	45.4
Barlings Beach	Western end	74.0	60.4
Tomakin Cove	Central	71.4	
Broulee Beach	Northern end	84.0	
Broulee Island Tombolo	Southern side	47.9	
Bengello Beach	Northern end	5.4	
	Central (windsock)	4.6	
	Southern end (north of training wall)	4.3	



Shelly quartzose beach sand with coarse dark lithic grains and predominantly marine quartz. Carbonate fraction consists of abraded mollusc fragments, whole gastropods (A), sponge spicules, echinoderms and foraminifera. Photogmicrograph shows (B) lithic fragment and (C) marine quartz.

Figure 2-1: Photomicrograph of a sand sample from the western end of Long Beach identifying carbonate sand (A) and marine quartz (C) (Source: WBM Oceanics, 2000)

3. Characteristic Geomorphology and Conceptual Sediment Transport Models

3.1 Preamble

This section investigates the morphodynamic characteristics and sediment mobility of the ten (10) beaches for which erosion/recession hazard modelling and mapping was undertaken. This includes their beach-barrier geomorphology, including their barrier type and volume, beach type-state, beach sediments, and degree of exposure to wave and tidal action. Following the site inspections (Section 2 and Appendix B), conceptual sediment compartment models were developed for each beach focusing on its sediments, their sources and sinks, and linkages, if any, to adjoining beach compartments.

This section is predominantly based on a review of existing literature. However, the following values were determined as part of this study and have been quoted throughout this section:

- Sediment characteristics (sand samples in Section 2 and Appendix B);
- Storm demand and beach demand (consensus values from expert panel in Section 5);
- Underlying shoreline movement and beach slope (photogrammetry analysis in Appendix C); and
- Nearshore wave climate (SWAN wave modelling in Appendix D).

3.2 Coastal Geomorphology

3.2.1 Background

The Eurobodalla coast occupies 110 km of the southern NSW coast, all located geologically in the rugged Lachlan Fold Belt that commences at the shire boundary at Durras and extends south to Tasmania. Along the Eurobodalla coast, the geology is predominately steeply dipping metasedimentary rocks, together with some local occurrences of basalt and granite. The rocks have been deeply weathered and eroded leading to the formation of numerous coastal valleys containing streams and a few moderate sized rivers. The Holocene sea level rise flooded the lower reaches of these valleys leading to the development of the present coast with its many small embayed estuaries and beaches located at the mouth of the valleys.

The coast is exposed to deepwater waves with a median H_s of 1.30 m, T_p = 9.5 s (Shand et al., 2010) and direction 130°TN (approximately south-east) (Coghlan, 2010). At the shore, however, the median significant wave height at the outer edge of the surf zone ranges from approximately zero well inside Batemans Bay shoaling up to 1.4 m on the more exposed open coast beaches. The spring tidal range (HHWSS-ISLW) is 1.655 m (MHL, 2012).

There are 128 beaches along the Eurobodalla coast, which average 0.65 km in length and occupy 70.6 km (55%) of the coast, the remainder being mainly bedrock and river or inlet mouths. There are at least 28 drainage systems reaching the coast, mostly associated with small streams and their estuaries and ICOLLs. The only rivers are:

- Clyde River (1,837 km^2 catchment),
- Moruya River (1,500 km^2);
- Tuross River (1,811 km^2); and
- Wagonga Inlet (144 km^2).

Each of these rivers has a relatively small catchment. However, given their steep catchments close to the coast, they all experience periodic flooding.

3.2.2 Sediment Compartments

The NSW Coastal Management Act (2016) identified 47 secondary coastal sediment compartments along the NSW coast as developed by the National Climate Change Adaption Research Facility (NCCARF, McPherson et al., 2015), including five (5) along the Eurobodalla coast which are all located in the south coast region (NSW02), in the Durras-Cape Howe primary compartment (PC 02). Two (2) of these cover the study area - the Batemans Bay secondary compartment (NSW02.06.02) extends from Three Islet Point to Mosquito Bay head, and the Broulee secondary compartment (NSW02.06.03) extends from Mosquito Bay head to Bingie Bingie Point (Figure 3-1).

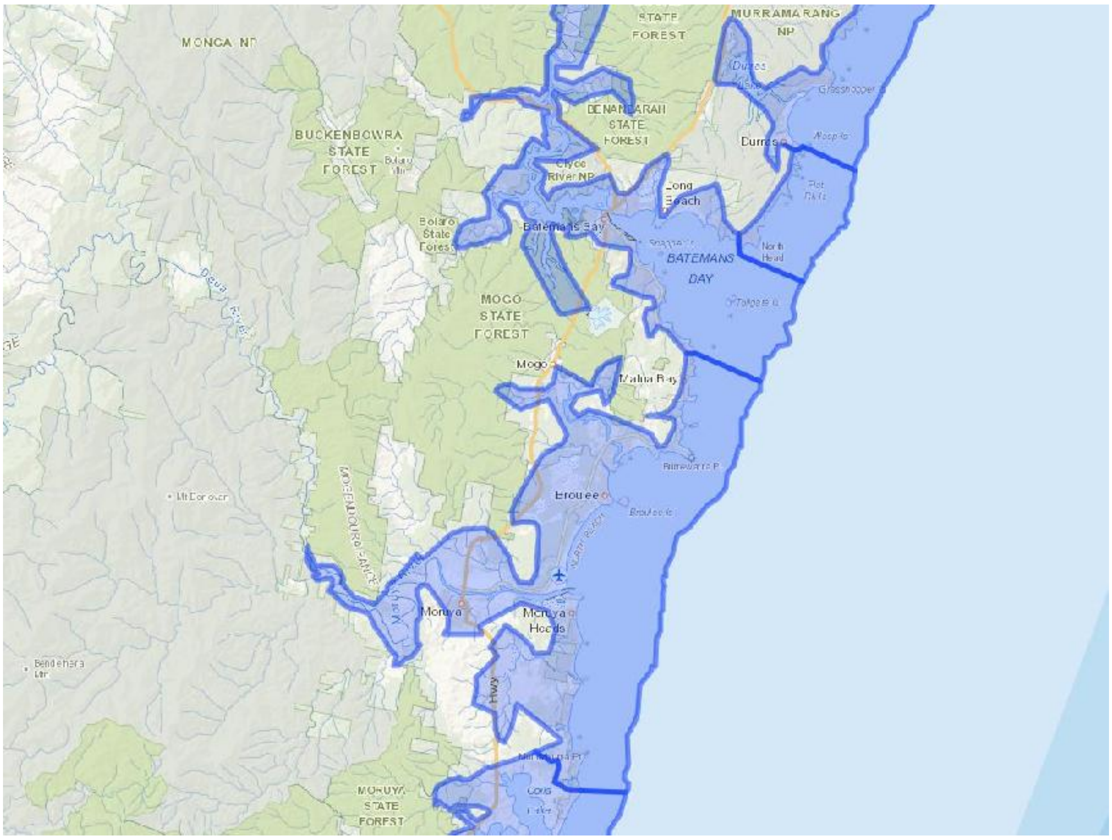


Figure 3-1: Secondary Sediment Compartments in the Study Area (CoastAdapt, 2017)

The purpose of the NCCARF compartment program is to encourage a sediment compartment approach to understanding the coast, its behaviour and management, as followed in this report. Shoreline behaviour (accretion, stability or recession) ultimately depends on the availability of sediment within a compartment. Subject to sea level change, if the sediment has a positive budget, the system can accrete and build seaward, as many beaches did in the mid-Holocene. If balanced, the shoreline remains stable; while if it is negative and sand is being lost from the system, the shoreline and beaches will recede. By understanding how sediment is operating within each compartment and linkages, if any, between adjacent compartments enables coastal managers to better understand the underlying causes of the shoreline behaviour and plan accordingly.

NCCARF assigned each secondary compartment with a sensitivity rating of 1 to 5 (where 1 = presently accreting and 5 = presently receding). The Batemans Bay secondary compartment is rated 3 (erosion and inundation issues) and the Broulee secondary compartment is rated 4 (erosion issues).

Five (5) of the beaches being assessed for erosion/recession are located in the Batemans Bay secondary compartment (SC 02) and five (5) on the open coast in the Broulee secondary compartment (SC 03) (Table 3-1). All of the beaches are also located within tertiary sediment compartments, where some are individual compartments while some are linked, such as Barlings Beach-Tomakin Cove and Beach.

Table 3-1: NCCARF classification of the Batemans Bay and Broulee primary sediment compartments and the tertiary sediment compartments containing the ten beaches

Province	Region	Primary Compartment	Secondary Compartment	Tertiary Compartments
Temperate South/ Southeast	NSW02 South Coast	06 Durras-Cape Howe	02 Batemans Bay	Maloneys Beach
				Long Beach
				Surfside Beach (east)
				Surfside Beach (west)
				Sunshine Bay
			03 Broulee	Malua Bay
				Guerilla Bay
				Barlings Beach-Tomakin Cove & Beach
				Broulee Beach-Bengello Beach

3.2.3 Holocene Evolution

The Eurobodalla coast was drowned by the Holocene sea level transgression, reaching its present level about 6,500 years ago and forming the present coast of rocky headlands, embayed beaches and estuaries. Both the Batemans Bay and Broulee compartments had a positive sediment supply in the mid-Holocene leading to the deposition of the beach systems and in some cases their accretion up to 2 km seaward, as occurred at Bengello (Thom, et al., 1978, 1981; Oliver, et al., 2015) and Moruya-Pedro (Oliver, et al., 2017). Most other Eurobodalla beach systems also underwent some degree of barrier accretion and sediment accumulation with sediment largely derived from the inner shelf, while the estuaries have been infilling with both fluvial, marine and in situ carbonate sediments.

Table 3-2 indicates the volume of marine sand transported into each of the nine beach-barrier systems (Surfside Beach (east) and Surfside Beach (west) are considered as one barrier) since the sea level stillstand. The greater volumes tended to occur where there was available accommodation space within the valleys combined with a suitable supply of sand. Four (4) of the beaches within the Batemans Bay secondary compartment accumulated substantial volumes of sand, which built the beaches 200-460 m into the bay and partially (Maloneys and Long) or completely (Surfside east and west) filled their embayments. The open coast beaches are bordered by prominent headlands, which break the Broulee compartment into a series of smaller tertiary sediment compartments, with no linkages between most of the compartments. Some of the compartments received abundant sand and/or have large accommodation space while some received very little and/or had little accommodation space, which explains the variations in volume shown in Table 3-2. Sunshine Bay and Guerilla Bay (south) have a beach and single foredune, with parts of each beach backed by cliffs and a very small stationary barrier. Malua Bay experienced minor accretion, while Barlings Beach and Broulee Beach underwent substantial accretion of several hundred metres, with some of the Broulee sand very likely to be Moruya River sand deposited in the inner shelf during the sea level lowstand. All ten (10) beaches received significant supply of carbonate sand derived from the adjacent rocks and sea floor. The considerable variation in tertiary sediment compartment behaviour is typical of the Eurobodalla and southern NSW coast with the coastal geology (headland, rocks and reefs) influencing the transport of sediment into each compartment. The sources of sand for the beach can also be gauged from the texture, that is, their size, sorting and composition. The sand sources for the ten beaches are a combination of fluvially derived quartz (lithic) sand deposited on the shelf at lower sea levels and reworked onshore during the sea level transgression and locally produced carbonate material (generally shell fragments derived from the rocks and sea floor immediately adjacent to each beach).

Table 3-2: Width and volume of the barrier systems supplied over approximately 6,000 years (Source: ABSAMP, 2009)

Beach/barrier	Maximum barrier width (m)	Volume		Comment
		Total (m^3 above 0 m AHD)	Lineal (m^3/m above 0 m AHD)	
Maloneys Beach	460	1,978,000	2,300	regressive beach-foredune ridges
Long Beach	200	2,300,000	1,000	regressive beach-foredune ridges
Surfside Beach (E & W)	440	780,000	867	regressive beach ridges
Sunshine Bay	~30	60,000	250	backed by cliffs, single low foredune
Malua Bay	440	275,000	550	single low foredune
Guerilla Bay (south)	~50	100,000	300	cliffs in north, single low foredune in centre
Barlings Beach	500	2,925,000	2,500	regressive foredune ridges
Tomakin Cove	650	1,950,000	3,250	regressive foredune ridges
Broulee Beach	500	5,635,000	2,500	regressive beach-foredune ridges

3.3 Beach-Barrier Sediment Compartments

3.3.1 Introduction

This section discusses each of the ten beaches and their barriers within the context of the their secondary and tertiary sediment compartments and develops conceptual models of beach behaviour. Table 3-3 summarizes the characteristics of the beaches and their sediments. Note that Cullendulla Beach and Bengello Beach have been included in this table because they are adjacent to, but not included in, the erosion/recession hazard assessment. The critical offshore wave direction identified for each beach was determined following modelling of waves from seven (7) different offshore wave directions, as described in Appendix D. The wave direction shown in Table 3-3 is the direction which results in the maximum wave conditions at each beach.

Table 3-3: Beach and Sediment Characteristics of the Study Sites

Beach	Embay. Ratio (-)[1]	Orient. (°TN)[2]	Critical (Design) Offshore Wave Direction (°TN)[3]	H_S (m)[3]		100 year ARI Storm Demand (m^3/m above 0 m AHD)	Beach State[4]	Beach Swash Slope (1V:?H)	Median Sand Size, D_{50} (mm)	Sand Sorting (Standard Deviation)		Carbonate Content of Sand (%)
				Median	100 year ARI					Quant. (Phi Units, Ø)	Qual.	
Maloneys Beach	0.58	200	180	0.4-0.5	1.5-1.9	50-80	R-LTT	10	0.21	0.90	Moderate	69-76
Long Beach	0.68	165	157.5	0.4-0.7	2.0-3.0	70-120	LTT-TBR	9-18	0.24	0.30	Very well	64-78
Cullendulla Beach	0.55	190	157.5	0.2	0.9	N/A	B+SF	24	0.18	1.60	Poor	62
Surfside Beach (east)	0.82	145	135-157.5	0.3-0.4	1.5-1.6	50-60	LTT	13-18	0.25	0.65	Moderate	20
Surfside Beach (west)	0.91	220	157.5	0.2	0.7	20	B+SF	20	0.21	0.64	Moderate	20
Sunshine Bay	0.52	70	112.5	0.4	4.0	25	R	11	0.21-1.01	0.90	Moderate	62 (sand), 1 (gravel)
Malua Bay	0.69	100	112.5	1.1	6.4	120	TBR	12	0.29-0.40	0.32	Very well	78
Guerilla Bay (south)	0.38	80	90	0.5	4.3	80	LTT	12	0.28-0.30	0.28	Very well	45
Barlings Beach	0.61	180	180	0.6-1.0	2.8-3.5	60-110	TBR	10-21	0.28-0.32	0.42	Well	74
Tomakin Cove	0.19	140	112.5	0.6	3.7	90	LTT	26	0.19	0.42	Well	71
Broulee Beach	0.60	70	90-112.5	0.4-0.9	1.8-3.5	70-110	TBR-LTT	23-30	0.21-0.22	0.42	Well	48-84
Bengello Beach	0.87	120	112.5	1.2-1.3	5.6-5.7	170	TBR-RBB	18	0.22-0.35	0.41	Well	4-5

(1) Embayment Ratio = straight line distance (chord) between controlling headlands / curved shoreline length (i.e. deeper bays have a lower embayment ratio)

(2) Beach Orientation

(3) The critical (design) offshore wave direction, median *H*s, and 100 year *H*s for each beach are specified with additional information (including bed elevation) in Appendix D, Table D-5.

(4) Beach States RBR = rhythmic bar and beach
TBR = transverse bar and rip
LTT = low tide terrace
R = reflective
B+SF = beach and sand flats

3.3.2 Batemans Bay Secondary Compartment

The Batemans Bay sediment compartment (NSW02.06.02) extends along 24 km of shoreline between Three Islet Point and Mosquito Bay head. The bay is 6 km wide at its entrance, narrowing to 300 m at the bridge. It is bedrock-controlled and has a series of ten (10) embayed beaches along its northern shore and eight (8) along its southern shore, including the artificially accreted Corrigans Beach. The bay faces south-east and has acted as sink for marine quartz and carbonate sand, which has built the beaches and barriers, the large shallow flood tide delta and more recently Corrigans Beach (Figure 3-2). Like most flood tide deltas, this is a dynamic system with the tidal delta channels and shoals in a state of dynamic equilibrium, which causes them to change location through time in response to tidal flows, waves and storms and sediment availability. This in turn can have substantial impacts on the adjacent shoreline, particular the inner parts of the bay including Surfside Beach and Corrigans Beach.

Wright and Thom (1976) investigated the nature of the surface sediments in Batemans Bay and identified three provinces. An *outer estuary-offshore* province occupies much of the flood tide delta and outer bay floor with fine to very fine lithic (quartz) sands and 35-50% fine calcareous sands; an *outer estuary-inshore* province extends around the perimeter of the bay shore, including its beaches, and has medium to coarse lithic (quartz) sand and ~50% carbonate; and an *inner estuary* province is located in the Clyde River west of the bridge with fine to medium lithic (quartz) sands and low carbonate. These results indicate three (3) sources of sediment. The lithic-quartz material is derived from both the Clyde River fluvial sands, as indicated by the lower carbonate west of the bridge, as well as inner shelf sands transported landwards during the sea level transgression, while the carbonate (molluscs and foraminferia) is produced in-situ. Wright and Thom (1976) also found that the sediments in Batemans Bay and the lower Clyde River show a high degree of mobility which lead to pronounced changes in the ebb tide channel that flows against the training wall, the ramp margin shoals that flank the channel along its northern boundary, and the ebb tide bar located at the eastern end of the channel. They also found the flood and ebb tides follow mutually exclusive paths with the tides flooding through the northern channel, close to Surfside Beach, and ebbing through the southern channel along the training wall. They found that while the channels occupy the same general position, over time the detailed configuration of the bars and shoals are continually changing. These changes affect wave refraction, direction, height and sediment movement at the shore and may have been a contributing factor to Surfside Beach erosion and accumulation of sand on Corrigans Beach.

The degree of sediment mobility within the bay is also demonstrated by the impact of the construction of the first training wall, completed in 1905, and its extension in 1991. At least 650,000 m^3 of sand accumulated in the lee of the wall to accrete the shoreline up to 600 m into the bay and build Corrigans Beach. The wall would have also modified the ebb tide channel by 'training' it along its length and thereby fixing the location of the ebb tide shoal or sand bar located at the eastern end of the channel (also illustrated in Figure B-29).

The northern four (4) Batemans Bay beaches face south into the prevailing swell, which is, however, increasingly lowered within the bay, resulting in four low to very low energy beaches, with Long Beach (west) being the most exposed and Surfside Beach (west), which faces southwest across the narrow inner bay, having the lowest energy. They all consist of fine sand (0.21-0.25 mm), which is very well-sorted on Long Beach grading to well-sorted on the others. Carbonate content is high at Maloneys (76%), Long (78%) and Cullendulla (62%) Beaches, then decreases markedly at Surfside Beach (20%). These figures show that Maloneys, Long and Cullendulla Beaches derived sediment from a similar source – the flood tide delta, whereas Surfside Beach with its substantially lower carbonate has a separate source, possibly fluvial sand

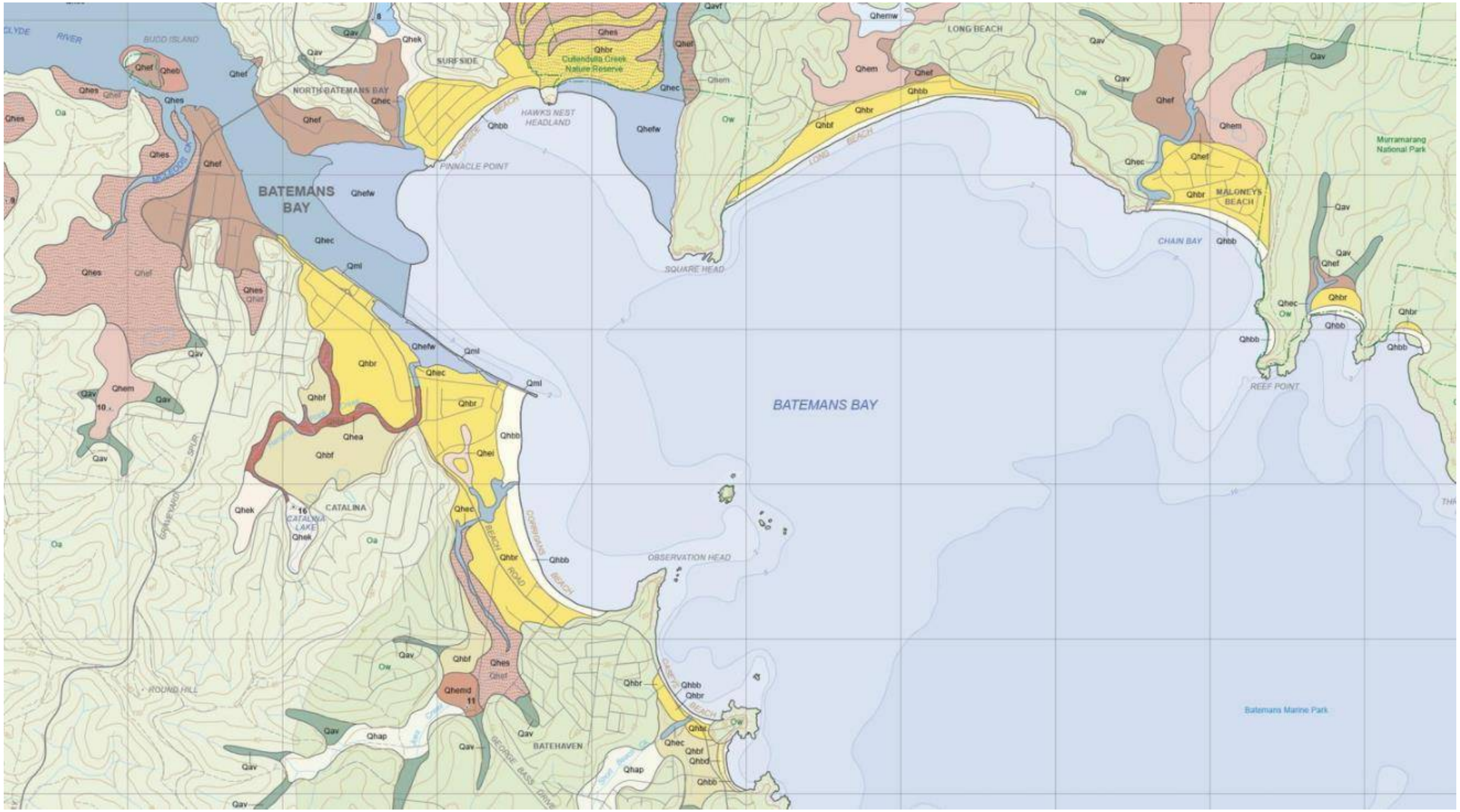


Figure 3-2: Quaternary geology of Batemans Bay clearly shows the infilling of the tributary valleys with river, estuarine and marine sediments, as well as the shallow flood tide delta (Source: Troedson and Hashimoto, 2013)

from the Clyde River. The two Surfside Beaches are also connected via sand moving around the dividing low point and have essentially identical sediment. Sunshine Bay on the southern side of the bay has no linkages with the northern beaches or the flood tide delta.

The following three types of rip currents can occur on the Batemans Bay beaches:

- Beach rips;
- Boundary, headland or topographic rips; and
- Mega-rips.

During and following periods of higher waves, beach rips are present on the most exposed beach (Long Beach), and these cut through the sand bar with a rip channel, with bars to either side.

Boundary rips occur on Maloneys Beach and Long Beach where waves break next to the rocky headlands. At the western end of both beaches, a boundary rip flows out against the rocks. These rips may be quite small during low waves increasing in size and intensity as wave height increases.

Mega-rips are large scale rips that occur on embayed beaches during periods of high waves ($H_s > 2\text{-}3$ m). As wave height, beach rip size and spacing increases on embayed beaches (<1-3 km in length), one rip cell can occupy the entire embayment. This can occur at Sunshine Bay. Where the rip is located and exits the embayment depends on wave height and direction, and the embayed configuration. Mega-rips are large, flow at speeds of up to 3 m/s and flow further seaward, depositing eroded sand in deeper water. The most severe erosion on embayed beaches usually occurs in association with mega-rips.

Maloneys Beach

Maloneys Beach (Figure 3-3) is an 810 m long embayed beach located just inside the northern entrance to Batemans Bay. It occupies a drowned valley that has been infilled with estuarine and marine sands, the latter building a 460 m regressive beach-foredune ridge barrier with a volume of ~2 M m^3 (Table 3-2). While it faces almost due south (200°) it is sheltered by its eastern Acheron Ledge and the Tollgate Islands, with waves averaging only about 0.4 m, increasing slightly east along the beach. Sediments are fine, moderately-sorted, carbonate-rich (78%) sand (Table 3-3), with some cobbles eroded from the adjacent headland present along the eastern end of the beach and a slight increase in grain size to the west. This increase suggests a stable sediment compartment usually free of beach rips, that is, the sand has rearranged itself over time with no longshore transport and little intra-beach transport. However, during high waves a temporary boundary rip flows out against the western rocks and would transport some sand out into the bay. The beach grades from a low energy low tide terrace (LTT) with no cusps in the east to a slightly higher energy LTT with high tide cusps in the west. It is narrow (~10 m), moderately steep (1V:10H) and backed by a low foredune and the now developed foredune ridge plain. The valley has acted as a sink for sand moving into the bay, which led to the development of the barrier system. This system now appears to be stable with the recent photogrammetry indicating no net recession, but a possible counter-clockwise rotation of the shoreline. It is very unlikely any sand is moving west around the Acheron Ledge, nor moving from Maloneys Beach around its western rocky point into the Long Beach compartment. While they may be similar in sediment texture and source, they do not appear to be laterally connected. It appears to be a compact tertiary sediment compartment with on-offshore transport during erosion-recovery events, but no lateral connections. Storm demand for Maloneys Beach is expected to be in the order of 50 m^3/m in the east increasing to 80 m^3/m in the west. This would equate to a total beach demand of ~50,000 m^3. In addition to beach erosion/recession, the system is exposed to both stream flooding and marine inundation along Maloneys Creek and into the backing wetland.

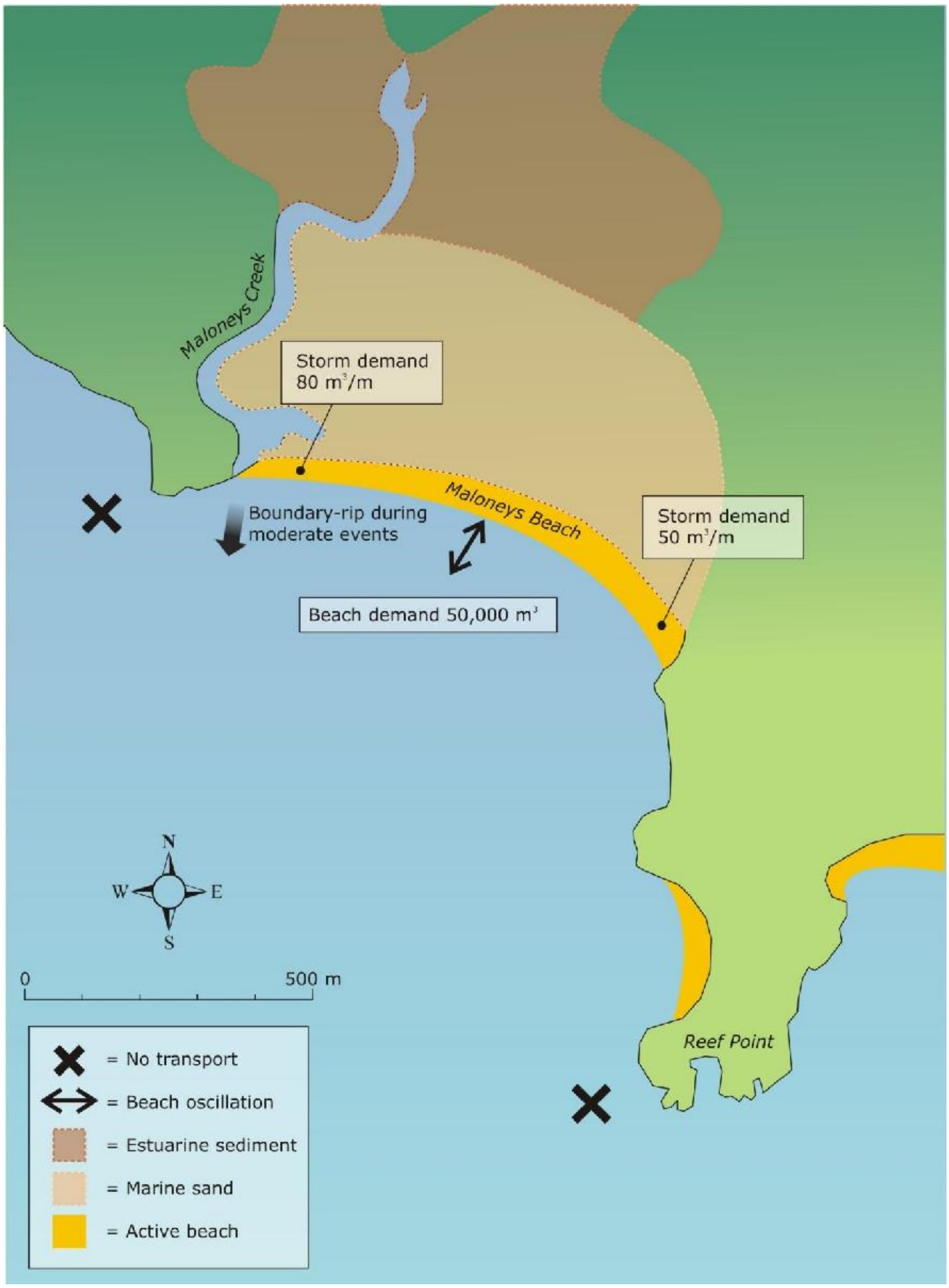


Figure 3-3: Conceptual model of sediment movement and storm demand at Maloneys Beach

Long Beach

Long Beach (Figure 3-4) faces south-east into the prevailing southeast swell and is the highest energy beach inside the bay. Nonetheless, it is afforded some protection by its eastern headland and reefs and the Tollgate Islands. Waves are low at the eastern end averaging about 0.4 m, increasing west of the creek to average about 0.7 m. The 2.15 km long beach is embayed between its eastern headland and Square Head. These and the backing central valley have acted as a sediment sink and lead to the formation of a 200 m wide regressive foredune ridge barrier which has a volume of about 2.3 M m^3 (Table 3-2). Reed Swamp, a wetland and lake occupies the central back-barrier depression. The beach sediments consist of very well sorted, fine (0.24 mm) carbonate-rich (63-78%) sand (Table 3-3), with a slight east to west increase in grain size which suggests a stable sediment compartment, that is, the sand has rearranged itself over time with no longshore transport and little if any intra-compartment transport with no apparent connection to the adjoining compartments (Maloneys and Cullendulla Beaches). There are some lithic pebbles-cobbles derived from the eastern headland along the eastern end of the beach and these cobbles may underlie the eastern end of the beach. The beach grades in the east from a low energy LTT, shifting to a higher energy LTT to low energy transverse bar and rip (TBR) in the west, with beach rip channels and currents occurring during and following periods of higher waves, and a boundary rip flowing out against Square Head which would transport sand deeper into the bay. The beach has a moderate slope (1V:9H-1V:18H) and is relatively narrow in the east (~15 m), widening as wave energy increases to ~25 m in the west. The beach undergoes a possibly slight rotation in response to changes in wave direction, but there is no apparent longshore transport, definitely not to east, unlikely to west. Storm demand for the beach has been estimated at 70 m^3/m in the east, increasing to 100 m^3/m in the centre and 120 m^3/m in the west, which would generate a beach storm demand of ~225,000 m^3. Photogrammetry indicates the beach has been accreting at 0.05-0.2 m/year since 1959, except for around the central creek mouth. To determine whether this is a long-term trend requires further data collection which is outside the scope of this study. This appears to be a compact individual tertiary sediment compartment with a longshore gradient in wave height, sediment size, beach slope and state, with no lateral connections and only on-offshore sand transport in response to storm events and recovery. It is also exposed to flooding and marine inundation via the central creek and the backing wetland, as well as inundation of the low eastern end of the beach.

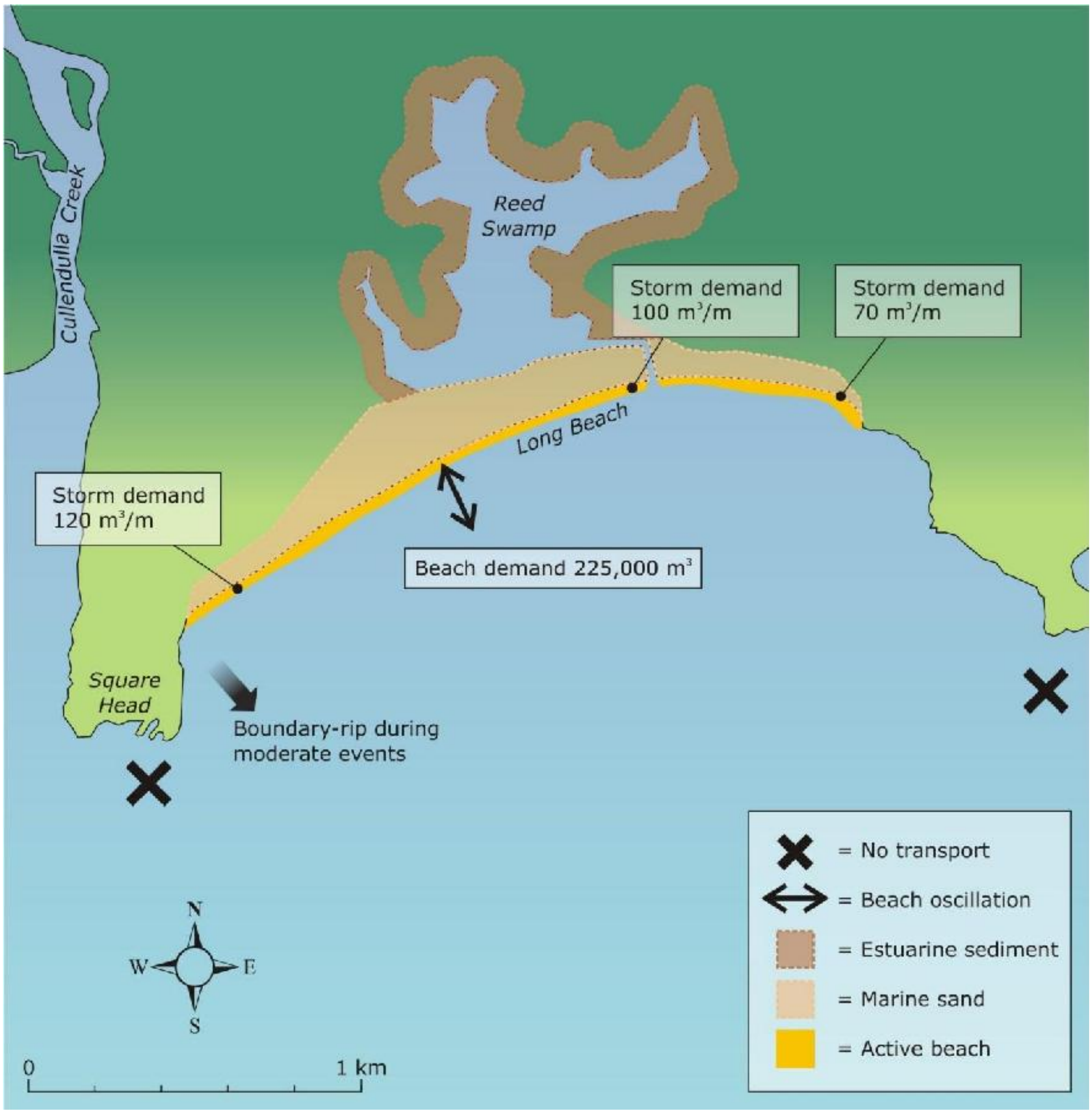


Figure 3-4: Conceptual model of sediment movement and storm demand at Long Beach

Surfside Beach (east and west)

The two adjoining Surfside Beaches (Figure 3-5 and more broadly in Figure 3-6) represent a transition to a lower energy system located deeper within the bay, one that is impacted by low waves, but increasingly by tides and periodic river flooding. The Surfside Beach (east) is 850 m long, faces south-east down the flood tide channel and receives waves averaging about 0.3 m in the north increasing to about 0.4 m in the south, while the shorter (270 m) Surfside Beach (west) faces south-west across the narrow bay, with waves averaging about 0.2 m. Both beaches are composed of identical moderately-well sorted fine sand, with 20% carbonate. The decrease in carbonate compared to the beaches to the east, suggests there is no westward sand transport to the beaches, rather they received sand from the flood tide delta and possibly the Clyde River. The longer Surfside Beach (east) is backed by a 440 m wide low regressive beach ridge plain, with the western beach forming the western side of the plain, with a total volume of ~780,000 m^3 (Table 3-2). Both beaches are low and prone to overtopping. They are also narrow (15 m in the east, 10 m in the west), with a low to moderate gradient (1V:13H-1V:20H, Table 3-3). The higher energy Surfside Beach (east) consists of a wave-dominated LTT which is usually free of beach and boundary rips, while the very low energy Surfside Beach (west) switches to a tide-dominated beach plus sand flats (B+SF) which extend up to 150 m off the shoreline. Sand is moving from Surfside Beach (east) around the low rocky point and is manifest on Surfside Beach (west) (Figure 3-5) as a series of 2-3 low, east trending sand waves. This sand moves west along the tidal flats and into the flood tide channel and becomes part of the flood tide delta. These sediments are likely to be recycled through the flood tide delta, its ebb and flood tide channels and associated tidal shoals (Figure 3-6). The rate of transport along the beach would be expected to be very low, in the order of 100's m^3/year, with most activity during periods of higher waves. There has been substantial erosion and property loss at Surfside Beach (west), which may be related to the dynamics and movement of the flood tide delta and its impact on the adjacent shorelines.

The northern end of Surfside Beach (east) was nourished with approximately 12,000 m^3 of sand (lineal placement extent unknown) obtained from the hind-dune area of Corrigans Beach in 1996 (WBM Oceanics, 2000). Surfside Beach (west) was nourished with 3,100 m^3 of sand (resulting in an addition of approximately 33 m^3/m above 0 m AHD) from routine dredging of the Batemans Bay bar in December 2016 (GPS & HS, 2017). The photogrammetry indicates a distinct counter-clockwise rotation of Surfside Beach (east). Surfside Beach (west) has greater shoreline oscillation owing to the impact of the migratory sand waves. Both beaches and their backing dunes are low and prone to creek flooding and coastal inundation. Their storm demands range from 50-60 m^3/m for Surfside Beach (east) and 20 m^3/m for the more sheltered Surfside Beach (west), which equates to beach storm demands of ~48,000 m^3 and 5,500 m^3, respectively.

Cullendulla Creek embayment is the only embayment within the Batemans Bay compartment that has been investigated in great detail. Lewis (1976) and Donner and Jungner (1981) cored and dated the regressive chenier to beach ridge sequence that has filled the embayment. They found the inner two cheniers were formed about 2,500-3,000 years ago, followed by accretion of the outer beach ridges from about 2,000 years ago, with the outermost beach ridge dating approximately 600 years ago, followed by a period of stability, though Cullendulla Beach is presently receding. This sequence of accretion cannot be directly applied to the neighbouring beaches because Cullendulla Beach is a substantially lower energy embayment which slowly filled with mud and then sand flats (between 6,500 to 3,000 years ago), following which the flats were capped by the cheniers then outer beach ridges. The higher energy regressive sandy barriers in Maloneys, Long and Surfside Beaches are more likely to have followed an evolutionary history like the Bengello and Pedro barriers, that is, accretion commencing about 6,500 years ago and continuing until they stabilised and formed the outer foredune.

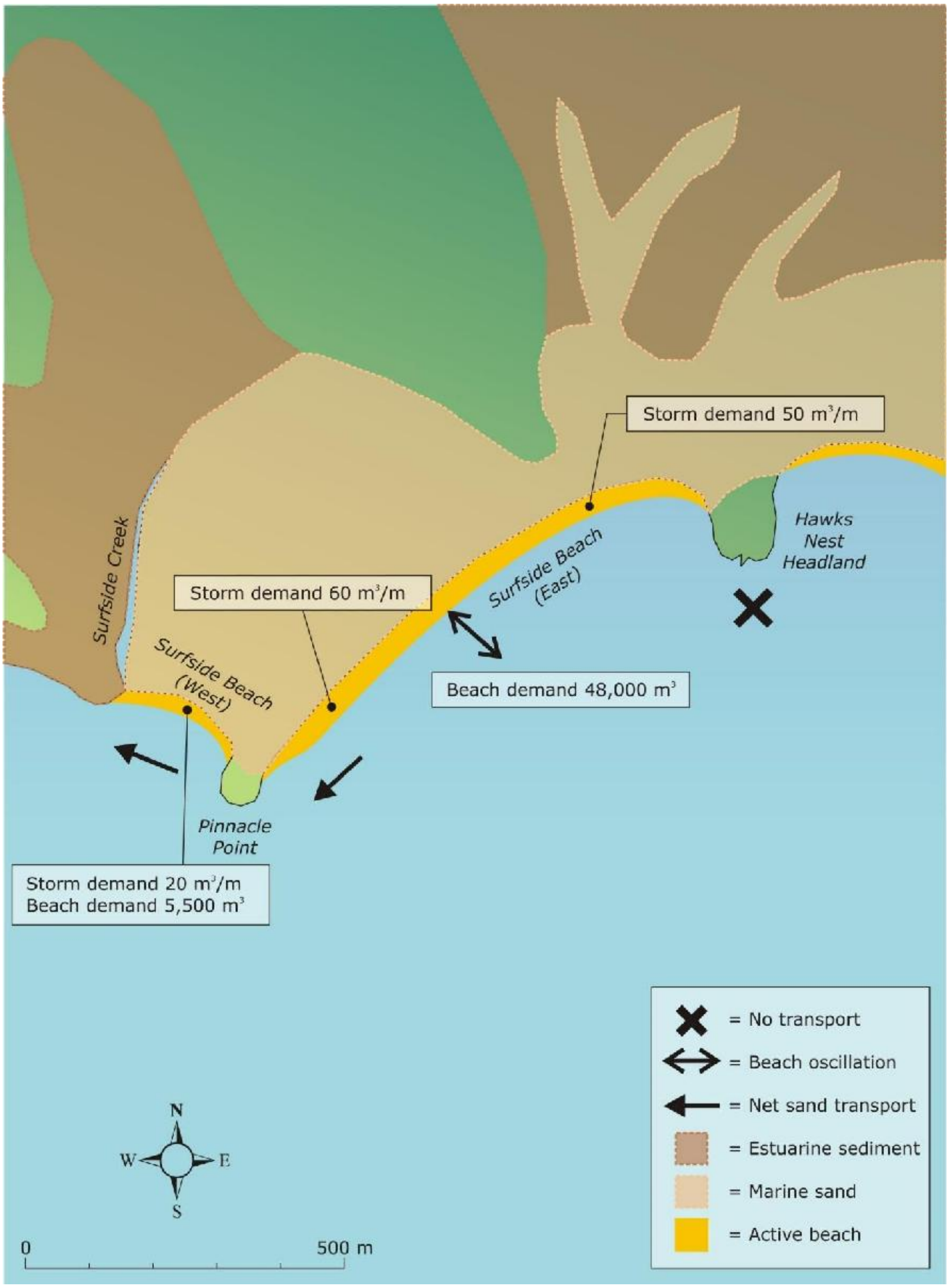


Figure 3-5: Conceptual model of sediment movement and storm demand at Surfside Beach

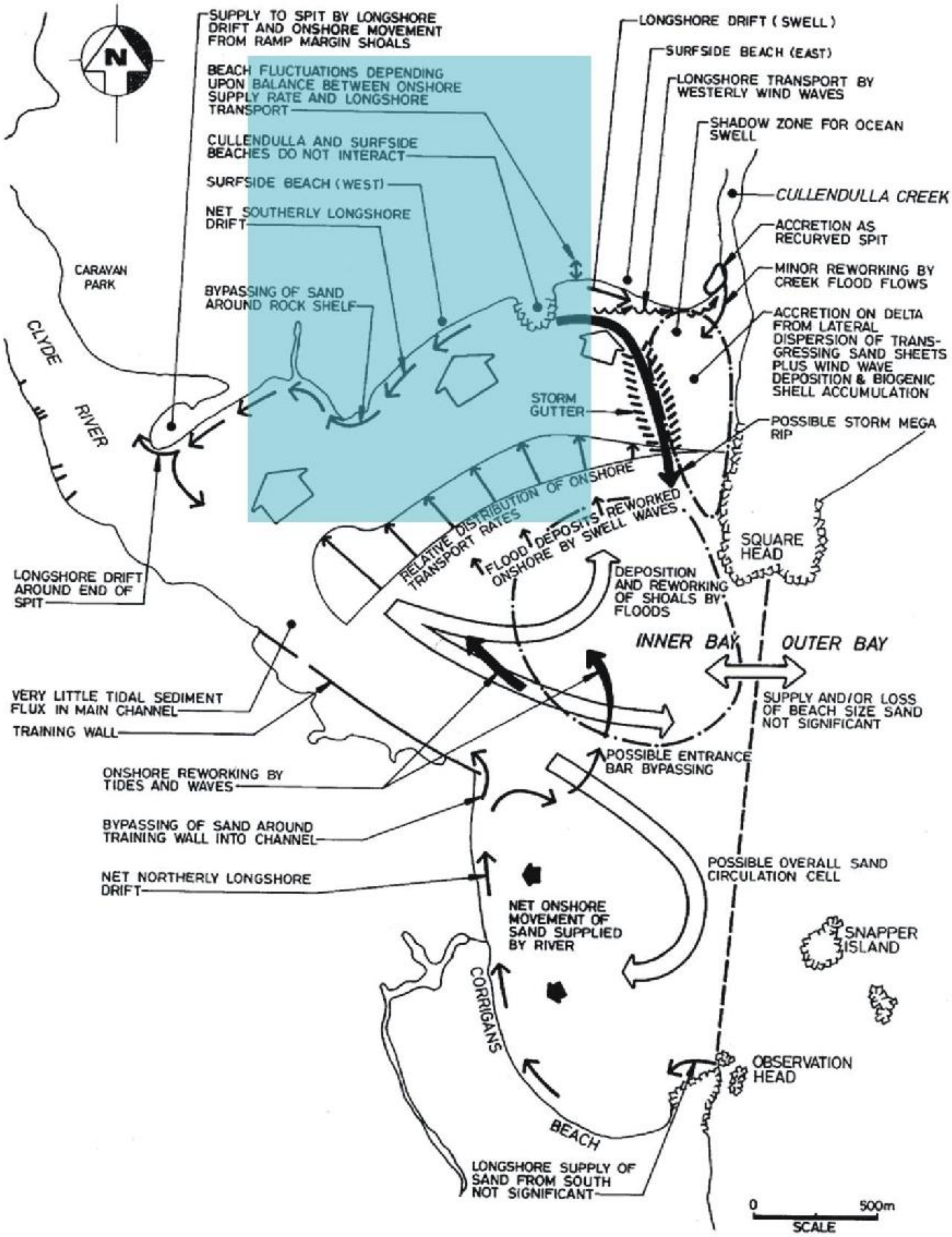


Figure 3-6: Conceptual model of sediment transport pathways within inner Batemans Bay (after Patterson Britton and Partners, 1992)

Sunshine Bay

Sunshine Bay is a small (520 m) curving beach located in a semi-circular embayment (embayment ratio = 0.52) (Figure 3-7) as well as being sheltered by rock reefs that occupy much of the bay floor. It faces east-northeast (70°) and as a result of its orientation and protecting headland and reef, receives low waves and is usually free of beach rips, averaging a wave height of only 0.4 m in the centre of the beach decreasing to the north and south. Its sediments are a bimodal mix of moderately-sorted fine sand and very coarse sand and cobbles, with the fine sand containing 62% carbonate and the coarser material just 1%. This is a distinct tertiary sediment compartment with no connection to the north or south and its own distinctive sediment suite, the coarser material derived from the surrounding rocks and reefs. The beach is moderately steep (1V:12H), reflective, with the coarser material arranged into prominent beach cusps. It is backed by steep cliffs to either end, and a small low central foredune, with essentially no barrier. Note that WRL considers that the coastal quaternary geology map (Figure 3-2) to be inaccurate along the central-northern end of Sunshine Beach. Based on multiple site inspections, this section of the beach is considered to be backed by cliffs and slopes composed of steeply dipping metasedimentary rocks (shales, siltstone and some sandstone) rather than marine sediment. This assumption has been reflected in the conceptual model (Figure 3-7) and erosion/recession hazard mapping for Sunshine Bay.

This self-contained beach and tertiary sediment compartment undergoes limited oscillation, with the photogrammetry indicating recent accretion of approximately 0.05 m/year since 1962. This is unlikely to be long-term owing to the small size of the existing beach and foredune, which shows no evidence of accretion and which has a volume of just 60,000 m^3. In addition, during large waves it is expected that water will build up inside the reefs and pulse seaward (flow out) through the reef-controlled centre of the bay as a small mega-rip, which could transport sediment out of the system leading to a net loss of sediment. This could explain the small size of the beach. The storm demand is estimated to be on the order of a low 25 m^3/m, which would equate to a beach storm demand of 14,000 m^3 (Table 3-2). In addition, overtopping could lead to future inundation of Beach Road located approximately 40 m west of the centre of the beach.

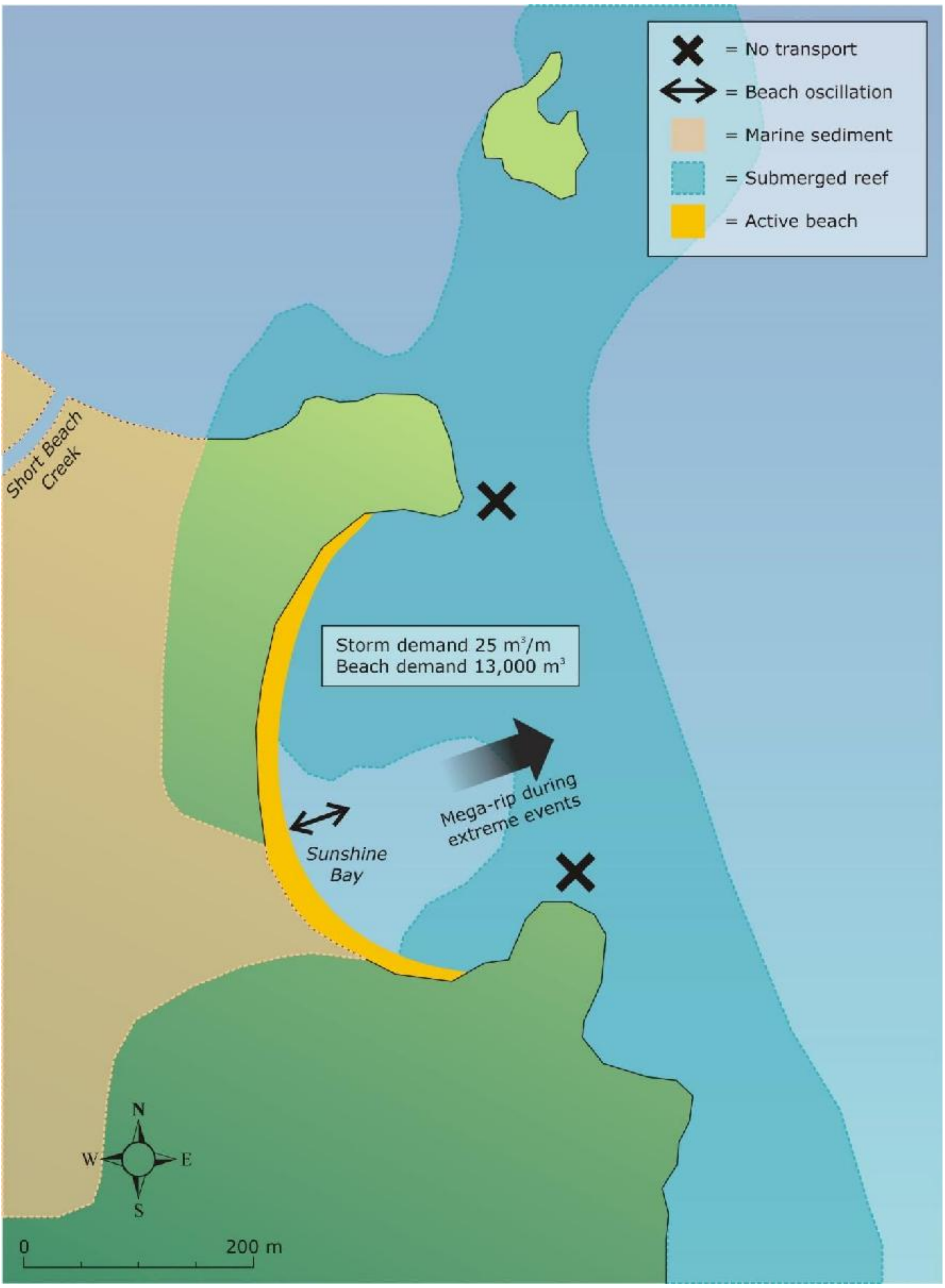


Figure 3-7: Conceptual model of sediment movement and storm demand at Sunshine Bay

3.3.3 Broulee Secondary Compartment

The Broulee sediment compartment consist of a series of embayed beaches and their associated tertiary sediment compartments including Malua Bay, Guerilla Bay, Barlings Beach-Tomakin Cove and Beach and Broulee Beach-Bengello Beach. The longer Broulee Beach does have periodic connection to Bengello Beach to the south when the tombolo to Broulee Island is severed. Figure 3-8 shows the dramatic change in the nature of the shoreline between the northern rocky shore with small embayed beaches with very small separate tertiary sediment compartments (Sunshine to Long Nose Point) and the large regressive barriers of Barlings Beach-Tomakin Cove and Beach and Broulee Beach-Bengello Beach and their larger and linked sediment compartments (Figure 3-9). This morphology is a reflection of the larger accommodation space available in each of the central bays and the abundant source of lithic quartz sediment from the Moruya River via the inner shelf, and north of Broulee Island, supplemented by local carbonate production.

Of most interest here is the very low carbonate (4%) and medium sand at the southern Bengello Beach (Table 3-3). At the northern end of Bengello Beach (southern side of Broulee Island tombolo) the carbonate increases to 48% and in the adjoining Broulee Beach it increases to 84% at its northern end. All the remaining beaches to the north remain high in carbonate (45-77%). This implies there is a major change in sediment texture and source between Bengello Beach and Broulee Beach, and beaches to the north. While Bengello Beach is composed of quartz-lithic sand ultimately derived from the Moruya River, the beaches to the north have a substantial amount of their sediment derived from the local marine biota. This was first observed by Hall (1981) and Ballard (1982) (as reported in Thom et al. 1986) who mapped the beach and seabed sediments between Tuross Head and Barlings Beach. They found the Bengello Beach sediments are fine, well-sorted quartz with low carbonate, extending up to 25 m depth, whereas the Broulee Beach to Barlings Beach nearshore sediments are medium grained, moderately to poorly-sorted carbonate-rich sands. The beach material therefore reflects the nearshore material, with Broulee Island separating the two compartments. However, as the tombolo to Broulee Island is breached during major storms, there is periodic leakage of the quartz-rich sand into the Broulee compartment, which explains the lower carbonate content on the southern side of Broulee Island tombolo.

Figure 3-8: Quaternary geology of the northern Broulee compartment. The section between Sunshine Bay and Long Nose Point (east of Barlings Beach) is dominated by metasedimentary rocky shore and small embayed beaches (Source: Troedson and Hashimoto, 2013)

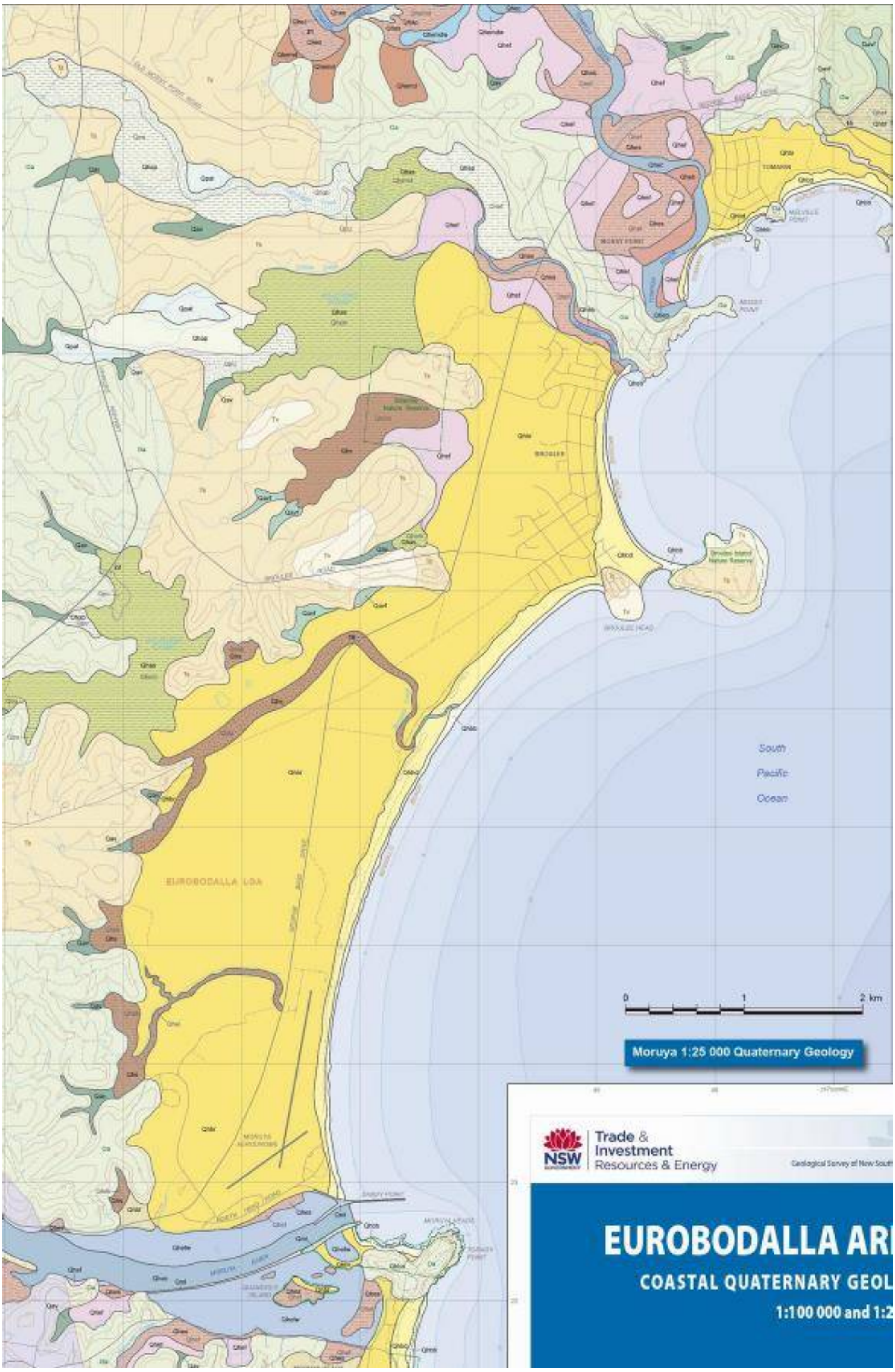


Figure 3-9: Quaternary geology of the central Broulee compartment. The Barlings Beach-Tomakin Cove and Beach and Broulee Beach-Bengello Beach embayments have accumulated large regressive barriers (Source: Troedson and Hashimoto, 2013)

Malua Bay

Malua Bay is a 510 m long east-facing (100°) embayed (0.69) beach bordered by Malua Head in the north and rocky shore leading to Pretty Point in the south (Figure 3-10). It is reasonably well exposed to waves from the east through south, with a median wave height of 1.1 m. The beach is composed of very well-sorted medium sand (0.29-0.40 mm), which increases slightly in size from north to south and contains 77% carbonate. The beach is moderately steep (1V:12H) with a 100-150 m wide TBR surf zone, 1-2 central beach rips and permanent boundary rips against the north and south headlands. During high south waves (the predominant storm condition), these combine to form a large mega-rip flowing out against the northern headland. Waves with incident directions between north and east could cause the mega-rip to flow out against the southern headland. The beach is backed by a low 50-100 m wide foredune region that may have been lowered when the park and road were constructed. This small barrier has a volume of approximately 275,000 m^3. There has been no substantial accretion of the barrier and the beach now appears stable. Photogrammetry indicates a dynamic but stable beach, with both erosion and recovery occurring. It is possible sand is lost via a mega-rip during major storm events to a depth from which it cannot return. If this is the case, the beach may be slightly erosional. The beach has a storm demand of 120 m^3/m, which equates to a beach storm demand of ~60,000 m^3. The beach is a closed tertiary sediment compartment with rocky coast extending more than a 1 km north and south and up to 500 m seaward and no longshore linkage to sand. While sand may be being lost offshore, most sand will be retained within the encircling rocks and reefs, however, the high carbonate content does indicate it can also receive carbonate material from the surrounding seabed.

Guerilla Bay

Guerilla Bay is a is a small (290 m) deeply embayed (0.38) beach sheltered to the south by the 1 km long Burrewarra Point and a tied-islet and rocky shore to the north (Figure 3-11). The beach is composed of very well-sorted medium sand (0.28-0.30 mm), with 45% carbonate material. It has a moderate slope of 1V:12H fronted by a 40 m wide LTT (Table 3-3). Wave average between 0.5 m and rip channels only occur during and following periods of higher waves, with a mega-rip draining the embayment during high wave conditions. The beach is backed by sea cliffs to either end, a small central creek and a small single 30-50 m wide foredune and a very small barrier with a volume of ~100,000 m^3 (Table 3-2). The limited amount of sand in the embayment and its moderate carbonate content indicates this is a separate small tertiary sediment compartment, with no longshore linkages, but with the possibility of offshore loss via a mega-rip, and offshore supply of carbonate material, the potential rates of which are unknown. Photogrammetry indicates the beach has accreted approximately 0.15 m/year since 1962, however, given its small size and limited sand sources it would be unlikely to be a long-term trend. The beach has a storm demand of 80 m^3/m which is equivalent to ~23,000 m^3 for the entire beach.

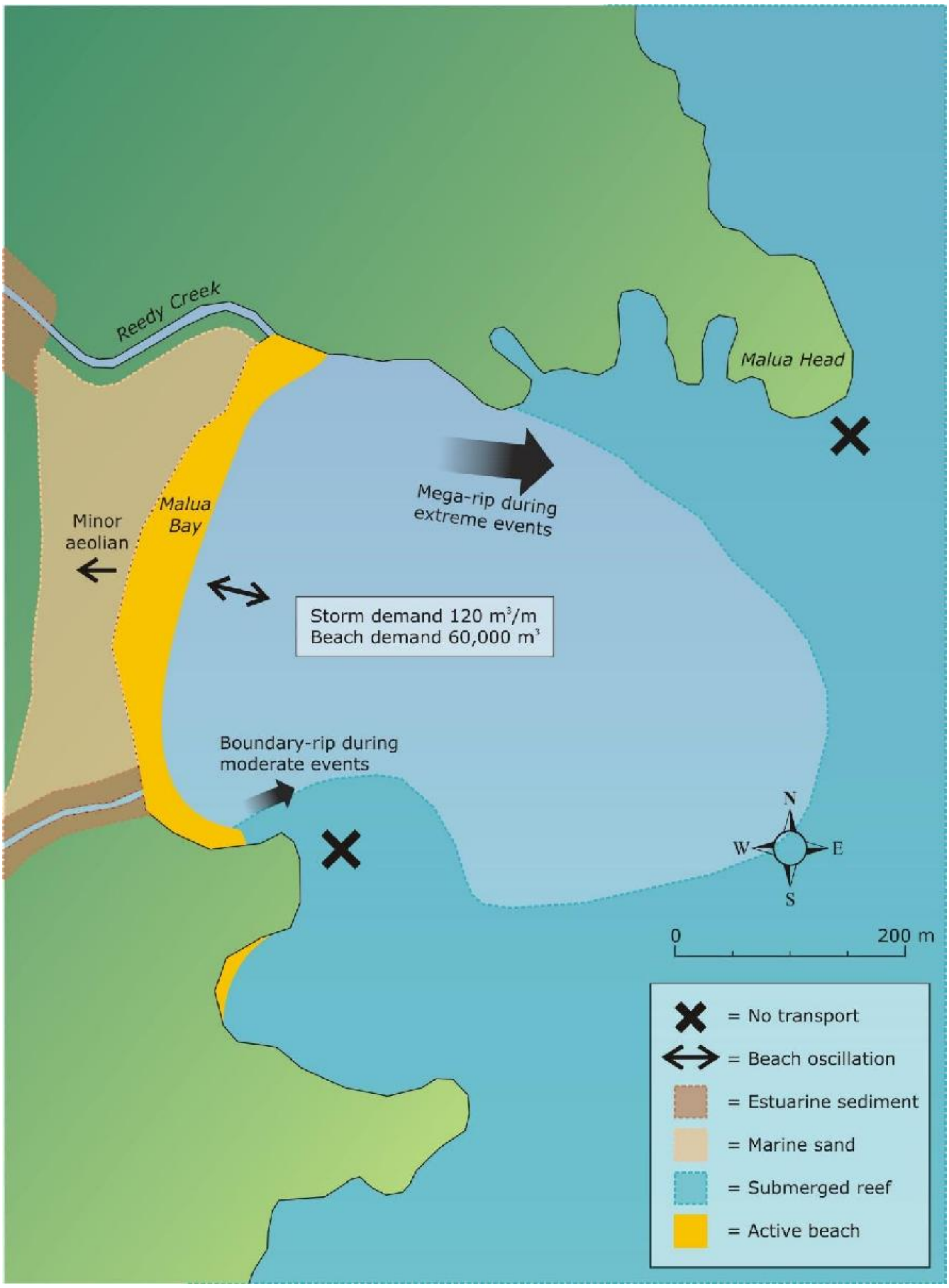


Figure 3-10: Conceptual model of sediment movement and storm demand at Malua Bay

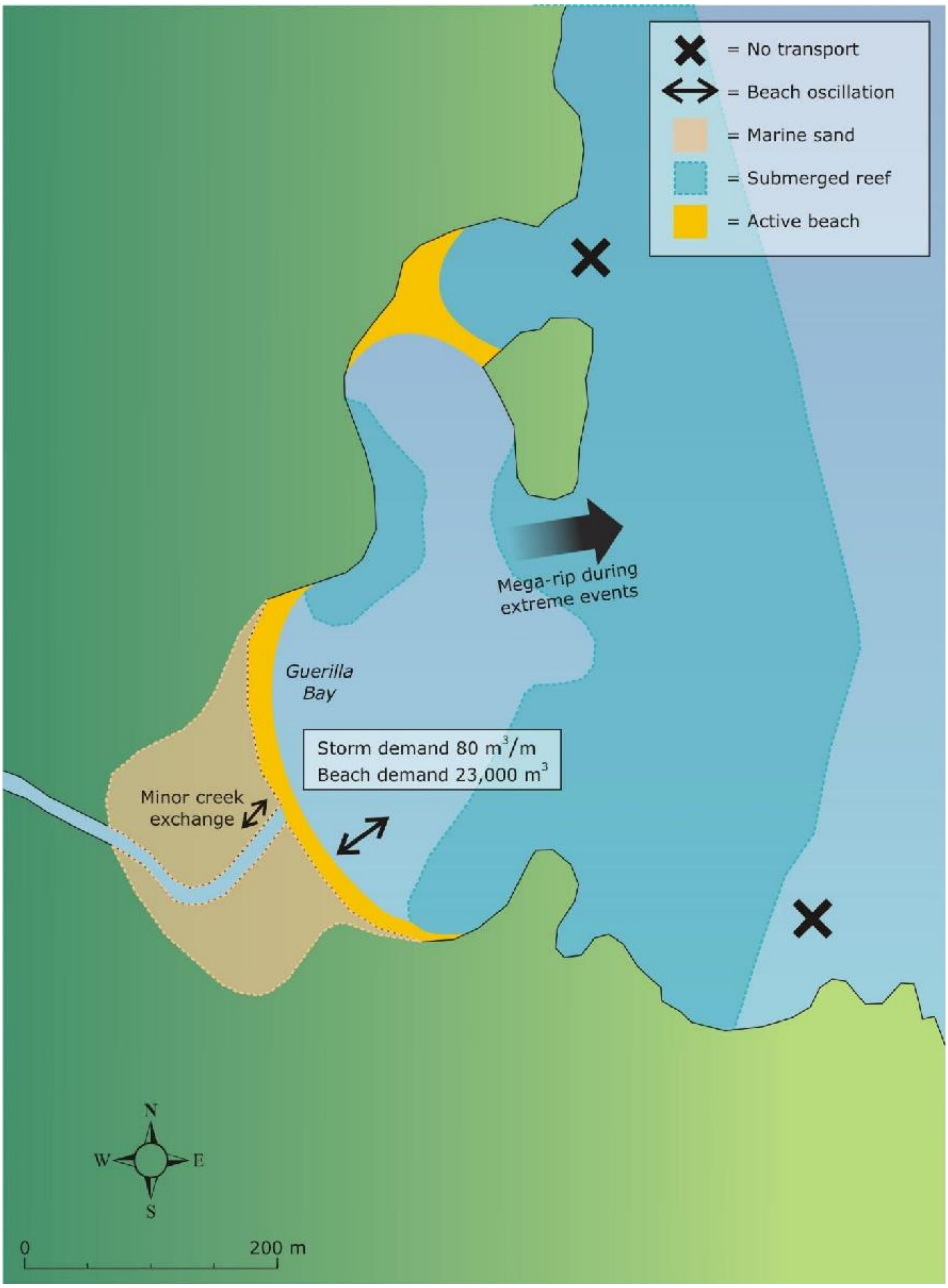


Figure 3-11: Conceptual model of sediment movement and storm demand at Guerilla Bay

Barlings Beach

Barlings Beach is located on the southern side of Burrewarra Point and faces due south (180°) into the prevailing southerly swell. It is moderately embayed (0.61) between the rocky Barlings Island and the high Melville Point, with Barlings Island and adjacent reefs providing some shelter to the eastern end of the beach (Figure 3-12). The 1.11 km long beach is composed of very well-sorted medium sand which increases in size form 0.28 mm in the east to 0.32 mm in the west and composed of 74% carbonate (Table 3-3). At the same time wave height also increases from an average of 0.6 m in the east to 1.0 m against Melville Point. The waves maintain a 40 m wide LTT in the eastern corner, with rips beginning about 200 m along the beach and usually 5-6 beach rips and a large boundary rip flowing out against Melville Point. During high waves, these combine to form a large mega-rip flowing out against Melville Point. The south-facing embayment, together with its Tomakin Cove and Beach neighbour has acted as a major Holocene sediment sink and the development of a regressive foredune ridge barrier that has accreted 500 m into the bay and has a volume of ~2.9 M m^3 (Table 3-2). This accretion appears to have ceased with the outer foredune the highest and widest, suggesting a period of stability. The beach is backed by a beachfront development which is set back behind the foredune and at least 100 m from the beach, the dune providing a natural buffer against erosion and inundation. Photogrammetry since 1964 indicates the beach is accreting (~0.1 m/year) in the west and eroding in the east (0.08-0.15 m/year), possibly a sign of counter-clockwise rotation or possibly slight erosion. Only further monitoring can confirm if this is a long-term trend. The beach has a storm demand of 60 m^3/m in the east, increasing to 110 m^3/m in the west, with a beach storm demand of ~95,000 m^3.

Sand transported offshore via a mega-rip against Melville Point would be deposited in Broulee Bay. While the sand is expected to stay within the bay, it may be transported back into the neigbouring Tomakin Cove or even Tomakin Beach and vice versa, with sand transported into the bay from the Tomakin beaches transported back into Barlings Beach. The entire bay can therefore be considered a single tertiary sediment compartment containing Barlings Beach and the two Tomakin beaches, as well as the mouth of the Tomaga River. It is unlikely the compartment is connected to beaches to the north (Guerilla Bay) or south (Broulee beach). However, more detailed field investigations are required to confirm the nature and extent of this compartment.

Tomakin Cove

Tomakin Cove is a small (270 m) curving, deeply embayed (0.19) beach that faces south-east (140°) out through an 80 m wide gap in the rock reefs that extend 600 m south of Melville Point (Figure 3-12). A cuspate foreland formed in the lee of the reefs separates it from the neighbouring Tomakin Beach. The beach is composed of well-sorted fine sand (0.19 mm), with 71% carbonate (Table 3-3). Median waves are 0.6 m which maintain a low gradient (1V:26H), 50 m wide LTT usually free of rip channels. During low waves, water returns seaward through the gap in the reefs. As wave height increases, this flow becomes a strong, pulsating mega-rip draining the whole cove.

The beach is backed by the Barlings-Tomakin regressive barrier, which extends 650 m inland in lee of the cove. The Tomakin part of the barrier has a volume of ~2 M m^3 (Table 3-2) and, like the Barlings barrier, the higher, wider seaward foredune indicates that accretion has ceased and the barrier is now stable. The foredune provides a 20-60 m wide natural buffer between the beach and the backing road and houses. Photogrammetry indicates that the beach has been recently receding at a rate of approximately 0.07 m/year since 1962. Only further monitoring will verify whether this is a long-term trend.

Tomakin Cove has a storm demand of 90 m^3/m and a beach storm demand of ~24,000 m^3. As mentioned earlier, Tomakin Cove is part of the Barlings Beach-Tomakin Cove and Beach tertiary sediment compartment and it is directly connected to Tomakin Beach via the cuspate foreland. It is also connected to Barlings Beach via sand transported by mega-rips to the bay sea floor. While the gap between rock reefs at Tomakin Beach is wider than at Tomakin Cove, a similar mega-rip (assisted by discharge from the Tomaga River on ebb tides) will flow out from the centre of Tomakin Beach under high wave conditions. Mapping of the seafloor sediments by Hall (1981) indicates a uniform fine to medium sized carbonate-rich sand, similar to that on the beaches.

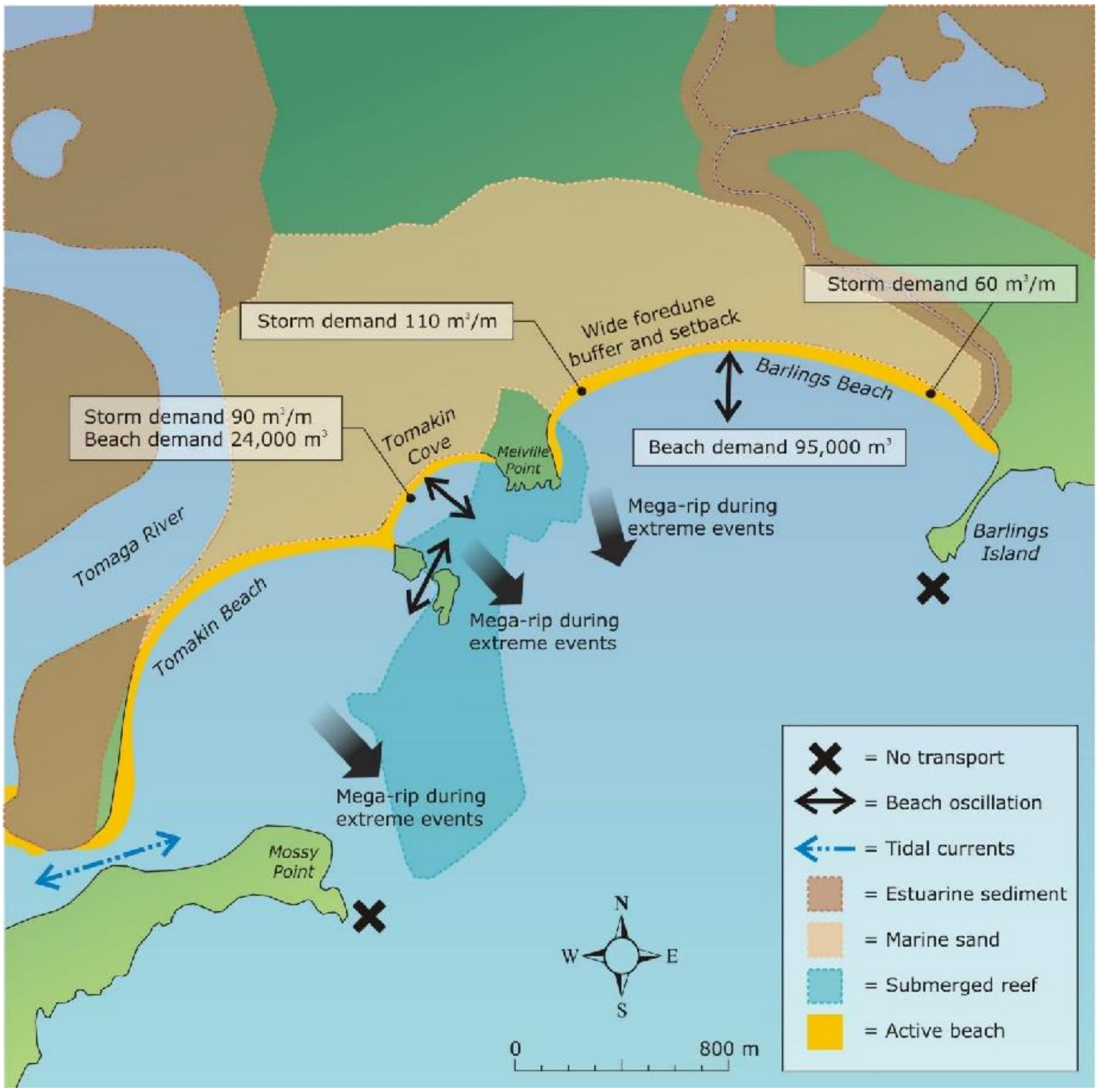


Figure 3-12: Conceptual model of sediment movement and storm demand at Barlings Beach and Tomakin Cove

Broulee Beach

Broulee Beach is a 1.74 km long, east-north-east facing (70°) curving embayed beach located between the northern Mossy Point and the large Broulee Island, which is tied by a tombolo to the southern end of the beach (Figure 3-13). The beach is moderately embayed (0.6), with the southern end very sheltered by the island, with median wave height increasing up the beach from 0.4 m in the south to 0.9 m in the north. The beach is composed of well-sorted, fine sand with carbonate content increasing from 48% on the southern side of the tombolo to 84% at the northern end of Broulee Beach (Table 3-3). The low waves maintain a reflective beach in the southern corner, grading northwards as wave increase to a LTT, then TBR with several beach rips usually present from about 1 km up the beach, extending to the northern end where a permanent boundary rip flows out against Mossy Point, assisted by flow from Candlagan Creek. During high southerly wave events, the rips increase in size and spacing, combining to form a mega-rip against the northern rocks of Mossy Point, with large rips also possibly operating down the beach.

The beach is backed by the northern part of the Broulee-Bengello barrier system, a large regressive beach to foredune ridge plain that is 1 km wide behind Broulee Beach, but up to 2 km wide behind Bengello Beach. The Broulee barrier has a volume of ~5.6 M m^3 (Table 3-2). The Bengello barrier has been investigated by Thom, et al. (1978, 1981) and more recently by Oliver et al. (2015). Oliver et al. found the barrier commenced accretion at the sea level stillstand approximately 6,500 years ago, and accreted seaward at a rate of 0.27 m/year or one foredune ridge every 110 years, until about 400 years ago when it appears to have stabilised and built the large outer foredune. A similar barrier evolution was recorded at Pedro Beach located 4 km to the south. Its 1.3 km wide regressive foredune ridge plain built seaward at a rate between 0.49-0.75 m/year, and ceased accreting about 700 years ago, followed by the accumulation of a large seaward foredune (Oliver, et al, 2017).

Broulee Beach is also linked to Bengello Beach via the tombolo at Broulee Island that divides the two embayments, forming one tertiary sediment compartment, which has a tenuous connection and periodic northward transport of low carbonate sand via the spit. This occurs when the spit is breached during major wave events and sand is washed into the Broulee Beach compartment (Ballard, 1982, Thom, et al., 1986). The photogrammetry data indicated overall beach accretion between 0.55-0.70 m/year since 1962, which decreases to the north, with slight recession at the northern end, which could be related to the mouth of Candlagan Creek. The recent accretion could be related to the last breach of the tombolo (sometime between May 1984 and May 1987), which would have supplied a pulse of sand to the southern end of the beach, which may have been reworked along the beach. The fact that the outer foredune is in the order of 400 years old suggests there has been no substantial accretion since that time. The beach has a storm demand of 110 m^3/m at the northern end, 90 m^3/m in the centre and 70 m^3/m at the southern end, with a total beach demand of ~155,000 m^3.

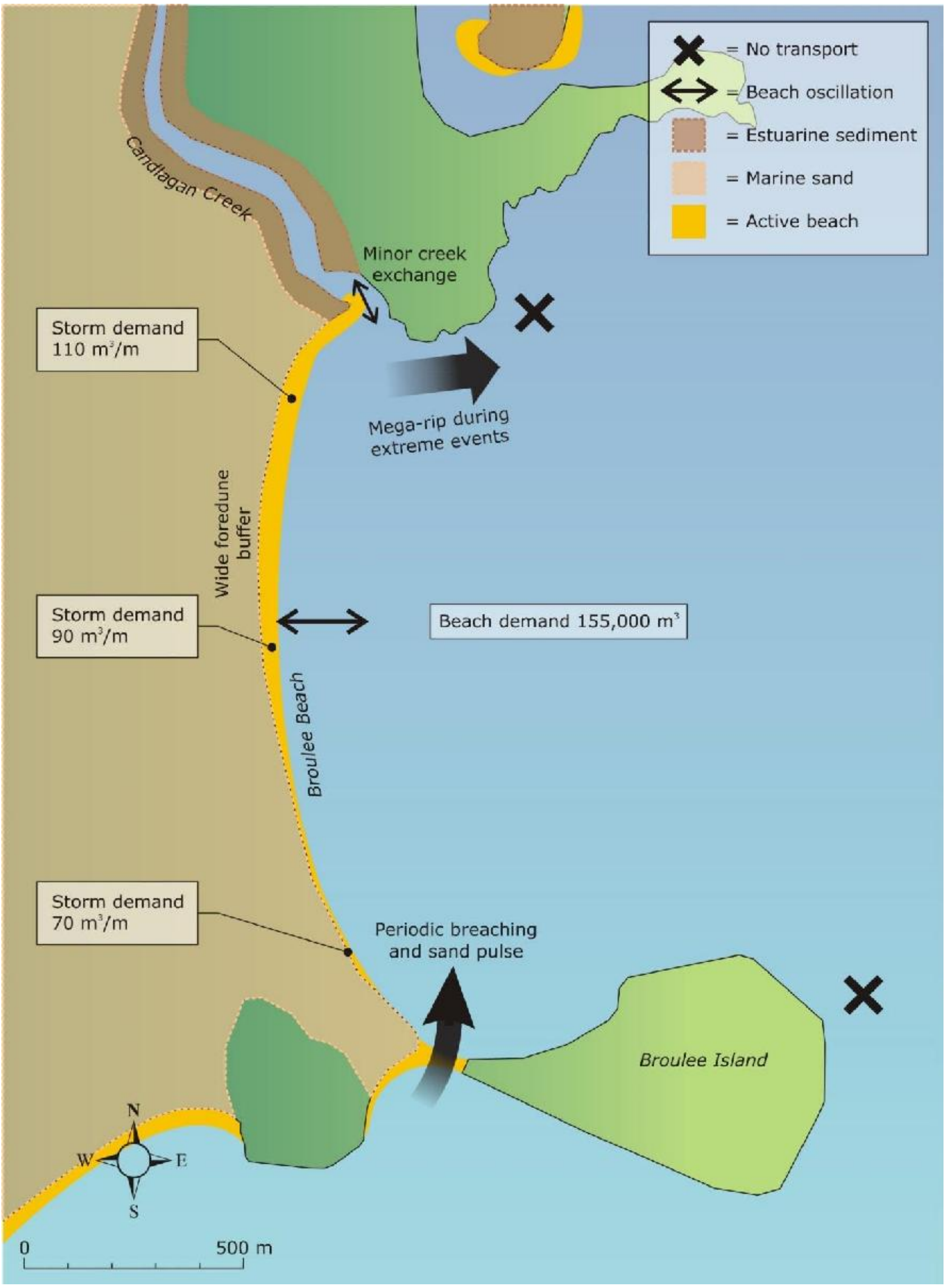


Figure 3-13: Conceptual model of sediment movement and storm demand at Broulee Beach

3.3.4 Summary of Geomorphology

The ten (10) beaches analysed in this section extend along 50 km of the Eurobodalla coast. They contain, however, considerable variation in their morphology, morphodynamics and storm demand. Their length ranges from 0.27-2.15 km, orientation (70-220°), embayment ratio (0.19-0.91), wave height (0.2-1.3 m) and beach state (B+SF to TBR). This variation is a product of the rugged coast with its numerous headlands, reefs, rocks and islands, which control coastal orientation and wave attenuation and refraction and thereby beach length, orientation, wave energy and ultimately sand supply. While the ten are similar in that they either consist of a small stable foredune (Sunshine Bay and Malua Bay) or regressive beach-foredune ridge system, their barrier volumes vary considerably from 0.06-3.0 M m^3. Likewise their storm demands vary both within some of the beaches (Broulee Beach: 70-110 m^3/m) and between all of the beaches (20-120 m^3/m). Most of the beaches are contained within their own separate tertiary sediment compartment, with weak transport linkages occurring within the Barlings-Tomakin compartment and the Broulee-Bengello compartment. This indicates the importance of considering each beach system and tertiary sediment compartment as a separate system that responds in its own way to storm events.

While the above provides a review about what we do know about the beach systems, there remain considerable unknowns. These include:

- the nature and scale of the on-offshore exchange of sand within compartments, and between the linked compartments;
- the potential permanent loss of sand offshore via mega-rips;
- the rate of carbonate production and its transport to the shore;
- the rate of carbonate abrasion and removal as fines (mud-silt);
- the supply of fluvial sand from the Clyde River into Batemans Bay;
- the supply of fluvial sediment from the Moruya River into the southern end of the Broulee Beach-Bengello Beach compartment.

Exploring these unknowns is outside the scope of this study and would require detailed field investigations to address them.

4. Assessment of Governing Physical Processes

4.1 Overview

Prior to assessing the coastal hazards, it was necessary to understand the coastal processes relevant to the study area. Coastal hazards are a direct consequence of coastal processes, which may adversely affect the built environment and the safety of people.

The coastal processes listed below are most relevant for this investigation and are assessed in the following sections.

- Water levels;
- Swells and local wind waves;
- Wave setup;
- Wave runup and overtopping; and
- Beach erosion and long-term shoreline recession.

The process of littoral drift (longshore sediment transport) was not directly assessed for this investigation due to the lack of connection between adjoining beach compartments, except between Surfside Beach (east and west). Long-term shoreline recession was assessed in two or three sections for longer beaches, allowing for the examination of long-term beach rotation or change due to gradients in net littoral drift.

The information presented in the following sections was acquired from the review of previous coastal processes reports, as well as from research, analysis and modelling undertaken specifically for this study.

4.2 Adopted Modelling Scenarios for the Coastal Hazard Assessment

Assessment of coastal erosion, shoreline recession, tidal inundation and coastal inundation was carried out for present day conditions and a set of future modelling scenarios.

Detailed information on the erosion/recession modelling and mapping is presented in Section 6, but a summary of the environmental conditions included in each map type and planning period is shown in Table 4-1.

Similarly the combinations of environmental conditions in each map type and planning period for tidal inundation and coastal inundation are shown in Table 4-2 and Table 4-3, respectively. Detailed information on inundation is presented in Section 7 (tidal) and Section 8 (coastal). These combinations were in accordance with the requirements of ESC and OEH.

Table 4-1: Modelling Scenarios for Erosion/Recession Hazard Mapping

Planning Period (Year)	**Modal SLR**[1] (m)	**Deterministic Method**			**Probabilistic Method**			
		100 year ARI Storm Demand (m^3 above 0 m AHD)	**Recession due to Sea Level Rise** (SLR× BF) [2]	**Underlying Shoreline Movement** (m/year ×years)	**Storm Demand PDF**[3] (m^3 above 0 m AHD)	**Recession due to Sea Level Rise** (SLR_PDF × BF_PDF) [2,3]	**Underlying Shoreline Movement PDF**[3] (m/year × years)	**Outputs**
2017	0.00	✓	×	×	✓	×	×	5% and 1% encounter probability
2050	0.22	✓	✓	✓	✓	✓	✓	5% and 1% encounter probability
2065	0.33	✓	✓	✓	✓	✓	✓	5% and 1% encounter probability
2100	0.71	✓	✓	✓	✓	✓	✓	5% and 1% encounter probability

Notes:

(1) Increase above 2017 Mean Sea Level.

(2) SLR: Sea Level Rise, BF: Bruun Factor

(3) PDF: Probability density function.

Table 4-2: Scenarios for Tidal Inundation Hazard Mapping (Excludes Wave Effects)

Planning Period (Year)	SLR[1] (m)	HHWSS Tidal Level Inundation			1 year ARI Inundation		
		Water Level	**Wind & Waves** (year ARI)	**Clyde River Flood** (year ARI)	**Water Level** (year ARI)	**Wind & Waves** (year ARI)	**Clyde River Flood** (year ARI)
2017	0.00	HHWSS[2]	nil	nil	1	nil	nil
2050	0.22	HHWSS[2]	nil	nil	1	nil	nil
2065	0.33	HHWSS[2]	nil	nil	1	nil	nil
2100	0.71	HHWSS[2]	nil	nil	1	nil	nil

Notes:

(1) Increase above 2017 Mean Sea Level.

(2) HHWSS: High High Water Solstices Springs tidal level.

Table 4-3: Modelling Scenarios for Coastal Inundation Hazard Mapping

Planning Period (Year)	SLR[1] (m)	1 year ARI Inundation			20 year ARI Inundation			100 year ARI Inundation		
		Water Level	**Wind & Waves** (year ARI)	**Clyde River Flood** (year ARI)	**Water Level**	**Wind & Waves** (year ARI)	**Clyde River Flood** (year ARI)	**Water Level**	**Wind & Waves** (year ARI)	**Clyde River Flood** (year ARI)
2017	0.00	MHW[2]	1	nil	20	20	10	100	100	50
2050	0.22	MHW[2]	1	nil	20	20	10	100	100	50
2065	0.33	MHW[2]	1	nil	20	20	10	100	100	50
2100	0.71	MHW[2]	1	nil	20	20	10	100	100	50

Notes:

(1) Increase above 2017 Mean Sea Level.

(2) MHW: Mean High Water tidal level.

4.3 Water Levels

4.3.1 Preamble

Coastal inundation is caused by elevated water levels coupled to extreme waves impacting the coast. Elevated water levels consist of (predictable) tides, which are forced by the sun, moon and planets (astronomical tides), and a tidal anomaly. Tidal anomalies primarily result from factors such as wind setup (or setdown) and barometric effects, which are often combined as "storm surge". Water levels within the surf zone are also subject to wave setup and wave runup. Figure 4-1 diagrammatically represents the different components contributing to coastal inundation.

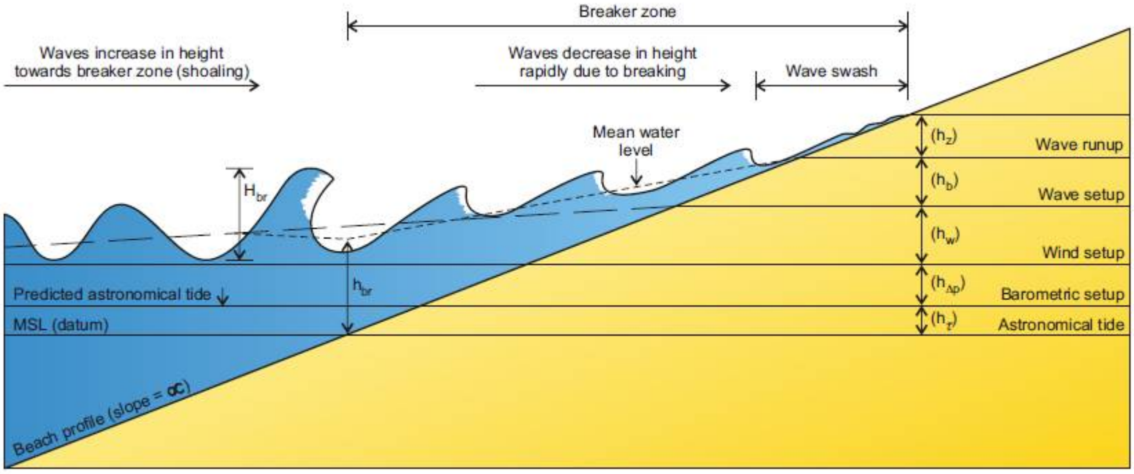


Figure 4-1: Components of Elevated Ocean Water Levels (Adapted from DECCW, 2010)

4.3.2 Storm Tide (Astronomical Tide + Anomaly)

Astronomical tidal planes for Batemans Bay, based on the Princess Jetty tide gauge record, are shown in Table 4-4 from MHL (2012). This tide gauge is located adjacent to the Batemans Bay Central Business District (CBD) in the Clyde River channel in a water depth of 10 m.

Table 4-4: Average Annual Tidal Planes (1990-2010) for Princess Jetty, Batemans Bay CBD (Source: MHL, 2012)

Tide	Level (m AHD)
High High Water Solstices Springs (HHWSS)	0.920
Mean High Water Springs (MHWS)	0.607
Mean High Water (MHW)	0.508
Mean High Water Neaps (MHWN)	0.408
Mean Sea Level (MSL)	0.048
Mean Low Water Neaps (MLWN)	-0.312
Mean Low Water (MLW)	-0.412
Mean Low Water Springs (MLWS)	-0.511
Indian Spring Low Water (ISLW)	-0.735

Tidal anomalies primarily result from factors such as wind setup (or setdown) and barometric effects, which are often combined as "storm tide". Additional anomalies occur due to "trapped" long waves propagating along the coast, the influence of the East Australia Current (EAC) and tsunamis. While a summary of recorded anomalies has not been published for Princess Jetty tide gauge, the gauge recently recorded an anomaly near low tide of 0.56 m on 6 June 2016 at 04:15 AM (Blacka and Coghlan, 2016). However, the tidal anomaly coinciding with the peak water levels during the same event was only approximately 0.2 m. The top 10 recorded anomalies at a Zwarts pole in the vicinity of Snapper Island are also reproduced in Table 4-5 (MHL, 1992). This gauge was deployed for a short period of time (1 July 1987 to 8 December 1990) in a water depth of 7 m.

Table 4-5: Ranking of Highest Recorded Anomalies (1987-1990) for Snapper Island Batemans Bay (Source: MHL, 1992)

Rank (on Anomaly)	**Peak Anomaly** (m)	**Date**	**Anomaly ARI** (1 in x years)
1	0.38	27/04/1990	5.0
2	0.30	01/12/1987	2.5
3	0.30	11/06/1989	1.7
4	0.29	10/12/1988	1.3
5	0.29	14/05/1990	1.0
6	0.28	04/07/1990	0.8
7	0.28	15/08/1990	0.7
8	0.27	17/11/1988	0.6
9	0.27	28/12/1989	0.6
10	0.25	13/03/1988	0.5

Design storm tide levels (astronomical tide + anomaly) are recommended in the Coastal Risk Management Guide (DECCW, 2010 after Watson and Lord, 2008) based on data from the Fort Denison tide gauge in Sydney and reproduced in Table 4-6 for a range of average recurrence intervals (ARI) – these values exclude wave setup and runup effects which can be significant where waves break on shorelines. However, these levels are predominantly applicable in the Newcastle - Sydney – Wollongong area and analysis of local tidal records on the NSW south coast is recommended.

Table 4-6: Tidal Water Levels + Anomaly (Newcastle – Sydney – Wollongong) (Source Watson and Lord, 2008 and DECCW, 2010)

ARI (years)	**2008 Water Level Excl. Local Wave Setup and Runup** (m AHD)
0.02	0.97
0.05	1.05
0.10	1.10
1	1.24
2	1.28
5	1.32
10	1.35
20	1.38
50	1.41
100	1.44
200	1.46

Storm tide levels for ARIs of 5 to 100 years (tabulated in Table 4-8) have previously been estimated for Batemans Bay based on further analysis of the Princess Jetty tide gauge (BMT WBM, 2009). Note that no attempt was made to remove non-tidal freshwater flooding events, local wind setup and "inner bay" wave setup from the raw data in the BMT WBM study. Since each of these coastal processes can contribute to increased water level elevations, the values calculated by BMT WBM (2009) may be slightly conservative.

Table 4-7: Tidal Water Levels + Anomaly (1985-2009) for Princess Jetty, Batemans Bay CBD (Source: BMT WBM, 2009)

Average Recurrence Interval ARI (year)	**2009 Water Level Excl. Local Wave Setup and Runup** (m AHD)
5	1.26
10	1.31
20	1.34
50	1.38
100	1.40

Since a 1 year ARI storm tide water level for Batemans Bay was not established in the BMT WBM (2009) study, WRL considered joint probability analysis undertaken for adjacent tide gauges by MHL (2010). This analysis was undertaken using the method described by Pugh and Vassie (1979). This calculates the chance that high astronomical tide levels and high anomaly levels occur together. The 1 year ARI elevated water level at five (5) adjacent nearshore tide gauges are reproduced in Table 4-8. Based on consideration of this information, the 1 year ARI water level at Fort Denison (storm tide levels in Batemans Bay are slightly lower than at Fort Denison for an equivalent ARI) and the trend in the BMT WBM (2009) data, WRL adopted a water level of 1.22 m AHD as the 1 year ARI storm tide level for Batemans Bay.

Table 4-8: 1 year ARI Water Levels (Astronomical Tide + Anomaly) (Source: MHL, 2010)

Tide Gauge Location	**2007 1 year ARI Water Level Excl. Local Wave Setup and Runup** (m AHD)
Crookhaven Heads	1.23
Jervis Bay	1.28
Ulladulla	1.17
Bermagui	1.16
Eden	1.21

From the consideration of this BMT WBM (2009) study and allowing for sea level rise between 2009 and 2017 (4.2 mm/year from 1996-2013 at Princess Jetty, Whitehead & Associates, 2014; see Section 0), water levels adopted by WRL for 2017 are also summarised in Table 4-9.

Table 4-9: Adopted Storm Tide (Astronomical Tide + Anomaly) Water Levels for Eurobodalla

Average Recurrence Interval ARI (year)	**2017 Adopted Water Level** (m AHD)
1	1.22*
20	1.37
100	1.43

*not calculated using BMT WBM (2009)

4.3.3 Batemans Bay Water Levels (Local Wind Setup and Coincident Flooding)

For open coast beaches, the still water level at the beach before the inclusion of wave setup is approximately equal to that offshore of the coast, and the levels provided in Table 4-9 provide an appropriate estimate of water levels. However, at the inner Batemans Bay sites, the shallow bathymetry and presence of the Clyde River provides conditions that allow even higher water level conditions, due to increase in water levels from wind setup and inland flood events.

Local Wind Setup

Since the bathymetry inside Batemans Bay is relatively flat and shallow and the bay itself has an open funnel shape, the super-elevation of water levels within the bay due to local wind setup requires consideration. The centre-line orientation of Batemans Bay is directed towards the south-east reducing from 5 km width near the Tollgate Islands to approximately 500 m at the Princes Highway bridge.

WRL adopted local wind setup levels from modelling undertaken for a previous inundation study of Batemans Bay (NSW PWD, 1989) using a two-dimensional SYSTEM 21 (Abbott et al, 1973) depth averaged hydrodynamic model. Peak water levels due to wind setup were determined at 17 locations around Batemans Bay (Figure 4-2). Three different water levels (-1.0, 0.0 and 1.0 m AHD) were used for the modelling runs in the initial study as wind setup is inversely related to water depth. However, for the purpose of this study, the 1 m AHD water level results have been adopted as this is closest to the relevant extreme water level conditions. Four different wind directions were modelled (NE, E, SE and S) with two different wind speeds (35 and 70 knots - 18 and 36 m/s over a 3 hour duration), the results of which are shown in Table 4-10. Wind setup for the 5% and 1% AEP storm events were linearly interpolated between the two different wind speeds modelled using the wind speed squared in Table 4-12. This interpolation technique was utilised in the previous oceanic inundation study (NSW PWD, 1989).

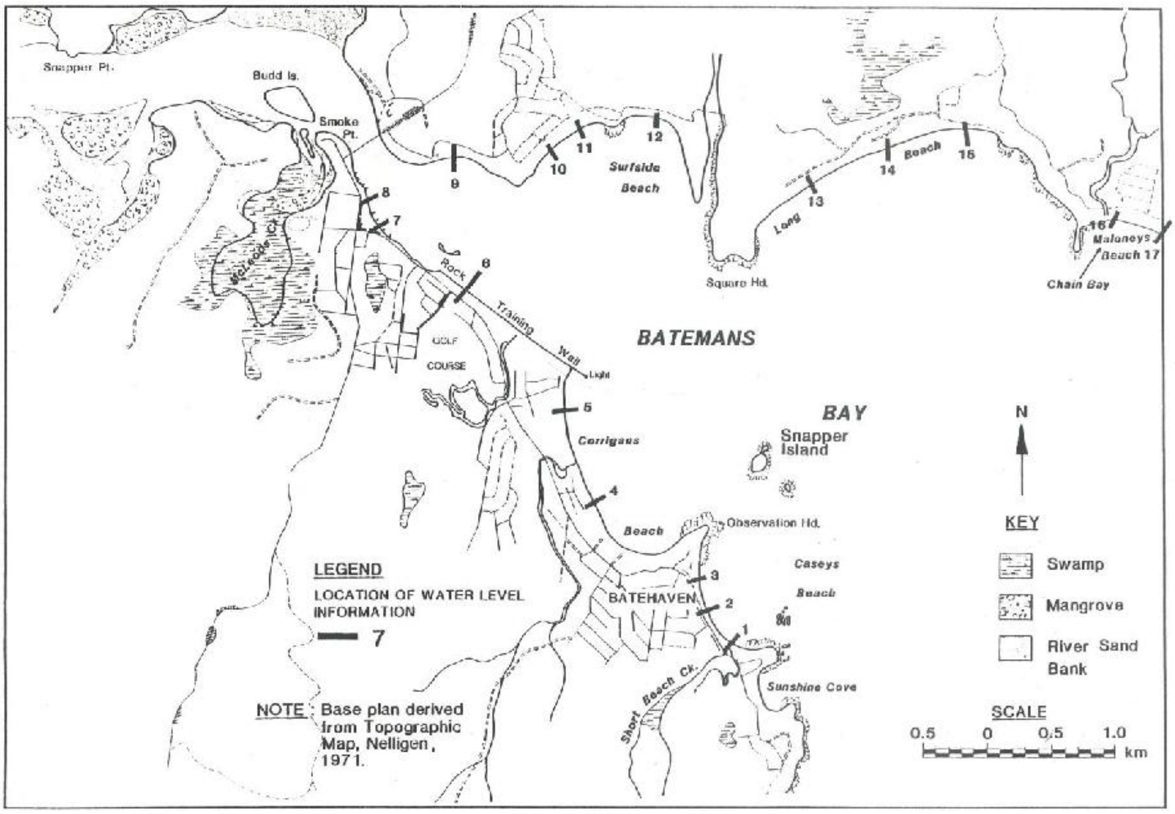


Figure 4-2: Water level output locations from NSW PWD (1989)

Table 4-10: Local Wind Setup in Batemans Bay as Output from SYSTEM 21 (NSW PWD, 1989)

Direction			**NE**		**E**		**SE**		**S**	
Wind Strength (m/s)			**18**	**36**	**18**	**36**	**18**	**36**	**18**	**36**
Location		**#**	**Wind Setup (m)**							
Maloneys Beach	Eastern End	17	-0.03	-0.07	0.05	0.29	0.06	0.27	0.07	0.19
	Western End	16	-0.02	-0.05	0.05	0.32	0.07	0.28	0.07	0.20
Long Beach	Eastern End	15	-0.03	-0.08	0.05	0.34	0.10	0.43	0.11	0.33
	Central	14	-0.02	-0.04	0.05	0.49	0.12	0.47	0.11	0.31
	Western End	13	-0.01	0.00	0.05	0.49	0.12	0.49	0.11	0.30
Cullendulla Beach	Central	12	-0.01	0.03	0.05	0.42	0.12	0.49	0.11	0.38
Surfside Beach (East)	Northern End	11	0.03	0.18	0.05	0.33	0.13	0.56	0.09	0.39
	Southern End	10	0.03	0.18	0.05	0.34	0.14	0.55	0.08	0.39
Wharf Road	Central	9	-0.04	0.03	0.04	0.35	0.08	0.09	0.06	0.40
Central Business District	Central +Western	8	0.02	0.14	0.05	0.22	0.08	0.35	0.05	0.32
	Eastern End	7	0.05	0.21	0.05	0.19	0.08	0.33	0.04	0.32
Boat Harbour	Central	6	0.01	0.10	0.06	0.21	0.05	0.23	0.04	0.28
Corrigans Beach	Northern End	5	0.02	0.12	0.05	0.22	0.05	0.23	0.03	0.23
	Southern End	4	0.02	0.12	0.05	0.20	0.04	0.19	0.02	0.17
Caseys Beach	Northern End	3	0.01	0.10	0.05	0.21	0.05	0.24	0.03	0.16
	Central	2	0.02	0.12	0.05	0.22	0.05	0.23	0.03	0.16
	Southern End	1	0.02	0.12	0.05	0.20	0.04	0.20	0.02	0.14

The wind conditions which develop wind setup were estimated using the design wind velocities for Australia excluding tornadoes set out in AS 1170.2 (2011). Design wind velocities (0.2 second gust, 10 m elevation, Terrain Category 2) applicable to coastal engineering assessments are given for average recurrence intervals of 1 to 1,000 years. Site wind speeds (V_{sit}), are calculated according to Equation 3.1 using multipliers for direction (M_d), terrain ($M_{z,cat}$), shielding (M_s) and topography (M_t).

$$V_{sit} = V_r M_d (M_{z,cat} M_s M_t) \quad \text{Equation 3.1}$$

The Eurobodalla coastline falls within Region A2 (AS 1170.2, 2011) and corresponding wind speed multipliers were adopted (see Table 4-11). For Terrain Category 1.5 (open water surfaces subjected to shoaling waves at serviceability and ultimate wind speeds), $M_{z,cat}$ at 10 m elevation (z) was adopted as 1.06 (AS1170.2:2011, S4.2.1). The adopted shielding or topography multipliers were both 1.0.

Table 4-11: Adopted Extreme Wind Speed Multipliers for Eurobodalla (Source: AS 1170.2, 2011)

Wind Direction		Multipliers			
		Direction (M_d)	Terrain ($M_{z,cat}$)	Shielding (M_s)	Topography (M_t)
NE	45.0	0.80	1.06	1.00	1.00
ENE	67.5	0.80	1.06	1.00	1.00
E	90.0	0.80	1.06	1.00	1.00
ESE	112.5	0.95	1.06	1.00	1.00
SE	135.0	0.95	1.06	1.00	1.00
SSE	157.5	0.95	1.06	1.00	1.00
S	180.0	0.90	1.06	1.00	1.00

Wind setup generated by winds blowing across Batemans Bay is the result of sustained winds rather than extreme gusts. Equivalent sustained 60 minute (1 hour) wind speeds were therefore calculated using the approach set out in Figure II-2-1 of Part II of the USACE Coastal Engineering Manual (2006). A 1 hour duration was selected to correspond with the 1 hour duration swell wave conditions for SWAN wave modelling (Section 4.4.1 and Appendix D). Similarly, equivalent 180 minute (3 hour) wind speeds were calculated to interpolate results from the NSW PWD (1989) wind setup values. Sustained (1 hour) wind speeds for annual recurrence intervals of 1, 20 and 100 years and 3 hour wind speeds for 20 and 100 year ARIs for all directions are presented within Table 4-12. The adopted wind setup values (the maximum wind setup from the four directions) are provided in Table 4-13.

Table 4-12: Adopted Extreme Wind Conditions for Eurobodalla (Source: AS 1170.2, 2011)

Wind Direction		1 Hour Average Wind Speed (m/s)			3 Hour Average Wind Speed (m/s)	
		1 year ARI	20 year ARI	100 year ARI	20 year ARI	100 year ARI
NE	45.0	16.3	20.1	22.2	18.6	20.6
ENE	67.5	16.3	20.1	22.2	18.6	20.6
E	90.0	16.3	20.1	22.2	18.6	20.6
ESE	112.5	19.3	23.8	26.4	22.1	24.5
SE	135.0	19.3	23.8	26.4	22.1	24.5
SSE	157.5	19.3	23.8	26.4	22.1	24.5
S	180.0	18.3	22.6	25.0	20.9	23.2

Table 4-13: Adopted Local Wind Setup throughout Batemans Bay

Location		#	Adopted Wind Setup (m)	
			20 year ARI	100 year ARI
Maloneys Beach	Eastern End	17	0.10	0.12
	Western End	16	0.11	0.13
Long Beach	Eastern End	15	0.16	0.19
	Central	14	0.18	0.22
	Western End	13	0.18	0.23
Cullendulla Beach	Central	12	0.18	0.23
Surfside Beach (East)	Northern End	11	0.20	0.25
	Southern End	10	0.21	0.26
Wharf Road	Central	9	0.10	0.13
Central Business District	Central and Western	8	0.13	0.16
	Eastern End	7	0.12	0.15
Boat Harbour	Central	6	0.08	0.10
Corrigans Beach	Northern End	5	0.08	0.10
	Southern End	4	0.07	0.08
Caseys Beach	Northern End	3	0.08	0.10
	Central	2	0.08	0.10
	Southern End	1	0.07	0.09

Coincident Freshwater Flooding

Fresh water floods are not expected to cause significant increase in ocean inundation levels in most of the study area. However, in inner Batemans Bay, flooding from the Clyde River may increase peak coastal inundation levels by up to 0.16 m. As agreed with OEH, WRL adopted the increase in inundation levels due to flooding from the Clyde River from the same study (NSW PWD, 1989) which used a one-dimensional SYSTEM 11 (Abbott, 1979) hydrodynamic model. This study found that flood and ocean storm events were neither dependent nor independent and adopted a flood discharge of twice the frequency of the ocean storm event (i.e. 50 year ARI river discharge with 100 year ARI storm). The flood contribution levels adopted for this study are provided in Table 4-14.

Table 4-14: Adopted Flood Contribution to Levels inside Batemans Bay

Location		#	Adopted Flood Contribution (m)	
			20 year ARI	100 year ARI
Cullendulla Beach	Central	12	0.01	0.02
Surfside Beach (East)	Northern End	11	0.02	0.03
	Southern End	10	0.02	0.02
Wharf Road	Central	9	0.04	0.07
Central Business District	Central and Western	8	0.06	0.16
	Eastern End	7	0.03	0.06
Boat Harbour West	Central	6	0.03	0.05
Corrigans Beach	Northern End	5	0.01	0.01

4.3.4 Sea Level Rise

Historical Measurements

This report used two different measurements of recent, historical sea level rise (SLR) rate in its analysis:

- To adjust the rates of underlying shoreline movement to account for existing Bruun recession due to sea level rise, a rate of 0.8 mm/year (White et al., 2014) was used. This was the mean sea level rise rate measured at Fort Denison from 1966 to 2010 which broadly coincides with the years of available photogrammetry data (1942 to 2014) from which the underlying shoreline movement trends were derived.
- To adjust the Batemans Bay storm tide water level statistics calculated based on the 2009 mean sea level to the 2017 mean sea level, a rate of 4.2 mm/year (Whitehead & Associates, 2014) was used. This was the mean sea level rise rate measured at Princess Jetty from 1996 to 2013. Note that measurements at this location are only available from 1985 onwards. This SLR rate, calculated over 18 years, reflects a wide range of local and regional influences on sea surface height superimposed on the underlying rate of SLR attributable to external forcings (i.e. climate change induced melting of snow and ice reserves and thermal expansion of the ocean water mass).

Future Projections

The SLR projections for various planning periods adopted in this study were equivalent to the values adopted by ESC on 25 November 2014 (ESC, 2014) and are shown in bold in Table 4-15. These benchmarks were established considering the most recent international (Intergovernmental Panel on Climate Change, IPCC, 2013 and 2014) projections. This policy includes locally adjusted projections for sea level rise (Whitehead & Associates, 2014) derived from Representative Concentration Pathway (RCP) 6.0 scenarios (upper bound of likely range; level exceeded by 5% of models) from the IPCC Assessment Report 5 (AR5).

The sea level rise trajectory described by Table 4-15 was used for deterministic erosion/recession mapping and inundation mapping. For probabilistic erosion/recession mapping, these sea level rise values were adopted as the modal sea level rise trajectory. However, the minimum and maximum sea level rise trajectories were established to cover the full range of IPCC projections, namely, to locally adjusted projections of RCP 2.6 (lower bound) and RCP 8.5 (upper bound), respectively, as documented by Whitehead & Associates (2014). These three (3) sea level rise trajectories are tabulated in Table 4-16 relative to the 2017 mean sea level.

Table 4-15 Sea Level Rise Projections (Adapted from ESC, 2014)

Planning Period (year)	**Sea Level Rise** (m)		
	Increase above 2015 Mean Sea Level	Increase above 2017 Mean Sea Level	Absolute Elevation of MSL (m Present AHD)[1]
2009	-0.03[2]	-0.04	0.05
2015	0.00	-0.01	0.08
2017	**0.01**[3]	**0.00**	**0.09**
2020	0.03	0.02	0.11
2030	0.10	0.09	0.18
2040	0.15	0.14	0.23
2050	**0.23**	**0.22**	**0.31**
2060	0.30	0.29	0.38
2065	**0.34**[3]	**0.33**	**0.42**
2070	0.39	0.38	0.47
2080	0.50	0.49	0.58
2090	0.61	0.60	0.69
2100	**0.72**	**0.71**	**0.80**

(1) Absolute elevation (m AHD) was determined by adding 0.08 m to values relative to 2015 MSL as per Whitehead & Associates (2014).

(2) Value extrapolated by WRL based on 4.2 mm/year SLR at Princess Jetty, Batemans Bay between 1996 and 2013 (Whitehead & Associates, 2014) to establish the 2009 MSL.

(3) Values interpolated by WRL using quadratic equations between adjacent planning periods.

Table 4-16: Sea Level Rise Projections for Probabilistic Erosion/Recession

Planning Period (year)	**Increase above 2017 Mean Sea Level (m)**		
	Minimum Trajectory	Modal Trajectory	Maximum Trajectory
	RCP 2.6 (lower bound)	RCP 6.0 (upper bound)	RCP 8.5 (upper bound)
2017	**0.00**	**0.00**	**0.00**
2020	0.01	0.02	0.02
2030	0.04	0.09	0.09
2040	0.09	0.14	0.16
2050	**0.12**	**0.22**	**0.25**
2060	0.14	0.29	0.36
2065	**0.15**	**0.33**	**0.42**
2070	0.17	0.38	0.49
2080	0.20	0.49	0.63
2090	0.22	0.60	0.78
2100	**0.24**	**0.71**	**0.97**

4.4 Ocean Swell and Local Wind Waves

4.4.1 Wave Height

The Eurobodalla LGA coastline is subject to waves originating from offshore storms (swell) and produced locally (wind waves) within the nearshore coastal zone. Swell waves reaching the coast may be modified by the processes of refraction, diffraction, wave-wave interaction and dissipation by bed friction and wave breaking. Locally generated waves undergo generation processes as well as the aforementioned propagation and dissipation processes.

A non-directional wave buoy operated offshore of Batemans Bay from 1986 to 2001 and was upgraded to measure wave direction in 2001. WRL, in conjunction with OEH (formerly DECCW) have completed an assessment of coastal storms and extreme waves for NSW which involves the identification of all measured coastal storms during the period 1971 – 2009 and derivation of the direction design storm events for annual recurrence intervals if 1 to 100 years (Shand et al. 2010). The results from the study for the wave buoy at Batemans Bay and two adjacent wave buoys at Port Kembla and Eden are tabulated for all wave directions in Table 4-17.

Table 4-17: Extreme Offshore Wave Climate (All Directions)
(Source: Shand et al. 2010)

Average Recurrence Interval (year)	One Hour Exceedance H_s (m)		
	Port Kembla	Batemans Bay	Eden
1	5.4	4.9	5.4
20	7.6*	6.8*	7.5*
100	8.8	7.7	8.5

* Note that the estimated 20 ARI values have been inferred by WRL for this study

Extreme wave heights extrapolated from the wave record of Batemans Bay are shown to be smaller than those from the wave record at Port Kembla and Eden. WRL, also in conjunction with OEH (formerly DECCW) and MHL, undertook a comprehensive study of the wave climate in the vicinity of Batemans Bay and confirmed that the wave buoy at this location is correctly measuring a less energetic wave climate than along the rest of the NSW coast (Coghlan et al, 2011). The reduced wave climate is attributed to land mass sheltering effects and wind field variations.

Directional extreme wave analysis for the one hour exceedance significant wave height are summarised for the 1, 20 and 100 year ARI, ranging from north-east to south swell directions in 22.5° increments in Table 4-18 (Shand et al, 2010). Note that the adopted 100 year ARI offshore significant wave height at the Batemans Bay wave buoy varies with incident wave direction. Extreme wave heights are predicted to be highest from the east-south-east to the south-south-east (112.5 to 157.5°).

Table 4-18: Batemans Bay One Hour Exceedance Wave Climate Conditions (Source: Shand et al. 2010)

Offshore Wave Direction		H_S (m)		
		1 year ARI	20 year ARI	100 year ARI
NE	45.0	3.0	5.0	6.2
ENE	67.5	3.0	5.0	6.2
E	90.0	3.7	6.1	7.3
ESE	112.5	4.9	6.8	7.7
SE	135.0	4.9	6.8	7.7
SSE	157.5	4.9	6.8	7.7
S	180.0	3.7	6.1	7.3

4.4.2 Wave Period

WRL, in conjunction with the Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI), reviewed Australian storm climatology and previous extreme wave analyses undertaken using instrument and numerical model data (Shand et al, 2011). Importantly, the study defined the peak spectral wave period during storm events around the Australian coast. The nearest location to the study area where this analysis was undertaken was Eden, with results presented in Table 4-19. The peak spectral wave periods presented in this table were adopted for the study.

Table 4-19: Associated Wave Period for Extreme Wave Events (Source: Shand et al., 2011)

Average Recurrence Interval ARI (year)	**Peak T_P (s)** **Eden**
1	11.6
20	12.8
100	13.4

4.4.3 Nearshore Wave Modelling

The Simulating WAves Nearshore (SWAN) numerical wave model (Booij et al, 1999) was used to quantify the change in wave conditions from the Batemans Bay wave buoy to the beaches included in the Coastal Hazard Assessment and to model the generation of local-waves. SWAN (version 41.10) is a third-generation wave model that was developed at Delft University of Technology (2016). Detailed information on the wave modelling is presented in Appendix D.

4.5 Wave Setup

Wave setup is defined as the local quasi-steady increase in water level inside a surf zone due to transfer of wave momentum. The numerical surf zone model of Dally, Dean and Dalrymple (1984) was implemented using SWAN wave modelling output to calculate local wave setup at 35 representative locations along the coastline of the study area. Detailed information on the wave setup determination is presented later in the report in Section 8.3.

4.6 Wave Runup and Overtopping

The 17 beaches for which inundation modelling and mapping was undertaken are backed by either sand dunes or seawalls. During storm events, waves frequently impact these features backing the beach and overtopping of the crests occurs in the form of bores of water being discharged inland or splashes of water being projected upwards and eventually transported inland by onshore winds. Wave overtopping can cause damage to the seawall crest and to beachfront structures.

Overtopping also constitutes a direct hazard to pedestrians and vehicles in the proximity of the dune or seawall during storm events.

Wave runup is defined as the extreme level the water reached on a structure slope by wave action. Unlike wave setup, wave runup is a highly fluctuating and dynamic phenomenon and it is commonly described using the runup parameter *R2%* which is the runup level exceeded by 2% of the waves.

Wave runup depends on the:

- Hydraulic parameters such as water level, wave height and period; and
- Structural parameters such as the seawall construction (sandstone masonry, precast concrete blocks, rock revetments etc.), slope of the seawall or the dune and crest levels.

Wave runup and bore propagation extents were calculated at each of the 35 representative locations along the Eurobodalla coastline based on:

- The extreme water levels incorporating storm surge and wave setup;
- The nearshore wave parameters (significant wave height and peak wave period) as derived from SWAN numerical wave modelling; and
- The dune or seawall geometry (crest level, slope etc.).

Detailed information on the wave setup determination is presented later in the report in Section 8.5.

4.7 Beach erosion and Long-term Shoreline Recession

4.7.1 Preamble

For the purposes of this study, the coastal hazard components can be described as follows:

- **Short Term Storm Erosion** – refers to the short-term response of a beach to changing wave and water level conditions during ocean storms. This response is generally manifested in a "storm bite" from the sub-aerial beach moving offshore during the storm; and

- **Shoreline Recession** – refers to the long-term trend of a shoreline to move landwards in response to a net loss in the sediment budget over time (hereafter referred to as negative Underlying Shoreline Movement). Shoreline recession is also predicted to result from sea level rise (Sea Level Recession).

It is important to differentiate the processes of erosion and recession as they occur on very different time-scales.

4.7.2 *Short Term Storm Erosion*

Beach erosion is defined as the erosion of the beach above mean sea level by a single extreme storm event or from several storm events in close succession. The amount of sand (above 0 m AHD) transported offshore by wave action is referred to as "storm demand" and expressed as a volume of sand per metre length of beach *(m^3/m)*. This can be converted to a horizontal "storm bite" which is easier to visualise. Figure 4-3 shows a photograph of Long Beach (east) in an moderately eroded state in June 2012.

Figure 4-3: Example Storm Erosion, Long Beach, 6 June 2012 (Mr Lindsay Usher)

Around the Eurobodalla coastline, storm demand varies depending on several factors such as:

- Exposure of the beach;
- Protection by offshore reefs and rock shelves;
- Nature of the coastline;
- Possibility of a mega-rip(s) forming during extreme wave conditions;
- Wave conditions (i.e. wave height, period and direction relative to the beach alignment);
- Water levels;
- Steepness of the profile offshore from the beach;
- Sand grain size;
- Beach type (i.e. reflective, low tide terrace, transverse bar and rip, etc.); and
- and the condition of the beach prior to the storm (i.e. accreted or already eroded).

Consensus design storm demands for the beaches of the Eurobodalla study area were developed by an expert panel (Section 5) through review of photogrammetry analysis (Appendix C), SBEACH numerical erosion modelling (Appendix E) and previously published estimates.

4.7.3 Shoreline Recession

Underlying Shoreline Movement

Ongoing underlying recession is the progressive onshore shift of the long term average land-sea boundary which may result from sediment loss. It is expressed in terms of loss over years in volume of sand within the beach *(m^3/m/year)* and/or corresponding negative landward shoreline movement *(m/year)*.

Underlying Shoreline Movement rates due to sediment loss or gain along the Eurobodalla beaches were derived through the analysis of long term changes in sand volumes (photogrammetric analysis). Consensus Underlying Shoreline Movement rates were also developed by an expert panel (Section 5) through review of photogrammetry analysis (Appendix C).

Recession due to Sea Level Rise

It is expected that the 10 beaches in the study area will recede in response to future sea level rise. Recession rates due to sea level rise were estimated using the Bruun Rule (Bruun, 1962, 1988) as the rate of sea level rise divided by the average slope ("Bruun Factor") of the active beach profile. This rule is based on the concept that the existing beach profile is in equilibrium with the incident wave climate and existing average water level. It also assumes that the beach system is two-dimensional and that there is no interference with the equilibrium profile by headlands and offshore reefs. Consensus Bruun factors were also developed by an expert panel (Section 5) through review of depth of closure analysis using up to five (5) methods (Appendix F) and previously published estimates.

5. Characteristic Erosion and Recession Values

To establish the characteristic erosion and recession values which would be used in subsequent modelling and mapping, WRL independently polled three (3) senior coastal engineers and scientists experienced on the Eurobodalla coast (Table 5-1). This structured communication technique, called the Delphi method, relies on the decisions of a panel of experts to achieve a consensus of the most probable future by iteration.

Table 5-1: Expert Panel Polled for Characteristic Erosion and Recession Values

Name	Affiliation	Role
Professor Andrew Short	University of Sydney, School of Geosciences	Honorary Coastal Geomorphologist
Mr James Carley	UNSW Water Research Laboratory	Principal Coastal Engineer
Mr Daniel Wiecek	NSW Gov., Office of Environment & Heritage	Senior Natural Resource Officer (Coast & Estuaries)

Each coastal expert was presented with the following information:

- Sediment characteristics (Section 2.1 and Appendix B);
- 100 year ARI SWAN numerical wave modelling results (Appendix D);
- 100 year ARI storm demand based on WRL photogrammetry analysis (Appendix C), WRL SBEACH numerical erosion modelling (Appendix E) and previously published estimates (Table 5-2);
- Bruun factor based on WRL depth of closure analysis using up to five (5) methods (Appendix F) and previously published estimates (Table 5-3); and
- Underlying shoreline movement trend based on WRL photogrammetry analysis (Appendix C).

They were then asked for their preferred values for 100 year ARI storm demand (best estimate only), Bruun factor (minimum, maximum and mode) and underlying shoreline movement trend (minimum, maximum and mode) at each beach section on the basis of the presented information and their own experience on the Eurobodalla coast.

Polling was not undertaken for minimum and maximum values at beaches where only the deterministic methodology was applied. While Bruun factors were assessed at more than one profile on longer beaches, only one Bruun factor value was adopted at each beach.

The experts' independently preferred values were then blended into a consensus range for input into the modelling (Table 5-4). Note that not all practitioners agreed with the full range of values but good agreement was achieved for mode values.

Finally, the consensus values for underlying trend were adjusted to account for existing Bruun recession under measured sea level rise (effectively making them slightly more accretionary) to avoid "double-dipping" with Bruun recession in the subsequent modelling (Table 5-5). This was done using the modal Bruun factor at each beach and a sea level rise rate of 0.8 mm/year

(White et al., 2014). This was the relative mean sea level rise at Fort Denison from 1966 to 2010 which broadly coincides with the years of available photogrammetry data from which the underlying shoreline movement trends were derived. Adjusting underlying trend rates to account for the contribution from existing Bruun recession to avoid "double counting" the effects of sea-level rise was recommended by Professor Paul Komar as part of an expert panel's peer review of a coastal hazard assessment for Kāpiti Coast District Council, New Zealand (Carley et al., 2014). A similar methodology has subsequently been applied on a range of coastal hazard assessments for other New Zealand councils (Tonkin & Taylor, 2015a; 2015b, 2016a and 2017).

Table 5-2: Summary of Storm Demand Estimates

Beach	Section	100 year ARI storm demand volume (m^3/m above 0 m AHD)			
		Photogrammetry†	SBEACH Modelling‡	Previous Estimates	Adopted Consensus Values
Maloneys Beach	East	31	73-96	12[1], 9[2], 45 (45-90)[3]	**50**
	West	26	113-156		**80**
Long Beach	East	19	68-87	15[1], 10[2], 70 (60-110)[3]	**70**
	Central	47	84-126	35[2]	**100**
	West	71	105-137	44[1], 20[2], 120 (80-130)[3]	**120**
Surfside Beach (East)	North	44	43-54	39[1], 25-40[2], 60 (60-110)[3]	**50**
	South	62	46-55		**60**
Surfside Beach (West)	Central	#	20*	20[5]	**20**
Sunshine Bay	Central	12	20*	(20-70)[3]	**25**
Malua Bay	Central	63	115-153	(20-70)[3]	**120**
Guerilla Bay (South)	Central	39	103-153	(60-110)[3]	**80**
Barlings Beach	East	53	50-64	79[4]	**60**
	West	113	60-106	170 (150-200)[3], 147[4]	**110**
Tomakin Cove	Central	90	84-132	(40-90)[3]	**90**
Broulee Beach	North	95	47-89	(150-200)[3]	**110**
	Central	45	34-56		**90**
	South	71-100 (spit influenced)	39-52		**70**

† For beaches where photogrammetry was available in 1972 and 1975 (Surfside Beach (east), Barlings Beach and Tomakin Cove) the maximum storm demand estimated from photogrammetry is considered a reasonable representation of the erosion that occurred due to the May-June 1974 storm sequence. The maximum storm demands estimated at the other beaches are considered to be an underestimate. Maximum storm demands are presented based on individual profiles rather than photogrammetry block averages to capture the influence of any rip cells (see Appendix C).
‡ The two SBEACH modelling storm demand estimates correspond to two calibration conditions: 4 profile average and single profile maximum erosion at Bengello Beach in 1974 (see Appendix E).
# A storm demand value for Surfside Beach (west) was not calculated from the photogrammetry as the volume changes between years at this location are considered to be associated with tide and flood driven shoreline re-alignment processes rather than erosion from wave attack.
* SBEACH modelling was not undertaken at Surfside Beach (west) and Sunshine Bay (see Appendix E). These storm demand values are based on WRL's expert coastal engineering judgment.

[1]DLWC (1996), [2]WMA (2006), [3]SMEC (2010), [4]GBAC (2010), [5]PBP (1994)

Table 5-3: Summary of Bruun Factor estimates

Beach	Section	Bruun Factors (-)					
		Inner Depth of Closure	Outer Depth of Closure	Divergence from Equilibrium	Break-point Depth	Rock/ Reef Depth	Previous Estimates
Maloneys Beach	East	10		59	10		50[1], 20-22[2]
	West	9		60	9		
Long Beach	East	25		60	22		40[1], 20-22[2]
	Central	16		56	17		-
	West	18		52	19		40[1], 23-25[2]
Surfside Beach (East)	North	31		25	23		25[1], 19-20[2]
	South	36		29	23		
Surfside Beach (West)	Central	#		#	#		20[4]
Sunshine Bay	Central	37	71		38	24	45-62[2]
Malua Bay	Central	28	44		31	33	40-49[2]
Guerilla Bay (South)	Central	20	34		22	21	25-35[2]
Barlings Beach	East	17	52		16		70-85[2]
	West	26	79*		22		85-95[2], 56[3]
Tomakin Cove	Central	24	74*		24	21	85-95[2], 40[3]
Broulee Beach	North	31	63*		28		65-75[2]
	Central	30	62		29		
	South	32	53		19		

Bruun factors for Surfside Beach (west) were not calculated using the five analysis methods since it is a tide-dominated beach with sand flats. The only estimate at this location is by BMT WBM (2009) which is based on the upper beach slope.
* Where the distance from the dune to the Hallermeier outer depth of closure was more than 1.5 km, depth of closure was assumed to be at 1.5 km offshore.

[1]DLWC (1996), [2]SMEC (2010), [3]GBAC (2010), [4]BMT WBM (2009)

Table 5-4: Adopted Consensus Input Values for Erosion/Recession Modelling and Mapping

Beach	Section	100 year ARI Storm demand volume (m^3/m)	Bruun factor (-)#			Underlying shoreline movement (m/year) #		
			min	mode	max	min	mode	max
Maloneys Beach	East	50		10			-0.05	
	West	80					0.04	
Long Beach	East	70	15	20	50	0.05	0.10	0.20
	Central	100	15	20	50	-0.10	0.00	0.10
	West	120	15	20	50	0.05	0.15	0.20
Surfside Beach (East)	North	50	20	25	30	-0.15	-0.08	-0.05
	South	60	20	25	30	0.05	0.10	0.15
Surfside Beach (West)	Central	20	15	20	30	-0.02*	-0.02*	-0.02*
Sunshine Bay	Central	25		40			0.05	
Malua Bay	Central	120	25	30	50	-0.20	-0.10	0.10
Guerilla Bay (South)	Central	80		25			0.15	
Barlings Beach	East	60		50			-0.05	
	West	110					0.05	
Tomakin Cove		90	20	25	60	-0.10	-0.07	-0.03
Broulee	North	110	25	30	65	-0.05	-0.01	0.05
	Central	90	25	30	65	0.20	0.30	0.40
	South	70	25	30	65	0.10	0.55	0.70

Note: Positive value = accretion trend

Negative value = recession trend

Minimum and maximum values have only been presented at beaches where the probabilistic methodology was applied.
* The minimum, mode and maximum underlying shoreline movement values for Surfside Beach (west) have been set to -0.02 m/year so that their values are 0.00 m/year when adjusted for existing Bruun recession (Table 5-5). This assumption has been made on the basis that there was no discernible trend for underlying shoreline movement at Surfside Beach (west).

Table 5-5: Summary of Adopted Consensus Values for Underlying Shoreline Movement

Beach	Section	Underlying shoreline movement (m/year)					
		Raw			Adjusted for Measured SLR *		
		min	mode	max	min	mode	max
Maloneys Beach	East		-0.05			-0.04	
	West		0.04			0.05	
Long Beach	East	0.05	0.10	0.20	0.07	0.12	0.22
	Central	-0.10	0.00	0.10	-0.08	0.02	0.12
	West	0.05	0.15	0.20	0.07	0.17	0.22
Surfside Beach (East)	North	-0.15	-0.08	-0.05	-0.13	-0.06	-0.03
	South	0.05	0.10	0.15	0.07	0.12	0.17
Surfside Beach (West)	Central	-0.02	-0.02	-0.02	0.00	0.00	0.00
Sunshine Bay	Central		0.05			0.08	
Malua Bay	Central	-0.20	-0.10	0.10	-0.18	-0.08	0.12
Guerilla Bay (South)	Central		0.15			0.17	
Barlings Beach	East		-0.05			-0.01	
	West		0.05			0.09	
Tomakin Cove	Central	-0.10	-0.07	-0.03	-0.08	-0.05	-0.01
Broulee Beach	North	-0.05	-0.01	0.05	-0.03	0.01	0.07
	Central	0.20	0.30	0.40	0.22	0.32	0.42
	South	0.10	0.55	0.70	0.12	0.57	0.72

Note: Positive value = accretion trend

Negative value = recession trend

* Adjusted with the modal Bruun factor and a SLR rate of 0.8 mm/year (White et al., 2014).

6. Probabilistic and Deterministic Erosion/Recession Hazard Assessment

6.1 Risk Definitions

Risk is defined as *likelihood* (or *probability*) times *consequence.* Probability is generally expressed in three formats:

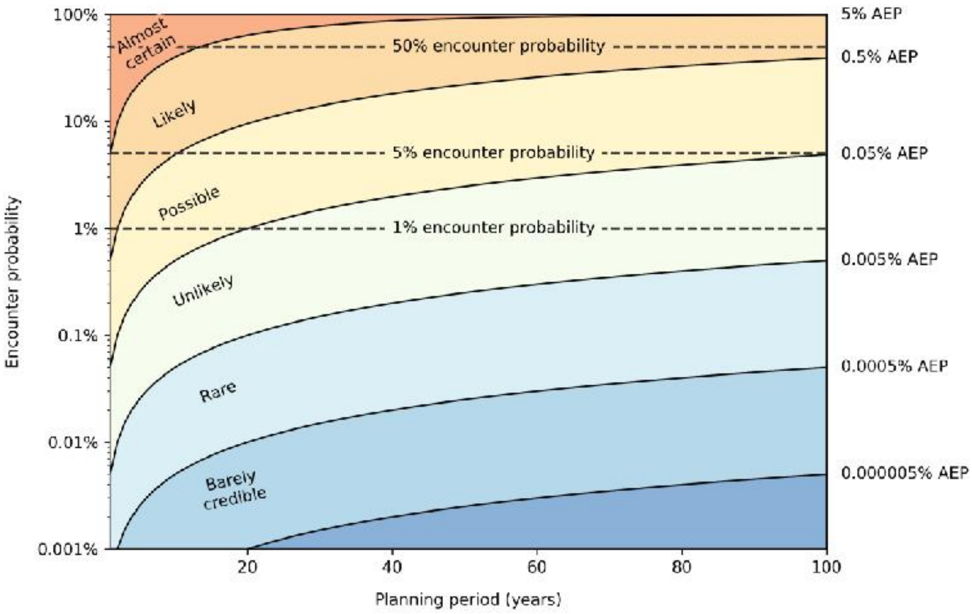
- Average Recurrence Interval (ARI);
- Annual Exceedance Probability (AEP); and
- Encounter Probability (EP) over the planning horizon.

The acceptable likelihood or acceptable risk for private dwellings is considered in several documents, but well accepted or legislated values for coastal hazards are not presently available.

The Building Code of Australia lists the following acceptable design probabilities for freestanding detached private houses:

- Water entry into building: 100 year ARI (1% AEP);
- Wind Load: 500 year ARI (0.2% AEP); and
- Earthquake load: 500 year ARI (0.2% AEP).

The coastal defences in parts of the Netherlands are designed to a 1% encounter probability over a 100 year planning period, which is equivalent to a 10,000 year ARI (Delta Commission, 1962). Figure 6-1 shows qualitative descriptions of likelihood for a range of encounter probabilities and planning periods.



Note: Figure adapted from AGS, 2007

Figure 6-1: Likelihood descriptions of encounter probabilities over a 100 year planning period

6.2 Probabilistic versus Deterministic Assessment of Coastal Hazards

In a deterministic approach, each input variable is assigned a single value and a single estimate (prediction) of shoreline movement is produced. This is usually a "design", "100 year ARI", "best estimate" or "conservative" value. In a probabilistic approach, each independent input variable is allowed to randomly vary over a range of values pre-defined through probability distribution functions. This range covers both uncertainty and error in a heuristic manner. The process of repeatedly combining these randomly sampled values is known as Monte-Carlo simulation.

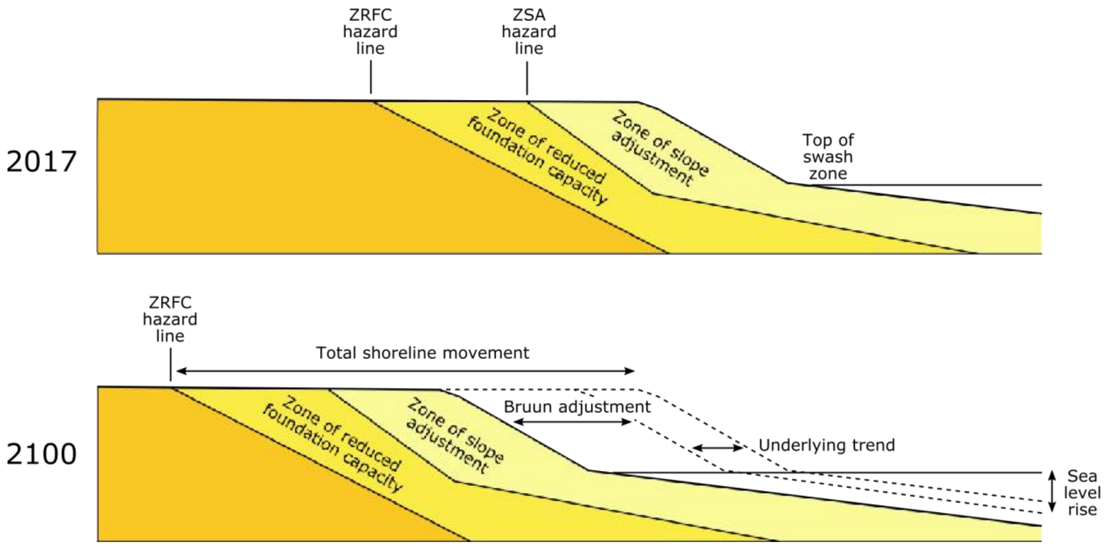
Probabilities of storm demand are also included in this assessment by combining them randomly with the recession probabilities in a further Monte-Carlo simulation. Note that by assuming that the storm demand represents a deviation from the long term average trend, and by expressing the combined probability as an AEP, the probability (AEP) of an eroded shoreline position each year does not need to consider beach recovery on the assumption that recovery occurs within one (1) year. The bounding still relies somewhat on engineering judgement and experience.

6.3 Erosion and Recession Hazards

The coastal erosion hazard lines in this study are based on the landward side of the *Zone of Reduced Foundation Capacity* (ZRFC), a potentially unstable region behind the theoretical erosion escarpment, as described by Nielsen et al., (1992; Figure 6-2). There are four (4) main components forming the position of the hazard line. Numerous other sub-components may aggregate to form these.

The four main components are:

- Shoreline movement due to sediment budget differentials;
- Sea level rise and the recession response to sea level rise (Bruun adjustment);
- Storm erosion; and
- Dune stability or zone of reduced foundation capacity (refer to Appendix G for details on this aspect of the methodology).



Note: Figure modified from Nielsen et al., 1992

Figure 6-2: Zone of Reduced Foundation Capacity (ZRFC) hazard lines

6.4 Probabilistic Input Values

The input variables for each beach in the probabilistic analysis were (Table 6-1):

1. Storm demand;
2. Bruun factor; and
3. Underlying shoreline movement.

Table 6-1: Adopted Input Values for Probabilistic Analysis

Beach	Section	Storm demand volume (m^3/m)[1]		Bruun factor			Underlying shoreline movement (m/year)[4]		
		1% EP[2]	5% EP[3]	min	mode	max	min	mode	max
Long	East	70	46	15	20	50	0.07	0.12	0.22
	Central	100	65	15	20	50	-0.08	0.02	0.12
	West	120	78	15	20	50	0.07	0.17	0.22
Surfside East	North	50	33	20	25	30	-0.13	-0.06	-0.03
	South	60	39	20	25	30	0.07	0.12	0.17
Surfside West		20	13	15	20	30	0.00	0.00	0.00
Malua Bay		120	78	25	30	50	-0.18	-0.08	0.12
Tomakin Cove		90	59	20	25	60	-0.08	-0.05	-0.01
Broulee	North	110	72	25	30	65	-0.03	0.01	0.07
	Central	90	59	25	30	65	0.22	0.32	0.42
	South	70	46	25	30	65	0.12	0.57	0.72

1. Storm demand is the quantity of sand removed during a single storm or a closely spaced series of storms.
2. 1% encounter probability is equivalent to a 100 year ARI storm demand in a single year.
3. 5% encounter probability is equivalent to a 20 year ARI storm demand in a single year.
4. Adjusted with the modal Bruun factor and a SLR rate of 0.8 mm/year (White et al., 2014), -ve= recession.

Sea level rise was considered to be uniform across all beaches, with the value in 2100 ranging from 0.24 m to 0.97 m, relative to the 2017 MSL (Figure 6-3). The modal sea level rise trajectory follows ESC's sea level rise policy and planning framework (RCP 6.0, upper bound – Whitehead & Associates, 2014). The minimum and maximum sea level rise trajectories were established to cover the full range of IPCC projections (IPCC, 2013 and 2014), namely, to locally adjusted projections of RCP 2.6 (lower bound) and RCP 8.5 (upper bound), respectively.

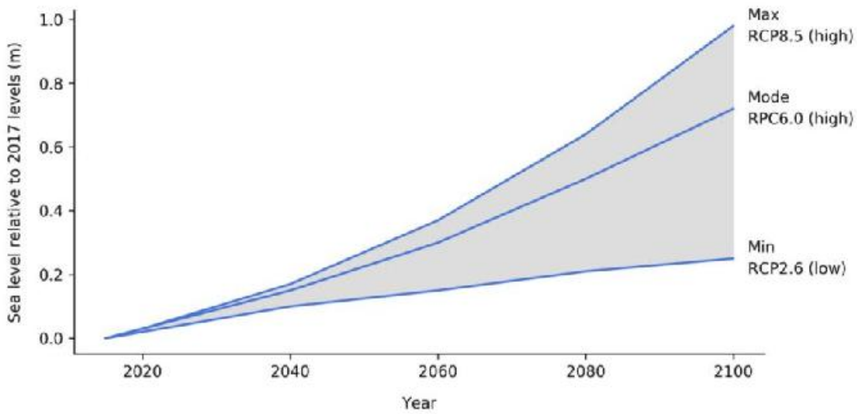


Figure 6-3: Sea level rise input values (Whitehead & Associates, 2014)

To provide an indication of possible shoreline movement due to Bruun recession at each beach and for each planning period, the minimum, mode and maximum Bruun Factors and SLR trajectories are combined in Table 6-2.

Table 6-2: Possible Shoreline Movement of Average Beach Position due to Sea Level Rise for Probabilistic Analysis

Beach	Section	Planning period	Possible Shoreline Movement due to SLR (m)		
			min BF, min SLR	mode BF, mode SLR	max BF, max SLR
Long Beach	East	2017	0.0	0.0	0.0
		2050	-1.7	-4.3	-13.0
		2065	-2.3	-6.8	-21.3
		2100	-3.6	-14.2	-48.5
	Central	2017	0.0	0.0	0.0
		2050	-1.7	-4.3	-13.0
		2065	-2.3	-6.8	-21.3
		2100	-3.6	-14.2	-48.5
	West	2017	0.0	0.0	0.0
		2050	-1.7	-4.3	-13.0
		2065	-2.3	-6.8	-21.3
		2100	-3.6	-14.2	-48.5
Surfside Beach (East)	North	2017	0.0	0.0	0.0
		2050	-2.3	-5.4	-7.8
		2065	-3.1	-8.5	-12.8
		2100	-4.8	-17.7	-29.1
	South	2017	0.0	0.0	0.0
		2050	-2.3	-5.4	-7.8
		2065	-3.1	-8.5	-12.8
		2100	-4.8	-17.7	-29.1
Surfside Beach (West)	West	2017	0.0	0.0	0.0
		2050	-1.7	-4.3	-7.8
		2065	-2.3	-6.8	-12.8
		2100	-3.6	-14.2	-29.1
Malua Bay	Central	2017	0.0	0.0	0.0
		2050	-2.9	-6.4	-13.0
		2065	-3.9	-10.2	-21.3
		2100	-6.0	-21.3	-48.5

Note: Negative value = recession

Table 6-2: Possible Shoreline Movement of Average Beach Position due to Sea Level Rise for Probabilistic Analysis (cont.)

Beach	Section	Planning period	Possible Shoreline Movement due to SLR (m)		
			min BF, min SLR	mode BF, mode SLR	max BF, max SLR
Tomakin Cove	Central	2017	0.0	0.0	0.0
		2050	-2.3	-5.4	-15.6
		2065	-3.1	-8.5	-25.6
		2100	-4.8	-17.7	-58.2
Broulee Beach	North	2017	0.0	0.0	0.0
		2050	-2.9	-6.4	-16.9
		2065	-3.9	-10.2	-27.7
		2100	-6.0	-21.3	-63.0
	Central	2017	0.0	0.0	0.0
		2050	-2.9	-6.4	-16.9
		2065	-3.9	-10.2	-27.7
		2100	-6.0	-21.3	-63.0
	South	2017	0.0	0.0	0.0
		2050	-2.9	-6.4	-16.9
		2065	-3.9	-10.2	-27.7
		2100	-6.0	-21.3	-63.0

Note: Negative value = recession

6.5 Monte-Carlo simulation

6.5.1 Sea level rise and underlying shoreline movement

Random values for sea level rise, Bruun factor, and underlying shoreline movement were simulated using triangular distributions (Figure 6-4), with the values from Table 6-1.

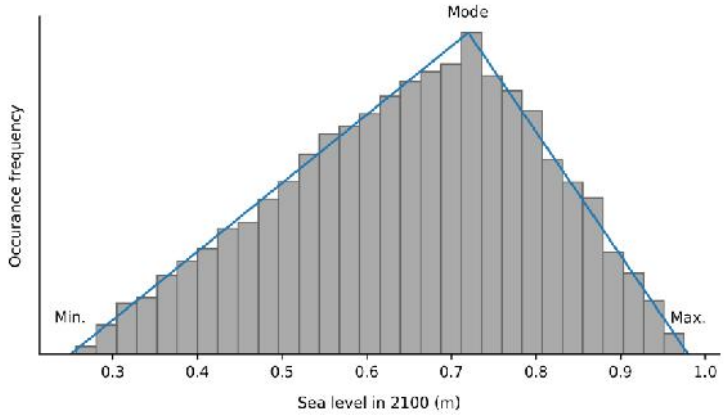


Figure 6-4: Triangular probability density function of sea level rise in 2100

The values for these variables were combined to give a total shoreline movement for each beach. Because the values were combined in a random order with 1,000,000 iterations, the probability density function for the total shoreline movement resembles a Gaussian distribution, rather than a triangular distribution (Figure 6-5). For example, this means that the larger sea levels were only combined with the larger Bruun factors for a small number iterations.

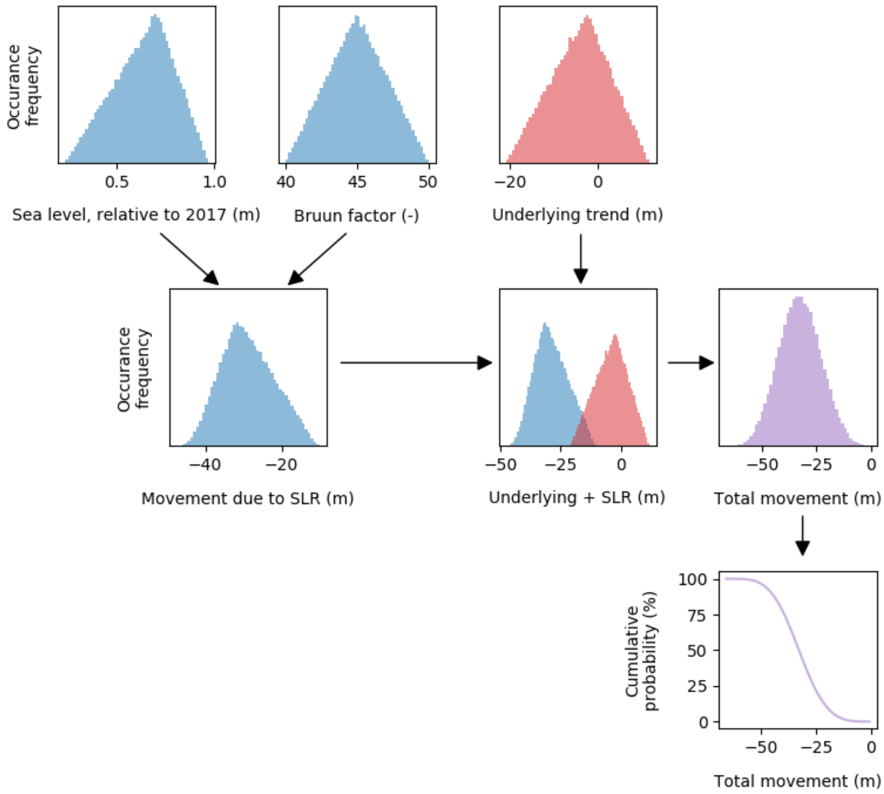
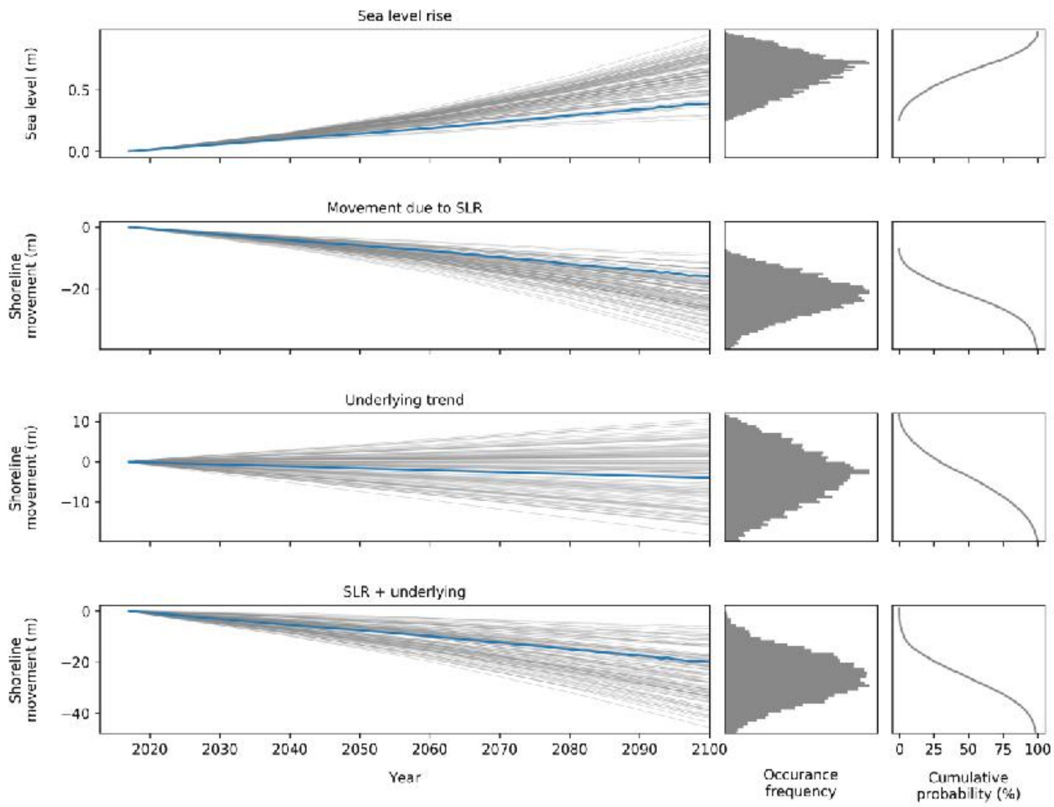


Figure 6-5: Methodology for combining random values to estimate shoreline movement

A set of 1,000,000 Monte-Carlo simulations were completed by randomly combining a constant Bruun factor, a discrete underlying shoreline movement rate, and a time-varying sea level rise trajectory, to create 1,000,000 different possible time series (Figure 6-6).



Note: Blue lines represent the shoreline trajectory for a single probabilistic model result.
Left panels only show the first 100 simulations to minimise clutter.

Figure 6-6: Simulated trajectories for sea level rise and underlying shoreline movement

6.5.2 Storm demand

Storm demand probabilities for each year were calculated using a uniform distribution of AEP values along an interval between 0 and 1 (Figure 6-7).

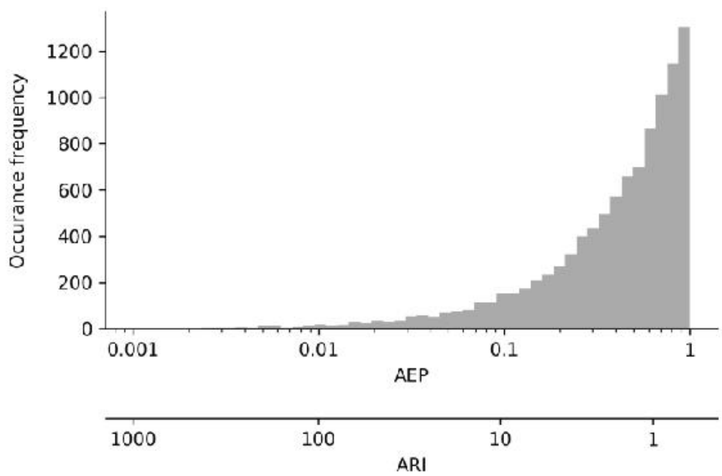


Figure 6-7: Uniform distribution of AEP values for generating storm demand volumes

The AEP values were converted to erosion volumes using the method described in Gordon (1987), based on the individual reference 100 year ARI storm demand volume for each beach. The Gordon method is only defined for 100 year ARI storm demand volumes between 140 m^3/m and 220 m^3/m. Many of the beaches in this study are somewhat sheltered, and have lower storm demand volumes. The defining equations Gordon (1987) were modified for these somewhat sheltered beaches to ensure that the storm demand was always greater than zero (Figure 6-8, Table 6-3).

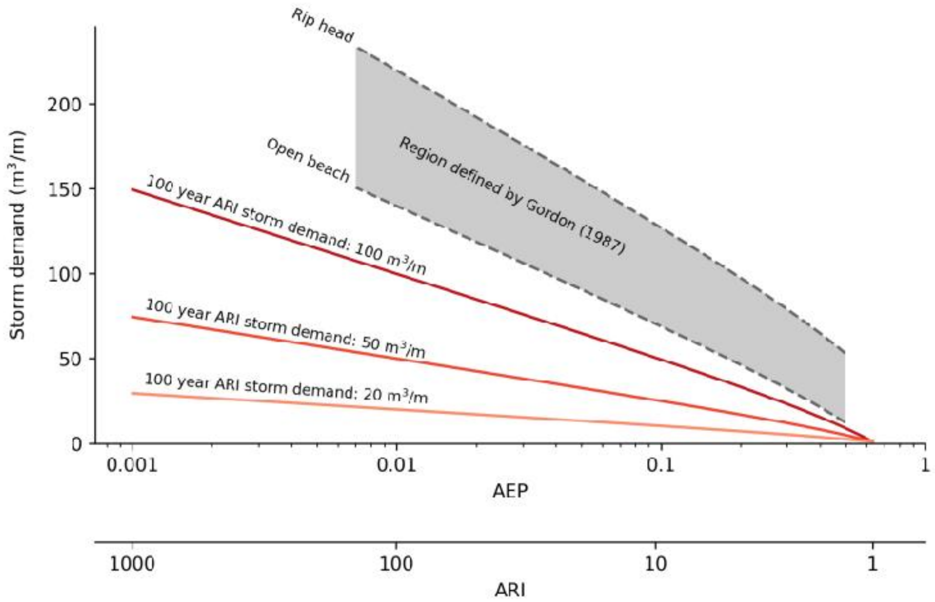


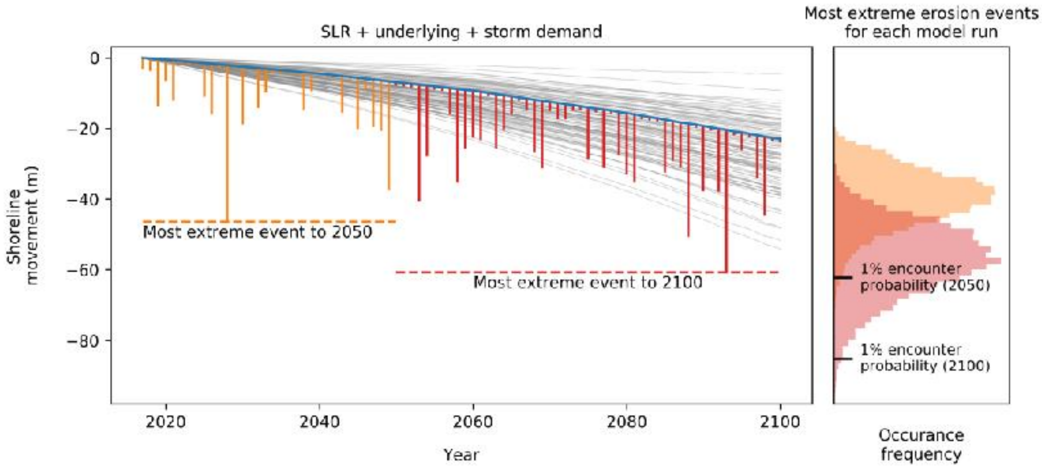
Figure 6-8: Storm demand volumes for exposed beaches in NSW (after Gordon, 1987)

Table 6-3: Adopted Storm Demand Values for Probabilistic Analysis

Beach	Section	Storm demand volume (m^3/m)						
		10,000 year ARI	1,000 year ARI	100 year ARI	20 year ARI	10 year ARI	1.4 year ARI	1 year ARI
		0.01% AEP	0.1% AEP	1% AEP	5% AEP	9.5% AEP	50% AEP	63% AEP
Long	East	139	105	70	46	36	6	1
	Central	199	150	100	65	51	8	1
	West	239	180	120	78	61	10	1
Surfside East	North	99	75	50	33	26	5	1
	South	119	90	60	39	31	5	1
Surfside West		39	30	20	13	11	2	1
Malua Bay		239	180	120	78	61	10	1
Tomakin Cove		179	135	90	59	46	8	1
Broulee	North	219	165	110	72	56	9	1
	Central	179	135	90	59	46	8	1
	South	139	105	70	46	36	6	1

6.6 Erosion Hazard Lines

The storm demand volumes were converted to horizontal erosion distances to the back of the ZRFC (Figure 6-2), based on the photogrammetry records for each beach profile. The storm demand was calculated separately for each Monte-Carlo simulation, and was combined with sea level rise, and underlying trend to calculate a receded shoreline position for each year. Each beach was allowed to recover from any storm-driven erosion at the beginning of the year. The most extreme erosion event was identified for all of the different planning periods in each simulation, and the erosion hazard lines were calculated from these events, for each encounter probability (Figure 6-9).

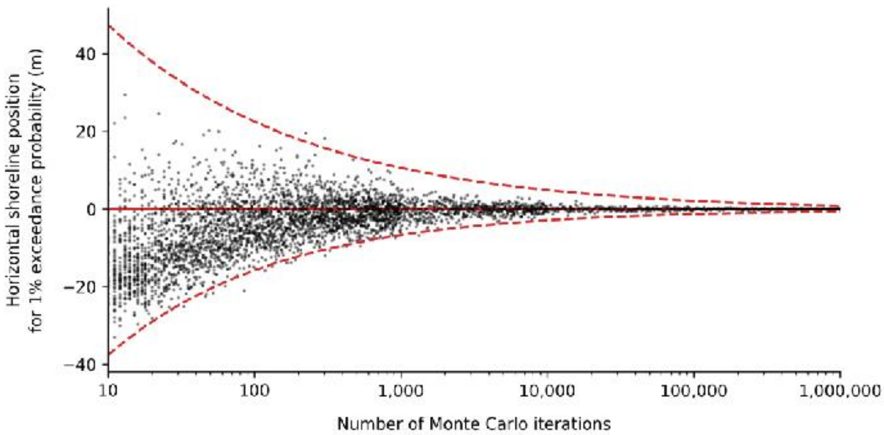


Note: Orange and red bars represent storm demand erosion for a single probabilistic simulation result.
Left panel only shows the first 100 simulations to minimise clutter.

Figure 6-9: Simulated storm demand superimposed on background shoreline movement

6.7 Sensitivity

A total of 1,000,000 runs were used for the Monte-Carlo simulation. The sensitivity of this number of runs was tested, and the scatter in the simulated shoreline position was found to be less than 1 m (Figure 6-10).



Note: Each dot shows unique simulation result for the same beach profile.

Figure 6-10: Sensitivity of Monte-Carlo simulation

6.8 Deterministic Assessment

The methodology for the deterministic assessment was similar to that of the probabilistic assessment, but a single value for each parameter was adopted, rather than a range (Table 6-4). This deterministic approach resulted in a single shoreline movement trajectory for each profile (Figure 6-11). The shoreline movement only due to Bruun recession at each beach and for each planning period is tabulated in Table 6-5. A single 100 year ARI storm demand volume was adopted for each beach (Figure 6-12).

Table 6-4: Adopted Input Values for Deterministic Aanalysis

Beach	Section	Storm demand (m^3/m)	Bruun factor (-)	Underlying shoreline movement[1] (m/year)
Maloneys Beach	East	50	10	-0.04
	West	80	10	0.05
Sunshine Bay	Central	25	40	0.08
Guerilla Bay	Central	80	25	0.17
Barlings Beach	East	60	50	-0.01
	West	110	50	0.09

1. Adjusted with a local Bruun factor and a SLR rate of 0.8 mm/year (White et al., 2014), -ve = recession

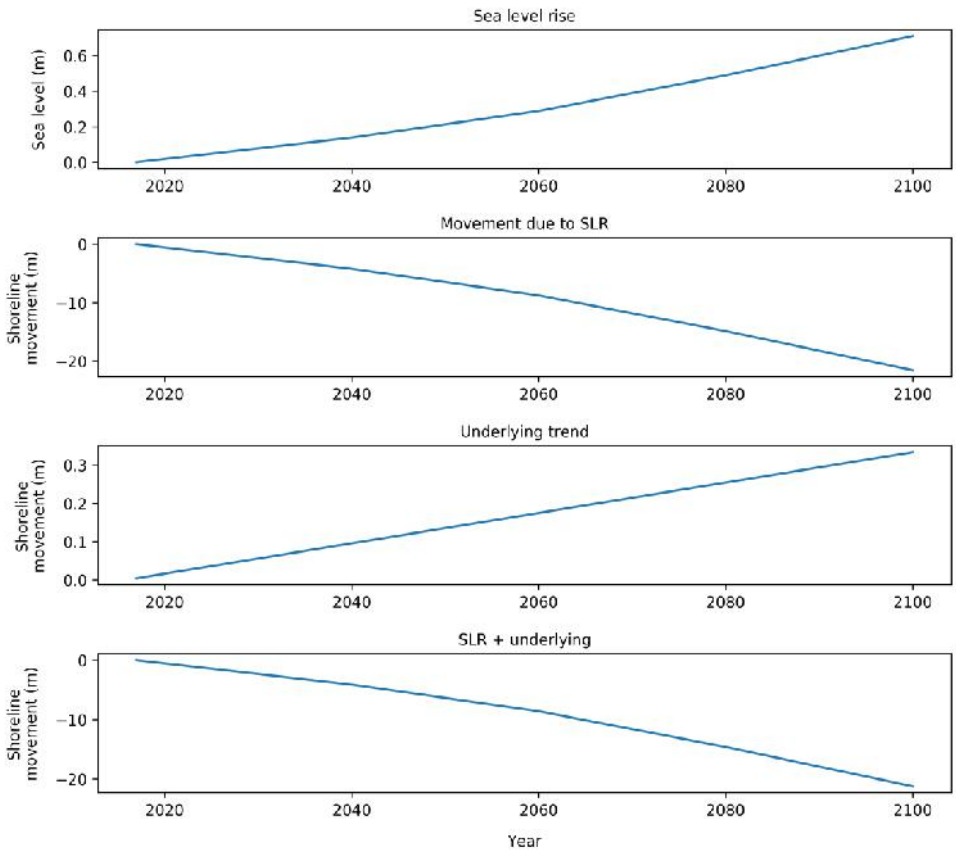


Figure 6-11: Calculated deterministic trajectories for sea level rise and underlying recession

Table 6-5: Estimated Shoreline Movement of Average Beach Position due to Sea Level Rise from Deterministic Analysis

Beach	Section	Planning period	Movement due to SLR (m)
Maloneys Beach	East	2017	0.0
		2050	-2.1
		2065	-3.4
		2100	-7.1
	West	2017	0.0
		2050	-2.1
		2065	-3.4
		2100	-7.1
Sunshine Bay	Central	2017	0.0
		2050	-8.6
		2065	-13.6
		2100	-28.4
Guerilla Bay (South)	Central	2017	0.0
		2050	-5.4
		2065	-8.5
		2100	-17.7
Barlings Beach	East	2017	0.0
		2050	-10.7
		2065	-17.0
		2100	-35.5
	West	2017	0.0
		2050	-10.7
		2065	-17.0
		2100	-35.5

Note: Negative value = recession

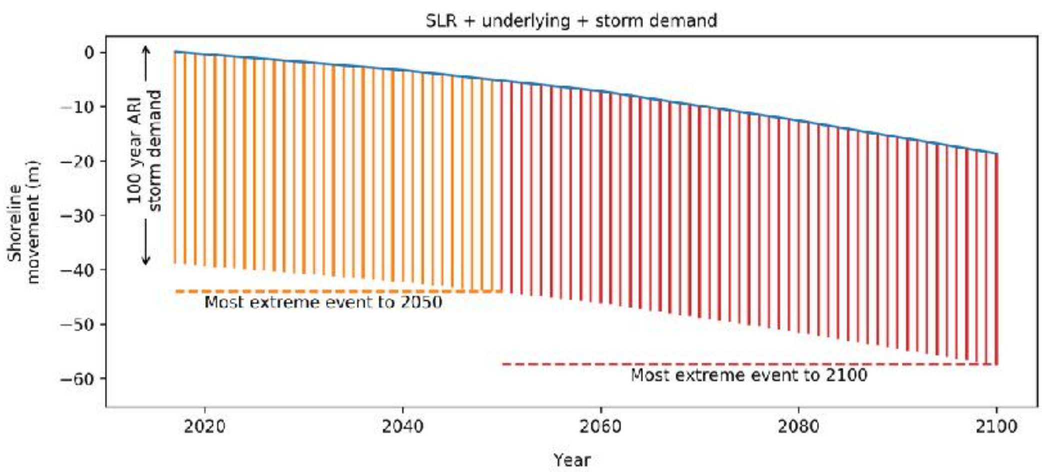


Figure 6-12: 100 year ARI storm demand superimposed on deterministic shoreline movement

6.9 Erosion/Recession Hazard Mapping

6.9.1 Overview

Table 6-6 summarises the list of maps prepared and shown in Appendix I for four planning periods (2017, 2050, 2065 and 2100). For Long Beach and Malua Bay, two scenarios were mapped; with the existing seawall in place and for the case of seawall failure. For Broulee Beach; two different scenarios were mapped; with Broulee Island attached by a tombolo and with Broulee Island detached.

Table 6-6: List of Erosion/Recession Hazard Maps

Beach	Erosion/Recession Methodology	Scenarios
Maloneys Beach	Deterministic	-
Long Beach	Probabilistic	With Existing Seawall and No Seawall
Surfside Beach (east)	Probabilistic	-
Surfside Beach (west)	Probabilistic	-
Sunshine Bay	Deterministic	-
Malua Bay	Probabilistic	With Existing Seawall and No Seawall
Guerilla Bay (south)	Deterministic	-
Barlings Beach	Deterministic	-
Tomakin Cove	Probabilistic	-
Broulee Beach	Probabilistic	Broulee Island attached and Broulee Island detached

6.9.2 Assumed Initial Beach Conditions

The most recent photogrammetry profiles for each beach were used for the erosion/recession mapping, except for Surfside Beach (west). These were generally from 2014, except for Barlings Beach and Broulee Beach, which were from 2011. Note that the 2011/2014 photogrammetry profiles have been considered equivalent to the present day (2017) beach condition without any adjustment for underlying movement or Bruun recession (i.e. no sea level rise response between 2011/2014 and 2017 due to the short time difference).

The most eroded beach alignment on record was developed for Surfside Beach (west) following a similar methodology previously used at Wharf Road (Webb, McKeown and Associates, 2005a and 2005b and BMT WBM, 2009). This approach ignored the presence of the dredged sand placed on the beach in December 2016 due to the strong influence of the flood tide delta on shoreline position. The 1942 (B1P4), 2011 (B1P5 and B1P6) and 1959 (B1P7) photogrammetry data were used for erosion/recession mapping at Surfside Beach (west). The 1980 photogrammetry profiles were also used for the central and southern sections of Broulee Beach for the Broulee Island detached scenario (this is only photogrammetry year when the salient/tombolo was classified as not being fully connected – see Appendix H).

6.9.3 Special Notations

For those beaches with non-erodible (over planning horizons, geological timescales) material landward of the present shoreline which may limit shoreline movement (erosion), a "bedrock (non-erodible)" line was included on the erosion/recession maps. This was mapped following consideration of observations during site inspections, coastal quaternary geological maps (Troedson and Hashimoto, 2013) and LIDAR elevation data. Where no erosion/recession hazard lines are shown landward of a "bedrock (non-erodible)" line, this feature represents the limit of erosion/recession (i.e. the cliff line is the erosion/recession hazard line). Areas landward of the "bedrock (non-erodible)" line could be subject to coastal cliff or slope instability hazards, which are beyond the scope of this study."

For those beaches with a watercourse entrance (Table 6-6), a "watercourse instability region" notation was included on the erosion/recession maps. This has been mapped qualitatively following consideration of historical aerial photography (where available), photogrammetry profiles adjacent to each watercourse entrance and any control points such as natural bedrock, bridge abutments, box culverts and pipe outlets. These regions should be considered representative of areas influenced by present day (2017) entrance dynamics. Assessment of the estimated influence of climate change (i.e. sea level rise, altered hydrology or suspended sediments) on entrance dynamics is outside the scope of works. In watercourse entrance instability regions, the shoreline could potentially move landward of the erosion/recession hazard lines due to lowering of the beach profile from entrance scouring and migration.

Table 6-7: Watercourse Entrances within the Beaches Requiring Detailed Erosion Mapping

Name	Location
Maloneys Creek	Western end of Maloneys Beach
Reed Swamp	Centre of Long Beach
Surfside Creek	Western end of Surfside Beach (West)
Reedy Creek	Northern end of Malua Bay
Unnamed Creek 1	Southern end of Malua Bay
Unnamed Creek 2	Centre of Guerilla Bay (South)
Unnamed Creek 3	Eastern end of Barlings Beach
Tomaga River	Southern end of Tomakin Beach
Candlagan Creek	Northern end of Broulee Beach

At Tomakin Cove only, a "potential salient loss region" notation was included on the erosion/recession maps. This has been mapped qualitatively following consideration of the present day (2017) beach planform and the landward penetration of the erosion/recession hazard lines at the centre of the cove. The rock/reef at the southern end of the cove presently influences the beach planform, and particularly controls the sand salient feature directly in its lee. While it is outside the scope of works to quantify in this study, at some quantum of future sea level rise, this rock/reef will have reduced control over the southern beach planform causing the loss of the coastal area composing the salient. As a result, the shoreline could potentially move landward of the erosion/recession hazard lines in this region. The effect of sea level rise (directly related to wave transmission over a reef) on the salient extent is shown in Figure 6-13. Figure 6-14 also provides an example of the loss of a salient/tombolo controlled by rock reef/island at Woody Bay. While salient loss at Woody Bay was related to reduced sediment supply (rather than sea level rise), it illustrates the dramatic change in planform that may occur with this coastal hazard.

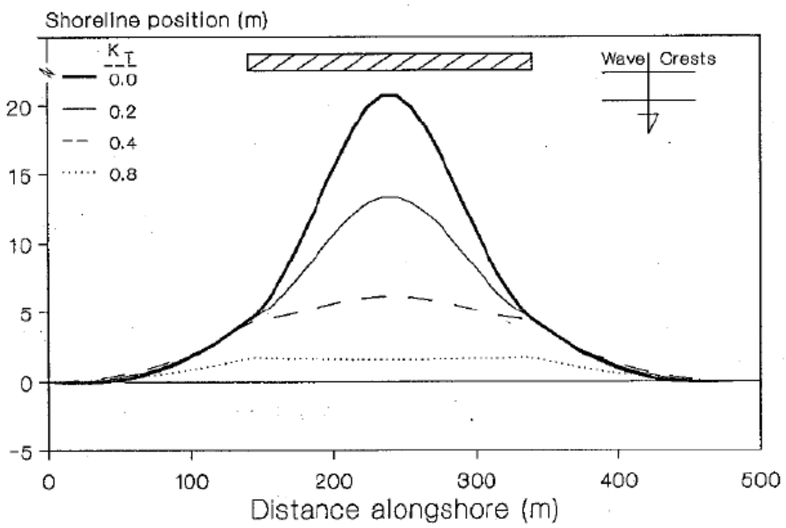


Figure 6-13: Effect of Wave Transmission (K_T) over a reef on the extent of a salient (Source: Hanson et al., 1990)

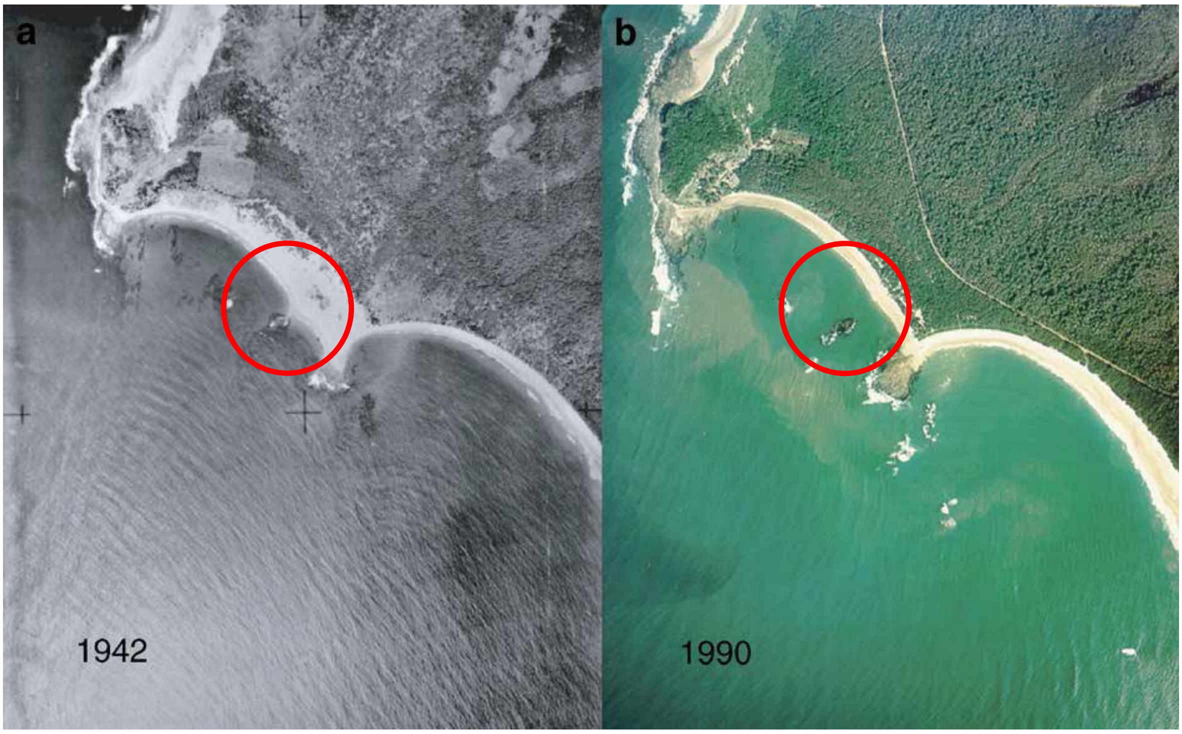


Figure 6-14: Aerial photographs taken in (a) 1942 and (b) 1990 at Woody Bay, NSW illustrates an example of salient loss. (Source: Goodwin et al., 2006)

At Broulee Beach only, an "ephemeral tombolo zone" notation was include on the erosion/recession map for the Broulee Island detached scenario. This has been mapped qualitatively following consideration of historical aerial photography at times when Broulee Island was not connected to Broulee Beach. This region should be considered as temporary land which will be eroded when/if the tombolo is severed again at some stage in the future.

6.9.4 Zone of Slope Adjustment

While all erosion/recession hazard lines in Appendix I are based on the landward side of the ZRFC, the distance from these lines to the seaward side of the ZRFC (the landward side of the ZSA) is tabulated for every photogrammetry profile in Appendix J.

7. Tidal Inundation Hazard Assessment

7.1 Preamble

Tidal inundation is the extent to which the land around the coastline is inundated by regular tide events without any further allowance for additional elevated components (storm surge, river flooding, wave setup or wave runup). It represents the level of nuisance flooding inundation that can be expected in low-lying coastal areas from tidal events. Tidal inundation is presented for the High High Water Solstices Springs (HHWSS) water level and the 63% AEP (1 year ARI) water levels (astronomical tide and anomaly but excluding wave effects - Figure 7-1) for present day (2017), 2050, 2065 and 2100 projected conditions.

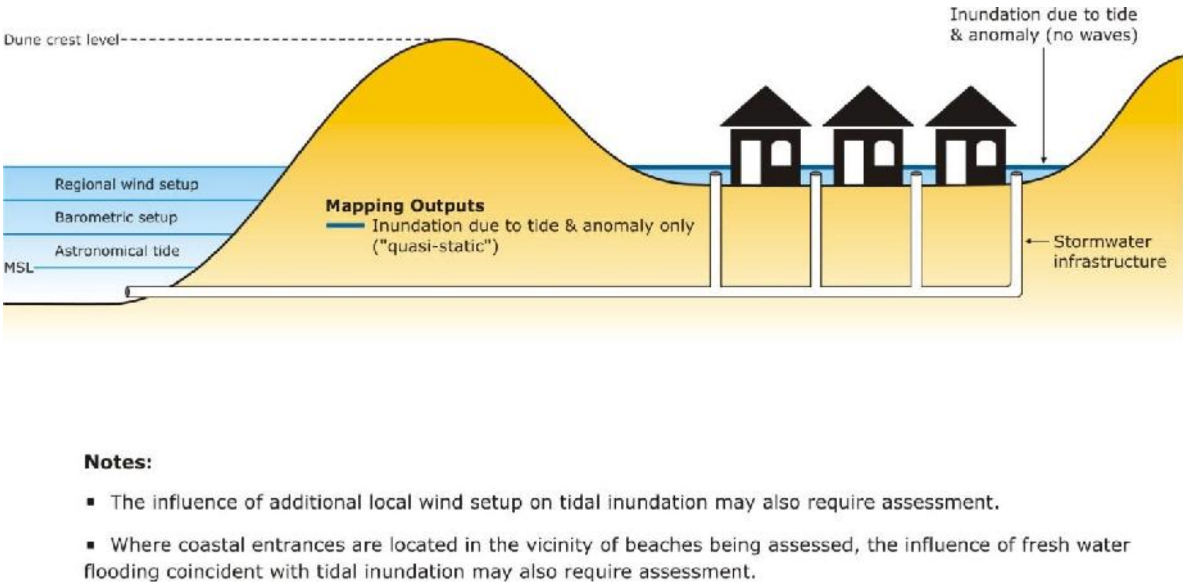


Figure 7-1: Components of Inundation Without Wave Effects

The HHWSS tidal level is reached by the higher of the two daily spring high water heights around the solstices in December and June each year. Colloquially, such astronomical tidal events are described as "king tides". Without additional elevated components, this tidal water level is expected to occur approximately three times per year (Willing and Partners, 1989c).

7.2 Mapping Methodology

The present day HHWSS level at Batemans Bay (Princess Jetty) is 0.92 m AHD (MHL, 2010), and the 63% AEP water level is 1.22 m AHD. Allowance for future sea level rise (SLR) has been included in accordance with ESC's planning policy. Present day and future inundation levels determined in accordance with ESC's sea level rise policy and planning framework, excluding wave effects, are shown in Table 7-1. Note that the future inundation levels in Table 7-1 do not take into account possible changes to tidal constituent amplitudes due to changes in local water depths or bed elevations. This assumption is most pertinent for inundation of tidal watercourses located behind most beaches. Hazard maps for potential inundation areas for the HHWSS tidal level and the 63% AEP (1 year ARI) water level (both excluding wave effects) for four planning periods (2017, 2050, 2065 and 2100) are shown in Appendix K.

Table 7-1: Levels Used in Tidal Flood Inundation Analysis

Planning Period	**HHWSS Tidal Level** (m AHD)	**63% AEP Water Level** (m AHD)	**Increase above 2017 Mean Sea Level** (m)
Present Day (2017)	0.92	1.22	0.00
2050	1.14	1.44	0.22
2065	1.25	1.55	0.33
2100	1.63	1.93	0.71

Hazard areas for inundation have been mapped using the most recent available LIDAR information (2005 inside Batemans Bay and 2011 elsewhere), based on elevation information only. A "quasi-static" methodology has been used to map the tidal inundation extents, which assumes that all areas below the specified coastal water level will be inundated. Specifically, the maps provided have not been adjusted at locations where there is isolated areas that appear to lack connection to the coastal tide events, or where channel constrictions, roughness or other similar flow impediments may prevent sufficient hydraulic connectivity for inland flood levels to reach the full extent of tidal levels. Should ESC identify areas of particular concern of tidal or nuisance flooding, it is suggested that more detailed hydraulic modelling be undertaken to eliminate or confirm their validity.

7.3 Historical Tidal Inundation Photos

In support of a report prepared by Watson and Frazer (2009), Mr Norman Lenehan of ESC photographed a large spring tide event on 12 January 2009. The water level peaked at 1.00 m AHD at 9:00 (AEDST) at the Princess Jetty tide gauge in Batemans Bay and coincided with relatively calm wave conditions. Example photographs at, or close to, this water level (which is higher than HHWSS but lower than the 1 year ARI water level) are shown for several locations within Batemans Bay in Figure 7-2, Figure 7-3, Figure 7-4 and Figure 7-5.

Figure 7-2: Surfside Beach (east): 1.0 m AHD Water Level - 12 January, 2009 (ESC, 2009)

Figure 7-3: Surfside Beach (west): 1.0 m AHD Water Level - 12 January, 2009 (ESC, 2009)

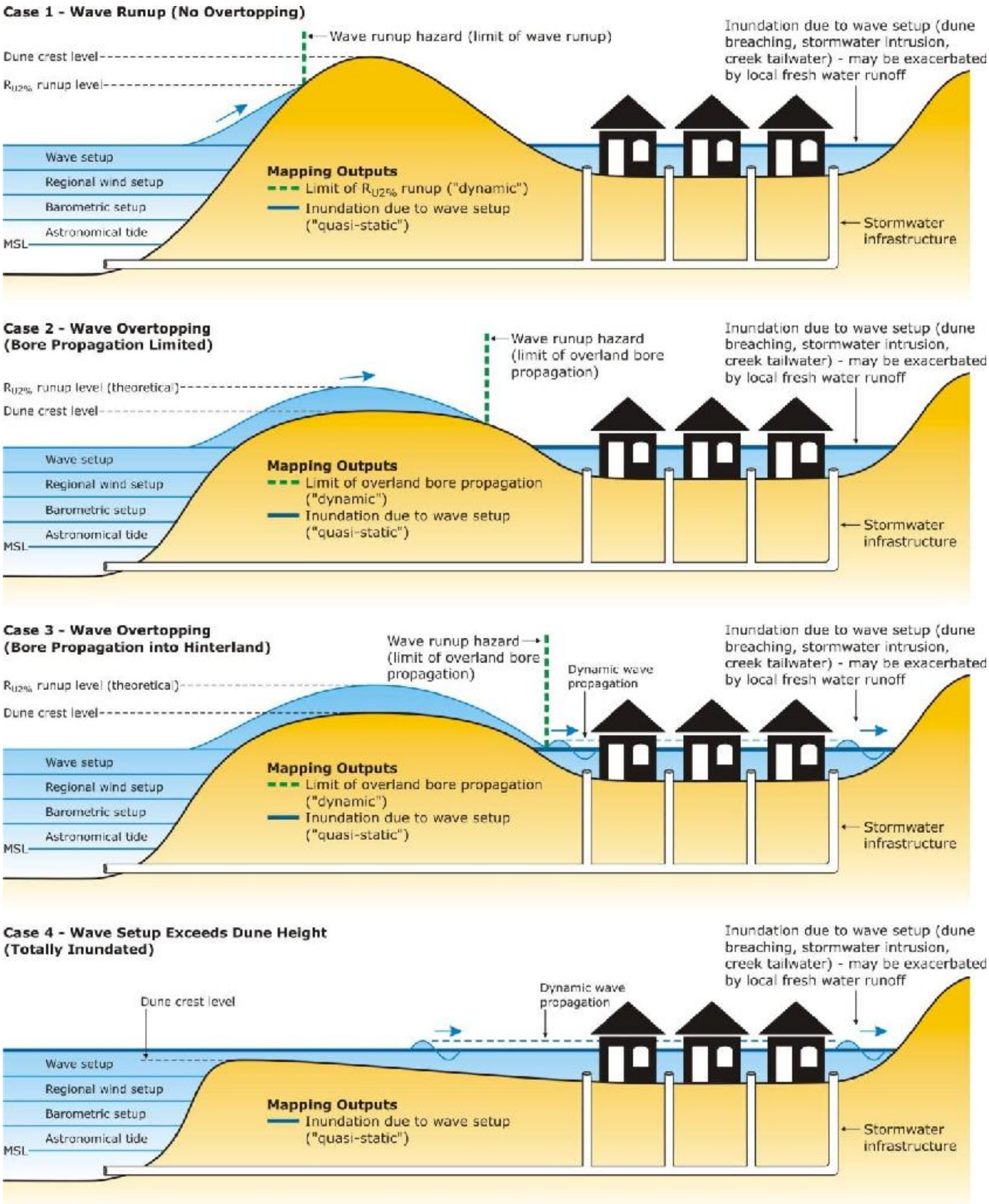
Figure 7-4: Wharf Road: 1.0 m AHD Water Level - 12 January, 2009 (ESC, 2009)

Figure 7-5: Central Business District: 1.0 m AHD Water Level - 12 January, 2009 (ESC, 2009)

8. Coastal Inundation Hazard Assessment

8.1 Preamble

Coastal inundation is the flooding of coastal areas by ocean waters and is typically caused by elevated water levels combined with extreme waves impacting the coast. Figure 8-1 diagrammatically represents the different components contributing to coastal inundation.



Notes:

- The influence of additional local wind setup on coastal inundation may also require assessment.
- Where coastal entrances are located in the vicinity of beaches being assessed, the influence of fresh water flooding coincident with coastal inundation may also require assessment.
- For cases 3 and 4 the influence of dynamic wave propagation on coastal inundation may also require assessment.

Figure 8-1: Components of Coastal Inundation

The "quasi-static" inundation level is the most representative inundation level for areas located away from direct impact of the waves (generally those properties which are not in the front row facing the water). Estimates of wave runup and overtopping are predictors of the wave impacts that beachfront structures are likely to suffer during extreme storm events.

This chapter outlines the methodologies supporting the coastal inundation hazard maps prepared for the 63% AEP (1 year ARI), 5% AEP (20 year ARI) and 1% AEP (100 year ARI) water levels for four planning periods (2017, 2050, 2065 and 2100) are shown in Appendix L.

8.2 Tide and Storm Surge Water Levels

As discussed in Section 4.3.2, adopted offshore extreme water levels for the study area are reproduced in Table 8-1. These levels do not include wave setup, wave runup or additional setup that occurs within Batemans Bay due to its shallow bathymetry and discharges from the Clyde River.

As agreed with OEH, the 5% and 1% AEP storm events are assumed to have complete dependence between extreme water levels and wave heights. However, this is considered overly conservative for the 63% AEP conditions. After consultation with OEH, the 63% AEP wave event was chosen to coincide with MHW as this provides a more realistic estimate of frequent coastal inundation levels.

Table 8-1: Adopted Present Day Extreme Water Levels (Excluding Wave Setup, Wave Runup and Additional Setup within Batemans Bay)

AEP %	ARI (years)	Water Level (m AHD)
63	1	1.22
5	20	1.37
1	100	1.43
Mean High Water		0.508

As discussed in Section 4.3.3, additional water level super-elevation is to be allowed for inside Batemans Bay due to due to wind setup (Table 4-13) and inland flood events (Table 4-14).

8.3 Wave Setup

8.3.1 General Methodology

Wave setup was calculated at 35 representative cross-sections across the study area. To determine the wave setup, H_{RMS} (m) corresponding to the adopted nearshore wave conditions extracted from the SWAN model (see Appendix D) were first calculated according to CIRIA (2007) in Equation 8.1.

$$H_{RMS} = 0.706 \times H_S \qquad (8.1)$$

Figure 8-2: SBEACH profiles - northern area

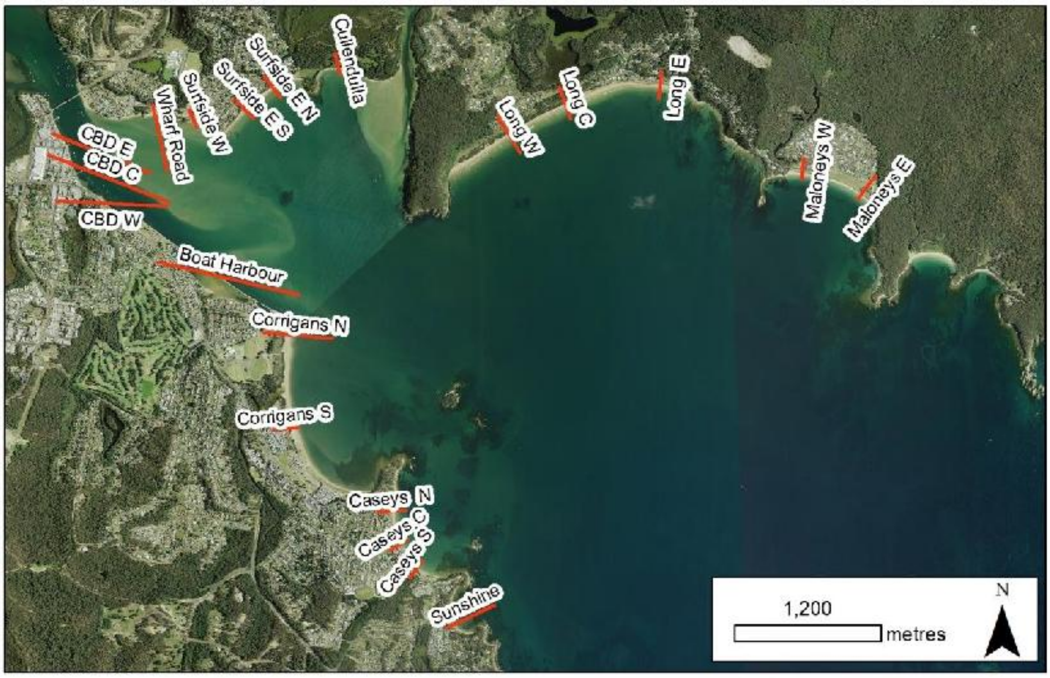


Figure 8-3: SBEACH profiles - inner Batemans Bay

Figure 8-4: SBEACH profiles - southern area

These wave heights, along with the corresponding peak spectral periods, were applied as a boundary condition to the Dally, Dean and Dalrymple (1984) two-dimensional surf zone model, implemented using the numerical modelling software SBEACH (Version 4.03). At each of the 35 profiles, topographic (LiDAR) and bathymetric data (nearshore bathymetric surveys and Australian Hydrographic Service bathymetry) were extracted as an input to the SBEACH model. The resultant water level was then extracted to determine the wave setup at each profile, an example of which is shown in Figure 8-5 for 1% AEP conditions at Malua Bay.

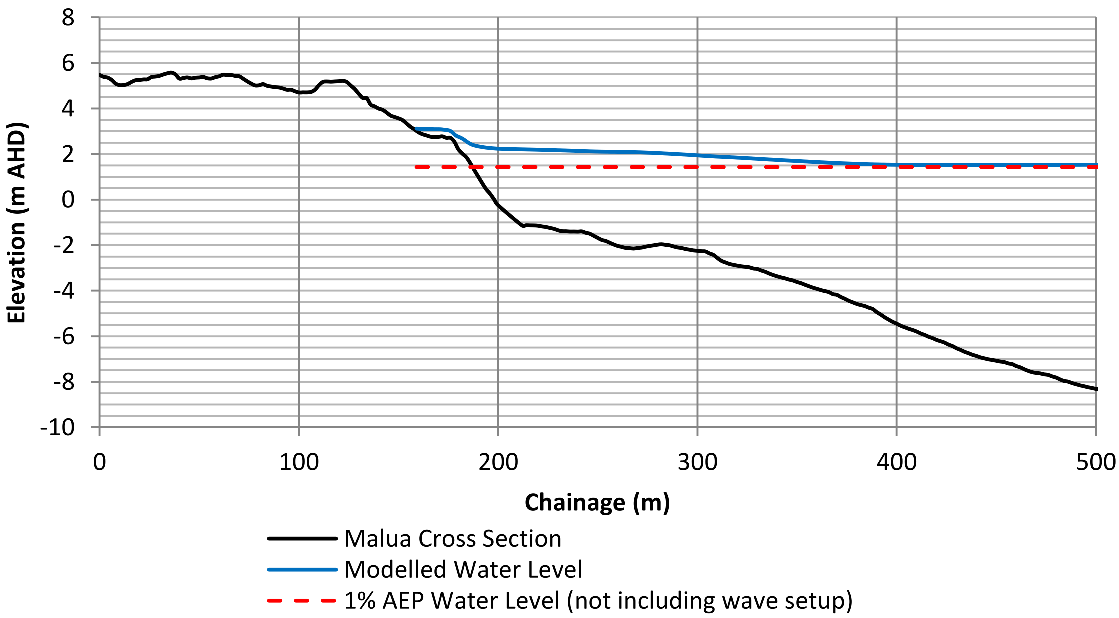


Figure 8-5: Example of SBEACH wave setup modelling at Malua Bay

8.3.2 Methodology for Beaches without Nearshore Bathymetric Survey Data

At Durras Beach and Cookies Beach, there was no available nearshore bathymetric surveys. The AHS bathymetric data in this area has contours starting at -15 m AHD, but very little available information closer to the shore. To fill the nearshore region, contours based on a Dean Equilibrium Profile (Dean, 1977) was assumed based on the measured grain size of 0.37 mm, shown in Figure 8-6. This equilibrium profile information was used as required for the bathymetry portion of the Durras Beach and Cookies Beach profiles.

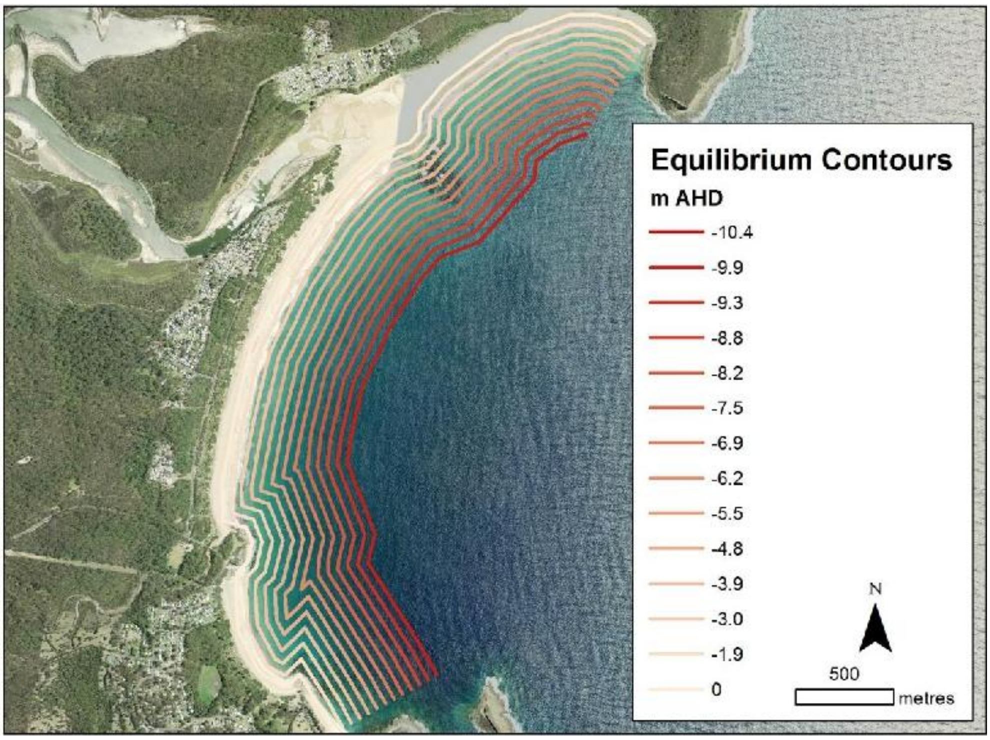


Figure 8-6: Dean equilibrium contours for Durras Beach and Cookies Beach

A similar process was undertaken at Sunshine Bay where the 1995 Batemans Bays survey only came inshore to approximately –5 m AHD. The nearshore region was filled with a Dean Equilibrium Profile based on a grainsize of 0.21 mm.

8.4 Summary of "Quasi-Static" Water Level Conditions

Table 8-2 summarises the "quasi-static" water level components for the present day planning period. Table 8-3 lists the total static water levels for the 2017, 2050, 2065 and 2100 planning periods in accordance with ESC's sea level rise policy and planning framework.

Table 8-2: Summary of Static Water Level Conditions for Present Day, Including All Elements

Beach	Profile	ARI (years)	Water Level (m AHD) - excluding setup and flood	Flood Contribution (m)	Bay Wind Setup (m)	Wave Setup (m)	Total SWL (m AHD)
Durras	North	1	0.51 (MHW)	0.00	0.00	0.78	1.29
		20	1.37	0.00	0.00	1.06	2.44
		100	1.43	0.00	0.00	1.12	2.55
	Central	1	0.51 (MHW)	0.00	0.00	0.97	1.48
		20	1.37	0.00	0.00	1.35	2.72
		100	1.43	0.00	0.00	1.45	2.89
	South	1	0.51 (MHW)	0.00	0.00	1.09	1.60
		20	1.37	0.00	0.00	1.50	2.87
		100	1.43	0.00	0.00	1.52	2.96
Cookies	-	1	0.51 (MHW)	0.00	0.00	0.67	1.17
		20	1.37	0.00	0.00	0.87	2.24
		100	1.43	0.00	0.00	0.91	2.34
Maloneys	East	1	0.51 (MHW)	0.00	0.00	0.20	0.71
		20	1.37	0.00	0.10	0.37	1.84
		100	1.43	0.00	0.12	0.46	2.01
	West	1	0.51 (MHW)	0.00	0.00	0.35	0.85
		20	1.37	0.00	0.11	0.55	2.03
		100	1.43	0.00	0.13	0.57	2.13
Long	East	1	0.51 (MHW)	0.00	0.00	0.42	0.93
		20	1.37	0.00	0.18	0.46	2.01
		100	1.43	0.00	0.23	0.48	2.14
	Central	1	0.51 (MHW)	0.00	0.00	0.41	0.92
		20	1.37	0.00	0.18	0.63	2.18
		100	1.43	0.00	0.22	0.66	2.31
	West	1	0.51 (MHW)	0.00	0.00	0.44	0.94
		20	1.37	0.00	0.16	0.62	2.15
		100	1.43	0.00	0.19	0.66	2.28
Cullendulla	-	1	0.51 (MHW)	0.00	0.00	0.22	0.73
		20	1.37	0.01	0.18	0.46	2.02
		100	1.43	0.02	0.23	0.47	2.15
Surfside E	North	1	0.51 (MHW)	0.00	0.00	0.61	1.12
		20	1.37	0.02	0.20	0.73	2.32
		100	1.43	0.03	0.25	0.62	2.33
	South	1	0.51 (MHW)	0.00	0.00	0.57	1.08
		20	1.37	0.02	0.21	0.76	2.36
		100	1.43	0.02	0.26	0.65	2.36

Table 8-2: Summary of Static Water Level Conditions for Present Day, Including All Elements (contd.)

Beach	Profile	ARI (years)	Water Level (m AHD) - excluding setup and flood	Flood Contribution (m)	Bay Wind Setup (m)	Wave Setup (m)	Total SWL (m AHD)
Surfside W	-	1	0.51 (MHW)	0.00	0.00	0.39	0.90
		20	1.37	0.04	0.10	0.45	1.96
		100	1.43	0.07	0.13	0.43	2.06
Wharf Rd	-	1	0.51 (MHW)	0.00	0.00	0.38	0.88
		20	1.37	0.04	0.10	0.47	1.98
		100	1.43	0.07	0.13	0.47	2.10
CBD	West	1	0.51 (MHW)	0.00	0.00	0.54	1.05
		20	1.37	0.04	0.13	0.41	1.95
		100	1.43	0.10	0.16	0.41	2.11
	Central	1	0.51 (MHW)	0.00	0.00	0.51	1.02
		20	1.37	0.03	0.13	0.36	1.90
		100	1.43	0.05	0.16	0.37	2.02
	East	1	0.51 (MHW)	0.00	0.00	0.49	1.09
		20	1.37	0.03	0.12	0.54	2.08
		100	1.43	0.05	0.15	0.56	2.22
Boat Harbour	-	1	0.51 (MHW)	0.00	0.00	0.67	1.18
		20	1.37	0.03	0.08	0.61	2.09
		100	1.43	0.06	0.10	0.61	2.21
Corrigans	North	1	0.51 (MHW)	0.00	0.00	0.31	0.82
		20	1.37	0.01	0.07	0.64	2.09
		100	1.43	0.01	0.08	0.67	2.19
	South	1	0.51 (MHW)	0.00	0.00	0.25	0.75
		20	1.37	0.00	0.08	0.27	1.72
		100	1.43	0.00	0.10	0.28	1.82
Caseys	North	1	0.51 (MHW)	0.00	0.00	0.48	0.98
		20	1.37	0.00	0.08	0.58	2.04
		100	1.43	0.00	0.10	0.54	2.08
	Central	1	0.51 (MHW)	0.00	0.00	0.75	0.81
		20	1.37	0.00	0.08	1.60	1.61
		100	1.43	0.00	0.10	1.68	1.70
	South	1	0.51 (MHW)	0.00	0.00	0.24	0.75
		20	1.37	0.00	0.07	0.30	1.74
		100	1.43	0.00	0.10	0.30	1.83
Sunshine	-	1	0.51 (MHW)	0.00	0.00	0.66	1.17
		20	1.37	0.00	0.00	1.08	2.45
		100	1.43	0.00	0.00	1.07	2.50

Table 8-2: Summary of Static Water Level Conditions for Present Day, Including All Elements (contd.)

Beach	Profile	ARI (years)	Water Level (m AHD) - excluding setup and flood	Flood Contribution (m)	Bay Wind Setup (m)	Wave Setup (m)	Total SWL (m AHD)
Malua	-	1	0.51 (MHW)	0.00	0.00	0.78	1.28
		20	1.37	0.00	0.00	1.36	2.73
		100	1.43	0.00	0.00	1.50	2.93
Guerilla	-	1	0.51 (MHW)	0.00	0.00	0.47	0.98
		20	1.37	0.00	0.00	1.03	2.40
		100	1.43	0.00	0.00	1.10	2.53
Barlings	East	1	0.51 (MHW)	0.00	0.00	0.41	0.92
		20	1.37	0.00	0.00	0.52	1.89
		100	1.43	0.00	0.00	0.59	2.02
	West	1	0.51 (MHW)	0.00	0.00	0.53	1.04
		20	1.37	0.00	0.00	0.77	2.14
		100	1.43	0.00	0.00	0.79	2.22
Tomakin	-	1	0.51 (MHW)	0.00	0.00	0.53	1.04
		20	1.37	0.00	0.00	0.52	1.90
		100	1.43	0.00	0.00	0.53	1.97
Broulee	North	1	0.51 (MHW)	0.00	0.00	0.46	0.97
		20	1.37	0.00	0.00	0.56	1.93
		100	1.43	0.00	0.00	0.77	2.20
	Central	1	0.51 (MHW)	0.00	0.00	0.31	0.82
		20	1.37	0.00	0.00	0.41	1.79
		100	1.43	0.00	0.00	0.45	1.89
	South	1	0.51 (MHW)	0.00	0.00	0.26	0.76
		20	1.37	0.00	0.00	0.33	1.70
		100	1.43	0.00	0.00	0.29	1.73

Table 8-3: Static Inundation Levels for All Planning Periods

Beach	Profile	ARI (years)	Planning Period			
			2017	2050	2065	2100
			Total Static Water Level (m)			
Durras	North	1	1.29	1.51	1.62	2.00
		20	2.44	2.66	2.77	3.15
		100	2.55	2.77	2.88	3.26
	Central	1	1.48	1.70	1.81	2.19
		20	2.72	2.94	3.06	3.43
		100	2.89	3.11	3.22	3.60
	South	1	1.60	1.81	1.93	2.30
		20	2.87	3.09	3.20	3.58
		100	2.96	3.18	3.29	3.67
Cookies	-	1	1.17	1.39	1.51	1.88
		20	2.24	2.46	2.57	2.95
		100	2.34	2.56	2.68	3.05
Maloneys	East	1	0.71	0.93	1.04	1.42
		20	1.84	2.06	2.18	2.55
		100	2.01	2.23	2.35	2.72
	West	1	0.85	1.07	1.19	1.56
		20	2.03	2.25	2.37	2.74
		100	2.13	2.35	2.47	2.84
Long	East	1	0.93	1.15	1.26	1.64
		20	2.01	2.23	2.35	2.72
		100	2.14	2.36	2.48	2.85
	Central	1	0.92	1.14	1.25	1.63
		20	2.18	2.40	2.52	2.89
		100	2.31	2.53	2.65	3.02
	West	1	0.94	1.16	1.28	1.65
		20	2.15	2.37	2.49	2.86
		100	2.28	2.50	2.62	2.99
Cullendulla	-	1	0.73	0.95	1.06	1.44
		20	2.02	2.24	2.35	2.73
		100	2.15	2.37	2.48	2.86
Surfside E	North	1	1.12	1.34	1.45	1.83
		20	2.32	2.54	2.66	3.03
		100	2.33	2.55	2.67	3.04
	South	1	1.08	1.30	1.41	1.79
		20	2.36	2.58	2.70	3.07
		100	2.36	2.58	2.70	3.07

Table 8-3: Static Inundation Levels for All Planning Periods (contd.)

Beach	Profile	ARI (years)	Planning Period			
			2017	2050	2065	2100
			Total Static Water Level (m)			
Surfside W	-	1	0.90	1.11	1.23	1.60
		20	1.98	2.20	2.31	2.69
		100	2.10	2.32	2.43	2.81
Wharf Rd	-	1	0.88	1.10	1.22	1.59
		20	2.00	2.06	2.17	2.55
		100	2.14	2.12	2.24	2.61
CBD	West	1	1.14	1.35	1.47	1.84
		20	1.97	2.19	2.30	2.68
		100	2.13	2.34	2.46	2.83
	Central	1	1.11	1.33	1.44	1.82
		20	1.91	2.12	2.24	2.61
		100	2.04	2.25	2.37	2.74
	East	1	1.09	1.31	1.42	1.80
		20	2.08	2.30	2.41	2.79
		100	2.22	2.44	2.55	2.93
Boat Harbour	-	1	1.24	1.46	1.57	1.95
		20	2.10	2.32	2.43	2.81
		100	2.23	2.44	2.56	2.93
Corrigans	North	1	0.88	1.10	1.21	1.59
		20	2.11	2.33	2.44	2.82
		100	2.23	2.45	2.57	2.94
	South	1	0.80	1.02	1.14	1.51
		20	1.72	1.94	2.06	2.43
		100	1.82	2.03	2.15	2.52
Caseys	North	1	1.04	1.26	1.38	1.75
		20	2.06	2.28	2.39	2.77
		100	2.10	2.32	2.43	2.81
	Central	1	0.81	1.03	1.14	1.52
		20	1.61	1.83	1.94	2.32
		100	1.70	1.92	2.03	2.41
	South	1	0.75	0.97	1.08	1.46
		20	1.74	1.96	2.07	2.45
		100	1.83	2.05	2.17	2.54

Table 8-3: Static Inundation Levels for All Planning Periods (contd.)

Beach	Profile	ARI (years)	Planning Period			
			2017	2050	2065	2100
			Total Static Water Level (m)			
Sunshine	-	1	1.17	1.39	1.50	1.88
		20	2.45	2.67	2.79	3.16
		100	2.50	2.72	2.84	3.21
Malua	-	1	1.28	1.50	1.62	1.99
		20	2.73	2.95	3.07	3.44
		100	2.93	3.15	3.27	3.64
Guerilla	-	1	0.98	1.20	1.31	1.69
		20	2.40	2.62	2.73	3.11
		100	2.53	2.75	2.86	3.24
Barlings	East	1	1.04	1.26	1.37	1.75
		20	2.14	2.36	2.47	2.85
		100	2.22	2.44	2.55	2.93
	West	1	0.92	1.14	1.25	1.63
		20	1.89	2.11	2.23	2.60
		100	2.02	2.24	2.36	2.73
Tomakin	-	1	1.04	1.26	1.37	1.75
		20	1.90	2.12	2.23	2.61
		100	1.97	2.19	2.30	2.68
Broulee	North	1	0.97	1.19	1.30	1.68
		20	1.93	2.15	2.27	2.64
		100	2.20	2.42	2.54	2.91
	Central	1	0.82	1.04	1.15	1.53
		20	1.79	2.01	2.12	2.50
		100	1.89	2.10	2.22	2.59
	South	1	0.76	0.98	1.10	1.47
		20	1.70	1.92	2.04	2.41
		100	1.73	1.94	2.06	2.43

8.5 Wave Runup and Bore Propagation

8.5.1 Wave Runup on Sandy Beaches

Wave runup is the maximum elevation water reaches on a slope due to wave action. Shand et al. (2011) evaluated a number of empirical equations that have been developed to measure wave runup on beaches, and found that the laboratory based equations developed by Mase (1989) provided the most accurate estimation. Mase (1989) developed Equation 8.2 based on laboratory experiments for irregular waves on impermeable beaches with a slope of 1V:5H to 1V:30H.

$$R_{2\%} = 1.86 \xi_0^{0.71} H_0 \tag{8.2}$$

Where H_0 = deepwater significant wave height (m)
L_0 = deepwater wave length (m)
tan α= beach slope
$R_{2\%}$ = wave runup level exceeded by 2% of waves above the storm tide level (wave setup excluded(m)
ξ_0 =deepwater Iribarren number, calculated as $\xi_0 = \frac{\tan\alpha}{\sqrt{H_0 / L_0}}$

This methodology was utilised at all profiles where there is no seawall present to develop a 2% wave runup level, reference to AHD. H_0 was considered equivalent to the H_s at the outer edge of the surf zone extracted from SWAN. L_0 was based on the peak wave period at the same location. Beach slope was estimated between the location of wave breaking and the ultimate wave runup height.

8.5.2 Wave Runup on Seawalls

Mase (1989) is not valid on seawalls, so a different methodology was pursued where there are seawalls present (CBD, Boat Harbour, Wharf Road, Caseys Beach and Corrigans Beach). The method was not applied where very short seawalls are present (Long Beach and Malua Bay). The state-of-the-art empirical technique for estimating overtopping is the EurOtop (2016) "Overtopping Manual", shown in Equations 8.3 – 8.6.

$$R_{2\%} = 1.65 \gamma_b \gamma_f \gamma_\beta \xi_{m-1,0} H_{m0} \tag{8.3}$$

With a maximum of:

$$R_{2\%} = 1.00 \gamma_{f,surging} \gamma_\beta \left(4 - \frac{1.5}{\sqrt{\gamma_b \xi_{m-1,0}}}\right) H_{m0} \tag{8.4}$$

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{m0} / L_{m0}}} \tag{8.5}$$

$$\gamma_{f,surging} = \gamma_f + (\xi_{m-1,0} - 1.8)(1 - \gamma_f)/8.2 \tag{8.6}$$

Where H_{m0} = spectral significant wave height at the toe of the structure (m)
$\xi_{m-1,0}$=spectral deepwater Iribarren number
γ_b = berm influence factor (1 if no or unknown berm present)
γ_f = roughness factor (0.55 for a double layer of rock armour)
γ_β = obliqueness factor (1 for waves perpendicular to the wall)
L_{m0} = spectral wave length (m)
tan α= structure slope
$R_{2\%}$ = wave runup level exceeded by 2% of the waves (m)

At most of the structures, the wave height at the structure will be depth limited in a storm event – that is the maximum wave height that can impact the structure is dependent on the depth of the water at, or just offshore of (plunge length), the toe. An empirical technique for estimating the breaker depth index (H_s/d_b) was derived from laboratory experiments by Goda (2007) on slopes between 1V:9H and horizontal, and was used for this study. From the significant wave height and peak period, two spectral wave parameters, spectral significant wave height (H_{m0}) at the structure and nearshore spectral mean energy wave period ($T_{m-1,0}$) were also calculated according to Equation 8.7 (USACE, 2006) and Equation 8.8 (USACE, 2006), respectively.

$$H_{m0} = \frac{H_s}{0.9} \tag{8.7}$$

$$T_{m-1,0} = \frac{T_p}{1.1} \tag{8.8}$$

An important input to the EurOtop (2016) runup calculation is the slope of the seawall. Some of the seawalls around the Batemans Bay region have been built without proper design or engineering guidance and the slope of the walls may be variable and not well documented. Where it was possible, WRL approximated the structure slopes from measurements during site visits if no design was available. The assumptions made about seawall slope are summarised below in Table 8-4.

Table 8-4: Summary of Adopted Seawall Slopes

Location	Slope
Wharf Road	1V:1.5H
CBD West (to 1 Clyde Street)	1V:1.2H
CBD Central 1 (1 Clyde St to the end of Mara Mia Walkway)	1V:2H
CBD Central 2 (end of Mara Mia Walkway to 8 Beach Road)	1V:1H
CBD East (8 Beach Road to 25 Beach Road)	1V:1.2H
Boat Harbour	1V:2H
Caseys Beach (North)	1V:1.2H
Caseys Beach (South)	1V:4H

8.5.3 Bore Propagation

If the wave runup level does not exceed the crest of the dune or seawall, mapping the extent of wave runup is a simple exercise. However, if the runup level exceeds the crest, the wave will propagate inland to a certain extent until gravitational and frictional forces prevent further landward attenuation. The landward propagation of the bore is dependent on the runup elevation, the crest elevation and the backshore slope, shown in Figure 8-7. Bore propagation distance has been calculated based on Equation 8.9, modified from FEMA (2005).

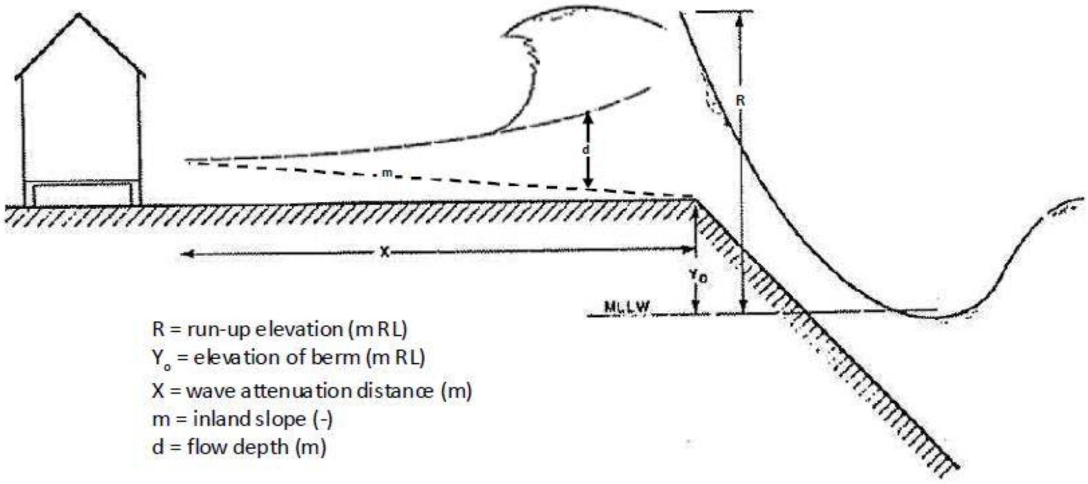


Figure 8-7: Bore propagation (Source: Tonkin & Taylor, 2016b and Cox and Machemehl, 1986)

$$X = \frac{\sqrt{R - Y_0}\, A(1-2m) g \sqrt{T}}{5} \qquad (8.9)$$

Where X = 2% bore propagation distance landward from crest (m)
R = 2% wave runup level (m AHD)
Y_0 = crest level (m AHD)
T = peak wave period (s)
g = 9.81 m/s^2
A = inland slope factor (default as 1)
m = positive upward inland slope valid for -0.5 < m < 0.25

8.5.4 Methodology for Mapping Wave Runup

For the "quasi-static" water levels, a "bathtub" method was employed to map the extent of inundation inland. However, due to the short temporal and dynamic nature of wave runup, this is not appropriate. Therefore the following methodology has been used to map the wave runup:

1. Wave runup levels was calculated at each of the 35 profiles described in Section 8.5, using the Mase (1989) method for sandy beach profiles or the EurOtop (2016) method for seawall profiles;
2. These runup levels were applied at photogrammetry profiles (or LIDAR where photogrammetry profiles were not available) at a profile spacing of 10 – 50 m. At each of these profiles:
    a. The crest level and position is extracted;
    b. If the "quasi-static" level exceeds the crest level, the backshore area is considered totally inundated and wave runup was not assessed;
    c. If the crest level exceeds the wave runup level, the extent of the wave runup was estimated based on elevation only;
    d. If the runup level exceeds the crest level, Equation 8.9 was used to estimate the propagation distance exceeded by 2% of wave bores, and the wave runup extent was established using this offset distance from the crest position.

8.5.5 Calibration at Caseys Beach

During a site inspection at Caseys Beach on 15 June 2016, WRL observed and surveyed the debris line after the East Coast Low event of June 5^{th}-6^{th} 2016 which caused significant overtopping of the northern section of seawall at Caseys Beach. Photos of the overtopping captured at the intersection of Batehaven and Beach Roads are shown in Figure 8-8). The most severe overtopping was within the area extending from approximately 50 m to the north of John Street to 140 m to the south of John Street, where overtopping wash completely crossed Beach Road and progressed into the front yards of the private properties in this area (Figure 8-9). Runup debris lines were surveyed by WRL in the front yard of the property at 382 Beach Road (Figure 8-10) and the observed approximate runup extent at 378 Beach Road was also surveyed during the seawall inspection for this project. It is difficult to precisely quantify the average recurrence interval of that runup and overtopping event, but WRL estimates it to be in the order of 15 – 25 years based on historical inundation damage (Blacka and Coghlan, 2016). This runup extent has therefore been used to calibrate the bore propagation methodology at this site, by adjusting the inland slope factor (A) in Equation 1.11 (Figure 8-11).

Figure 8-8: Overtopping at Intersection of Batehaven and Beach Road, 6/6/2016 10:00 pm (Source: Facebook)

Figure 8-9: Post June 2016 Storm Damage to South of John Street (ESC, 2016)

Figure 8-10: Runup Debris Line surveyed by WRL in the front yard of 382 Beach Road (ESC, 2016)

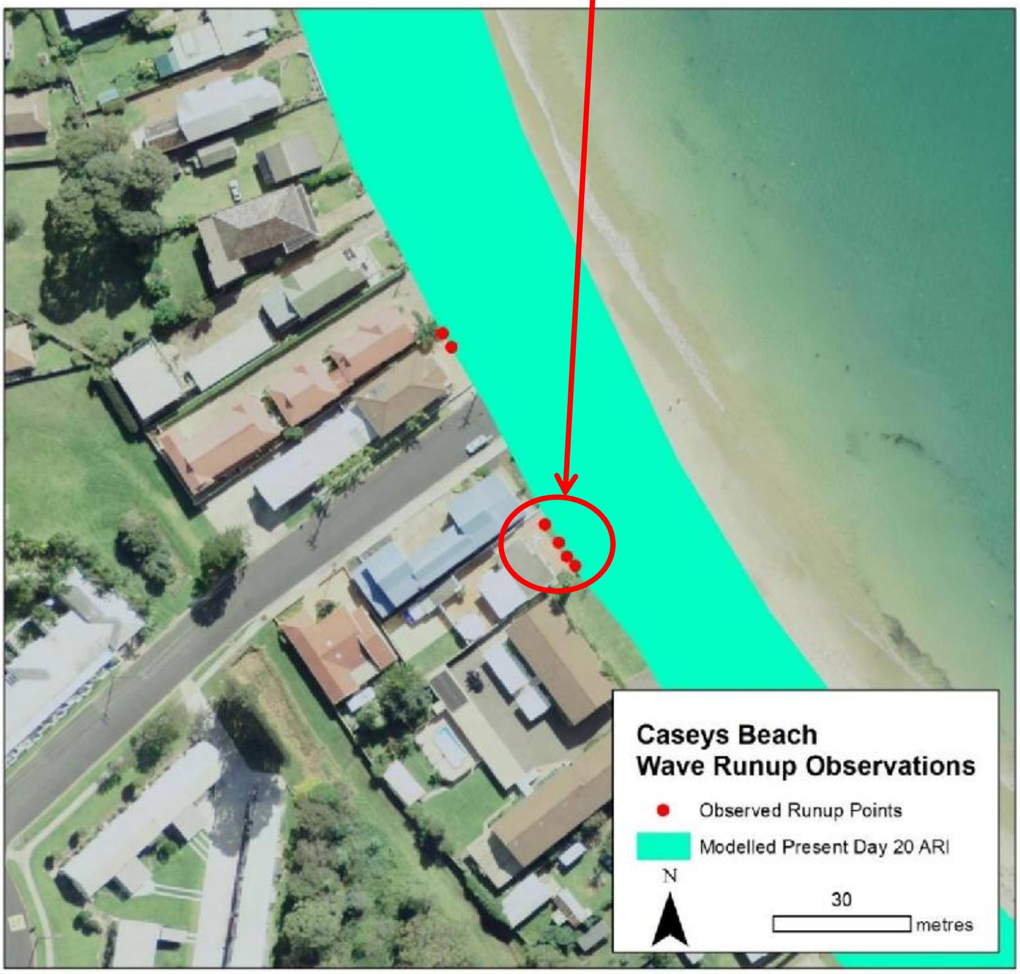


Figure 8-11: Calibration of bore propagation methodology for Caseys Beach

By adjusting the inland slope factor to 1.5, the modelled 20 year ARI wave runup extent agreed well with the observed runup extent. Therefore, an inland slope factor of 1.5 has been adopted at Caseys Beach behind the seawalls. Without observed runup at other locations, the inland slope factor was kept as 1.

8.6 Summary of Dynamic Wave Runup Levels and Wave Bore Propagation Distances

Table 8-5 summarises the wave runup levels under present day (2017) conditions, and the resulting bore propagation distances. Note that if the wave runup does not exceed the dune crest, bore propagation distance was not calculated.

Table 8-6 summarises the wave runup levels only for all planning periods. Note that for future planning periods, the runup elevations for sandy beach sections have been increased by the same value as projected sea level rise (relative to the 2017 mean sea level). However, for seawall sections, runup was calculated for each future planning period using the depth limited wave height at the toe of the structure (which increases in height with projected sea level rise).

Table 8-5: Summary of Wave Runup Levels, Resulting Bore Propagation for Present Day Conditions

Beach	Profile	ARI (year)	Wave Runup (m AHD)	Dune/Seawall Crest Elevation Range (m AHD)	Bore Propagation Range (m)
Durras	North	1	3.4	3.9 – 9.0	N/A
		20	5.0	3.9 – 9.0	2.6 - 8.4
		100	5.5	3.9 – 9.0	6.2 - 11.1
	Central	1	3.4	7.7 - 8.9	N/A
		20	5.1	7.7 - 8.9	N/A
		100	5.6	7.7 - 8.9	N/A
	South	1	3.0	6.8 - 8.2	N/A
		20	4.7	6.8 - 8.2	N/A
		100	5.2	6.8 - 8.2	N/A
Cookies	-	1	3.2	2.9 - 13.1	1.5 – 4.0
		20	4.9	2.9 - 13.1	8.45 - 10.8
		100	5.3	2.9 - 13.1	10.8 - 13.1
Maloneys	East	1	2.7	2.0 - 15.2	1.8 - 6.1
		20	5.9	2.0 - 15.2	5.0 - 16.3
		100	6.3	2.0 - 15.2	7.6 - 17.1
	West	1	3.5	3.2 - 5.7	3.8 - 4.0
		20	6.2	3.2 - 5.7	5.5 - 13.2
		100	6.7	3.2 - 5.7	8.6 - 15.7
Long	East	1	2.6	2.1 - 3.3	0.6 - 4.4
		20	4.5	2.1 - 3.3	8.3 - 10.8
		100	4.9	2.1 - 3.3	10.6 - 13.0
	Central	1	2.9	3.9 - 5.7	N/A
		20	4.8	3.9 - 5.7	0.8 - 7.1
		100	5.3	3.9 - 5.7	1.5 - 9.6
	West	1	3.1	2.9 - 11.0	1.5 - 1.5
		20	5.1	2.9 - 11.0	1.7 - 7.9
		100	5.6	2.9 - 11.0	3.6 - 10.6

Table 8-5: Summary of Wave Runup Levels, Resulting Bore Propagation for Present Day Conditions (contd.)

Beach	Profile	ARI (year)	Wave Runup (m AHD)	Dune/Seawall Crest Elevation Range (m AHD)	Bore Propagation Range (m)
Cullendulla	-	1	2.2	1.4 - 2.0	2.6 - 6.4
		20	3.6	1.4 - 2.0	9.8 - 11.5
		100	4.0	1.4 - 2.0	12.1 - 13.7
Surfside E	North	1	1.8	2.4 - 3.4	N/A
		20	4.0	2.4 - 3.4	6.0 - 9.8
		100	4.6	2.4 - 3.4	9.5 - 12.7
	South	1	2.0	2.4 - 3.8	N/A
		20	4.3	2.4 - 3.8	5.6 - 10.9
		100	4.7	2.4 - 3.8	8.4 - 13.2
Surfside W	-	1	0.8	1.6 - 10.1	N/A
		20	2.5	1.6 - 10.1	4.5 - 7.5
		100	2.7	1.6 - 10.1	6.3 - 9.1
Wharf Rd	(Dune)	1	2.1	1.1 - 12.4	3.5 - 5.4
		20	2.8	1.2 - 12.4	6.1 - 8.3
		100	3.0	1.2 - 12.4	7.3 - 9.9
Wharf Rd	(Seawall)	1	2.5	1.7 - 2.4	1.6 - 6.3
		20	4.9	1.7 - 2.4	12.4 - 14.0
		100	5.2	1.7 - 2.4	14.3 – 16.0
CBD	West	1	2.1	1.8 - 2.2	3.3 - 4.5
		20	4.5	1.8 - 2.2	12.4 - 13.6
		100	4.8	1.8 - 2.2	14.3 - 15.6
	Central	1	2.6	1.7 - 2.4	3.7 – 7.0
		20	4.7	1.7 - 2.4	12.3 - 13.9
		100	5.0	1.7 - 2.4	14.5 - 16.3
	East	1	2.6	1.4 - 2.6	1.8 - 7.4
		20	4.6	1.5 - 2.6	11.4 - 13.9
		100	5.0	1.5 - 2.6	13.6 - 16.3
Boat Harbour	-	1	4.3	1.2 - 1.6	11.0 - 12.3
		20	6.2	1.2 - 1.6	15.8 - 17.6
		100	6.7	1.2 - 1.6	18.2 - 20.3
Corrigans	North	1	3.2	1.1 - 3.2	1.4 - 9.7
		20	5.0	1.1 - 3.2	10.4 - 14.9
		100	5.4	1.1 - 3.2	12.7 - 17.2
	South	1	2.0	1.8 - 13.2	1.6 - 3.4
		20	2.8	1.8 - 13.2	2.6 - 7.9
		100	3.0	1.8 - 13.2	2.7 - 9.5

Table 8-5: Summary of Wave Runup Levels, Resulting Bore Propagation for Present Day Conditions (contd.)

Beach	Profile	ARI (year)	Wave Runup (m AHD)	Dune/Seawall Crest Elevation Range (m AHD)	Bore Propagation Range (m)
Caseys	North	1	3.6	3.3 – 19.0	0.8 - 3.7
		20	4.4	3.2 – 19.0	1.4 - 6.9
		100	5.0	3.3 – 19.0	5.6 - 9.4
	Central	1	2.9	3.2 – 4.0	N/A
		20	4.7	3.2 – 4.0	3.96 - 8.18
		100	4.9	3.2 – 4.0	4.9 – 9.0
	South	1	1.8	2.6 - 3.2	N/A
		20	3.8	2.6 - 3.2	9.3 - 11.2
		100	4.1	2.6 - 3.2	10.9 - 12.9
Sunshine	-	1	3.4	3.6 - 26.2	N/A
		20	4.9	3.6 - 26.3	9.3 - 9.3
		100	5.3	3.6 - 26.3	4.4 - 11.7
Malua	-	1	3.0	2.2 - 5.8	2.5 - 6.4
		20	5.2	2.2 - 5.8	2.3 - 13.4
		100	5.9	2.2 - 5.8	2.8 - 16.4
Guerilla	-	1	3.5	3.0 - 22.7	2.9 - 4.4
		20	5.4	3.0 - 22.7	10.3 - 11.1
		100	6.0	3.0 - 22.7	13.2 - 13.9
Barlings	East	1	2.5	3.4 - 6.7	N/A
		20	4.3	3.4 - 6.7	6.5 - 6.7
		100	4.9	3.4 - 6.7	5.1 - 9.9
	West	1	3.1	5.2 - 13.5	N/A
		20	4.6	5.2 - 13.5	N/A
		100	5	5.2 - 13.5	N/A
Tomakin	-	1	2.9	3.8 - 7.6	N/A
		20	4.2	3.8 - 7.6	3.4 - 4.6
		100	4.6	3.8 - 7.6	1.8 - 7.3
Broulee	North	1	2.4	3.7 - 8.3	N/A
		20	3.9	3.7 - 8.3	3.7 - 3.7
		100	4.2	3.7 - 8.3	6.1 - 6.1
	Central	1	2.0	5.3 - 7.4	N/A
		20	3.5	5.3 - 7.4	N/A
		100	3.9	5.3 - 7.4	N/A
	South	1	2.0	1.9 - 9.0	2.1 - 2.1
		20	3.7	1.9 - 9.0	3.7 - 10.4
		100	3.8	1.9 - 9.0	1.7 - 11.8

Table 8-6: Wave Runup Levels for All Planning Periods

Beach	Profile	ARI (years)	Planning Period 2017	2050	2065	2100
			Wave Runup Level (m AHD)			
Durras	North	1	3.4	3.6	3.7	4.1
		20	5.0	5.2	5.3	5.7
		100	5.5	5.7	5.8	6.2
	Central	1	3.4	3.6	3.7	4.1
		20	5.1	5.3	5.4	5.8
		100	5.6	5.8	5.9	6.3
	South	1	3.0	3.2	3.3	3.7
		20	4.7	4.9	5.0	5.4
		100	5.2	5.4	5.5	5.9
Cookies	-	1	3.2	3.4	3.5	3.9
		20	4.9	5.1	5.2	5.6
		100	5.3	5.5	5.6	6.0
Maloneys	East	1	2.7	2.9	3.0	3.4
		20	5.9	6.1	6.2	6.6
		100	6.3	6.5	6.6	7.0
	West	1	3.5	3.7	3.8	4.2
		20	6.2	6.4	6.5	6.9
		100	6.7	6.9	7.0	7.4
Long	East	1	2.6	2.8	2.9	3.3
		20	4.5	4.7	4.8	5.2
		100	4.9	5.1	5.2	5.6
	Central	1	2.9	3.1	3.2	3.6
		20	4.8	5.0	5.1	5.5
		100	5.3	5.5	5.6	6.0
	West	1	3.1	3.3	3.4	3.8
		20	5.1	5.3	5.4	5.8
		100	5.6	5.8	5.9	6.3
Cullendulla	-	1	2.2	2.4	2.5	2.9
		20	3.6	3.8	3.9	4.3
		100	4.0	4.2	4.3	4.7
Surfside E	North	1	1.8	2.0	2.1	2.5
		20	4.0	4.2	4.3	4.7
		100	4.6	4.8	4.9	5.3
	South	1	2.0	2.2	2.3	2.7
		20	4.3	4.5	4.6	5.0
		100	4.7	4.9	5.0	5.4

Table 8-6: Wave Runup Levels for All Planning Periods (contd.)

Beach	Profile	ARI (years)	Planning Period 2017	2050	2065	2100
			Wave Runup Level (m AHD)			
Surfside W	-	1	0.8	1.0	1.1	1.5
		20	2.5	2.7	2.8	3.2
		100	2.7	2.9	3.0	3.4
Wharf Rd	(Dune)	1	2.1	2.3	2.4	2.8
		20	2.8	3.0	3.1	3.5
		100	3.0	3.2	3.3	3.7
Wharf Rd	(Seawall)	1	2.5	2.7	3.2	4.3
		20	4.9	5.2	5.3	5.6
		100	5.2	5.4	5.5	5.9
CBD	West	1	2.1	2.3	2.5	2.8
		20	4.5	4.8	4.9	5.2
		100	4.8	5.0	5.1	5.5
	Central	1	2.6	2.8	2.9	3.3
		20	4.7	4.9	5.0	5.4
		100	5.0	5.2	5.4	5.7
	East	1	2.6	2.8	2.9	3.3
		20	4.6	4.8	5.0	5.3
		100	5.0	5.2	5.3	5.7
Boat Harbour	-	1	4.3	4.5	4.6	5.0
		20	6.2	6.4	6.6	6.9
		100	6.7	6.9	7.0	7.4
Corrigans	North	1	3.2	3.4	3.5	3.9
		20	5.0	5.2	5.3	5.7
		100	5.4	5.6	5.7	6.1
	South	1	2.0	2.2	2.3	2.7
		20	2.8	3.0	3.1	3.5
		100	3.0	3.2	3.3	3.7
Caseys	North	1	3.6	3.8	3.9	4.3
		20	4.4	4.6	4.7	5.1
		100	5.0	5.2	5.3	5.7
	Central	1	2.2	3.0	3.2	4.2
		20	4.7	5.5	5.7	6.8
		100	4.9	5.6	5.9	7.0
	South	1	1.8	2.3	2.5	3.4
		20	3.8	4.3	4.5	5.3
		100	4.1	4.5	4.7	5.6

Table 8-6: Wave Runup Levels for All Planning Periods (contd.)

Beach	Profile	ARI (years	Planning Period			
			2017	2050	2065	2100
			Wave Runup Level (m AHD)			
Sunshine	-	1	3.4	3.6	3.7	4.1
		20	4.9	5.1	5.2	5.6
		100	5.3	5.5	5.6	6.0
Malua	-	1	3.0	3.2	3.3	3.7
		20	5.2	5.4	5.5	5.9
		100	5.9	6.1	6.2	6.6
Guerilla	-	1	3.5	3.7	3.8	4.2
		20	5.4	5.6	5.7	6.1
		100	6.0	6.2	6.3	6.7
Barlings	East	1	2.5	2.7	2.8	3.2
		20	4.3	4.5	4.6	5.0
		100	4.9	5.1	5.2	5.6
	West	1	3.1	3.3	3.4	3.8
		20	4.6	4.8	4.9	5.3
		100	5	5.2	5.3	5.7
Tomakin	-	1	2.9	3.1	3.2	3.6
		20	4.2	4.4	4.5	4.9
		100	4.6	4.8	4.9	5.3
Broulee	North	1	2.4	2.6	2.7	3.1
		20	3.9	4.1	4.2	4.6
		100	4.2	4.4	4.5	4.9
	Central	1	2.0	2.2	2.3	2.7
		20	3.5	3.7	3.8	4.2
		100	3.9	4.1	4.2	4.6
	South	1	2.0	2.2	2.3	2.7
		20	3.7	3.9	4.0	4.4
		100	3.8	4.0	4.1	4.5

8.7 Comparison with Observations and Previous Studies

8.7.1 Static Water Levels

Two previous studies (NSW PWD, 1989 and DLWC, 1996) have comprehensively examined coastal inundation around Batemans Bay. Each included an allowance for uncertainty in their "quasi-static" inundation levels (0.3 and 0.2 m, respectively). While inundation levels are unavailable for the 1 year ARI event, the levels calculated by WRL are compared with those from the previous studies for the 20 and 100 year ARI events in Table 8-7. A third study (WMA, 2006) also examined coastal inundation for the 100 year ARI event only. This study did not include an allowance for uncertainty in its inundation levels which are also shown in Table 8-7.

Table 8-7: Comparison of "Quasi-static" Coastal Inundation Levels Estimated by WRL and Previous Reports

Beach	Section	Ref #	Present 20 Year ARI (5% AEP) Inundation Level (m AHD)					Present 100 Year ARI (1% AEP) Inundation Level (m AHD)					
			WRL (2017)	NSW PWD (1989)		DLWC (1996)		WRL (2017)	NSW PWD (1989)		DLWC (1996)		WMA (2006)
				without uncertainty allowance	with 0.3 m allowance	without uncertainty allowance	with 0.2 m uncertainty allowance		without uncertainty allowance	with 0.3 m uncertainty allowance	without uncertainty allowance	with 0.2 m uncertainty allowance	
Maloneys	East	17	1.8	2.4	2.7	2.5	2.7	2.0	2.6	2.9	2.7	2.9	2.7
	West	16	2.0	2.5	2.8	2.6	2.8	2.1	2.7	3	2.8	3	2.7
Long	East	15	2.0	2.3	2.6	2.2	2.4	2.1	2.5	2.8	2.4	2.6	2.5
	Central	14	2.2	2.3	2.6	2.3	2.5	2.3	2.5	2.8	2.5	2.7	2.5
	West	13	2.2	2.3	2.6	2.3	2.5	2.3	2.5	2.8	2.5	2.7	2.5
Cullendulla	Central	12	2.0	1.5	1.8	1.6	1.8	2.2	1.7	2	1.8	2	1.8
Surfside (East)	North	11	2.3	2.2	2.5	2.4	2.6	2.3	2.4	2.7	2.6	2.8	2.6
	South	10	2.4	2.2	2.5	2.4	2.6	2.4	2.4	2.7	2.6	2.8	2.6
Wharf Road	Central	9	2.0	2.2	2.5	2.2	2.4	2.1	2.4	2.7	2.3	2.5	1.6
Central Business District	Central	8	2.0	2.2	2.5	2.1	2.3	2.1	2.5	2.8	2.3	2.5	1.8
	East	7	1.9	2.2	2.5	2.1	2.3	2.0	2.3	2.6	2.2	2.4	1.8
Boat Harbour	Central	6	2.1	1.8	2.1	1.7	1.9	2.2	2.2	2.5	2	2.2	1.8
Corrigans	North	5	2.1	2.1	2.4	1.9	2.1	2.2	2.3	2.6	2.1	2.3	2.3
	South	4	1.7	2	2.3	1.9	2.1	1.8	2.2	2.5	2	2.2	2.3
Caseys	North	3	2.0	2.2	2.5	2.1	2.3	2.1	2.4	2.7	2.2	2.4	2.4
	Central	2	1.6	2.2	2.5	2.1	2.3	1.7	2.3	2.6	2.2	2.4	2.4
	South	1	1.7	2.3	2.6	2.2	2.4	1.8	2.4	2.7	2.3	2.5	2.4

The values adopted in this study are of a similar magnitude to those found previously. In general, the modelling undertaken in this study has found slightly lower levels than the previous studies undertaken over the last 30 years. While the model framework for the NSW PWD (1989) study was different to those used in WRL's study, the key difference is considered to be that the 100 year ARI significant wave height adopted for the older study was 10.6 m based on 14 years of wave buoy data collected at Botany Bay. Recall that the 100 year ARI wave height adopted in WRL's study is lower (7.7 m) based on 24 years of wave buoy data collected at Batemans Bay. The NSW PWD (1989) framework and its 100 year ARI wave height were also adopted for the DLWC (1996) study even though 10 years of wave buoy data was then available at Batemans Bay. As a result, WRL estimates that the level of "quasi-static" coastal inundation risk in Batemans Bay is slightly less than previously reported primarily due to improved knowledge of the wave climate acquired through ongoing data collection.

WMA (2006) did not comprehensively detail the methodology or input values used to determine the inundation levels they reported. As such, it is not possible to comment on the differences between inundation levels estimated by WRL and WMA (2006).

8.7.2 Wave Runup Levels

Higgs and Nittim (1988) undertook a comprehensive study on the wave runup at beaches in Batemans Bay during storms on 4-9 August and 17-23 November 1986. A variety of oceanographic and meteorological data was collected with wave buoys (offshore of Batemans Bay), tide gauges (Snapper Island and Princess Jetty) and an anemometer (Moruya Heads). The August storm had a peak H_S of 5.6 m and typical T_P of 10-13.5 s. Local winds were from the SSW-SSE. The maximum water level recorded at the Snapper Island tide gauge was 0.86 m AHD. The November storm had a peak H_S of 6.0 m and typical T_P of 10-13.5 s. Local winds were from the S-SW. The maximum water level recorded at the Snapper Island tide gauge was 1.02 m AHD.

The location and elevation of maximum runup were pegged and surveyed after both storm events and are shown in Table 8-8.

Table 8-8: Runup Levels during Storms in 1986

Site	**Maximum Runup Elevation** (m AHD) 4-9 August	**Maximum Runup Elevation** (m AHD) 17-23 November
Maloneys Beach	1.9-2.2	2.2-3.7
Long Beach	2.7	2.1-3.7
Cullendulla Beach	-	1.4-1.8
Surfside Beach	-	2.3-2.8
Wharf Road	2.0	1.5-1.7
Central Business District	-	1.4
Boat Harbour West	-	1.5
Boat Harbour East	-	1.4
Corrigans Beach	2.2-2.8	2.2-2.3
Caseys Beach	-	2.5-3.2
Malua Bay	5.5	-

A verification case at Malua Bay was run to assess the appropriateness of the Mase (1989) method for estimating wave runup for the August 1986 storm. The following comparison is available from this event:

- Observed debris line: 5.5 m AHD
- Calculated Mase R_{max}: 5.5 m AHD
- Calculated Mase $R_{2\%}$: 4.9 m AHD

The predictions were in good agreement with the observed debris line, and the method is therefore considered appropriate for the wider study area.

8.7.3 Historical Coastal Inundation Photos

The extents of and damage from historical coastal inundation are mapped in great detail in NSW PWD (1989). A selection of key photos from this report are reproduced as Figure 8-12 through Figure 8-16.

Figure 8-12: Soldiers Club, Beach Road, CBD, 29-30 August 1963 (NSW PWD, 1989)

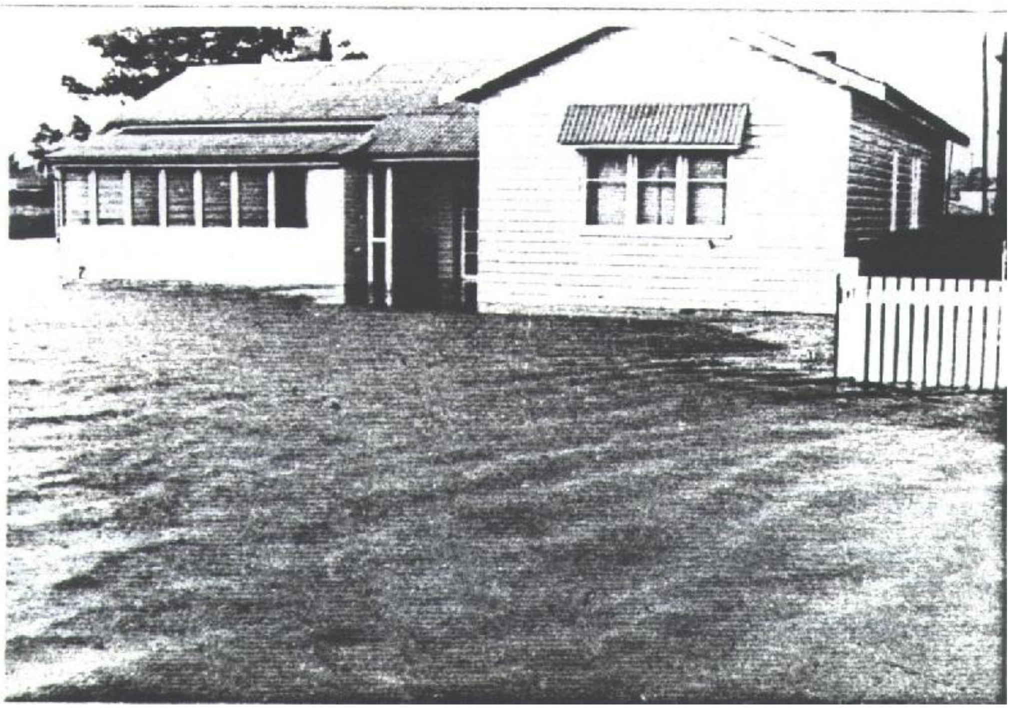


Figure 8-13: Corner of Bavarde Avenue and Golf Links Drive (Hanging Rock) 29-30 August 1963 (NSW PWD, 1989)

Figure 8-14: Mariners on the Waterfront, CBD, 1 July 1984 (NSW PWD, 1989)

Figure 8-15: Overtopping of Caseys Beach Seawall 1 July 1984 (NSW PWD, 1989)

Figure 8-16: Overtopping of Myamba Parade at Surfside Beach (west) 13 August 1986 (NSW PWD, 1989)

On 4-6 June 2012, a severe storm with offshore significant wave heights of 6 m (typical T_P = 13 s, south-easterly wave direction, maximum water level 1.3 m AHD) had a large impact upon beaches within Batemans Bay. A series of photos, collated by ESC, documenting the extent of coastal inundation are reproduced in Figure 8-17 through Figure 8-28.

Figure 8-17: Overtopping of Bay Road, Long Beach, 6 June 2012 (Mr Lindsay Usher)

Figure 8-18: Backshore Inundation at Cullendulla Beach, 6 June 2012 (Mr Lindsay Usher)

Figure 8-19: Inundation Debris Line at Surfside Beach (East), 6 June 2012 (Mr Lindsay Usher)

Figure 8-20: Inundation Debris Line at Surfside Beach (West), 6 June 2012 (Mr Lindsay Usher)

Figure 8-21: Overtopping of Myamba Parade at Surfside Beach (West), 6 June 2012 (Mr Dick Crompton)

Figure 8-22: Inundation at Wharf Road (1 of 3), 6 June 2012 (Mr Dick Crompton)

Figure 8-23: Inundation at Wharf Road (2 of 3), 6 June 2012 (Mr Dick Crompton)

Figure 8-24: Inundation at Wharf Road (3 of 3), 6 June 2012 (Mr Dick Crompton)

Figure 8-25: Inundation at CBD near Starfish Deli, 6 June 2012 (Mr Mark Swadling)

Figure 8-26: Inundation Damage to CBD Foreshore, 6 June 2012 (Mr Lindsay Usher)

Figure 8-27: Overtopping Extents at CBD, 7 June 2012 (Mr Norman Lenehan)

Figure 8-28: Backshore Inundation at Corrigans Beach, 6 June 2012 (Mr Dick Crompton)

9. Review of Additional Coastal Hazards

9.1 Windblown Sand

Site visits and analysis of aerial photos indicate that there are no substantial hazards due to windblown sand (aeolian drift) in the Eurobodalla study area. A quantity of windblown sand will reach the built environment during strong winds, but as all dunes are vegetated, this quantity is anticipated to be minor and mobile dunes are not expected to threaten the built environment. The exception is some beach access points, such as Malua Bay, where pedestrian traffic has removed vegetation, lowered the sand levels and has formed a potential dune breach point.

For a typical Eurobodalla median sand grain size of 0.19 mm to 0.40 mm, sand movement is initiated for the following velocities referenced to an anemometer elevation of 10 m (USACE, 2006):

- Dry sand ~6.4 to 9.2 m/s (~12 to 18 knots, 23 to 33 km/hour);
- Wet sand ~11.4 to 14.2 m/s (~22 to 28 knots, 41 to 51 km/hour).

Note that much higher wind speeds are required to mobilise wet sand compared to dry sand. Sand can become wet through waves and tide, or through precipitation. Therefore, reduced rainfall due to climate change has the potential to increase windblown sand volumes. The modelling of this is beyond the scope of this study.

Based on daily average wind data from the BoM meteorological station at Moruya Heads, consideration of the median grain size and the orientation of each beach, the monthly percent occurrence of dune building winds at each beach is presented in Figure 9-1 (dry sand) and Figure 9-2 (wet sand). Natural dune building can occur when the winds are close to perpendicular to the shoreline, directed onshore and exceed the threshold of motion. It can be seen that, in general, the potential for dune building is lowest in May, June, July and August. Broulee Beach has the highest overall dune building potential and Surfside Beach (West) has the least.

Note that a future adaptive response may require dunes to be raised, in which case detailed vegetation management plans and dune designs would need to be prepared. Works for dune reconstruction may need to involve detailed studies of aeolian mobilisation during the revegetation phase. Future climate change may alter the range of viable dune vegetation species.

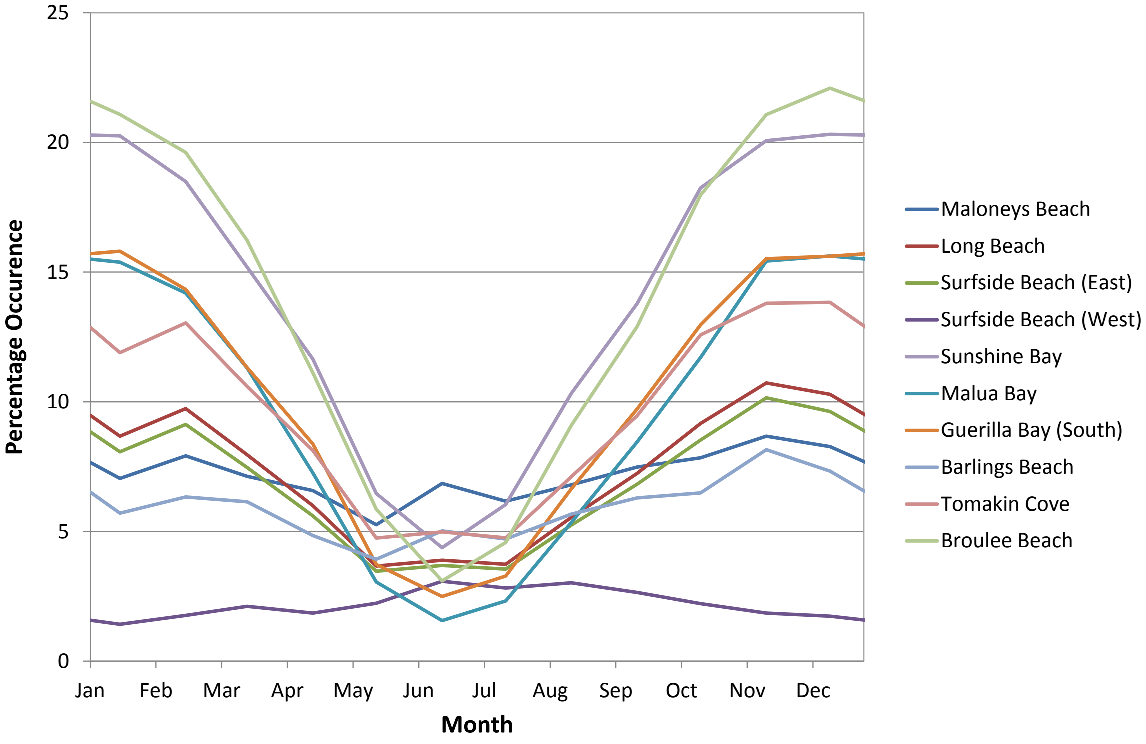


Figure 9-1: BoM Moruya Heads Pilot Station Daily Average Wind - Occurrence of Winds for Dune Building – Dry Sand

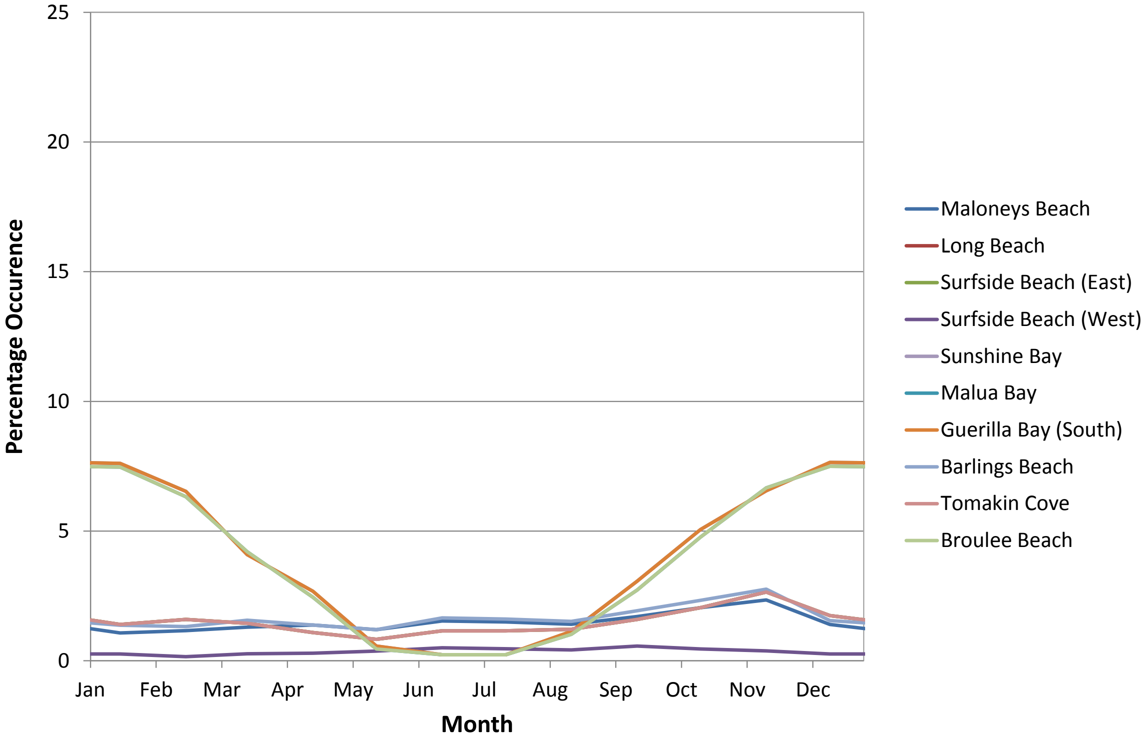


Figure 9-2: BoM Moruya Heads Pilot Station Daily Average Winds - Occurrence of Winds for Dune Building – Wet Sand

9.2 Stormwater Erosion

Stormwater erosion is a relatively minor hazard on the ten (10) beaches for which erosion/recession maps were prepared as there are no large conveyance structures discharging directly onto sandy beaches. There are mid-sized discharge structures at the northern and southern ends of Surfside Beach (East). However, the additional erosion resulting from these is minor and limited to within several metres of the structure as they are located on mainly rocky platforms.

The design of future stormwater outfalls needs to consider coastal processes, such as:

- The effect of elevated ocean water levels on the hydraulic performance of the system; and
- Local erosion caused by stormwater discharge and/or wave scour around the outfall.

Water quality from discharged stormwater is likely to be a hazard, but is beyond the scope of this study to consider this issue.

10. Assumptions and Limitations

10.1 Introduction

The methodology applied in this report for the Eurobodalla Coastal Hazard Assessment was developed in consultation with Eurobodalla Shire Council and the NSW Office of Environment and Heritage (NSW OEH), and considers the following documents:

- NSW Coastal Management Act (2016) ;
- *Draft* NSW Coastal Management Manual (OEH, 2016);
- Coastal Risk Management Guide (DECCW, 2010);
- ESC sea level rise policy and planning framework (ESC, 2014;Whitehead & Associates, 2014);
- NSW Coastline Management Manual (NSW Government, 1990).

The assumptions and limitations applicable to the analysis and the data used in this study are described below.

10.2 Site Inspections

A visual assessment of the dunes and seawalls allowed general and qualitative observations of the present seawall conditions. A detailed stability assessment was not part of the scope of works and a geotechnical investigation was not undertaken for this study. Representative crest levels and foreshore geometry were estimated by experienced coastal engineers, however, in some locations these levels vary along the dune or seawall.

10.3 Sea Level Rise

The sea level rise projections adopted in this investigation were based ESC's sea level rise policy and planning framework (ESC, 2014). No further reassessment of these benchmarks was undertaken by WRL. These locally adjusted sea level rise benchmarks are based on projections from the IPCC and actual sea level rise may be higher or lower than these benchmarks over the planning period. The IPCC reviews and revises sea level projections at generally 5-7 year intervals, with the most recent revision (Assessment Report 5) being in 2013/14, and Assessment Report 6 due in 2021/2022.

10.4 Water Levels and Wave Climate

For erosion modelling purposes, a Mean High Water Spring (MHWS) tide time series was assumed, to which a tidal anomaly was added, such that the peak water level corresponded to the 100 year ARI storm surge water level. For modelling purposes the peak in predicted tide and tidal anomaly was assumed to coincide with the peak wave height of the storm.

The nearshore wave climate around the beaches of Eurobodalla Shire was determined using a numerical wave propagation model (SWAN version 41.10). The model inputs were offshore boundary conditions and bathymetric data. Offshore boundary conditions relied on extreme wave and wind statistics analysis undertaken by WRL (Shand et al., 2011) for the Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI). Bathymetric data was obtained from NSW OEH, NSW RMS and AHS. Data collection and analysis was undertaken by reputable organisations, however, minor survey errors are possible. Some temporal change in the seabed after surveys is almost certain which adds further uncertainty to the impacts of coastal hazards.

10.5 Beach Erosion and Recession

The volumes of storm erosion adopted in this study were informed by two methods undertaken by WRL: analysis of photogrammetry and numerical SBEACH erosion modelling.

For beaches where photogrammetry was available in 1972 and 1975 (Surfside Beach (East), Barlings Beach and Tomakin Cove) the maximum storm demand estimated from photogrammetry is considered a reasonable representation of the erosion that occurred due to the May-June 1974 storm sequence. However, the maximum storm demands estimated at the other beaches are considered to be an underestimate because the available photogrammetry dates do not capture the pre- and post-storm-sequence (i.e. beach recovery has occurred following the erosion event).

The SBEACH model has previously been calibrated and validated at numerous places around Australia. For this study, SBEACH was calibrated nearby to the study area against measured erosion at Bengello Beach. The sand grain size modelled at each beach was equivalent to the sediment samples acquired during the site inspections. Based on the experience of this report's authors, their engineering judgement, and consultation with OEH for this project, it was elected to model "design" erosion volumes using 2 x 100 year ARI storm events to account for storm clusters. Note that the Western Australian *Statement Of Planning Policy No. 2.6 (*Western Australian Planning Commission, 2003), specifies 3 x design storms to simulate clusters. Note also that changes to coastal geomorphology since 2014/2015 (when the majority of topographic and nearshore bathymetric survey data was recorded) will not be fully captured. The SBEACH model was calibrated under two separate conditions – aiming to achieve the maximum storm erosion observed at a single profile at Bengello Beach in 1974 (170 m^3/m above 0 m AHD) and, over the four (4) modelled profiles, to achieve the average erosion observed across the whole beach over the same period (95 m^3/m above 0 m AHD). These two target values were established because it is not known whether the singe profile maximum volume coincided with a rip-head embayment (three-dimensional dynamic formations like rip-heads are not included in SBEACH). Since SBEACH calibration was based on a high energy calibration location with a low beach slope, modelled erosion volumes at beaches with steep slopes may be over-predicted. WRL considers that this is likely to be the case at Maloneys Beach and Guerilla Bay (south).

The rates of recession adopted in this study ultimately relied on the analysis of temporal data sets of beach profile fluctuations. These were obtained using photogrammetric data made available by the OEH and ESC. The accuracy of this information rests with OEH and Jacobs (for photogrammetry data commissioned directly by ESC), however, photogrammetric analysis is undertaken to best current practice by skilled and experienced staff. The temporal resolution of the dataset limits the accuracy and reliability of the estimates.

Future shoreline recession as a result of sea level rise was estimated using the Bruun rule and the NSW Government's *Coastal Risk Management Guide* (DECCW, 2010). The limitations of this methodology are well recognised (Ranasinghe et al., 2007) and were taken into consideration. However, no robust and scientifically recognised alternative currently exists. Where known or obvious, the presence of underlying bedrock shelves was taken into account in the initial Bruun factor estimates in this study. However, there may be bedrock present in other areas where it is not visible.

10.6 Wave Runup and Overtopping

Best practice empirical prediction methods based on the most current published literature (Cox and Machemehl, 1986; Mase, 1989; FEMA, 2005 and EurOtop, 2016) were applied to estimate wave overtopping extents and runup levels at the dunes and seawalls. Statistical and data uncertainties related to these methodologies are discussed in the referenced literature (Shand et al., 2011 and EurOtop, 2016). The effect of wind on overtopping rates was not considered. Site specific physical modelling is the only available method offering greater certainty than the methods used.

10.7 Mapping of Coastal Hazard Lines

Mapping of coastal hazard lines was produced to provide general guidance for coastal planning and to identify areas prone to coastal hazards. Mapping was undertaken using state-of-the-art methodologies. Mapping was based on the most recent photogrammetry profiles for each beach (generally 2014, except 2011 for Barlings Beach and Broulee Beach). The limitations of the temporal and spatial resolution of the available photogrammetry data applies to the mapping. Site specific investigations and surveys are encouraged to overcome such limitations. WRL is not responsible for the accuracy of the photogrammetry data.

10.8 Modelling and Mapping of Coastal Inundation Zones

Mapping of coastal inundation zones was produced to provide general guidance for coastal planning and to identify areas prone to coastal inundation. Mapping was undertaken using state-of-the-art methodologies. Assessment of coastal inundation was performed using a combination of three methods at each beach section:

- A “bathtub” method was employed to map the extent of “quasi-static” inland inundation;
- If the dune or seawall crest level exceeds the “quasi-static” water level, the extent of the wave runup was estimated based on elevation using the Mase (1989) method for dunes and EurOtop (2016) for seawalls; and
- If the runup elevation exceeds the crest level, the Cox and Machemehl (1986) method, as adjusted by FEMA (2005), was used to estimate the landward propagation distance of wave bores.

Mapping of inland inundation assumed that topography remains as it was from the 2005 and 2011 LiDAR data provided by NSW LPI and did not consider flow paths, flow velocities, loss of flow momentum or wave propagation into creek areas. No changes were made to isolated “quasi-static” inundated areas that appear to be hydraulically disconnected; further detailed hydraulic modelling considering localised effects would be required to eliminate or confirm their validity. A qualitative check indicated that the LiDAR data was consistent with the observed land forms, however, WRL is not responsible for the accuracy of the LiDAR data.

Mapping of runup and overtopping wave bores was based on the 2011 or 2014 photogrammetry data or 2005 LiDAR data and did not include any allowance for future landward recession. Mapping of runup and overtopping was only undertaken along the crest of the dune or seawall along each beach section; it was not mapped inside watercourse entrances, inside the Batemans Bay Boat Harbour, at rock platforms or cliffed regions.

11. Recommended Further Work

Throughout this report, WRL has recommended that a number of additional investigations be undertaken. These further assessments are summarised in this section

Tidal and Coastal Inundation (Sections 7 and 8)

Mapping of inland tidal and coastal inundation assumed that all areas below the specified coastal water level will be inundated and did not consider connectivity of flow paths, flow velocities, loss of flow momentum or wave propagation into creek areas. Specifically, the maps provided have not been adjusted where channel constrictions, roughness or other similar flow impediments may prevent sufficient hydraulic connectivity for inland flood levels to reach the full extent of "quasi-static" inundation levels. No changes were made to isolated "quasi-static" inundated areas that appear to be hydraulically disconnected. Should ESC identify areas of particular concern for inland inundation, it is suggested that more detailed hydraulic modelling be undertaken to eliminate or confirm their validity. Local surveys by a registered surveyor are also recommended to determine local inundation extents.

Seawall Condition Assessments (Appendix B)

Overall, the condition of the rock revetment wall around the CBD is considered to be reasonable. However, WRL recommends that ongoing monitoring of the condition of the wall by ESC according to coastal engineering guidelines (CEM, 2006).

Between the Batemans Bay Boat Harbour and CBD, WRL understands that ESC is responsible for the maintenance of the revetment where Beach Road is located immediately in its lee (up to 50 m east of Herarde Street). The condition of the rock revetment wall under the responsibility of ESC is considered to be fair, however, one section opposite "The Old School House" (TOSH, 10 Beach Road) requires immediate attention. The rock type is unknown with an approximate size of 0.4 m and a structure slope of 1V:1.0H. No geotextile underlayer was visible. In this section, the crest of the revetment is below the level of Beach Road and fines are being lost through the wall over a distance of approximately 100 m. Ongoing monitoring of the condition of the remainder of the wall between the Boat Harbour and the CBD should be undertaken by ESC.

Overall, the condition of the seawall along the northern part of Caseys Beach is considered to be poor and requires immediate action and ongoing monitoring by ESC. The reader is referred to WRL's detailed condition assessment and design advice report for this seawall (Blacka and Coghlan, 2016).

Durras Lake Tailwater Conditions (Appendix M)

At Durras Beach and Cookies Beach, there was no available nearshore bathymetric surveys. The AHS bathymetric data in this area has contours starting at -15 m AHD, but very little available information closer to the shore. To fill the nearshore region, depth contours based on a Dean Equilibrium Profile (Dean, 1977) were assumed. Since the quality of this assumption is unknown, WRL recommends that the tailwater condition assessment for the entrance to Durras Lake be repeated when a bathymetry survey is undertaken offshore of Durras Beach (South).

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