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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).

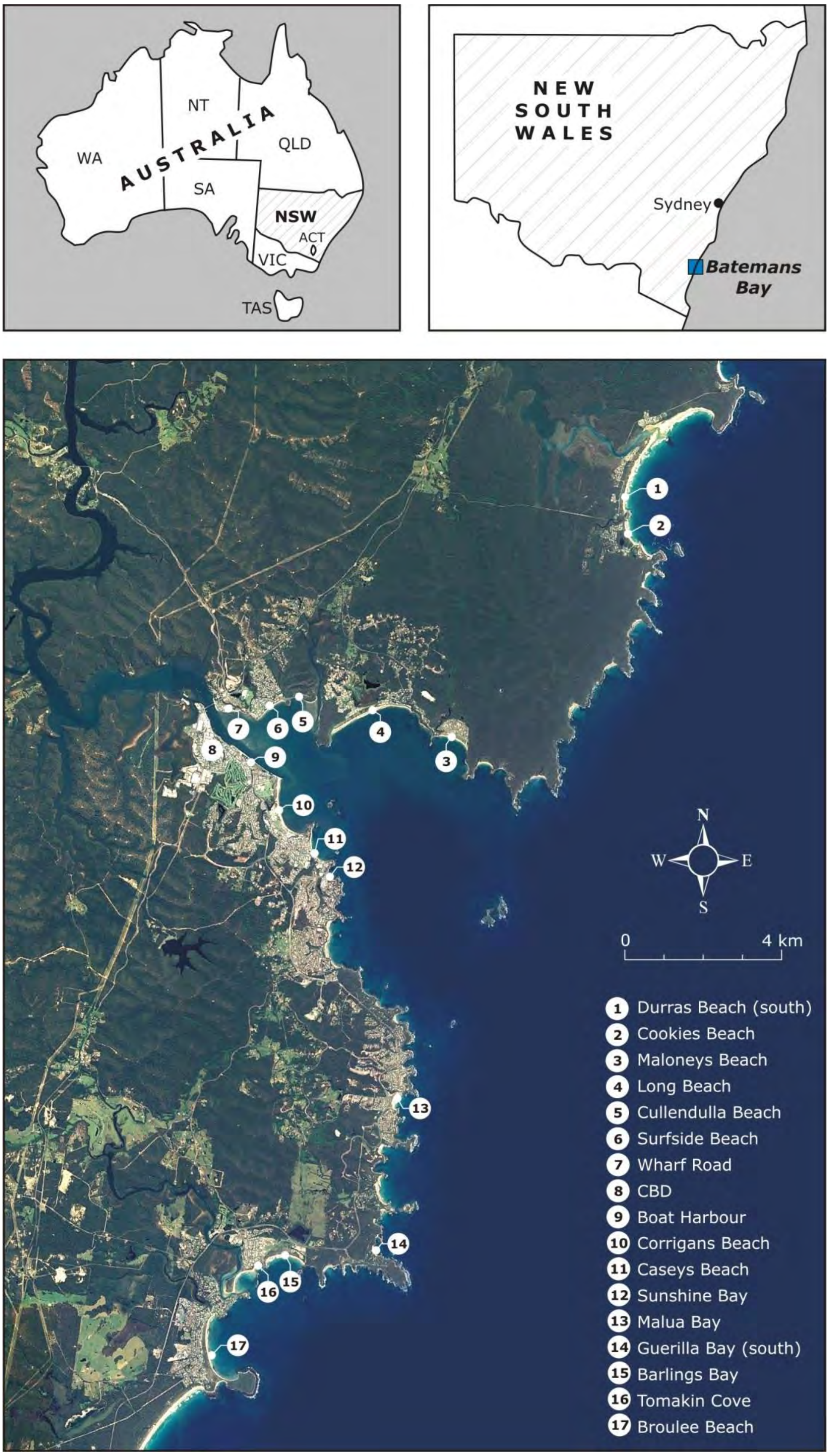


Figure 1-1: Location and Study Area

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

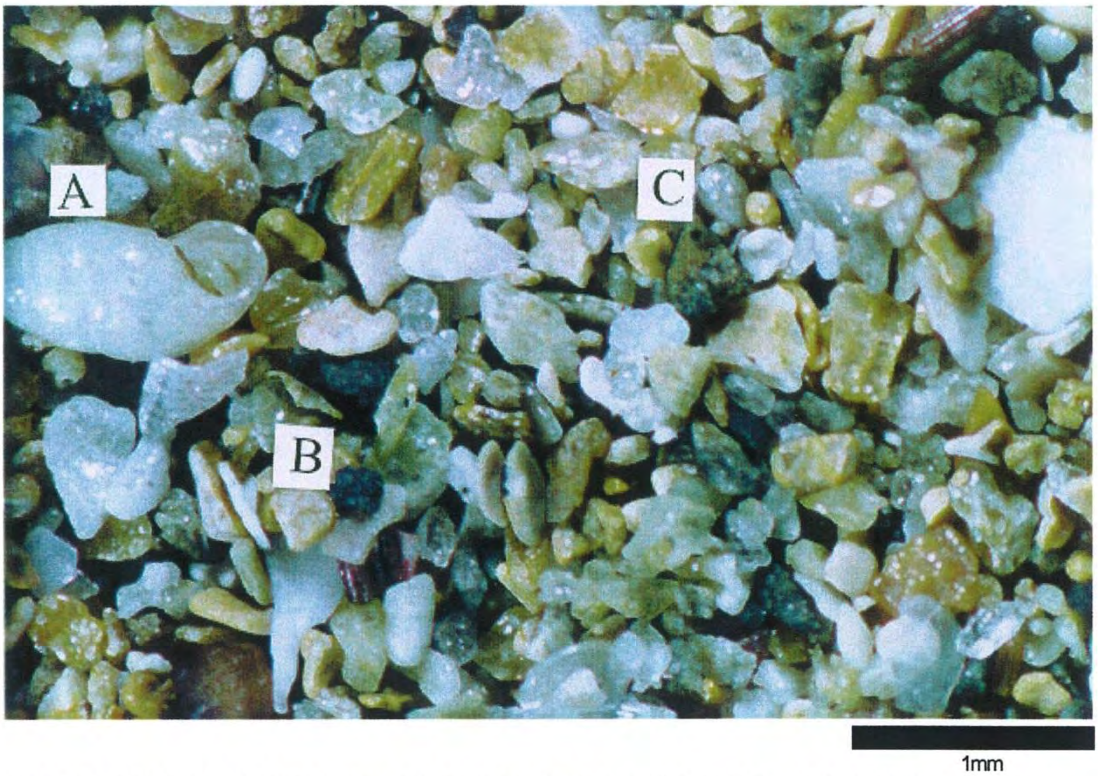
Generally, the sediment from each of the beaches is characterised as medium grained sand. However, it also important to note exceptions to this within the coastline sub-sections. The sediment from Cullendulla Beach, Sunshine Bay, Tomakin Cove & Beach and Broulee Beach has a relatively high fraction of 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).

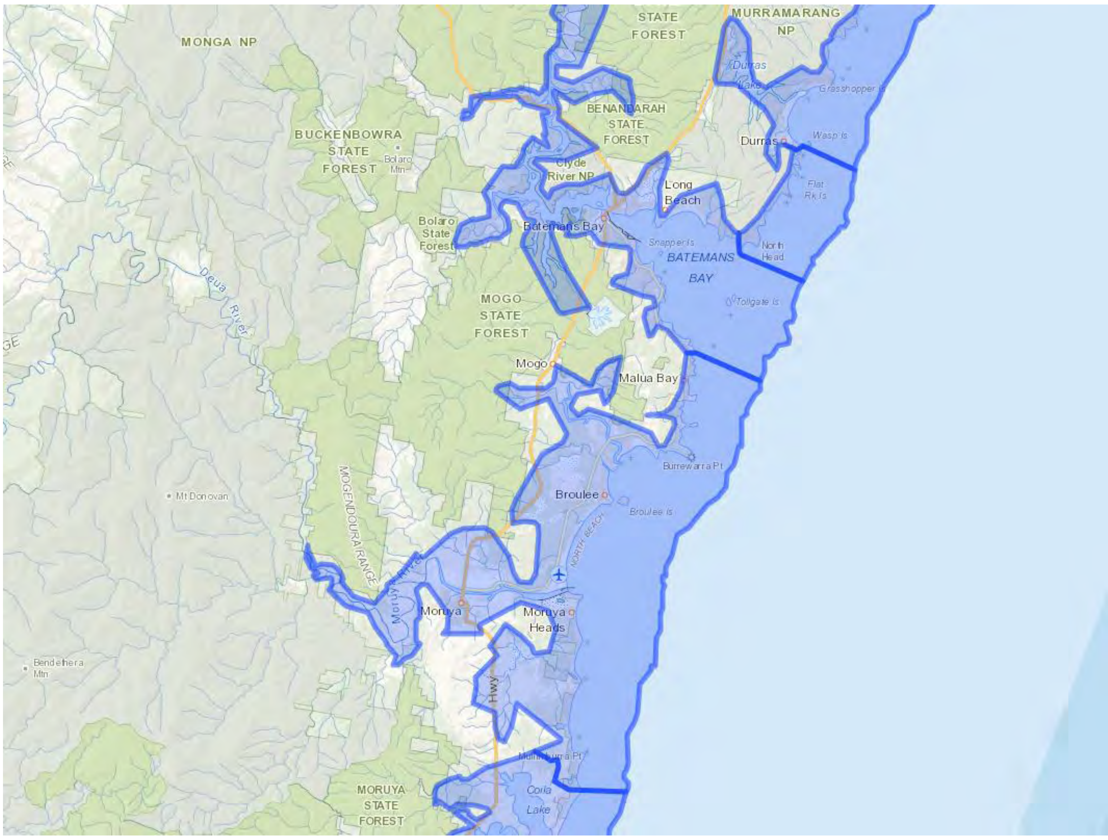


Figure 3-1: Secondary Sediment Compartments in the Study Area (CoastAdapt, 2017)

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 > 2-3 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.

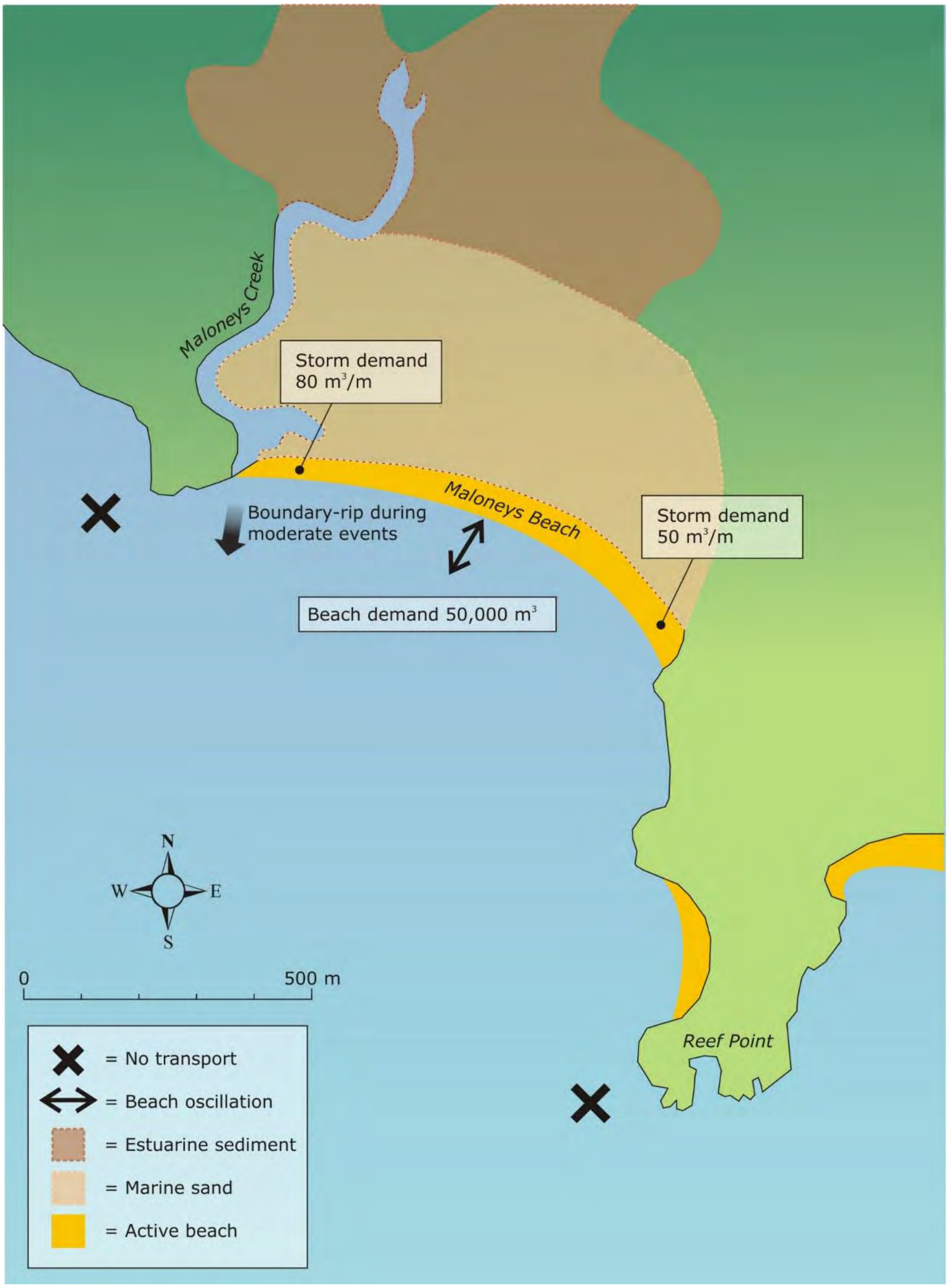


Figure 3-3: Conceptual model of sediment movement and storm demand at Maloneys Beach

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.

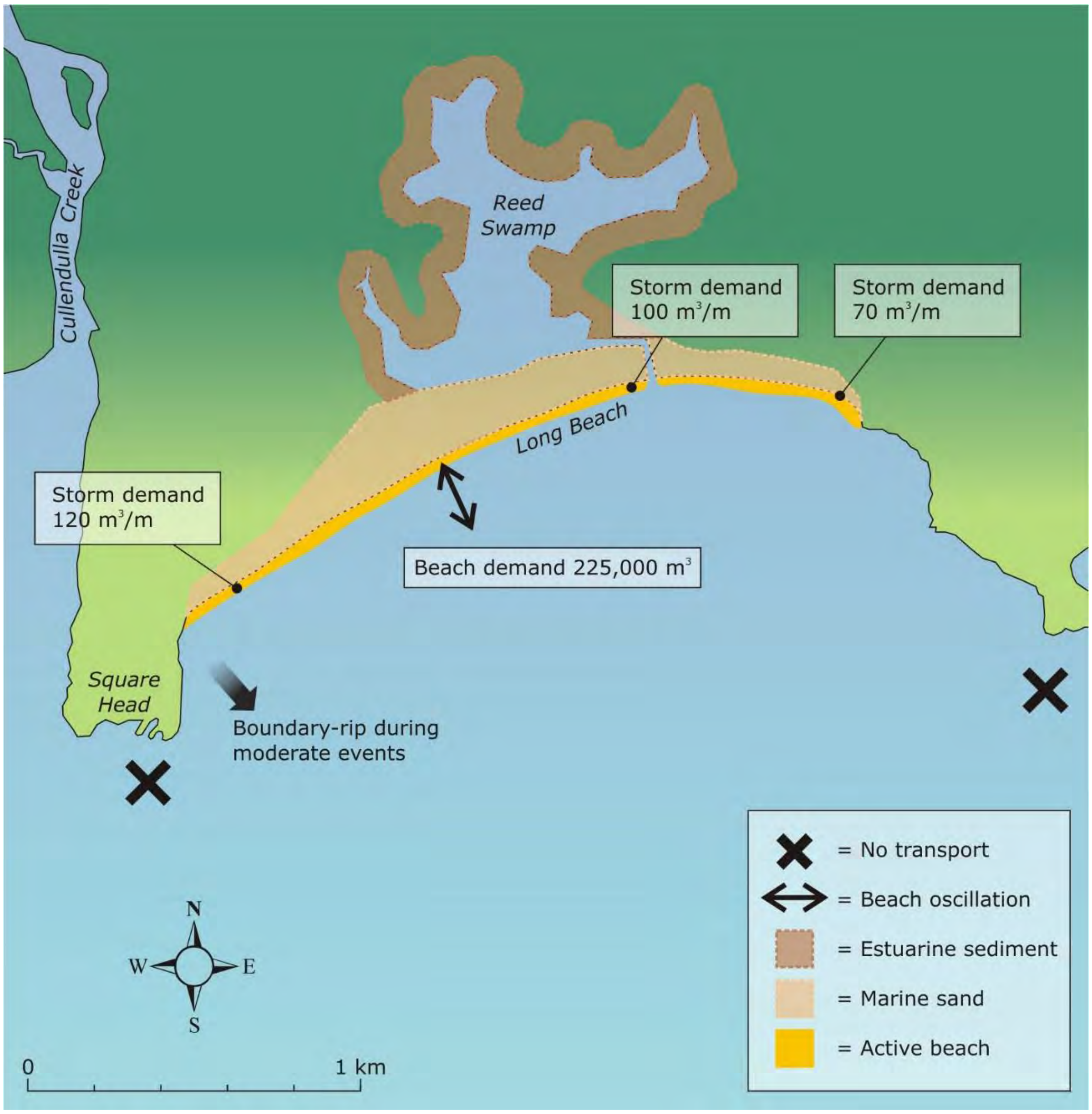


Figure 3-4: Conceptual model of sediment movement and storm demand at Long Beach

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

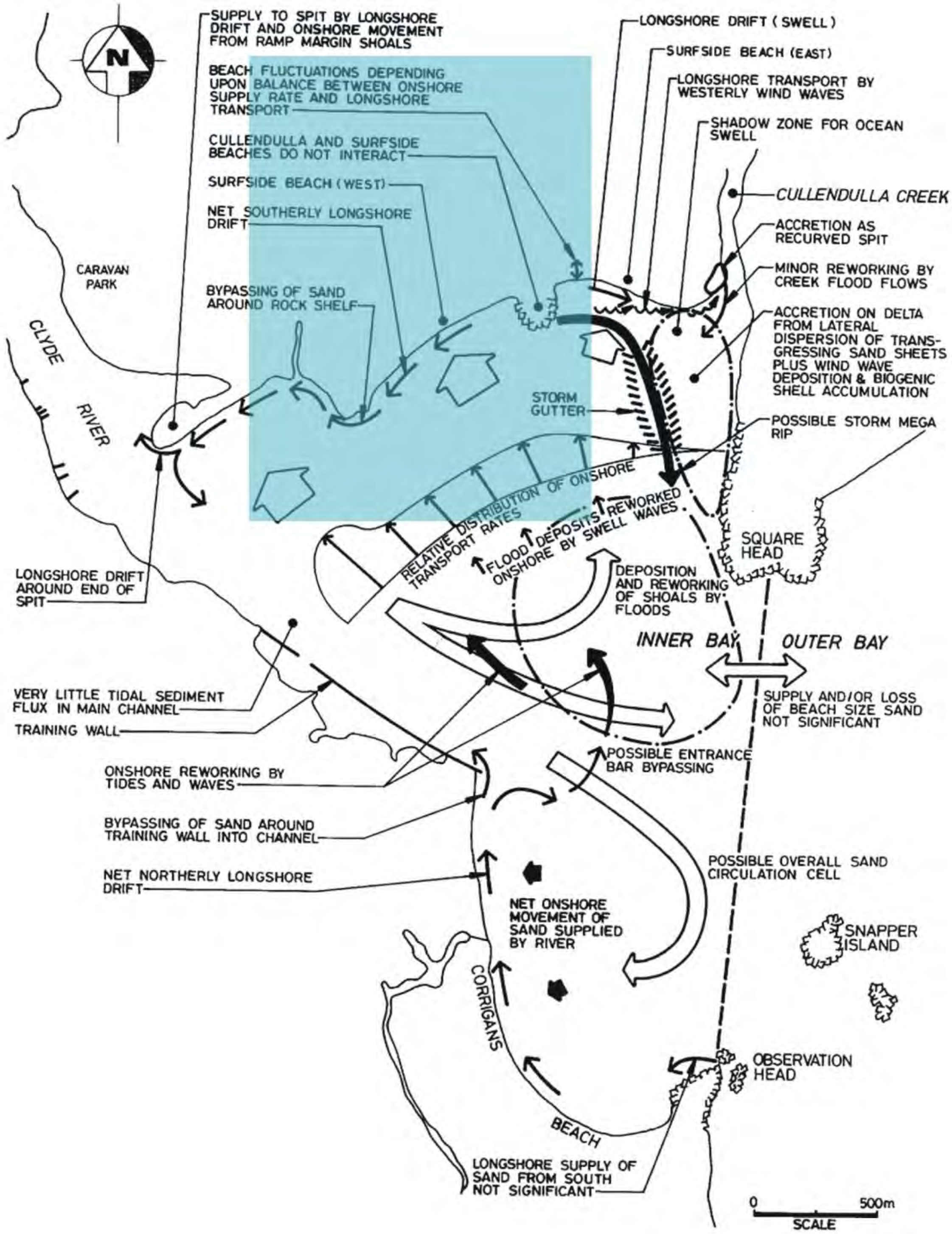


Figure 3-6: Conceptual model of sediment transport pathways within inner Batemans Bay (after Patterson Britton and Partners, 1992)

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.

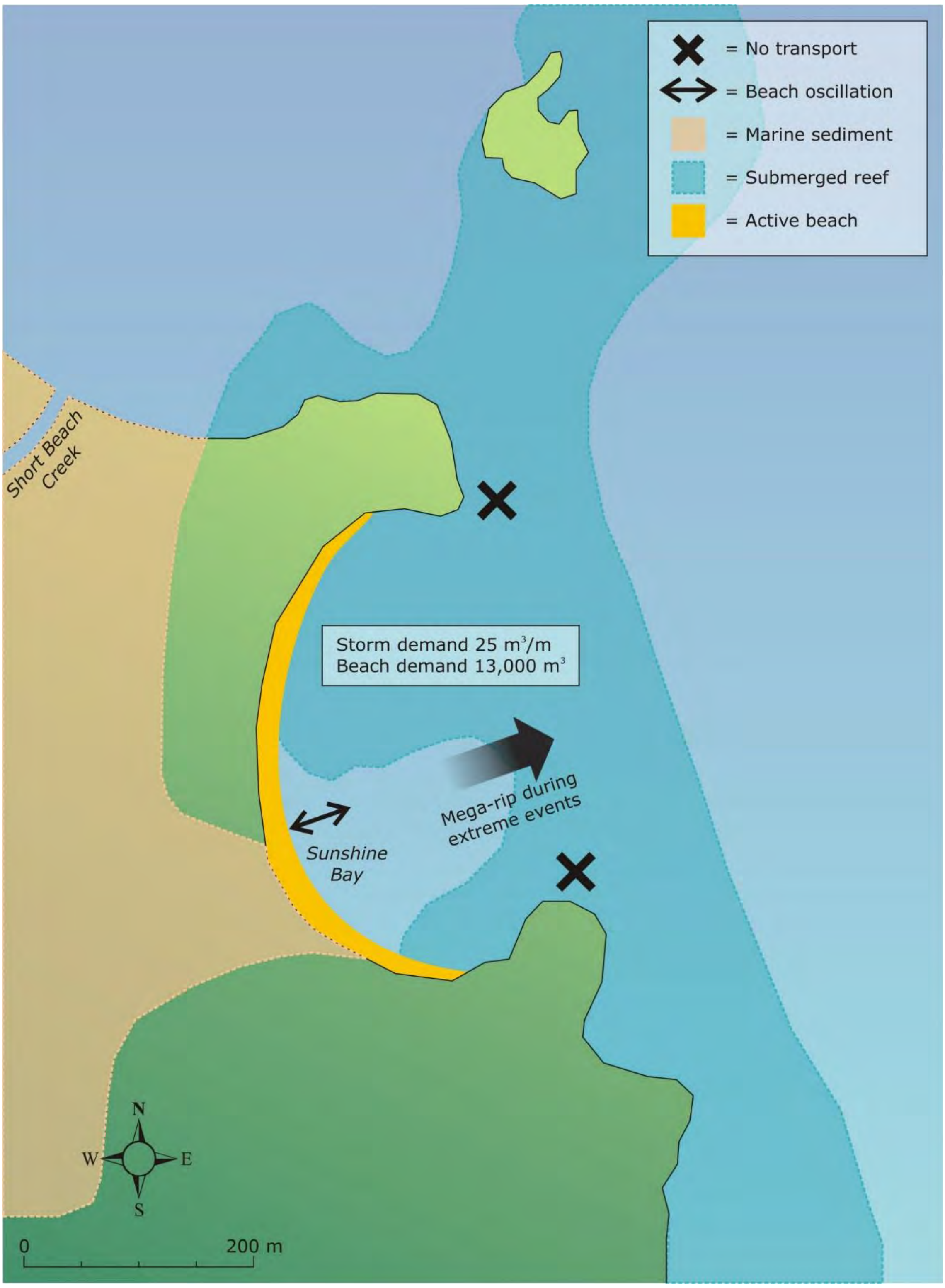


Figure 3-7: Conceptual model of sediment movement and storm demand at Sunshine Bay

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)

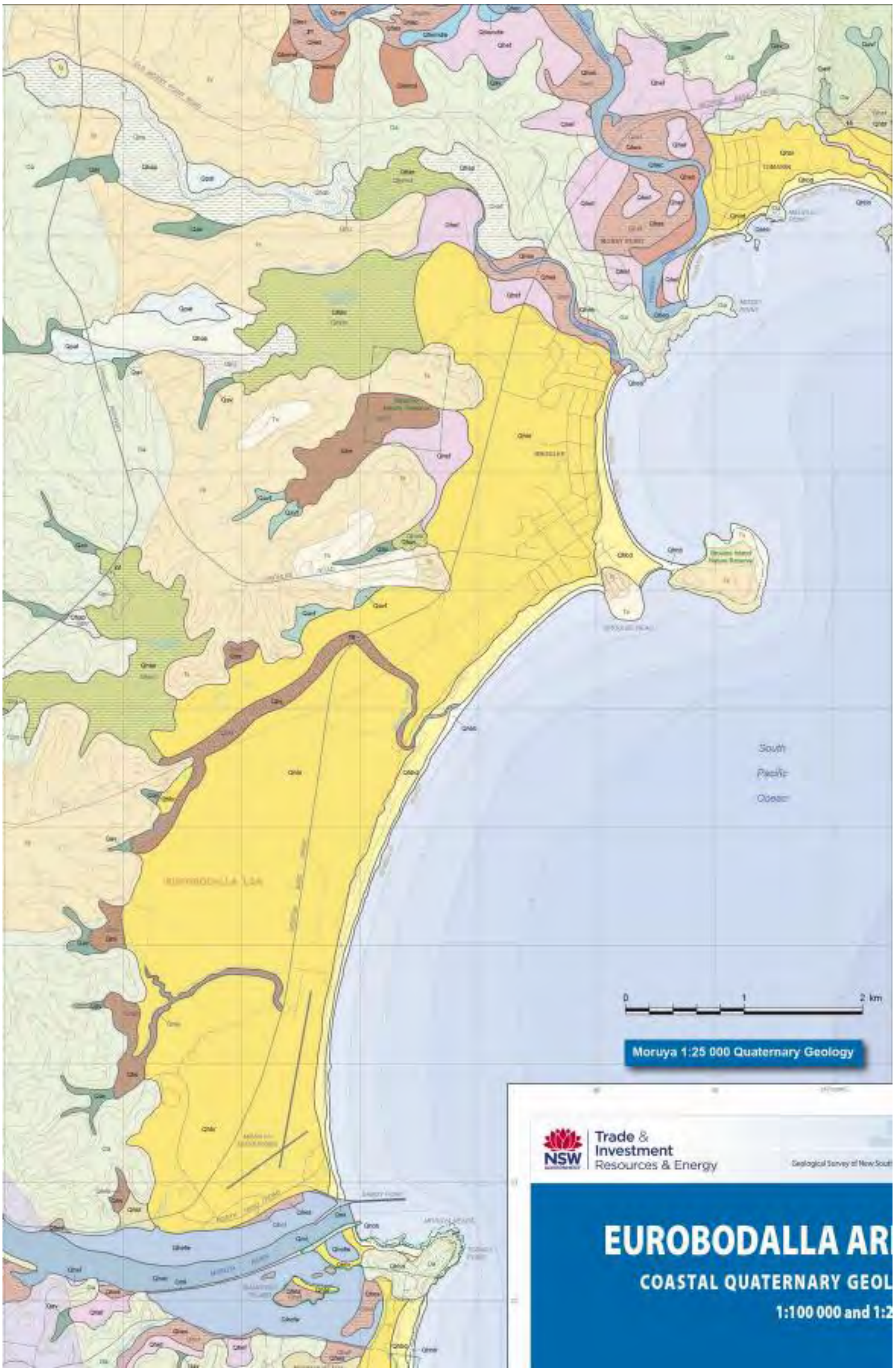


Figure 3-9: Quaternary geology of the central Broulee compartment. The Barlings Beach-Tomakin Cove and Beach and Broulee Beach-Bengello Beach embayments have accumulated large regressive barriers (Source: Troedson and Hashimoto, 2013)

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.

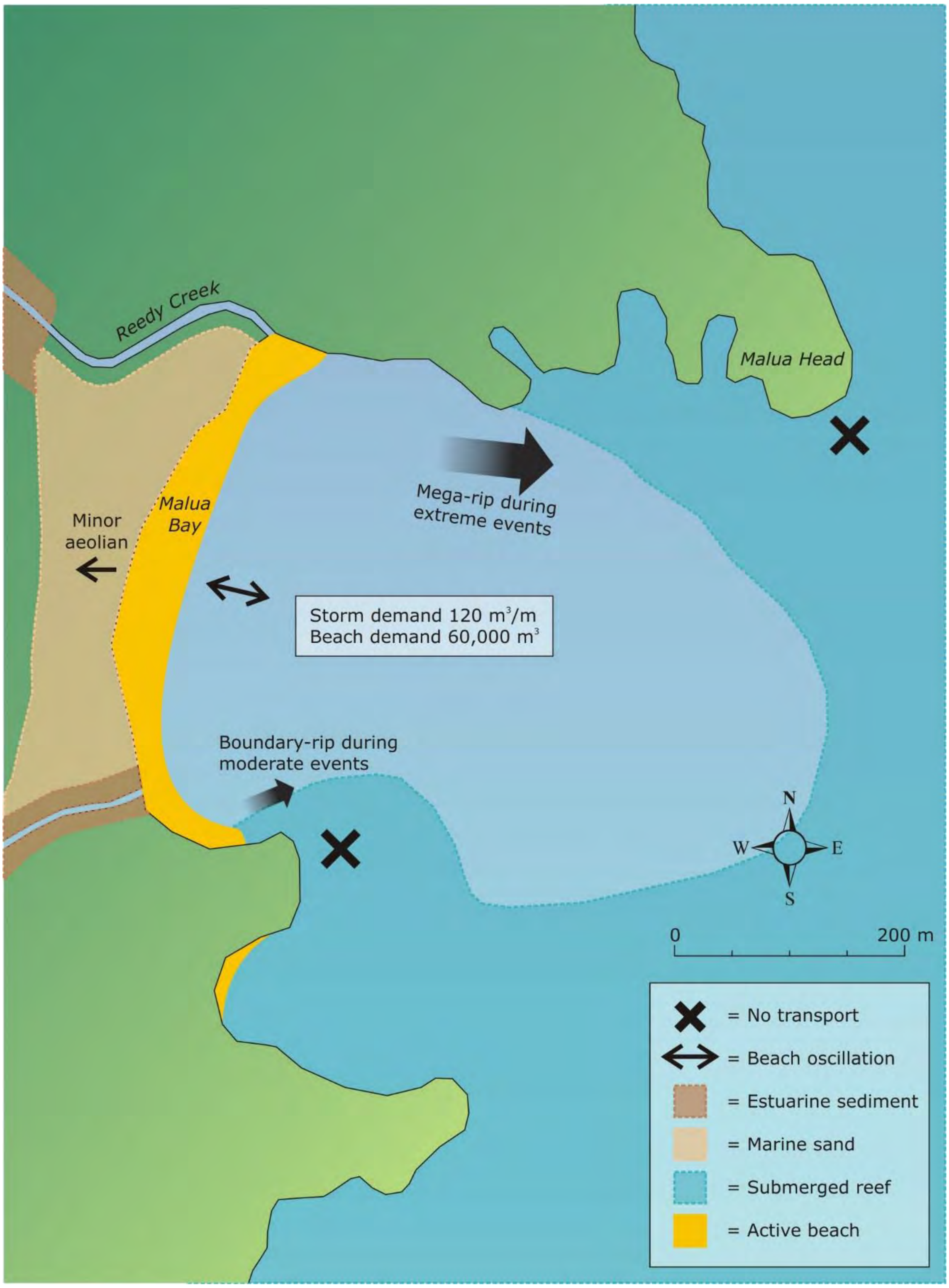


Figure 3-10: Conceptual model of sediment movement and storm demand at Malua Bay

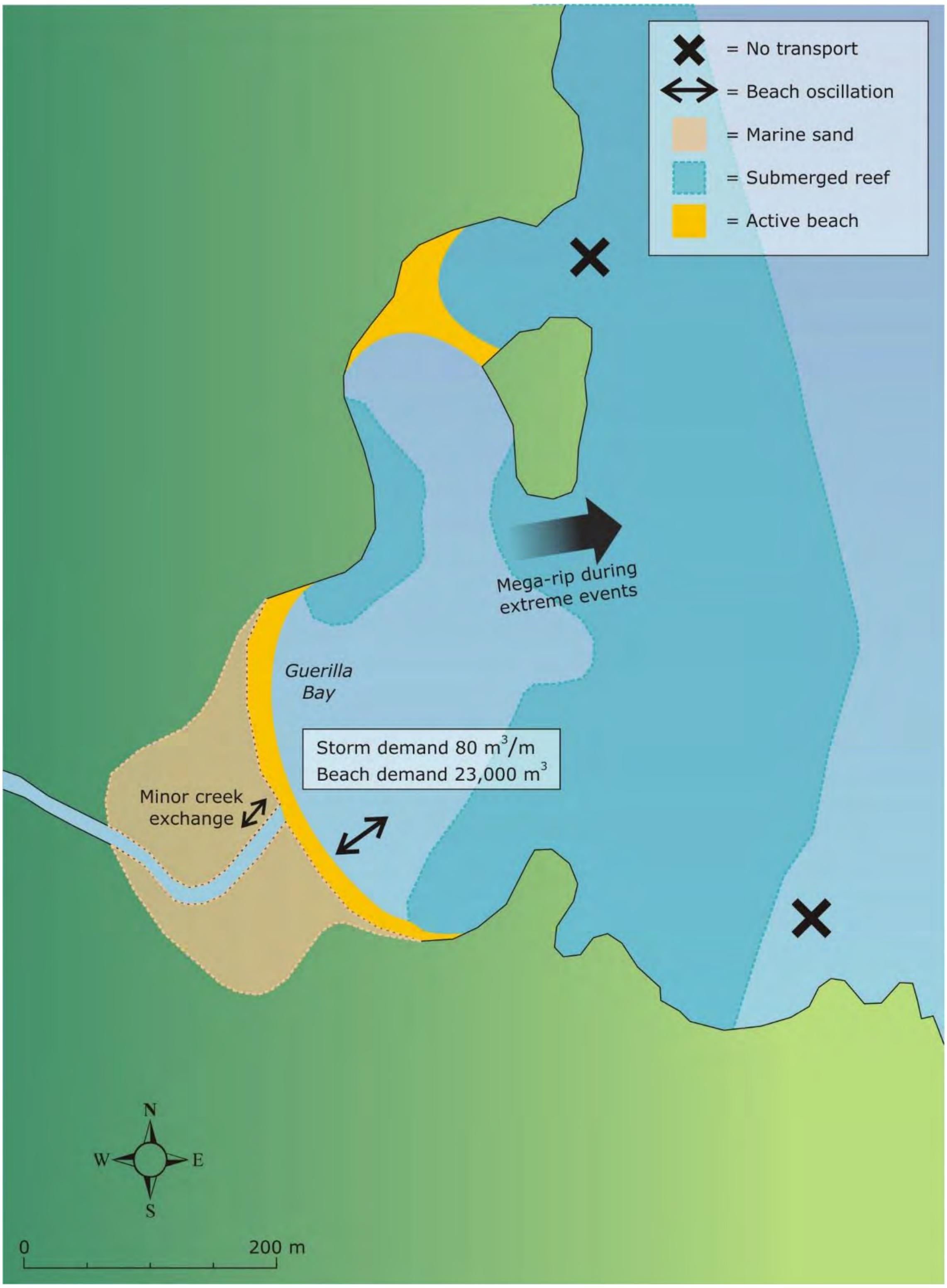


Figure 3-11: Conceptual model of sediment movement and storm demand at Guerilla Bay

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.

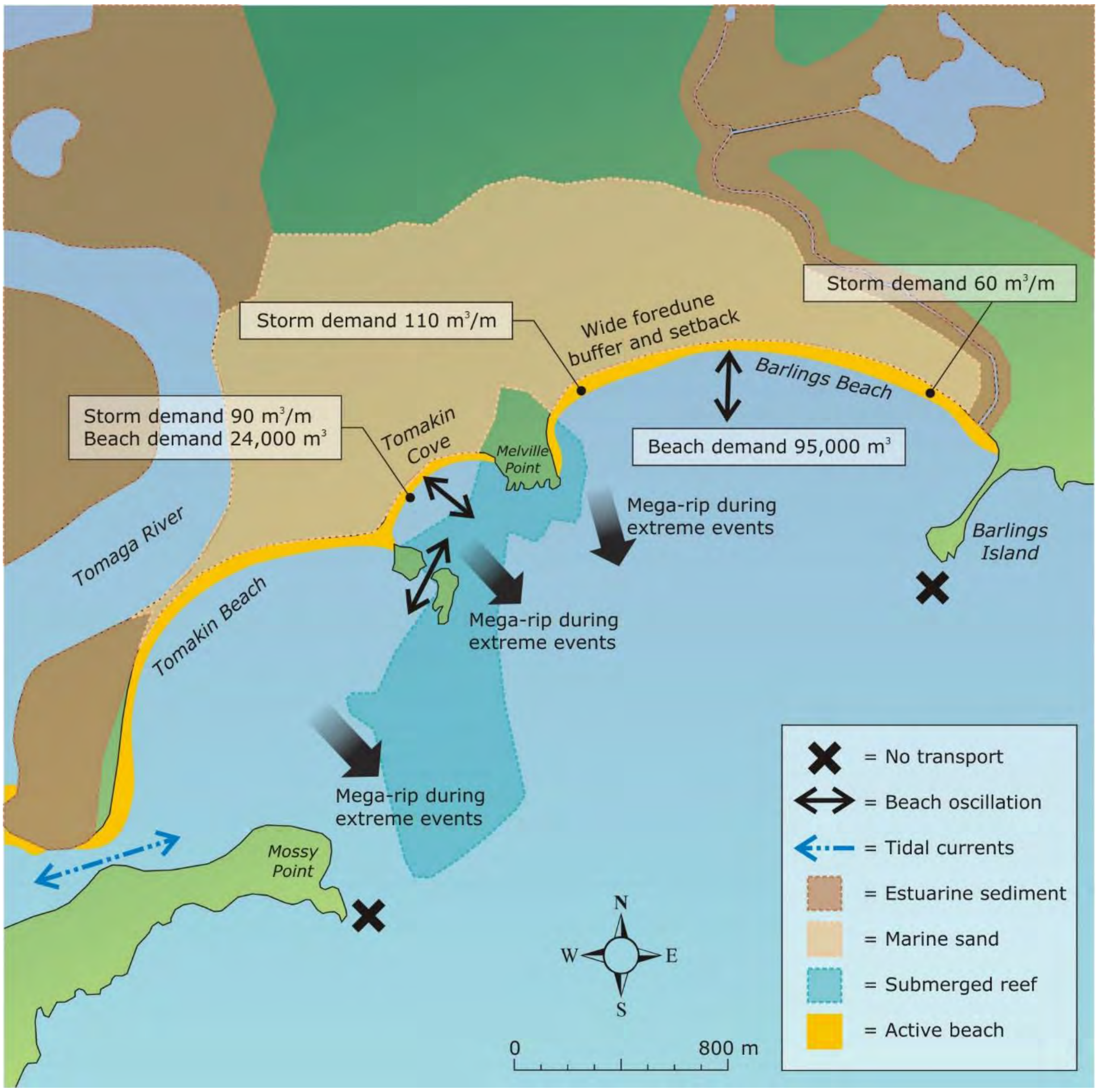


Figure 3-12: Conceptual model of sediment movement and storm demand at Barlings Beach and Tomakin Cove

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.

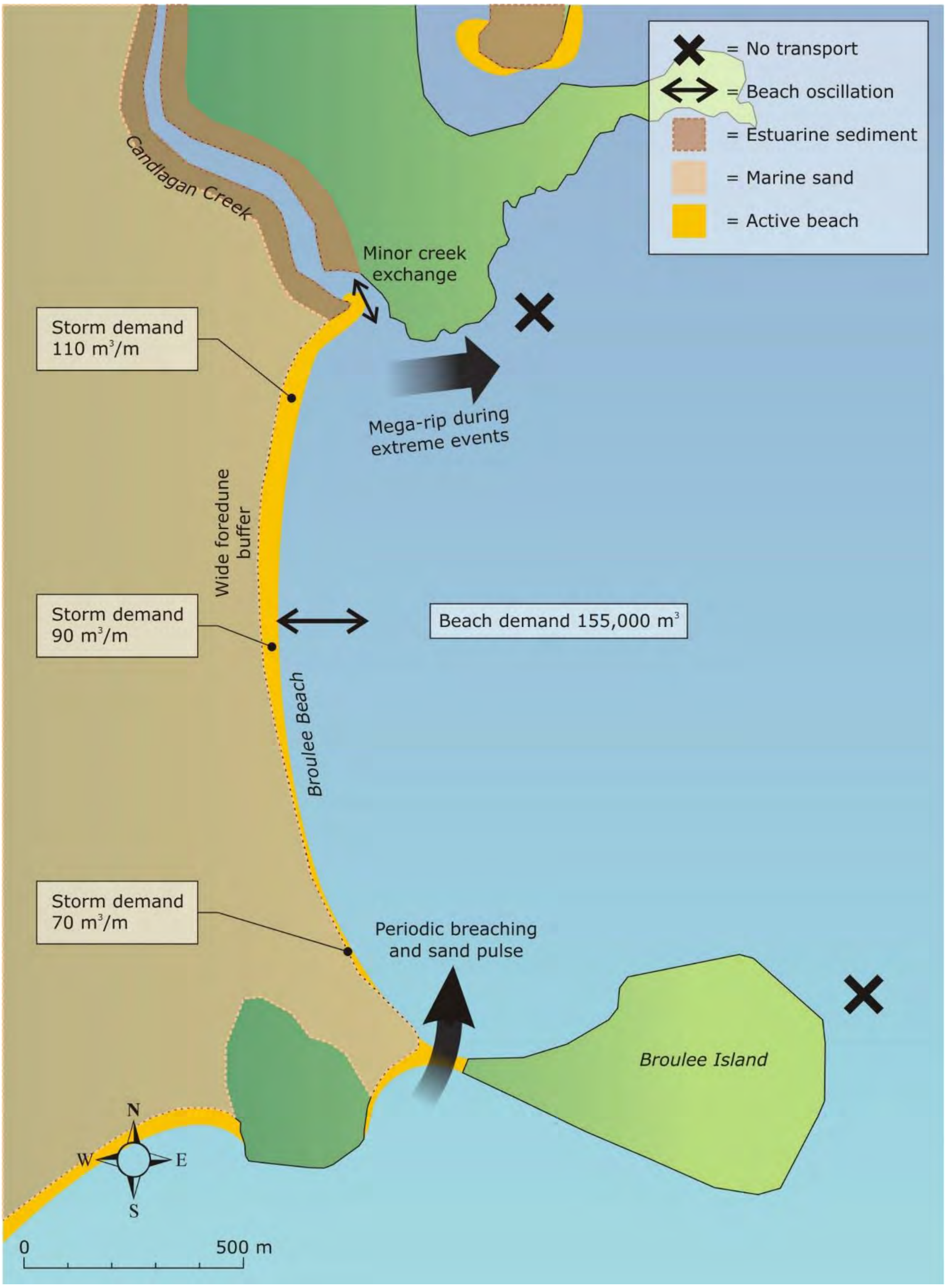


Figure 3-13: Conceptual model of sediment movement and storm demand at Broulee Beach

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.

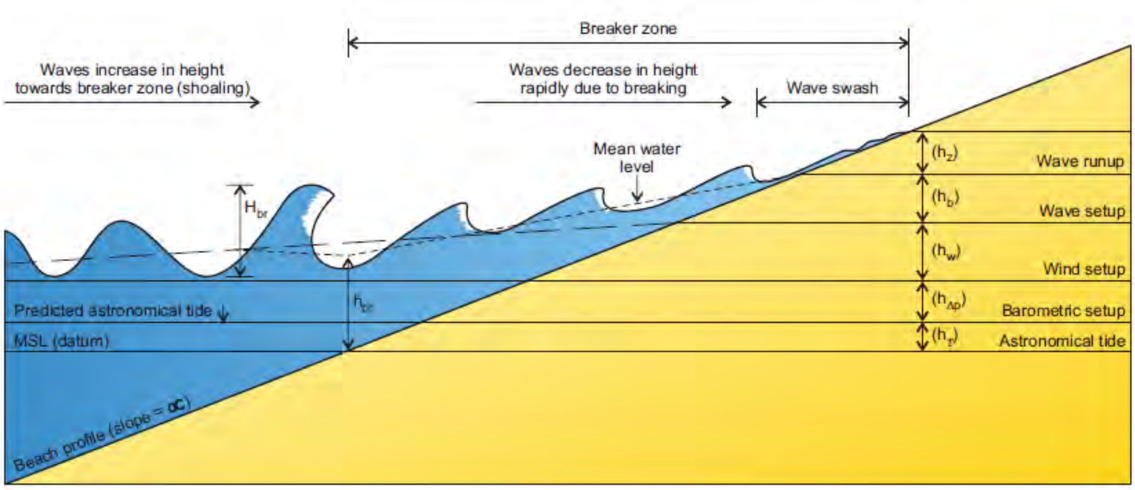


Figure 4-1: Components of Elevated Ocean Water Levels (Adapted from DECCW, 2010)

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).

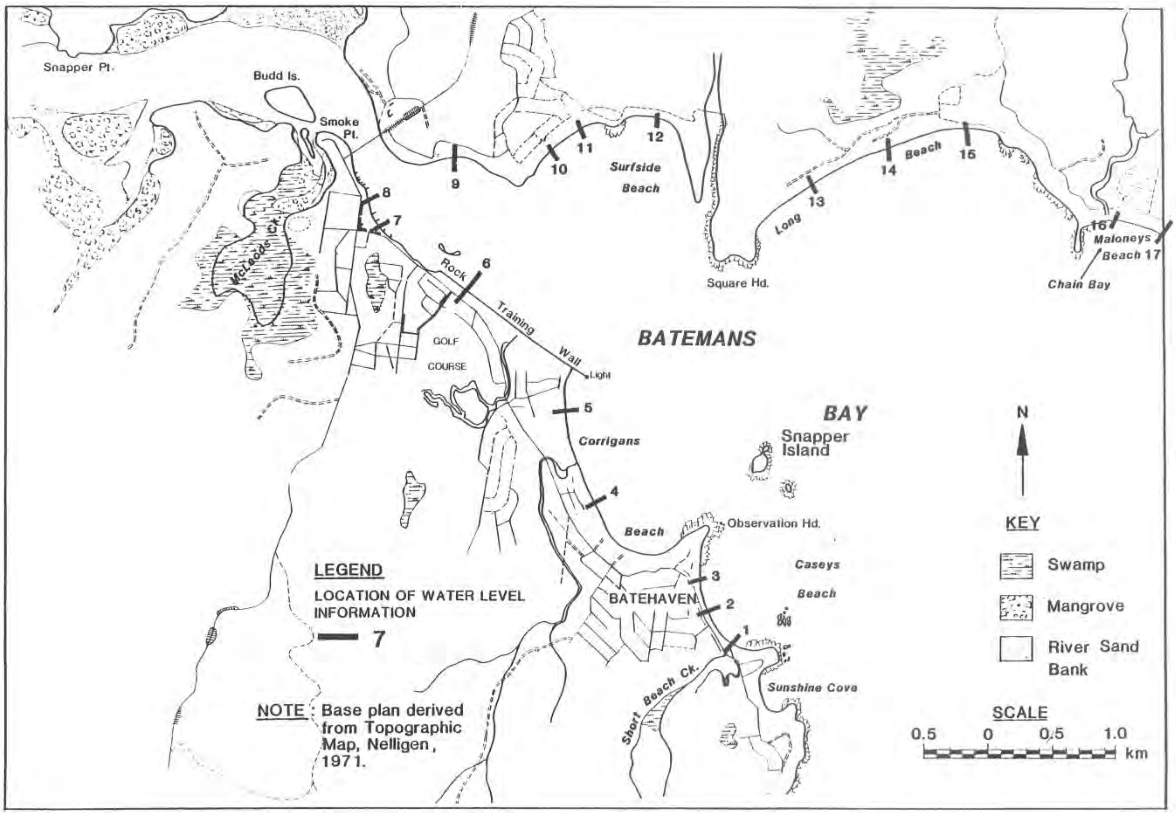


Figure 4-2: Water level output locations from NSW PWD (1989)

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).

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

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:

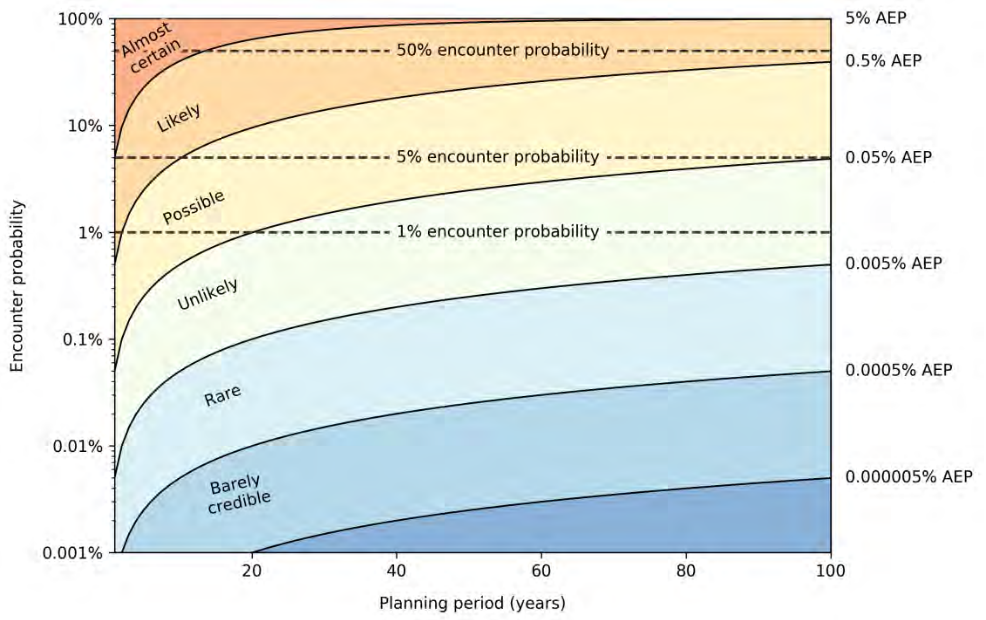
- Average Recurrence Interval (ARI);
- Annual Exceedance Probability (AEP); and
- Encounter Probability (EP) over the planning horizon.

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.

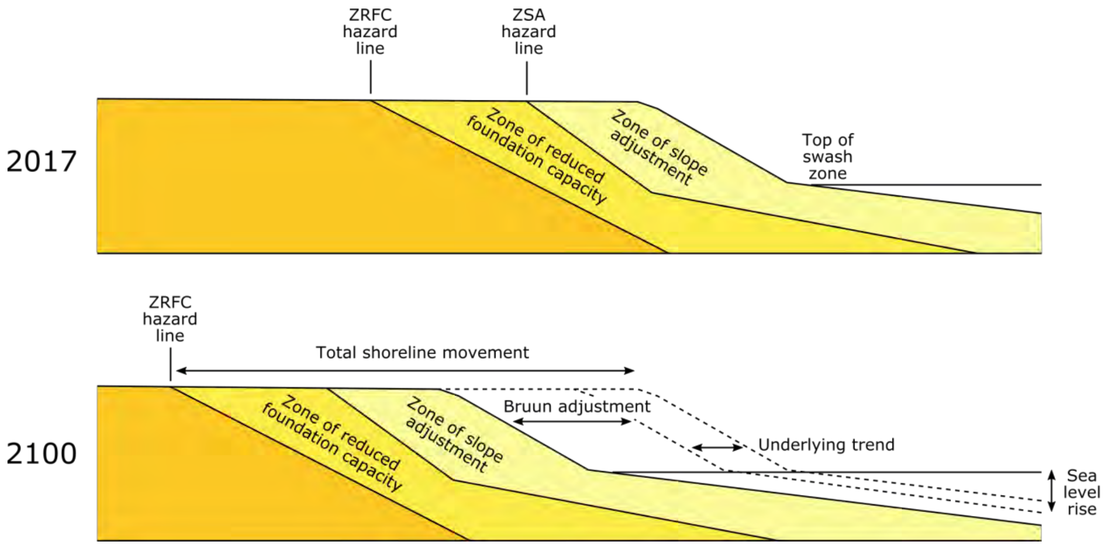
Probabilities of storm demand are also included in this assessment by combining them randomly with the recession probabilities in a further Monte-Carlo simulation. Note that by assuming that the storm demand represents a deviation from the long term average trend, and by expressing the combined probability 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.

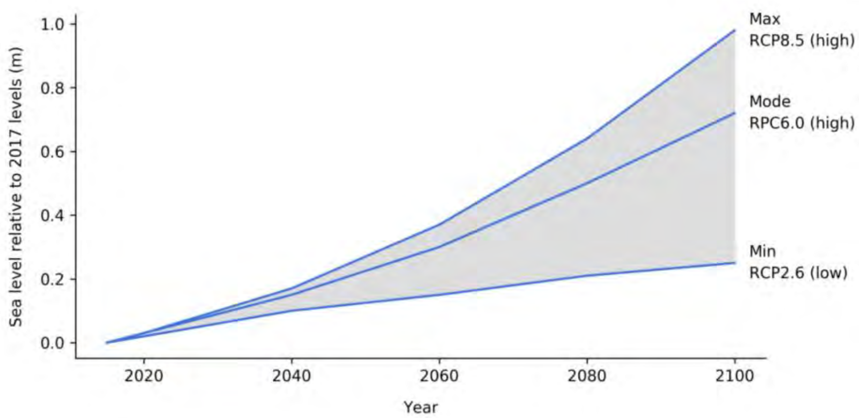


Figure 6-3: Sea level rise input values (Whitehead & Associates, 2014)

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.

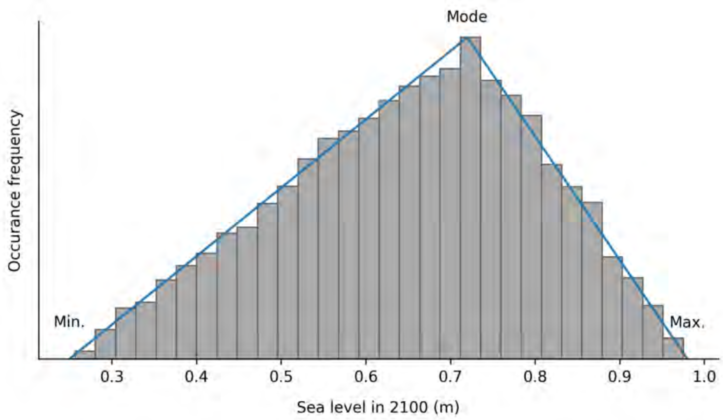


Figure 6-4: Triangular probability density function of sea level rise in 2100

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.

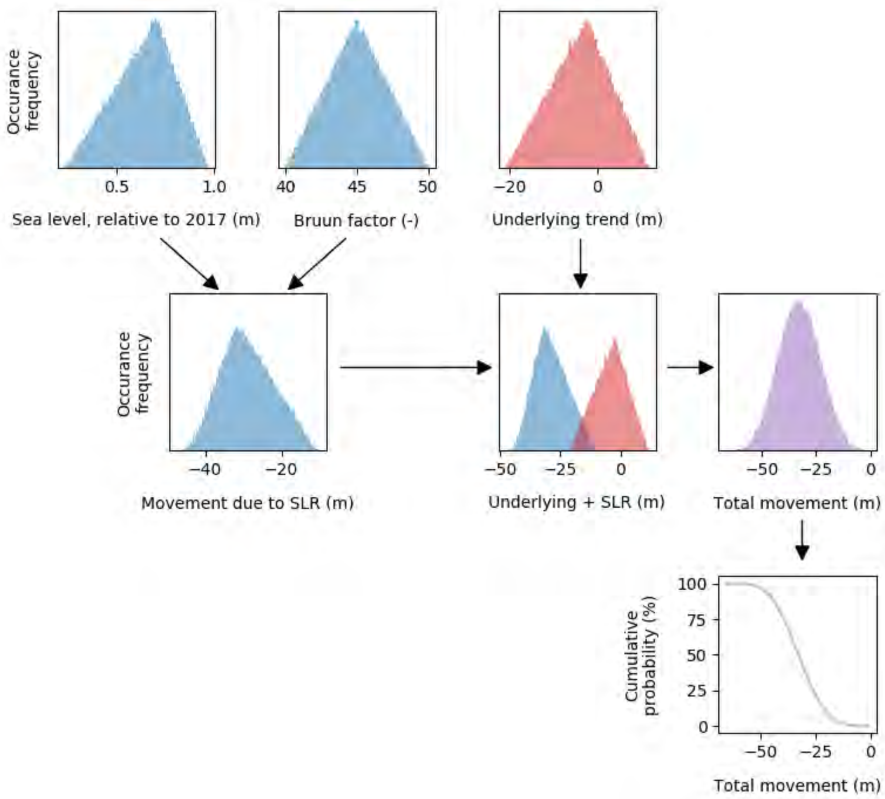
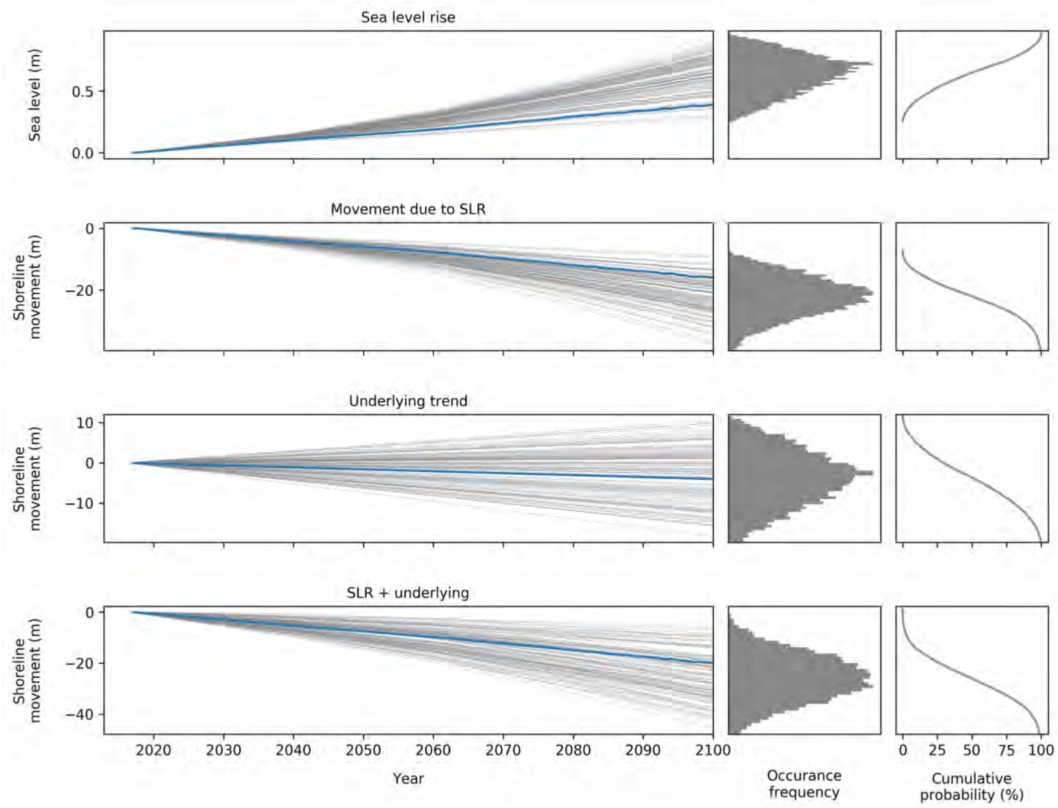


Figure 6-5: Methodology for combining random values to estimate shoreline movement

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).

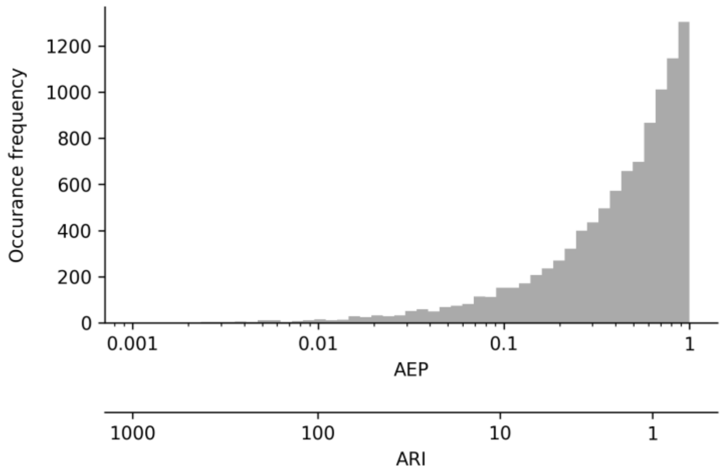


Figure 6-7: Uniform distribution of AEP values for generating storm demand volumes

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).

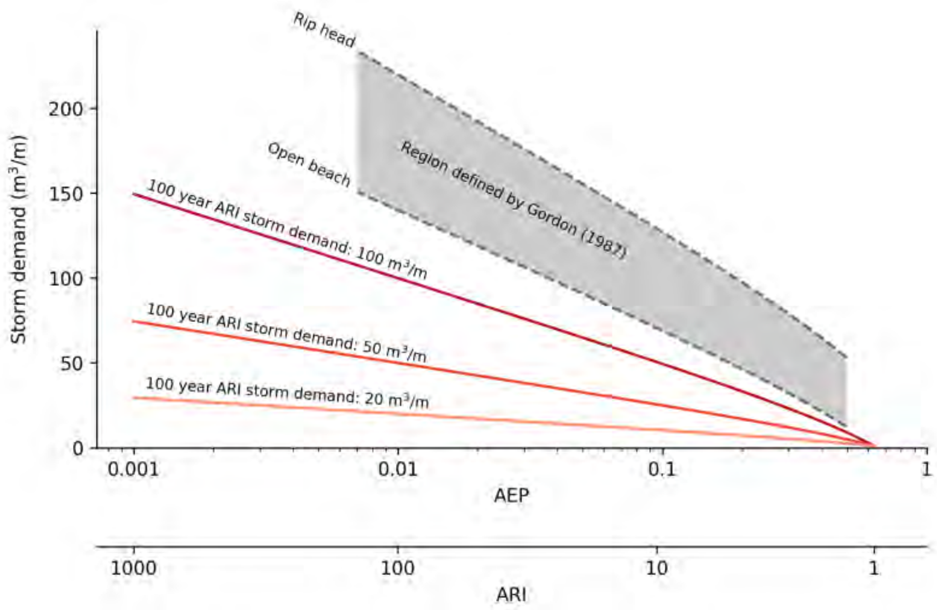


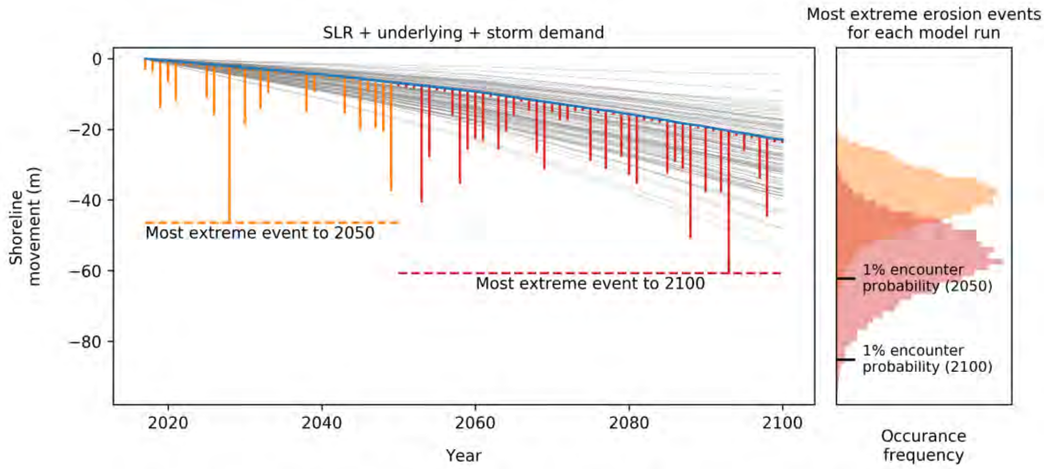
Figure 6-8: Storm demand volumes for exposed beaches in NSW (after Gordon, 1987)

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).

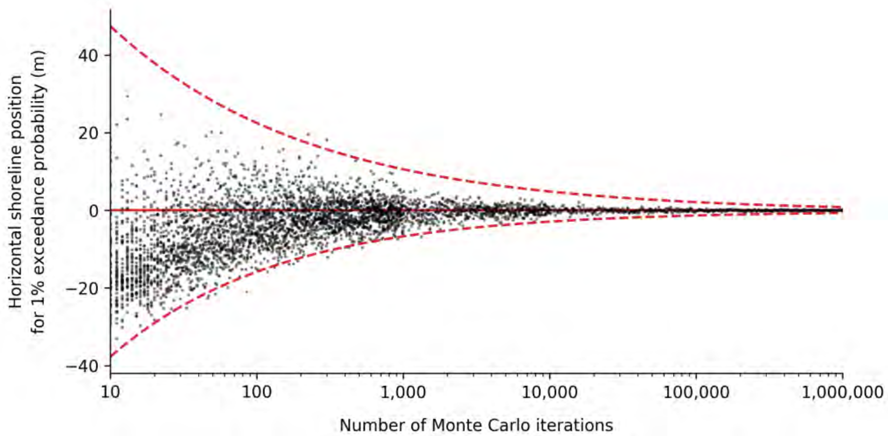


Note: Orange and red bars represent storm demand erosion for a single probabilistic simulation result. Left panel only shows the first 100 simulations to minimise clutter.

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

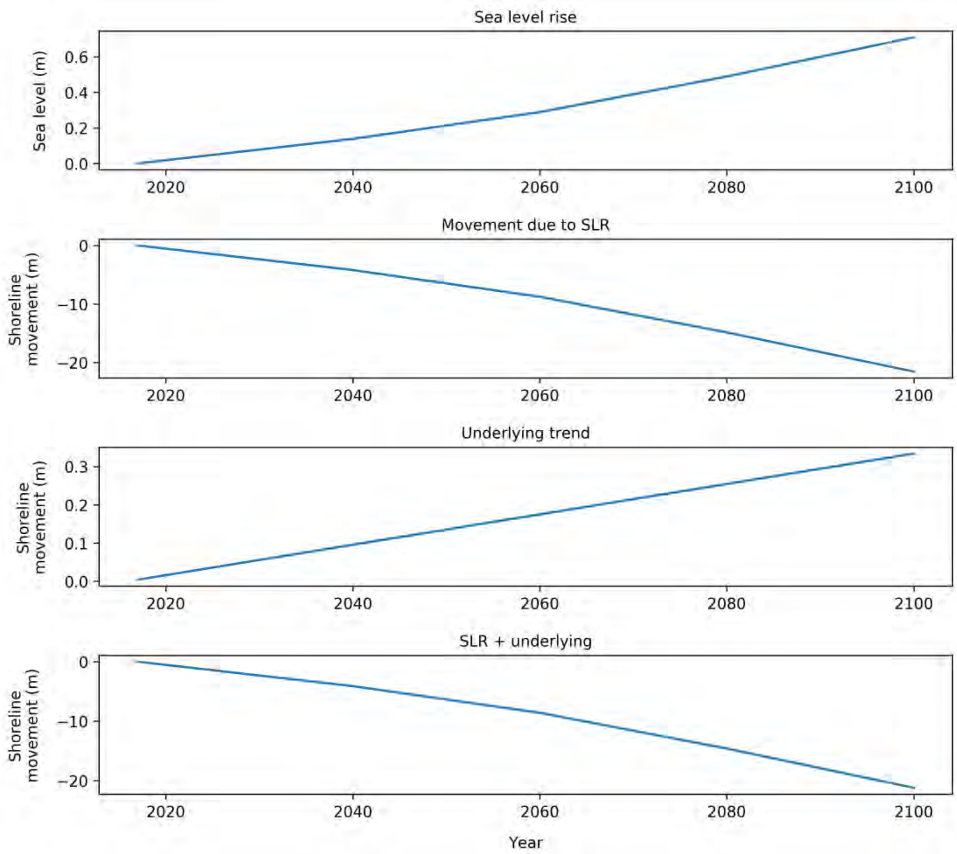


Figure 6-11: Calculated deterministic trajectories for sea level rise and underlying recession

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

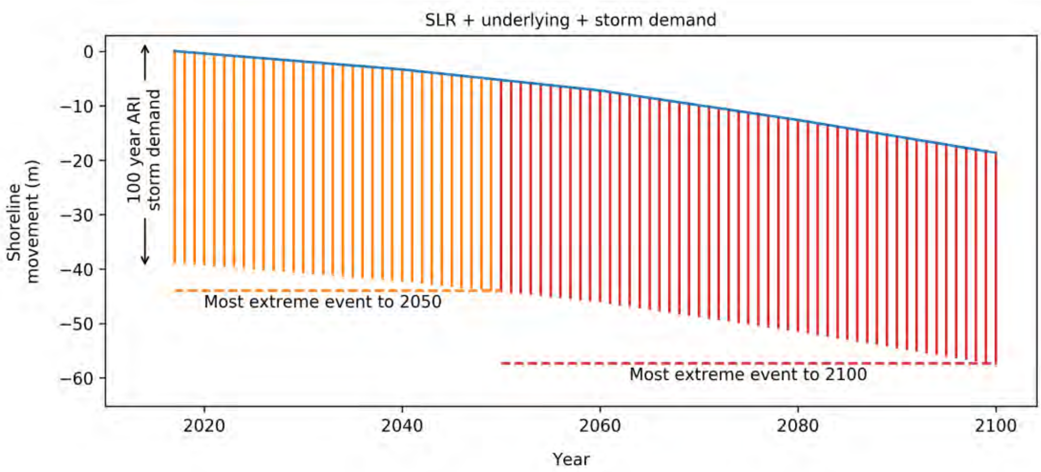


Figure 6-12: 100 year ARI storm demand superimposed on deterministic shoreline movement

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.

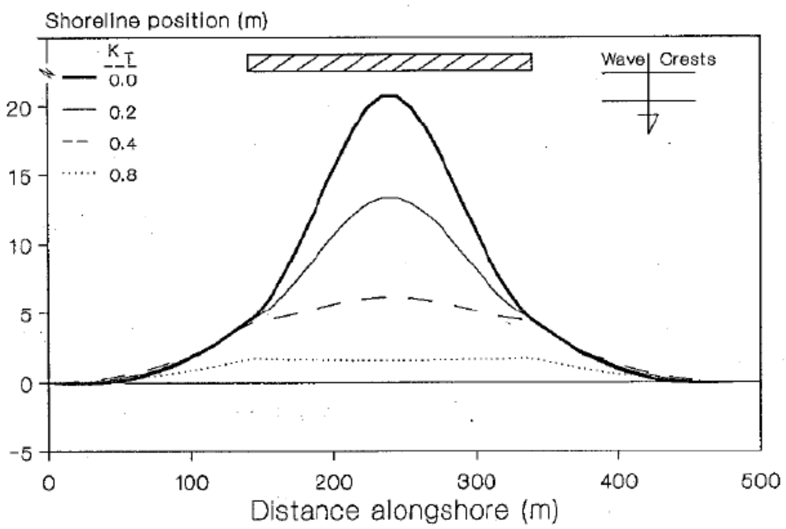


Figure 6-13: Effect of Wave Transmission (K_T) over a reef on the extent of a salient (Source: Hanson et al., 1990)

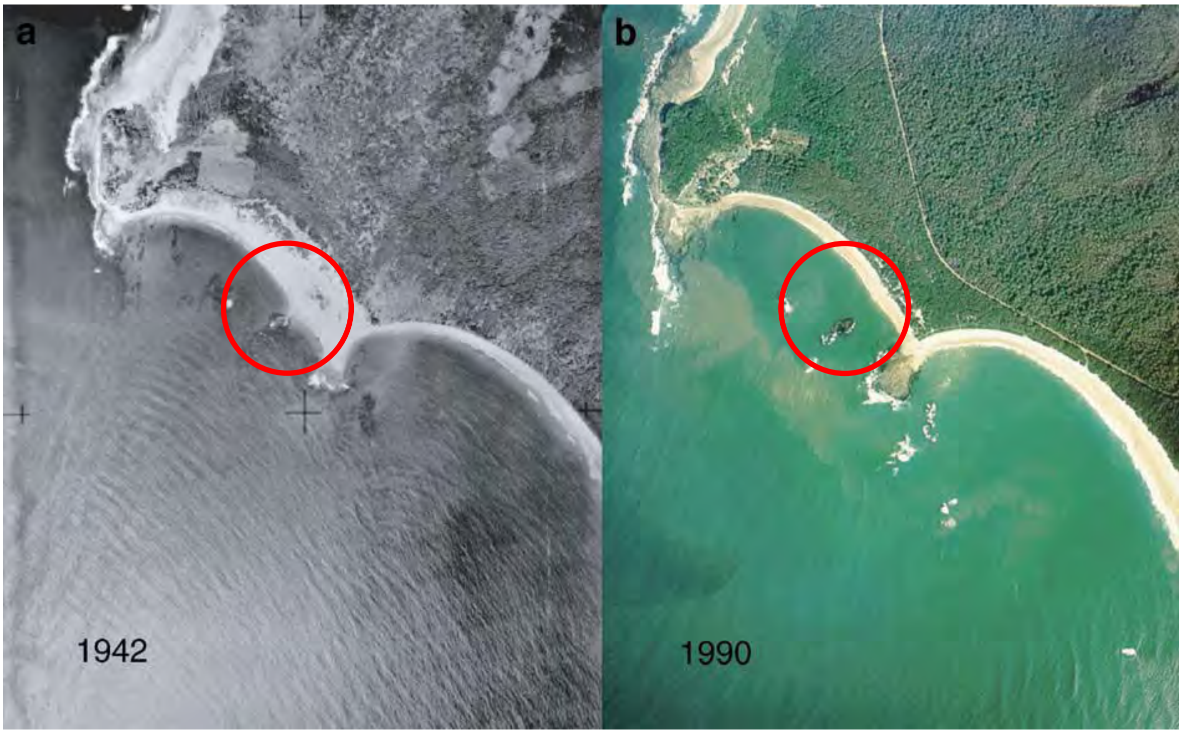


Figure 6-14: Aerial photographs taken in (a) 1942 and (b) 1990 at Woody Bay, NSW illustrates an example of salient loss. (Source: Goodwin et al., 2006)

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.

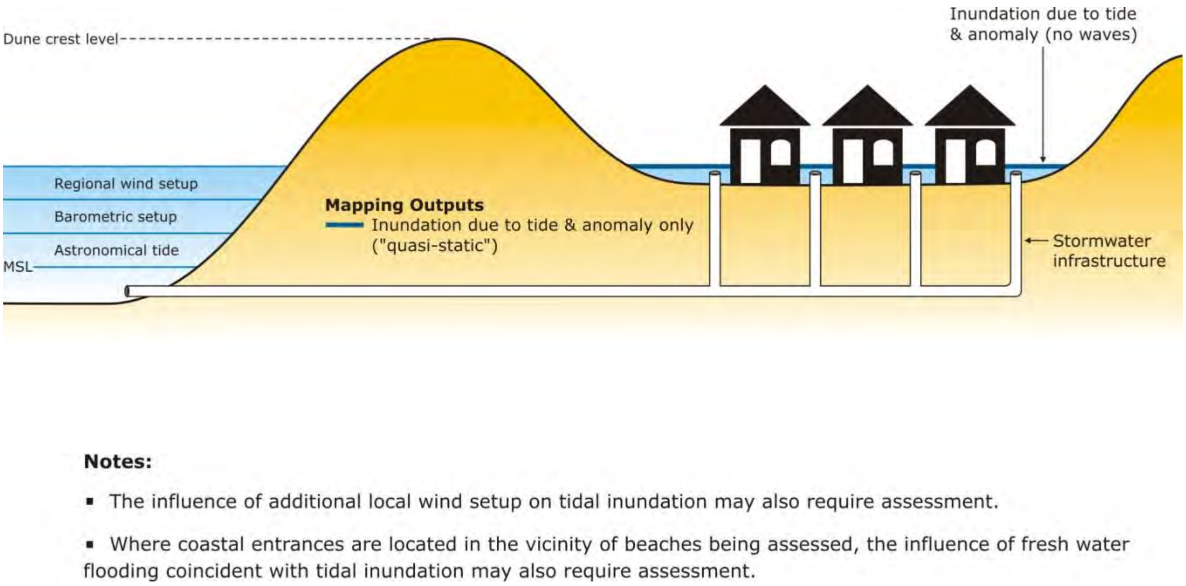


Figure 7-1: Components of Inundation Without Wave Effects

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.

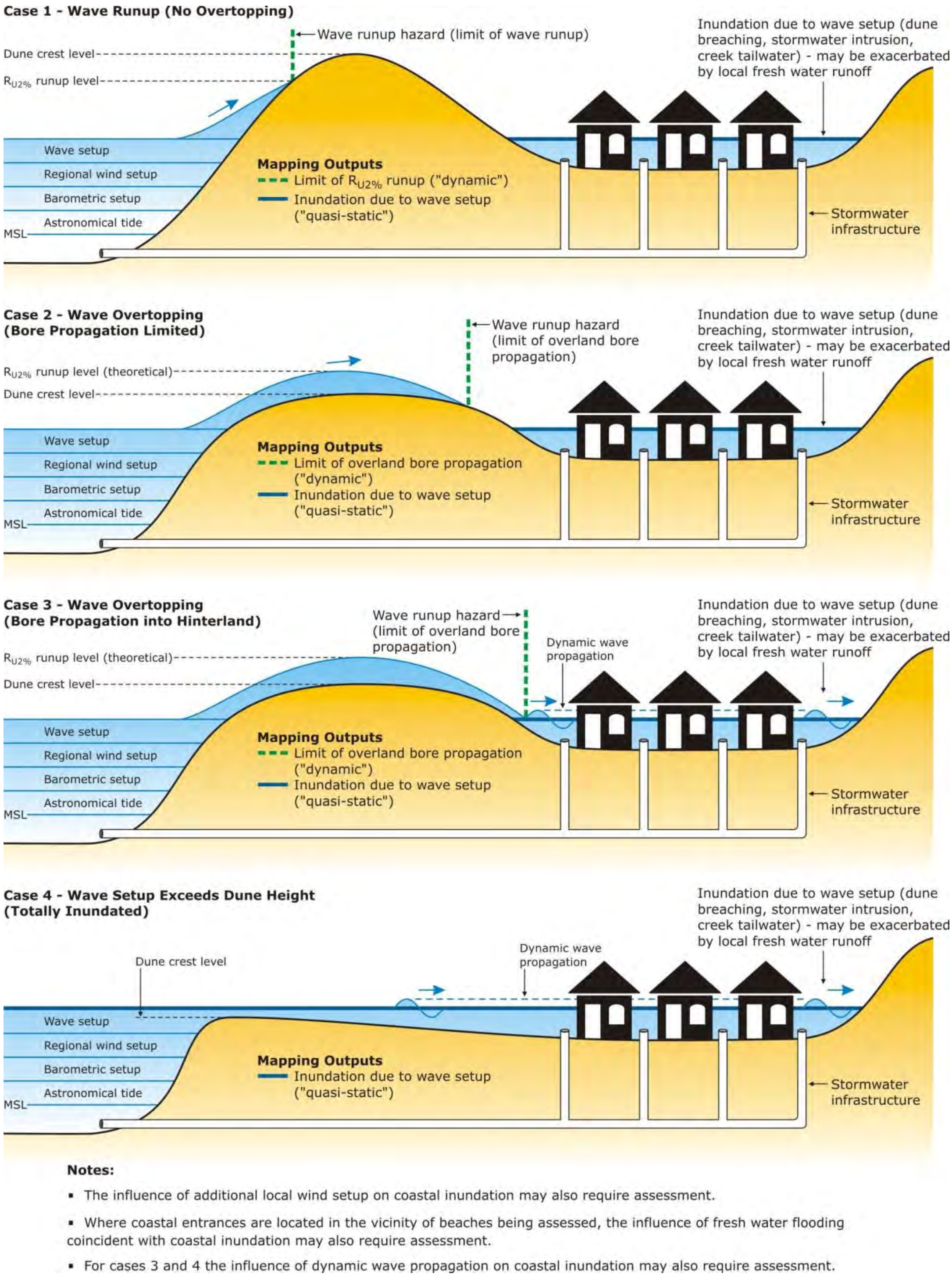


Figure 8-1: Components of Coastal Inundation

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.

$$H_{RMS} = 0.706 \times H_S \tag{8.1}$$

Figure 8-2: SBEACH profiles - northern area

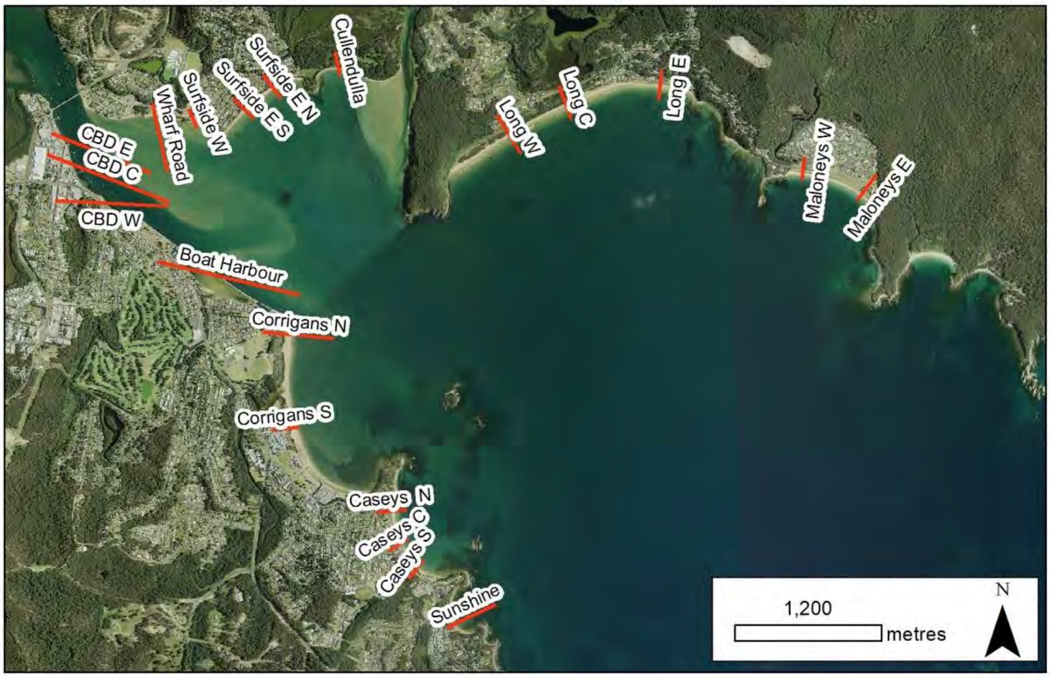


Figure 8-3: SBEACH profiles - inner Batemans Bay

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.

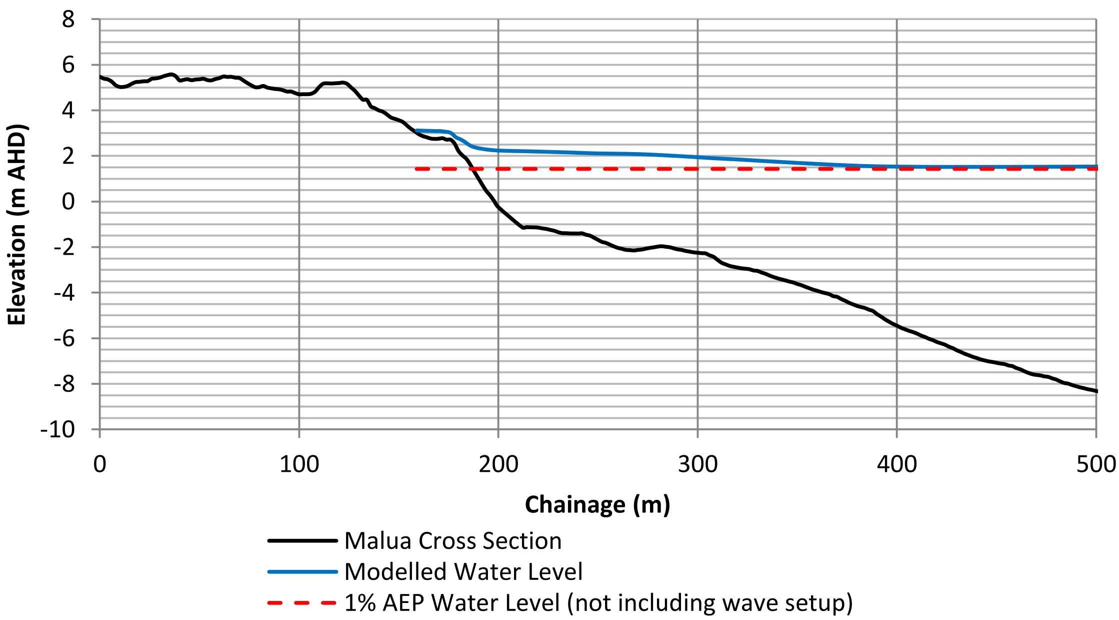


Figure 8-5: Example of SBEACH wave setup modelling at Malua Bay

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.

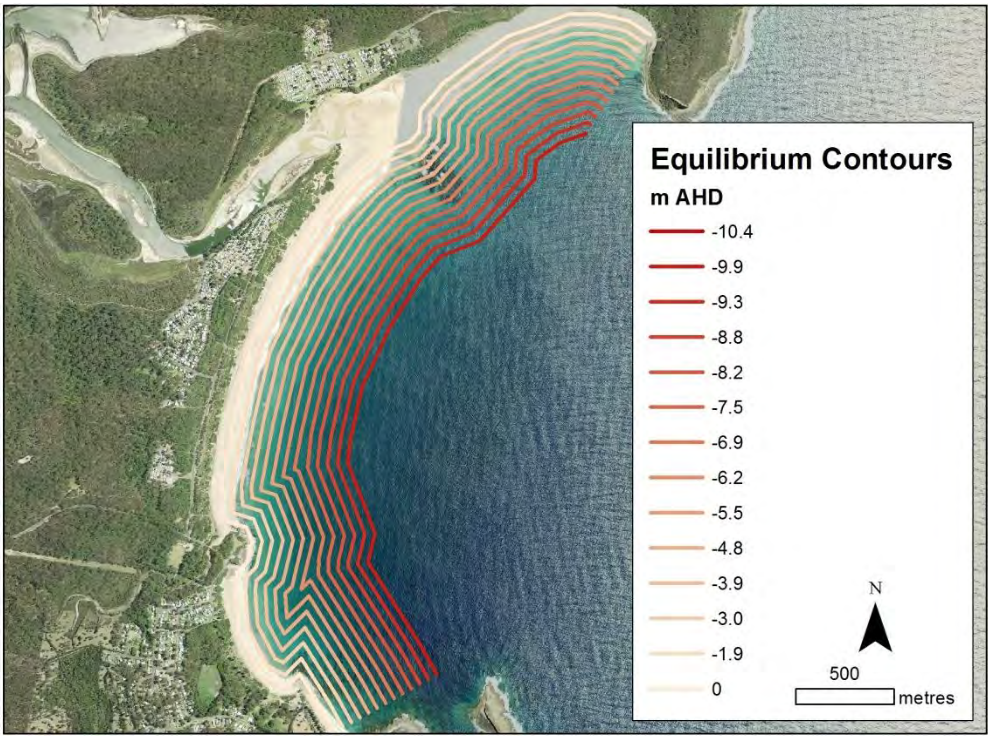


Figure 8-6: Dean equilibrium contours for Durras Beach and Cookies Beach

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.

$$R_{2\%} = 1.86\xi_0^{0.71} H_0 \quad (8.2)$$

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.

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

With a maximum of:

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

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

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

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).

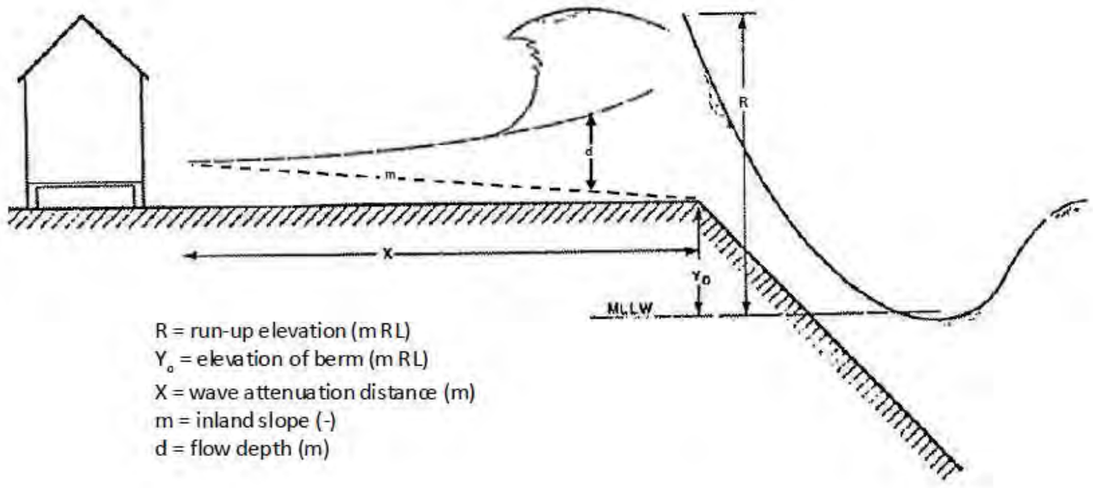


Figure 8-7: Bore propagation (Source: Tonkin & Taylor, 2016b and Cox and Machemehl, 1986)

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

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 5th-6th 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)

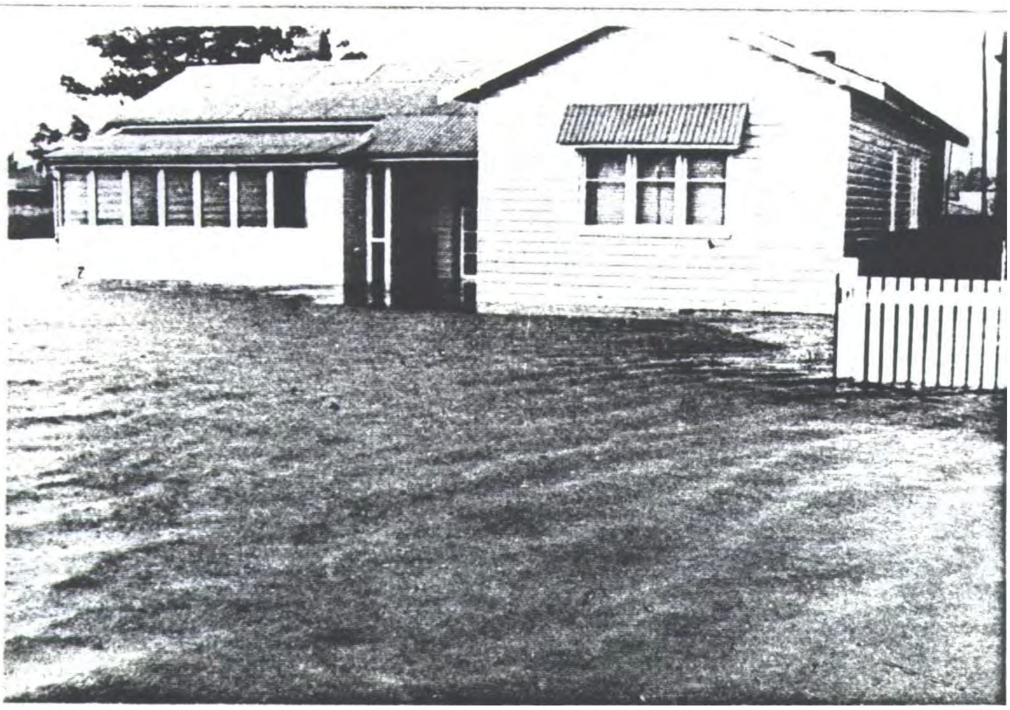


Figure 8-13: Corner of Bavarde Avenue and Golf Links Drive (Hanging Rock) 29-30 August 1963 (NSW PWD, 1989)

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.

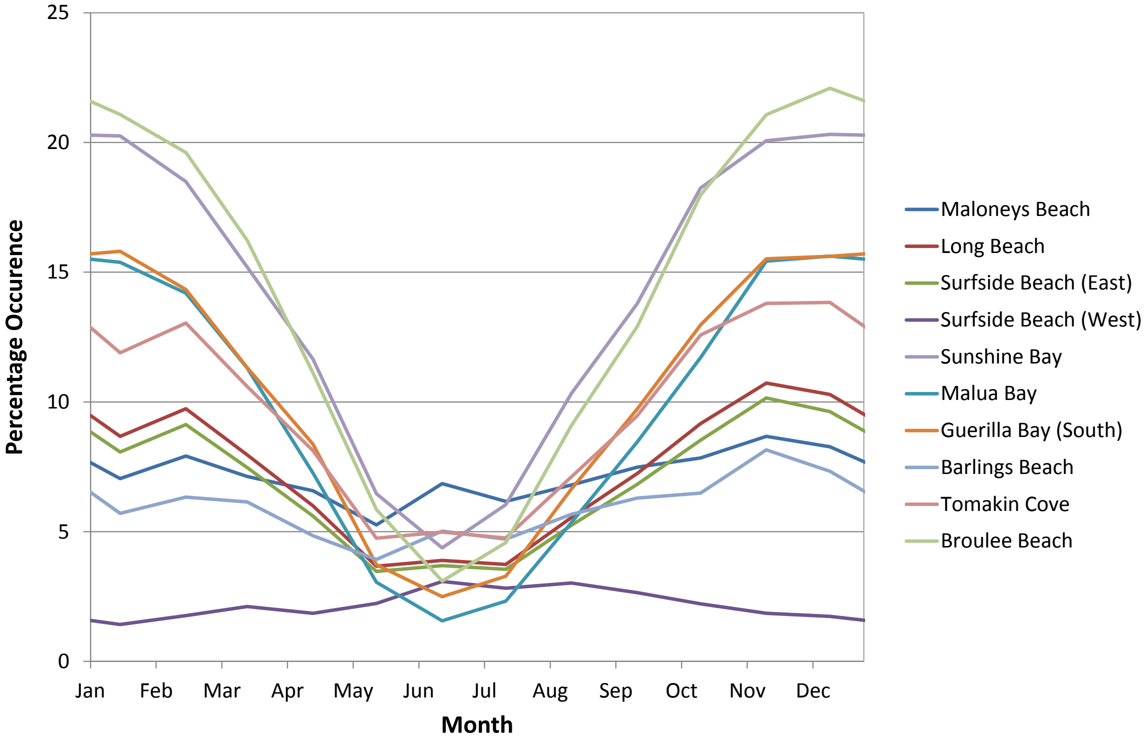


Figure 9-1: BoM Moruya Heads Pilot Station Daily Average Wind - Occurrence of Winds for Dune Building – Dry Sand

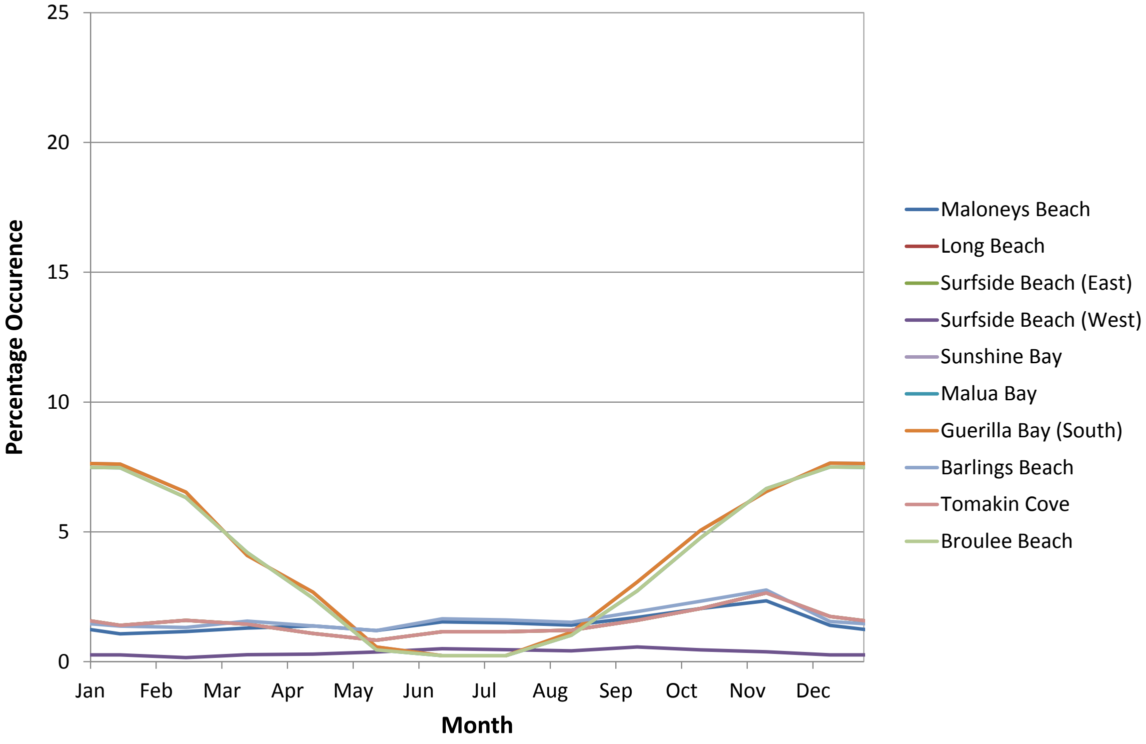


Figure 9-2: BoM Moruya Heads Pilot Station Daily Average Winds - Occurrence of Winds for Dune Building – Wet Sand

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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Appendix A: Literature Review

A.1 Preamble

A substantial body of literature in the form of consultant and government technical and management reports exists for beaches within Batemans Bay, but there is a paucity of coastal science and engineering literature in the wider Eurobodalla local government area (LGA). All the available literature addressing coastal processes, coastal protection works and coastal management within the Eurobodalla LGA was consulted. This included the management of risks to public safety and built assets, as well as risks from climate change. A brief summary of key documents (where it is relevant to the study area and the scope of the Coastal Hazard Assessment) is presented in the following discourse. The quality and reliability of the data and information was also assessed. Historical context to contemporary issues was provided where possible.

A.2 The Persistence of Rip Current Patterns on Sandy Beaches (Eliot, 1973)

This conference paper outlined the results of 20 current measurement campaigns undertaken at South Durras Beach over 37 days in November and December 1972. Measurements were taken at 50 m intervals covering the full 2.25 km length of the beach. Analysis of the current measurements was used to infer nearshore water circulation patterns. The number of rips along South Durras Beach varied with incident wave energy, incident wave direction and other parameters affecting the longshore current velocity. The range of the number of rips observed along South Durras Beach as a function of incident wave height and direction is shown in Table A-1. The data indicated that there was an inverse relationship between the prevailing energy conditions on South Durras Beach and the number of rip currents which occurred along it. The average rip spacing under high energy conditions (wave height > 1.5 m) was 905 m and 200 m under low energy conditions (wave height > 1.5 m). It was noted that there were places where rips tend to occur frequently and that these places appeared to be regularly spaced. It was also noted that the more permanent rip locations were those established during high energy conditions. The drop in the number of rip currents from low to high energy conditions was accompanied by a widening of the surf zone. During low energy conditions, the width of the South Durras Beach surf zone varied from 75 to 125 m. For high energy conditions, the surf zone width was approximately 200 m.

Table A-1: Number of Rips along South Durras Beach (Source: Eliot, 1973)

Wave Direction	Number of Rips	
	Wave Height < 1.5 m	Wave Height > 1.5 m
N to ENE	7-9	2-3
ENE to ESE	6-9	2-1
ESE to S	7-11	2-3

The forms which occurred along the low water line on South Durras Beach were sinuate in shape. Their wavelengths (the distances between projections) varied from 75 to 425 m. Ninety per cent of them had wavelengths between 75 and 250 m. The projections were located landward of sandbars and shoals and the depressions landward of pools, troughs and feeder channels. There appeared to be no direct relationship between the nearshore water circulation system and the forms that developed along the shoreline. That is, the rip currents did not show any consistent locations with respect to projections or depressions along the shoreline.

A.3 Seasonal Beach Change, Central and South Coast, NSW (Thom, McLean, Langford-Smith and Eliot, 1973)

This conference paper related beach surveys at South Durras Beach and Bengello Beach with observed weather systems during 1972. The surveys at South Durras Beach were those described in detail by Eliot (1973). The surveys at Bengello Beach included four profiles from the foredune to the offshore bar measured at fortnightly intervals. The envelope of profile change relative to Mean Low Low Water (MLLW) ranged from 2 m in elevation to 56 m horizontally. The mean change in volume between successive fortnightly surveys was 25 m^3/m (with range of 7 to 85 m^3/m). During 1972, Bengello Beach generally built upwards and seawards although phases of short-term erosion were noted. The envelope of profile change at South Durras Beach was quite similar to that at Bengello Beach with an overall accretionary trend during 1972. However, recovery after an erosion phase was more rapid at South Durras Beach when compared to Bengello Beach.

A.4 Beach Changes at Moruya, 1972-1974 (McLean and Thom, 1975)

This conference paper related beach surveys undertaken at Bengello Beach with observed weather systems from January 1972 to October 1974. Bengello Beach was described as being a relatively undisturbed, crescent-shaped beach facing slightly south of east and exposed to moderate to high energy waves emanating from directions between NE and S. Headlands at the extremities of the beach cause refraction of ocean swell from the north and south and act as barriers to littoral drift from adjacent beaches. The active beach is backed by a series of parallel relict beach ridges (or foredunes) 5 to 8 m high which have accumulated since the Postglacial Marine Transgression. Sediments at Bengello Beach were described as predominantly well sorted, fine to medium grained (d_{50} range of 0.15 to 0.35 mm) clean quartz sands; the proportion of shell being less than 10% on the sub-aerial portion of the beach, although it increases seawards of this zone. Waldrons Swamp is located landward of the active beach. It drains Waldrons Creek towards the northern end of Bengello Beach.

The analysis for 1972 was presented in WRL's review of Thom et al (1973) and is not reproduced for brevity. From January to June in 1973, Bengello Beach continued to accrete. However, in mid-June 1973, the beach was severely depleted by a storm. The authors identified that the storm in June 1973 marked an abrupt change from an accretionary period to an erosional regime. For the remainder of 1973 and into 1974, the general tendency was one of gradual depletion. In February, March and April 1974, storms further eroded Bengello Beach which was left relatively undernourished. Bengello Beach was then further changed dramatically during late May and June 1974. Over three weeks of successive storms, with two major storms from 24-27 May and 9-15 June, the mean change in volume was 130 m^3/m above -0.94 m AHD. From July to October 1974, Bengello Beach was observed to begin recovery. The envelope of profile change relative to Mean Sea Level (MSL) ranged from over 3 m in elevation to 60 m horizontally from January 1972 to October 1974. Finally, the authors asserted that frequent monitoring of one beach which is considered "representative" of the region will shed more light on temporal variations than infrequent monitoring of many beaches. As such, extrapolation of behaviour at Bengello Beach to other beaches in the region was considered to be reliable.

A.5 Observations of Resonant Surf and Current Spectra on a Reflective Beach and Relationships to Cusps (Wright, Thom, Cowell, Bradshaw and Chappell, 1977)

This journal paper outlined observations of inshore wave and current behaviour at McKenzies Beach on 9 December 1976. It was described as being a pocket beach with a steep linear beachface with slopes of 1V:7H to 1V:10H. It was noted that a gravel step is consistently present at the subaqueous base of the beach face, and beach cusps are invariably present. This experiment provided evidence that beach cups are related to low-mode edge waves which oscillate parallel to as well as perpendicular to the beach (strong inshore resonance).

A.6 Batemans Bay Waterway Planning Study (Laurie, Montgomerie and Pettit, 1978)

This report by Laurie, Montgomerie and Pettit examined hydraulic and engineering aspects of Batemans Bay to inform its use and management. It attempted to delineate between sensitive and inherently stable areas. The report provided a preliminary plan for conservation and future development with respect to ecology and urban planning. It was asserted that care for the Clyde River and Cullendulla Creek requires skilful management for effects on the inner bay due to erodible catchment slopes. The Clyde River was described as a well-mixed estuary system with a wide and deep mouth and extensive headwaters draining a basin with an area in excess of 1,600 km^2. The principal characteristics of the area were deemed to be:

- exposure to easterly storms;
- navigation restrictions imposed by the Clyde River bar and extensive sand shoals in the inner bay;
- relative frequency of high river discharges; and
- topographical limitations on public access to the water.

The report defined the inner bay as the region between the highway bridge and a line between Square Head and Observation Head. This is equivalent to the current study area with the exclusion of Maloneys, Long and Caseys Beaches. The inner bay was found to be a complex area with the greatest degree of fluvial and marine interaction. Sediment in this area was found to be largely fluvial in origin but bi-directionally forced by the tides, river flows and wave action. This influx of riverine sands is from the Clyde River, smaller creeks and rapid weathering of local headlands. Sand in the inner bay is finer than in the outer bay, with the coarsest sand accumulating on the Clyde River bar and at the northern end of Corrigans Beach. Following construction of the training wall in the early 1900s, the river mouth bar moved further east and accretion occurred behind the wall and at the northern end of Corrigans Beach. Erosion of the inner bar northern shoreline was observed during this time. The Clyde River bar is mobile but generally located within a few hundred metres east and north of the end of the training wall. As early as 1864, bathymetric charts show it in this same position even prior to dredging and construction of the training wall. Commercial shipping services ceased in 1955. In 1964, dredging was stopped since, although desirable, it was not economically practical. Infilling of the boat harbour on the south side of the inner bay was deemed to be primarily due to wind and wave transport through the entrance and to a lesser extent wave overtopping and sediments from Hanging Rock creek. Virtually the whole north side of the inner bay was considered to be in a state of instability or fragile stability and not suited to development of waterfront structures other than for several hundred metres downstream of the highway bridge. The most severe conditions for erosion were deemed to be when flooding and associated channel scour occurred just prior to a large wave event. The south side of the inner bay was generally considered stable. The report noted that accretion at the northern end of Corrigans Beach had seemed to have temporarily ceased but that it may occur again in the future and recommended ongoing monitoring in this area. The report also cautioned against development in the Cullendulla Creek catchment, which is a shallow estuary in its own right with a small input of freshwater. This area is a depositional plain with unique geomorphic qualities. It has "chenier-like" plains of sand-shell ridges separated by saltmarsh and mangrove flats. The report recommended that it should be protected for its geomorphic uniqueness, rich oysters, flora and Aboriginal middens. It was noted that between 1864 and 1899, the location of the Cullendulla Creek mouth moved westward from its current position by 200 m during a period of accretion, but had returned to its present position by 1922.

In the outer bay (seaward of a line between Square Head and Observation Head), there was little evidence of long term variations in bathymetry between 1893 and 1960. The beaches on the southern side (including Caseys Beach) were generally described as pocket beaches between rocky headlands with minor depositional plains from creeks. The southern beaches were noted to be generally protected from southerly, south-easterly and westerly storms with highest wave impacts during summer. Shell (and hence sand) production in shallow waters offshore of the southern beaches was considered to be effective in maintaining beach sediment budgets. It was recommended that dredging not be undertaken in this area. Caseys Beach was described as an independent sediment unit with little longshore movement of sand beyond the platforms and headlands at both ends of the beach. On the northern side of the outer bay, Maloneys and Long Beaches were considered not be influenced by river mouth processes; with erosion and accretion only occurring due to wave variability. It was recommended that building and construction at Maloneys, Long and Surfside Beaches should be avoided and, where practicable, the width of the foreshore reservation be extended to at least 100 m.

A significant storm in June 1975 was described with overtopping and damage to structures and vessels. It was considered that seiching may have occurred in the inner bay. High flows in the Clyde River were noted to mainly pass under the northern half of the highway bridge before heading towards the south side of the inner bay and along the training wall. This sudden channel width expansion also caused high velocity eddy currents downstream of the highway bridge.

The report considered a series of proposals to improve boat moorings including:

- a breakwater at the southern end of Caseys Beach;
- a marina at Corrigans Beach;
- a breakwater wall just downstream of the highway bridge on the northern bank; and
- dredging and improvement works behind the training wall (to raise the crest).

A.7 Surf-Beach Dynamics in Time and Space – An Australian Case Study, and Elements of a Predictive Model (Chappell and Eliot, 1979)

This journal paper outlined the results of 20 beach survey campaigns undertaken at South Durras Beach over 37 days in November and December 1972. These were undertaken in parallel with the current measurements outlined in Eliot (1973). Profiles were taken at 50 m intervals covering the full 2.25 km length of the beach. South Durras Beach is described as being a medium to high energy surf beach. The beach fronts a Holocene barrier structure which test drilling has shown to have a 25 m thickness above bedrock. The beach sediment is dominated by medium sand compromised largely of shell carbonate and quartz. The bathymetry offshore of South Durras Beach is inherited from Pleistocene subaerial erosion subdued by

Holocene sediment cover, is moderately complex and refraction thus significantly affects the longshore distribution of wave energy. It was noted that the inshore morphology and circulation patterns are very changeable and the beach is not homogenous along its length. Statistical analysis of the inshore morphology behaviour through varying energy conditions and modelling of the general inshore/nearshore profile under different wave energies was also presented.

A.8 Experimental Control of Beach Face Dynamics by Water-Table Pumping (Chappell, Eliot, Bradshaw and Lonsdale, 1979)

This journal paper outlined the results of the first known field experiments of beach groundwater manipulation undertaken in Australia at South Durras Beach. Beach groundwater manipulation, or beach dewatering, is an alternative to more traditional coastal stabilisation methods. Beach dewatering consists of the artificial lowering of the groundwater table with its proponents suggesting that this results in enhancing infiltration losses during wave uprush/backwash cycles while promoting sediment deposition at the beach face. Two beach dewatering experiments were undertaken on a 150 m long segment of South Durras Beach 7 October 1973 and 22 January 1975. An array of wells plus a large pump were used to regulate the intertidal beach water table while inshore and nearshore morphologies, water circulation and sedimentary processes were monitored adjacent to and away from the well array. The first experiment involved four pumped wells at 2 m centres while the second involved 24 wells at 1.5 m centres. The experiments indicated that beach dewatering has potential as an effective means of beach stabilisation.

A.9 Surf Zone Resonance and Coupled Morphology (Chappell and Wright, 1978)

This conference paper discussed the results of field experiments involving direct measurements of inshore current spectra, inshore circulation patterns and depositional morphology at McKenzies Beach and Bengello Beach. For brevity, WRL has not reviewed this paper as its content is discussed in greater detail in Wright et al (1979) and Wright (1982).

A.10 Morphodynamics of Reflective and Dissipative Beach and Inshore Systems: Southeastern Australia (Wright, Chappell, Thom, Bradshaw and Cowell, 1979)

This journal paper compared the results of field experiments involving direct measurements of surf and inshore current spectra, inshore circulation patterns and depositional morphology at McKenzies Beach, Broulee Beach and Bengello Beach. With the exception of McKenzies Beach which is composed of a bimodal population of sand and gravel, the beaches are primarily composed of medium sand.

McKenzies Beach was described as being a relatively high energy, reflective beach. Runup (relative to breaker amplitude) was noted as being high. Two experiments examining the spectral characteristics of and cross-spectral relationships between water surface and horizontal flow oscillations at different locations in the inshore system were conducted on 9 December 1976 (see Wright et al, 1977) and 26 May 1977. Wave data measured at McKenzies Beach showed pronounced narrow spectral peaks centred at swell frequencies. The peaks were noted to be conspicuously narrower and sharper than is the case for Broulee Beach and Bengello Beach, owing to sheltering.

Broulee Beach was described as being a partially protected dissipative beach. It is sheltered from the dominant south-easterly swell and from the south-easterly storm waves and exhibits a narrow range of temporal variability (compared to Bengello Beach), typically having a low tide terrace beach typography year round. An experiment conducted at the northern end of Broulee Beach on 31 July 1976 was discussed.

Bengello Beach was described as being a relatively high energy, dissipative beach. It is long and weakly embayed with the full spectrum of beach typographies evident along it. Wave exposure is greatest in the middle of the beach, slightly reduced at the northern end and most protected at the southern end. Two experiments conducted at the northern end of Bengello Beach (30 July 1976 and 27 May 1977) and three experiments conducted at the middle of the Bengello Beach (8 December 1976, 24 May 1977 and 25 May 1977) were discussed.

Runup (relative to breaker amplitude) at Broulee Beach and Bengello Beach was noted as being lower than at McKenzies Beach. Wave spectra from the surf zones at Broulee Beach and Bengello Beach showed significant energy at a much wider range of frequencies than at McKenzies Beach.

A.11 Field Observations of Long Period, Surf-Zone Standing Waves in Relation to Contrasting Beach Morphologies (Wright, 1982)

This journal paper extended the work presented by Wright et al (1979) at McKenzies Beach and Bengello Beach. In addition to the field experiments on 9 December 1976 and 26 May 1977 at McKenzies Beach, results from supplementary experiments on 10 and 11 December 1977 were outlined. In addition to the experiments at the northern end of Bengello Beach (30 July 1976 and 27 May 1977), results from a more extensive experiment on 12-14 December 1977 were presented. Analysis of the measurements of surf and inshore current spectra, inshore circulation patterns and depositional morphology and their inter-relationships were set out.

A.12 Transgressive and Regressive Stratigraphies of Coastal Sand Barriers in Southeast Australia (Thom, 1983)

This journal paper discussed the stratigraphic characteristics of the coastal sand barrier at Bengello Beach. The author asserted that the series of parallel relict beach ridges, which back the active beach, were deposited during the Postglacial Marine Transgression. Radiocarbon dating results from sediment cores forming a cross-section through the middle of Bengello Beach were presented.

A.13 Batemans Bay Drainage Study (Willing and Partners, 1984)

This report by Willing and Partners concerns the construction of a shopping complex upstream of the Soldiers Club in the CBD. The catchment was considered to be a single valley with an area of 50 ha which discharges into the Clyde River with varying degrees of tidal inundation. During extremely high water levels, water was noted to back up in existing drainage works. Rainfall and runoff analysis and retardation effects were undertaken with the RAFTS (Runoff Analysis and Flow Training Simulation) numerical model. 10 and 100 year ARI rainfall events were considered. The design of the shopping complex was based on a river water level of 1.5 m AHD and required the infilling of an existing swamp which acted as a natural retarding basin. The report discussed the requirements of a new retarding basin to offset this impact and other necessary drainage requirements.

A.14 Coastal Storms in NSW in August and November 1986 (Higgs and Nittim, 1988)

This report by WRL documented 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.

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.

The location and elevation of maximum runup were pegged and surveyed after both storm events and are shown in Table A-2.

Table A-2: Runup Levels During 1986 Storms

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.15 Batemans Bay Inundation Study (Willing and Partners, 1988)

This report by Willing and Partners followed the 1984 Drainage Study (Willing and Partners, 1984). It reviewed the 100 year ARI oceanic still water level at the CBD (2.60 m AHD) and recalculated flood levels with 1, 5, 20, 50 and 100 year ARI rainfall for additional flooding impacts. It was noted that if the 100 year ARI rainfall was coincident with the 100 year ARI oceanic still water level, the CBD tail water level would rise by 0.16 m. As such, the effect of additional rainfall under such an oceanic flooding event was considered minimal.

A.16 Batemans Bay Oceanic Inundation Study (NSW PWD, 1989)

This report by the NSW Public Works Department was commissioned to determine the likely water levels during extreme storm events in Batemans Bay. The bay was described as funnel shaped; reducing from 5 km width near the Tollgate Islands to approximately 500 m at the Princes Highway bridge. Most of the beaches, dunes and hind dunes are typically 2 m AHD. Oceanic flooding had historically occurred at Surfside Beach, Wharf Road, the CBD, the boat harbour (east and west), Corrigans Beach and Caseys Beach. At the time of writing, the mid-range sea level rise estimate was described as 1.0 m by 2100 but this was not taken into account in the calculated design water levels. The area was considered to be tectonically stable and the impact of tsunamis was not considered.

A brief outline of historic oceanic inundation and river flooding was presented. To the south-east of Wharf Road, a survey in 1898 showed that a high sand spit existed 1.5 m above the high water mark. However, in 1959 this sand spit (and the associated subdivisions) were washed away during a flood event coinciding with spring tides. On 22 May 1960, a severe earthquake in Chile triggered a tsunami that caused oscillations of approximately 0.84 m at 45 minute intervals below the highway bridge. In August 1963, flooding occurred mainly due to rainfall combined with a high tide. In the storms during May and June in 1974, the peak still water level at Wharf Road was observed as 1.5 m AHD, with runup exceeding 3.4 m AHD at Surfside Beach. In June 1975, 90 m of Beach Road at Caseys Beach was damaged due to wave overtopping. In June and July 1984, wave overtopping and sand deposition occurred along Beach Road and the CBD foreshore (peak H_S of 5.6 m). In August 1986, waves overtopped the culvert at McLeod Street on the northern shoreline of the inner bay. In November 1986, wave runup was within approximately 0.2 m of the seawall crest of the CBD. The highest still water level observed in Batemans Bay is approximately 1.85 m AHD at the Princes Highway bridge (date unknown).

This study focused on storm events with significant offshore wave heights greater than 5 m. A bathymetry survey of Batemans Bay was commissioned as part of the project. Water levels were derived at 17 locations around the bay through a series of modelling exercises. Storm surge (determined by Monte Carlo analysis) was found to be common to all parts of Batemans Bay, but other components of elevated water levels (such as wave setup and river flooding) may vary. While joint probability analysis was undertaken for the ocean water levels, the probability of their occurrence with river flooding was not included in the simulations. However, it was noted that some dependency exists between the occurrence of river floods and elevated ocean water levels. A hydraulic flood model was constructed to determine the contribution of flooding to elevated water levels between Surfside Beach and the boat harbour. Wave setup was found to be greatest at Maloneys and Long Beaches. Northerly winds were found to be unlikely to generate high elevated water levels as they generate an offshore current due to the Coriolis force. A 0.3 m uncertainty factor was applied to each of the design water levels. Wave runup was then calculated for each of the 17 locations based on the 20 and 100 year ARI wave events (determined from 5 years of wave data at Jervis Bay, 1982-1986). Except at the western end of Long Beach, wave runup exceeded the nominal crest level at each location for the 100 year ARI event. Importantly, at Cullendulla Beach, Wharf Road, the CBD, the boat harbour and the southern end of Corrigans Beach, the 100 year ARI design still water level is above the nominal crest elevation. At these locations, the crest would be inundated even without wave runup. The crest levels would need to be raised by 1 to 4 m to prevent inundation and wave overtopping.

Finally, due to the protection offered by Square Head, the modelling indicated that a wave setup (and consequent pressure head) differential exists between Surfside and Cullendulla Beaches. It speculated that this difference in head drives a current which continues to supply sand to the shoal on the western side of Square Head.

A.17 Joes Creek Flood Study (Willing and Partners, 1989a)

This report by Willing and Partners reviewed present and future flooding conditions for Joes Creek as a result of the proposed George Bass Drive extension. Joes Creek catchment has an area of 536 ha, discharges under Beach Road and terminates at Corrigans Beach. The RAFTS numerical model was used to simulate the 5, 20, 50 and 100 year ARI rainfall events. Modelling was undertaken with three different tail water conditions: 0.94 m AHD (High High Water Solstices Springs tidal level which occurs approximately 3 times per year), 2.25 m AHD and 2.55 m AHD. The latter two tail water conditions included wave setup and were derived from the Batemans Bay Oceanic Inundation Study (NSW PWD, 1989). Peak flood levels were determined for a number of outlet configurations before and after the road alignment for George Bass Drive.

A.18 Short Beach Creek Flood Study (Willing and Partners, 1989b)

Short Beach Creek catchment has an area of 350 ha and an outlet at the southern end of Caseys Beach. A tributary to Short Beach Creek flows past a caravan park (Caseys Beach Holiday Park) and joins the creek approximately 200 m upstream of the outlet at the beach. After recent flooding, this report by Willing and Partners was initiated to investigate the sufficiency of five pipe culverts under Sunshine Bay Road and also consider the future effects of the proposed George Bass Drive extension. The RAFTS numerical model was again used to simulate the 5, 20, 50 and 100 year ARI rainfall events. Modelling was again undertaken with three different tail water conditions: 0.94 m AHD, 2.43 m AHD and 2.70 m AHD. It was noted that the bridge over Short Beach Creek acts as a control point for upstream water levels. Modelling also considered the build-up of sand blocking the outlet with sand bar elevations between 1.40 and 3.20 m AHD considered. The bar was expected to scour out during minor floods and hence the risk of Beach Road acting as an overland spillway is minimal. It was noted that tail water conditions lower than 0.94 m AHD did not affect upstream water levels as critical depth is achieved immediately downstream of the bridge. A range of short and long term flood mitigation options were set out for reducing post-development flows to pre-development values.

A.19 Batemans Bay Oceanographic and Meteorological Data (MHL, 1990)

This report by Manly Hydraulics Laboratory describes a range of data collected at Batemans Bay between 1986 and 1989 for the Batemans Bay Oceanic Inundation Study (1989). The data collection project involved commissioning a network of data recorders to measure offshore and inshore waves, offshore and inshore tides, wave runup and wind data. Waves were recorded at the newly installed offshore buoy and inshore on a Zwarts pole near Snapper Island. Tides were measured near the Tollgate Islands, Snapper Island and at Princess Jetty (CBD). Poles were used to measure wave runup at Long and Surfside Beaches. It was found that the tides recorded at Princess Jetty correlated well to Snapper Island except during floods and within periods with strong onshore winds where higher tidal anomalies were recorded at the CBD. Generally, the tide at Snapper Island leads Princess Jetty by approximately 22 minutes with a slight reduction in amplitude due to energy loss over the sand shoals. The most intense storm during the data collection period occurred on 23-24 May 1988 during a neap tidal cycle. The tidal anomaly was 0.23 m offshore and 0.15 m inshore. The deep water H_S was 3.9 m and 2.1 m at Snapper Island. However, the maximum wave runup level recorded at Surfside Beach during this storm was only 1.0 m AHD.

A.20 Behaviour of Beach Profiles During Accretion and Erosion Dominated Periods (Thom and Hall, 1991)

This journal paper discussed beach surveys undertaken at Bengello Beach from January 1972 to December 1987. Analysis from January 1972 to October 1974 was presented in WRL's reviews of Thom et al (1973) and McLean and Thom (1975) and is not reproduced for brevity. It was noted that beach surveys had been undertaken fortnightly up until January 1976, after which time they were undertaken monthly. An erosion dominant period including the May/June storms of 1974 extended to June 1978 (when the beach reached its most eroded state since measurements commenced) after which Bengello Beach returned to an accretion dominated period up until the latest available surveys (December 1987). The maximum accretionary rate was 0.419 m^3/m/day but 0.120 m^3/m/day was typical. It was noted that the subaerial beach volume had remained approximately constant from 1981 to 1987. The maximum change in beach volume above -0.94 m AHD varied between 279 and 298 m^3/m between 1972 and 1987.

The authors asserted that the pre-1974 beach may not have been indicative of a long-term equilibrium beach. It was suggested that the mean beach volume in 1981 was approximately equivalent to the mean beach volume in 1973. A small amount of additional accretion from 1981 to 1987 was attributed to sediment contributions from offshore of Bengello Beach. The authors noted that additions to the compartment's total sand store from external sources (e.g. the Moruya River or alongshore) were considered insignificant but that further work was required to conclusively determine this. The Moruya River, which terminates at the southern end of Bengello Beach, experienced large-scale flooding in 1975 and 1976.

A.21 Reed Swamp – Long Beach Flood Study (Willing and Partners, 1991)

Reed Swamp is located behind Sandy Place at Long Beach. It has a catchment area of 136 ha and a wetland which occupies 33 ha of which 5.4 ha is a permanent lagoon. This report by Willing and Partners revised the flood levels presented in previous studies. The existing culverts under Sandy Place were found to be inadequate to discharge a design flood event within the existing drainage channels. The study estimated the 5, 20 and 100 year flood levels and investigated upgrade options for the culverts including protection and augmentation. In June 1991, high flows (estimated to be greater than a 20 year ARI rainfall event) bypassed the culvert and flowed through adjacent properties to Long Beach. It was noted that the outflow from Reed Swamp is primarily governed by downstream tail water levels.

A.22 Land at Cullendulla Creek, Surfside (Patterson, Britton and Partners, 1992)

This report by Patterson, Britton and Partners reviewed oceanic inundation, beach stability and stormwater drainage at Cullendulla Beach. It is an engineering assessment concerning a proposed caravan park development in the lee of the beach. Specifically it reviewed the results of a Local Environmental Study (LES) commissioned by ESC (Kinhill Engineers, 1990).

The LES found that flood flows from Cullendulla Creek were not sufficient to generate water surface gradients and increase tailwater levels under oceanic inundation. The coastal engineering report commented that wave setup at Cullendulla Creek was expected to be lower than on the adjacent beach. The report also discussed at length the 0.7 m difference in design inundation levels between Surfside and Cullendulla Beaches (PWD, 1989) and the potential for overland flow between the two. It was asserted that the Cullendulla Creek estuary essentially behaves as a flood storage basin. The potential for an increase in storage level is determined by the discharge capacity of the Cullendulla Creek outlet relative to overland flow from Surfside Beach. It was concluded that since the Cullendulla Creek outlet is very efficient, the rise in water level from any overland flow would be less than a few millimetres.

With regard to beach stability, the report commented that the inner part of Batemans Bay is essentially a closed sediment system. It asserted that the exchange of sediments between the inner bay and the outer bay is not significant except for very fine to fine sand transported into the outer bay during flood events. Historically, the inner bay may actually have been a mud basin separated from the ocean by a barrier. Approximately 3,000 years before present, this barrier failed and the inner bay was connected to the ocean. The report considered sediments from Cullendulla Creek to be a minor if not insignificant source of sediment for the Square Head shoal seaward of the eastern end of the beach. The main contributors to sediment at Cullendulla Beach were deemed to be the Clyde River and shells produced offshore. The report hypothesised that Cullendulla Creek receives less sediment than the adjoining Surfside Beach, with little littoral exchange between the two. During storms, it was postulated that a mega-rip would tend to form against the Square Head shoal. Westerly winds were deemed to be very effective at generating littoral drift between the western end of Cullendulla Beach and the Square Head shoal. The report asserted that the historical connection of Hawkes Nest to the western end of Cullendulla Beach was not the cause of ongoing recession as progradation had previously occurred between 1864 and 1930 under this arrangement. From 1930 to 1990, Cullendulla Beach receded by 40-60 m (typical) and up to 100 m its eastern end. The report noted that the 1990 shoreline position was still located seaward of the 1864 shoreline. A review of photogrammetric data between 1942 and 1990 indicated typical recession of 1-1.2 m per year. Recession for the western and central parts of the beach (0.4-0.5 m per year) was lower than at the eastern end (1.0-2.0 m per year). Also, recent recession in the western and central parts of the beach was lower than the long term average, whereas the rate at the eastern end was consistent over the analysis period. It was asserted that erosion in the western and central parts of the beach were dominated by storm (swell) waves. Erosion in the eastern part was contended to be from local south-westerly and westerly wind waves. The annual total sediment loss from Cullendulla Beach was estimated to be 3,000 m^3 per year (1,000 m^3 per year above 0 m AHD). The report concluded that in the absence of a major flood or a series of smaller floods, Cullendulla Beach would continue to recede due to swell and wind wave attack. A beach management concept design involving a groyne field and nourishment was also set out. The preferred fill source for beach nourishment was sand extracted from the Square Head shoal.

Finally, the report undertook a preliminary review of stormwater drainage for the proposed development and noted that the detailed design should maximise natural infiltration and recommended that drainage be directed towards Cullendulla Creek and/or the wetland. It was noted that water quality control ponds would be required prior to drainage into the wetland.

A.23 Coastal Engineers Report, Timbara Crescent (Patterson, Britton and Partners, 1994)

This letter report by Patterson, Britton and Partners addresses the coastal hazards relevant to a private property at Timbara Crescent on the northern shoreline of the inner bay. A 50 year planning period was adopted and as no photogrammetry existed, the storm demand for the site was conservatively estimated to be 20 m^3/m. A conservative profile when the beach was slightly eroded (December 1986) was used as the average profile for determination of the hazard lines. It was noted that the present day sediment processes were both event driven (flood and coastal storms) and responsive to relatively sustained periods of accumulation or loss (over several decades). No long term recession was observed at the site. The 50 year design water level was adopted as 2.3 m AHD (from the Batemans Bay Oceanic Inundation Study, less the 0.3 m uncertainty allowance). Under these conditions, the relevant property would be inundated by water to a depth of up to 1.3 m with maximum breaking wave heights of 1.0 m. The best estimate of sea level rise for 2045 at the time of writing was 0.24 m and the Bruun rule was applied to estimate recession at the site. However, it was noted that the ongoing supply of sand from the Clyde River and offshore shell production may nullify shoreline recession due to sea level rise. The letter report recommended that development on the property should consider raised floor levels, structural members designed for wave loadings, the addition of a wall between the adjacent property to the east to prevent wave reflection impacts and preparation of a flood evacuation plan.

A.24 Coastal Processes of Cullendulla Creek (Short, 1995a)

This report is the first of two by Short concerning a revised tourism development proposal at Cullendulla Beach. It was commissioned by the NSW state government. This report discusses the coastal processes operating in the area and the impact of the proposed development on the natural processes. Cullendulla Creek is described as a barrier estuary containing a tidal creek, flats and delta together with a chenier beach ridge sequence and the modern beach. It was noted that oceanic inundation of the entire site will occur approximately every 20 years with an oceanic water level of 1.8 m AHD. Cullendulla Beach has a relatively steep reflective high tide beach (3°) fronted by a wide, low gradient low tide beach/terrace (1°). Maxima for recession were noted to occur at both the western and eastern ends of the beach (where there is shoreline instability from the creek entrance) with a minimum in the lee of the western side of the ebb tide delta. The author reviewed previous work in the area but asserted that there is insufficient information on coastal processes operating in the inner bay and at Cullendulla Beach to conclusively attribute the exact cause of recession and its future rate and duration. However, it is likely to be related to both wave and tidal current impacts on sediments in the inner bay. Recession was considered likely to continue for the next few years to decades. The report asserted that cycles of recession and progradation at Cullendulla Beach were in the order of hundreds of years. The author also contended that a mega-rip would not tend to form against the Square Head shoal. Instead, if a mega-rip does occur, it was more likely to occur at the western end of Cullendulla Beach. The report concluded that the proposed development and its protective works were not in accord with the goals of the NSW Coastal Policy.

A.25 Geomorphology of Cullendulla Creek (Short, 1995b)

This report is the second of two by Short concerning a revised tourism development at Cullendulla Beach. This report discusses the impact of the proposed development on the geomorphology of the system, in particular the outer beach ridges. Cullendulla Creek is the only known chenier site in NSW and one of only two known and documented sites in southern Australia. Cheniers are defined as low, shore linear, swash deposited sand and shell that overlie and are separated by inter-tidal and sub-tidal mud. They represent episodic wave deposition in a muddy tidal flat environment. Such sites are rare in NSW due to the lack of pre-existing fine sediments and high wave energy which removes any fine sediments from the shoreline. The entire system represents a unique coastal system and preserves an excellent record of sea level rise, estuary infilling and shoreline progradation over the past 10,000 years. The author asserts that the cheniers and beach ridges clearly and dramatically illustrate past positions of the shoreline. The report describes the nature of fluvial and marine sediments and the infilling sequence of the creek in six phases. The present geomorphology was categorised into the following major terrain units: outer beach ridge and cheniers, inner beach ridge and cheniers, tidal creeks, a tidal delta and shore platforms. The area also contains numerous Aboriginal occupation sites. The report contended that Cullendulla Beach has the best developed ebb tide delta in NSW. The entire system was asserted to be of additional importance due to its occurrence in a relatively small area (180 ha) with good access from a major town (Batemans Bay) and highway. The report concluded that the proposed development would completely cover and "destroy" the outer beach ridges and thereby severely downgrade the scientific and natural integrity of the entire system. It was also asserted that there was no practicable way that the development could be modified to mitigate its impacts.

A.26 Batemans Bay Vulnerability Study (NSW DLWC, 1996)

The Federal Government was interested in documenting examples of typical climate change vulnerability in each state of Australia. Batemans Bay was selected as the representative site for NSW. The project was jointly funded by ESC, the NSW Government and the Commonwealth Government. The study by the NSW Department of Land and Water Conservation adopted a 50 year planning horizon and a mid-range sea level rise projection of 0.24 m in 2045. Impact assessments were then prepared for beaches, buildings and habitats. The impacts of climate change quantitatively considered included sea level rise, sea surface temperature, rainfall and runoff, storm wave heights and suspended sediment yield from the catchment. Storminess and shoreline re-alignment were also considered qualitatively. Photogrammetry was used to estimate storm demand and long term recession.

The report noted that a low carbonate content of sand in the inner bay appeared to suggest accretion due to fluvial infilling. Maloneys and Long Beaches were characterised by onshore/offshore sand transport only. In contrast, Cullendulla, Surfside and Corrigans Beaches responded to a combination of onshore/offshore and longshore sediment transport. Aeolian losses were not considered to be a major issue as most beaches had well developed dune vegetation. Human intervention in the coastal zone included the rock training wall, dune reconstruction at the northern end of Corrigans Beach in 1988, dredging and terminal revetments at Long, Corrigans and Caseys Beaches. As a result of the intervention at Corrigans Beach, photogrammetry was analysed separately and normalised prior to 1988 and post 1988. The report noted that five major storms occurred in the photogrammetry between 1972 and 1977 at Cullendulla and Corrigans Beaches. However, photogrammetry was only available between 1972 and 1990 at Maloneys, Long, and Surfside Beaches. It was noted that a flood in February 1992 brought a large amount of debris and sediment onto Corrigans Beach.

In comparison to the Batemans Bay Oceanic Inundation Study, a lower uncertainty level of 0.2 m was adopted in the design still water levels. In comparison with the previous study, design water levels on the northern side of Batemans Bay increased by approximately 0.1 m and there was also a small decrease (< 0.1 m) on the southern side. This change was only due to variations between the bathymetric surveys used to develop meshes for the numerical models. The boundary conditions derived for the revised model were also based on wave data from Batemans Bay rather than from Jervis Bay and Botany Bay. On the basis that either bathymetry condition was possible, the study applied the higher design water level of the two studies at each site.

For the 50 year planning period, the study adopted increased design rainfall projections. As such, a hydraulic flood model was constructed to model the increased flood levels from this runoff under climate change. The report commented that suspended sediment load in Batemans Bay is proportional to discharge and rainfall erosivity and speculated that there would likely be a small increase in sediment supply to the bay under climate change. It was also speculated that any damage to seagrass beds in Batemans Bay would lead to increased wave heights at the shoreline. Climate change may cause damage to the seagrass beds by salinity change, sediment smothering following floods, higher waves and sea temperature change. Limited data was available at the time of writing regarding future changes in wind patterns under climate change, although speculative commentary was provided.

Hazard lines at each site were determined from storm demand and recession due to sea level rise using the Bruun rule. Ongoing long term recession was noted at Maloneys, Long and

Cullendulla Beaches and included in the respective hazard lines. Surfside Beach also included an additional parameter, an erosion escarpment, in determination of its hazard line to account for mid-term shoreline fluctuations. The applicability of the Bruun Rule at Maloneys, Long and Surfside Beaches was questioned due to the presence of rock reef nearshore.

Site specific management options considering environmental planning, development controls and protection works were set out for each site around Batemans Bay. Cullendulla Beach was characterised by its lack of an incipient dune and vegetation due to ongoing long term recession. Cullendulla Creek was described as a barrier estuary containing a tidal creek, tidal flats and an ebb tide delta. Long term recession at Cullendulla Beach will likely lead to the loss of a vehicle track, a Telstra cable and a rising main. The dune at Surfside Beach was considered stable except at the northern end which was recently eroded (at the time of writing) with a 1 m high scarp. The beach there appeared to be stable as a result of waves moving flood deposited sand onshore. The report indicated that the revetment around the CBD was necessary primarily for protection against flood flows rather than for protection against the structural impacts of waves. The northern end of Corrigans Beach was accreting due to flood deposition and longshore sediment transport (northward) being trapped against the training wall. A sewage pumping station at the southern end of Caseys Beach was also considered to be at risk from coastal hazards.

Finally, the report reproduced the findings of Short (1995b), who noted that to fully understand the processes operating in the inner bay and at Cullendulla Creek, accurate information regarding the following processes is required:

- transport of Clyde River sediment into, within and through the inner bay, particularly associated with major floods;
- transport of marine sands from the outer bay to inner bay;
- the impact of major storm wave events on sediment transport within the inner bay and the impact of the waves and associated setup and runup on bay shores;
- the sequential modification of the depth and morphology of the bay associated with such events
- the impact of modification of adjacent coastal processes and sedimentation;
- the impact of the southern training wall on processes and sedimentation within the inner bay; and
- the interaction of all these processes within the inner bay over years and decades.

A.27 Batemans Bay Wave Penetration and Run-Up Study (Lawson and Treloar, 1996)

This study by Lawson and Treloar was commissioned as a sub-component of the Batemans Bay Vulnerability Study. It was intended to recalculate the wave propagation, wave runup and design still water levels (including setup) at 17 selected sites (as with previous Inundation Study) with updated bathymetric data. The report examined changes in bathymetry between 1987 and 1995. The training wall was extended in 1987 leading to accretion at the northern end of Corrigans Beach. The bathymetry adjacent to Acheron Ledge (separating Maloneys and Long Beaches) had also changed between 1987 and 1995 and affected propagation to the northern shoreline. The study adopted the same tide and storm surge levels as in the previous study. A reverse ray frequency-direction spectral wave refraction method was used to developed nearshore wave coefficients (RAYTRK). It was not possible to propagate waves seaward of Cullendulla Beach, Wharf Road and the CBD at mean sea level. The study found that Caseys and Corrigans Beaches were more sheltered from the southerly sector than determined previously and Maloneys, Long and Surfside Beaches were similarly more exposed. Design still water levels included an uncertainty allowance of 0.2 m. Overall design still water level changes were in the order of 0.1 m. The 100 year ARI design still water level (without sea level rise) varied between 2.2 and 3.0 m AHD around the selected sites of Batemans Bay.

A.28 Drainage Report Wharf Road-Surfside (ESC, 1997)

This report by ESC reviewed the existing drainage in the Wharf Road catchment and considered mitigation options for minor flooding. Rainfall and runoff modelling was undertaken with the XP RAFTS model which was setup with six separate sub-catchments. The hydraulic grade line method was used for hydraulic modelling with a tail water level of 0.6 m AHD (approximately mean high water). The study considered the effects of localised flooding in isolation of oceanic inundation levels. However, the effects of oceanic inundation on the site based on still water levels presented in the Batemans Bay Vulnerability Study were qualitatively discussed.

A.29 The Impact of a Major Storm Event on Entrance Conditions of Four NSW South Coast Estuaries (McLean and Hinwood, 1999)

This conference paper discussed the effects of a major storm event on 8-13 May 1997 on four estuaries, one of which was the Tomaga River. The Tomaga River was described as being a permanently open, elongated coastal river with no associated lake. Under typical conditions the authors indicated that it has an area of 1.6 km^2 and a catchment of 98 km^2. At each estuary, the influx of sand through storm induced washover deposits provided a noticeable change (restriction) to pre-existing entrance conditions. Analysis of the water level inside the Tomaga River indicated that immediately following the storm, the mean water level increased by 0.033 m. Weak changes to the tidal constituents within Tomaga River as result of the storm were detected (reduced tidal amplitude and increased phase lag). The modified regime persisted for less than a week following the major storm event. The authors concluded that a dominant negative feedback mechanism in the Tomaga River encouraged the rapid recovery of pre-storm flows and amplitudes. This negative feedback mechanism was the increased still water level which promoted higher efflux velocities and bed shear stresses. At the same time, a positive feedback mechanism existed in the reduced tidal amplitude. The balance between these two mechanisms at Tomaga River was biased towards the negative feedback allowing relatively rapid "recovery" from a major storm event.

A.30 Batemans Bay Primary School Relocation Surfside Stormwater Drainage Study (ESC, 2000)

This report by ESC considered stormwater drainage for the proposed Batemans Bay primary school relocation site at Surfside. It superseded an earlier drainage study undertaken in 1986. The catchment is immediately upstream of the culverts under Wharf Road. Rainfall and runoff modelling was undertaken with the XP RAFTS model for the 1, 20 and 100 year ARI events. The one-dimensional HEC-RAS surface profile model was used to estimate water surface levels. Four different tail water conditions were considered: 0.6, 1.1, 1.5 and 2.3 m AHD. Calculations were undertaken with unblocked and blocked culverts. The report concluded that the effect of the primary school relocation on stormwater would be minimal and that the benefits exceeded a slight rise (0.1 m) in flood levels at Wharf Road in a 20 year ARI event.

A.31 Batemans Bay/Clyde River Estuary Processes Study (WBM Oceanics, 2000)

This study by WBM Oceanics addressed the water quality and sedimentation aspects of the estuary and associated catchment.

A two-dimensional (2D) RMA hydrodynamic model was developed for the area downstream of Nelligen. Flows from smaller tributaries were defined using the AQUACM-XP model. The RMA advection-dispersion module was also used. The model was calibrated and validated to available data on water levels and flows in the estuary for a range of measurement sites. Tidal water level data from Eden was determined to be most representative of the tide across the entrance to Batemans Bay. The report notes that the ocean tide attenuates slightly (approximately 10%) between the entrance and the highway bridge. It was also noted that the annual median rainfall over the Clyde River coastal plain varies between 900 and 1,150 mm. The highest daily rainfall (24 hours to 9 AM) on record at Batemans Bay was 363 mm on 9 April 1945. Preliminary (un-calibrated) water quality modelling was also undertaken using a module in the MIKE11 model. Predictive assessments for sewer spill scenarios and land use changes with water quality impacts were simulated.

Batemans Bay was described as having an open funnel shape with its centre-line orientation directed towards the south-east. Direct ocean wave access to the inner bay area is available from a range of ocean wave directions between the east 90° and approximately the south-south-east (150°). Waves from further north and south than this are subject to substantial refraction, diffraction and associated attenuation in propagating to the inner bay. It was noted that the shape of Batemans Bay may exacerbate wind-induced setup in certain conditions. Wave attenuation by bed friction across Batemans Bay causes a tendency for setup to occur further offshore with a slightly lower resultant setup at the shore. Where much wave refraction occurs in offshore areas, the same wave setup tendencies may also result. As part of the study, 2D preliminary (un-calibrated) wave propagation modelling was undertaken to derive refraction, diffraction and shoaling characteristics for several representative wave cases. The modelling of wave induced currents was used to infer indicative sand transport patterns. Wave related radiation stresses within the inner bay were found to induce significant current circulations including a consistent clockwise current circulation in the deeper regions (especially significant for accretion at the northern end of Corrigans Beach) and westward current flow across the ramp margin shoals past the Wharf Road area to the main river channel.

The study provides an excellent overview of sediment transport processes within Batemans Bay including Holocene sediment supply, river delta morphology, shoreline evolution and historical bathymetric changes. With respect to sedimentation aspects of the estuary, the annual average flood fluvial sand supply at the highway bridge was calculated to be approximately 22,000 m^3 per year. A substantial proportion of this volume is transported by the most infrequent flood events. The seabed across the inner bay was described as being highly mobile over a wide area out to and beyond Square Head and Snapper Island. Since 1898, approximately 800,000 m^3 of sand has accumulated on Corrigans Beach. The extent of accretion on Corrigans Beach is largely determined by the length of the training wall. The study noted that tidal currents act with wave action to move sand on the river bar, the ramp margin shoals and the Wharf Road area. The study described the Wharf Road area as having two fundamental configuration categories for the shoreline and shoals. The first is a nearshore current dominated configuration with a shoreline shape which runs approximately east-west. The second is a wave dominated shoreline evolution forming a well-established sand spit alignment approximately parallel to wave crests in the area.

In general, it was asserted that sand movement is westwards at Wharf Road. Only infrequent floods were deemed to have capacity for substantial reworking of the Wharf Road shoals seaward to the Surfside Beach nearshore area.

Finally, the study documented several important changes introduced to the Batemans Bay area. In response to progressive erosion of the northern end of Surfside Beach commencing around 1991, 12,000 m^3 of sand was placed on the beach in 1996. The sand for this beach nourishment exercise was sourced from the hind-dune area of Corrigans Beach. This was in turn replaced by sand removed from the marina basin during maintenance dredging. The report also noted that the highway bridge was constructed between 1951 and 1956, replacing the ferry crossing which had operated since the 1800s.

A.32 Moruya Odyssey: Beach Change at Moruya, 1972-2000 (Shen, 2001)

This conference paper discussed beach surveys undertaken at Bengello Beach from January 1972 to December 2000. Analysis from January 1972 to December 1987 was presented in WRL's reviews of Thom et al (1973), McLean and Thom (1975) and Thom and Hall (1991) and is not reproduced for brevity. It was noted that beach surveys had been undertaken fortnightly between January 1972 and January 1976, monthly between February 1976 and January 1989 and approximately every six weeks between February 1989 and December 2000. An accretion dominant period extended from June 1978 to January 1993 after which Bengello Beach entered a period of gradual erosion up until the latest available surveys (December 2000). The author noted that Bengello Beach had not yet experienced a full low-frequency cycle of erosion and accretion since monitoring commenced in 1972.

A.33 Batemans Bay Waterway Infrastructure Strategy (Webb, McKeown and Associates, 2002)

This report by Webb, McKeown and Associates formed part of the Estuary Processes Study and its outcomes would be included in the Estuary Management Plan for Batemans Bay. It considered the long term planning for, and provision and management of public waterway infrastructure. Specifically it assessed the current condition of public wharves and jetties, boat launching/retrieval facilities, car parking and other land based facilities, fish cleaning tables and boat access for persons with a disability. A 25 year planning timeframe was adopted with the greatest population growth in the area expected at Long Beach. The boat ramp at Maloneys Beach was considered to be dangerous as it often had a drop-off at the seaward end of the ramp due to erosion. Permanent waterway facilities at Cullendulla Beach, Surfside Beach and Wharf Road would be subject to shoaling and erosion which require extensive maintenance. Beach accretion around such facilities was noted to be just as significant for maintenance as erosion. Facilities constructed at Maloneys Beach, Long Beach, Surfside Beach, the northern end of Corrigans Beach and Caseys Beach would also be vulnerable to storm damage from wave action. Unofficial boat launching at the southern end of Corrigans Beach was noted to have caused damage to the dune back beach area. The report proposed that a boat harbour could be created at the eastern end of Long Beach with the construction of a breakwater.

A.34 McLeods Beach [Surfside Beach (West)] Emergency Response Plan - Draft (WBM Oceanics, 2003)

Following more than a decade of accretionary trends on the northern shoreline of the inner bay, severe erosion of the foreshore in December 2001 led to initiation of an Emergency Response Plan for Surfside Beach (West), also known as McLeods Beach. The storm had a peak H_S of

4.0 m coinciding with a spring tide and storm surge of approximately 0.4 m. This report by WBM Oceanics prepared management options for irregular and severe erosion at the site.

It was noted that sand transport is dominated by tidal currents on a day-to-day basis and wave induced currents when swell propagates into the bay. Erosion of the foreshore is dependent on the location and volume of sand in the shoals offshore of the beach, the volume of sand on the beach and the location of the Surfside Creek outlet (which is intermittently closed). The sediment at Surfside Beach (West) is dominated by flood and non-flood cycles. During major floods, scour processes deposit sand offshore of Wharf Road and offshore of Surfside Beach (East). Flooding may cause erosion or accretion at Surfside Beach (West). During periods of high waves (without flooding), strong westward longshore transport from Surfside Beach (East) occurs along the Wharf Road foreshore. This wave action generally causes accretion at Surfside Beach (West). The natural recovery of Surfside Beach was noted to be dependent on major floods to provide new sand deposits offshore. The report concluded that there are no theories or models which can reproduce or represent the processes over the timeframe of natural changes for the northern shoreline of Batemans Bay. Paradoxically, if a significant flood or large wave event does not take place for several years, persistent beach erosion may be apparent in some areas of the inner bay.

The study commented that a seawall (preferably composed of sand-filled geotextile containers) positioned landward of the historical eroded foreshore alignment should have a minimal probability of adversely affecting coastal processes. Sand nourishment was considered as an emergency response option combined with monthly monitoring. An alert was to be noted when the beach scarp came within 5 m of private property boundaries and nourishment was to commence if the scarp reached the boundaries. The construction of a groyne field was also appraised. The preferred solution set out in the report was immediate construction of a seawall composed of 2 tonne sand-filled geotextile containers. It was also recommended that sand build-up at the Surfside Creek outlet be monitored and relocated to adjacent beaches when necessary.

A.35 Batemans Bay and Clyde River Estuary Management Study (WBM Oceanics, 2004a)

This report by WBM Oceanics considered the current uses and values of the estuary and provided strategies for addressing issues and conflicts. Significantly, it found that the Clyde River is one of the few coastal rivers in NSW known to deliver significant sand to the coastal zone. The chenier sand plain forming part of the Cullendulla Wetlands was considered to be of national scientific significance. The report noted that this sand plain provides one of the few remaining intact sites which demonstrate shoreline evolution. It cautioned that development in the Surfside Creek catchment would increase discharge volumes (and hence sediments) if not carefully managed. This would increase the frequency at which the Surfside Creek outlet is blocked. One of the management strategies proposed was to undertake a cost-benefit analysis of dredging the Clyde River bar. It also recommended that additional technical studies regarding the bar be commissioned in parallel with ongoing monitoring of its elevation.

A.36 Batemans Bay and Clyde River Estuary Waterway User Management Plan (WBM Oceanics, 2004b)

This study by WBM Oceanics was commissioned as a sub-component of the Estuary Management Plan which was forthcoming at the time of writing. The plan was designed to protect recreational and environmental values of the waterway and ensure boating practices which maximised user safety and enjoyment. The report concluded that the boat ramp at Maloneys Beach (composed of board and chain) was no longer usable. It speculated that the impact of boat wash on bank erosion was minimal.

A.37 Background Information Document for Joes, Wimbie, Short Beach and Surfside Creeks (WBM Oceanics, 2004c)

This report is the first of three by WBM Oceanics concerning four creeks in the Batemans Bay area. It provides relevant technical information to inform subsequent reports. It was noted that breaching of entrance barriers at the four creeks is periodically undertaken by ESC to alleviate odour problems, as flood prevention strategy or to mitigate water quality issues. The creeks in the study are all small Intermittently Closed and Open Lakes and Lagoons (ICOLLs). Each creek has a relatively high catchment runoff to estuary volume ratio.

While not examined in this report, it was noted that within Eurobodalla Shire there are approximately 30 ICOLLs, of between 200 metres and 1 km in length and less than 20 to 40 m in width, which have lagoons located behind the beach berm.

Surfside Creek has a catchment area of 2.1 km^2. The creek extends 400 m from the opening where it joins the southern extent of a freshwater wetland. This wetland is protected from tidal flushing by a bund wall. The entrance to this creek requires relatively frequent opening by ESC. Scour of the beach is dependent on the beach condition and the volume of water in the creek when it is opened. The elevation of the berm seaward of the outlet causing complete blockage is estimated to be 1.62 m AHD.

A.38 Creek Management Policies for Joes, Wimbie, Short Beach and Surfside Creeks (WBM Oceanics, 2004d)

This report is the second of three by WBM Oceanics concerning four creeks in the Batemans Bay area. It presents the adopted creek management policies. The adopted policy for Surfside Creek is to excavate sand blocking the culvert when the upstream water level reaches 1.5 m AHD.

A.39 Review of Environmental Factors for Joes, Wimbie, Short Beach and Surfside Creeks Creek Management Policies (WBM Oceanics, 2004e)

This report is the third of three by WBM Oceanics concerning four creeks in Eurobodalla Shire. It documents the magnitude and nature of potential environmental impacts associated with entrance management policies. It is considered that artificial opening of the creeks is merely an early facilitation of a natural process to temporarily re-establish a tidal connection between the creeks the ocean. The study discusses the natural berm elevation variability without human intervention. It concluded that the creek management policies are unlikely to have significant environmental impacts upon the respective ecosystems in the short term.

A.40 Comprehensive Coastal Assessment #06: New South Wales Coastal Lands Risk Assessment – Draft Issue 3 (Patterson, Britton and Partners, 2005)

This report by Patterson, Britton and Partners developed a whole-of-coast comparative risk assessment, identifying those parts of the NSW coastal zone that are at risk from a range of coastal hazards for one probability scenario (1% AEP) in 2005 and in the future (2015). The project was funded by the NSW Department of Infrastructure, Planning and Natural Resources. The results were intended to alert local councils and state agencies to areas requiring more detailed scrutiny when planning future land use. "Broad-brush" methodologies were developed and used for all localities regardless of whether or not detailed coastal processes investigations had previously been completed. That is, the output from this report at any on locality includes greater uncertainties than a coastal hazard study focused only on that locality. A total of 99 discrete coastal "localities" were broadly assessed in Eurobodalla Shire including 46 open beaches, 10 pocket beaches and 43 cliffs/bluffs. Note that areas within tidal rivers/estuaries and the inner part of Batemans Bay; Central Business District, Boat Harbour West and Boat Harbour East, were not included. The coastal hazards considered included erosion, recession, cliff instability and overwash potential (dynamic inundation). The primary coastal vegetation line was used as a baseline for mapping coastal hazard lines as it was considered to be representative of an erosion escarpment during past erosion events. This line was defined as the distinct sand/vegetation interface (i.e. the edge of significant vegetation rather than sparse dunal vegetation) and was manually digitised for the NSW coastline using aerial photographs. Unfortunately, the coastal hazard figures for each of the localities within Eurobodalla Shire were not available for review by WRL (report text available only). The key assumptions in the "broad-brush" assessment are presented below:

- Present Day Setbacks from the Primary Coastal Vegetation Line (storm bite distance)
  - Pocket Beach: 10 m
  - Open, Exposed Beach, Dune Elevation > 8 m AHD: 15 m
  - Open, Exposed Beach, Dune Elevation 6-8 m AHD: 20 m
  - Open, Exposed Beach, Dune Elevation < 6 m AHD: 25 m
  - Open, Sheltered Beach, Dune Elevation > 8 m AHD: 10 m
  - Open, Sheltered Beach, Dune Elevation 6-8 m AHD: 15 m
  - Open, Sheltered Beach, Dune Elevation < 6 m AHD: 20 m
- Underlying recession rate: 0.3 m/year
- Recession due to sea level rise
  - Bruun Factor: 50
  - Sea Level Rise (Relative to 1990 MSL): 0.50 m (2105)
- Present Day Wave Runup level: 6 m AHD

A.41 Batemans Bay Wharf Road Development - Soft Option Coastal Engineering Assessment (Webb, McKeown and Associates, 2005a)

This report by Webb, McKeown and Associates concerns a residential development at Wharf Road on the northern shoreline of Batemans Bay. Three structures up to 4.5 storeys high were proposed, with a total of 33 residential units. The development application required a coastal engineer to demonstrate that the site would be secure from flooding and coastal processes and not impact upon flooding and coastal processes. The proponent wished to install a buried seawall with a wave return wall along the 100 % historical line (the most eroded beach alignment on record). However, ESC requested that a "soft option" without a seawall be considered by the proponent.

The report considered that 1964 was the most eroded beach state between 1898 and 1999 and this was adopted as the 100 % historical line. For erosion beyond this line to occur, the report speculated that a very large flood would have to occur when the main channel and margin shoals were highly shoaled. Such conditions would direct flood flows into the Wharf Road area, particularly if combined with a low tide. Peak flood velocities would be in the order of 2 to 4 m/s. While the report acknowledged that the effect of climate change on sediment movement along Wharf Road is not clear, it was speculated that the probability of future erosion extending beyond the 100 % historical line was small. Despite this assertion, the report applied the Bruun rule to estimate recession beyond the 100 % historical line due to sea level rise (0.2 m) in 2050. No additional storm demand was allowed for in these calculations. The report concluded that the development could proceed if structural members were designed for wave loading and inundation, and beach nourishment was planned following any major erosion events.

A.42 Addendum to Batemans Bay Wharf Road Development Soft Option - Coastal Engineering Assessment (Webb, McKeown and Associates, 2005b)

Further to the previous study, this report by Webb, McKeown and Associates considered additional requests from ESC regarding the development at Wharf Road. It was noted that the present vegetation line (2005) was actually located up to 15 m landward of the 100 % historical line (1964). The report also considered impacts on the proposed development from recent coastal protection in its vicinity. Upstream of the development, the foreshore revetment in front of the caravan park had been extended by 5 m in early 2005. Immediately downstream of the development a temporary rock wall had also been constructed in May 2005.

A.43 From Foreshore to Foredune: Foredune Development Over the Last 30 Years at Moruya Beach, New South Wales, Australia (McLean and Shen, 2006)

This journal paper discussed beach surveys undertaken at Bengello Beach from January 1972 to June 2004. Analysis from January 1972 to December 2000 was presented in WRL's reviews of Thom et al (1973), McLean and Thom (1975), Thom and Hall (1991) and Shen (2001) and is not reproduced for brevity. A period of gradual erosion extended from 1993 to 1999 after which Bengello Beach entered a period of gradual accretion up until mid-2001. From this time up until the latest available surveys (June 2004), Bengello Beach has been relatively stable. At this time, the beach was considered to be in a well-nourished state. The maximum change in beach volume above 0 m AHD (averaged across the four profiles) was an increase of 210 m^3/m between 1975 and 1994. The authors also noted that during the 7 weeks from 6 May to 21 June 1974, 95 m^3/m of sand above 0 m AHD (averaged across the four profiles) was eroded (2 m^3/m/day).

A.44 Flood Risk Assessment (URS, 2006)

This report by URS presents findings and recommendations on Floodplain Risk Management within ESC. It is a strategic document for the development of more specific floodplain risk management studies and plans within the local government area. The report recommended that after a flood has occurred, flood damage and other data should be collected as quickly as possible. Potential flood prone properties were noted at Maloneys Beach (5), Long Beach (65), Surfside Beach and Wharf Road (180), the CBD and boat harbour (200), Corrigans and Caseys Beaches (100), Malua Bay (20), Rosedale Beach (15), Guerilla Bay (2), Tomakin Cove and Beach (60) and Broulee Beach (10).

A.45 Batemans Bay Coastline Hazard Management Plan (Webb, McKeown and Associates, 2006)

This report by Webb, McKeown and Associates reviewed the findings of the Batemans Bay Vulnerability Study and further reports since this time (including the Clyde River/Batemans Bay

Estuary Processes Study) to determine hazard management options for the region. The study area was the same as that for the present Coastal Hazard Zone Management Plan. The study did not consider rocky foreshores as they were not deemed to be significantly affected by coastal hazards. The present value of likely damage due to inundation was estimated using an Average Annual Damage approach. This estimated the damage for each event multiplied by its probability of occurrence.

Revetment stability was also assessed in the study. Small seawalls at Long Beach and Corrigans Beach were not considered to have the ability to withstand a major storm. After damage sustained in a severe storm in the early 1990s, the seawall at Caseys Beach was topped up with 0.6 to 0.9 m size granite.

The report reviewed water level records during three recent severe storms in the area. On 9-11 May 1997, a maximum water level of 1.08 m AHD was measured at Princess Jetty tide gauge with a maximum anomaly of 0.32 m. On 22-25 June 1998, a maximum water level of 1.25 m AHD was measured with a maximum anomaly of 0.51 m. On 6-10 August 1998, a maximum water level of 1.07 m AHD was measured with a maximum anomaly of 0.35 m.

The study adopted sea level rise projections of 0.2 m in 2050 and 0.5 m in 2100. Since flooding and storm surge are not entirely mutually dependent, the study combined a 20 year ARI flood with 100 year ARI storm surge in determination of 100 year ARI oceanic still water levels in the inner bay. At most, flooding contributed 0.1 m to these design water levels. Unlike the Batemans Bay Oceanic Inundation and Vulnerability Studies, no uncertainty allowance was included in these levels. A hydrodynamic model was set-up for the CBD only to examine the combined effects of local runoff and oceanic inundation. Storm demands were calculated in a different manner to those in the Batemans Bay Vulnerability Study (DLWC, 1996), but not consistently based on an eroded volume above 0 m AHD. The storm demand for Cullendulla Beach was estimated in an unconventional manner and assumed to be equivalent to the volume of sand removed by ongoing recession over 5 years. The Bruun rule was applied to determine recession due to sea level rise, but these estimates were considered to be conservative.

The following recommended coastal hazard management options were presented for each part of Batemans Bay:

<u>Maloneys Beach:</u> No major recommendations.
<u>Long Beach:</u> Continue and strengthen existing development controls.
<u>Cullendulla Beach:</u> Relocate assets when required in the medium term.
<u>Surfside Beach:</u> Continue minimum floor level policy but large scale land filling is not feasible.
<u>Wharf Road:</u> Continue existing development controls but large scale land filling is not feasible.
<u>CBD:</u> Continue and strengthen existing development controls.
<u>Boat Harbour West:</u> Continue existing minimum floor level policy.
<u>Boat Harbour East:</u> Consider construction of a levee around the caravan park.
<u>Corrigans Beach:</u> Continue minimum floor level policy with planned retreat.
<u>Caseys Beach:</u> Sustain ongoing maintenance and possible upgrade of the seawall.

A.46 Southern Rivers Catchment Action Plan (SRCMA, 2006)

This plan by the Southern Rivers Catchment Management Authority broadly addresses six subregions from Stanwell Park to the Victorian Border of which Eurobodalla is one. It identifies the desired condition of natural resources and sets out priority targets towards achieving this condition over 10 years. Adaptive management is considered an important principle to deal with fire, flood, drought, storms and climate change. The main threats to the quality of ecosystems were deemed to come from historic and current impacts. Ecologically sustainable development principles and climate change impacts were taken into account in development of the Catchment Action Plan. Specifically, it targets the identification, auditing and rehabilitation of erosion "hotspots".

A.47 South Coast Regional Strategy 2006-31 (NSW Department of Planning, 2007)

This strategy by the NSW Department of Planning represents an agreed NSW government position on the future of the south coast to complement and inform other state government planning documents. Batemans Bay is identified as a major regional centre in the document, while Moruya and Narooma are identified as major towns. Its purpose is to ensure the adequate and appropriately located supply of land to sustainably accommodate housing and employment needs over 25 years. Various adaptation strategies for climate change were presented. Future urban development will not be located in high risk areas from natural hazards (flooding, inundation, erosion and recession). The strategy deems that Local Environmental Plans should provide adequate setbacks in high risk areas. The document specifically identified Long Beach and Malua Bay as an area potentially suitable for future urban growth. However, future development in northern Batemans Bay was preferred in the first instance due to its proximity to the CBD.

A.48 Projected Changes in Climatological Forcing for Coastal Erosion in NSW (McInnes et al, 2007)

This report is the first of two by CSIRO concerning climate changes projections in the Batemans Bay area. This study demonstrates the expected range of variability of climate parameters that influence coastal erosion in Batemans Bay. Two high resolution regional climate models were forced with the same greenhouse gas emission scenarios with markedly different responses. Depending on the model considered, a climate variable may both increase or decrease such that the range of possible changes spans zero. This was an artefact of the differences in the formulation of the models and their treatment of physical processes. Both models were forced with the IPCC A2 scenario which is considered to be a sufficiently conservative future scenario to base risk averse planning decisions on.

Wave growth and propagation were not included in the regional climate models but were inferred from the wind outputs with application of desktop wave hindcast techniques. These techniques treated waves with a fetch less than 200 km as locally generated and waves with a fetch between 200 and 500 km as swell.

Conclusions were drawn from comparison of the two models based on whether a trend (i.e an increase or decrease) was consistent for each variable between both models over the projected period. Sea level rise was expected to be 0-4 cm greater than the global mean at Batemans Bay in 2030 and 4-12 cm greater in 2070. Projected changes to wind speed, direction and frequency were inconclusive. Correspondingly, the projected changes to mean wave climate (height, direction and period) were considered small to negligible. Changes to extreme wave climate and storm surge behaviour were also inconclusive.

A.49 Climate Change Projections for the Wooli Wooli Estuary and Batemans Bay (Macadam et al, 2007)

This report is the second of two by CSIRO concerning climate changes projections in the Batemans Bay area. This study complements the previous report but examines additional variables with two different global mean temperature increases (low and high) for the same two regional climate models. The annual average maximum temperature at Batemans Bay was found to increase by 0.5 to 1.1° in 2030 and 1.1 to 4.6° in 2070. The annual average minimum temperature was found to also increase by 0.4 to 1.0° in 2030 and 1.0 to 4.3° in 2070. The annual average solar radiation was found to increase by 0.1 to 0.3% in 2030 and 0.2 to 0.8% in 2070. Projected changes to annual average rainfall, extreme rainfall and extreme drought were inconclusive.

A.50 Wharf Road Coastal Hazard Assessment and Hazard Management Plan (BMT WBM, 2009)

This report by BMT WBM was commissioned to consider the extent of coastline hazards effecting beachfront properties in the eastern end of Wharf Road. The area was subdivided in 1883 and is currently zoned for residential and tourism development. 80 per cent of the original subdivision is now below the high water mark. At the time of writing, coastal hazards were managed with minimum floor levels and additional development control. As a result of the construction of the training wall, the report asserted that 80 per cent of the sand supplied by the Clyde River into the inner bay had accreted on Corrigans Beach. It was speculated that this has reduced the supply of fluvial sediment to Cullendulla Beach, Surfside Beach and Wharf Road. It was noted that the northern end of Corrigans Beach had nearly accreted to the end of the training wall and its capacity as a sediment sink would correspondingly reduce. Extreme water levels were derived from the Princess Jetty tide gauge (including wave set-up and flood effects). A groyne constructed on private land without approval from ESC had maintained the updrift (eastern) shoreline position but exacerbated downdrift (western) shoreline erosion between 2005 and 2007.

In contrast to two previous studies of the area, the 100 % historical line (the most eroded beach alignment on record) was identified as having occurred in 1977. The extent of maximum erosion had previously been nominated to have occurred in 1964 or 2005. A smooth equilibrium planform was also fitted through this 1977 foreshore alignment. The Bruun rule was also applied to estimate the recession due to sea level rise beyond the 1977 vegetation line.

The suitability of a range of management options including environmental planning, development control conditions, "soft" protective works, "hard" protective works and "hybrid" protective works were considered. It was determined that any structural protection works or mitigation options would only be able to use a small amount of backfill due to its impacts on stormwater drainage. The report concluded that a second groyne or extension of the seawall fronting the caravan park would be required to offset the downdrift erosion caused by the unapproved groyne. However, it was considered unlikely that such "hard" protective works would satisfy ecologically sustainable development principles and so the recommended management option was rezoning of the Wharf Road area with possible voluntary resumption.

A.51 Eurobodalla Interim Sea Level Rise Adaptation Policy (ESC, 2010)

This policy by ESC initiates the process of providing long term management options for the ESC coastline under sea level rise projections and gives guidance on how development applications will be assessed until the Coastal Zone Management Plan is completed. Coastal risk areas are defined as those deemed to be at risk in a coastal hazard or floodplain study or within 100 m of the 1% still water level (1.435 m AHD) and/or at an elevation less than or equal to 5 m AHD. A 100 year planning period was used for residential land and a 50 year period for commercial or public facilities. The policy adopted the now withdrawn NSW sea level rise benchmarks of 0.4 m rise by 2050 and 0.9 m rise by 2100 relative to 1990 levels. The degree of risk was discretised as follows: immediate risk (at risk to the current 1% AEP event), high risk (at risk to 1% AEP event by 2050), medium risk (at risk to 1% AEP event from 2050 to 2100) and low risk (at risk to 1% AEP event after 2100). ESC promotes a policy of planned retreat for sites with immediate, high and medium risk (i.e. those sites at risk to 1% AEP event before 2100). New developments in the coastal zone must not create community risk, manage coastal hazard risk, not require protection works, not create community cost and be able to be removed or relocated at no cost to the community. As such, the policy noted that compliance with engineered property protection works is difficult to achieve. It was noted that planning control exclusions may be modified for coastal erosion protection in the CBD, the boat harbour (east and west) and the northern end of Corrigans Beach.

A.52 Concept Plan for the Batemans Bay Coastal Headlands Walking Trail (Gondwana Consulting, 2010)

This report by Gondwana Consulting presented a concept plan to guide the planning and development of a three phase formal walking trail linking the coastal headlands and beaches of the southern shoreline of Batemans Bay (Observation Head to Pretty Point). Parts of the trail are in the Coastal Hazard Assessment study area and include Caseys Beach, Sunshine Bay and Malua Bay. Acid sulphate soils were noted to occur along and landward of Sunshine Bay (west of Beach Road) and landward of Malua Bay (and along Reedy Creek).

In the first phase, the works proposed at each of the coastline sub-sections within the Coastal Hazard Assessment study area are minimal (i.e. addition of signage, drainage treatments and upgrade of existing walking track surfaces). The exception to this is at Caseys Beach, where a new footpath is to be built on the western side of Beach Road to allow travel between the northern end of Caseys Beach and Short Beach Creek before crossing back to the eastern side of the Beach Road. This footpath is to be constructed to avoid wave impacts on walkers under high tides and/or storm conditions.

During the second phase, the following significant works are proposed:

- picnic furniture is to be installed at the southern end of Caseys Beach;
- picnic furniture is to be installed at Sunshine Bay; and
- a footbridge is to be built over Reedy Creek at the northern end of Malua Bay.

In the third phase, it is proposed that Beach Road may be converted to a one-way road system at Caseys Beach and a foreshore reserve be established for the trail.

A.53 Eurobodalla Shire Coastal Hazards Scoping Study (SMEC, 2010)

This report by SMEC reviewed all existing coastal hazard studies for the whole of the ESC local government area and provided recommendation for future studies. The review specifically focused on the findings of previous reports with regard to current climate change projections. Specifically, it was commented that the Batemans Bay Vulnerability Study (DLWC, 1996) should have considered the 2100 planning period. It was noted that the storm demand values derived in that study appeared to be generally too low. The review found that there is a lack of coastal hazard information for the shire outside of Batemans Bay. It recommended that given the length of coastline and vast network of estuaries, beaches and lagoons within the shire, there is a need to target comprehensive coastal hazard investigations to priority areas. This report identified the priority areas for targeted assessments, as well as critical data acquisition requirements for the development of a Coastal Management Program for the entire coastline.

This scoping study analysed photogrammetry to estimate storm demand and long term recession at Maloneys Beach, Long Beach, Cullendulla Beach, Surfside Beach and Barlings Beach based on the storms of May-June 1974 and May 1978. Estimates for storm demand at other beaches were also presented based on approximate wave climate exposure. A small amount of recession was noted at Maloneys Beach and Barlings Beach, but it was speculated that this may be the result of sea level rise and not a sediment transport imbalance, and hence the long term recession at this beach was considered to be nil. Long term recession at Cullendulla Beach was not analysed. It was assumed that the median sand grain size for the beaches was 0.25 mm.

It was noted that the Bruun rule is a two-dimensional model which does not take into account three-dimensional effects. However, due to the lack of a more satisfactory model at the time of writing, it had been assumed that the Bruun rule could be applied uniformly along the beaches. However, the beaches within estuaries such as Cullendulla Beach, Wharf Road and the CBD would not undergo sea level rise recession that could be accurately calculated using the Bruun rule. It was asserted that this was because their offshore profiles are dominated by the dynamics of the tidal delta and three-dimensional sediment transport processes. Bruun factors were calculated from bathymetric and topographic data for Maloneys, Long and Surfside Beaches. A Bruun factor of 50 was specifically adopted for Barlings Beach as bathymetric data was unavailable. Bruun factors for the remaining beaches were approximated with more limited data. As the study concerned the whole of the local government area, the extreme water level estimates were generic in nature. However, the 1,000 year ARI water level was used for maximum wave runup calculations. Hazard lines were plotted for Maloneys, Long, Surfside, Barlings and Moruya Heads Beaches.

Run-up and overtopping areas were plotted for the following coastline sub-sections:

- Durras Beach (south);
- Maloneys Beach;
- Long Beach;
- Surfside Beach;
- Wharf Road and the CBD;
- Corrigans Beach;
- Caseys Beach and Sunshine Bay;
- Denhams, Surf and Wimbie Beaches;
- Mosquito Bay and Garden Bay;
- Malua Beach;
- Rosedale Beach;
- Guerrilla Bay;
- Barlings Beach;
- Tomakin Cove and Beach;
- Broulee Beach;
- Bengello Beach;
- Moruya Heads Beach;
- Congo Beach;
- Tuross Beach;
- Potato Point;
- Yabbara, Dalmeny and Kianga Beaches;
- Bar Beach;
- Narooma Beach; and
- Mystery Bay.

A risk assessment indicated that extreme (immediate) risk was present for the eastern end of Long Beach (inundation and erosion), Surfside Beach (West) (inundation), the CBD (inundation), Caseys Beach (erosion) and the southern end of Tomakin Beach (inundation). It was also noted that the Durras Creek, Maloneys Creek, Surfside Creek and Short Beach Creek outlets are constricted, which may cause issues under significant flows.

Finally, the report concluded that at the time of writing, future studies would require an updated bathymetric survey of Batemans Bay, ongoing LIDAR collection and ongoing photogrammetry collection. It was noted that gaps existed in the historical photogrammetry record which could not be retrospectively filled. The report also recommended that a wave climate study of Batemans Bay be undertaken to update the previous analysis conducted for the Vulnerability Study in 1996.

A.54 Beach Change at Bengello Beach, Eurobodalla Shire, New South Wales: 1972-2010 (McLean, Shen and Thom, 2010)

This conference paper discussed beach surveys undertaken at Bengello Beach from January 1972 to October 2010. Analysis from January 1972 to June 2004 was presented in WRL's reviews of Thom et al (1973), McLean and Thom (1975), Thom and Hall (1991), Shen (2001) and McLean and Shen (2006) and is not reproduced for brevity. A period of relative beach stability (consistent mean volume) in a well-nourished state continued from mid-2001 up until October 2010, albeit with high variations in sand volume. Important erosional events occurred in July 2001, October 2004, July 2005, June-July 2007, October 2009 and May-June 2010. The most significant of these events was in June-July 2007 when the mean sea level intercept moved 20-30 m landward leaving a 1.5 to 2 m high vertical scarp. The post-storm state of Bengello Beach was approximately equivalent to when surveys commenced in January 1972. However, within a year of the June-July 2007 event, the beach had recovered to its pre-storm state and continued accreting to reach its most accreted state since surveys commenced.

A.55 The Cause of Breaks in Holocene Beach Ridge Progradation at Bengello Beach (Rae, 2011)

This postdoctoral thesis evaluated whether any, if not all, of the breaks in beach ridge progradation at Bengello Beach throughout the Late Holocene, from approximately 7,000 years before present (BP) to the present, may have been caused by possible sea level, sediment supply and/or wave climate changes.

Bengello Beach was described as a transgressive-regressive barrier infilling part of a drowned river valley. Between 10,000 to 8,500 years BP, a low relief barrier existed 30-40 m below the present sea level,. A rapid marine transgression between 8,000 to 6,000 years BP caused Bengello Beach valley to flood. Open-ocean sand gradually blocked off the drowned valley causing estuarine mud to accumulate. The low-gradient coastal embayment and an excess of sediment caused episodic beach ridge progradation to occur, mostly between 6,000 to 2,500 years BP, thereby blocking several small drainage basins to create Waldrons Swamp. This resulted in the formation of 40-50 parallel beach ridges, each of which represent a former shoreline position. The source of this infilling sediment was concluded to be the offshore shelf deposit as the bounding headlands exclude littoral inputs of sediments and sediment input from the Moruya River was not considered significant. It was asserted that analysis of these beach ridges may be used as an indicator of the future response of Bengello Beach to changes in sea level and wave climate.

Hand augering of the beach ridges indicated that median sand grain size increased with depth. The Bruun Rule (Bruun Factor approximately 72) was used to estimate recession distances at Bengello Beach at three periods during the Holocene when the sea level rose. An analysis of aerial photographs of Bengello Beach indicated that there has been no significant changes in the orientation of the ridges along the centre of the embayment. However, to the south, the ridges curved to a common point along the airport indicating that this was the edge of the Bengello Beach barrier prior to draining (sea level fall) around 2,000 years BP. It was noted that Moruya airport was constructed in 1942.

It was concluded that the causes of the breaks in the beach ridges were attributed to a combination of wave climate and sea level changes and not sediment supply. Wave energy changes effecting the beach ridges were considered to include periods of increased wave heights and periods but without wave direction changes. The use of ground penetrating radar also indicated that a previous erosion event, approximately 1,900 years BP, had resulted in a 5 m scarp at the beach face. Since modern monitoring began in 1972, observed beach scarps have not exceeded 2 m in height.

A.56 Review of Environmental Factors for Clyde River Entrance Bar Maintenance Dredging and Beach Nourishment, Batemans Bay (NSW Department of Primary Industries, 2011)

This report by the NSW Department of Primary Industries documents the magnitude and nature of potential environmental impacts associated with dredging of the Clyde River bar and nourishment of Corrigans Beach. At the time of writing, shallowing of the entrance bar had limited the ability of recreational and commercial boats to safely cross the bar. NSW Crown Lands proposed to undertake dredging of the bar to maintain public safety and amenity followed by placement of sand on Corrigans Beach to minimise wave overtopping during storm conditions. 10,000 m^3 of sand is to be dredged from the bar to the east of the end of the training wall. The dredged sand is to be placed at two different sites. 7,500 m^3 sand is to be placed seaward of Batemans Bay resort at Corrigans Beach. This is to form a 320 m long dune with a maximum elevation of 3.7 m AHD. Placement of this sand at other beaches such as Long, Surfside and Caseys Beaches was considered and rejected. The remaining sand is to be placed at a second site seaward of the Hanging Rock playing fields. This sand will be stored at this location for future nourishment use as required elsewhere in Batemans Bay.

The report noted that on the opposite side of the bay, there was little movement of sediment around Square Head into Cullendulla Beach. It was noted that the 1989 extension of the training wall (by 150 m) was intended to prevent leakage of sand from Corrigans Beach past the training wall tip, along the channel and into the boat harbour ramp area. Ongoing beach accretion will eventually lead to repetition of this process.

The dredged depth was not to exceed approximately -2.8 m AHD to minimise the rate of infilling. It was speculated that dredging to a greater depth (say -3.3 m AHD) would result in faster infilling of the dredged area.

A.57 Eurobodalla Local Environmental Plan (ESC, 2012)

This plan by ESC aims to restrict development of land subject to flooding, coastal hazards, bush fire and land slip. It embraces ecologically sustainable development principles and aims to minimise any off and on site impacts on biodiversity, water resources and natural landforms. There are 22 different land zonings stipulated in the Local Environment Plan. Zone E2 is entitled Environmental Conservation and one of its objectives is to identify those areas at risk from coastline hazards, including sea level rise. Section 5.5 specifically discussed development within the coastal zone. New development must not be significantly affected by coastal hazards, have a significant impact on coastal hazards or increase the risk of coastal hazards in relation to any other land. Section 5.7 identifies that any development carried out below the high water mark requires consent. Section 6.5 discusses the development of land subject to flooding. The flood planning level was identified as being the 100 year ARI flood level with an additional 0.5 m freeboard. If such land is also affected by coastal hazards, the authority must consider the potential to relocate, modify or remove the development.

Appendix B: Location Summaries

B.1 Durras and Cookies Beaches

Durras Beach is a 2.3 km long beach with a low gradient, facing south-east and east (Figure B-1). It is backed by a continuous well-vegetated foredune and a high, healthy hind dune, which is subsequently backed by the 1 km long entrance channel to Durras Lake (Figure B-2). A reef exists 100 m offshore of the entrance to Durras Lake at the northern end of the beach. When the lake is open (it is generally closed) as it was at the time of the inspection (5 December 2012), the reef and tidal shoals produce additional bars, channels and currents in this area. At mid tide, heavy shoaling is observed within the entrance and a sand bar forms seaward of the entrance. Protected shore birds were observed on the shoals near the Durras Lake entrance. The centre of the beach is backed by urban development including the Durras Lake community and a caravan park (Lakesea Park).

The foredune continues to the southern rocks where a small creek (Durras Lake) crosses the beach. The entrance to the creek is controlled by the Durras Road bridge and existing scour protection for the concrete abutments. Scour protection on the northern side of the entrance is composed of granite rock primary armour with an approximate size of 0.5 m. No secondary armour or geotextile underlayer was observed on the structure. There is a natural wall of rock along the southern side of the creek.

To the south-east of Durras Beach, Wasp Island provides protection from wave attack. Therefore, the wave climate exposure generally reduces from north to south along Durras Beach.

Durras Beach has several informal access points across the dune which are found along Durras Lake Road. The road is well protected by the fronting foredune and the beach is not visible from the road.

Cookies Beach (South Durras) is an 800 m long beach with a low gradient, facing east and north-east (Figure B-1). It is located between two low, unnamed rocky headlands. It is backed by a continuous, well-vegetated foredune, which is subsequently backed by a 2 ha lake and the surrounding Cookies wetland area (Figure B-3). The wetland drains via a small creek in the southern corner. At the time of the site inspection (5 December 2012), the creek entrance was closed.

Cookies Beach is exposed to a moderate wave climate, which usually maintains an attached bar with a rip against the northern rocks. The southern end of the beach appears to be exposed to a lower wave climate due to the rocky outcrops around the southern point. During a higher wave climate, rips can form against the southern rocks.

Beach access is available from the northern and southern ends of the beach. The northern end provides pedestrian access to the beach from a small community of houses situated landward of Dilkera Street. Beach access at the southern end is via a designated pedestrian track and a concrete boat ramp, which is situated adjacent to the southern rocks and crosses the beach. A small picnic area and car park at the southern end are in close proximity to the beach and lie at low elevations. A summary of sand sample analysis is shown in Figure B-4

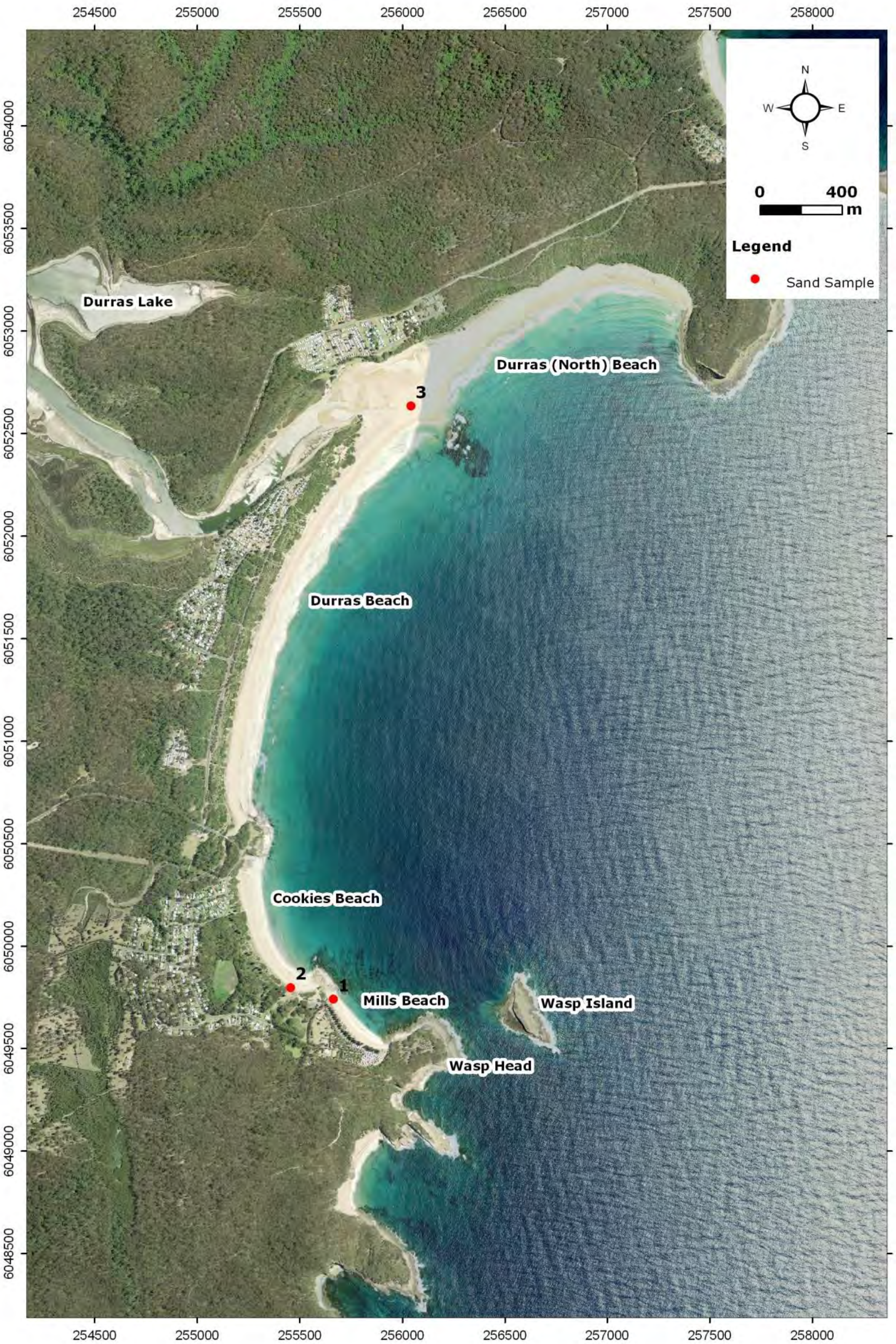


Figure B-1: Durras and Cookies Beaches Site Details


a) View of beachface looking north


b) Durras Lake entrance at northern end


c) Well vegetated dune looking south


d) Well vegetated dune looking north


e) Durras Creek entrance at southern end


f) Durras Road bridge at southern end


g) Dune scarp at the centre of the beach


h) Rock/reef at southern end

Figure B-2: Durras Beach (south) Site Inspection

a) View of beach looking north

b) View of beach looking south

c) Boat ramp at the southern end

d) Rock/reef at southern end

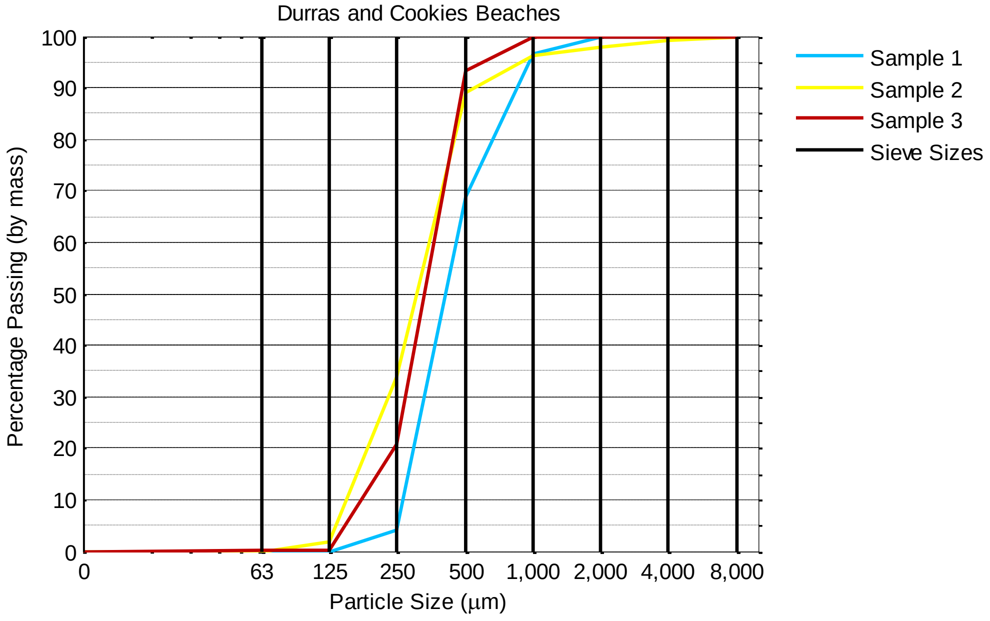
e) Moderate dune vegetation

f) Position of property relative to dune

g) Stormwater outlets at the southern end

h) Formal access way and toilet amenities

Figure B-3: Cookies Beach Site Inspection



Sample 1

Sample 2

Sample 3

Figure B-4: Durras and Cookies Beaches Sediment Samples

B.2 Maloneys Beach

Maloneys Beach is 810 m long with a low gradient facing south (Figure B-5). A well vegetated and relatively steep, stabilised dune exists with a crest level of approximately 6 m AHD for most of the beach, decreasing to 3 m AHD at the eastern and western ends (Figure B-6). There are several breaks in the dune to allow for public pedestrian beach access. At the time of the inspection (1 November 2011), a small scarp was noted at the western end of the beach with one of the public beach access points closed as a result. The entrance to Maloneys Creek exists at the western end of the beach but was not open at the time of the inspection. This creek entrance appears to be quite stable as it is controlled by a box culvert at the bridge and constrained by rock walls on its western side. This creek connects to a large freshwater wetland approximately 800 m upstream. It is understood that beach boat launching occurs from the eastern end of the beach. The beach is backed by a small urban settlement with a ground level of approximately 5.0 m AHD, although there are lower lying areas near the wetland (3.5 m AHD). Car parks exist at both ends of the beach. Reefs exist off the eastern and western ends of the beach providing some protection from wave attack. A summary of sand sample analysis is shown in Figure B-7.

Figure B-5: Maloneys Beach Site Details

a) Rock/reef at eastern end

b) View of beach face looking west

c) Area leeward of the dune (east)

d) Area leeward of the dune (west)

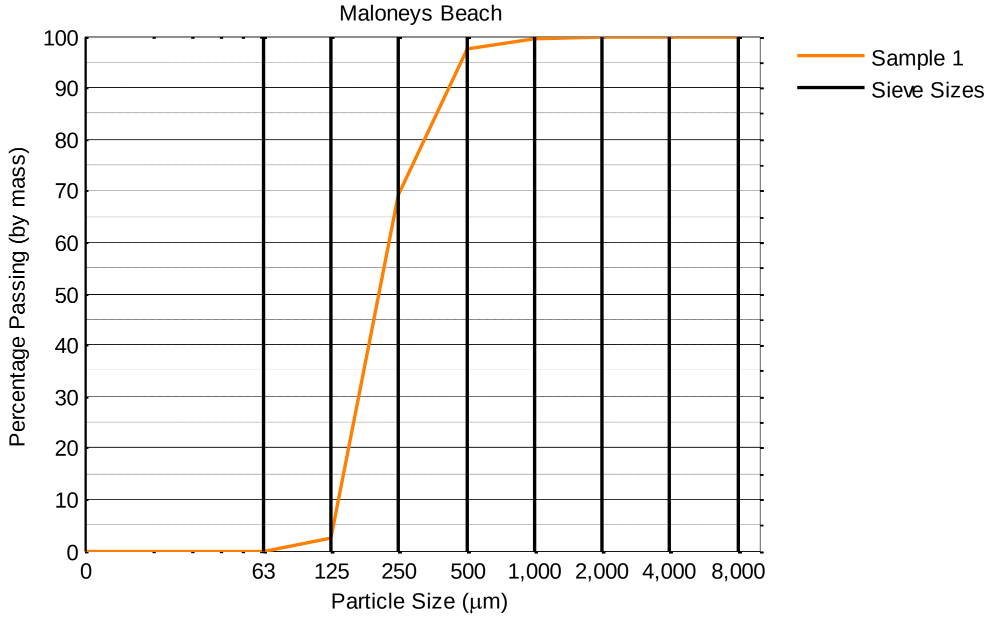
e) Typical well vegetated dune face

f) Scarp at western end of the beach

g) Maloneys Creek outlet

h) Rock/reef at western end

Figure B-6: Maloneys Beach Site Inspection



Sample 1

Figure B-7: Maloneys Beach Sediment Sample

B.3 Long Beach

Long Beach is 2.15 km long with a low gradient facing south and south-east (Figure B-8). Wave climate exposure increases from east to west along Long Beach. Reef exists off the eastern end of Long Beach providing some protection from wave attack.

The eastern third of Long Beach has private properties facing the beach on the northern side of Bay Road (Figure B-9). Two concrete stormwater outlets are located within this section. The foreshore either side of the westernmost stormwater outlet where Fauna Avenue intersects with Bay Road is protected by a rock revetment wall constructed during the 1980s (WMA, 2006). The condition of this wall is unknown (SMEC, 2010), however, WRL estimates that the primary armour is basalt with a typical size of 0.3 to 0.4 m. The wall is largely buried and has an irregular crest level of 3.0 to 3.5 m AHD. Since the structure is largely buried and construction details are unavailable, the alongshore extent of the seawall in Figure 2.7 is approximate only. The dune either side of this wall is poorly vegetated and relatively low, with a similar elevation as the wall. At the time of the inspection (31 October 2011), a small scarp was noted at the eastern end. The ground levels for most properties within the eastern third of Long Beach are above 3.5 m AHD, however, several properties have an elevation of 3.0 m AHD. Four wheel drive (4WD) beach access is available from the eastern end for boat launching. A car park also exists at the western end of this section.

Reed Swamp backs the central third of Long Beach and has an outlet at Sandy Place (100 m west of Long Beach Road). This creek entrance appears to be moderately stable as it is controlled by a box culvert. It is constrained by rock gabions upstream of the culvert, but there is no sidewall protection downstream. The dune height is approximately 5 m AHD except near the Reed Swamp outlet where it drops to approximately 3 m AHD. The back beach area is developed on the southern side of Sandy Place. Seaward of this development the dune is moderately vegetated, but to the west of the developed section, the dune is well vegetated.

The western third of the beach has a dune height of approximately 5 m AHD and is well vegetated (Figure B-10). There are several breaks in the dune to allow for public pedestrian beach access. The back beach area is relatively undeveloped except for a new sub-division on the southern side of Sandy Place. This development is well setback compared to those properties in the eastern and central thirds of the beach. A summary of sand sample analysis is shown in Figure B-11.

Figure B-8: Long Beach Site Details

a) Scarp at eastern end of the beach

b) View of beach looking west

c) Stormwater Outlet 1

d) Partially buried rock revetment wall

e) Stormwater Outlet 2

f) View of beach looking east (1/3)

g) Reed Swamp outlet

h) Erosion due to Reed Swamp outlet

Figure B-9: Long Beach Site Inspection (1 of 2)

a) Vegetation at Reed Swamp outlet

b) Pedestrian beach access at western end

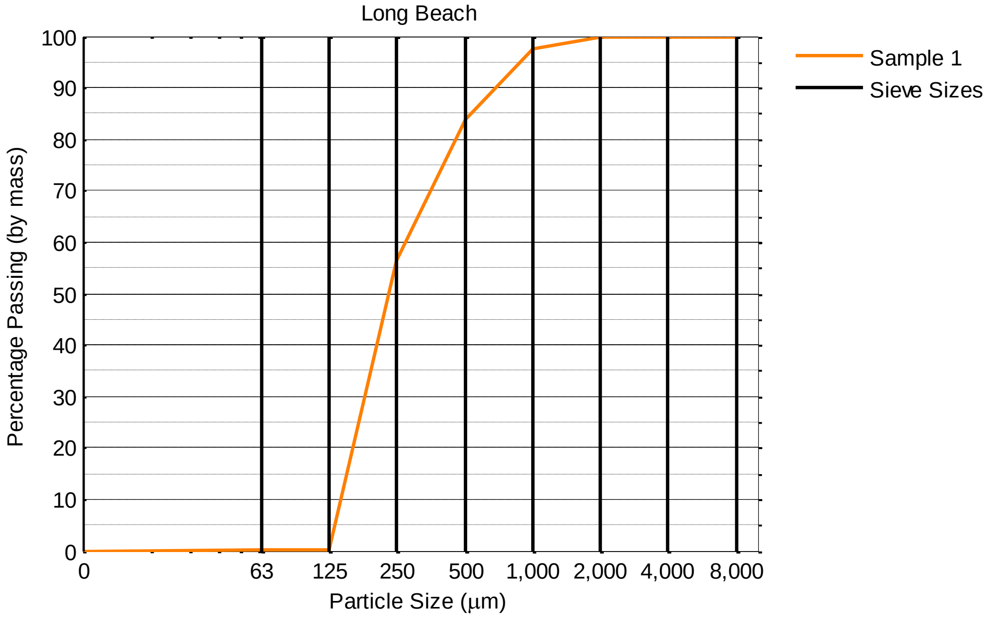
c) View of beach looking west (1/3)

d) View of beach looking east (2/3)

e) View of beach looking west (2/3)

f) Setback development at western end

Figure B-10: Long Beach Site Inspection (2 of 2)



Sample 1

Figure B-11: Long Beach Sediment Sample

B.4 Cullendulla Beach

Cullendulla Beach is a 660 m long beach with a low gradient facing south (Figure B-12). The beach is backed by a series of low beach ridges, then mangroves and inner beach ridges. The back beach area is well vegetated and has a relatively low elevation of between 1.5 and 2.0 m AHD. A scarp running along the length of the beach and vegetation loss due to recession were evident at the time of the site inspection (1 November 2011, Figure B-13). There is no residential development landward of this beach. However, an important sewer rising main, a telecommunications cable and a disused access track run along the back of the beach. The entrance to Cullendulla Creek does not have any artificial training structures, but is naturally constrained on its eastern side by rock shelves and cliffs. It was open at the time of the inspection. The creek connects to a large wetland upstream. A large ebb tide delta extends up to 1 km offshore at the eastern end of the beach. Square Head provides significant protection from swell wave attack for most of the beach. Reef also exists off the western end of the beach (Hawks Nest) providing some additional protection from wave attack. A summary of sand sample analysis is shown in Figure B-14.

Figure B-12: Cullendulla Beach Site Details

a) View of beach face looking north

b) Rock/reef at eastern end

c) View of beach face looking east

d) Scarp at eastern end of the beach

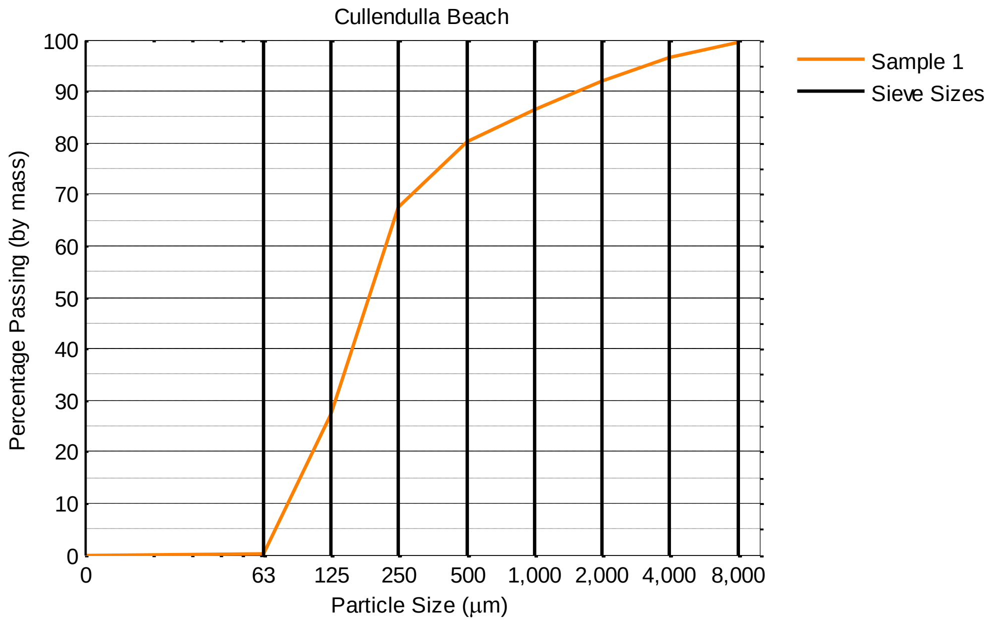
e) Scarp at the centre of the beach

f) Loss of vegetation due to recession

g) Evidence of recession along the beach

h) View of Square Head looking east

Figure B-13: Cullendulla Beach Site Inspection



Sample 1

Figure B-14: Cullendulla Beach Sediment Sample

B.5 Surfside Beach (East and West)

Surfside Beach is sub-divided into two different compartments for the purposes of this Coastal Hazard Assessment, Surfside Beach (East) and Surfside Beach (West) (Figure B-15).

Surfside Beach (East) is an 850 m long beach with a low gradient facing south-east (Figure B-16). The beach dune height is approximately 2.5 m AHD and is moderately well vegetated (although highly variable) along its length. The beach is backed by residential development (seaward of Myamba Parade) with a relatively low ground level of approximately 2.3 m AHD along its full length. Stormwater outlets exist at the northern (outlet damaged at the time of inspection, 31 October 2011) and southern ends of the beach and a sewage pumping station is also located behind the dune. The are several breaks in the dune to allow for public pedestrian beach access, and a car park also exists at the eastern end of the beach. Reefs exist off the northern and southern ends of the beach.

Surfside Beach (West) is a 270 m long beach with a low gradient facing south (Figure B-17). In other reports this same beach has also been referred to as McLeods Beach, Timbara Beach or Wharf Road (East). The beach dune crest is approximately 1.6 m AHD and is vegetated only with grass. The beach is backed by residential development (seaward of Myamba Parade) at the eastern end of the beach. The entrance to Surfside Creek exists at the western end of the beach but was not open at the time of inspection (31 October 2011). This creek extends 400 m from the opening where it joins a freshwater wetland. The creek entrance appears to be quite stable as it is controlled by a three pipe culvert under McLeod Street. The pipe culverts were half blocked due to sediment infilling from the beach. Reefs exist off the eastern and western ends of the beach providing some protection from wave attack. During the site inspection (31 October 2011), it was noted that waves approached the beach at an oblique angle producing a net longshore current westwards along the beach. A summary of sand sample analysis is shown in Figure B-18 and Figure B-19.

Figure B-15: Surfside Beach (east and west) Site Details

a) Rock/reef at northern end

b) Sewage pumping station

c) View of beach face looking north

d) View of beach face looking south

e) Pedestrian beach access at the centre

f) Typical setback of development

g) Stormwater outlet at southern end

h) Rock/reef at southern end

Figure B-16: Surfside Beach (east) Site Inspection

a) Rock/reef at eastern end

b) Typical oblique wave approach

c) View of beach face looking east (1/2)

d) View of beach face looking west

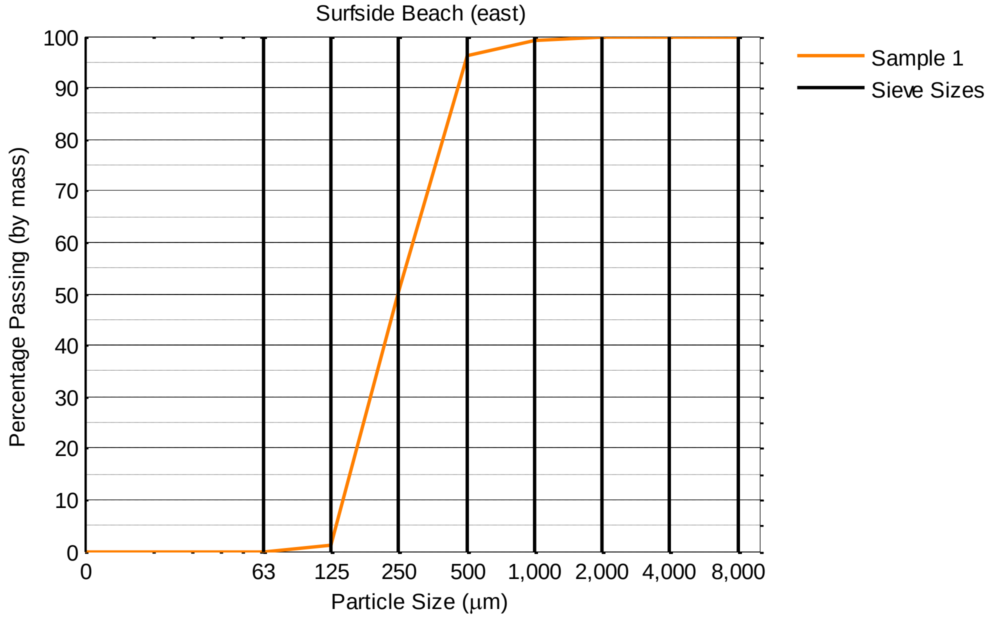
e) Typical setback of development

f) Surfside Creek outlet

g) View of beach face looking east (2/2)

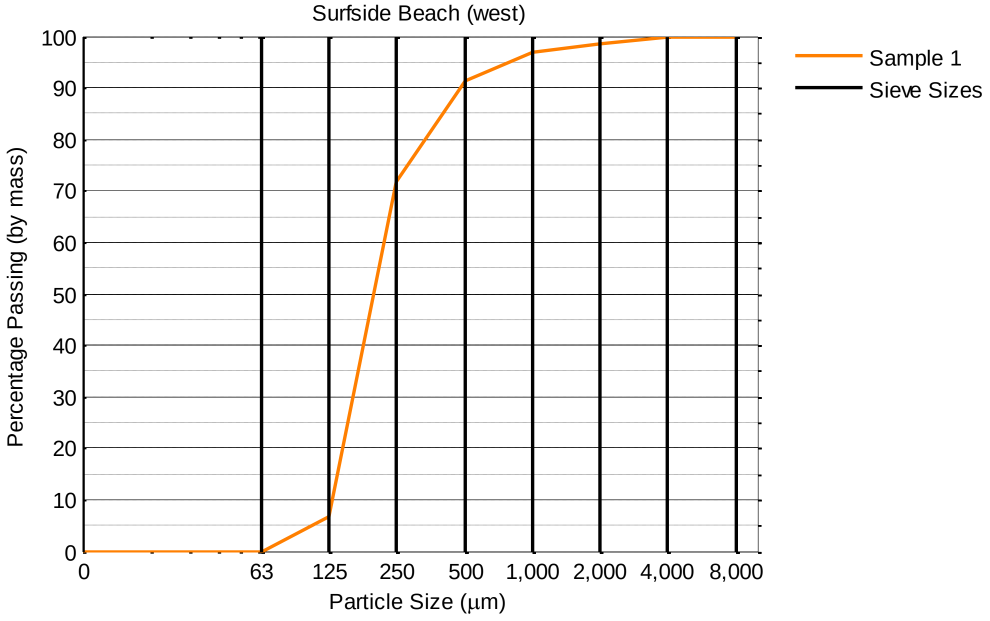
h) Rock/reef at western end

Figure B-17: Surfside Beach (west) Site Inspection



Sample 1

Figure B-18: Surfside Beach (east) Sediment Sample



Sample 1

Figure B-19: Surfside Beach (west) Sediment Sample

B.6 Wharf Road

Wharf Road is a 900 m crenulate strip of sand fronted by dynamic tidal sand flats up to 200 m wide, then a series of tidal channels and shoals extending up to 600 m into the bay and edge of the deep channel (Figure B-20). This sub-section of the coastline is bound to the east by Surfside Beach (West) and to the west by the entrance to the Clyde River with a control point on the northern Princes Highway bridge abutment. It faces to the south-east around to the south-west. At the eastern end of this sub-section, several properties are located seaward of McLeod Street. Access to these areas by WRL was limited as it was private property. The westernmost of these properties is fronted by an unapproved groyne which ESC has requested to be removed (Figure B-21). It appears to WRL that the primary armour on the groyne is basalt, however, building waste has also been included. At a bend in Wharf Road itself, a rock revetment wall protects the road from erosion. WRL considers that at least two different types of rock have been used as armour on this rock revetment. At the eastern end of this wall, the armour had been grouted together with mortar. A caravan park (BIG4 Batemans Bay at Easts Riverside Holiday Park) is located seaward of Wharf Road in the central part of this coastline sub-section. A car park also exists at the western end of the sub-section. A second rock revetment wall protects the caravan park and the car park (Figure B-22) with a crest level between approximately 1.5 and 1.9 m AHD. Again, it appeared that at least two different types of rock with a wide grading have been used as armour on this second revetment. A local stormwater outlet from the caravan park is located within the face of the revetment. At the western end of Wharf Road, the rock revetment wall is completely buried. Note that there is an unprotected section of coastline between the revetment protecting the road and the revetment protecting the caravan park. At the time of the inspection (31 October 2011), a large sand spit was located seaward of the western end of Wharf Road. A small, moderately vegetated dune was located in its lee. It was observed that wave energy at Wharf Road is highly dependent on the tide.

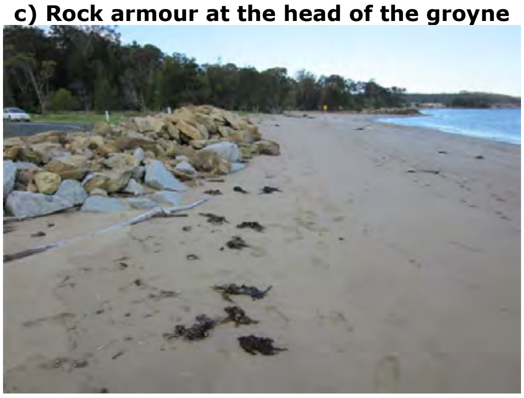
Figure B-20: Wharf Road Site Details

a) View of beach looking east

b) Unapproved groyne

c) Rock armour at the head of the groyne

d) Building waste used as groyne armour



e) Rock revetment wall

f) Wall armour is grouted together

g) Rock wall armour properties are mixed

h) View of beach looking west

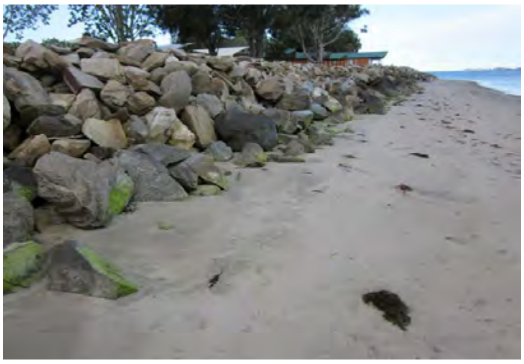
Figure B-21: Wharf Road Site Inspection (1 of 2)

a) Eastern end of rock revetment wall

b) View of revetment wall looking west

c) Rock wall armour properties are mixed

d) Stormwater outlet from caravan park



e) View of revetment wall looking east

f) Moderately vegetated dune face

g) Sand spit at western end

h) View of beach looking east

Figure B-22: Wharf Road Site Inspection (2 of 2)

B.7 Central Business District

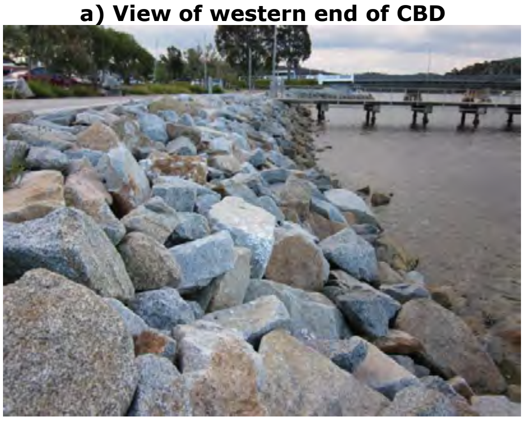
The Central Business District (CBD) coastline sub-section has a length of 680 m (Figure 2.18) facing north-east. It is bound to the west by the entrance to the Clyde River and to the east by the Boat Harbour sub-section. The entire coastline of the CBD is highly developed and armoured. A rock revetment wall protects the full length of the CBD from erosion (Figure 2.19). A pedestrian footpath with elevation varying between 1.7 and 2.2 m AHD is located leeward of the rock revetment wall. Most of the CBD area is at or below the level of the revetment crest. At the time of the inspection (1 November 2011), a small pocket beach existed at the western end of the sub-section. Two box culverts (at the centre and eastern ends of the sub-section) drain stormwater into Batemans Bay. Many smaller local stormwater outlets are also located within the face of the revetment, particularly at the eastern end (Figure 2.20). There are three timber structures creating public space above and seaward of the revetment wall and four wharves for vessels to dock against. This sub-section receives protection from offshore shoals which induce incident wave breaking. At the time of the site inspection (1 November 2011), an onshore breeze was blowing white water up and onto the pedestrian footpath from waves breaking on the revetment.

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 be undertaken by ESC according to coastal engineering guidelines (USACE, 2006). At the western end of the CBD (up to the second wharf), the revetment is mainly composed of granite with an approximate size of 0.4 m. In this region, the armour appears to have recently been topped up. The structure slope in this region is relatively steep at approximately 1V:1.2H. There is a change in armour between the second wharf and the third wharf (Innes Boatshed), with at least two different types of rock (granite and another unknown material) used on the revetment. The granite has an approximate size of 0.3 m and the unknown rock type has a size of 0.9 m. The structure slope in this region is relatively flat at approximately 1V:2.0H. It should be noted that directly under the Innes Boatshed there is no rock revetment. Instead, the rock wall temporarily discontinues and is replaced by a vertical concrete besser block wall. Some pavers in the footpath were noted to be settling in this area, probably due to undermining or loss of fill through the wall. East of the third wharf (Innes Boatshed) the armour was mainly composed of granite with an approximate size of 0.4 m and a structure slope of 1V:2.0H. A geotextile filter was generally evident under the armour along the full length of the revetment.

Figure B-23: CBD Site Details

a) View of western end of CBD

b) Wharf 1 of 4



c) View of revetment wall looking west 1

d) Public Space 1 of 3

e) View of revetment wall looking west 2

f) Wharf 2 of 4

g) Stormwater outlet

h) Wharf 3 of 4

Figure B-24: CBD Site Inspection (1 of 2)

a) View of revetment wall looking south

b) Public Space 2 of 3

c) Public Space 3 of 3

d) Wharf 4 of 4

e) Typical local stormwater outlet

f) View of revetment wall looking north

g) Geotextile underlayer visible

h) Stormwater outlet

Figure B-25: CBD Site Inspection (2 of 2)

B.8 Boat Harbour

The Boat Harbour coastline sub-section has a length of 2.07 km (Figure B-26) facing north-east. It is bound to the west by the CBD and to the east by Corrigans Beach. A single, medium-density residential building is located seaward of Beach Road at the centre of the sub-section. Further to the east are a car park and buildings associated with the marina. A rock wall protects the full length of the Boat Harbour sub-section from erosion (Figure B-27). Where this structure runs parallel to the river channel it is considered to be a training wall (crest 1.8 to 2.2 m AHD). It is considered to act as a breakwater for the Boat Harbour itself. For the remainder, it is considered to be a revetment (crest approximately 1.5 to 2.0 m AHD). Three stormwater outlets are located within the face of the revetment. Each of these was originally fitted with a floodgate. At the time of the inspection (1 November 2011), one floodgate was missing and two were blocked due to sediment infilling from the beach. Three distinct reef sections run perpendicular to the revetment, trapping sand in a manner similar to groynes. Again, this sub-section receives protection from offshore shoals which induce incident wave breaking.

WRL understand 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 requires immediate attention. Ongoing monitoring of the condition of the remainder of the wall should be undertaken by ESC. At the western end, the revetment is mainly composed of granite with an approximate size of 0.7 m, a geotextile underlayer and a relatively steep slope of approximately 1V:1.0H. Opposite "The Old School House" (TOSH, 10 Beach Road), the revetment structure and its condition changes considerably; this region requires immediate attention from ESC. 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 region, 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. East of this section, the revetment rock changes back to granite with an approximate size of 0.7 m and a slope varying between approximately 1V:1.2H and 1V:1.6H. No geotextile underlayer was visible in this region.

While a review of the internal marina and the intertidal basin is beyond the scope of this study, it has significantly infilled with sediment. The outlet of Hanging Rock Creek is also within the marina. The Hanging Rock boat ramp is located towards the eastern end of the sub-section (Figure B-28). Extensive urban development is located landward of the marina and the boat ramp. Properties seaward of Beach Road and Tuna Street have variable degrees of protection from coastal processes. Again, this sub-section receives protection from offshore shoals which induce incident wave breaking.

Figure B-26: Boat Harbour Site Details

a) Western end of rock revetment wall

b) Stormwater outlet (missing floodgate)

c) Revetment rock armour type 1 of 2

d) Revetment rock armour type 2 of 2

b) Stormwater outlet (blocked floodgate)

f) Stormwater outlet (blocked floodgate)

g) Eastern end of rock revetment wall

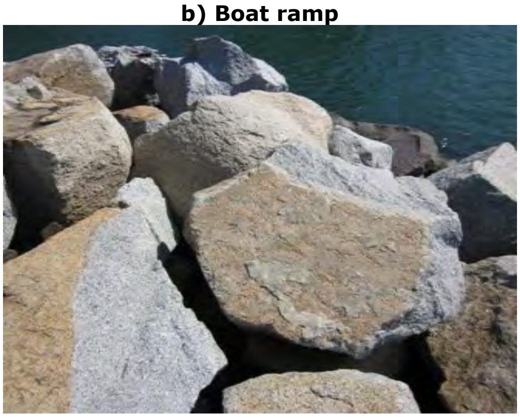
h) Western end of rock training wall

Figure B-27: Boat Harbour Site Inspection (1 of 2)

a) View of boat harbour looking west

b) Boat ramp

c) Concrete cube wall seaward of development



d) Typical primary rock armour

b) View of rock training wall looking east

f) View of rock training wall looking west

g) Typical secondary rock armour

h) Eastern end of rock training wall

Figure B-28: Boat Harbour Site Inspection (2 of 2)

B.9 Corrigans Beach

Corrigans Beach is a 1.8 km long artificially accreted beach with a low gradient facing north-east (Figure B-29). This beach commences at the eastern end of the Boat Harbour training wall (Figure B-30). Construction of the training wall was initially completed in 1905 but it was extended eastward in 1991. As a result of both these works, sand accumulated on the southern side of the training wall, accreting the shoreline by up to 600 m since 1905 to form Corrigans Beach. The foredune dune is low (typical elevation of 2.5 to 3.0 m AHD), wide and moderately vegetated. The low foredune height is largely attributable to the rapid accretion rate experienced here for more than 110 years (i.e. insufficient time for a higher foredune to develop). The dune is backed by a large flat area of relatively new, accreted land.

The entrance to Joes Creek exists at the centre of the beach but was not open at the time of the inspection. The seaward part of the creek entrance is not visibly structurally controlled save for the Beach Road bridge further inland. Three smaller creeks also have outlets at the southern end of the beach. 4WD beach access for boat launching and a car park exist at the southern end.

Development in the lee of Corrigans Beach consists of two caravan parks at the centre (BIG4 Batemans Bay Beach Resort) and southern end of the beach (Clyde View Holiday Park) with typical ground elevations of 1.6 m AHD and several freestanding buildings at the southern end.

In addition to sand accumulation due to the presence of the training wall, sand dredged from the Clyde River (ebb tide) bar has repeatedly been placed at the northern and centre thirds of Corrigans Beach over many years. Buildings in the northernmost caravan park are set back further from the shoreline that those in the southern caravan park. The foreshore of the southern caravan park is protected by a rock revetment wall. The condition of this wall is unknown and was predominantly buried at the time of the inspection (1 November 2011). Since construction details are unavailable, the alongshore extent of the seawall in Figure B-29 is approximate only.

Reef exists off the southern end of the beach providing some protection from wave attack. Additional protection from wave attack is provided by Observation Head and Snapper Island.

Figure B-29: Corrigans Beach Site Details

a) Training wall at northern end of beach

b) View of beach looking south

c) View of beach looking north

d) Moderately vegetated dune face

b) Joes Creek outlet

f) Typical setback of northern tourist park

g) 4WD beach access corridor

h) Predominantly buried rock revetment wall

Figure B-30: Corrigans Beach Site Inspection

B.10 Caseys Beach

Caseys Beach is an 850 m long beach with a low gradient facing east (Figure B-31). A rock revetment wall protects most of the foreshore from erosion (Figure B-32). Beach Road is located immediately in the lee of the revetment and its elevation varies between 3 and 4 m AHD. The beach itself does not have a notable dune system. While there is significant urban development landward of Beach Road, a sewage pumping station is the main asset seaward of the road. There are several breaks in the revetment to allow for public pedestrian beach access typically via stairs. Three stormwater outlets are located within the face of the revetment. One of these was fitted with a floodgate which was blocked due to sediment infilling from the beach at the time of the inspection (31 October 2011, Figure B-33). Short Beach Creek also has an outlet at the southern end of Caseys Beach. It runs under the Beach Road bridge and is constrained by the bridge abutments. At commencement of the site inspection, the creek entrance was initially closed but "broke out" during the inspection. Car parks exist at both ends of the beach. Reefs exist off the northern and southern ends of the beach providing some protection from wave attack.

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).

Since WRL's original site inspection on 31 October 2011, WRL also prepared a detailed condition assessment and design advice report for the seawall along the southern part of Caseys Beach which protects the sewage pumping station (Coghlan and Drummond, 2013). In April 2017, upgrade works on this seawall section were completed.

Figure B-31: Caseys Beach Site Details

a) Rock/reef at northern end

b) View of beach looking south

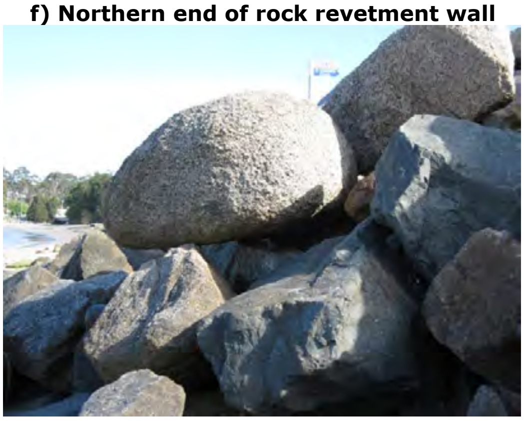
c) Stormwater outlet

d) Stormwater outlet

e) Typical pedestrian beach access

f) Northern end of rock revetment wall

g) Some slumping of armour at the crest



h) Example of precariously positioned rock

Figure B-32: Caseys Beach Site Inspection (1 of 2)

a) Short Beach Creek outlet

b) Creek outlet following "break out"

c) Scour protection at bridge abutments

d) Unprotected section of beach

e) Sewage pumping station

f) Stormwater outlet (blocked floodgate)

g) Southern end of rock revetment wall

h) Rock/reef at southern end

Figure B-33: Caseys Beach Site Inspection (2 of 2)

B.11 Sunshine Bay

Sunshine Bay is a 520 m long beach with a low gradient, facing east to north-east (Figure B-34). The bay is semi-circular and is located between two well vegetated 20 m high headlands. The beach gradient reduces north to south with scarps noticeable along the foreshore. It is bordered and fronted by considerable rock and reef resulting in a low wave climate. There is no natural dune at Sunshine Bay (Figure B-35). A natural rock outcrop at the northern end of the beach had formed a small salient at the time of the site inspection (5 December 2012). Additional protection from wave attack is also provided by the Tollgate Islands to the south-south-east.

Beach Road runs just behind the beach with a small parking area towards the southern end, opposite a caravan park (Pleasurelea Tourist Resort). The car park is positioned landward of a low point in the beachface providing informal pedestrian access to the beach. Adjacent to the south side of the car park are several houses including a boatshed with a launching ramp and small wooden retaining wall. These structures are located at low elevations in close proximity to the beach. A summary of sand sample analysis is shown in Figure B-36.

Figure B-34: Sunshine Bay Site Details

a) View of beach looking north

b) View of beach looking south

c) Loss of vegetation

d) Scarp at the southern end of the beach

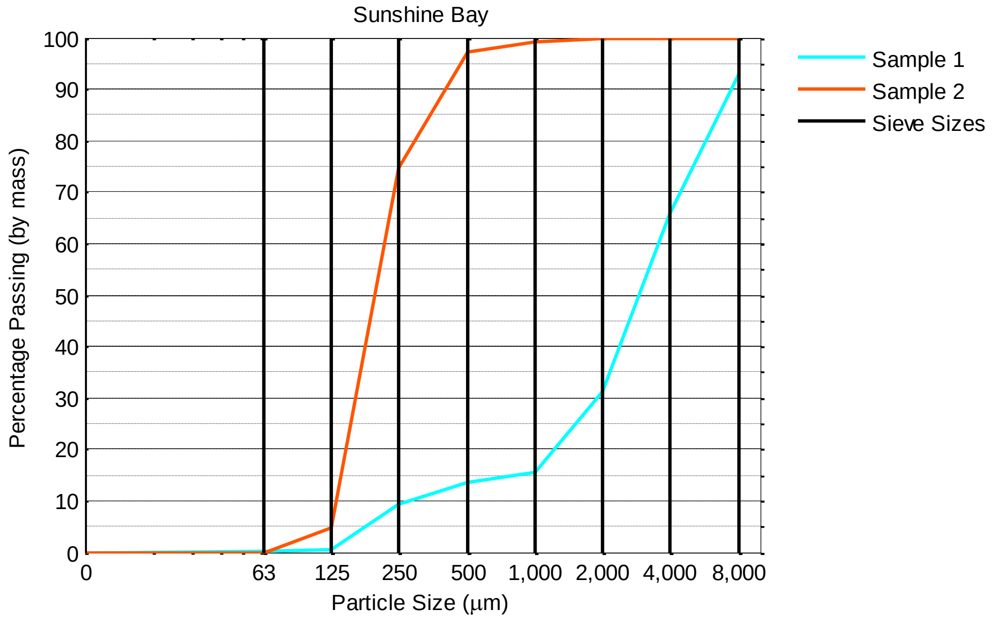
e) Wooden retaining wall at southern end

f) Rock outcrop with salient

g) Informal pedestrian beach access

h) Typical house located at northern end

Figure B-35: Sunshine Bay Site Inspection



Sample 1

Sample 2

Figure B-36: Sunshine Bay Sediment Samples

B.12 Malua Bay

Malua Bay is a 510 m long beach with a low gradient, facing east (Figure B-37). It is bordered by Malua Head to the north and the base of Pretty Point to the south. Two creeks drain across the beach: Reedy Creek at the northern end and a small creek at the southern end (Figure B-38). Reedy Creek entrance is typically open and drains across a rock bed at the base of Malua Head, whereas, the creek mouth at the southern end is rarely open. These typical entrance conditions were observed at the time of the site inspection (5 December 2012). Both creeks are heavily vegetated along their banks and are controlled by culverts under George Bass Drive. Reedy Creek is controlled by three 3 m wide box culverts approximately 200 m landward of the beach and the southern creek is controlled by three 1.8 m concrete pipes approximately 100 m landward of the beach.

Malua Bay is exposed to a moderate wave climate which usually maintains an attached bar with a rip against the northern rocks. Higher waves produce a southern boundary rip and a shifting central rip, which are at times linked by a continuous trough.

There is no natural dune at Malua Bay, but the beach is backed by a grassed picnic area allowing uncontrolled pedestrian access along the entire beach. The park contains the Malua Bay SLSC, a picnic area with public amenities, walking paths, a playground and shops fronting George Bass Drive. Parking is available next to the shops at the northern end and on landward of Malua Bay SLSC.

At the eastern end of Kuppa Avenue, an apartment block and a telecommunications pit are located at low elevations in close proximity to the beach. Some houses at the southern end of Malua Bay are also in close proximity to the beach.

Some houses at the southern end of Malua Bay are protected by a revetment wall. Primary armour on the revetment is composed of approximately 0.7 m size rock but no underlayer or secondary armour are apparent. The structure slope of the revetment wall is steep at approximately 1.0V:1.0H. A summary of sand sample analysis is shown in Figure B-39.

Figure B-37: Malua Bay Site Details


a) View of beach looking north


b) View of beach looking south


c) Blocked stormwater pit at southern end


d) Reedy Creek outlet at northern end


e) Well vegetated dunes


f) Box culverts under George Bass Drive


g) Rock revetment at southern end

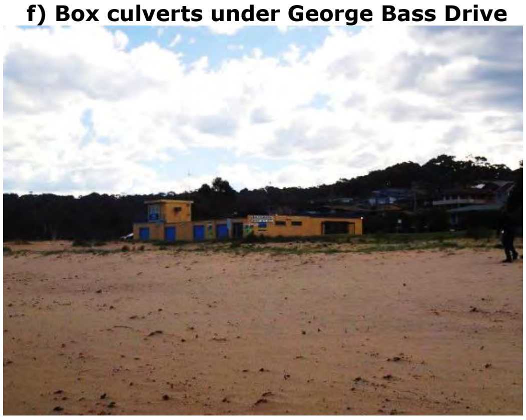
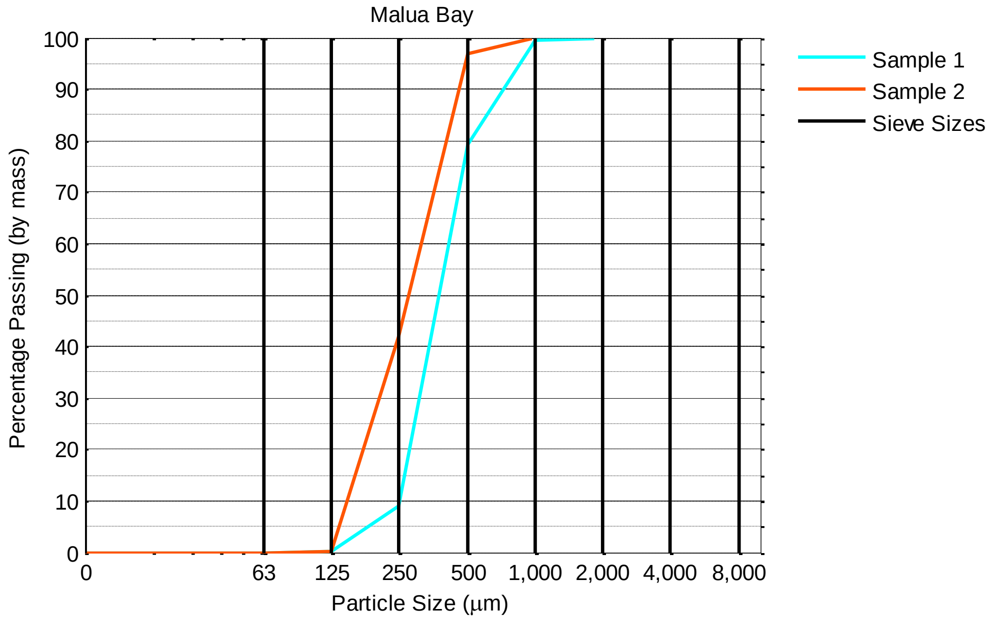

h) Malua Bay Surf Life Saving Club

Figure B-38: Malua Bay Site Inspection



Sample 1

Sample 2

Figure B-39: Malua Bay Sediment Samples

B.13 Guerilla Bay

Guerilla Bay is a 290 m long beach with a high gradient, facing north-east (Figure B-40). It is located at the southern end of a 500 m wide bay. The beach is bordered by a 200 m long rock platform to the north and the base of Burrewarra Point to the south. It is backed by vegetated bluffs with a creek mouth at its centre (Figure B-41). This creek entrance is typically closed; this was the case at the time of the site inspection (5 December 2012). The control point for the creek is two 0.5 m culverts located under Beach Parade.

The beach is well protected from incident waves from all directions except north-east. It is generally free of rips except during higher seas when one flows out against the northern rocks.

A large, healthy dune is located landward of the beach. Guerilla Bay can be accessed via a small car park off Ocean Street. Houses at the northern and southern ends of the beach are located on rock cliffs. At the time of the site inspection, recent undercutting of the cliff was observed at the southern end. There are several properties with varying degrees of development located at low elevations in close proximity to the beach. A summary of sand sample analysis is shown in Figure B-42.

Figure B-40: Guerilla Bay Site Details

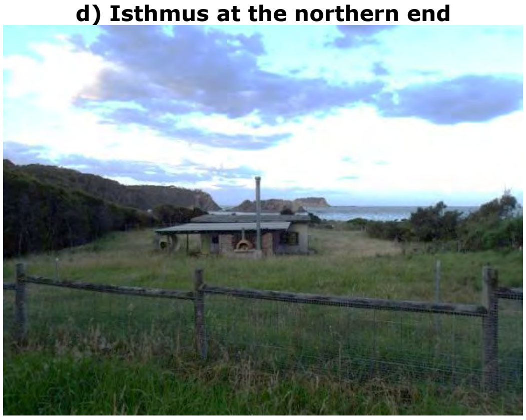

a) View of beach looking north


b) Rock/reef at southern end


c) Stormwater outlets


d) Isthmus at the northern end

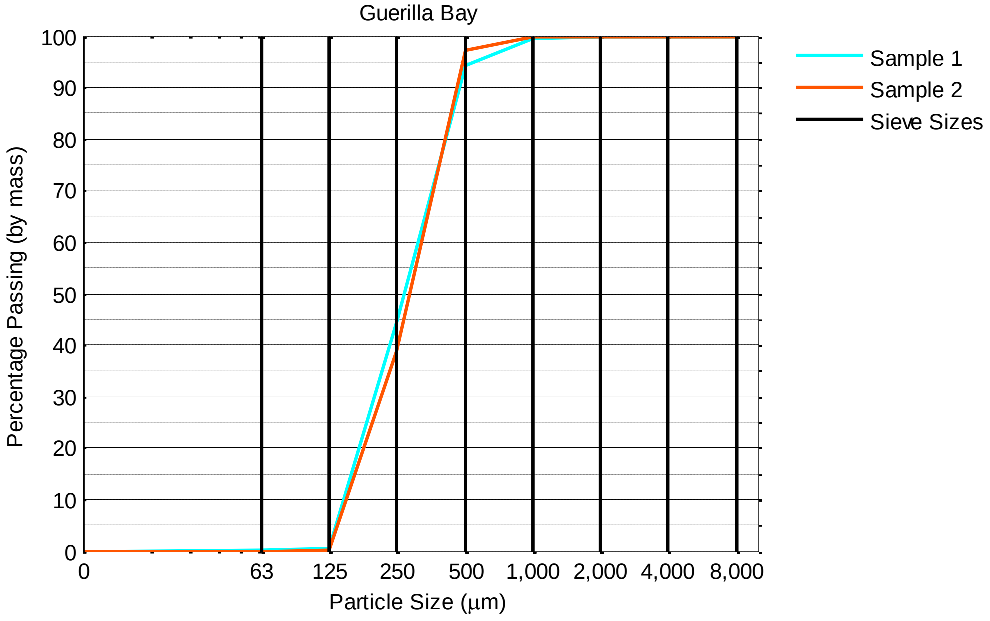

e) Backing rocky bluffs with rockfall


f) Cabin at the centre of the beach


g) House at the southern end


g) Scour of the banks of the creek

Figure B-41: Guerilla Bay Site Inspection



Sample 1

Sample 2

Figure B-42: Guerilla Bay Sediment Samples

B.14 Barlings Beach

Barlings Beach is a 1.11 km long beach with a low gradient, facing south (Figure B-43) which was inspected on 8 December 2012. It is located between the 15 m high Barlings Island (eastern end) and the conical 25 m high Melville Point (western end). The beach is part of a 500 m wide series of regressive foredune ridges, fronted by 100 m of now vegetated transgressive dunes. A creek is landward of the sand dune and terminates in a small wetland area at the eastern end of the beach (Figure B-44). The creek is controlled by five box culverts (2.7 x 1.5 x 1.22 m) and granite rock scour protection located landward of the beach.

Wave climate exposure generally increases from the east to the west along Barlings Beach. This results in a near permanent rip against Melville Point and up to six rips up the beach, usually separated by an attached bar.

A large, well vegetated dune is located landward of the beach. Behind the eastern end of the beach is a caravan park with pedestrian access to the beach. Four-wheel drive beach access for boat launching is available along a dirt track at the eastern end too. At the western end of the beach, a new development is set back behind the foredune (at least 100 m from the beach), with the dune providing a natural buffer against erosion and inundation. Uncontrolled pedestrian access along the western end of the beach is available via the development and a small car park located off Sun Patch Parade. A summary of sand sample analysis is shown in Figure B-45.

Figure B-43: Barlings Beach Site Details

a) View of beach looking east

b) View of beach looking west

c) Box culverts landward of the beach

d) Backing rocky bluffs at western end

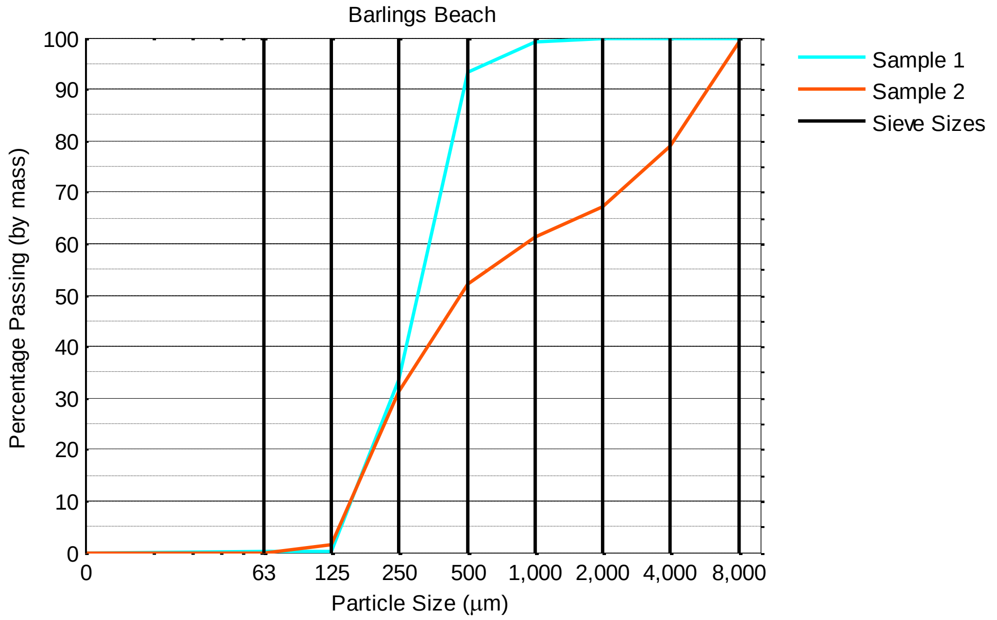
e) Barlings Island at eastern end of beach

f) New development at eastern end

g) 4WD beach access corridor

h) Creek entrance

Figure B-44: Barlings Beach Site Inspection



Sample 1

Sample 2

Figure B-45: Barlings Beach Sediment Samples

B.15 Tomakin Cove and Tomakin Beach

Tomakin Cove is a 270 m long beach with a low gradient, facing south-east (Figure B-46) which was inspected on 8 December 2012. Tomakin Cove is located between Melville Point in the north and a sand tombolo in the lee of a reef in the south. Shallow reefs extend near continuously between these two boundaries resulting in low wave energy impacting the beach (Figure B-47).

A lagoon is located between the base of the cove and the reefs. The cove is backed by a densely vegetated foredune, which provides a buffer against erosion for the houses located in its lee. However, there is no additional set back between the rear of the foredune and the houses. A small stormwater outlet with a diameter of 0.4 m is located at the centre of Tomakin Cove just north of an informal beach access point.

Tomakin Beach is a 900 m long beach with a low gradient, facing south-east (Figure B-46). It is located between Tomakin Cove in the north and the mouth of the Tomaga River in the south. Reefs at the northern end of the beach extend 400 m southward and Mossy Point (south of Tomaga River) protects the southern end of the beach. Wave climate exposure is generally greatest at the centre of the beach, reducing towards either end. During higher waves, the beach terrace is cut by rips in the centre of the beach (Figure B-48).

Shoaling of the river mouth extended up to 200 m behind the beach, creating a flat, wide spit and constricting the narrow river channel between the shoals and the rocks of Mossy Point. The concrete abutments on the Tomakin Bridge on George Bass Drive provide a control point approximately 2 km upstream of the mouth of the Tomaga River. The southern side of the river mouth is constrained by natural rock shelves and cliffs at Mossy Point.

The beach is backed by a vegetated foredune with formal pedestrian beach access available from a small car park off the end of Reid Street. The dune narrows to a 10 m wide sand spit at its centre. There are several houses at the end of Kingston Place (at the northern end of the beach) which are located at low elevations in close proximity to the beach. Several houses, boat sheds and jetties on the southern side of the Tomaga River (Mossy Point) are also located at low elevations. A summary of sand sample analysis is shown in Figure B-49.

Figure B-46: Tomakin Cove and Beach Site Details

a) View of cove looking north

b) View of cove looking south

c) Backing rocky bluffs at the northern end

d) Scarp in the middle of the cove

e) Pedestrian beach access at northern end

f) Houses at the middle of the cove

g) Houses at the southern end

h) Stormwater outlet

Figure B-47: Tomakin Cove Site Inspection

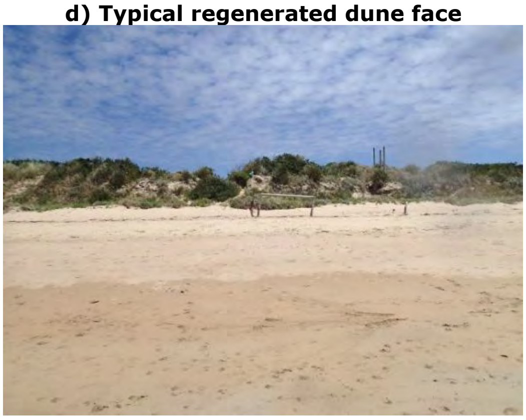

a) Tomakin Beach looking east


b) Rock/reef at northern end


c) View of beach looking south


d) Typical regenerated dune face

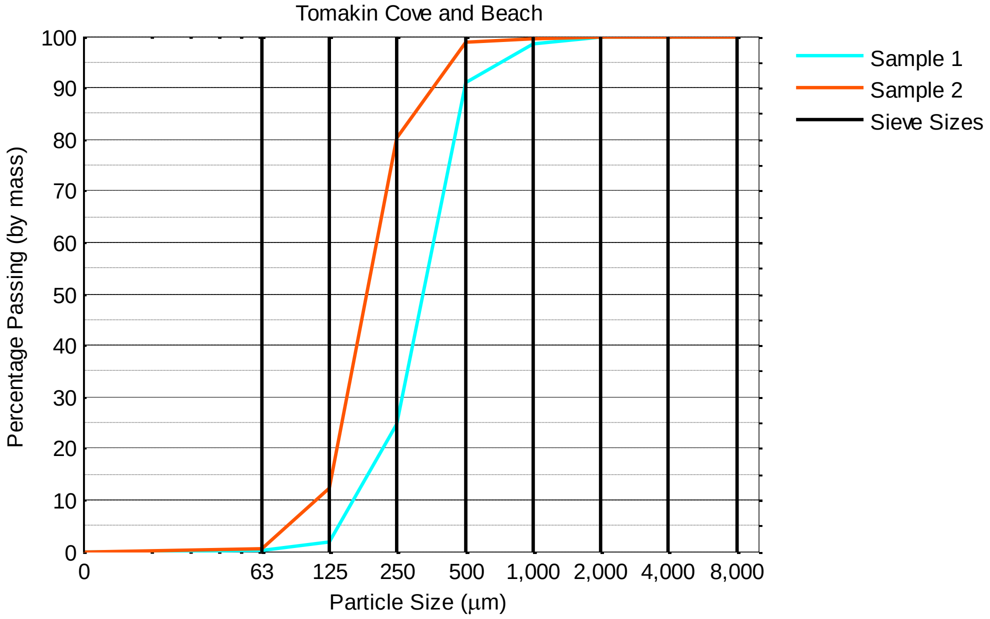

e) View of Tomaga River from the centre


f) Scarp in the middle of the beach


g) Houses at northern end


h) Tomaga River entrance

Figure B-48: Tomakin Beach Site Inspection




Sample 1


Sample 2

Figure B-49: Tomakin Cove and Beach Sediment Samples

B.16 Broulee Beach

Broulee Beach is a 1.74 km long beach with a low gradient, facing east-north-east (Figure B-50). It is bordered by Mossy Point to the north and to the south by a tombolo, known as Broulee Spit, which connects Broulee Island to the mainland. At the time of the site inspection (8 December 2012), Broulee Island was connected to the mainland, that is, Broulee Spit was closed (Figure B-51). However, the island has been separated from the mainland at times in recent decades (see Appendix H). The beach forms the seaward boundary of a 1 km wide foredune ridge plain which has accumulated over the past 6,000 years. The entrance to Candlagan Creek is located at the northern end of the beach across a rock platform at the base of Mossy Point. The Beach Road bridge crosses the creek just upstream of the mouth and its concrete abutments are protected on both sides by revetments comprising approximately 0.7 m rock. The rock protection extends further to the east on the northern side of Candlagan Creek to protect a small car park.

Broulee Beach experiences a moderate wave climate with exposure generally reducing from north to south. Under typical conditions, the beach maintains an attached bar with a rip against the northern rocks (assisted by flow from Candlagan Creek) and several beach rips are usually present up to the middle of the beach, grading southwards to a low tide terrace along most of the southern half before finishing in a reflective beach in the southern corner. Low wave energy conditions at the southern end creates a wide, flat beach. On the southern side of Broulee Spit, the beach face has a high gradient.

A healthy, vegetated dune exists along the entire beach with several formal beach access points along Coronation Drive. This foredune provides a buffer between the beach and the road. Houses landward of Coronation Drive appear to be located well landward of the active beach. However, five houses at the northern end of the beach and on the southern side of Candlagan Creek are located in close proximity to the beach. These houses sit on the crest of the sand dune with limited sand stores and vegetation on the seaward side.

The southern end of the beach can be reached on foot from Bayside Street, Harbour Drive or a small headland car park at the end of Albert Street. The crest of the dune along the tombolo was approximately 3 m wide at the time of the site inspection (8 December 2012), although it is infrequently cut by large seas from the south (Appendix H). Houses at this end of Broulee Beach were set well back from the shoreline at the time of the site inspection. A sewage pumping station is also located on Bayside Street and is similarly set well back. A summary of sand sample analysis is shown in Figure B-52.

Figure B-50: Broulee Beach Site Details


a) View of the beach looking south


b) Vegetated sand reserve at southern end


c) Candlagan Creek outlet at northern end


d) Beach Road bridge over Candlagan Creek

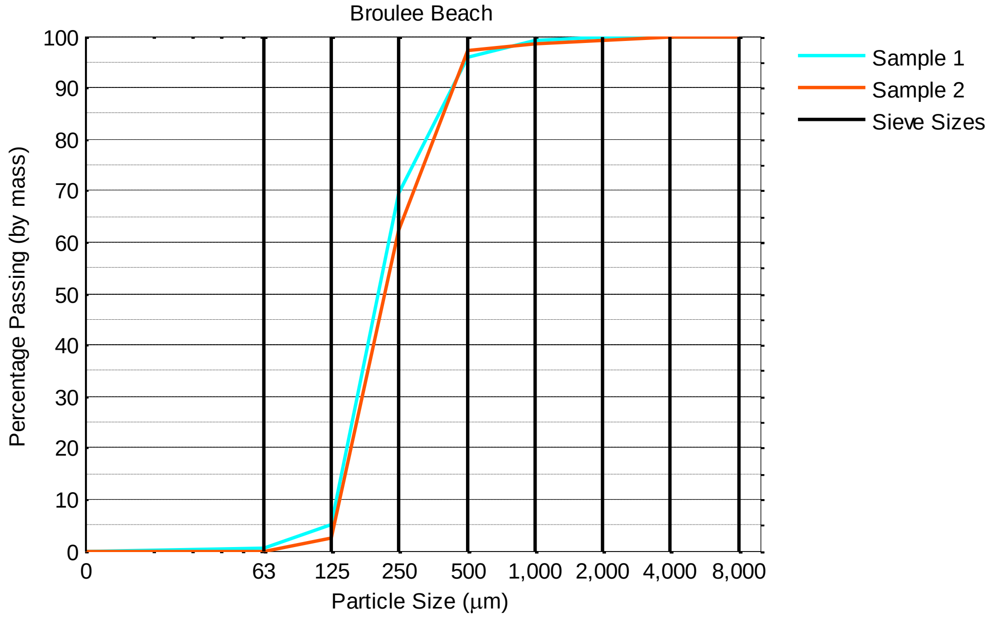

e) Rock protection for small car park


f) Houses at the northern end


g) Sewage pumping station


h) Broulee Tombolo and Island

Figure B-51: Broulee Beach Site Inspection



Sample 1

Sample 2

Figure B-52: Broulee Beach Sediment Samples

B.17 Bengello Beach

Bengello Beach is a 6 km long beach with a low gradient, facing south-east and east (Figure B-53). It is one of the longest beaches on the NSW south coast. Bengello Beach is bordered to the north by Broulee Head and to the south by the northern training wall of the Moruya River. The entire beach is backed by a 1 to 2 km wide series of low, densely vegetated foredune ridges, which formed when the shoreline built out seaward between 6,000 and 3,000 years ago. The usually closed mouth of Waldrons Creek is located near the centre of the beach (Figure B-54). At the time of the site inspection (4 December 2012), Waldrons Creek entrance was closed. Several 3.6 m wide box culverts under George Bass Drive are a control point for the creek.

Wave climate exposure is greatest at the centre of the beach, with Broulee Head (and the surrounding reefs) protecting the northern end of the beach from incident waves north of east and the northern training wall, tidal shoals and Toragy Point (Moruya Heads) protecting the southern end from waves south of east.

The beach is accessible at the northern end where Broulee Surfers SLSC is located. There is a car park next to the SLSC and a second car park at the base of the headland near a small boat ramp and 0.8 m diameter stormwater outlet. The SLSC and caravan park (Big4 Broulee Beach Holiday Park) are located well landward of the dunes, whereas the car park at the base of the headland and the road (Heath Street) are located at low elevations in close proximity to the beach. A gated gravel road runs south behind the beach for 2 km providing vehicular access as far as Waldrons Creek which breaks out across the beach during floods. Large seas in 1975 resulted in the permanent closure of a section of this gravel road which ran the full length of the beach (Short, 2007). George Bass Drive now runs south behind the dunes and is located 1 to 2 km inland. George Bass Drive meets Bruce Cameron Drive which runs along the northern side of the Moruya River and terminates at Moruya Airport. There is also a car park at the southern end providing access to the beach. In addition to Moruya Airport, a caravan park (North Head Camp Ground) is also located landward of the southern end of Bengello Beach.

There are two coastal structures at either end of Bengello Beach. At the northern end of the beach, a small revetment protects the car park situated just off Heath Street. The revetment structure is composed of granite rock primary armour with an approximate size of 0.5 m, with no underlayer or secondary rock visible. The structure slope of the revetment wall is approximately 1.0V:1.5H. At the southern end of the beach, the northern training wall of the Moruya River interrupts littoral sand transport. The northern training wall has a pedestrian footpath along its entire length and has primary armour on both sides consisting of granite rock with an approximate size of 1.0 m, a geotextile underlayer and secondary rock armour. A summary of sand sample analysis is shown in Figure B-55.

Figure B-53: Bengello Beach Site Details

a) View of beach looking south

b) View of beach looking north

c) Boat ramp and rock revetment

d) Moruya River training wall (north)

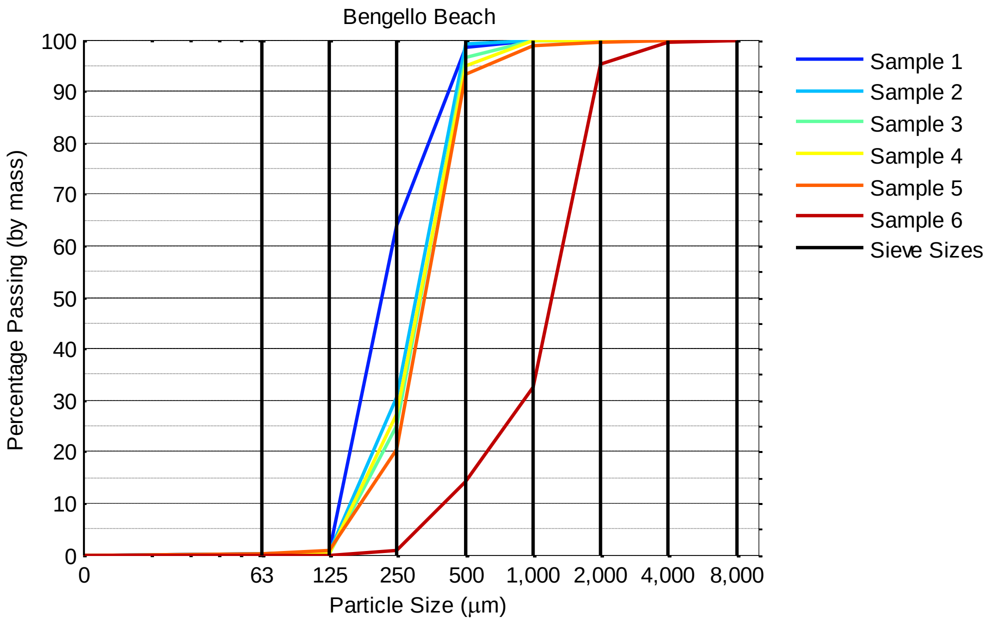
e) Broulee Surfers SLSC

f) Dune scarp in the middle of the beach

g) Waldrons Creek outlet

h) Northern end of Moruya Airport

Figure B-54: Bengello Beach site Inspection



Sample 1

Sample 2

Sample 3

Sample 4

Sample 5

Sample 6

Figure B-55: Bengello Beach Sediment Samples

Appendix C: Photogrammetry

C.1 Preamble

Photogrammetry data was available for a number of the beaches within the Eurobodalla Shire Council region, provided to WRL by NSW OEH and ESC (Jacobs, 2015). Photogrammetry data is one of the only survey datasets sets available in NSW to assess historical, long term changes on the NSW coastline. This appendix summarises the available photogrammetry data, and the analysis of this data undertaken by WRL.

C.2 Photogrammetry Data

Photogrammetry data is available for all of the beaches where erosion modelling has been undertaken. The years of available data are summarised in Table C-1. Every photogrammetry dataset was used in the analysis of underlying recession and storm demand (except for 1942 at Long Beach and 1972 Broulee Beach, Block M).

Table C-1: Summary of Photogrammetric Data (Source: NSW OEH, 2015 and Jacobs, 2015)

Coastline Sub-Section	Year
Maloneys Beach	1942, 1972, 1990, 1993, 1999, 2007, 2011, 2014
Long Beach	1942*, 1959, 1972, 1990, 1993, 1999, 2007, 2011, 2014
Surfside Beach (east)	1942, 1959, 1972, 1975, 1977, 1990, 1993, 1999, 2007, 2011, 2014
Surfside Beach (west)	1942, 1959, 1972, 1990, 1993, 1999, 2007, 2011, 2014
Sunshine Bay	1962, 1972, 1991, 2011, 2014
Malua Bay	1962, 1972, 1984, 1991, 2003, 2011, 2014
Guerrilla Bay	1962, 1972, 1984, 1991, 2011, 2014
Barlings Beach	1964, 1972, 1975, 1980, 1984, 1993, 2003, 2011
Tomakin Cove	1962, 1972, 1975, 1984, 1993, 2001, 2011, 2014
Broulee Beach	1962, 1972*, 1980, 1993, 2011

* NSW OEH has advised that the 1942 photogrammetry data at Long Beach (both blocks) and 1972 photogrammetry data for Broulee Beach Block M (southern third of the beach) is comparatively less accurate (possibly due to datum shifts) than the other data sets. WRL has excluded this data from its analysis.

The accuracy of photogrammetry is dependent on many factors, including the height at which the image was taken, distortions for physical features of the land (including the curvature of the earth and relief displacement), and distortions from the camera. While all modern cameras used for photogrammetry are calibrated to allow such corrections, no such calibrations were performed for camera distortions prior to 1960 (Hanslow, 2007). Pre-1960's surveys are therefore less accurate. DWLC (1996) provides a summary of photogrammetric accuracy for the photogrammetry surveys around Corrigans Beach, stating accuracies post the 1960's to be 0.4 – 0.5 m in the horizontal direction and 0.3 – 0.4 in the vertical direction. This is similar, although slightly less accurate in the vertical direction, to the approximate general accuracy of all the NSW photogrammetry stated in Evans and Hanslow (1996) and summarised in Table C-2.

Table C-2: Photogrammetry Accuracy, Where No Specific Analysis of Accuracy is Available (Source: Evans and Hanslow, 1996)

Year	Vertical Accuracy (m)	Horizontal Accuracy (m)
Pre-1960	±0.7	±1
Post-1960	±0.2	±0.5

C.2.1 Locations of Photogrammetry

At each beach, the processed photogrammetry data has been provided at discrete profiles along the beach. Profile spacing varies between 20 m – 50 m, depending on the location. Figures C-1 to C-10 show the profile locations at each of the beaches where erosion modelling was undertaken. Each profile is identified by a block name (letter or number) and profile number (e.g. BEP19 – Block E, Profile 19).

Figure C-1: Maloneys Beach profile locations

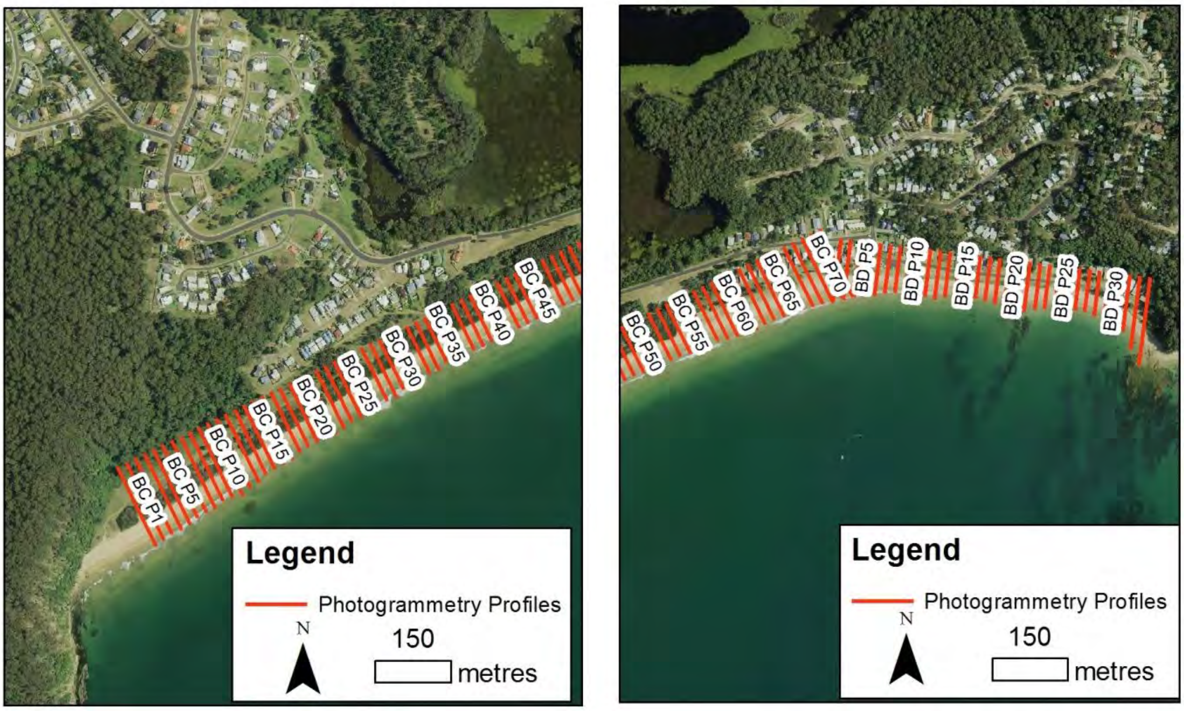


Figure C-2: Long Beach profile locations (not all profiles are labelled)

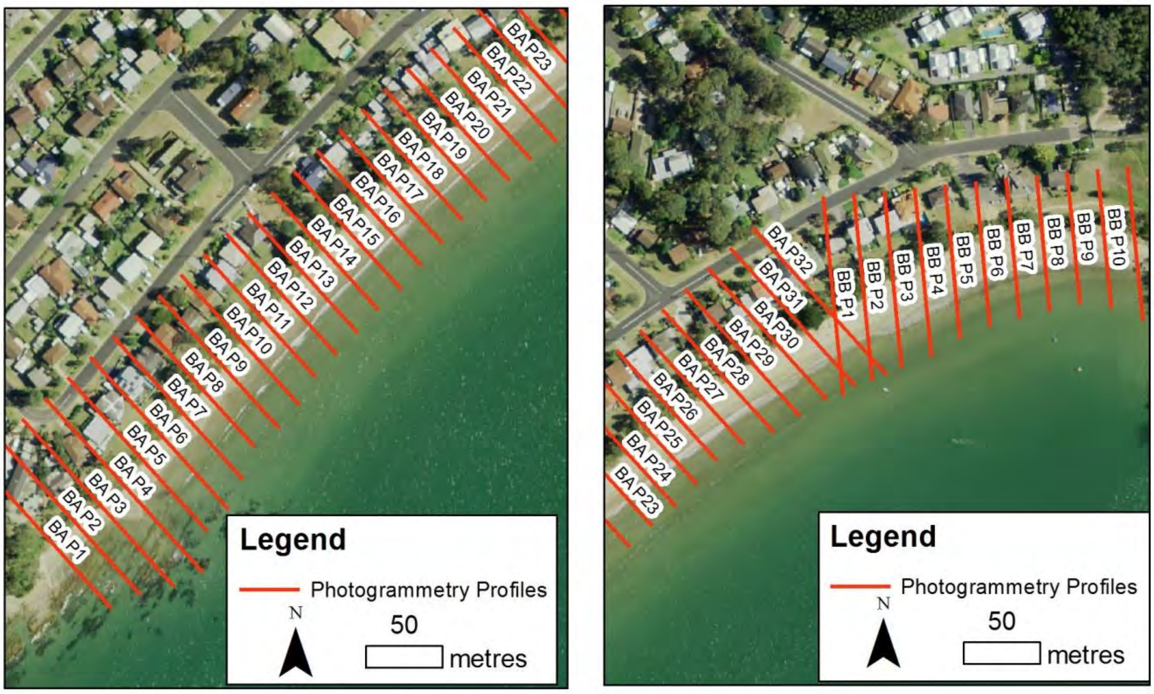


Figure C-3: Surfside Beach (east) profile locations

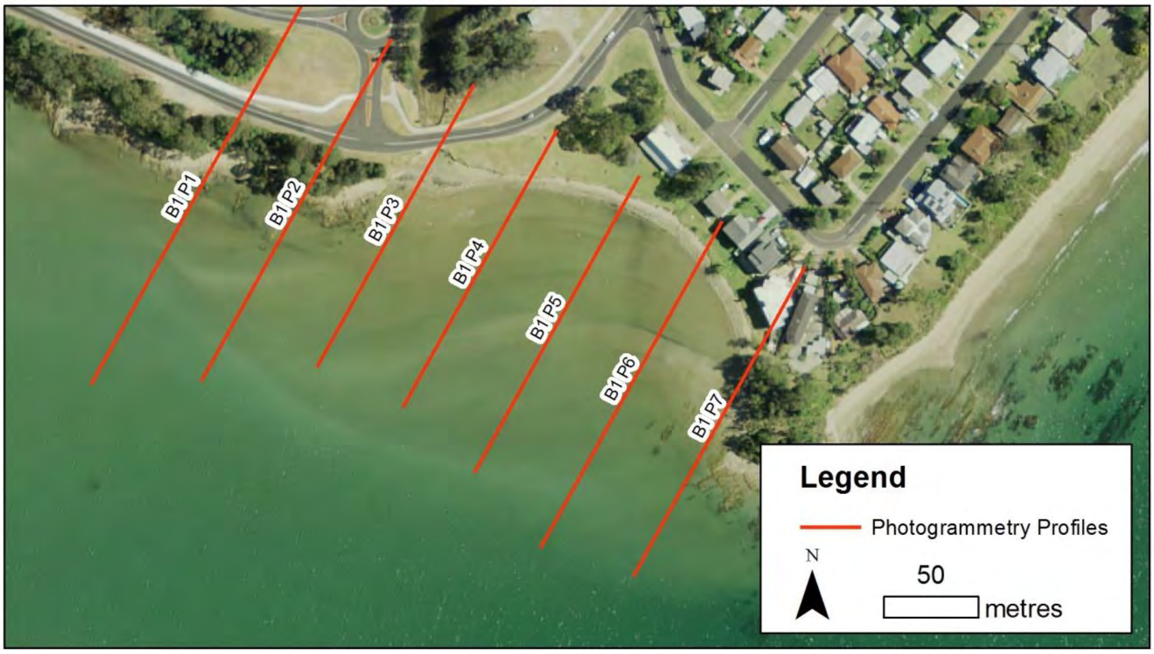


Figure C-4: Surfside Beach (west) profile locations

Figure C-5: Sunshine Bay profile locations

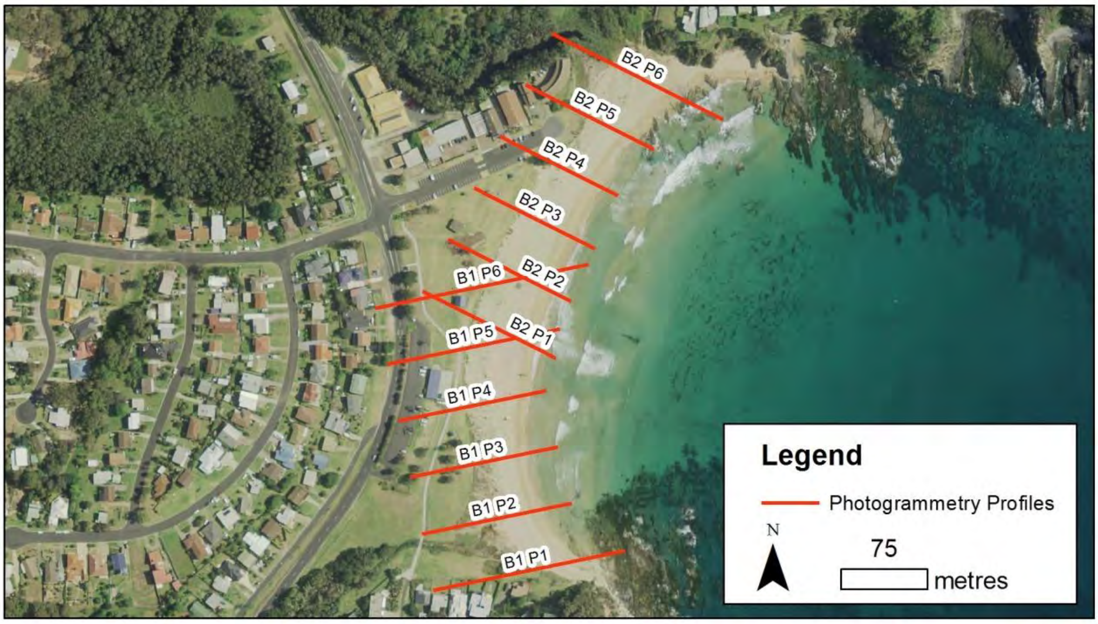


Figure C-6: Malua Bay profile locations

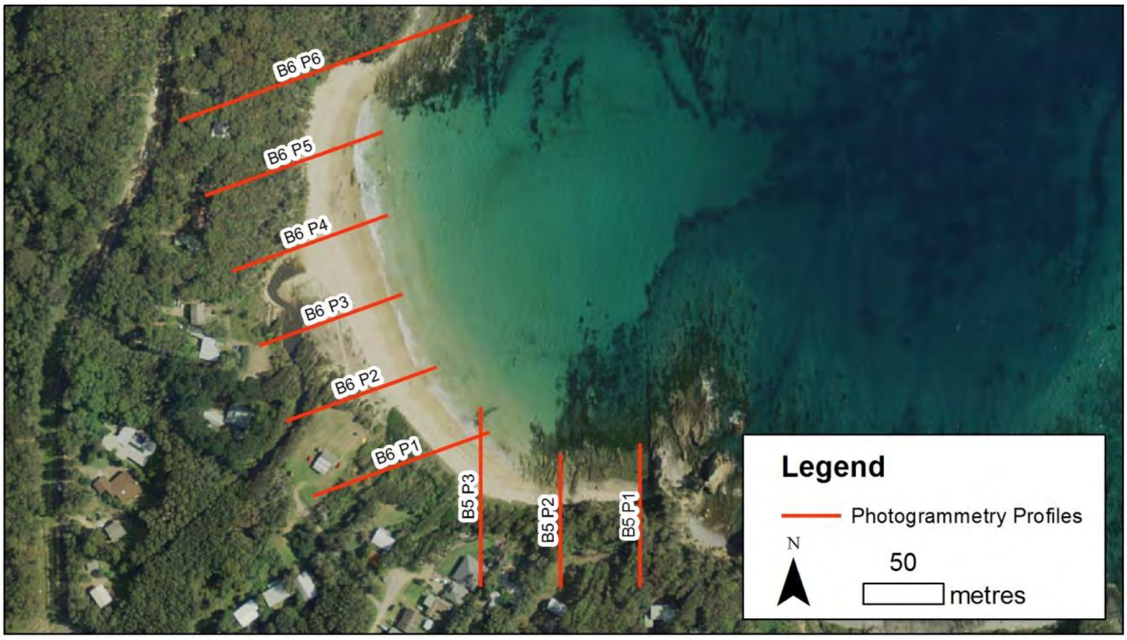


Figure C-7: Guerilla Bay profile locations

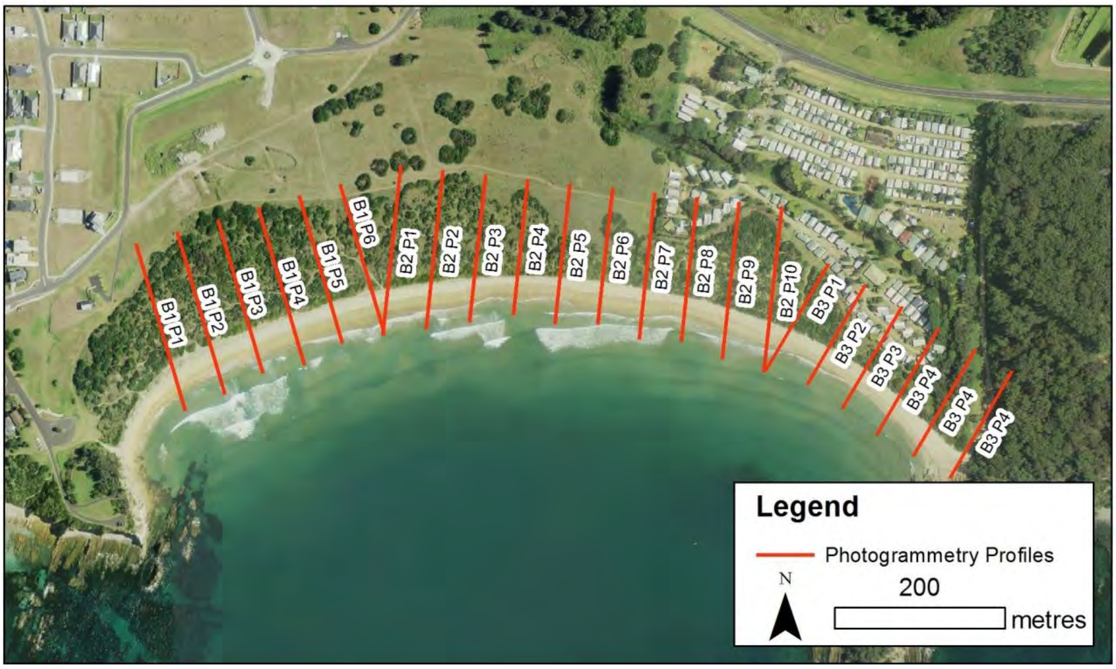


Figure C-8: Barlings Beach profile locations

Figure C-9: Tomakin Cove profile locations

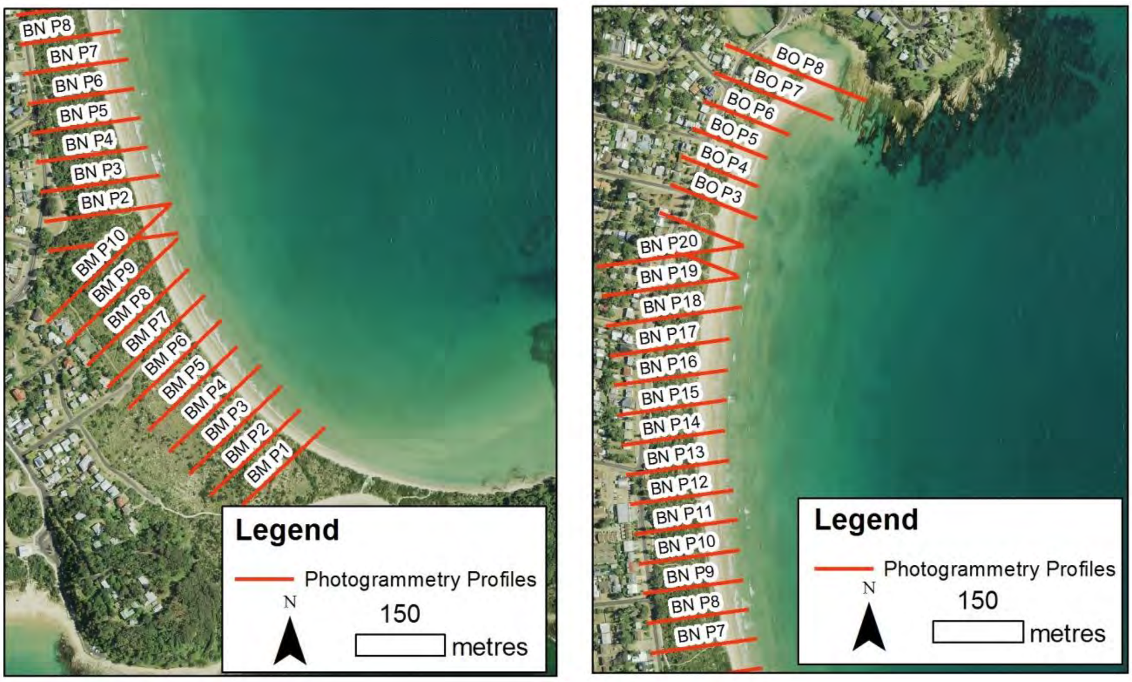


Figure C-10: Broulee Beach profile locations

C.2.2 Photogrammetry Cross Sections

Example photogrammetry cross sections for each beach section where erosion modelling was undertaken are shown in Figures C-11 to C-27.

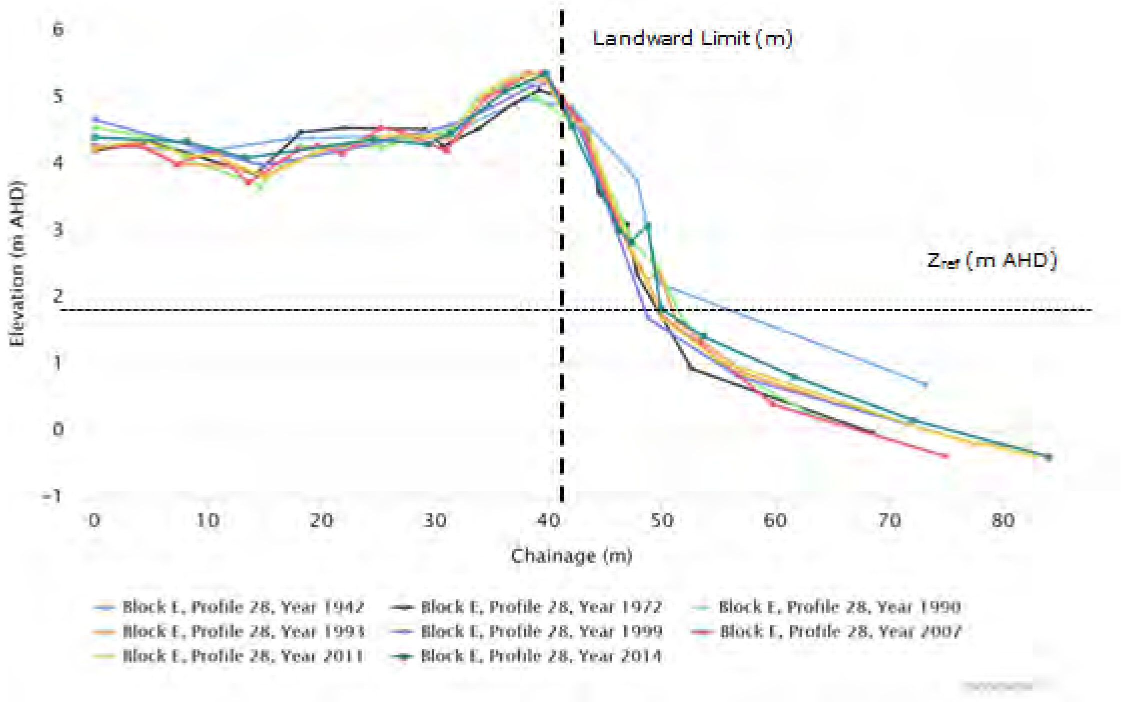


Figure C-11: Example photogrammetry cross sections at Maloneys Beach East (Block E, Profile 28)

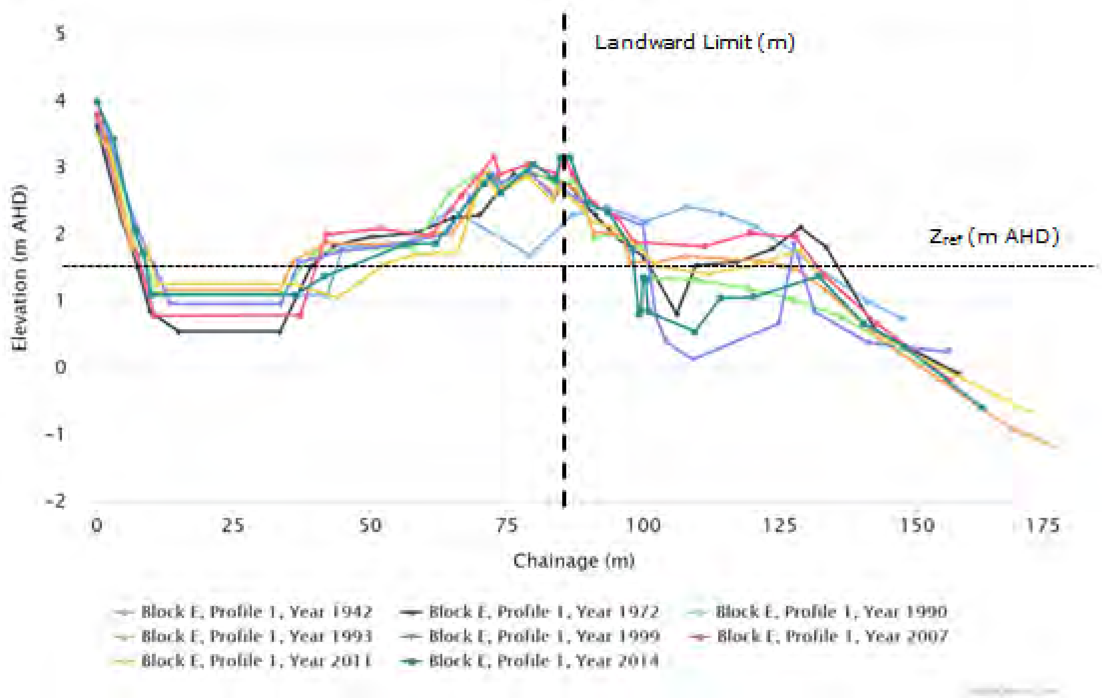


Figure C-12: Example photogrammetry cross sections at Maloneys Beach West (Block E, Profile 1)

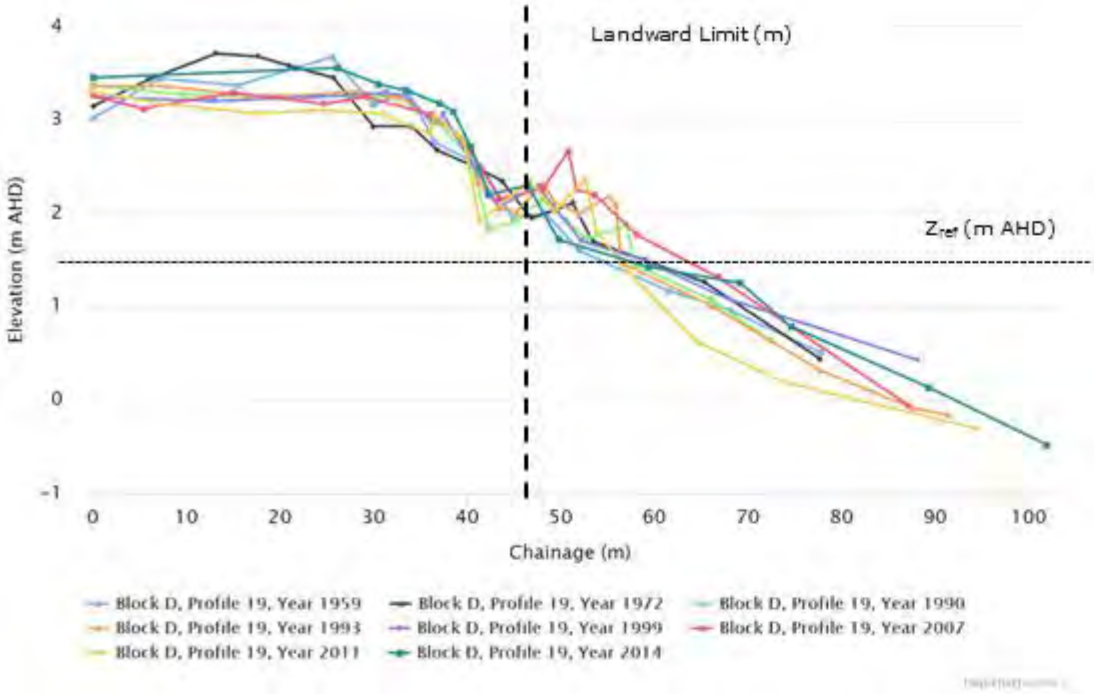


Figure C-13: Example photogrammetry cross sections at Long Beach East (Block D, Profile 19)

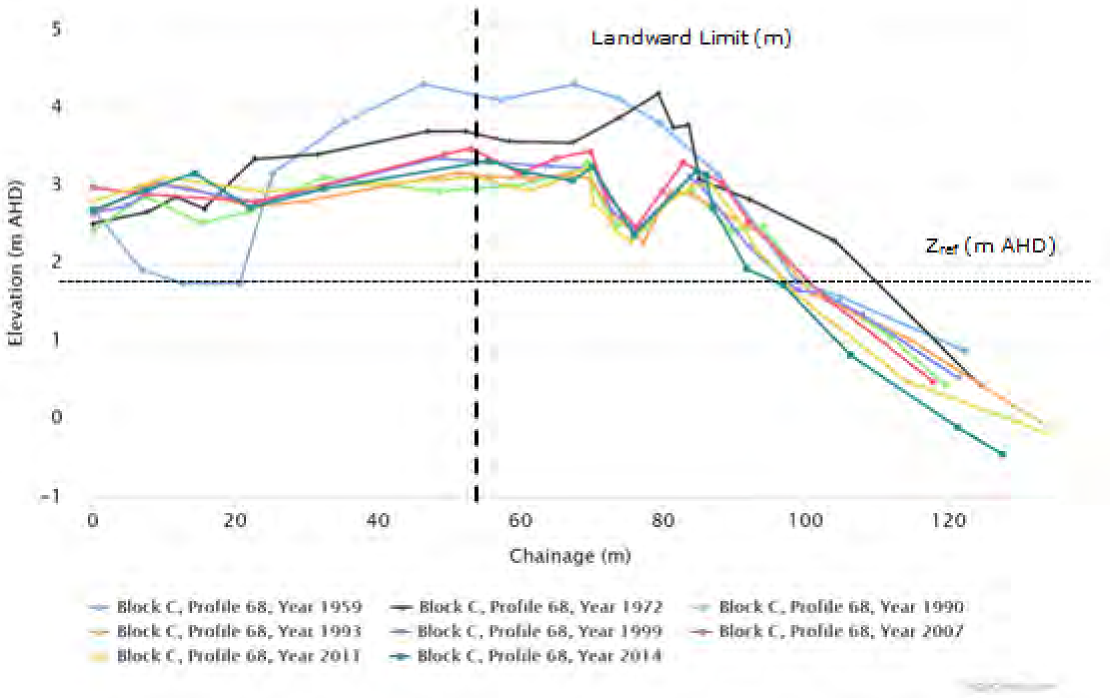


Figure C-14: Example photogrammetry cross sections at Long Beach Central (Block C, Profile 68)

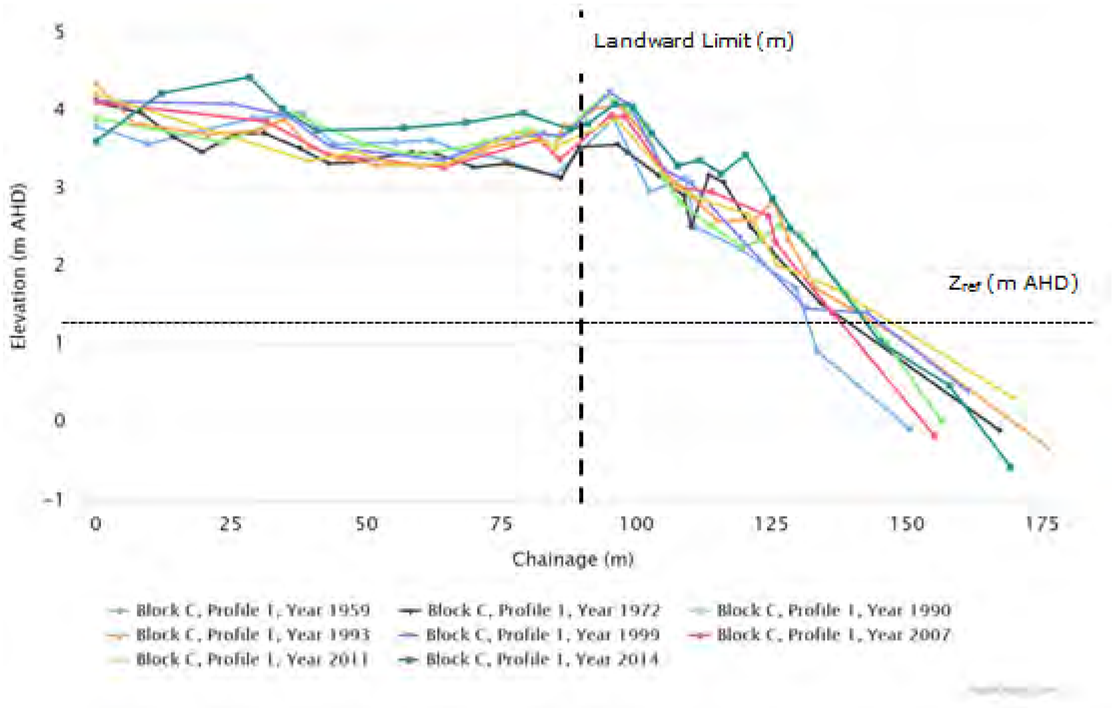


Figure C-15: Example photogrammetry cross sections at Long Beach Central (Block C, Profile 1)

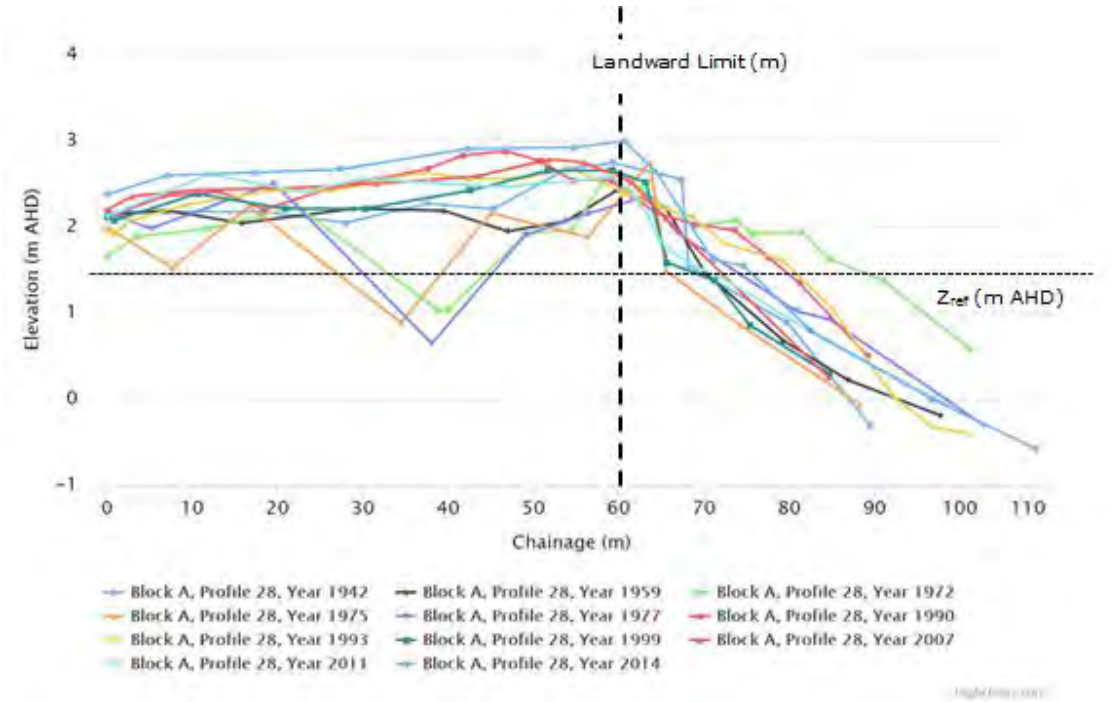


Figure C-16: Example photogrammetry cross sections at Surfside Beach (east) North (Block A, Profile 28)

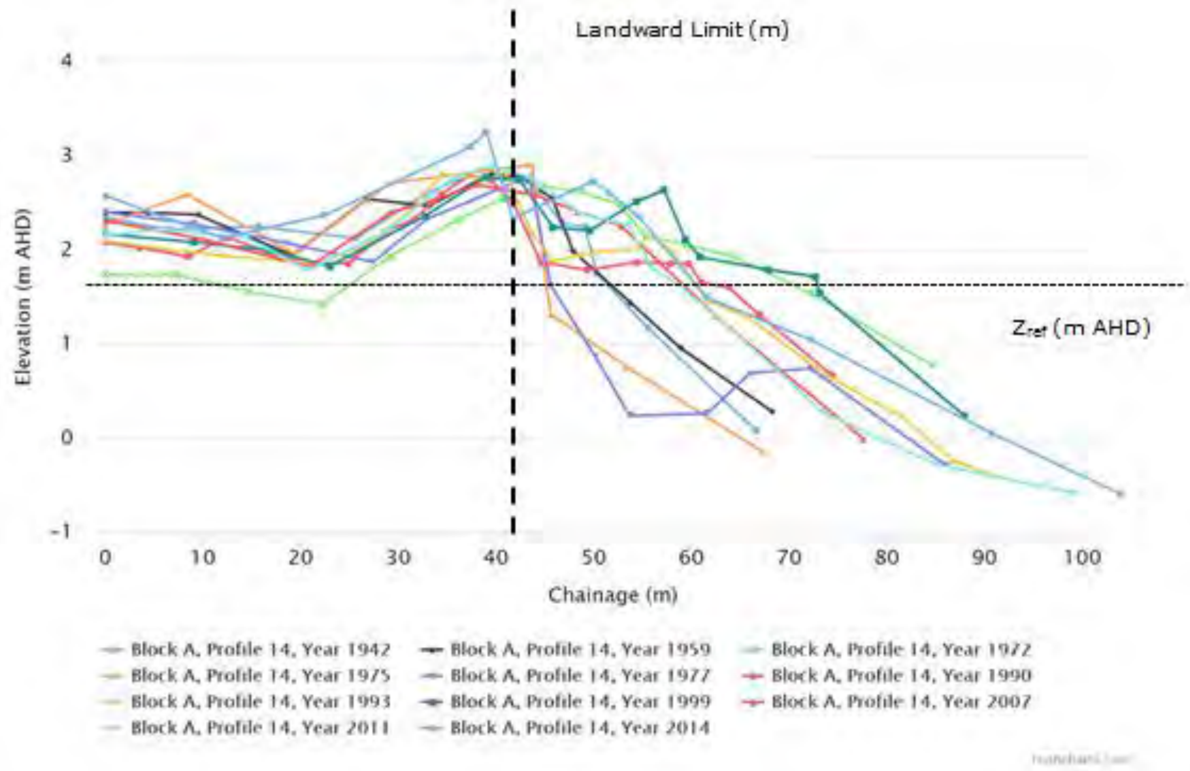


Figure C-17: Example photogrammetry cross sections at Surfside Beach (east) South (Block A, Profile 14)

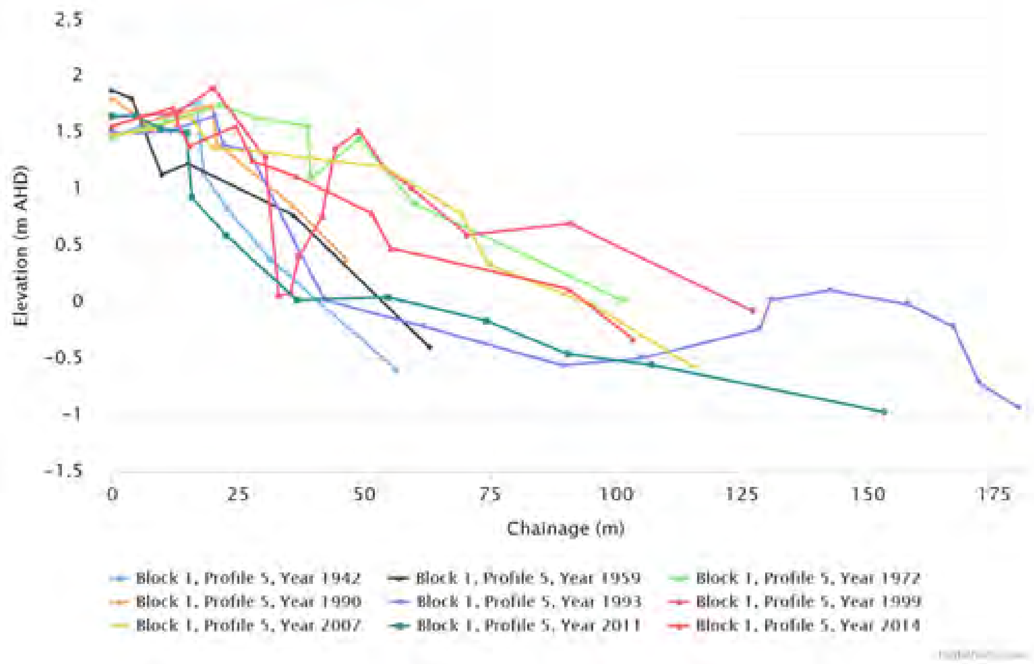


Figure C-18: Example photogrammetry cross sections at Surfside Beach (west) (Block 1, Profile 5)

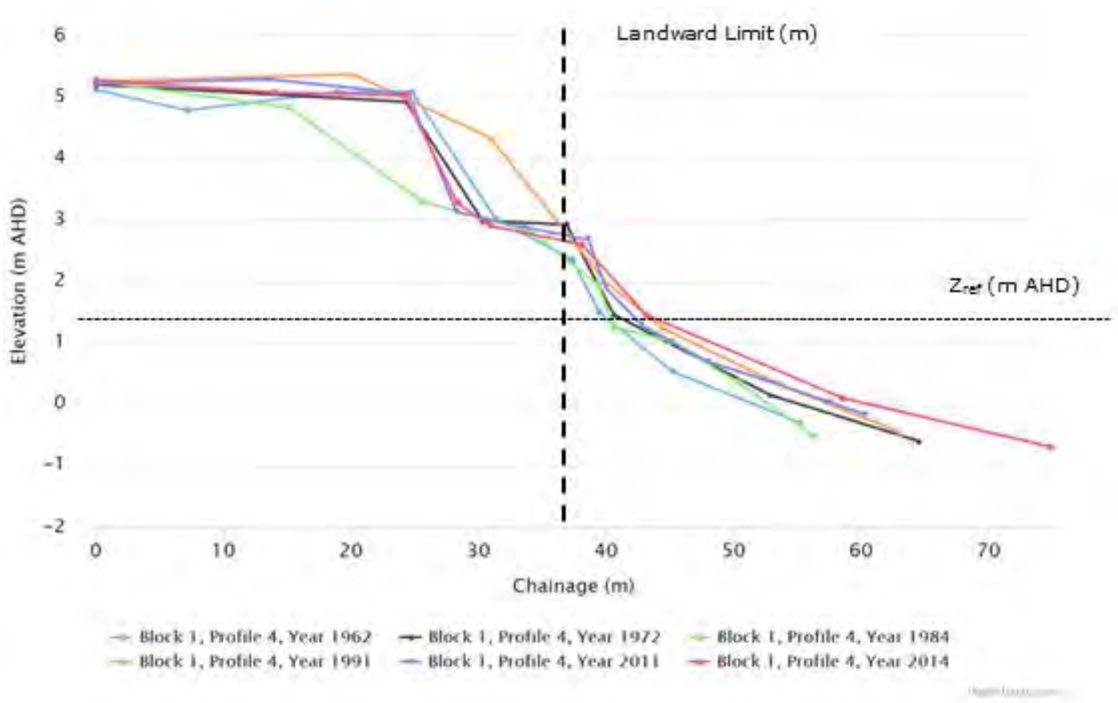


Figure C-19: Example photogrammetry cross sections at Sunshine Bay (Block 1, Profile 4)

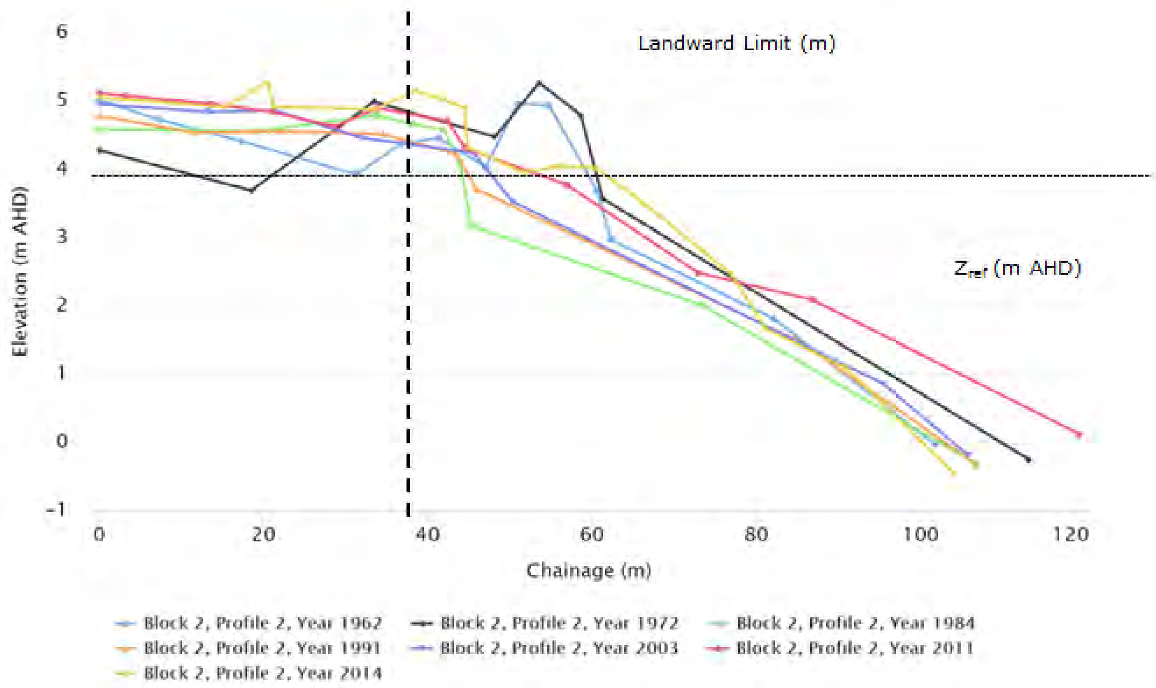


Figure C-20: Example photogrammetry cross sections at Malua Bay (Block 2, Profile 2)

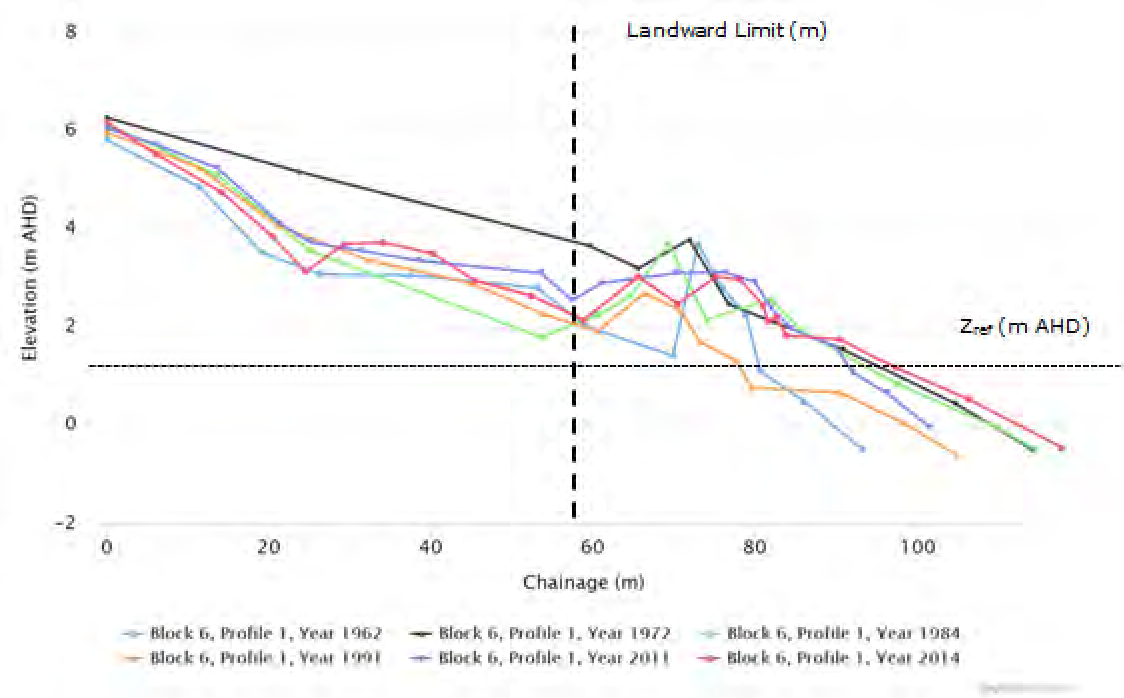


Figure C-21: Example photogrammetry cross sections at Guerilla Bay (Block 6, Profile 1)

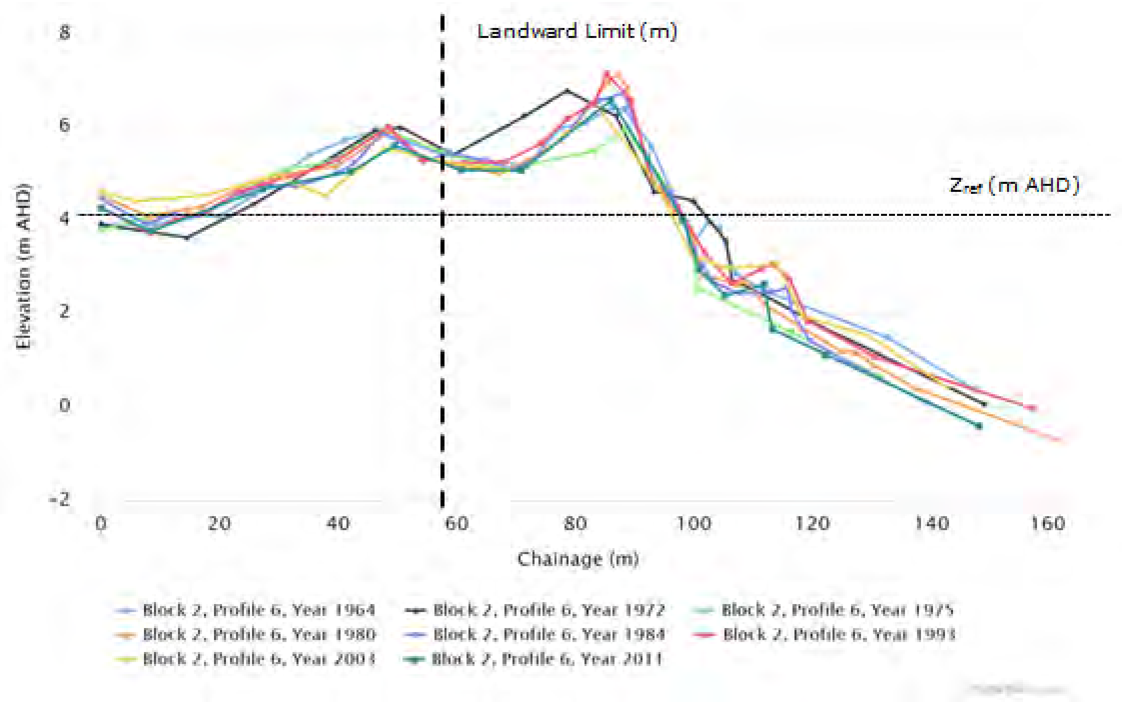


Figure C-22: Example photogrammetry cross sections at Barlings Beach East (Block 2, Profile 6)

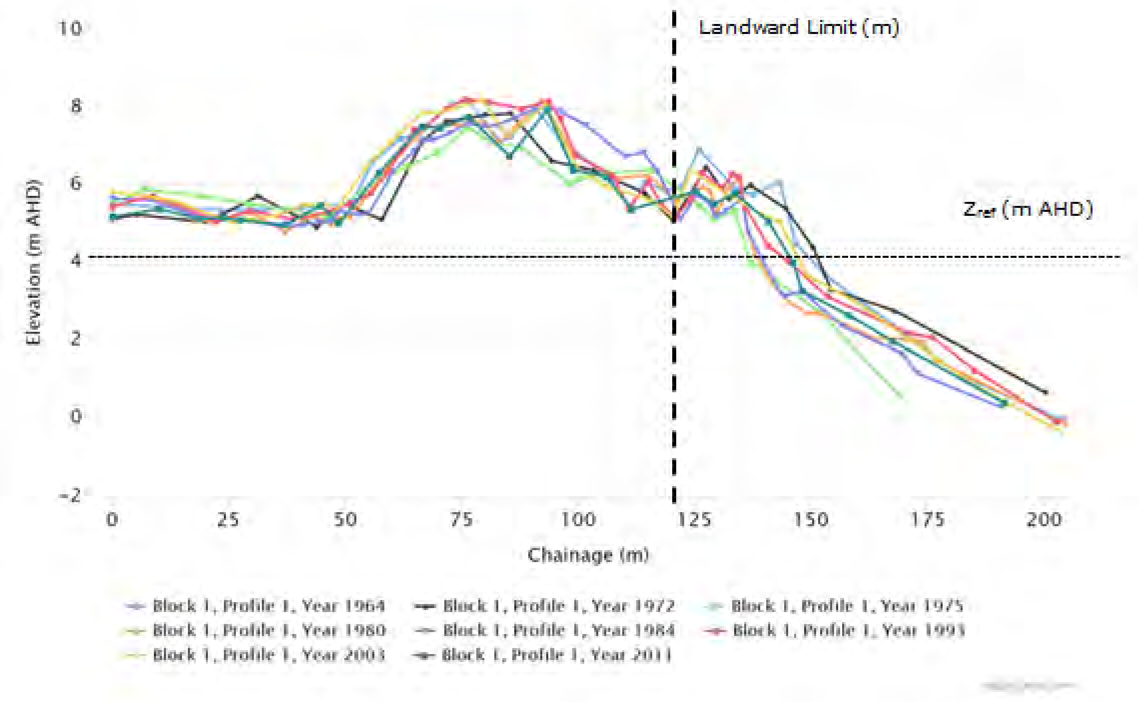


Figure C-23: Example photogrammetry cross sections at Barlings Beach West (Block 1, Profile 1)

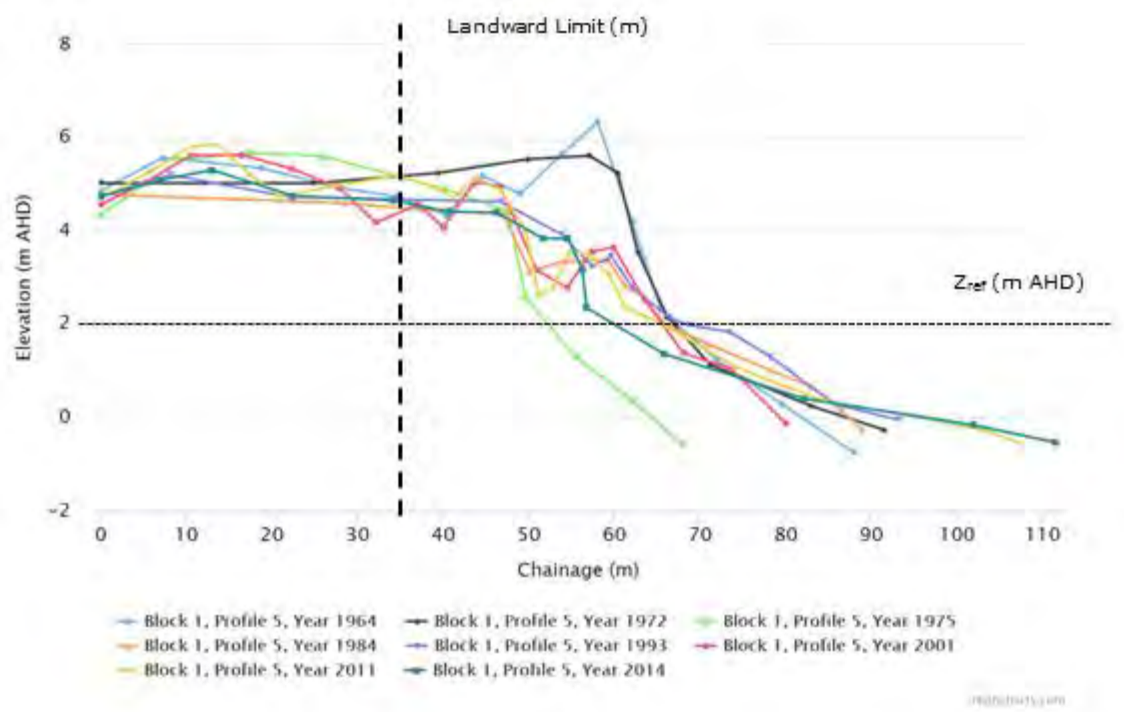


Figure C-24: Example photogrammetry cross sections at Tomakin Cove (Block 1, Profile 5)

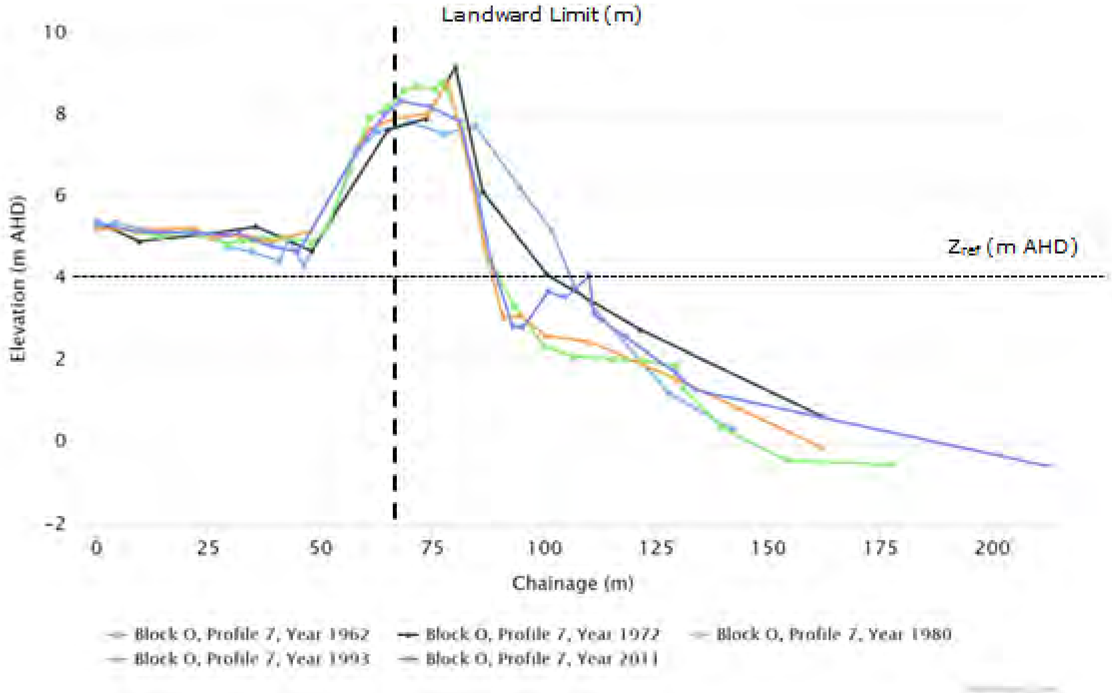


Figure C-25: Example photogrammetry cross sections at Broulee Beach North (Block O, Profile 7)

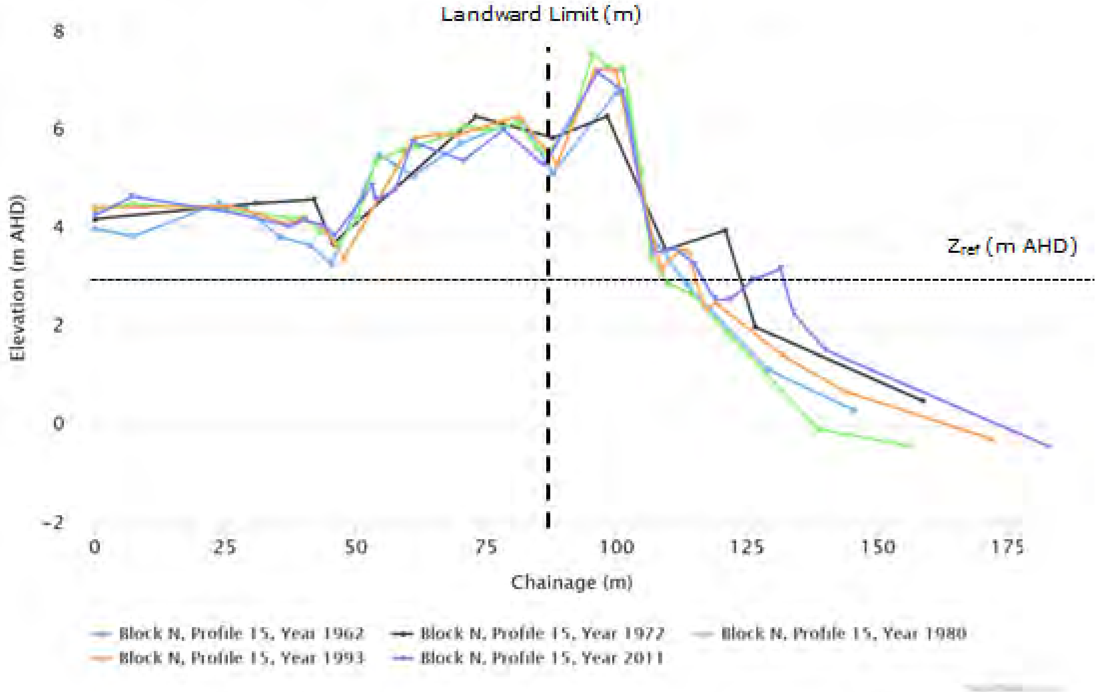


Figure C-26: Example photogrammetry cross sections at Broulee Beach Central (Block N, Profile 15)

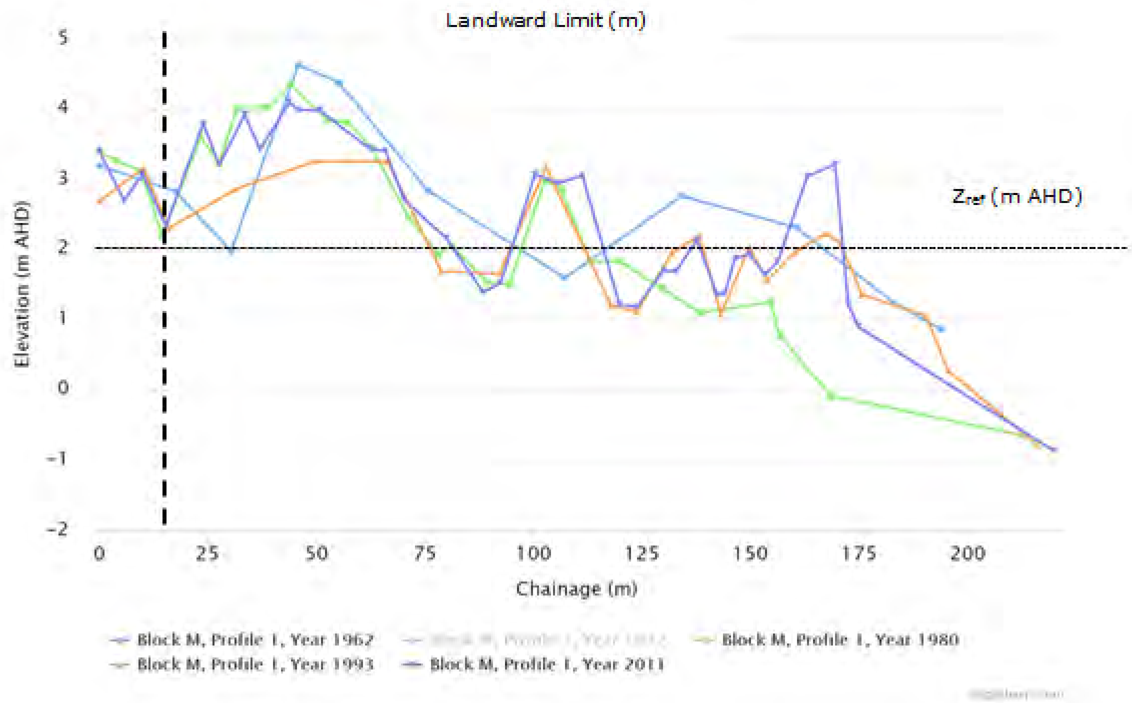


Figure C-27: Example photogrammetry cross sections at Broulee Beach South (Block M, Profile 1)

C.3 Analysis of Photogrammetry

The photogrammetry records have been used to quantify both long term historical changes (underlying recession) and short term storm erosion at each of the beaches. This section summarises the methodology used to undertake these analyses, and the corresponding results.

C.3.1 Underlying Recession

Underlying recession defines the natural recessionary (or accretionary) trends of a beach, and is typically measured in horizontal lineal metres per year. Unless otherwise stated, recession is indicated as a negative value and accretion as a positive value. As beaches are dynamic environments, the long term recessionary trends can be difficult to separate from short term fluctuations in the beach state. In order to deal with this uncertainty, two methods have been used to measure the underlying recession:

1. Movement of a representative contour (Z_{ref}) (m/year); and
2. Rate of volumetric change (m^3/m/year), converted to a linear trend (m/year) by dividing by the average dune height at each profile.

Where a dune exists, the representative contour has been chosen on the lower dune face, such that is not typically influenced by daily wave action, but will move on a long term scale. Volumetric analysis is defined as the sub aerial beach volume seaward of a defined landward limit (landward of which profile changes are not associated with coastal processes). Where necessary, profiles have been extrapolated seaward to 0 m AHD using a 1V:10H slope. Table C-3 summarises the representative contour and the landward limits used at each profile.

Examples of analysis for underlying shoreline movement using the two methods are presented in Figure C-28 (Z_{ref} trend) and Figure C-29 (volumetric trend) for Surfside Beach (east) (Block B, Profile 2).

The results of the analysis on underlying movement (accretion – positive, recession – negative) are provided in Figure C-30 to Figure C-38. Note that some profiles have been excluded from the analysis due to the presence of rocky cliffs that are considered non erodible or creek mouth where sediment transport is not primarily driven by wave action. No results have been provided for Surfside Beach (west) as the photogrammetry showed no discernible trend. This is likely to be due to the tide-dominated nature of Surfside Beach (west).

Table C-3: Summary of Landward Limits and Representative Contours

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Maloneys Beach	E	1	85	1.5
	E	2	80	1.9
	E	3	85	1.5
	E	4	83	1.5
	E	5	77	1.8
	E	6	75	2
	E	7	65	1.9
	E	8	63	2
	E	9	61	1.9
	E	10	50	1.5
	E	11	50	2
	E	12	45	2
	E	13	44	1.9
	E	14	41	2
	E	15	37	2
	E	16	35	2
	E	17	32	2
	E	18	35	1.9
	E	19	33	2
	E	20	26	2
	E	21	23	2
	E	22	30	1.8
	E	23	32	1.8
	E	24	32.5	1.8
	E	25	32.5	2
	E	26	35	2
	E	27	39	1.5
	E	28	41	1.9
	E	29	41	2
	E	30	52	1.6
	E	31	55	1.5
	E	32	60	1.5
	E	33	65	1.9
	E	34	75	1.5
	E	35	83	1.2
	E	36	96	1.5
	E	37	110	1.2
	E	38	105	1.2

Table C-3: Summary of Landward Limits and Representative Contours (cont…)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Long Beach	C	1	89	1.25
	C	2	85	1.9
	C	3	84	1.8
	C	4	80	2
	C	5	70	1.8
	C	6	66	2
	C	7	72	2
	C	8	63	2
	C	9	66	2
	C	10	60	1.8
	C	11	59	1.9
	C	12	51	1.8
	C	13	54	1.5
	C	14	48	1.8
	C	15	52	1.7
	C	16	45	1.6
	C	17	48	1.7
	C	18	43	2
	C	19	36	1.5
	C	20	36	1.5
	C	21	40	1.8
	C	22	41	1.7
	C	23	41	2
	C	24	42	1.8
	C	25	32	1.8
	C	26	27	1.5
	C	27	24	1.8
	C	28	38	1.9
	C	29	35	1.7
	C	30	33	1.8
	C	31	15	1.9
	C	32	23	2
	C	33	30	1.7
	C	34	30	1.7
	C	35	29	2
	C	36	29	1.8
	C	37	34	1.8
	C	38	34	1.75
	C	39	35	1.7
	C	40	30	1.9
	C	41	28	2
	C	42	15	1.7
	C	43	15	2
	C	44	17	2
	C	45	27	1.8
	C	46	14	2
	C	47	9	1.6
	C	48	30	1.5
	C	49	36	1.9
	C	50	35	2
	C	51	31	1.5
	C	52	31	1.7
	C	53	18	1.5

Table C-3: Summary of Landward Limits and Representative Contours (cont…)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Long Beach (cont…)	C	54	18	1.6
	C	55	17	1.7
	C	56	21	1.6
	C	57	22	2
	C	58	33	1.8
	C	59	35	1
	C	60	30	1.5
	C	61	48	2
	C	62	46	1.7
	C	63	37	1.6
	C	64	42	1.7
	C	65	46	1.9
	C	66	60	2
	C	67	52	2
	C	68	54	1.9
	C	69	65	1.6
	C	70	70	1.9
	C	71	86	1.5
	C	72	80	1.8
	D	1	55	1.5
	D	2	48	1.8
	D	3	50	2
	D	4	45	1.9
	D	5	42	1.5
	D	6	48	1.8
	D	7	46	1.5
	D	8	36	1.5
	D	9	34	1.5
	D	10	37	1.5
	D	11	36	1.5
	D	12	37	1.4
	D	13	39	1.5
	D	14	40	1.4
	D	15	50	1.4
	D	16	47	1.5
	D	17	45	1.5
	D	18	46	1.5
	D	19	46	1.5
	D	20	47	0.7
	D	21	43	0.5
	D	22	41	0.6
	D	23	40	0.8
	D	24	40	1.2
	D	25	40	1.3
	D	26	40	1.3
	D	27	40	1
	D	28	41	1.1
	D	29	50	1.2
	D	30	64	1
	D	31	70	1
	D	32	90	1

Table C-3: Summary of Landward Limits and Representative Contours (cont...)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Surfside Beach (east)	A	1	67	1.4
	A	2	67	1
	A	3	66	1
	A	4	70	1.3
	A	5	65	1.5
	A	6	62	1.5
	A	7	52	1.5
	A	8	58	1.5
	A	9	56	1.6
	A	10	54	1.5
	A	11	45	1.5
	A	12	44	1.6
	A	13	49	1.5
	A	14	42	1.6
	A	15	44	1.5
	A	16	42	1.7
	A	17	41	2.1
	A	18	40	1.6
	A	19	40	1.8
	A	20	42	1.9
	A	21	40	1.75
	A	22	47	1.6
	A	23	44	1.8
	A	24	43	1.5
	A	25	53	1.6
	A	26	55	1.5
	A	27	58	1.6
	A	28	60	1.5
	A	29	55	1.5
	A	30	60	1.4
	A	31	64	1.5
	A	32	67	1.5
	B	1	73	1.3
	B	2	52	1.3
	B	3	47	1.5
	B	4	45	1.1
	B	5	30	1.7
	B	6	28	1.6
	B	7	34	1.55
	B	8	23	0.7
	B	9	22	1
	B	10	19	0.8
	B	11	26	0.7
	B	12	35	1.1
Surfside Beach (west)	1	1	74	1.5
	1	2	84	1.5
	1	3	40	0
	1	4	30	0.5
	1	5	18	0.5
	1	6	20	0.5
	1	7	50	0.6

Table C-3: Summary of Landward Limits and Representative Contours (cont…)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Sunshine Bay	1	1	51	1
	1	2	22	1.5
	1	3	29	1
	1	4	37	1.5
	2	1	58	1.5
	2	2	32	1.5
	2	3	27	1.5
	2	4	30	1
	2	5	28	1.5
	3	1	37	1.4
	3	2	29	1.5
	3	3	39	0.8
Malua Bay	1	1	75	1
	1	2	25	1.3
	1	3	53	2.5
	1	4	55	3
	1	5	56	3.3
	1	6	61	3.5
	2	1	35	3.5
	2	2	37	4
	2	3	28	4
	2	4	36	4.6
	2	5	49	3.5
	2	6	20	1
Guerilla Bay	5	1	51.75	1.3
	5	2	51.25	1.1
	5	3	44	1.7
	6	1	58	1.3
	6	2	27	1.3
	6	3	42	1.3
	6	4	35	1.5
	6	5	66	1.7
	6	6	148.5	1.1

Table C-3: Summary of Landward Limits and Representative Contours (cont...)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Barlings Beach	1	1	120	4
	1	2	108	4
	1	3	100	4
	1	4	95	4
	1	5	95	4
	1	6	95	4.5
	2	1	101	4.5
	2	2	65	4.5
	2	3	61	4
	2	4	55	4
	2	5	60	4
	2	6	59	4
	2	7	60	4
	2	8	70	4
	2	9	72	3
	2	10	68	4
	3	1	76	3.5
	3	2	41	4
	3	3	53	4
	3	4	72	3.5
Tomakin Cove	1	1	10	1.2
	1	2	26	1.9
	1	3	32	2
	1	4	35	1.9
	1	5	36	2
	1	6	39	2
	2	1	46	2
	2	2	39	2
	2	3	40	1.7
	2	4	38	2
	2	5	42	1.8
	2	6	40	1.6
	3	1	55	1.7
	3	2	52	1.9
	3	3	42	2
	3	4	42	1.6
	3	5	74	1.8

Table C-3: Summary of Landward Limits and Representative Contours (cont…)

Beach	Block	Profile	Landward Limit	Zref (m AHD)
Broulee Beach	M	1	14	2
	M	2	72	2
	M	3	63	2.5
	M	4	101	2.5
	M	5	106	2.5
	M	6	44	2.5
	M	7	74	2.5
	M	8	111	2.8
	M	9	115	2.5
	M	10	122	2.5
	N	1	94	2.5
	N	2	87	3
	N	3	70	3
	N	4	76	3
	N	5	75	3
	N	6	83	3
	N	7	60	3
	N	8	81	3
	N	9	73	3
	N	10	70	3
	N	11	60	2.8
	N	12	80	2.8
	N	13	81	3
	N	14	88	3
	N	15	86	3
	N	16	95	3
	N	17	108	3
	N	18	93	3
	N	19	126	3
	N	20	149	3
	O	1	65	4
	O	2	63	4
	O	3	61	4.5
	O	4	47	4.5
	O	5	59	5.2
	O	6	72	4.5
	O	7	65	4
	O	8	58	2.5

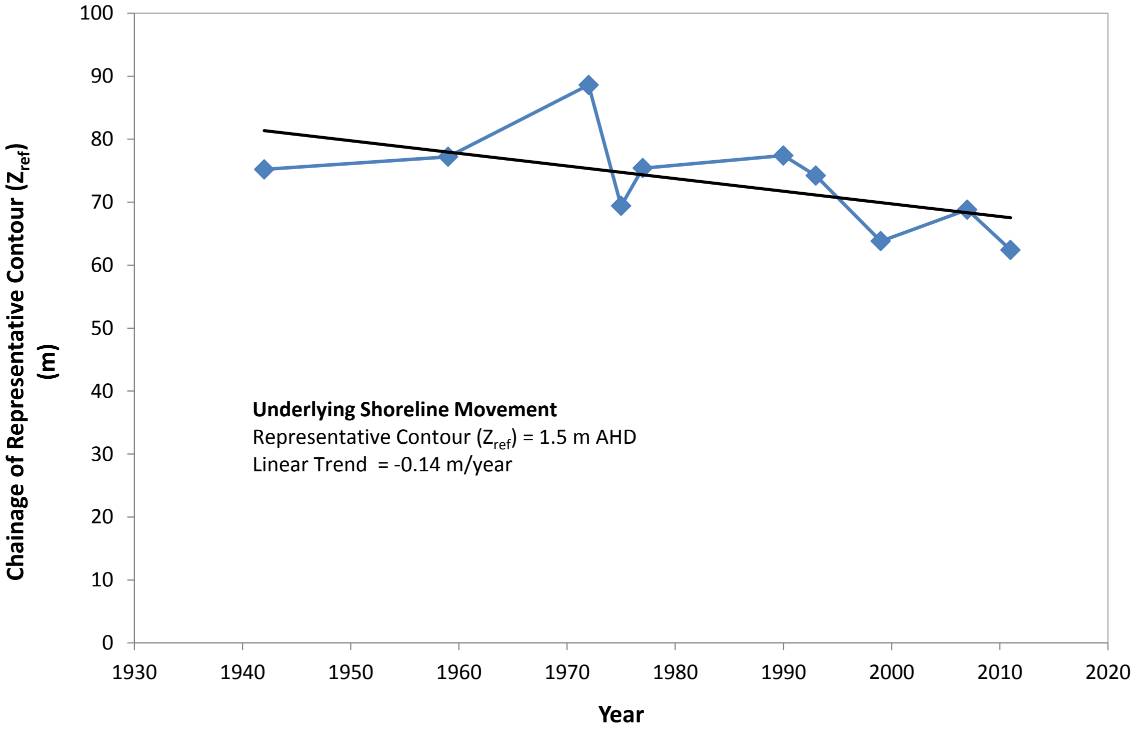


Figure C-28: Example Underlying Shoreline Movement Analysis from Photogrammetry Data - Movement of Representative Contour, Surfside Beach (east), Block B, Profile 3

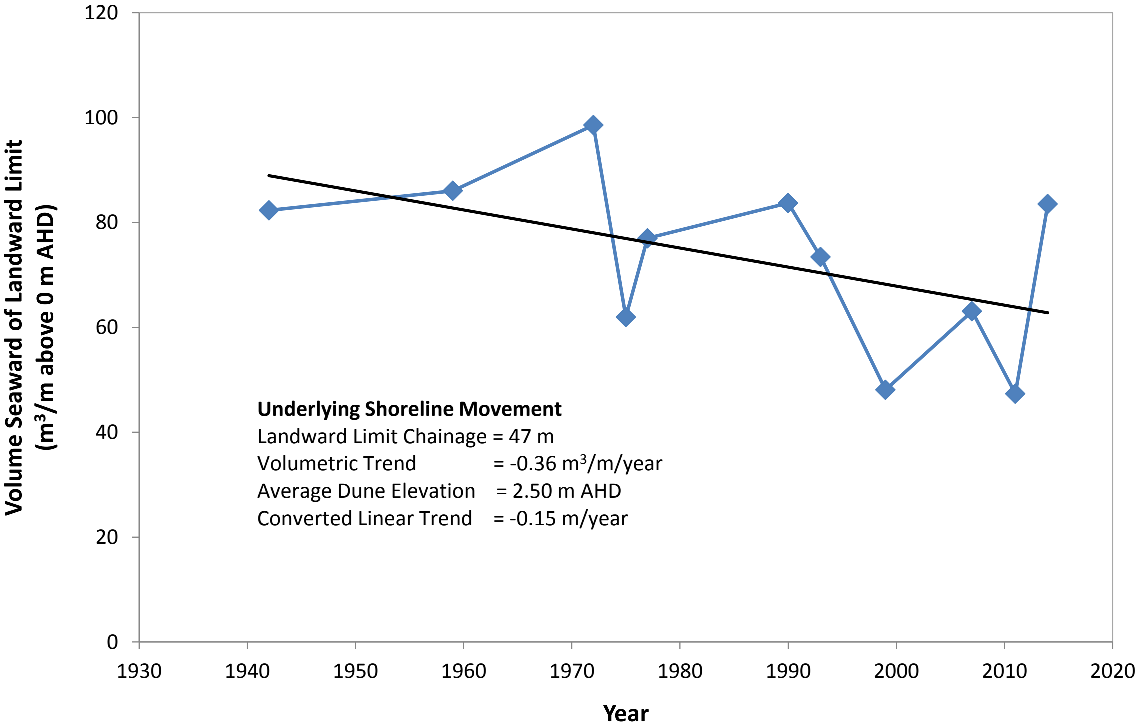


Figure C-29: Example Underlying Shoreline Movement Analysis from Photogrammetry Data - Rate of Volumetric Change, Surfside Beach (east), Block B, Profile 3

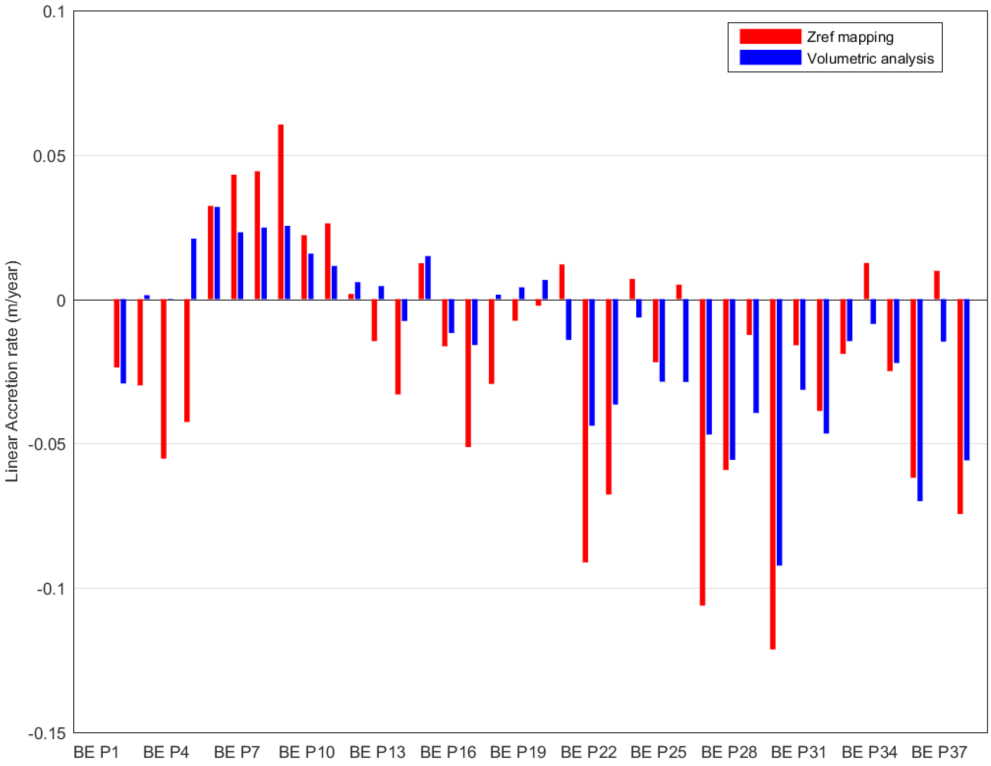


Figure C-30: Maloneys Beach Underlying Movement

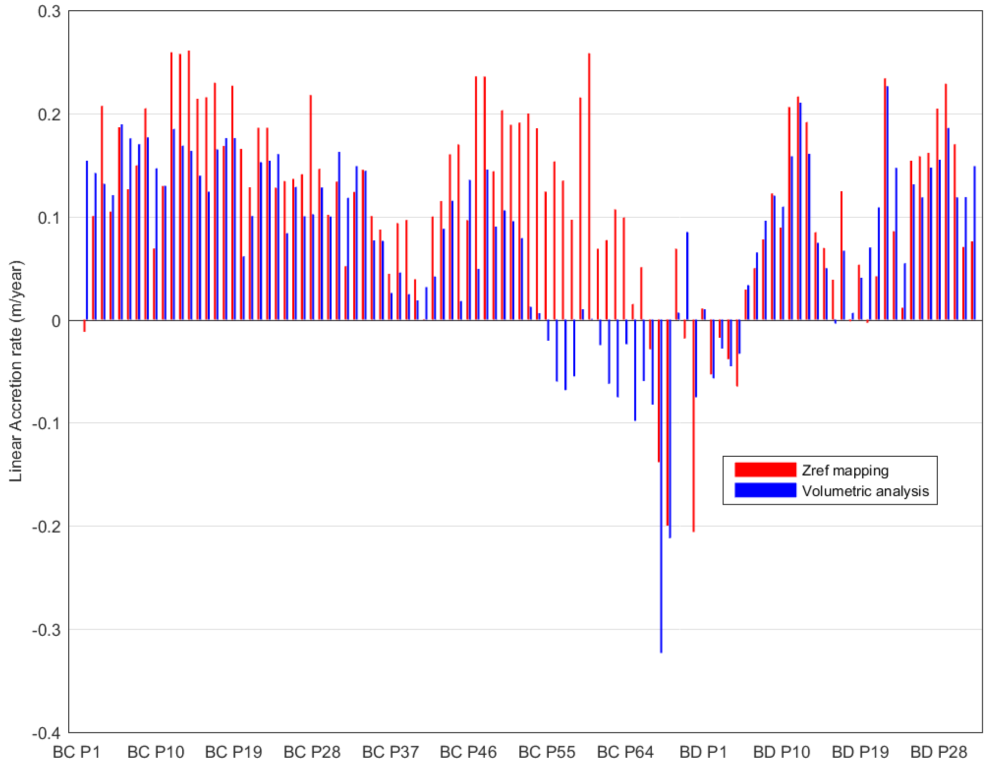


Figure C-31: Long Beach Underlying Movement

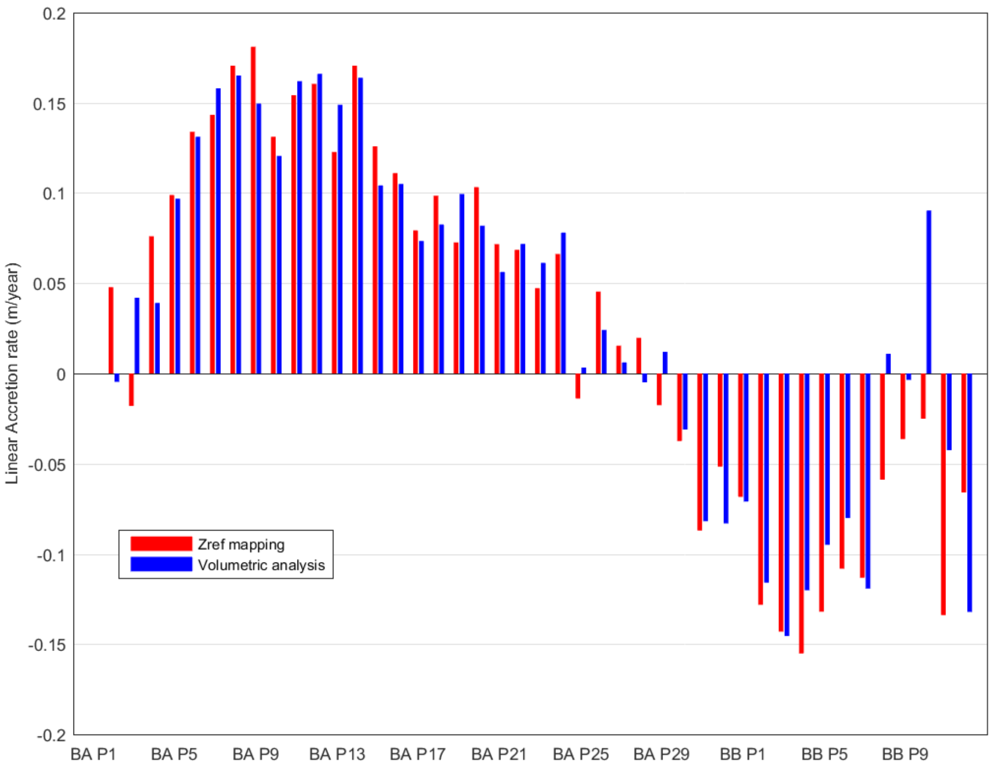


Figure C-32: Surfside Beach (east) Underlying Movement

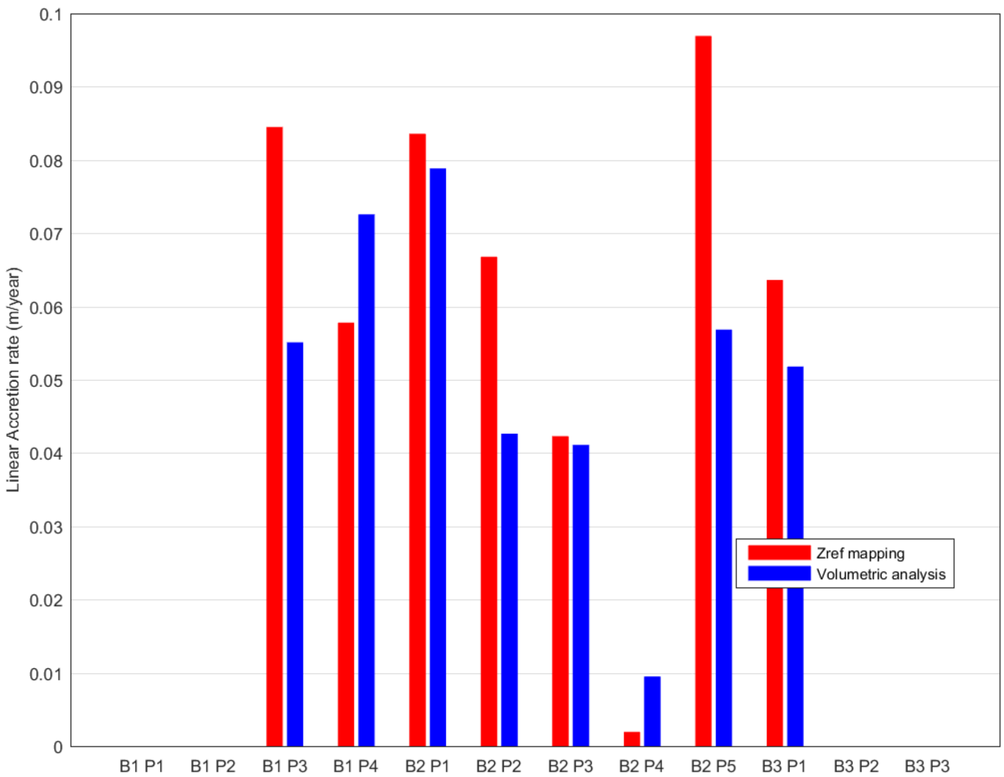


Figure C-33: Sunshine Bay Underlying Movement

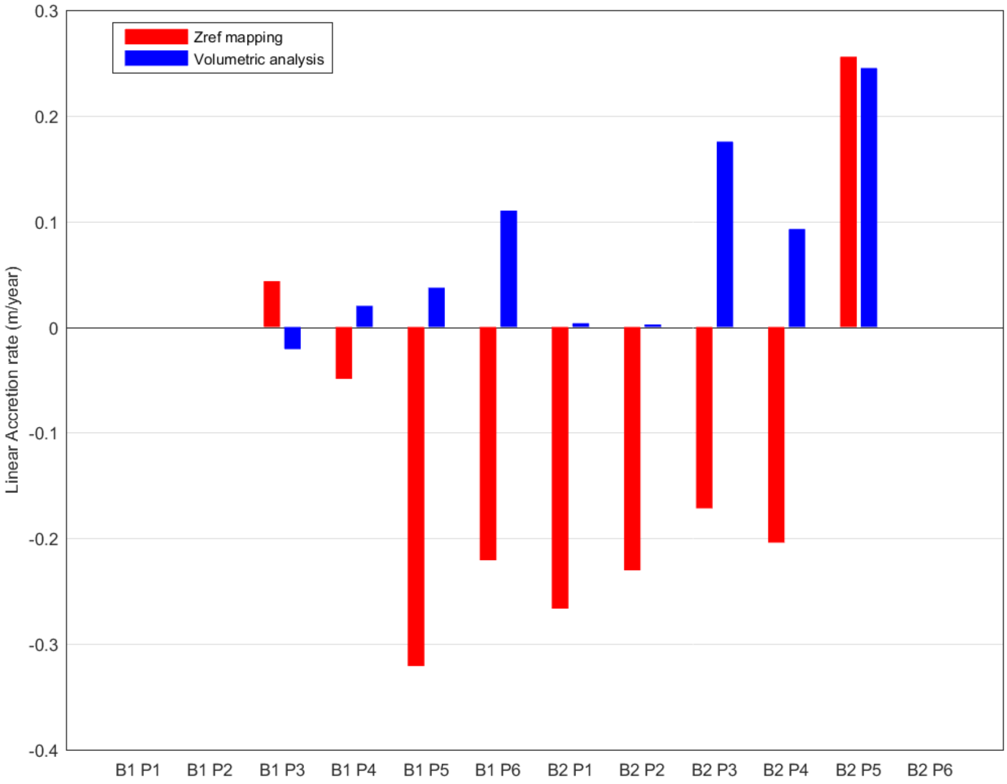


Figure C-34: Malua Bay Underlying Movement

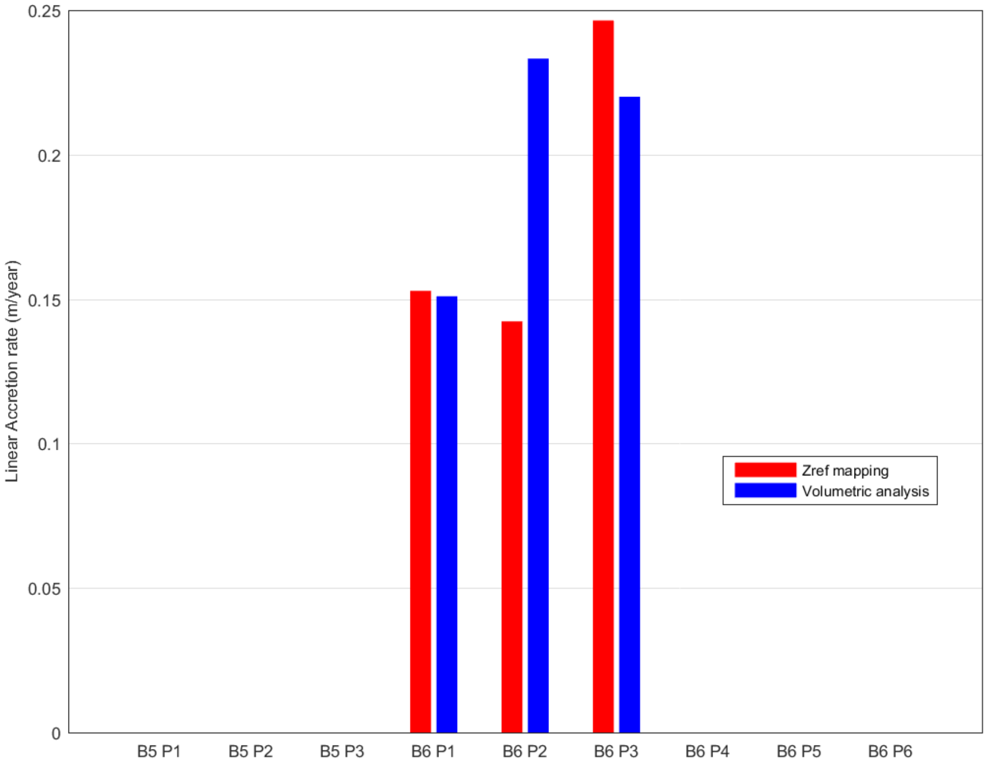


Figure C-35: Guerilla Bay Underlying Movement

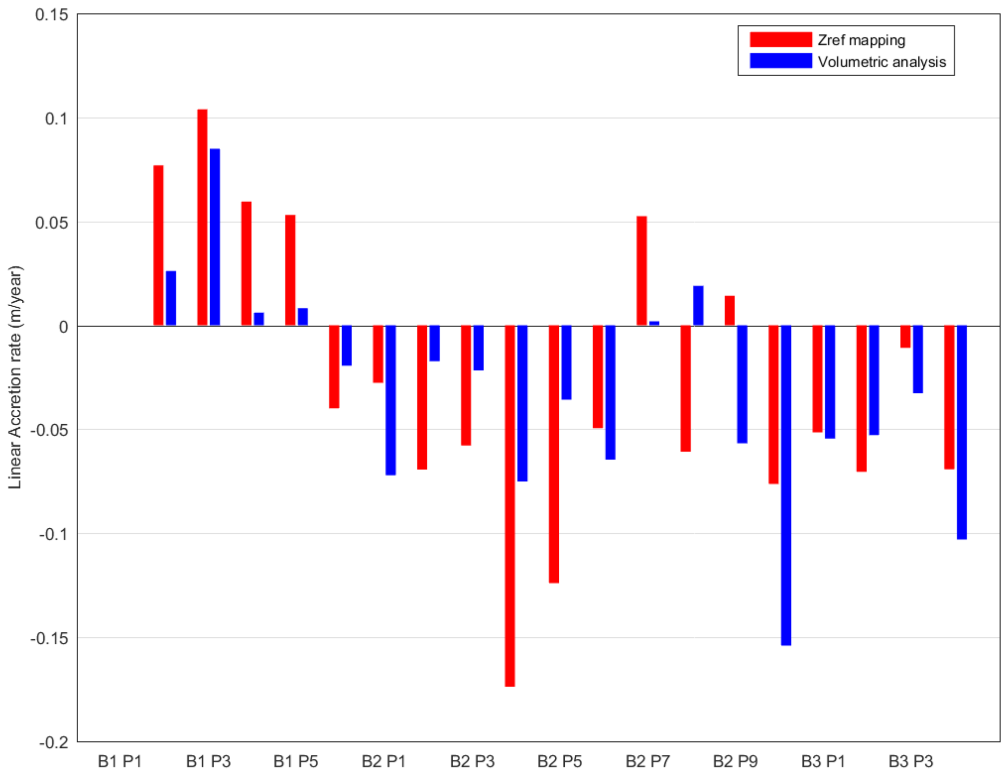


Figure C-36: Barlings Beach Underlying Movement

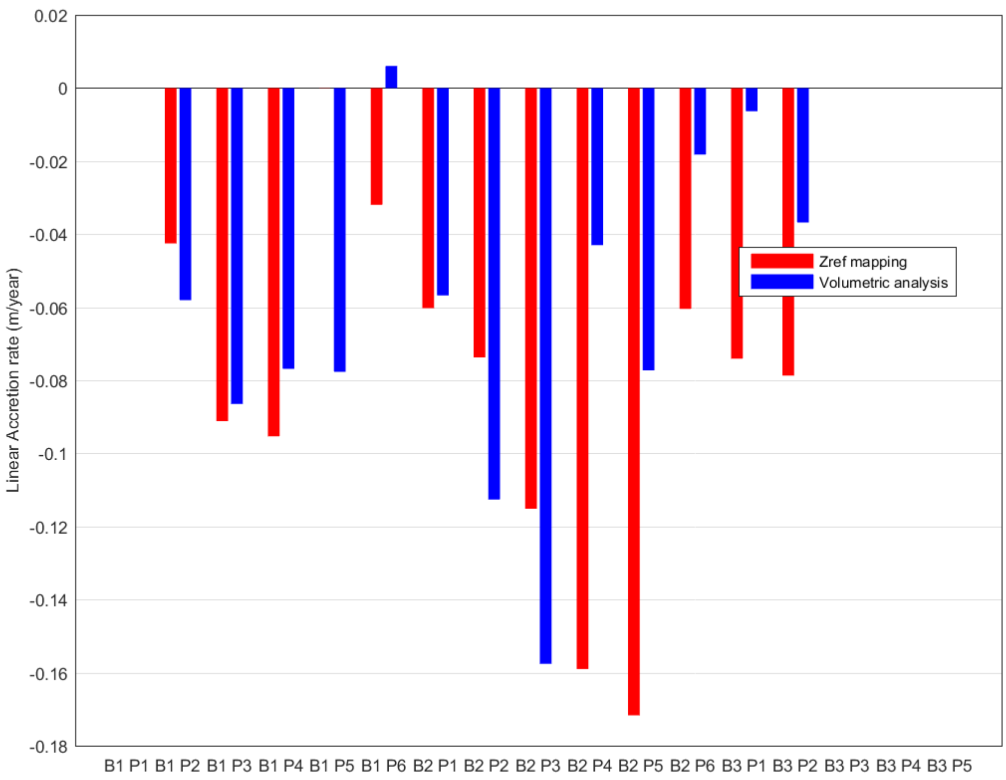


Figure C-37: Tomakin Cove Underlying Movement

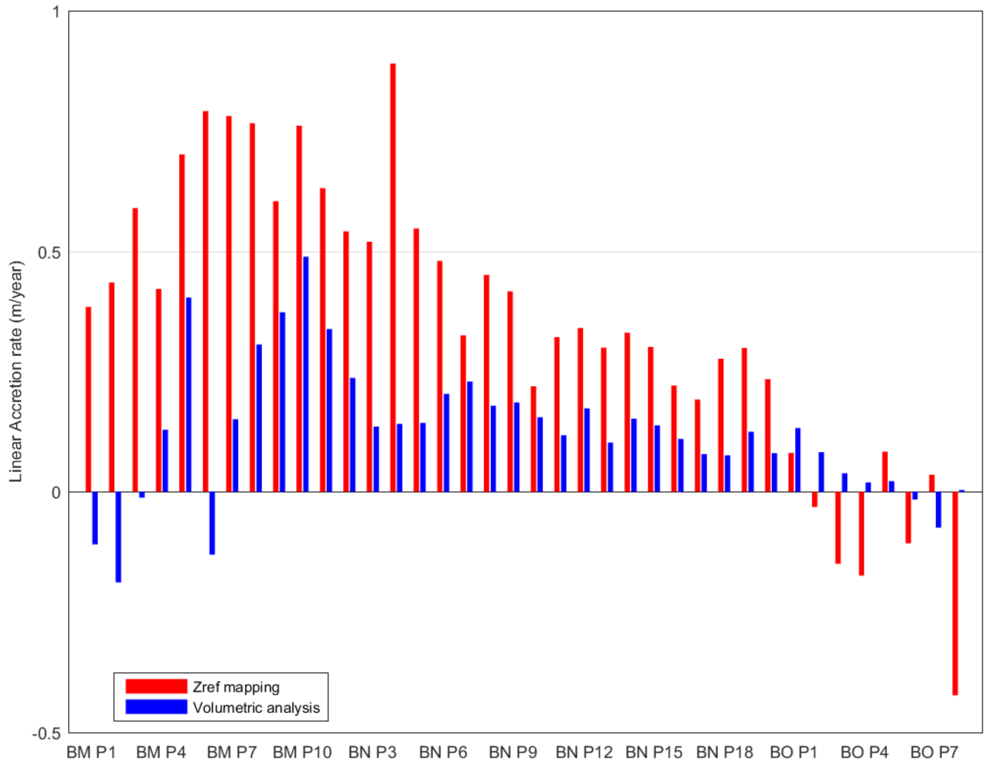


Figure C-38: Broulee Beach Underlying Movement

C.3.2 Maximum Historical Storm Erosion

The maximum historical storm erosion is defined here as the maximum negative change in sub aerial beach volume between two consecutive photogrammetry years at each profile location. Similar to the volumetric analysis, profiles have been extrapolated to 0 m AHD (as necessary) at a slope of 1V:10H and volume was only calculated seaward of the landward limit described in Table C-3.

The results of the maximum historical storm erosion are provided in Figure C-39 to Figure C-47 and summarised in Table C-5. As per Section C.3.1, no results are provided at profiles backed by a rocky cliff or a creek mouth, nor are any results presented for Surfside Beach (west).

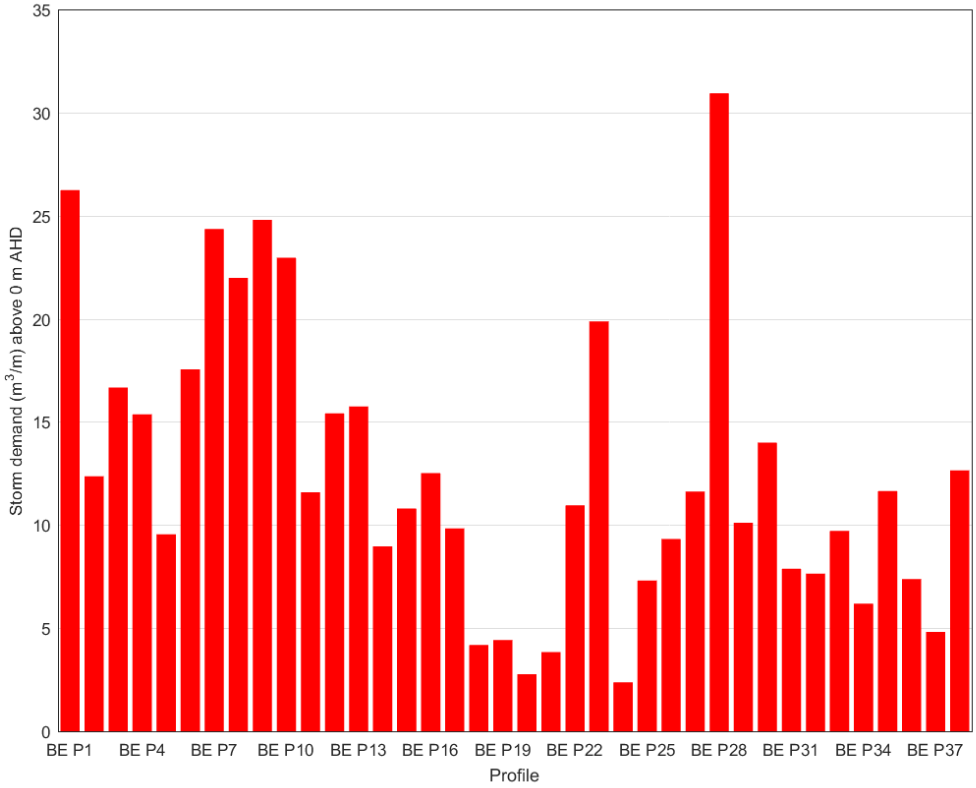


Figure C-39: Maloneys Beach Maximum Storm Demand

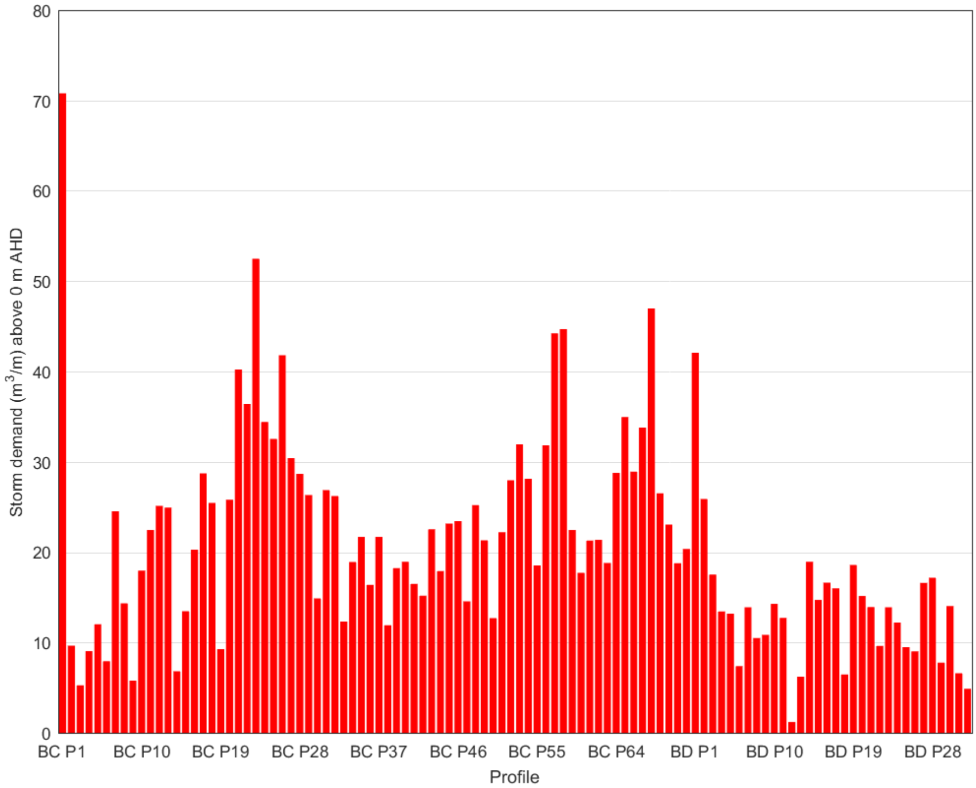


Figure C-40: Long Beach Maximum Storm Demand

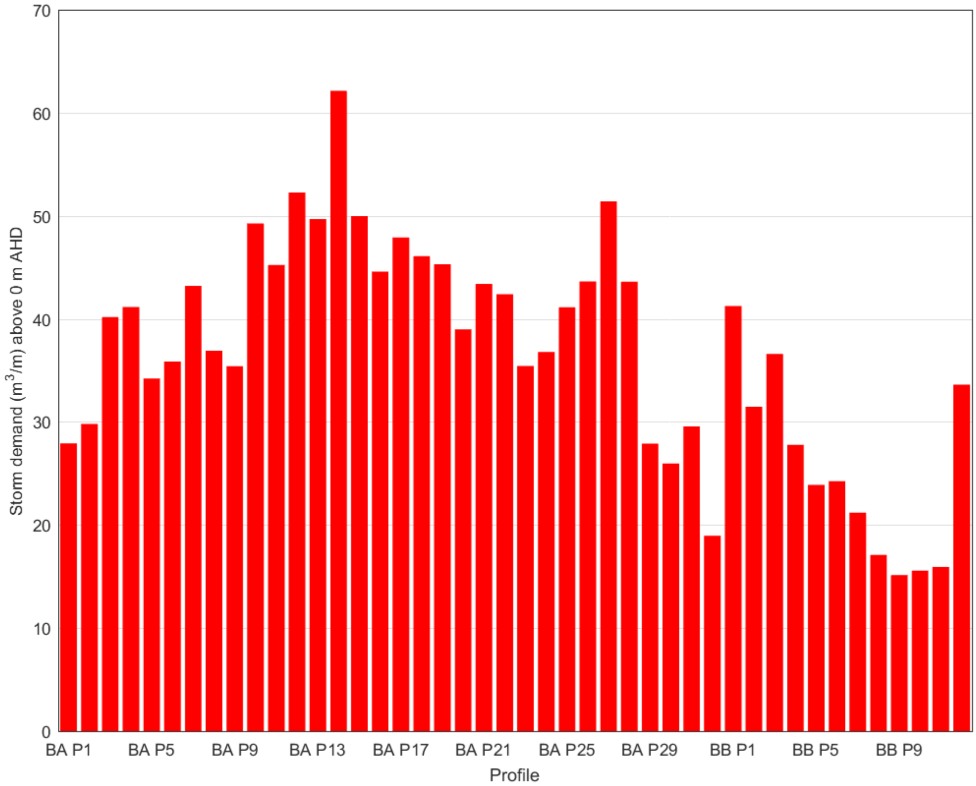


Figure C-41: Surfside Beach (east) Maximum Storm Demand

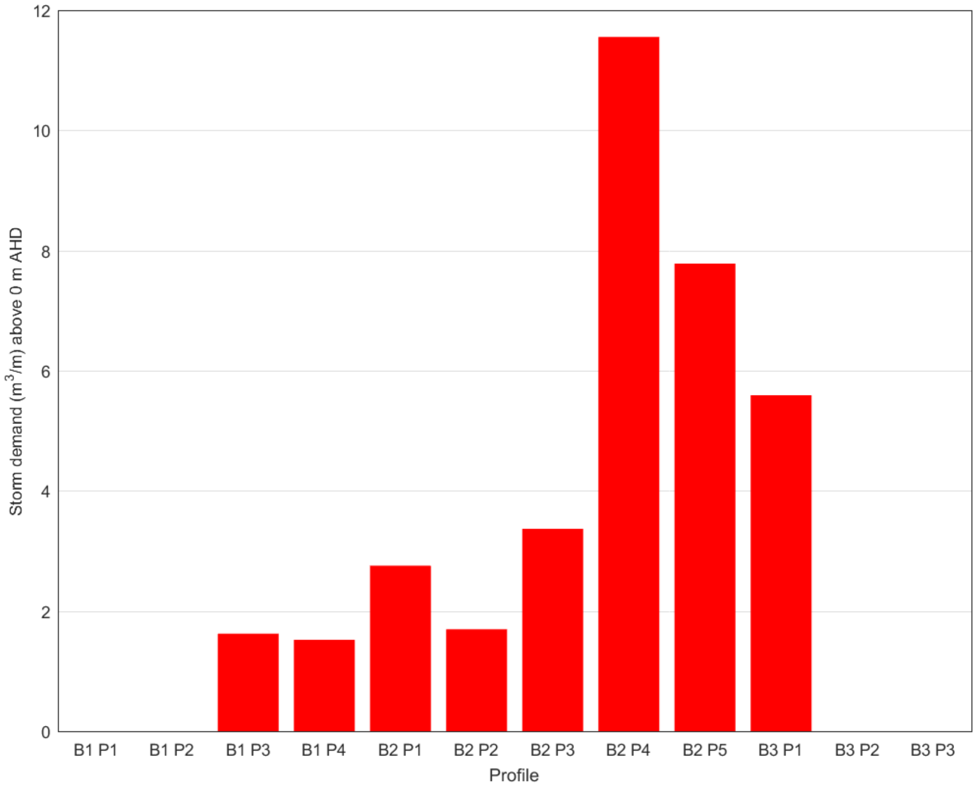


Figure C-42: Sunshine Bay Maximum Storm Demand

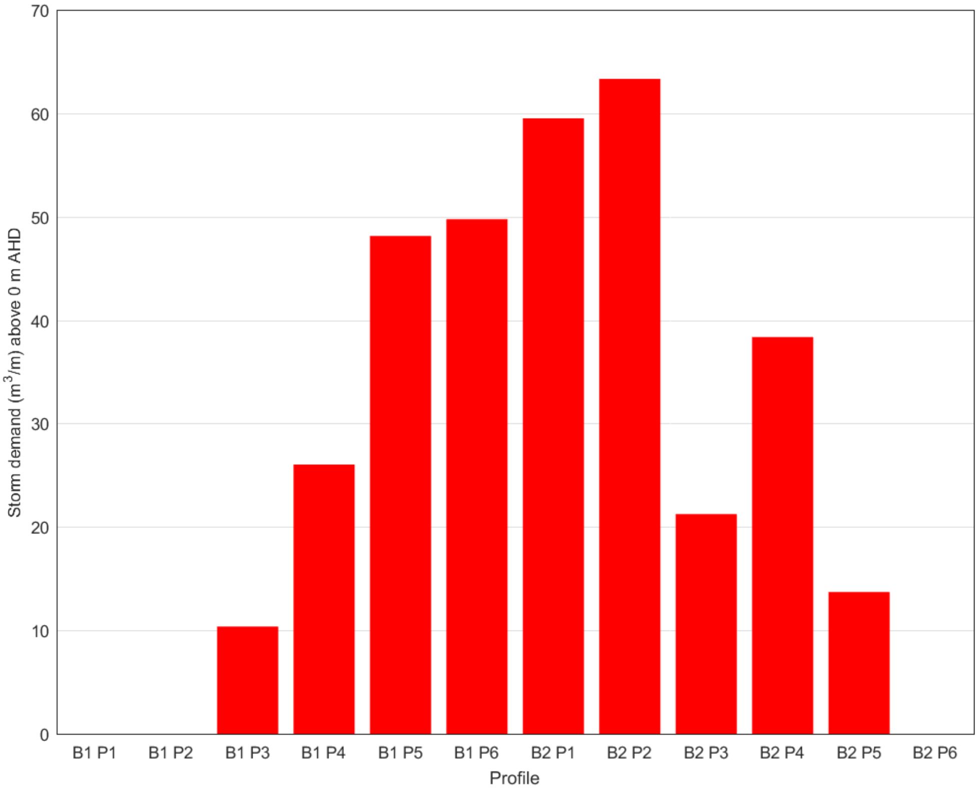


Figure C-43: Malua Bay Maximum Storm Demand

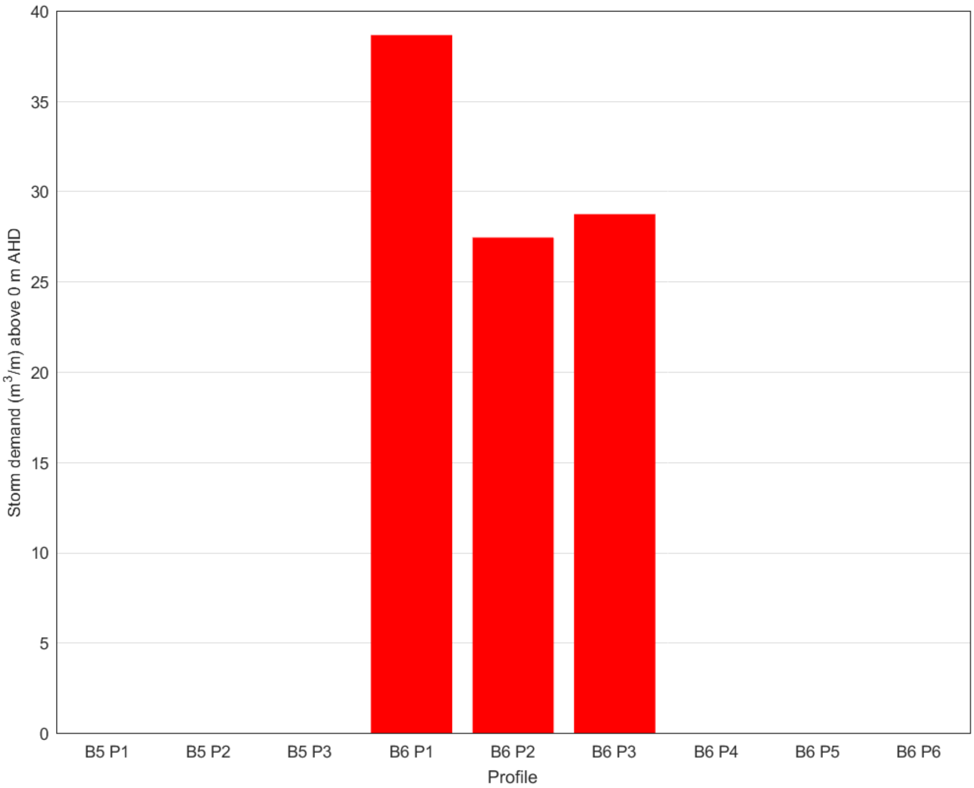


Figure C-44: Guerilla Bay Maximum Storm Demand

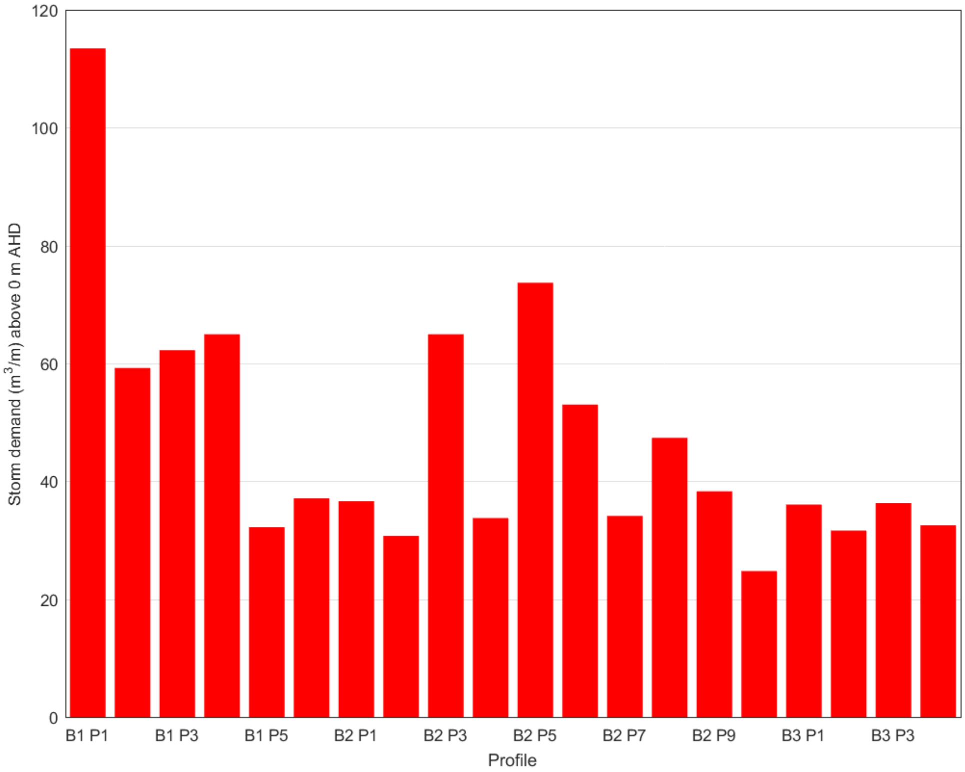


Figure C-45: Barlings Beach Maximum Storm Demand

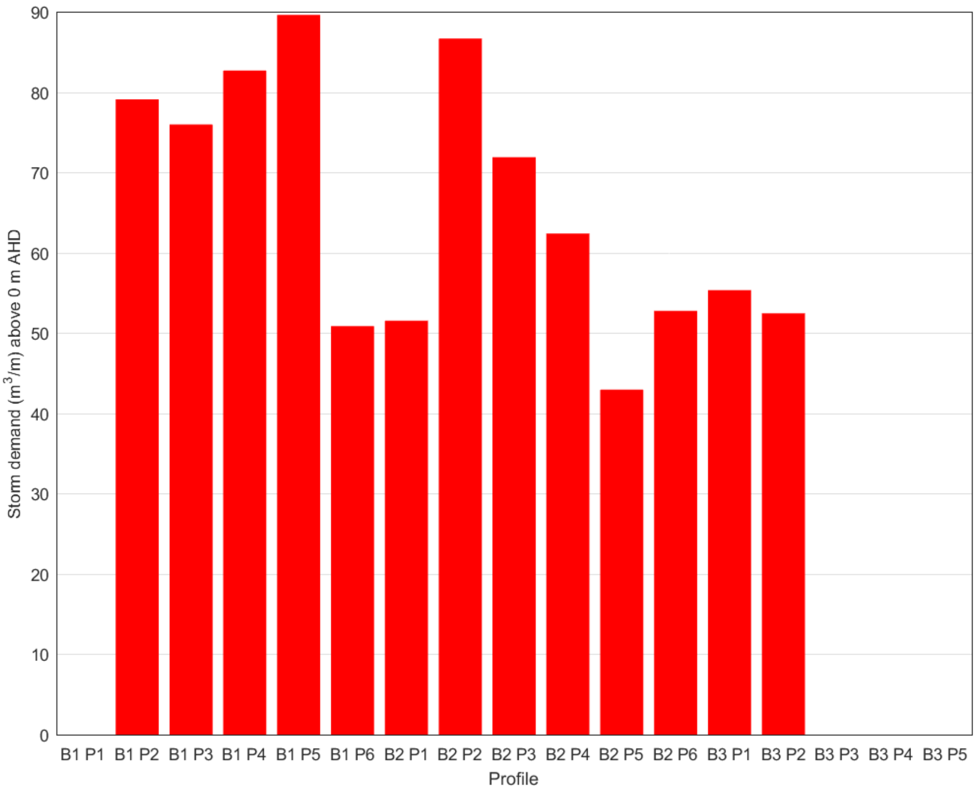


Figure C-46: Tomakin Cove Maximum Storm Demand

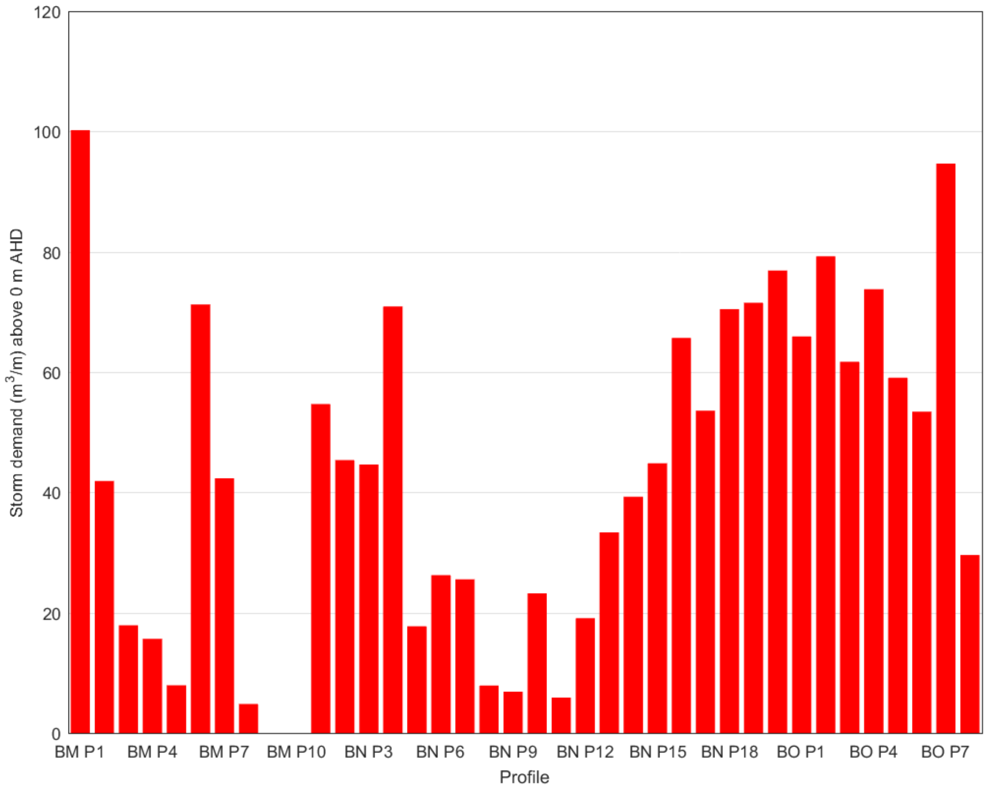


Figure C-47: Broulee Beach Maximum Storm Demand

C.4 Summary

Some of the beaches were treated as separate sections, when the underlying shoreline movement or storm demand varied along the length of the beach (Table C-4). The erosion hazard lines for the two profiles on either side of the intersection points were blended to give a smooth transition between the beach sections.

A summary of maximum storm demand volumes derived from photogrammetry records is provided in Table C-5.

Based on measurements commencing in 1972 at nearby Bengello Beach, the study area's most erosive period in the last 45 years occurred due to a sequence of storm events in May-June 1974. Photogrammetry was recorded at all beaches in 1972, providing an indicative "pre-storm-sequence" condition. For beaches where "post-storm-sequence" photogrammetry was available for 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 this erosive period. Indeed, the consensus values for 100 year ARI storm demand adopted by the expert panel at these beaches are similar to the values calculated from the photogrammetry. However, for beaches where photogrammetry was not available for a significant period of time following May-June 1974 and beach recovery had occurred over a number of years (particularly Maloneys Beach, Long Beach and Sunshine Bay where at least 16 years elapsed), the maximum storm demand estimated from photogrammetry is considered an underestimate of the erosion attributable to this storm sequence and alternative methods were used to estimate storm demand.

Table C-4: Extents of Different Beach Sections

Beach	Section	Section Start		Section End	
		Block	Profile	Block	Profile
Maloneys Beach	West	E	1	E	17
	East	E	17	E	38
Long Beach	West	C	1	C	55
	Central	C	55	D	7
	East	D	7	D	32
Surfside Beach (East)	South	A	0	A	28
	North	A	28	B	12
Barlings Beach	West	1	1	2	4
	East	2	4	3	4
Broulee Beach	South	M	1	N	5
	Central	N	5	N	15
	North	N	15	O	8

Table C-5: Summary of Maximum Storm Demand Volumes Derived from Photogrammetry

Beach	Section	Maximum Storm Demand (m^3/m above 0 m AHD)	Comment
Maloneys Beach	Eastern End	31	Block E, Profile 28: 1942 - 1972
	Western End	26	Block E, Profile 1: 1993 - 1999
Long Beach	Eastern End	19	Block D, Profile 19: 2007 - 2011
	Central	47	Block C, Profile 68: 1972 - 1990
	Western End	71	Block C, Profile 1: 1993 - 1999
Surfside Beach (East)	Northern End	44	Block A, Profile 28: 1972 - 1975
	Southern End	62	Block A, Profile 14: 1972 - 1975
Surfside Beach (West)	Central	-	-
Sunshine Bay	Central	12	Block 1, Profile 4: 1962 - 1972
Malua Bay	Central	63	Block 2, Profile 2: 1972 - 1984
Guerilla Bay (South)	Central	39	Block 6, Profile 1: 1984 - 1991
Barlings Beach	Eastern End	53	Block 2, Profile 6: 1972 - 1975
	Western End	113	Block 1, Profile 1: 1972 - 1975
Tomakin Cove	Central	90	Block 1, Profile 5: 1972 - 1975
Broulee Beach	Northern End	95	Block O, Profile 7: 1972 - 1980
	Central	45	Block N, Profile 15: 1972 - 1980
	Southern End	71-100 (spit influenced)	Block M, Profile 1: 1962 - 1980

Appendix D: SWAN Wave Modelling

D.1 Preamble

The Eurobodalla Shire Council area is subject to extreme waves originating from offshore storms. When swell waves approach the coast, they are modified by the processes of refraction, diffraction, wave-wave interaction and dissipation processes. The model SWAN (Simulating WAves Nearshore) was used to quantify the change in wave conditions from an offshore boundary to nearshore locations of interest. Details of SWAN can be found in Booij et al*.* (1999) and Ris et al. (1999) and is described in brief below.

D.2 SWAN Wave Model

SWAN (version 41.10) is a third-generation wave model that computes random, short-crested, wind-generated waves in coastal regions and inland waters. The SWAN model is based on the wave action balance equation with sources and sinks, and accommodates the processes of wind generation, white capping, bottom friction, quadruplet wave-wave interactions, triad wave-wave interactions and depth induced breaking (Ris et al., 1995). It was developed at Delft University of Technology (2016).

The formulation of the SWAN wave model imposes a number of restrictions which should be acknowledged. While the model may be used on domains of any scale, its use in oceanic scale domains is not recommended for reasons of computation efficiency compared to models such as WAM and WaveWatchIII. Additionally, the spectral formulation of the model limits its ability to accurately model wave diffraction and some surf zone processes such as wave setup (in a two-dimensional simulation).

Despite these limitations, the SWAN model is considered an industry-standard spectral wave generation and propagation model and, with appropriate acknowledgment and allowance for such limitations, provides accurate and robust values.

D.3 Computational Domain

Correct representation of natural bathymetry within the model computational domain is critical to simulating representative wave propagation and transformation processes.

D.3.1 Data Sources

Sources of bathymetric data of the Eurobodalla Shire region used within this study are presented in Table D-1. Topographic data was sourced from the most recent available LIDAR (2005 within Batemans Bay and 2011 elsewhere). Bathymetry is shown for each model domain in Figure D-2 to Figure D-5.

Table D-1: Available Bathymetric Survey Data for the Study Region

Location	Source	Survey Date	Grid Reference System	Datum(1,2,3)
Maloneys Beach	OEH Hydrosurvey	24/7/2014	GDA94/MGA Zone 56	AHD
Long Beach	OEH Hydrosurvey	29/7/2014	GDA94/MGA Zone 56	AHD
Surfside Beach (east and west)	OEH Hydrosurvey	30/7/2014	GDA94/MGA Zone 56	AHD
Malua Bay	OEH Hydrosurvey	30/7/2014	GDA94/MGA Zone 56	AHD
Guerilla Bay	OEH Hydrosurvey	29/7/2014	GDA94/MGA Zone 56	AHD
Barlings Beach	OEH Hydrosurvey	1/4/2015	GDA94/MGA Zone 56	AHD
Tomakin Cove	OEH Hydrosurvey	1/4/2015	GDA94/MGA Zone 56	AHD
Broulee Beach	OEH Hydrosurvey	1/4/2015	GDA94/MGA Zone 56	AHD
Bengello Beach	OEH Hydrosurvey	1/4/2015	GDA94/MGA Zone 56	AHD
Batemans Bay Bar	RMS Hydrosurvey	8/7/2015	GDA94/MGA Zone 56	BBHD
Batemans Bay Marine Parks	NSW Marine Park Authority Survey	2007 - 2012	GDA94/MGA Zone 56	AHD
Batemans Bay	OEH Hydrosurvey	1986-87	AGD66/ISG Zone56/1	BBHD
Batemans Bay	OEH Hydrosurvey	5-8/12/1995	AGD66/ISG Zone56/1	BBHD
Deepwater Bed Elevation	Australian Hydrographic Service (AHS)	varied	GDA94/MGA Zone 56	LAT

Notes:
(1) BBHD = Batemans Bay Hydro Datum = -0.889 m AHD.
(2) LAT = Lowest Astronomical Tide = -0.850 m AHD.
(3) AHD = Australian Height Datum ≈ Mean Sea Level (MSL).

D.3.2 Model Domains

Four (4) model domains were constructed to represent different scales and regions of the ESC region, shown in Figure D-1. These domains include:

1. Coarse Eurobodalla Domain (Figure D-2) – extends 56 km from Brush Island (Bawley Point) in the north to Toragy Point (Moruya Heads) in the south and at least 15 km offshore to ensure that model boundary effects did not influence the wave characteristics reaching the beaches in the study area. This coarse model domain has a spatial resolution of 100 m and was primarily used as a transformation model to simulate wave propagation from an offshore location to the nearshore.
2. Durras Model Domain (Figure D-3) – this is a nested grid with a spatial resolution of 25 m. This model domain is centred around Durras Beach and Cookies Beach, and is used to simulate wave propagation at these locations.
3. Batemans Bay Domain (Figure D-4) – this is a nested grid with a spatial resolution of 25 m, centred around Batemans Bay. It is used to simulate wave propagation for beaches between (and including) Maloneys Beach and Sunshine Bay.
4. Moruya Domain (Figure D-5) - this is a nested grid with a spatial resolution of 25 m, covering the southern extent of the study area. It is used to simulate wave propagation for beaches between (and including) Malua Bay and Bengello Beach.

The parameters used to define the position of each model domain are provided in Table D-2.

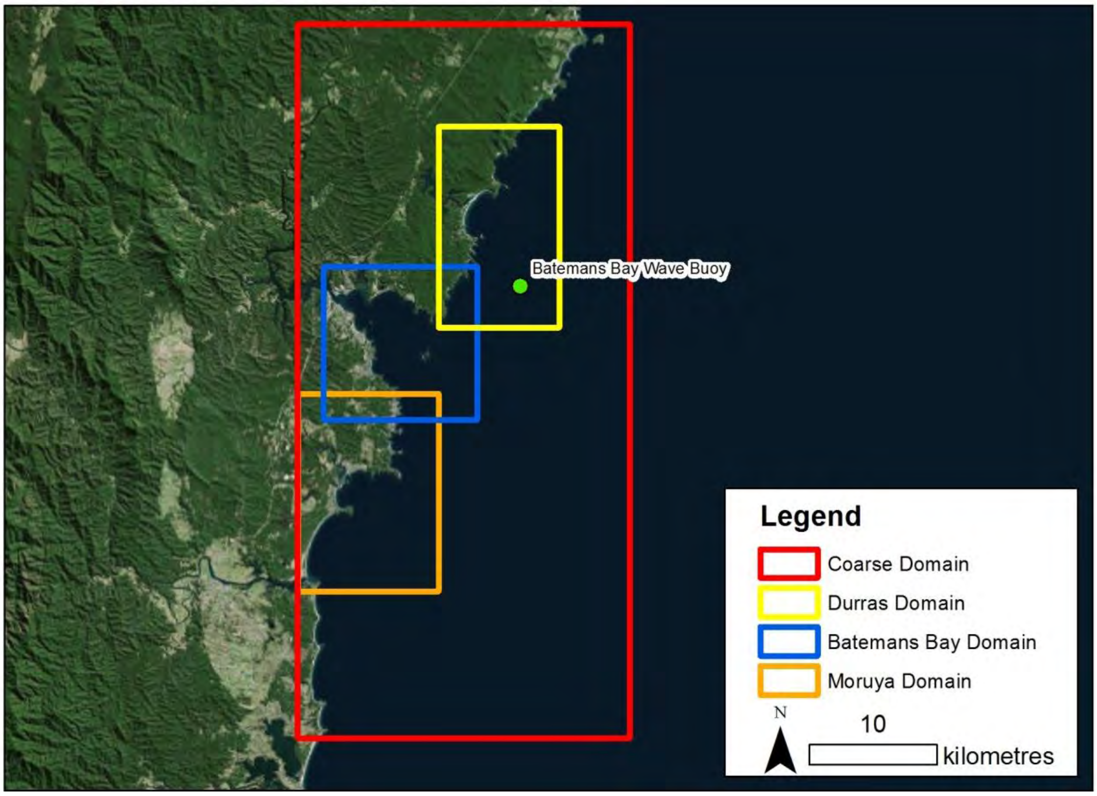


Figure D-1: SWAN model domains

Table D-2: Summary of Model Domain characteristics

Domain	Parameter	East	North
Coarse	Coordinate - Lower Left Corner	242,250	6,010,250
	Size (km)	56	26
	Resolution	100	100
	# Cells	260	560
Durras	Coordinate - Lower Left Corner	253,250	6,042,500
	Size (km)	9.5	15.75
	Resolution	25	25
	# Cells	380	630
Batemans Bay	Coordinate - Lower Left Corner	244,250	6,035,250
	Size (km)	12	12
	Resolution	25	15
	# Cells	480	480
Moruya	Coordinate - Lower Left Corner	242,250	6,021,750
	Size (km)	11	15.5
	Resolution	25	25
	# Cells	440	620

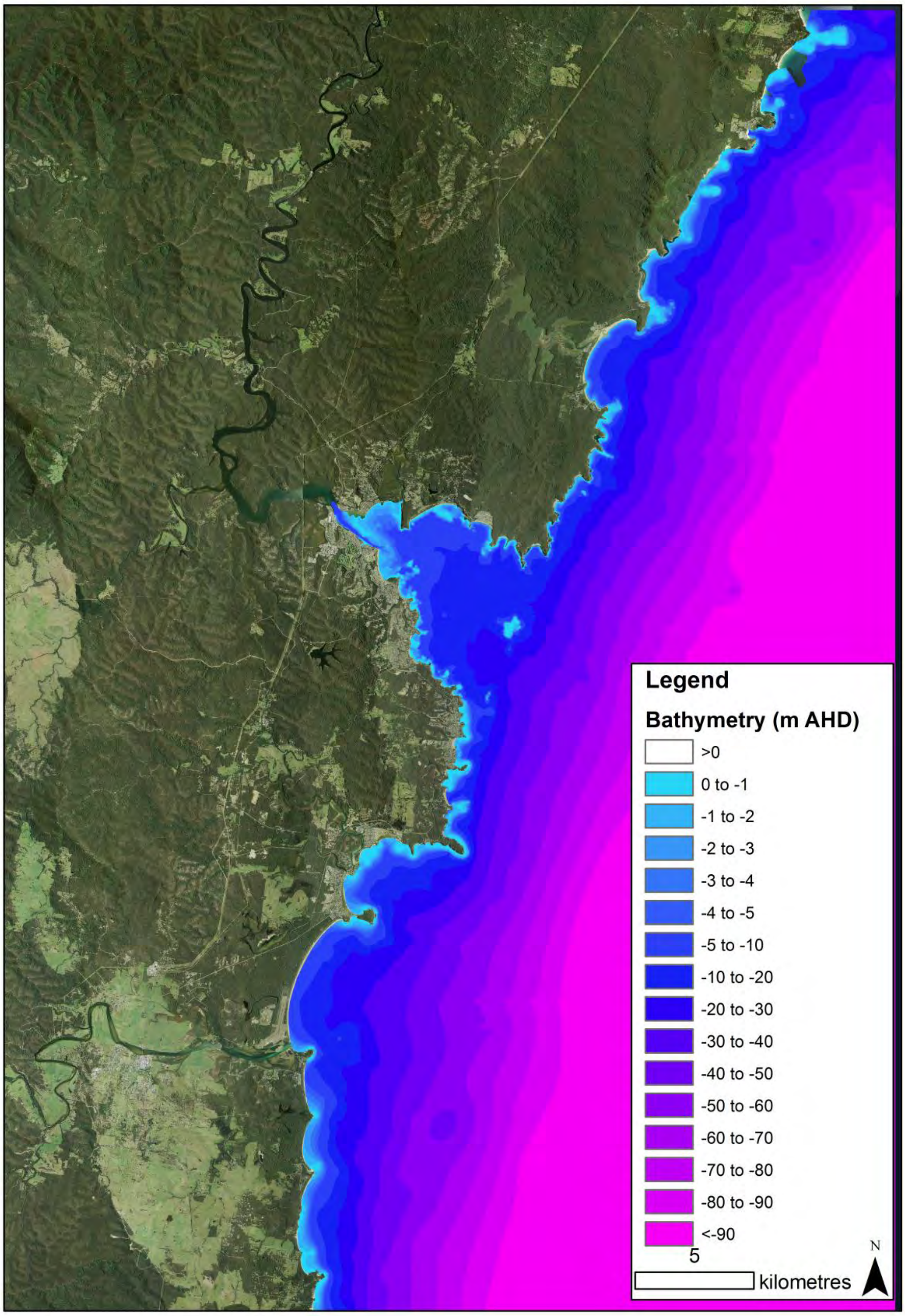


Figure D-2: Eurobodalla Coarse model domain bathymetry

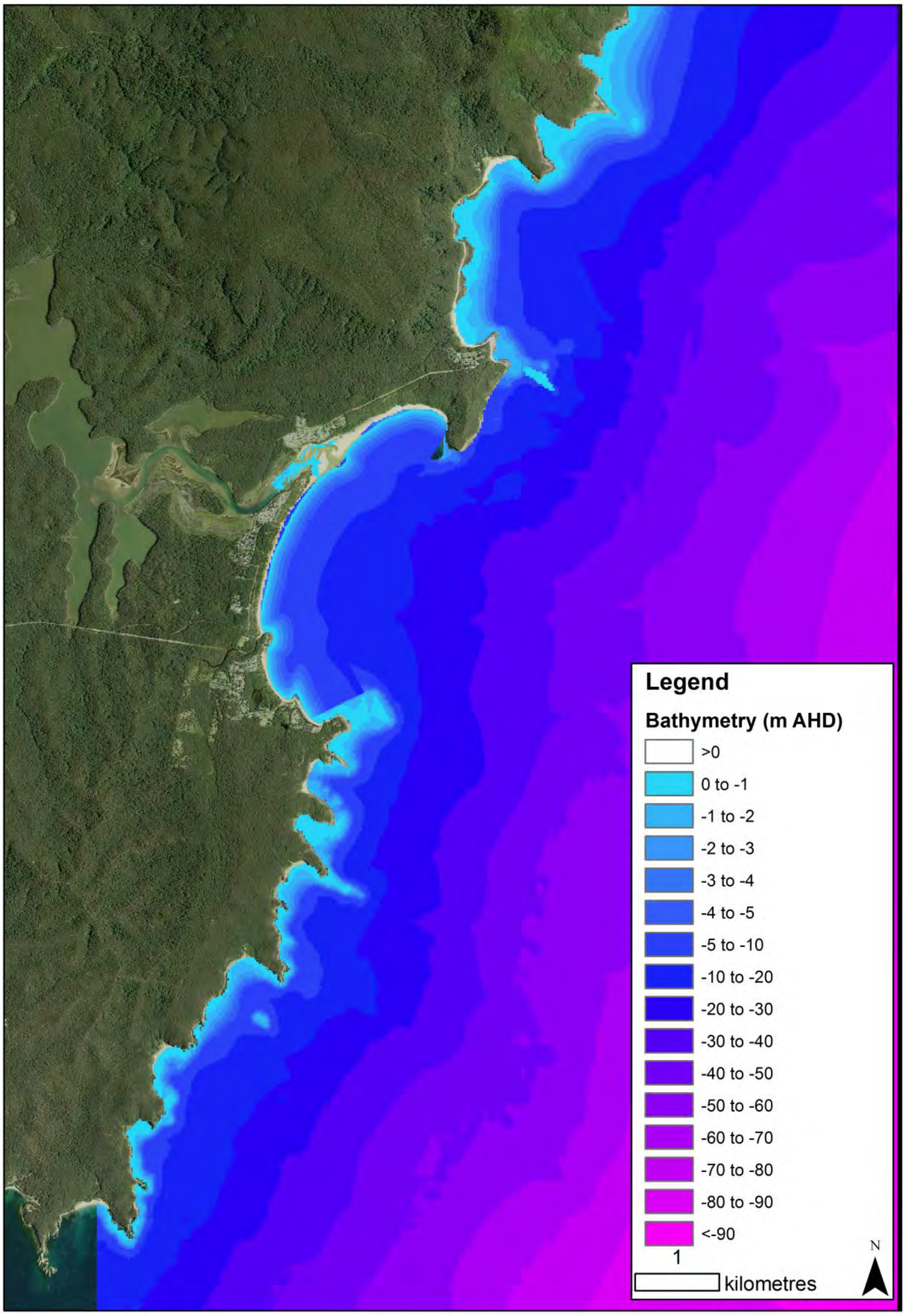


Figure D-3: Durras model domain bathymetry

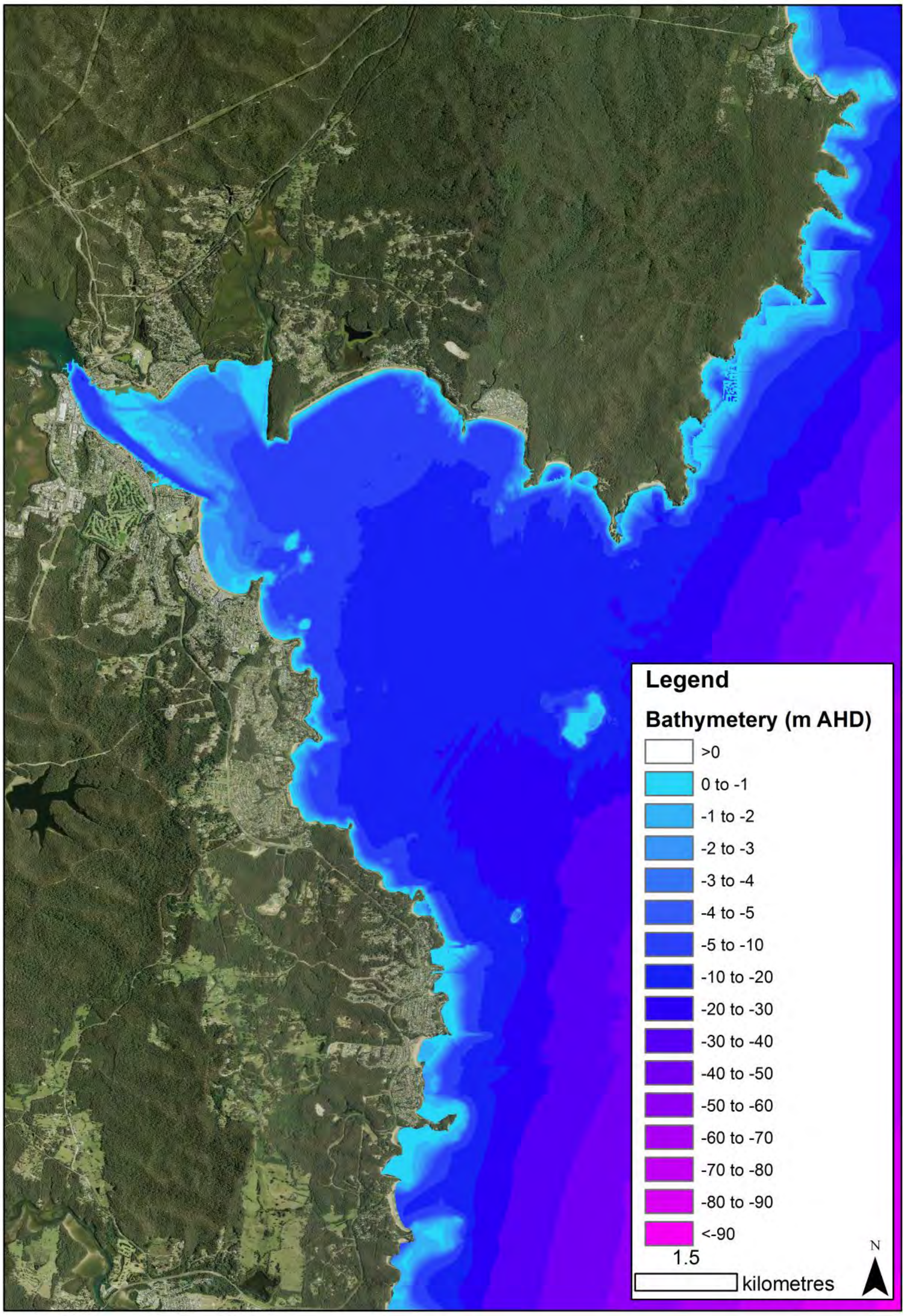


Figure D-4: Batemans Bay model domain bathymetry

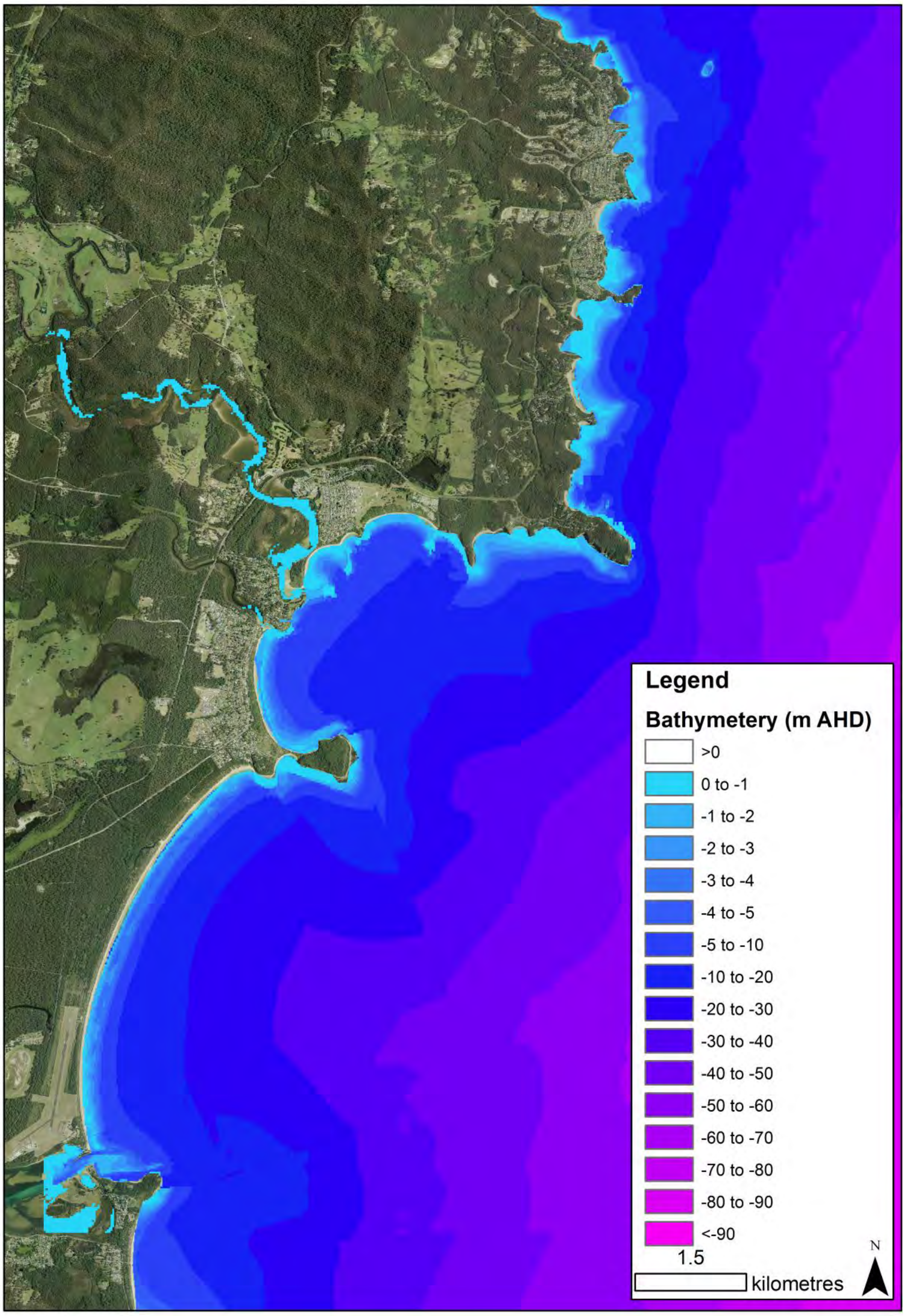


Figure D-5: Moruya model domain bathymetry

D.3.3 Output Locations

For each model simulation, spatial maps of wave height, period and direction have been generated for the four (4) model domains. More detailed information including significant wave height, mean and peak period and direction and depth have also been provided for 33 locations within the study area. These locations are shown in Figure D-6, Figure D-7 and Figure D-8.

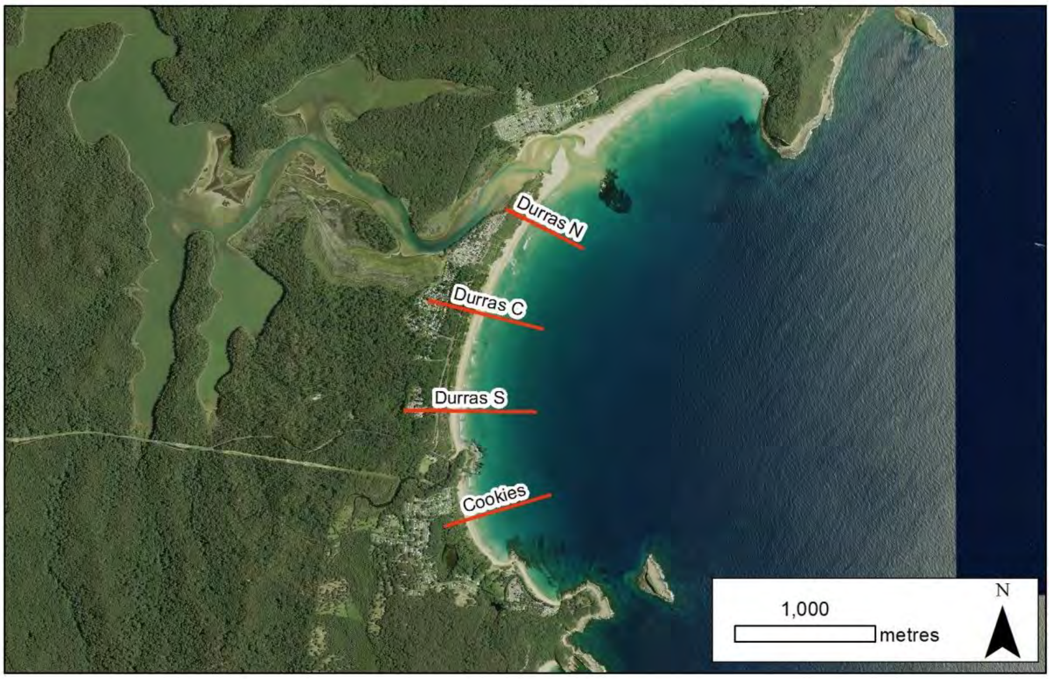


Figure D-6: Output locations for the Durras region

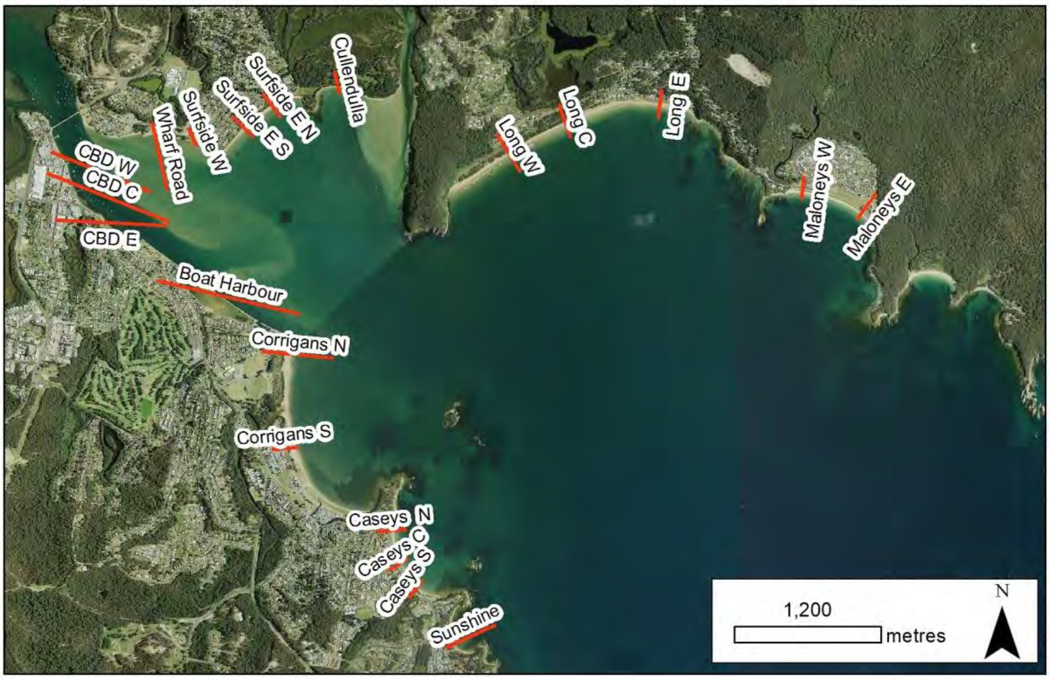


Figure D-7: Output locations for the Batemans Bay region

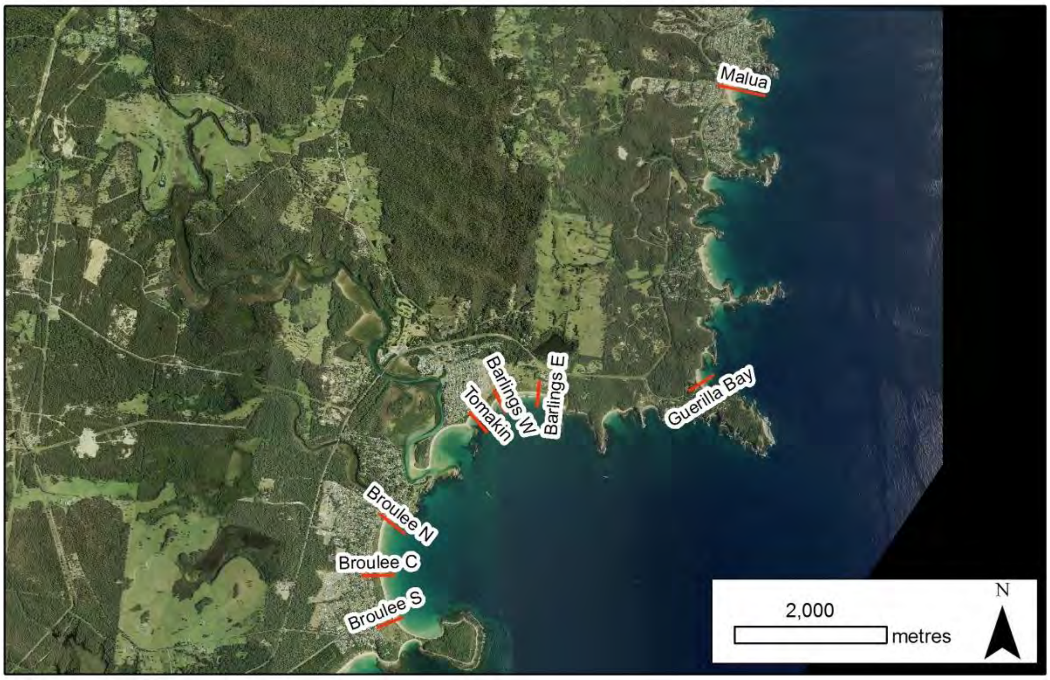


Figure D-8: Output locations for the Moruya region

Rather than specify a single point offshore of a coastal location, information was extracted along transect lines from pre-breaking to the shoreline. This is due to the significant amount of wave transformation which occurs immediately prior to breaking. When an arbitrary output point is specified, the location may be well offshore of the surf zone and will not include final, nearshore transformation or may be inside the surf zone where some loss of spectral wave height through offshore breaking of larger waves has already occurred. By extracting information along a transect, wave conditions at the outer edge of the surf zone may be extracted using the wave breaking fraction. The outer edge of the surf zone is assumed to occur when the wave breaking fraction reaches 1% and wave conditions were extracted and output for that location.

D.4 SWAN Wave Simulations

D.4.1 Wave Parameters

SWAN modelling was undertaken using the model parameters and coefficients shown in Table D-3. Sensitivity tests were undertaken on some coefficients, however some were determined based on WRL's past wave modelling experience.

Table D-3: SWAN Modelling Setup and Parameters

Model Physics	
Physics mode (generation)	3rd
Wave growth formulation	Komen et al. (1984)
Triad wave-wave interaction	On
Nonlinear quadruplet wave interaction	On
Whitecapping	On
Wave breaking model	Battjes and Janssen (1978)
A	1
H_{max}/d (γ)	0.73
Bottom friction (JONSWAP)	0.067 (default)
Model Numerics	
Model Run Mode	Stationary, Two dimensional
Iterations	15
Spectral Parameters	
Spectral Shape at Boundary	JONSWAP
Peak Enhancement Factor	3.3 (default)
Period	Peak
Standard Deviation of Directional Spreading	30 º
Diffraction	Off (recommended)
Directional Space Parameters	
Directional Range	360 º
Directional Resolution	10 º
Frequency Space Parameters	
No. Frequency Bins	32
Min. Frequency	0.05
Max. Frequency	1

D.4.2 Scenarios

Main model scenarios corresponding to 1, 20 and 100 year ARI events (1 hour duration) from directions between north-east and south have been simulated. Event directions have been undertaken at 22.5° increments. The wave height and direction at the model boundaries of the coarse grid were manually adjusted to ensure that the target wave conditions were reproduced at the Batemans Bay wave buoy location. A summary of main scenarios is presented within Table D-4. Table D-4 also shows the median wave conditions at each beach (based on the following wave conditions at the Batemans Bay wave buoy: H_S = 1.30 m, T_P = 9.5 s, SE direction – see Shand et al. 2010 and Coghlan, 2010), which are provided for comparison.

Table D-4: SWAN Main Model Scenarios and Environmental Forcing Factors (1 hour duration)

Scenario	ARI	Water Level	Conditions at Batemans Bay Offshore Wave Buoy			Domain Wind Conditions	
		(m AHD)	Hs (m)	Tp (s)	Dp (°)	V (m/s)	Dir (°)
NE	1	0.51	3	11.6	45	16.3	45
	20	1.37	5	12.8		20.1	
	100	1.43	6.2	13.4		22.2	
ENE	1	0.51	3	11.6	67.5	16.3	67.5
	20	1.37	5	12.8		20.1	
	100	1.43	6.2	13.4		22.2	
E	1	0.51	3.7	11.6	90	16.3	90
	20	1.37	6.1	12.8		20.1	
	100	1.43	7.3	13.4		22.2	
ESE	1	0.51	4.9	11.6	112.5	19.3	112.5
	20	1.37	6.8	12.8		23.8	
	100	1.43	7.7	13.4		26.4	
SE	1	0.51	4.9	11.6	135	19.3	135
	20	1.37	6.8	12.8		23.8	
	100	1.43	7.7	13.4		26.4	
SSE	1	0.51	4.9	11.6	157.5	19.3	157.5
	20	1.37	6.8	12.8		23.8	
	100	1.43	7.7	13.4		26.4	
S	1	0.51	3.7	11.6	180	18.3	180
	20	1.37	6.1	12.8		22.6	
	100	1.43	7.3	13.4		25.0	
SE	median	0.00	1.3	9.5	135	-	-

Additional wave model scenarios were run, but the environmental conditions and resulting wave heights are not presented in this report for brevity. Scenarios include:

- The 100 year ARI wave conditions with a duration of 3 hours, 6 hours, 12 hours, 24 hours, 48 hours, 96 hours and 144 hours (for SBEACH erosion modelling and Durras Lake tailwater conditions);
- The 20 year ARI wave conditions with a duration of 3 hours, 6 hours, 12 hours, 24 hours, 48 hours, 96 hours and 144 hours (for Durras Lake tailwater conditions);
- The 1 year ARI wave conditions with a duration of 3 hours, 6 hours, 12 hours, 24 hour and 48 hours (for Durras Lake tailwater conditions);
- The wave height exceeded for 12 hours per year (0.137% exceedance) for Hallermeier (1983) outer depth of closure calculations; and
- The 10% exceedance wave height (for SBEACH erosion modelling and Durras Lake tailwater conditions).

While the 20 and 100 year ARI events wave conditions have been combined with the 20 and 100 year ARI events for water level conditions (tide plus anomaly), respectively, 1 year ARI wave conditions have been combined with the Mean High Water (MHW) level (0.51 m AHD) as previously agreed with OEH. This wave and water level combination is considered more representative of that which would result in 1 year ARI coastal inundation rather than assuming complete dependence of the variables (i.e. 1 year ARI waves and 1 year ARI water level).

D.4.3 *Diffraction at Cullendulla*

The dominant dissipative processes behind Square Head at Cullendulla are diffraction and refraction. Therefore, at this location, desktop methods were preferred to using the SWAN wave model whose diffraction approximation does not properly handle diffraction into bays around large headlands (Delft University of Technology, 2016).

To estimate the diffracted wave height at Cullendulla, the irregular wave diffraction diagram for waves passing through a structure gap with *B*/*L* = 2.0 (ratio of entrance width to local wavelength) and S_{MAX} = 75 (directional spreading function; value appropriate for swell waves) published by Goda (2000) was overlain on the study area as shown in Figure D-9. A diffraction factor of 0.35 was used for design.

Figure D-9: Irregular Wave Diffraction Coefficients at Cullendulla

D.5 Results

Examples of SWAN output for each model grid for the 100 year ARI event from the east-south-east (ESE) are shown in Figure D-10, Figure D-11, Figure D-12 and Figure D-13.

For each output location (coordinates in MGA Zone 56), wave conditions for each direction scenario were evaluated and maximum conditions (at outer breakpoint) are summarised in Table D-5. The median wave statistics originating from a south easterly (SE) direction are also included for comparison. Output locations are shown in Figure D-14 to Figure D-23.

Note that the bed elevations referred to in Table D-5 are for present day (2017) conditions only. No changes were made to the bathymetry grids to represent possible future changes to the seabed due to projected sea level rise.

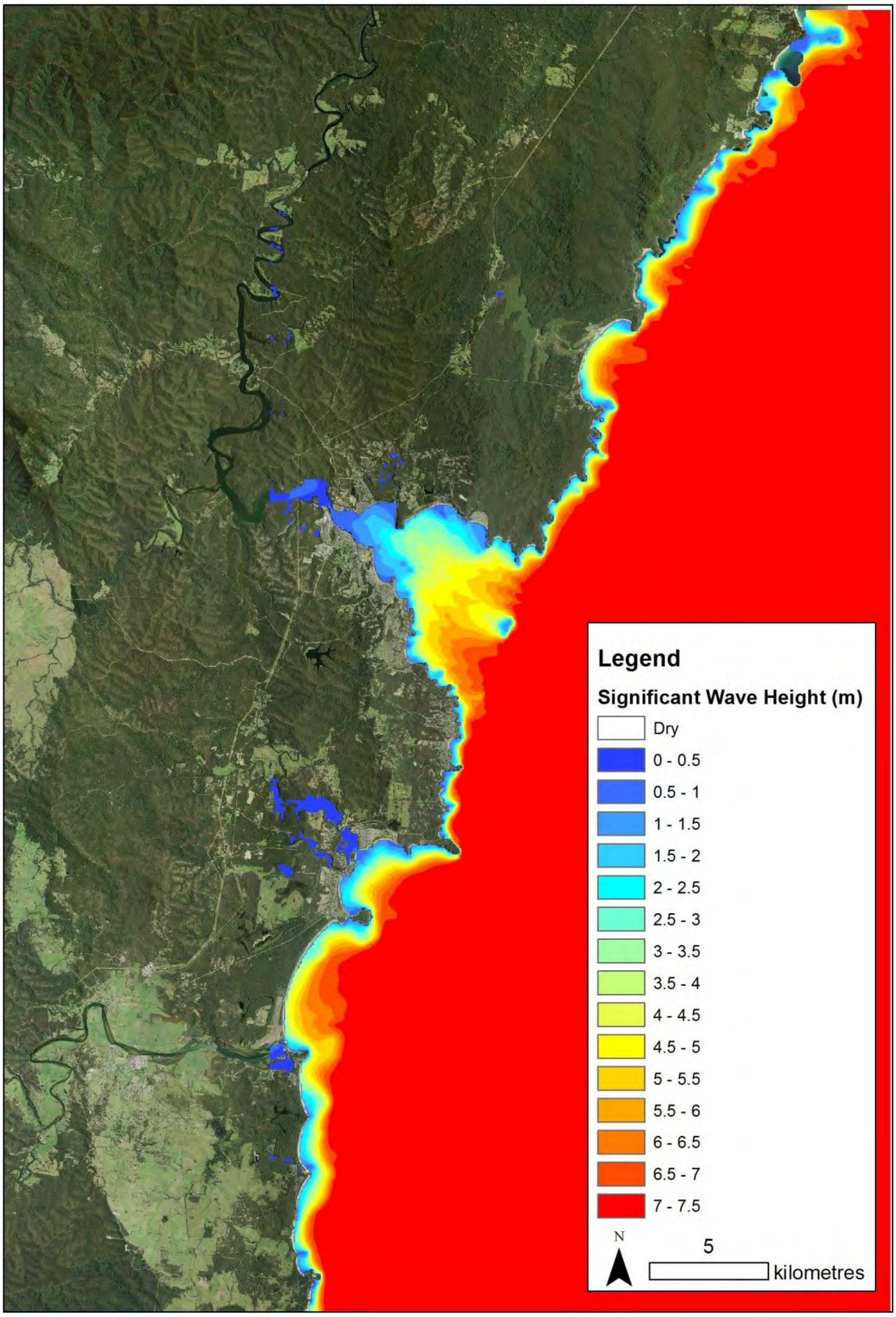


Figure D-10: Example of 100 year ARI east south-east event for the Eurobodalla Coarse model domain

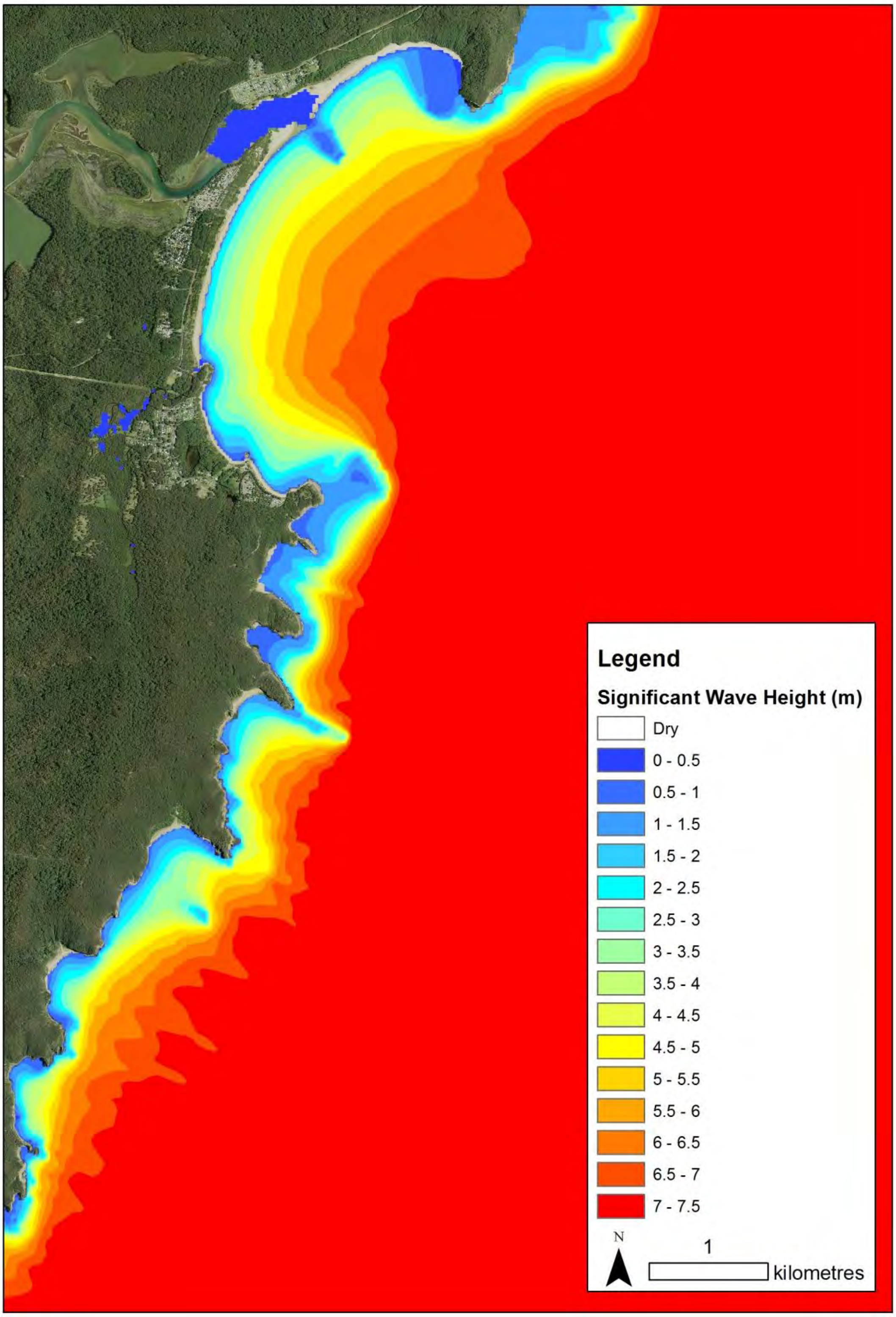


Figure D-11: Example of 100 year ARI east south-east event for the Durras model domain

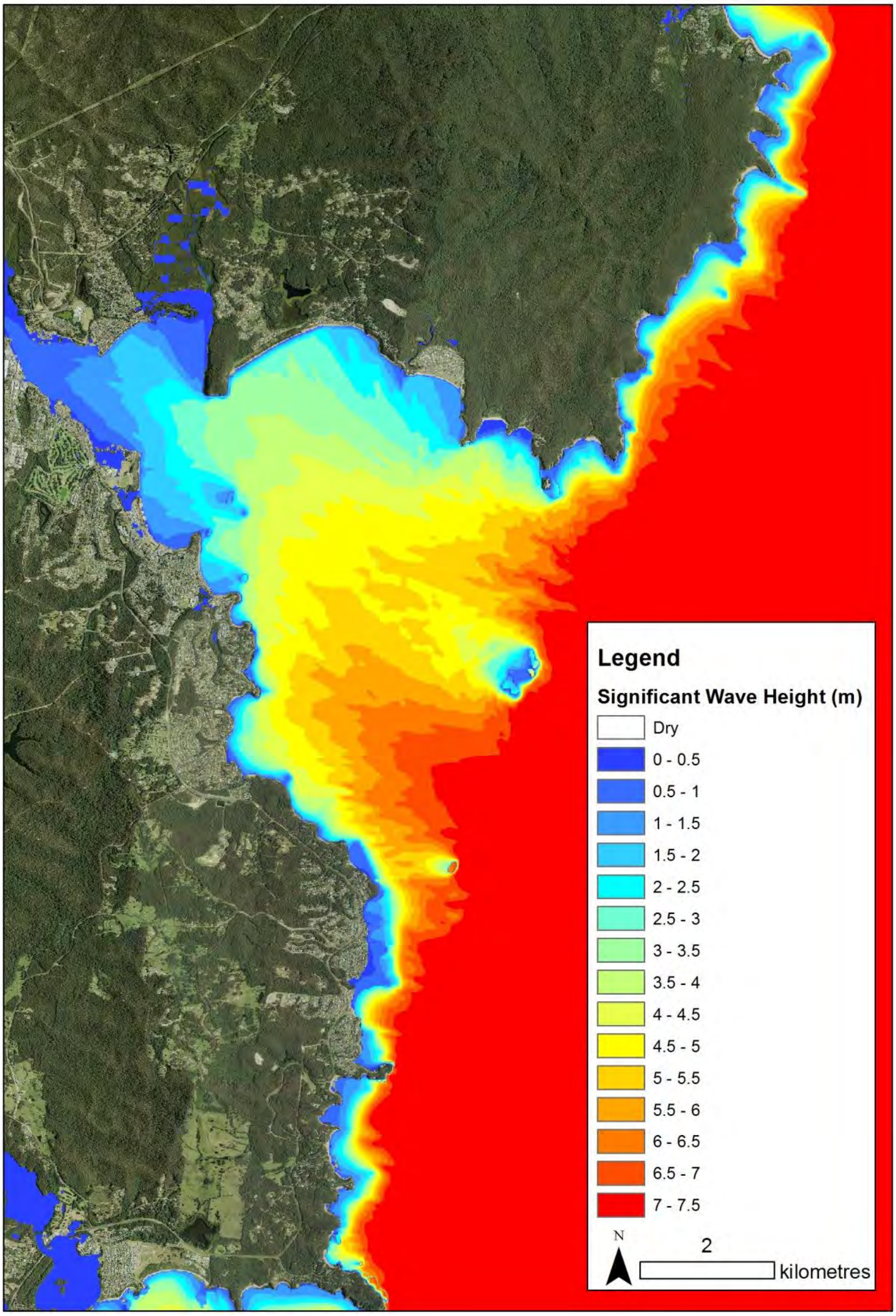


Figure D-12: Example of 100 year ARI east south-east event for the Batemans Bay model domain

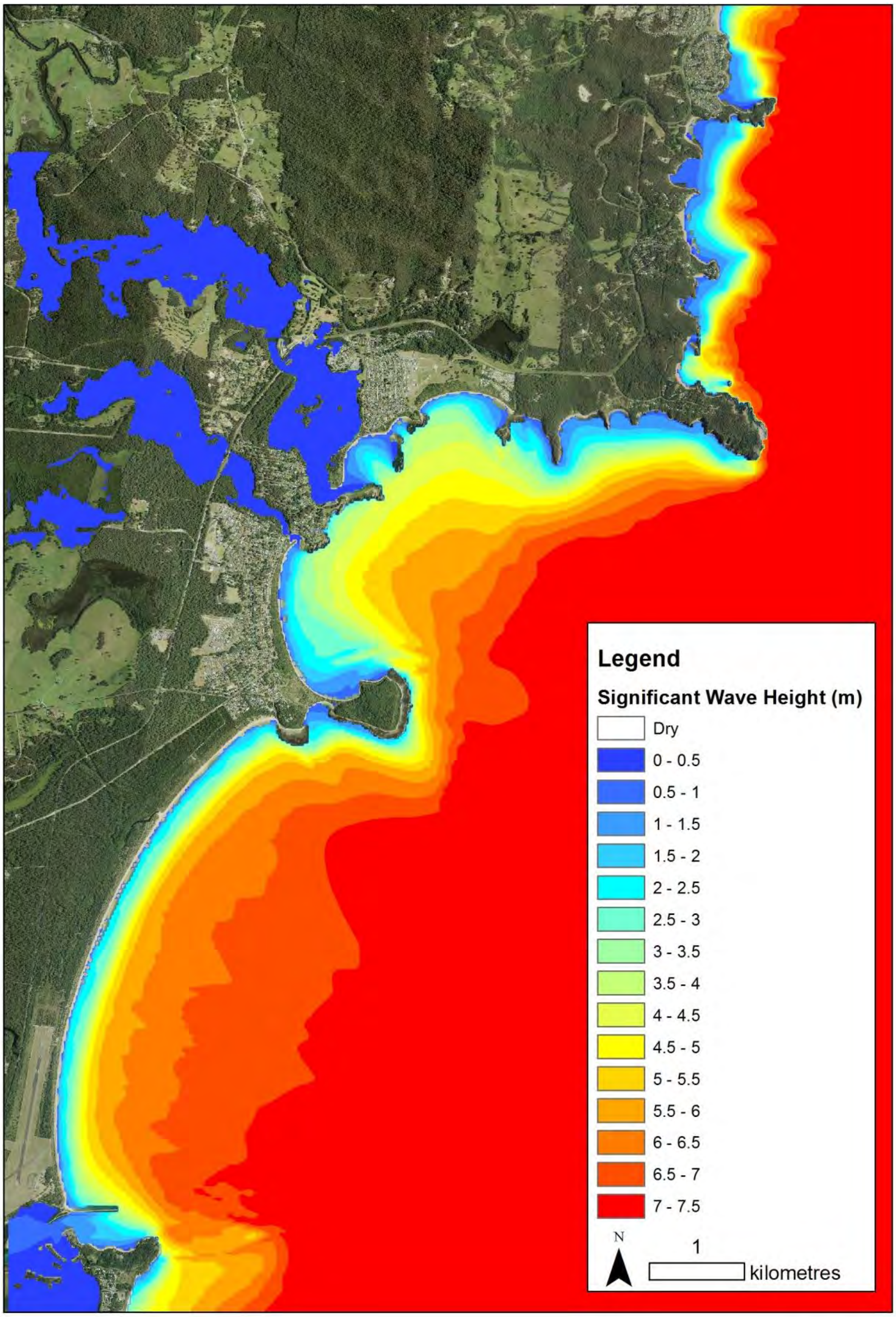


Figure D-13: Example of 100 year ARI east south-east event for the Moruya model domain

Table D-5: Maximum Wave Conditions at the Outer Breakpoint of the Surf Zone

Beach	Profile	ARI (year)	Easting (m)	Northing (m)	Bed Elevation (m AHD)	Offshore Wave Direction (°)	Hs (m)	Tp (s)	Peak Direction (°)
Durras	North	Median	255806	6052180	-3.3	SE	1.1	9.2	125
		1	255962	6052102	-6.0	SE	3.4	11.2	125
		20	256051	6052057	-7.4	SE	4.2	12.3	125
		100	256095	6052034	-8.0	SSE	4.5	13.6	135
	Central	Median	255447	6051552	-3.1	SE	1.0	9.2	115
		1	255617	6051510	-5.8	ESE	3.4	11.2	115
		20	255762	6051474	-7.7	ESE	4.4	12.3	115
		100	255811	6051461	-8.4	ESE	4.7	13.6	115
	South	Median	255260	6050876	-2.3	SE	0.8	9.2	95
		1	255410	6050874	-5.2	ESE	3.0	11.2	95
		20	255708	6050872	-8.3	ESE	4.6	12.3	95
		100	255758	6050871	-8.9	ESE	5.0	13.6	95
Cookies	-	Median	255364	6050132	-2.8	SE	0.7	9.2	75
		1	255723	6050238	-5.9	ESE	3.4	11.2	85
		20	255818	6050266	-7.8	ESE	4.4	12.3	95
		100	255866	6050280	-8.6	E	4.8	13.6	95
Maloneys	East	Median	251181	6044556	-2.9	SE	0.4	9.2	215
		1	251196	6044576	-1.4	SSE	1.1	12.3	215
		20	251196	6044576	-1.4	S	1.3	13.6	215
		100	251181	6044556	-2.9	S	1.5	13.6	205
	West	Median	250736	6044748	-1.6	SE	0.5	9.2	185
		1	250733	6044724	-2.8	SSE	1.5	11.2	175
		20	250733	6044724	-2.8	SSE	1.8	12.3	185
		100	250733	6044724	-2.8	S	1.9	13.6	185

Table D-5: Maximum Wave Conditions at the Outer Breakpoint of the Surf Zone (cont…)

Beach	Profile	ARI (year)	Easting (m)	Northing (m)	Bed Elevation (m AHD)	Offshore Wave Direction (°)	Hs (m)	Tp (s)	Peak Direction (°)
Long	East	Median	249569	6045434	-1.8	SE	0.4	9.2	175
		1	249566	6045410	-2.5	SSE	1.6	11.2	175
		20	249563	6045386	-3.2	SSE	1.9	12.3	175
		100	249563	6045386	-3.2	SSE	2.0	13.6	175
	Central	Median	249315	6045432	-3.1	SE	0.4	9.2	165
		1	249315	6045432	-3.1	SSE	1.8	11.2	165
		20	249318	6045408	-4.4	SSE	2.3	12.3	165
		100	249318	6045408	-4.4	SSE	2.4	13.6	165
	West	Median	248403	6045008	-2.9	SE	0.7	9.2	145
		1	248429	6044966	-5.0	SSE	2.6	11.2	155
		20	248442	6044944	-6.0	SSE	3.0	12.3	155
		100	248442	6044944	-6.0	SSE	3.1	13.6	155
Cullendulla	-	Median	246963*	6045588*	-1.0	SE	0.2	9.2	185
		1	246963*	6045588*	-1.0	SSE	0.8	11.2	175
		20	246963*	6045588*	-1.0	SSE	0.9	12.3	175
		100	246963*	6045588*	-1.0	SSE	0.9	13.6	175
Surfside E	North	Median	246480	6045396	-2.3	SE	0.3	9.2	145
		1	246465	6045416	-1.5	SSE	1.2	11.2	145
		20	246465	6045416	-1.5	SSE	1.4	12.3	145
		100	246480	6045396	-2.3	SSE	1.5	13.6	145
	South	Median	246246	6045242	-1.7	SE	0.4	9.2	135
		1	246246	6045242	-1.7	SE	1.4	11.2	135
		20	246263	6045224	-2.3	SSE	1.6	12.3	135
		100	246263	6045224	-2.3	SE	1.6	13.6	135
Surfside W	-	Median	245793	6045159	-1.1	SE	0.2	9.2	145
		1	245793	6045159	0.1	SE	0.6	11.2	155
		20	245793	6045159	0.1	SSE	0.7	12.3	155
		100	245793	6045159	0.1	SSE	0.7	13.6	155

*Approximate location only, determined using a diffraction factor of 0.35

Table D-5: Maximum Wave Conditions at the Outer Breakpoint of the Surf Zone (cont…)

Beach	Profile	ARI (year)	Easting (m)	Northing (m)	Bed Elevation (m AHD)	Offshore Wave Direction (°)	Hs (m)	Tp (s)	Peak Direction (°)
Wharf Rd	-	Median	245555	6044795	-0.8	SE	0.2	2.4	105
		1	245555	6044795	-0.6	ESE	0.9	11.2	105
		20	245555	6044795	-0.6	ESE	1.0	12.3	105
		100	245555	6044795	-0.6	ESE	1.1	13.6	105
CBD	West	Median	245419	6044788	-0.3	SE	0.1	1.2	105
		1	245419	6044788	-0.3	SE	0.8	11.2	105
		20	245419	6044788	-0.3	ESE	0.9	12.3	105
		100	245419	6044788	-0.3	ESE	0.9	13.6	105
	Central	Median	245567	6044546	-0.5	SE	0.4	9.2	85
		1	245567	6044546	-0.5	ESE	0.9	11.2	85
		20	245567	6044546	-0.5	ESE	1.0	12.3	85
		100	245567	6044546	-0.5	ESE	1.0	13.6	85
	East	Median	245567	6044546	-0.5	SE	0.4	9.2	85
		1	245567	6044546	-0.5	ESE	0.9	11.2	85
		20	245567	6044546	-0.5	ESE	1.0	12.3	85
		100	245567	6044546	-0.5	ESE	1.0	13.6	85
Boat Harbour	-	Median	246636	6043788	-2.0	SE	0.6	9.2	95
		1	246636	6043788	-2.0	ESE	1.5	11.2	105
		20	246636	6043788	-2.0	ESE	1.7	12.3	105
		100	246636	6043788	-2.0	ESE	1.7	13.6	105
Corrigans	North	Median	246598	6043462	-2.3	SE	0.5	9.2	85
		1	246598	6043462	-2.3	ESE	1.4	11.2	85
		20	246918	6043432	-2.8	ESE	2.0	12.3	95
		100	246918	6043432	-2.8	ESE	2.0	13.6	95
	South	Median	246629	6042705	-1.9	SE	0.4	4.7	75
		1	246629	6042705	-1.9	ESE	1.1	11.2	85
		20	246629	6042705	-1.9	ESE	1.2	12.3	85
		100	246629	6042705	-1.9	ESE	1.3	13.6	85

Table D-5: Maximum Wave Conditions at the Outer Breakpoint of the Surf Zone (cont…)

Beach	Profile	ARI (year)	Easting (m)	Northing (m)	Bed Elevation (m AHD)	Offshore Wave Direction (°)	Hs (m)	Tp (s)	Peak Direction (°)
Caseys	North	Median	247457	6042028	-1.8	SE	0.5	9.2	95
		1	247481	6042030	-2.9	ESE	1.8	12.3	95
		20	247481	6042030	-2.9	ESE	2.0	12.3	95
		100	247506	6042032	-3.6	ENE	2.2	13.6	95
	Central	Median	247507	6041785	-1.9	SE	0.4	9.2	75
		1	247507	6041785	-1.9	ESE	1.4	12.3	75
		20	247527	6041798	-2.7	E	1.7	13.6	75
		100	247527	6041798	-2.7	ENE	1.8	13.6	75
	South	Median	247642	6041622	-1.8	SE	0.4	9.2	35
		1	247627	6041602	-1.3	ESE	1.2	11.2	55
		20	247642	6041622	-1.8	E	1.4	12.3	55
		100	247642	6041622	-1.8	NE	1.5	13.6	45
Sunshine	-	Median	247963	6041129	-1.4	SE	0.4	4.7	55
		1	248167	6041224	-5.9	ESE	3.1	11.2	115
		20	248234	6041254	-7.0	ESE	3.9	12.3	115
		100	248234	6041254	-7.0	ESE	4.0	13.6	115
Malua	-	Median	249857	6035412	-2.9	SE	1.1	9.2	115
		1	249977	6035384	-6.9	ESE	3.7	11.2	115
		20	250122	6035351	-10.7	ESE	5.5	12.3	115
		100	250194	6035334	-12.0	ESE	6.4	13.6	115
Guerilla	-	Median	249374	6031600	-2.6	SE	0.5	9.2	65
		1	249415	6031626	-4.2	ESE	2.5	11.2	75
		20	249498	6031678	-7.5	ESE	4.0	12.3	85
		100	249498	6031678	-7.5	E	4.3	13.6	75

Table D-5: Maximum Wave Conditions at the Outer Breakpoint of the Surf Zone (cont…)

Beach	Profile	ARI (year)	Easting (m)	Northing (m)	Bed Elevation (m AHD)	Offshore Wave Direction (°)	Hs (m)	Tp (s)	Peak Direction (°)
Barlings	East	Median	247172	6031325	-2.2	SE	0.6	9.2	185
		1	247170	6031300	-3.1	SSE	2.0	11.2	175
		20	247168	6031275	-3.9	S	2.5	12.3	165
		100	247165	6031250	-4.4	S	2.8	13.6	165
	West	Median	246731	6031218	-3.8	SE	1.0	9.2	145
		1	246754	6031174	-5.4	SSE	3.2	11.2	145
		20	246765	6031152	-6	SSE	3.4	12.3	155
		100	246765	6031152	-6	S	3.5	13.6	155
Tomakin	-	Median	246403	6031042	-2.0	SE	0.6	9.2	135
		1	246502	6030930	-6.3	SE	3.2	11.2	145
		20	246519	6030912	-6.8	ESE	3.6	12.3	145
		100	246519	6030912	-6.8	ESE	3.7	13.6	145
Broulee	North	Median	245306	6029683	-2.5	SE	0.9	9.2	115
		1	245404	6029606	-5.0	ESE	2.9	11.2	115
		20	245443	6029576	-5.7	ESE	3.4	12.3	115
		100	245443	6029576	-5.7	ESE	3.5	13.6	115
	Central	Median	245224	6029049	-1.9	SE	0.5	9.2	85
		1	245273	6029050	-3.2	ESE	1.9	11.2	95
		20	245298	6029050	-3.6	ESE	2.4	12.3	95
		100	245322	6029051	-3.9	E	2.6	13.6	95
	South	Median	245359	6028478	-1.6	SE	0.4	9.2	55
		1	245406	6028496	-2.4	ESE	1.4	11.2	65
		20	245406	6028496	-2.4	ESE	1.7	12.3	65
		100	245406	6028496	-2.4	E	1.8	13.6	65

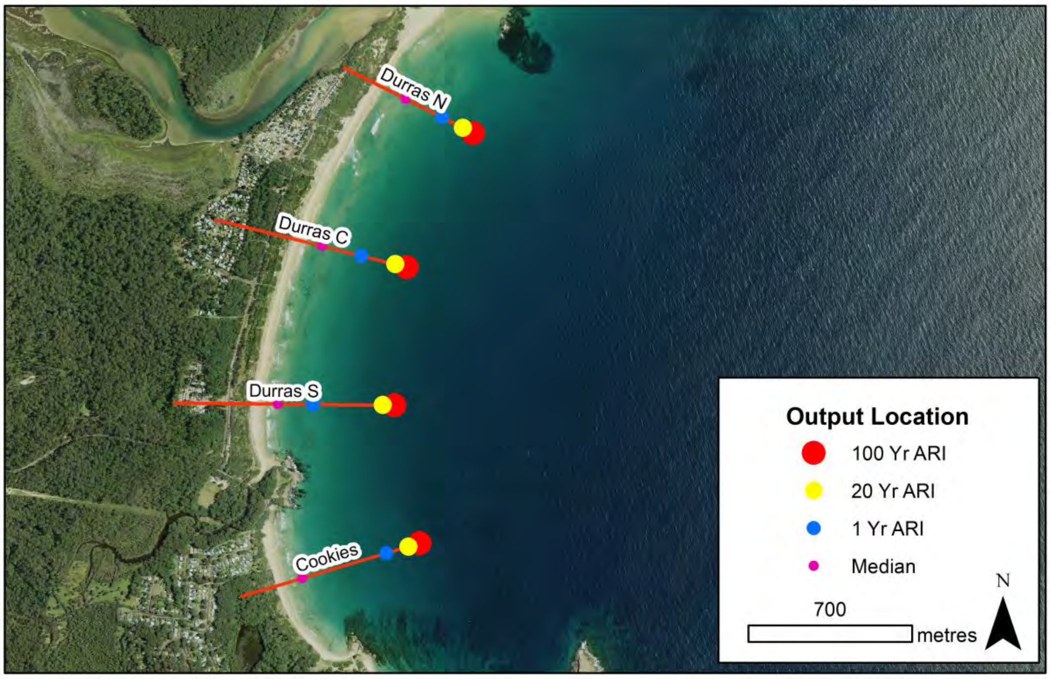


Figure D-14: SWAN Output Locations: Durras Beach and Cookies Beach

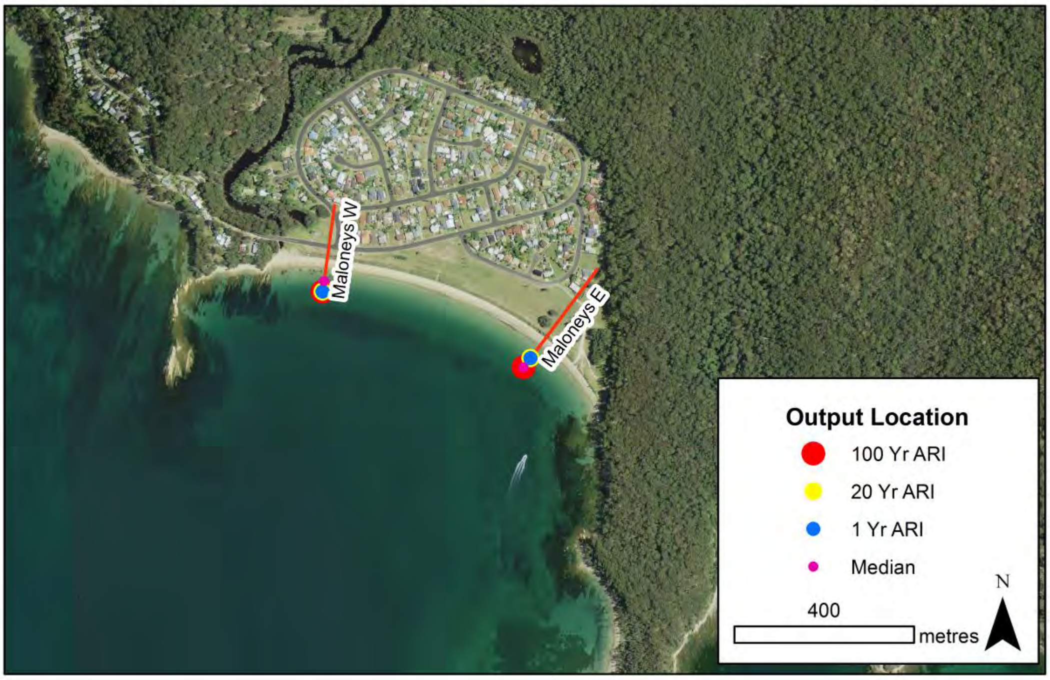


Figure D-15: SWAN Output Locations: Maloneys Beach

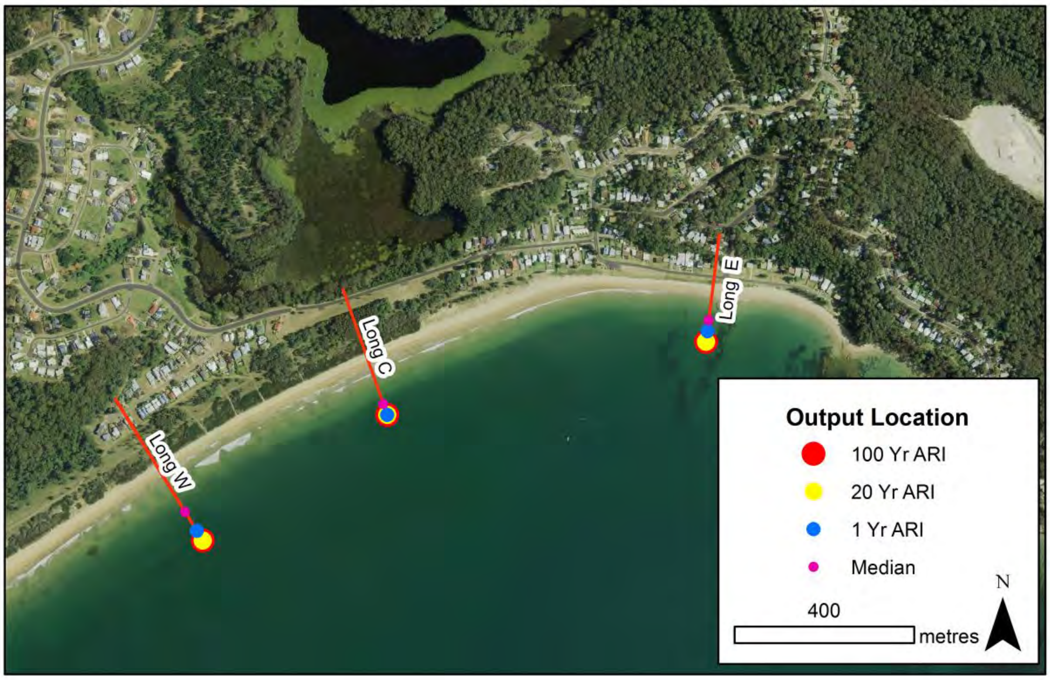


Figure D-16: SWAN Output Locations: Long Beach

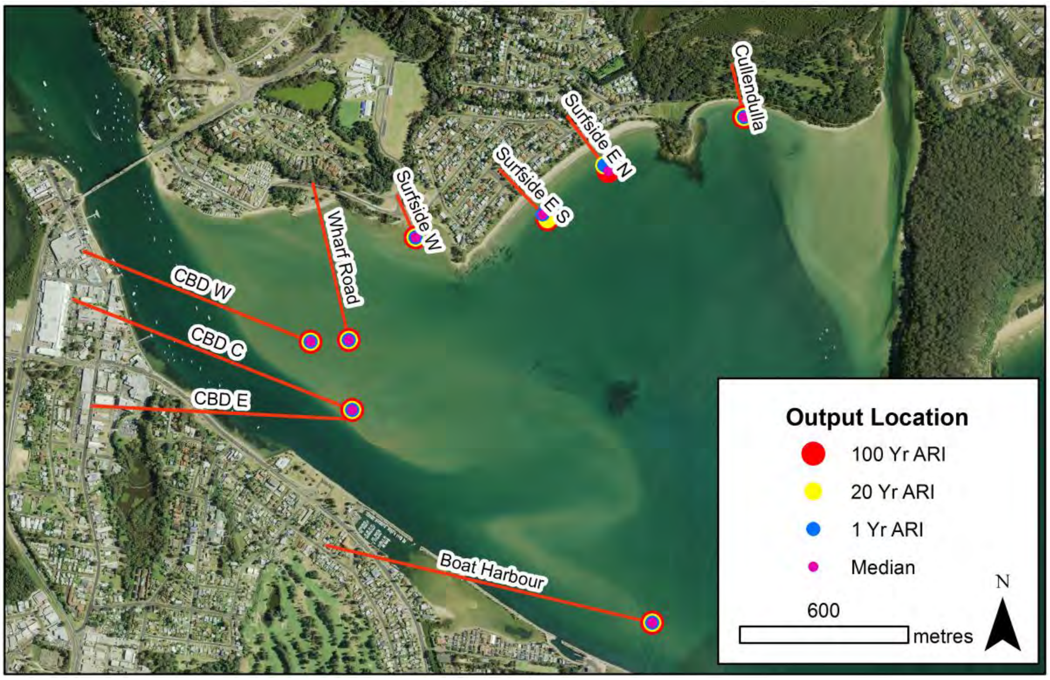


Figure D-17: SWAN Output Locations: Inner Batemans Bay Beaches

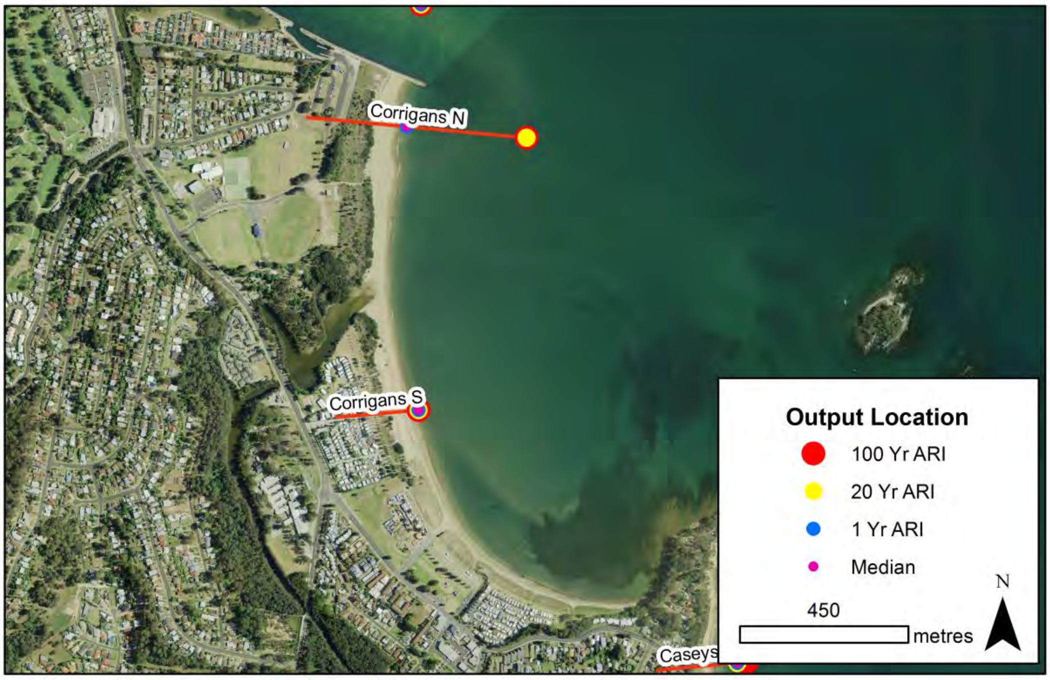


Figure D-18: SWAN Output Locations: Corrigans Beach

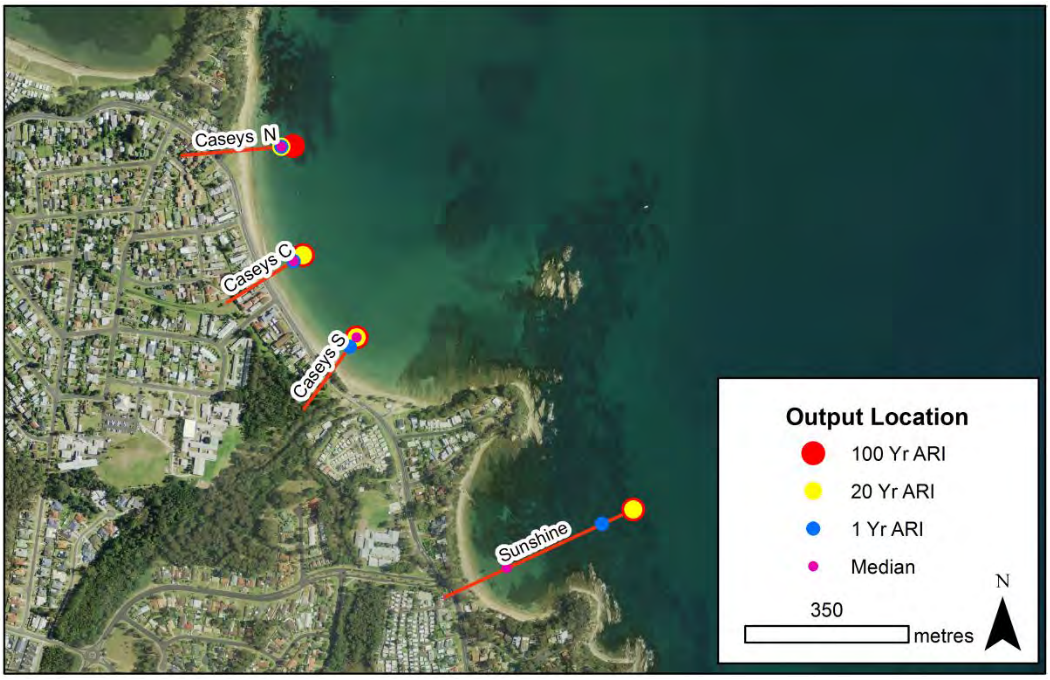


Figure D-19: SWAN Output Locations: Caseys Beach and Sunshine Bay

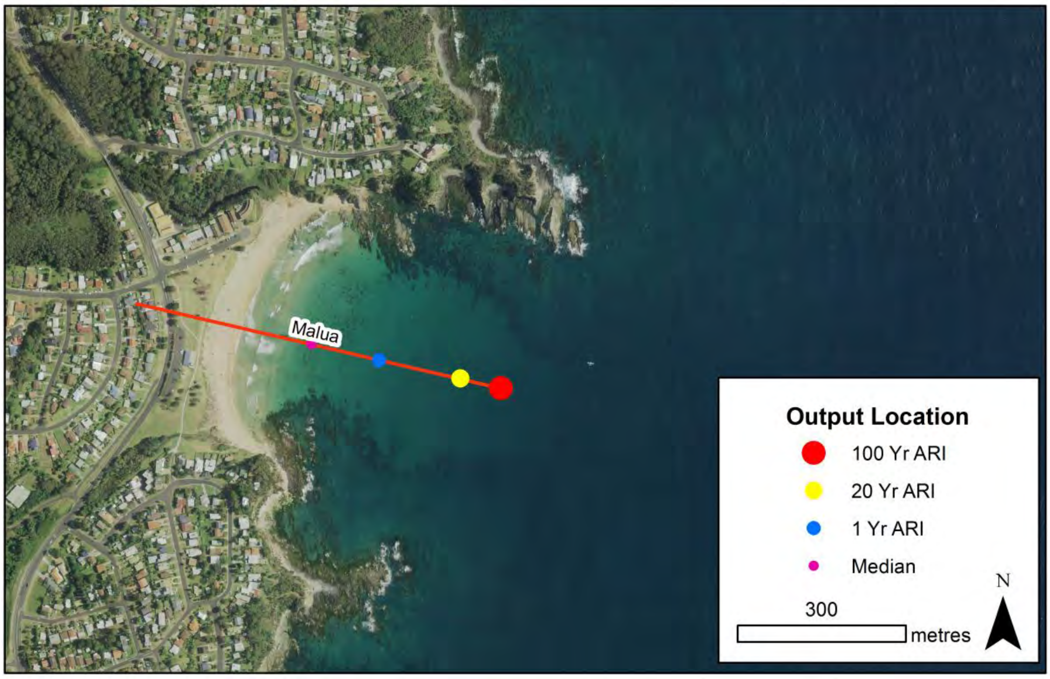


Figure D-20: SWAN Output Locations: Malua Bay

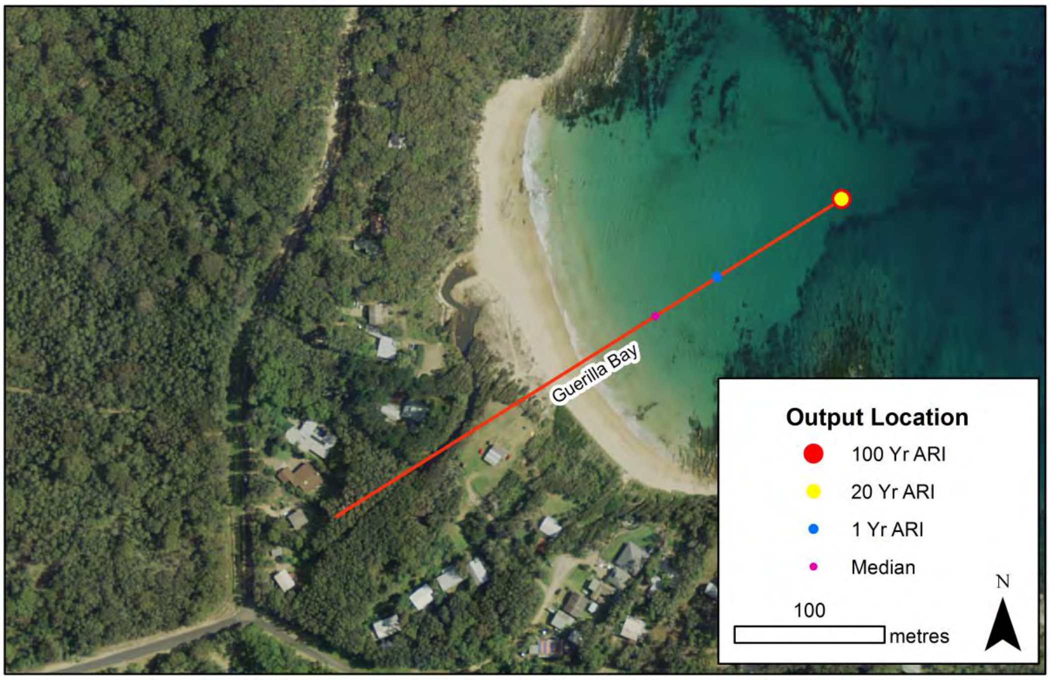


Figure D-21: SWAN Output Locations: Guerilla Bay

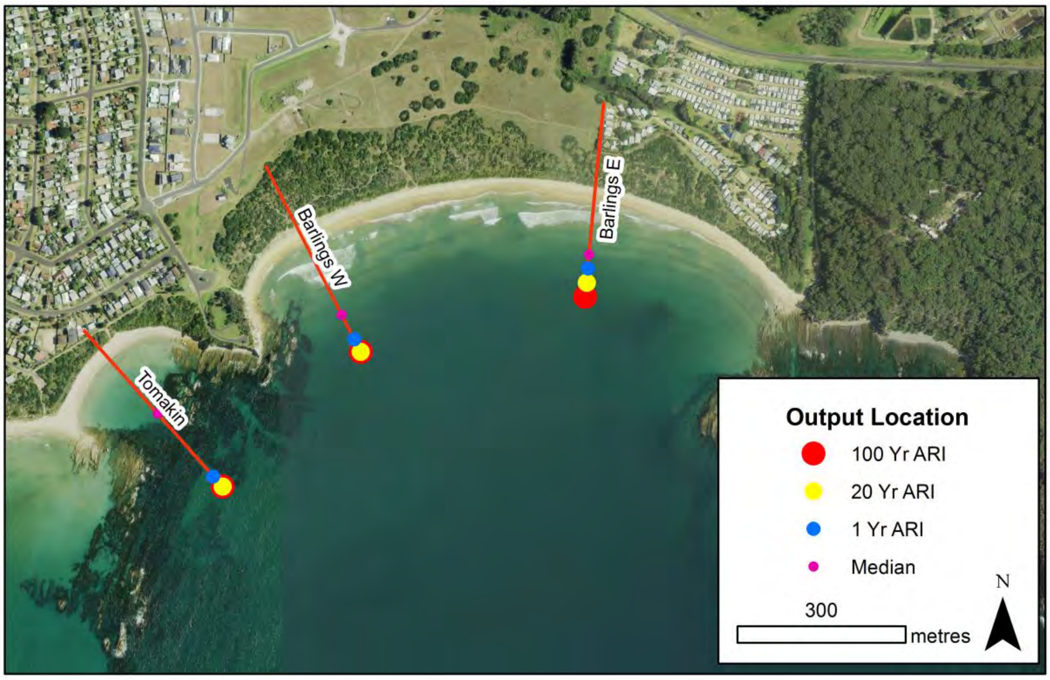


Figure D-22: SWAN Output Locations: Tomakin Cove and Barlings Beach

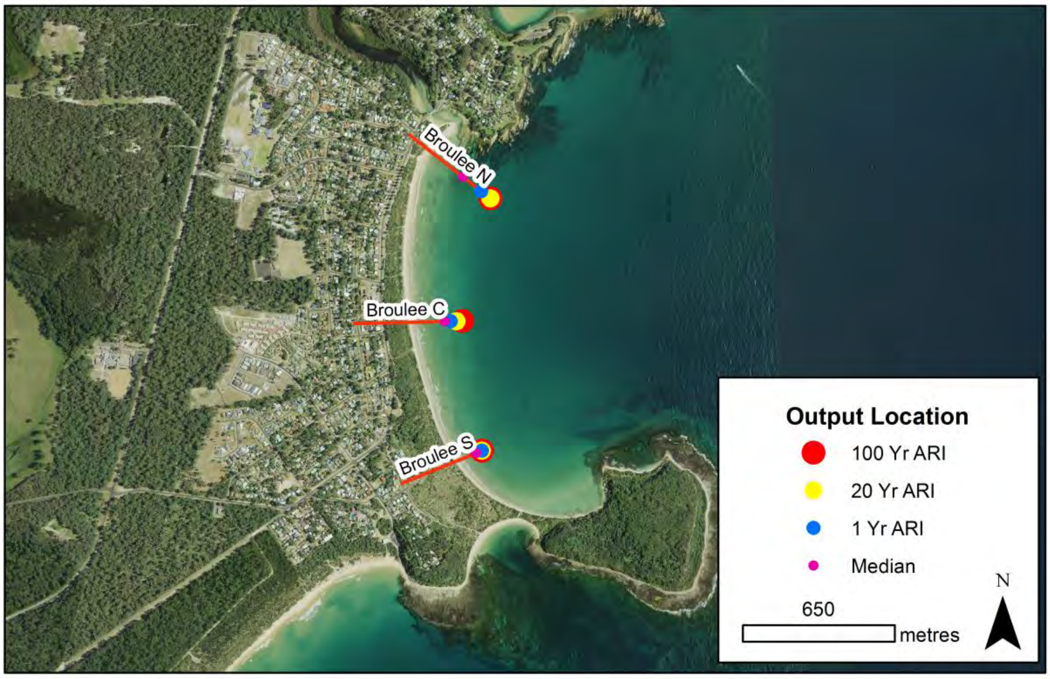


Figure D-23: SWAN Output Locations: Broulee Beach

D.6 Historical Nearshore Wave Photos

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. During this event, Mr Lindsay Usher of ESC photographed the nearshore wave conditions on 6 June 2012 at Long Beach (central section, Figure D-24) and Surfside Beach (east) (northern end, Figure D-25). Based on interpolation of existing SWAN results, WRL estimates that the significant wave heights at outer edge of the surf zone of these beaches at the peak of the storm to be 2.0 m and 1.3 m, respectively. These local wave heights are considered to have an approximate average recurrence interval of 5 years.

Figure D-24: Nearshore Waves at Long Beach, 6 June 2012 (Mr Lindsay Usher)

Figure D-25: Nearshore Waves at Surfside Beach (East), 6 June 2012 (Mr Lindsay Usher)

Appendix E: SBEACH Model Methodology and Calibration

E.1 Preamble

The modelling program SBEACH (Storm-induced Beach Change) was developed by the U.S. Army Corps of Engineering (USACE) Coastal Engineering Research Center and is an empirically based two dimensional model used to examine the short-term response to beach, berm and dune profiles to storm events. Details of the model are given in Larson and Kraus (1989) and Larson, Kraus and Byrnes (1990). SBEACH considers sand grain size, the pre-storm beach profile and dune height, plus time series of wave height, wave period and water level in calculating a post-storm beach profile. In this study, SBEACH (version 4.03) has been used to quantify the estimated storm demand at each of the beaches in response to a synthetic design storm. This appendix outlines the methodology used in the SBEACH modelling and the calibration of the model to beaches within the ESC region.

E.2 Available Observed Profile Data at Bengello Beach

Through discussions with ESC and OEH, it was agreed that the SBEACH model would be calibrated at Bengello Beach, where reliable monitoring data exists. Bengello Beach has been monitored (approximately monthly) since 1972 with traditional survey techniques (Thom and Hall, 1991, McLean and Shen, 2006). The four profiles used for model calibration are shown in Figure E-1.

Figure E-1: SBEACH erosion profiles Bengello Beach (calibration only)

In the last 45 years, the most erosive period occurred over three weeks during May – June 1974 (shown in Figure E-2). Measured profile data was recorded at four profiles before and after this storm period, with recorded erosion summarised in Table E-1. The maximum storm erosion over this period was 200 m^3/m above -0.94 m AHD (estimated to be approximately 170 m^3/m above 0 m AHD) and the average storm erosion across the four profiles was estimated at 95 m^3/m

above 0 m AHD (McLean et al*.* 2010). Both the maximum and average storm erosion was used in model calibration to provide a range of estimated storm demands.

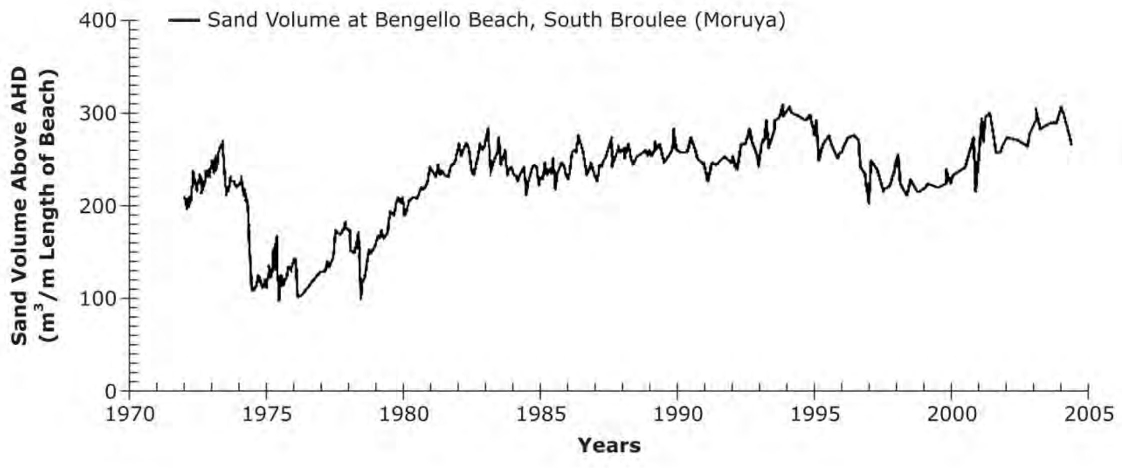


Figure E-2: Timeseries of sand volume change at Bengello Beach (Source: McLean et al., 2010)

Table E-1: Summary of Storm Demand at Bengello Beach for the 1974 Storm

Profile	**Volume of Storm Demand** (m^3/m above -0.94 m AHD)
1	150
2	200
3	130
4	160

Erosion data for the four (4) profiles at Bengello Beach from the 1974 storm period is available for calibration of the SBEACH model, however, the wave and water level data is not. The methodology used to create a synthetic design storm is described in the sections below.

A photograph of Profile 3 during the May-June 1974 storm sequence is shown in Figure E-3. The final scarp for this storm sequence is now degraded and vegetated but still visible in a photograph of Profile 4 taken on 30 June 2007 (Figure E-4).

Figure E-3: Bengello Beach (Profile 3 Looking North) 25 May 1974 - Further erosion occurred in early June 1974 after this photograph was taken (McLean et al., 2010)

Figure E-4: Bengello Beach (Profile 4 Looking North) 30 June 2007 after the "Pasha Bulker" Storm - left arrow indicates 1974-76 scarp, right indicates 1996-98 scarp (McLean et al., 2010)

E.3 Synthetic Design Storms

E.3.1 Design Offshore Wave Conditions

Shand et al. (2011) developed deepwater synthetic design storms, including a timeseries of significant wave height and peak spectral period, for a number of locations on the NSW coast, including Eden, south of Batemans Bay. In this study, the wave period and duration of the Eden offshore design storm has been adopted, however wave statistics from the Batemans Bay wave buoy have been used (generally 5% - 15% smaller than the wave climate at Eden). Note that the 100 year ARI offshore wave statistics vary with incident wave direction resulting in the development of three (3) offshore design storms for the 100 year ARI storm event (Figure E-5). The storm with the highest wave heights was applied for east-south-east, south-east and south-south-east directions. The storm with the lowest wave heights was utilised from the north-east and east-north-east directions. An intermediate storm was used for waves with incident directions of east and south.

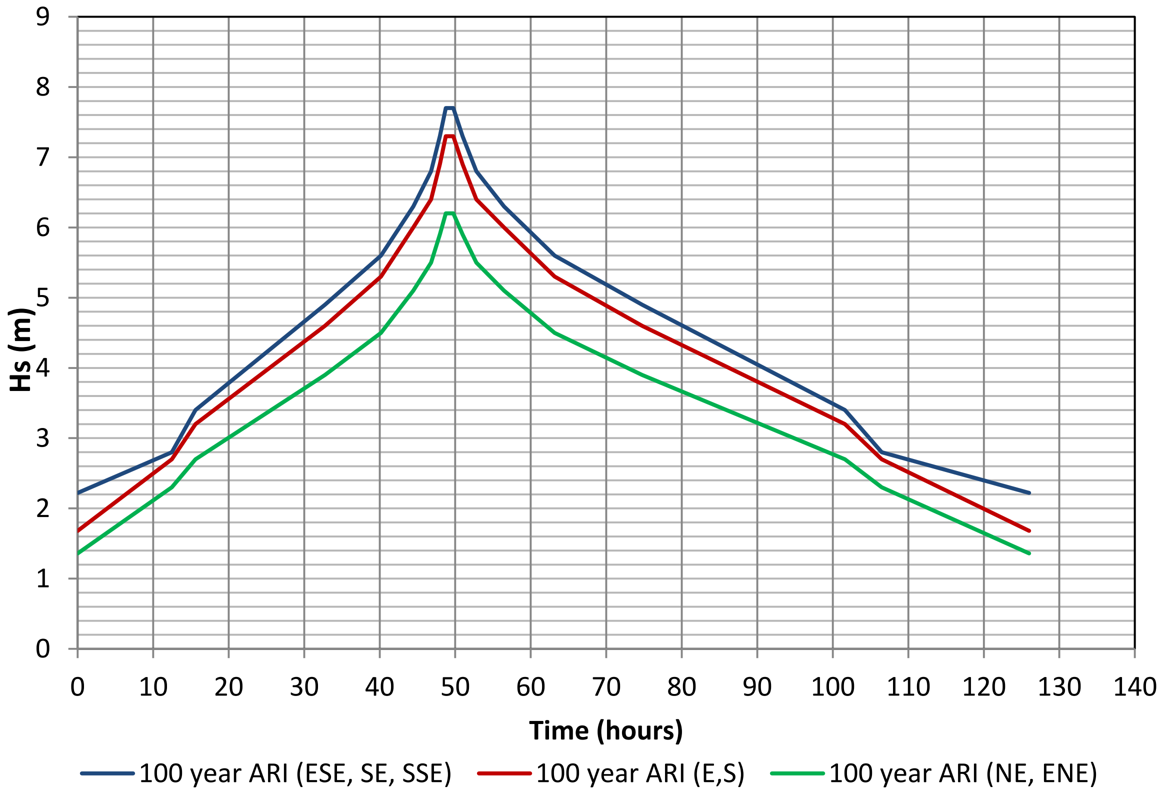


Figure E-5: Offshore design wave conditions for Batemans Bay

E.3.1.1 Storm Clustering

The worst case erosion events experienced by a beach are generally caused by the clustering of large storm events. Since beach recovery occurs over a much longer time frame than storm erosion, when the time between storms is sufficiently small, the beach is unable to recover to its accreted state. Major historical erosion events in NSW, such as the storms of 1974 and 1986, have been a result of multiple storms over a short (several months) period. To account for the effects of storm clustering, WRL has adopted a methodology of running two sequential 100 year ARI storms.

Thom and Hall (1991) showed that the timescales at which the beach recovery takes place is sufficiently slow, of the order of a week to several months, that the beach response to multiple erosion events would be relatively insensitive to the time gap between the storms. Therefore, for the purpose of SBEACH erosion modelling, the time gap between the storm is considered inconsequential.

E.3.2 Nearshore Design Waves

A SWAN model was developed to transform the offshore wave heights to local, nearshore waves (see Appendix D for more details on the SWAN modelling undertaken). Two (2) sequential (clustered) 100 year ARI storms were modelled with SWAN for seven (7) incident wave directions. The wave heights and directions at the model boundaries of the coarse grid were manually adjusted to ensure that the target wave conditions were reproduced at the Batemans Bay wave buoy location.

The local wave heights resulting from the seven (7) offshore design storm directions were extracted from the SWAN model at each transect where waves were beginning to break (1% of waves were broken). Using the output of the SWAN model, a single nearshore synthetic design storm was developed for the most critical (design) wave direction for each transect, using the same duration as the offshore synthetic storm. An example of the nearshore transformation of the waves at Bengello Profile 3 is shown in Figure E-6. The nearshore synthetic design storm extracted from the SWAN wave model at each transect became the input to the SBEACH erosion model.

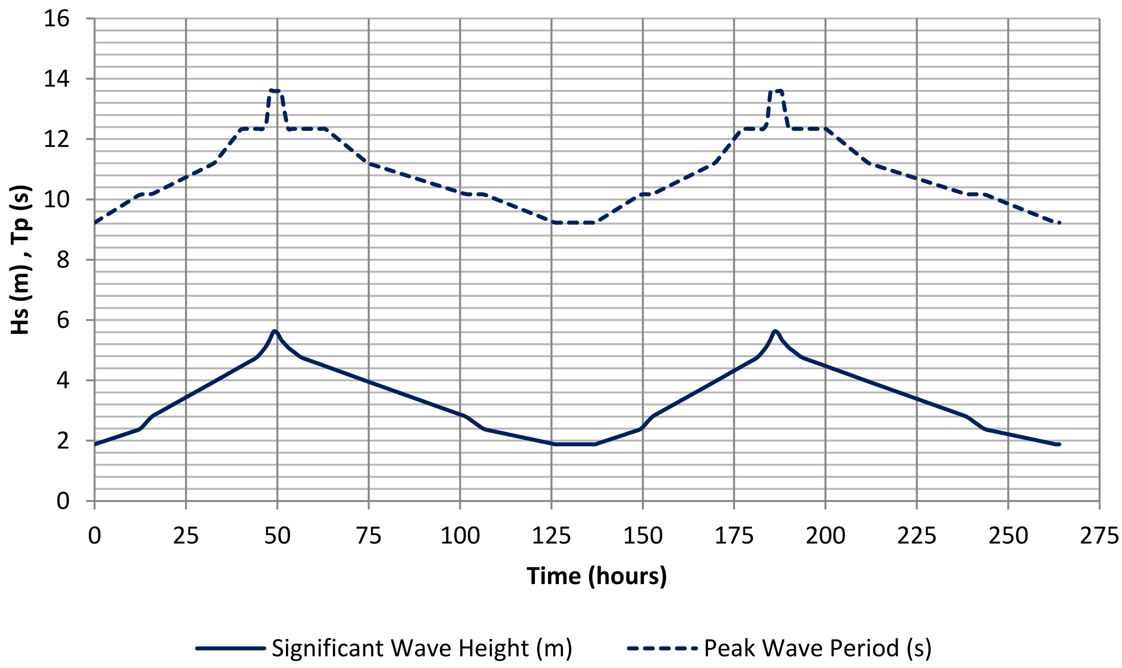


Figure E-6: Example of nearshore synthetic design storm used at Bengello Profile 3

E.3.3 Design Water Levels

Ocean water levels consist of (predictable) tides which are forced by the sun and moon (astronomical tides), and a tidal anomaly. The largest positive anomalies are associated with major storms and are driven by barometric setup (associated with low barometric pressure) and coastal wind setup, which are often combined as "storm surge". Water levels within the surf zone are also subject to wave setup, although this is modelled within the SBEACH package and is not required to be in the input water levels.

For storm erosion modelling purposes, a spring tide timeseries was generated (based on tidal constituents for Princess Jetty) using harmonic analysis with a peak water level of 0.82 m AHD (between 14/06/2011 – 20/06/2011). Extreme values analysis has been undertaken at Batemans Bay and defined the 1% AEP (100 year ARI) water level offshore to be 1.43 m AHD. This level includes both tide and storm surge, implying a maximum storm surge of 0.51 m must be applied at the peak of the storm to meet the required water level. It would be overly conservative to apply the maximum storm surge over the entire modelling period, so to better model the storm, the surge is allowed to increase linearly from nil to the maximum level and back to nil over the course of the storm. The resulting water levels for locations outside of Batemans Bay are shown in Figure E-7.

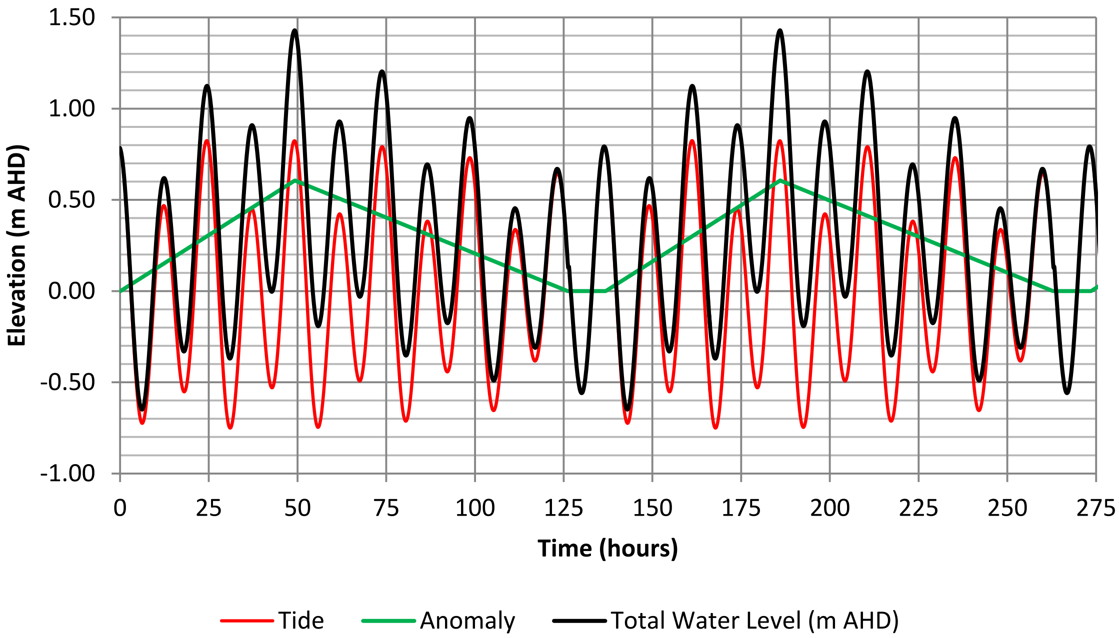


Figure E-7: Water levels used for SBEACH modelling at locations outside of Batemans Bay

For the beaches inside Batemans Bay (Surfside Beach, Long Beach and Maloneys Beach), the shallow bathymetry provides conditions that allow even higher water level conditions, due to the increase in water levels due to wind setup and inland flood events. The calculation of wind setup and flood levels is explained extensively in Section 3.3.3, however, the maximum water level at each location is presented in Table E-2. These water levels were achieved by adjusting the maximum storm surge level at these locations.

Table E-2: Summary of Maximum Water Levels for 100 year ARI Event used for SBEACH Erosion Modelling

Location	Maximum Water Level (m AHD)
Outside Batemans Bay	1.43
Maloneys Beach	1.54
Long Beach (Western End)	1.61
Long Beach (Central and Eastern End)	1.60
Surfside Beach (East) (Northern End)	1.75
Surfside Beach (East) (Southern End)	1.74

Figure E-8 to Figure E-22 show the local wave height, wave period and water level used for SBEACH modelling at each location.

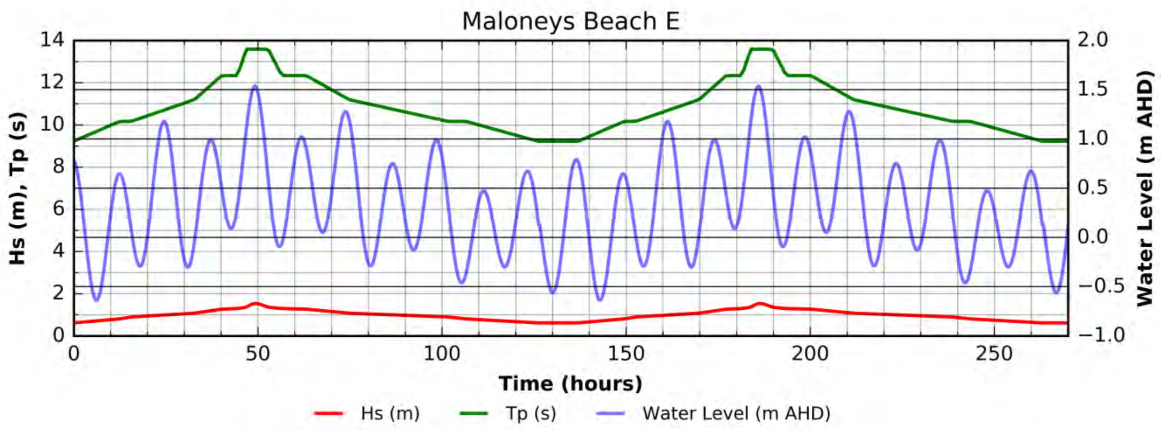


Figure E-8: Wave Height, Water Level and Peak Period for Maloneys Beach East

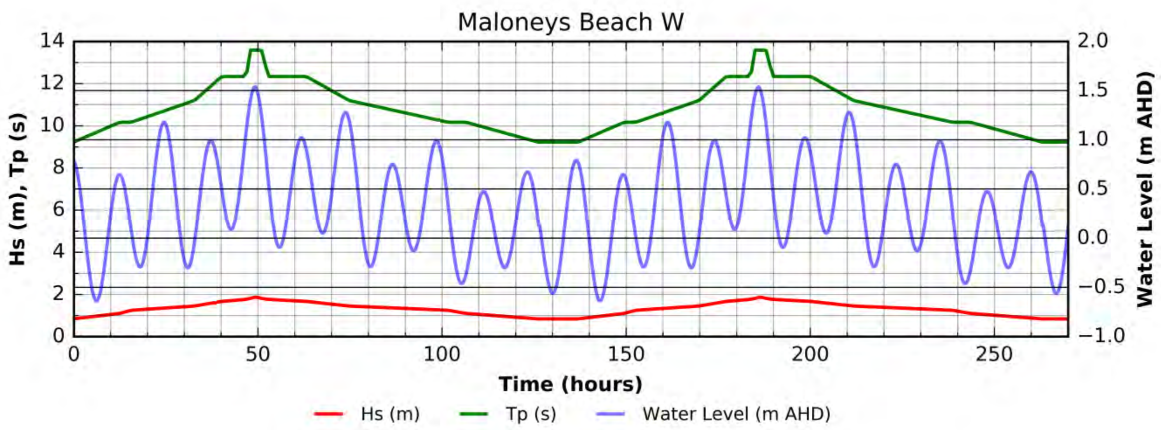


Figure E-9: Wave Height, Water Level and Peak Period for Maloneys Beach West

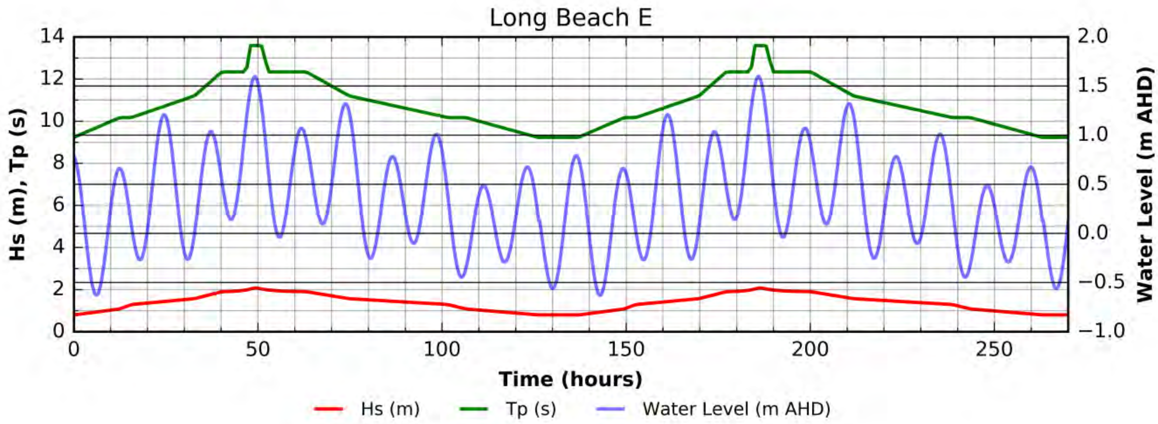


Figure E-10: Wave Height, Water Level and Peak Period for Long Beach East

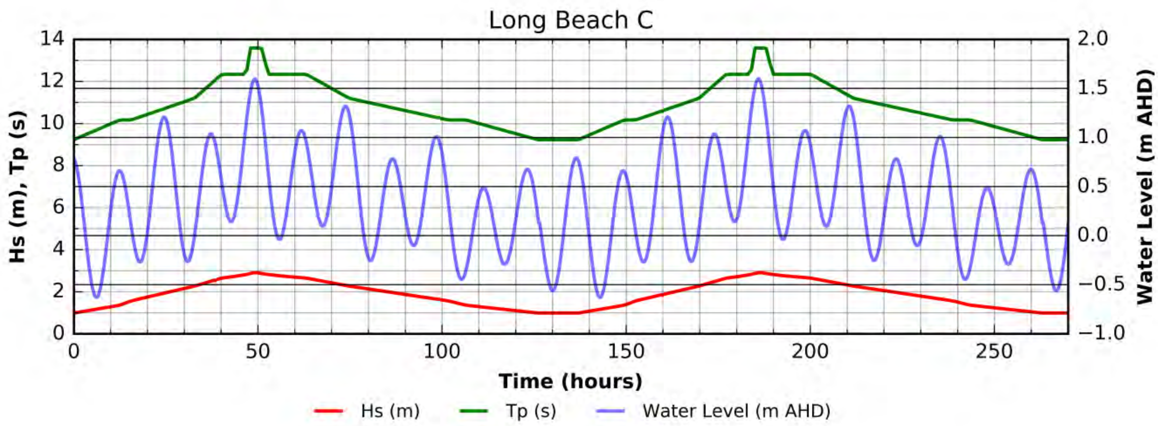


Figure E-11: Wave Height, Water Level and Peak Period for Long Beach Central

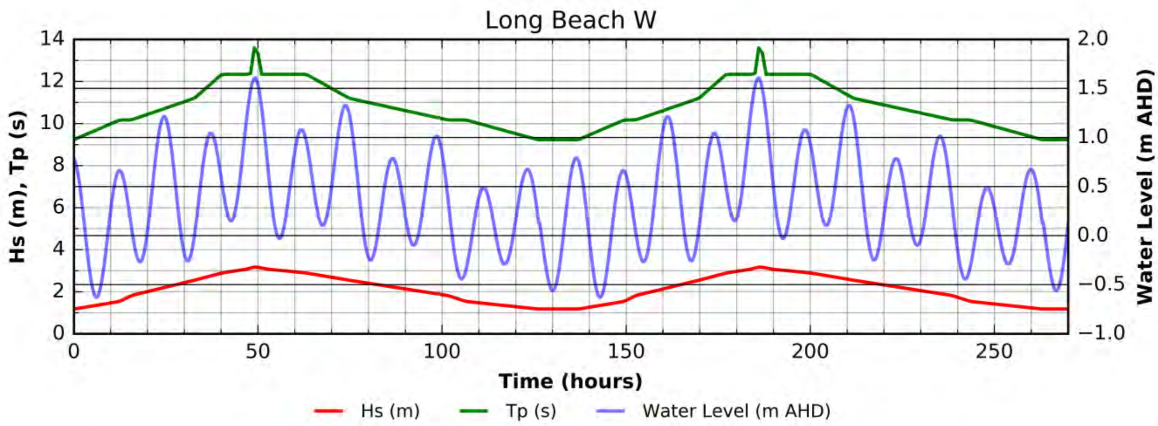


Figure E-12: Wave Height, Water Level and Peak Period for Long Beach West

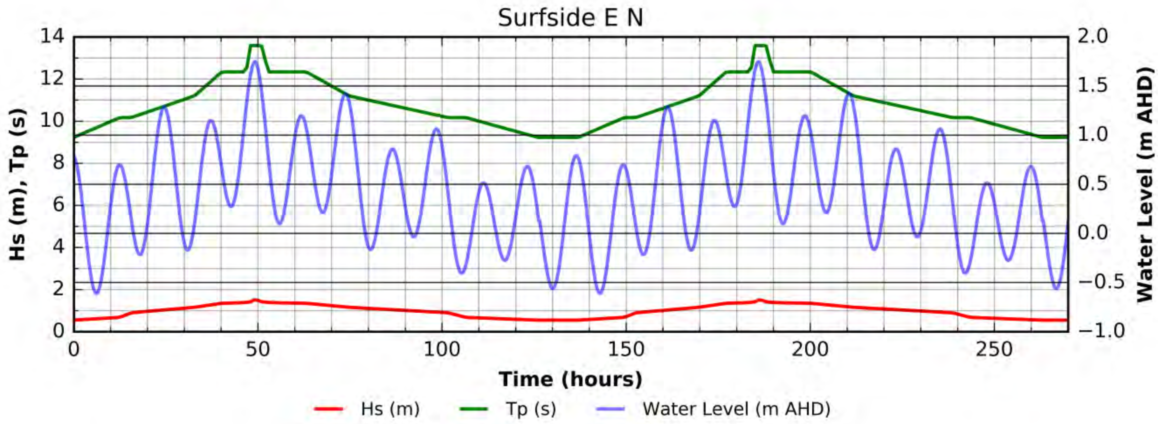


Figure E-13: Wave Height, Water Level and Peak Period for Surfside Beach East North

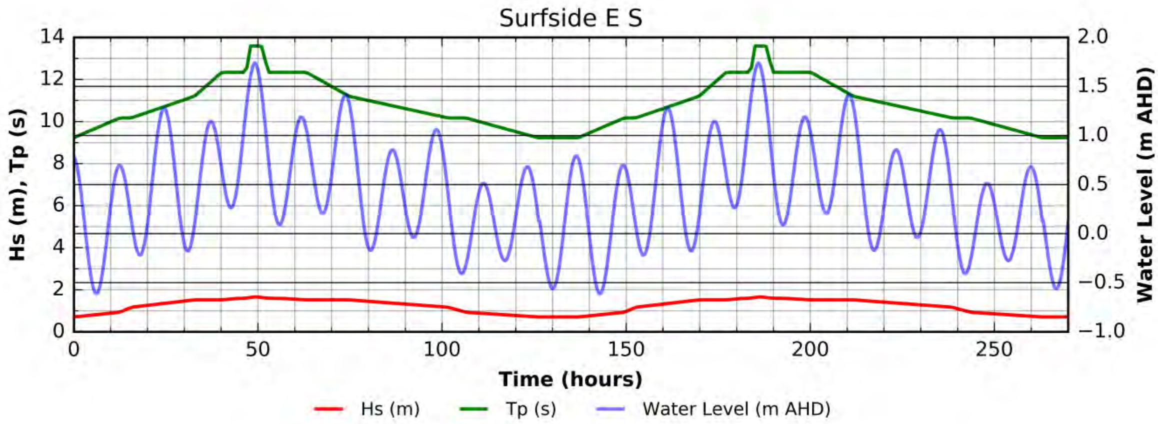


Figure E-14: Wave Height, Water Level and Peak Period for Surfside Beach East South

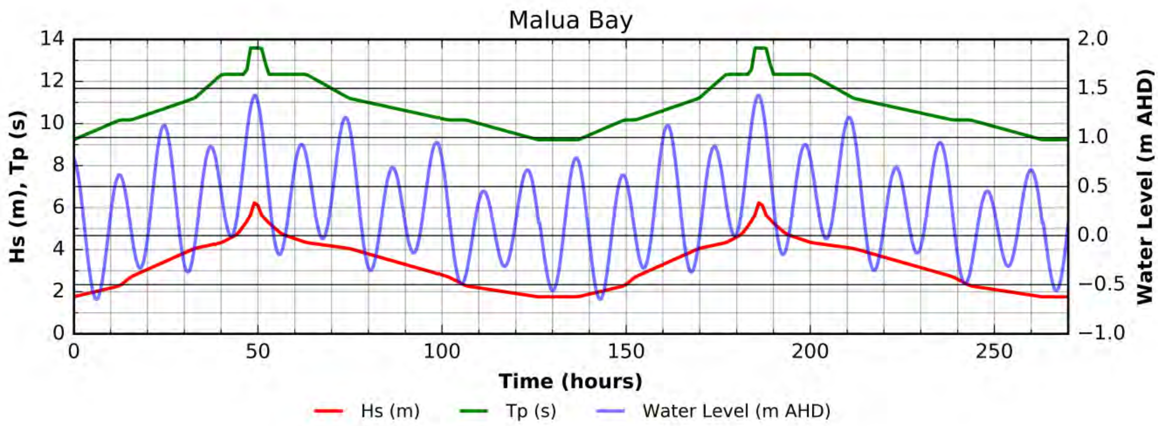


Figure E-15: Wave Height, Water Level and Peak Period for Malua Bay

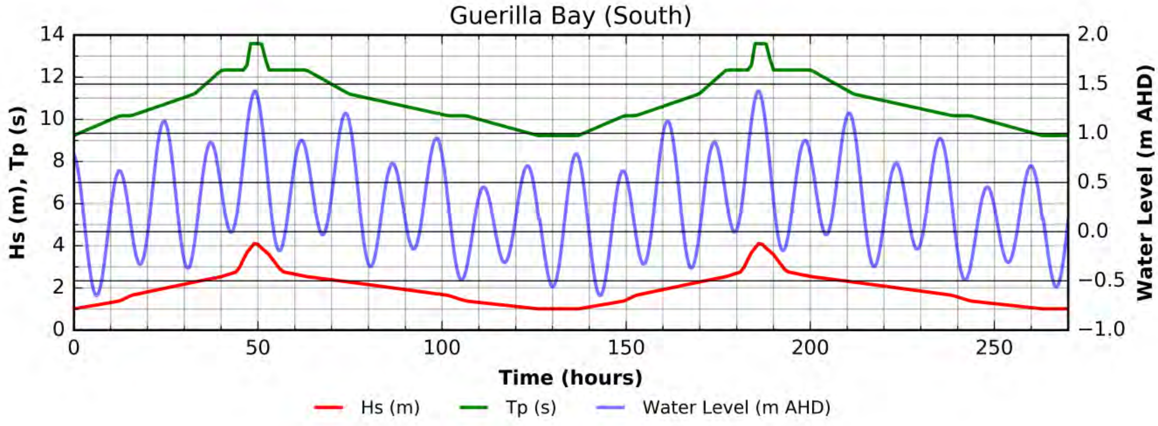


Figure E-16: Wave Height, Water Level and Peak Period for Guerilla Bay (South)

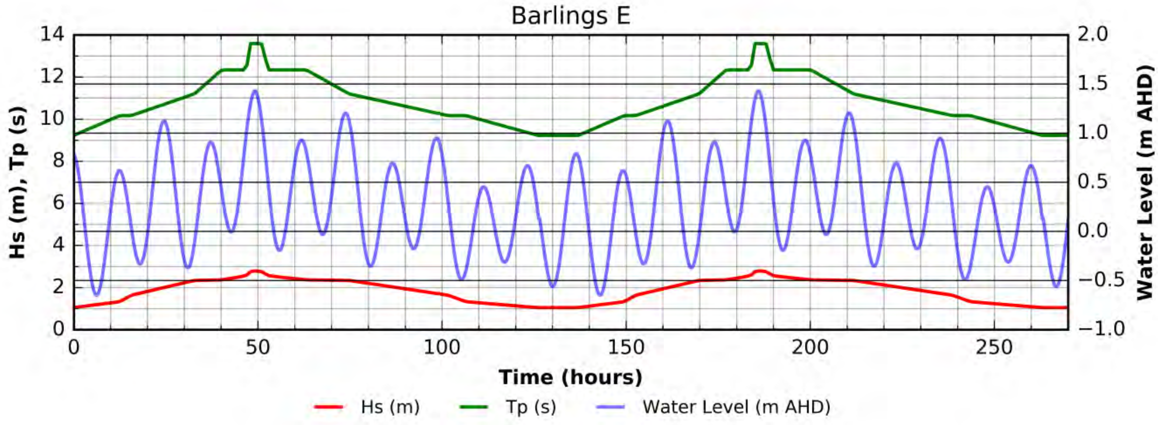


Figure E-17: Wave Height, Water Level and Peak Period for Barlings Beach East

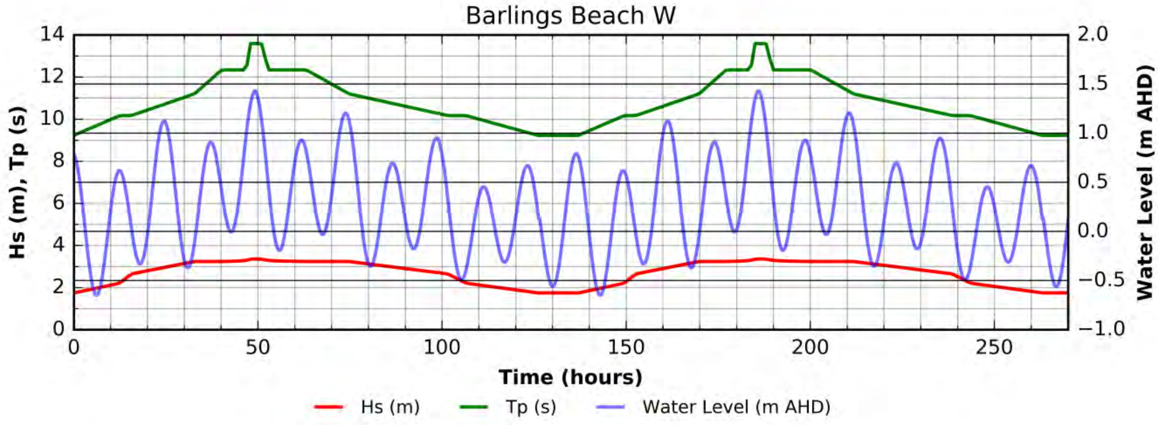


Figure E-18: Wave Height, Water Level and Peak Period for Barlings Beach West

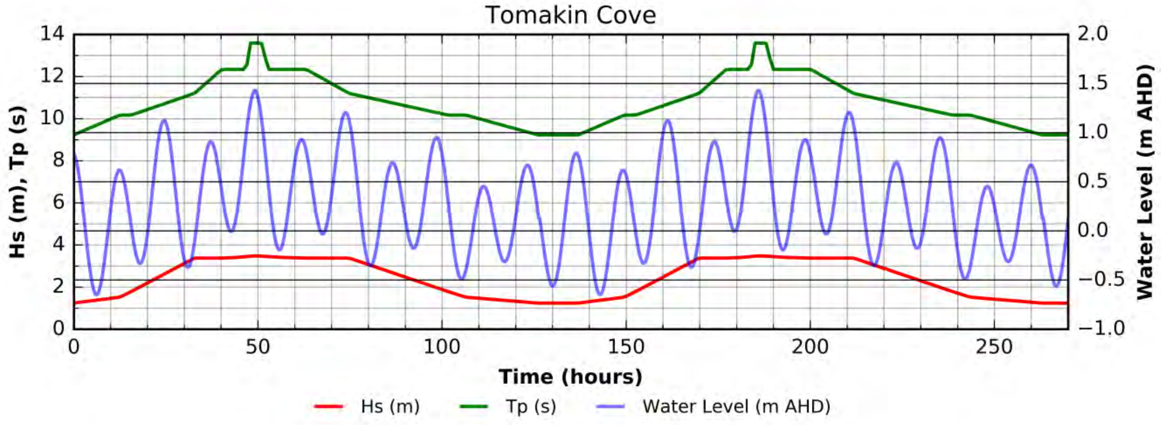


Figure E-19: Wave Height, Water Level and Peak Period for Tomakin Cove

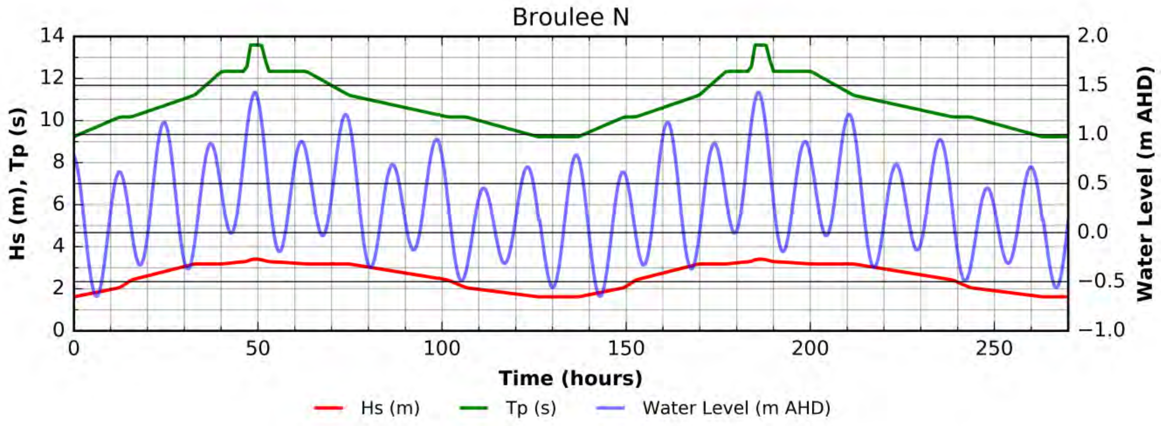


Figure E-20: Wave Height, Water Level and Peak Period for Broulee Beach North

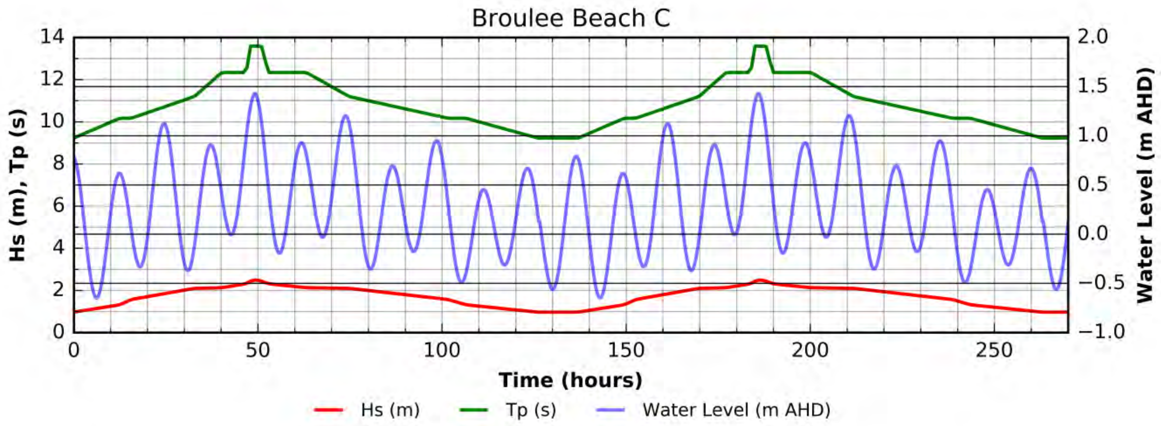


Figure E-21: Wave Height, Water Level and Peak Period for Broulee Beach Central

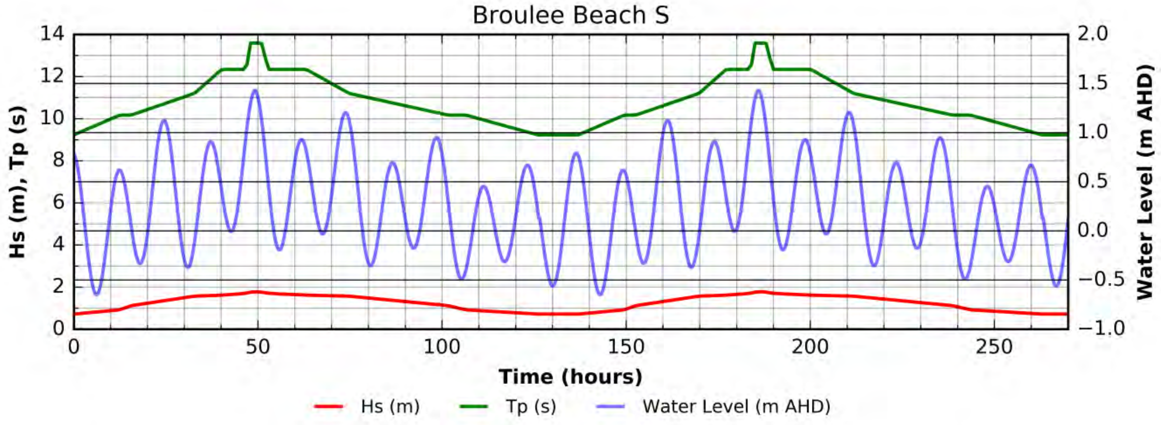


Figure E-22: Wave Height, Water Level and Peak Period for Broulee Beach South

E.3.4 Phasing of Extreme Ocean Water Levels and Design Wave Conditions

WRL, in conjunction with NSW OEH (formerly NSW DECCW) completed a detailed joint probability analysis of significant wave height and tidal residual for Sydney. The analysis showed that for design where both tidal residual and wave height are of interest, their occurrence cannot be assumed to be independent and the joint occurrence of extreme events should be considered. At locations where there is a lack of sufficient data, marginal extremes should be combined assuming complete dependence of the variables (Shand et al., 2012). Since no joint probability assessment has been undertaken for Eurobodalla local government area, complete dependence of extreme water levels and wave heights has been assumed for the 1% AEP (100 ARI) storms for the purpose of SBEACH erosion modelling.

E.4 Calibration at Bengello Beach

In calibrating the SBEACH model, the aim is to reproduce a surveyed change in beach profile in response to known climatic conditions. In the absence of wave and water level conditions at Bengello Beach, a 100 year ARI synthetic storm was developed for each of the four (4) profiles for use in calibration. While the exact recurrence interval of the 1974 storm period is not known, Figure E-2 shows the erosion event caused by that storm was significantly greater than observed erosion at any other period during 45 years of monitoring. Without further monitoring, it is considered appropriate to assume this erosion event was approximately equivalent to a 100 year ARI erosion event at Bengello Beach.

Since the methodology was developed so that it could be used at every other beach location, the elevations along the initial profile was extracted at a 2 m spacing from the available survey data. At Bengello Beach, there were profile surveys for the beach face (survey taken 17/11/2014) and a hydrosurvey of the nearshore bathymetry (survey taken on 18/11/2014). This data was supplemented with the 2011 LIDAR and AHS bathymetry as required to make a measured profile that was sufficiently long to be appropriate for SBEACH modelling.

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 maximum volume at Profile 2 coincided with a rip-head embayment (rip-heads are not included in SBEACH). This resulted in two sets of model sediment transport rate (k) coefficients that were used to give a range of values to represent the 100 year ARI storm demand elsewhere in the study area.

Table E-3 summarises the calibrated SBEACH parameters under the two calibration cases. The final eroded profiles for each Bengello Beach profile are provided in Figure E-23, Figure E-24, Figure E-25 and Figure E-26. By decreasing the sediment transport rate coefficient from 2.5 x 10-6 to 1.5 x 10-6 m4/N, the erosion of the dune face significantly decreases, allowing the much lower average storm demand figure to be achieved. Table E-4 shows the SBEACH modelled storm demands at each of the four (4) profiles at Bengello Beach. Using the parameters described above, the SBEACH model is considered to represent the observed erosion figures well, although it is noted that maximum single profile erosion occurred at Profile 3 (based on 2014 survey data) rather than at Profile 2. The two sets of model parameters were then used at each subsequent beach to obtain an upper and lower limit of erosion expected at each location.

Table E-3: Summary of Calibrated SBEACH Parameters

Coefficient/ Variable (notation used in model)	Value (calibrated to average erosion)	Value (calibrated to maximum erosion)	Brief Description
DXC	Variable (1 and 2 m)	Variable (1 and 2 m)	Model grid size
DT	20 minutes	20 minutes	Time step
K	1.5×10^{-6} m^4/N	2.5×10^{-6} m^4/N	Sediment transport rate coefficient
KB	0.005	0.005	Overwash transport parameter
EPS	0.002 m^2/s	0.002 m^2/s	Slope dependent transport rate coefficient
LAMM	0.5	0.5	Transport rate decay coefficient multiplier
TEMPC	20°C	20°C	Temperature
ISEED	4567	4567	Seed for random number generation
RPERC	20%	20%	Random variation in wave heights
DFS	0.3	0.3	Landward surfzone depth
D50*	0.33	0.33	Effective median grainsize in the surfzone
BMAX	30°	30°	Avalanching angle

*D50 varied across other beaches depending on the observed grainsize

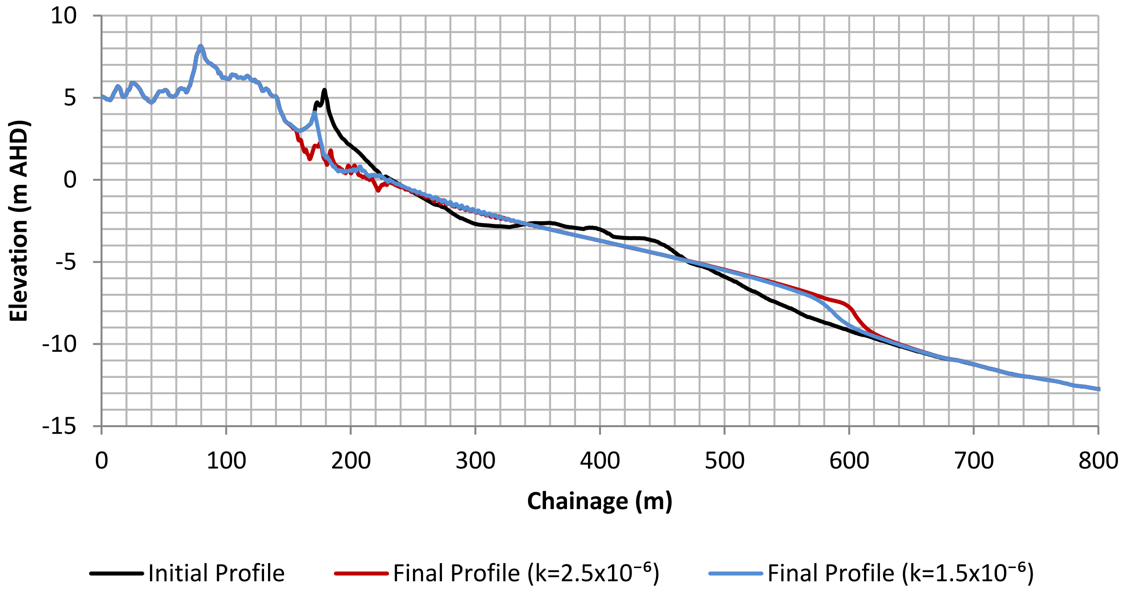


Figure E-23: SBEACH results at Bengello Beach Profile 1

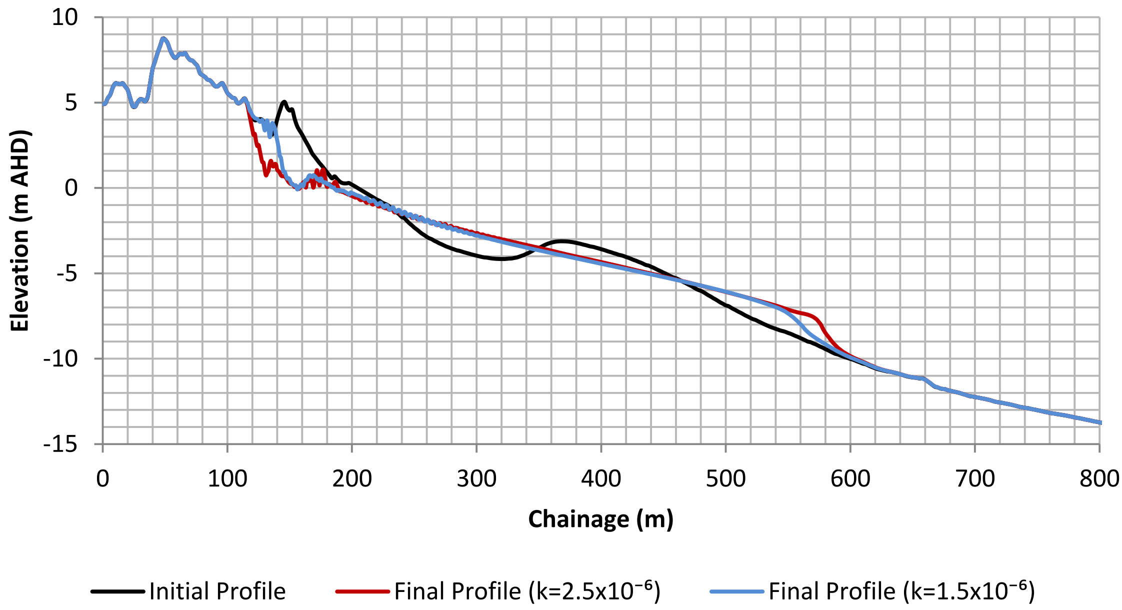


Figure E-24: SBEACH results at Bengello Beach Profile 2

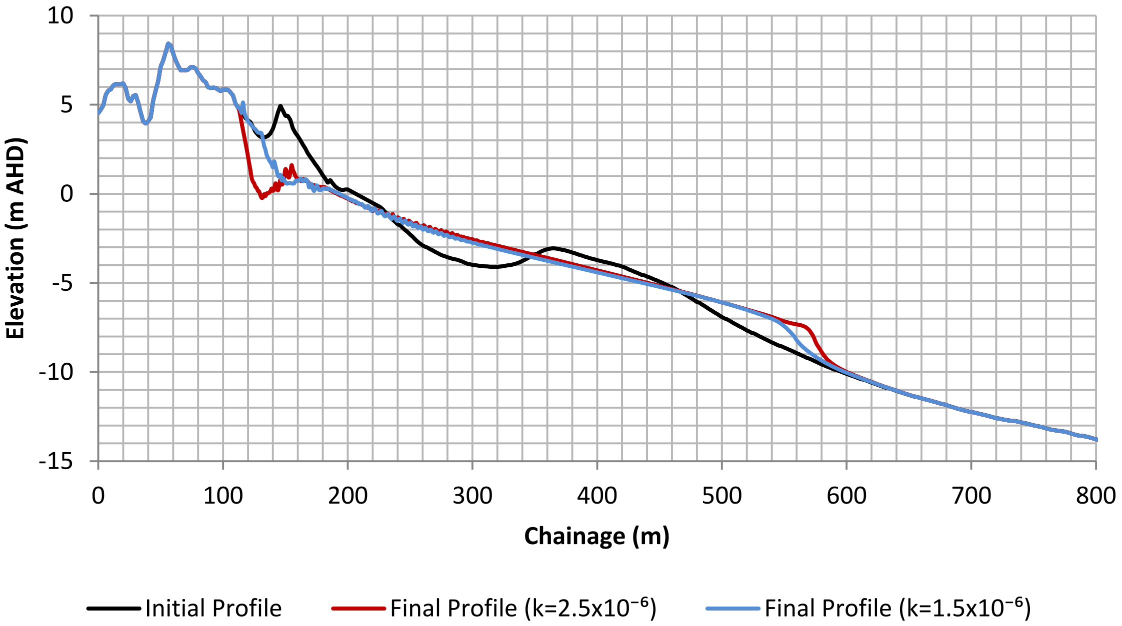


Figure E-25: SBEACH results at Bengello Beach Profile 3

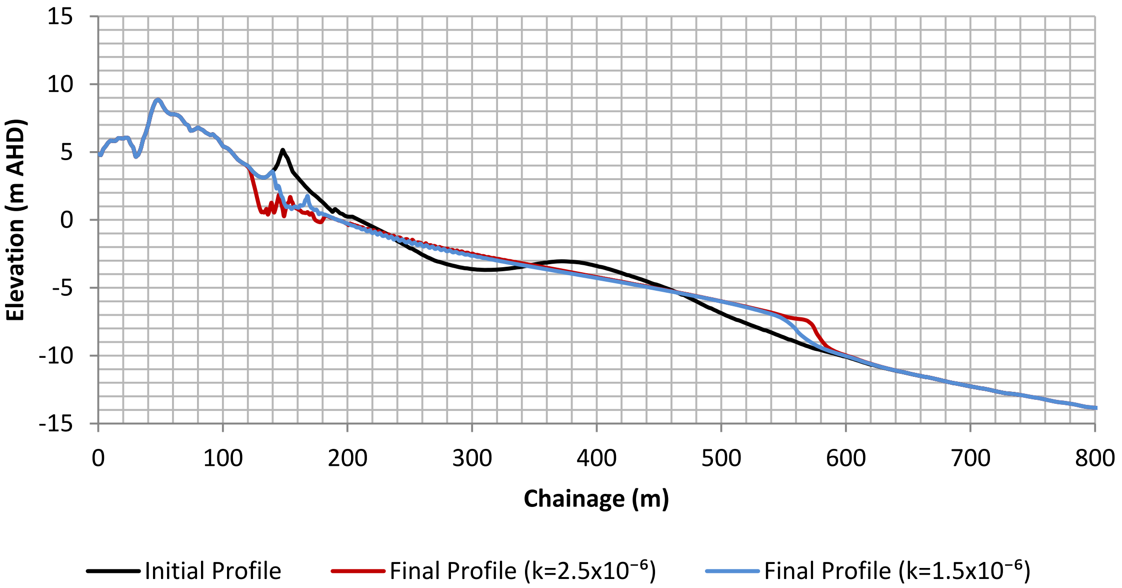


Figure E-26: SBEACH results at Bengello Beach Profile 4

Table E-4: Summary of Calibrated Storm Demands at Bengello Beach

Profile	Storm Demand $k=1.5x10^{-6}$ m^4/N (m^3/m above 0 m AHD)	Storm Demand $k=2.5x10^{-6}$ m^4/N (m^3/m above 0 m AHD)
Bengello Beach 1	86	125
Bengello Beach 2	109	155
Bengello Beach 3	113	174
Bengello Beach 4	88	146
Maximum	113	174
Average	99	150

E.5 SBEACH Modelling Locations

SBEACH erosion modelling was undertaken at nine (9) beaches in the study area, as shown in Figure E-27 and Figure E-28. Where the beaches were long enough that the wave climate would vary significantly along the beach, multiple representative transects were used (15 total transects). WRL has previously collected and analysed sediment samples collected at each location to determine sediment size, and this is summarised in Table E-5. At Sunshine Bay, a distinct bimodal distribution of sediment size was observed and both grainsizes are provided. This type of sediment distribution is also observed at Caseys Beach (adjacent beach to the north) as discussed in NSW PWD (1987).

Figure E-27: SBEACH erosion profile inner Batemans Bay

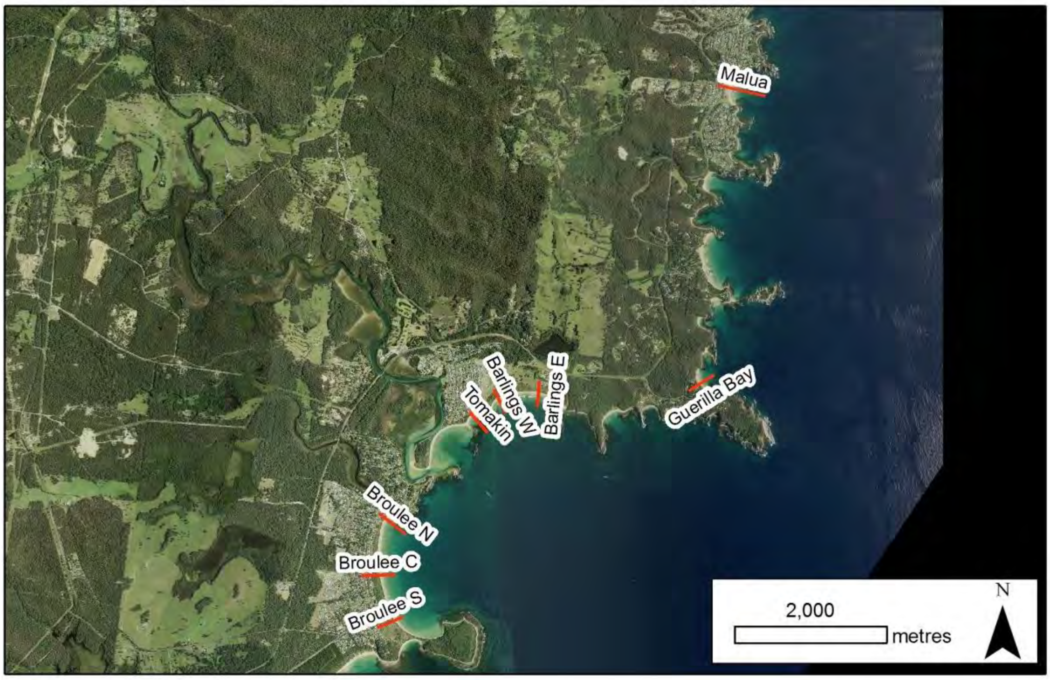


Figure E-28: SBEACH erosion profiles southern area

Table E-5: Summary of Grain Size at Each Location

Beach	Section	D_{50} (mm)
Maloneys Beach	East	0.21
	West	0.21
Long Beach	East	0.24
	Central	0.24
	West	0.24
Surfside Beach (east)	North	0.25
	South	0.25
Surfside Beach (west)	Central	0.21
Sunshine Bay	Central	0.21/1.01*
Malua Bay	Central	0.34
Guerilla Bay (south)	Central	0.29
Barlings Beach	East	0.28
	West	0.32
Tomakin Cove	Central	0.19
Broulee Beach	North	0.21
	Central	0.21
	South	0.21

* Sediment at Sunshine Bay has a bimodal distribution as discussed in Sections 2.2 and 5.3.2.

At each of the profile locations indicated in Figure E-27 and Figure E-28, profile data was extracted from the best available topographic and bathymetric surveys as shown in Table E-6.

Table E-6: Date of Available Surveys for Study Area

Beach	Topographic Survey Date	Bathymetric Survey Date
Maloneys Beach	23/7/2014	24/7/2014
Long Beach	29/7/2014	29/7/2014
Surfside Beach (east)	30/7/2014	30/7/2014
Sunshine Bay	24/11/2016	-
Malua Bay	30/7/2014	30/7/2014
Guerilla Bay	-	29/7/2014
Barlings Beach	31/3/2015	1/4/2015
Tomakin Cove	31/3/2015	1/4/2015
Broulee Beach	31/3/2015	1/4/2015
Batemans Bay	-	8/7/2015

Where no site specific surveys were available, or data was required beyond the extents of the surveyed areas, the most recent LIDAR (2011 outside of Batemans Bay and 2005 inside the bay) and the AHS bathymetry dataset was used to supplement the surveys.

E.6 Results of SBEACH Modelling

Table E-7 summarises the modelled storm demand for the 100 year ARI event at each of the beaches. No results are presented for Sunshine Bay, as the bimodal nature of the sediment distribution makes it inappropriate for SBEACH modelling. Additionally, Surfside Beach (west) has also not been modelled as the strongly refractive wave conditions mean that erosion processes are not cross shore.

Table E-7: Results of SBEACH Modelling for the 100 Year ARI Storm

Beach	Profile	**Storm Demand Average Erosion** (m^3/m above 0 m AHD)	**Storm Demand Maximum Erosion** (m^3/m above 0 m AHD)
Maloneys Beach	East	73	96
	West	113	156
Long Beach	East	68	87
	Central	92	132
	West	105	137
Surfside Beach (east)	North	43	54
	South	46	55
Surfside Beach (west)	-	n/a	n/a
Sunshine Bay	-	n/a	n/a
Malua Bay	-	115	153
Guerilla Bay	-	103	153
Barlings Beach	East	50	64
	West	60	106
Tomakin Cove	-	84	132
Broulee Beach	North	47	89
	Central	34	56
	South	39	52

The SBEACH erosion modelling has included measurements of the local bathymetry and sand grain size and modelled nearshore wave conditions specific to each transect. As discussed in Section E.4, the model was calibrated at an open coast location (Bengello Beach) and the same model parameters were applied at lower energy sites. Although based on a limited dataset, Leadon (2015) has suggested that the model sediment transport rate (k) coefficient is inversely proportional to beach slope. Since constant k values were used in this study 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). However, the consensus values for 100 year ARI storm demand adopted by the expert panel considered multiple factors and at these two beaches are significantly lower than the SBEACH predictions.

Appendix F: Assessment of Bruun Factor

F.1 Preamble

The most commonly applied and well known model for beach response to SLR is that of Bruun (1962, 1988) which assumes that an elevation in sea level will result in a recession of the coastline. This model assumes that as the sea level is raised, the equilibrium profile is moved upward and landward, conserving mass and original shape, based on the concept that the existing beach profile is in equilibrium with the incident wave climate and existing average water level (shown in Figure F-1). A recession rate can be estimated using the Bruun Rule (Bruun, 1962, 1988) as the rate of sea level rise divided by the average slope of the active beach profile. Bruun's Rule is expressed as:

$$R = \frac{rX}{h+d_c} \qquad (F.1)$$

where R is horizontal recession (m)
r is sea level rise (m)
X is the horizontal distance between h and d_c
h is active dune/berm height (m)
d_c is profile closure depth (m, expressed as a positive number)

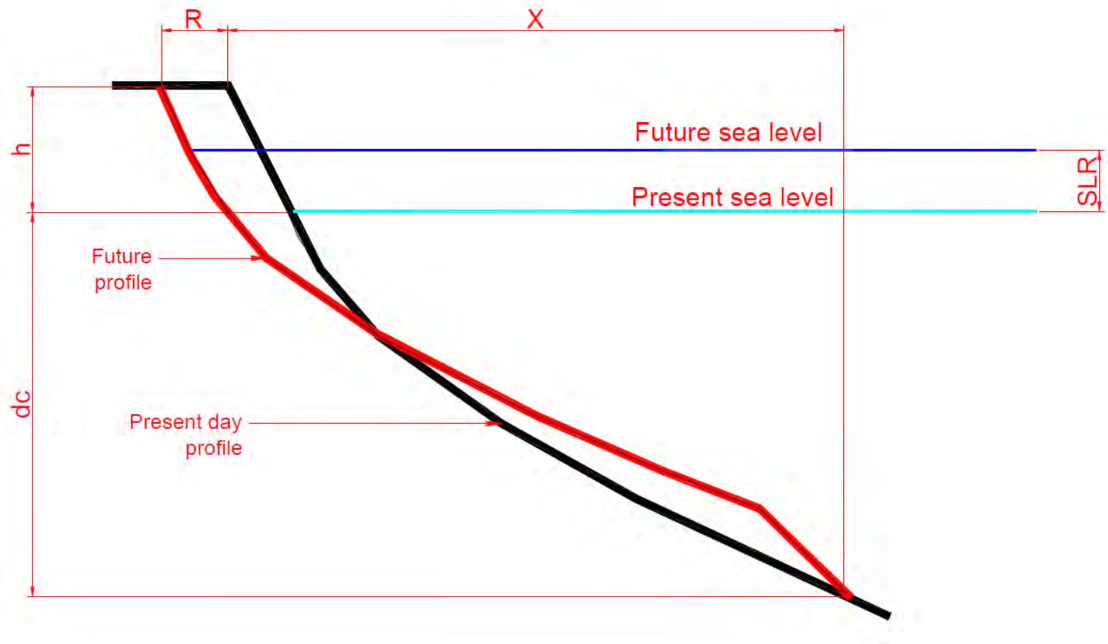


Figure F-1: Illustration of Bruun Rule

Typically, a Bruun factor, which incorporates profile slope at a particular site and thus gives horizontal recession distance as a function of sea level rise is used to calculate recession due to sea level rise at a given location. This appendix summarises the methodology to estimate the depth of closure, and therefore Bruun factor, for each of the beaches in the study area.

F.2 Depth of Closure

The depth of closure is defined as the depth corresponding to the offshore limit of active sediment transport. Its determination is subject to large uncertainty, and is commonly assessed using empirical methods or relying on site specific geology or sedimentology methods.

The primary method for establishing depth of closure in NSW coastal hazard assessments is generally the Hallermeier (1981) inner depth of closure. This method, four other alternative techniques and previously published estimates were collated for consideration by the expert panel. It should be emphasised that the purpose of estimating the depth of closure in this study was to provide an input to the Bruun Rule.

This section summarises the various methodologies used to assess the depth of closure at sites across ESC.

F.2.1 Hallermeier Depth of Closure

Hallermeier (1981) stated that there were three simplified zones of sediment transport: the very active littoral zone closest to the shore, a buffer zone in which the bed is impacted by surface waves but to a lesser extent and an outer zone where surface waves have a negligible effect on the profile bed. Therefore two depths of closures can be established, the inner depth of closure indicates the end of the highly active littoral zone and the outer depth of closure, seaward of which surface waves have little effect on littoral transport. Hallermeier (1981) states that the inner depth of closure on a sandy beach as shown in Equation F.2 and Hallermeier (1983) expressed the outer depth of closure as per Equation F.3.

$$d_l = 2.28H_{s,t} - 68.5\left(\frac{H_{s,t}}{gT_s{}^2}\right) \qquad \text{(F.2)}$$

where d_l = inner depth of closure (m) below mean low water (MLW) level
$H_{s,t}$ = wave height exceeded 12 hours a year (m)
T_s = wave period corresponding with $H_{s,t}$ (s)

$$d_i = 0.018H_mT_m\sqrt{\frac{g}{D_{50}(s-1)}} \qquad \text{(F.3)}$$

where d_i = outer depth of closure (m) below mean low water (MLW) level
H_m = annual median significant wave height (m)
T_m = wave period corresponding with H_m (s)
s = specific gravity of sand grains, taken as 2.65
D_{50} = median grain size

For computation of the Hallermeier outer depth of closure, distance from the dune to the depth of closure was limited to 1500 m. Where the computed depth of closure exceeded this point, the depth 1500 m offshore from the dune was adopted.

F.2.2 Equilibrium Profile

Bruun (1954) (and later Dean, 1977) proposed the concept of beach profiles, such that the relationship between cross shore distance and depth could be related using equation F.4.

$$h = Ax^{\frac{2}{3}} \qquad \text{(F.4)}$$

where h = depth (m)
x = cross shore distance (m)
A = sediment scale parameter (-)

Since the sediment scale parameter, A, is dependent on the median grainsize, this equilibrium profile can be rapidly generated for each site. At the inner sections of Batemans Bay, the bathymetry is relatively flat and shallow and sediment movement is driven not only by wave forces, but through water movement from the Clyde River and Cullendulla Creek. Therefore, the outer depth of closure method of Hallermeier (1983) may not be appropriate at these locations. For this study, an alternate calculation of the depth of closure was to estimate the location where the observed profile begins to significantly deviate from the Dean equilibrium profile, as shown in Figure F-2. This methodology assumes that where the profile significantly deviates in shape and slope to the equilibrium profile, the sediment transport is no longer dominated by waves, and can therefore be considered the depth of closure for the purpose of using Bruun's rule. This approach to estimate depth of closure has previously been used within Batemans Bay (SMEC, 2010) and at other international locations (e.g. NASA, 2010). In the absence of repeat bathymetry surveys offshore of the Batemans Bay beaches, this alternative depth of closure estimate is considered instructive. However, it is acknowledged that the equilibrium concept assumes constant wave conditions and does not include the presence of bars. Furthermore, the point at which the observed profile deviates from the equilibrium profile may be influenced by the timing of the profile measurement with respect to erosion and accretion modes.

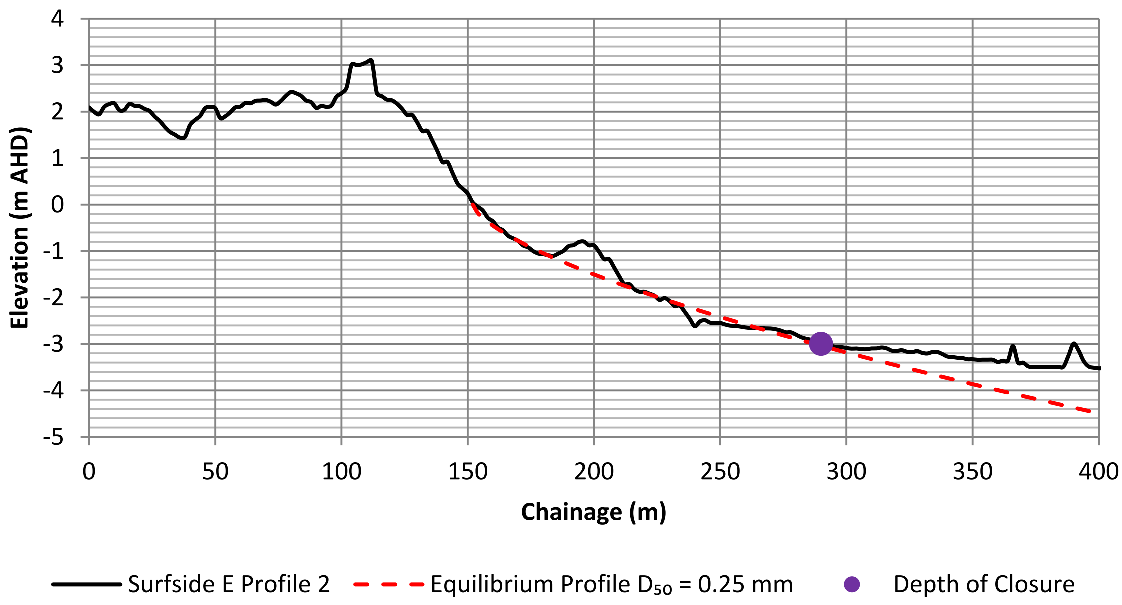


Figure F-2: An example of estimating the depth of closure using the equilibrium profile at Surfside Beach (East) profile 2

F.2.3 Site Specific Geology and Bathymetry

At a number of the beaches in this study, there are specific geological features and bathymetry that can be used to estimate the depth of closure. At these locations, there is an offshore reef which can be identified using aerial images, such as at Tomakin Cove in Figure F-3.

After identifying the location of the reef feature that indicates the position of the depth of closure, local bathymetric surveys were used to estimate the depth at this point. This methodology was used at all beaches which have an obvious reef feature and included Sunshine Bay, Malua Bay, Guerilla Bay and Tomakin Cove.

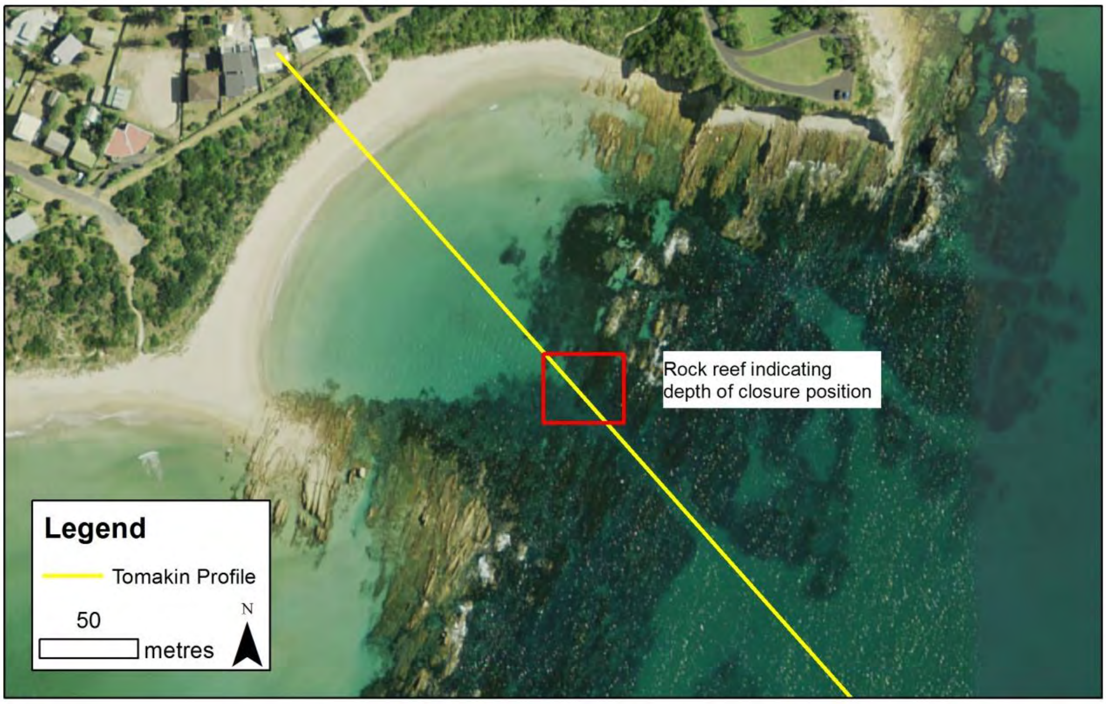


Figure F-3: Using the rock reef to identify the position of the depth of closure at Tomakin Cove

F.2.4 Wave breakpoint depth

Given that the depth of closure is the point at which sediment movement ceases to be driven by surface wave movement, it is conversely true that it is also approximately equal to the point where waves are no longer significantly influenced by the water depth. A simplistic approximation of this spatial position is the point at which waves first begin to break.

Using the SWAN model developed for the region (see Appendix D), the water depths at which 1% of the 100 year ARI waves were breaking were extracted at each location. This depth was then assumed as an alternate depth of closure at each location. WRL considers that closure depths estimated using this approach represent the lower limit (i.e. possibly unconservative) of sediment movement. That is, the adopted depth of closure should be at least the depth of 1% wave breaking.

F.3 Bruun Factor

The Bruun factor is calculated using Equation F.5 (refer also to Figure F-1) for a given dune location and depth of closure. This methodology was used considering the four (4) methodologies for depth of closure described above as they were appropriate and the resulting Bruun factors are collated in Table F-1. Note that sediment size was determined by mechanical sieving of samples collected at each location.

$$BF = \frac{h_D - h_c}{x_c - x_D} \qquad \text{(F.5)}$$

Where BF = Bruun factor
h_c = elevation of depth of closure (m AHD)
h_D = elevation of dune (m AHD)
x_c = relative cross shore chainage of depth of closure (m)
x_D = relative cross shore chainage of dune (m)

For reference, Table F-1 also states the previous estimates made in other studies, including NSW DLWC (1996), SMEC (2010), GBAC (2010) and BMT WBM (2009). Table F-2 summarises the distances used in the Bruun factor calculations.

As discussed in Section 4, Table F-1 (except for the last three columns) and Table F-2 were presented to each member of the expert panel. They were then asked for their preferred values for Bruun factor (minimum, maximum and mode) at each beach section on the basis of the presented information and their own experience on the Eurobodalla coast. The experts' independently preferred values were then blended into a consensus range shown in the last three columns of Table F-1.

Table F-1: Depth of Closure and Bruun Factor Estimates for the Study Area

Beach	Section	WRL D_{50} (mm)	Elevations (m AHD)						Bruun Factors (-)								
			Dune	Inner Depth of Closure	Outer Depth of Closure	Divergence from Equilibrium	Break-point Depth	Rock/ Reef Depth	Inner Depth of Closure	Outer Depth of Closure	Divergence from Equilibrium	Break-point Depth	Rock/ Reef Depth	Previous Estimates	Adopted Consensus Values		
															min	mode	max
Maloneys Beach	East	0.21	5.9	-2.7		-11.0	-2.9		10		59	10		50[1], 20-22[2]		**10**	
	West	0.21	6.3	-3.6		-11.0	-2.8		9		60	9					
Long Beach	East	0.24	3.4	-3.9		-8.2	-3.2		25		60	22		40[1], 20-22[2]	**15**	**20**	**50**
	Central	0.24	3.8	-4.2		-7.5	-4.4		16		56	17		-	**15**	**20**	**50**
	West	0.24	5.3	-5.7		-8.2	-6.0		18		52	19		40[1], 23-25[2]	**15**	**20**	**50**
Surfside Beach (East)	North	0.25	2.9	-3.0		-2.6	-2.3		31		25	23		25[1], 19-20[2]	**20**	**25**	**30**
	South	0.25	3.1	-3.3		-3.0	-2.3		36		29	23			**20**	**25**	**30**
Surfside Beach (West)	Central	0.21	1.9											20[4]	**15**	**20**	**30**
Sunshine Bay	Central	0.21/1.01	3.8	-6.7	-11.0		-7.0	-4.4	37	71		38	24	45-62[2]		**40**	
Malua Bay	Central	0.34	5.2	-8.0	-21.1		-12.0	-14.7	28	44		31	33	40-49[2]	**25**	**30**	**50**
Guerilla Bay (South)	Central	0.29	4.2	-5.0	-14.8		-7.5	-11.4	20	34		22	21	25-35[2]		**25**	
Barlings Beach	East	0.28	7.2	-3.7	-11.1		-3.1		17	52		16		70-85[2]		**50**	
	West	0.32	6.2	-6.7	-11.5*		-6.0		26	79*		22		85-95[2], 56[3]			
Tomakin Cove	Central	0.19	6.6	-6.9	-11.5*		-6.3	-2.8	24	74*		24	21	85-95[2], 40[3]	**20**	**25**	**60**
Broulee Beach	North	0.21	7.5	-6.4	-15.0*		-5.7		31	63*		28		65-75[2]	**25**	**30**	**65**
	Central	0.21	6.6	-4.3	-15.6		-3.9		30	62		29			**25**	**30**	**65**
	South	0.21	4.6	-3.5	-11.3		-2.4		32	53		19			**25**	**30**	**65**

* 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 at 1.5 km offshore

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

Table F-2: Distances for Bruun Factor Estimates for the Study Area

Beach	Section	Distances from Dune to ... (m)				
		Inner Depth of Closure	Outer Depth of Closure	Divergence from Equilibrium	Break-point Depth	Rock/Reef
Maloneys Beach	East	85		997	89	
	West	91		1037	79	
Long Beach	East	181		693	144	
	Central	130		638	139	
	West	195		700	211	
Surfside Beach (East)	North	179		136	118	
	South	231		176	123	
Surfside Beach (West)	Central					
Sunshine Bay	Central	387	1043		405	194
Malua Bay	Central	363	1167		542	647
Guerilla Bay (South)	Central	185	653		256	326
Barlings Beach	East	190	957		169	
	West	338	1500*		266	
Tomakin Cove	Central	322	1500*		317	197
Broulee Beach	North	432	1500*		366	
	Central	325	1375		302	
	South	260	837		133	

* 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 at 1.5 km offshore

Appendix G: Dune Stability Schema for Erosion Mapping

First pass, or "rule of thumb" erosion distance assessments are calculated by dividing the storm demand by the beach dune height. Nielsen et al. (1992) describes a more robust method, where a storm demand volume is converted to a horizontal distance by defining a zone of slope adjustment and a zone of reduced foundation capacity (Figure G-1).

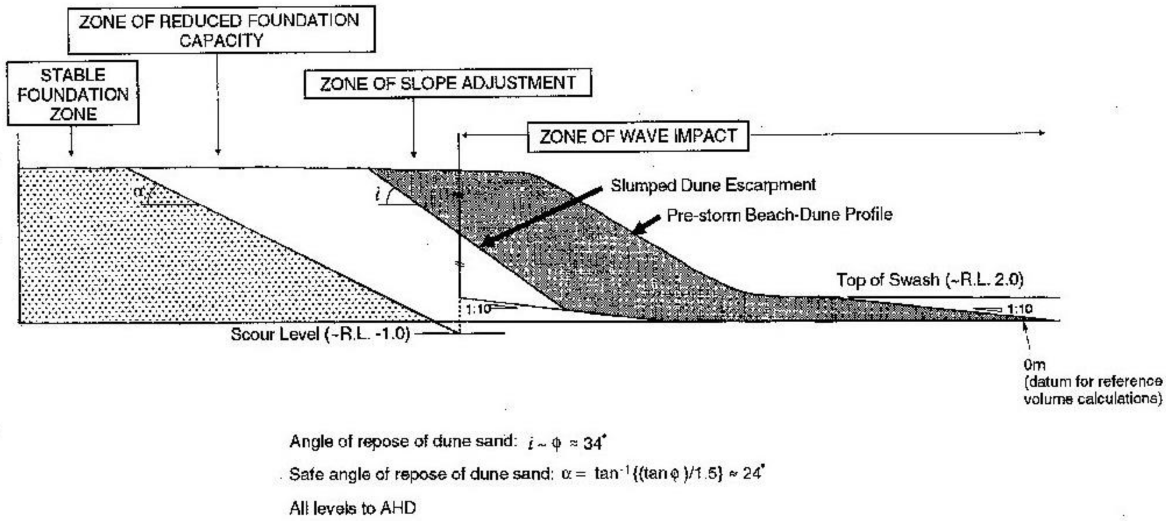


Figure G-1: Zone of slope adjustment (Nielsen et al, 1992)

Typically, a constant dune height is adopted for each beach when using this method, but this approach is not ideal because beach profiles are often irregular, and vary alongshore. Measured cross sections of each beach's dune system were used for this investigation (instead of a constant dune height), to accurately determine the relationship between storm demand volumes and erosion distances (Figure G-2). All calculations were undertaken volumetrically rather than using the simplified empirical equations in Nielsen et al., (1992) to define the ZWI, ZSA and ZRFC. That is, it was not necessary to define an average ground level to solve for each of these zones.

The only inputs were storm demand, swash elevation, scour level and the angle of repose.

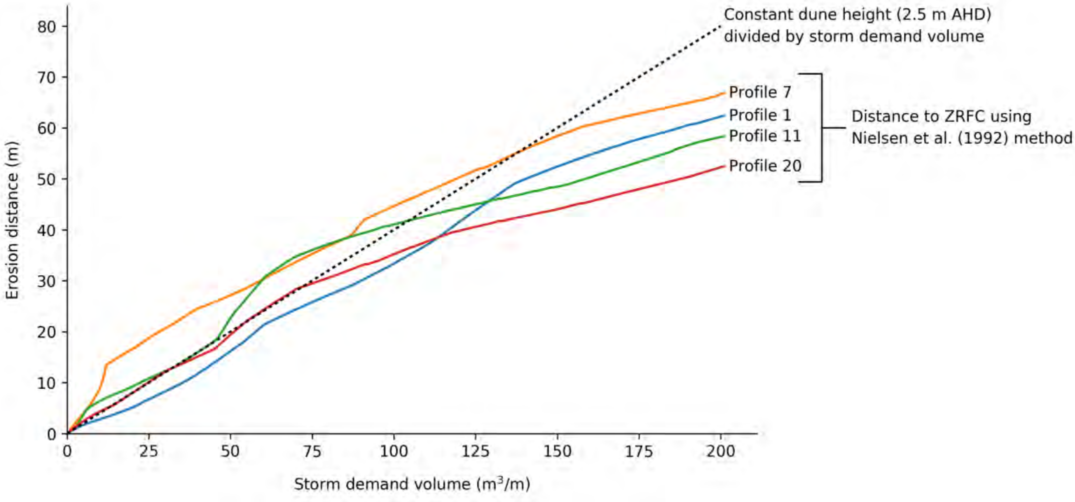


Figure G-2: Erosion distances for a range of storm demand volumes at Broulee Beach (block N)

The Nielsen et al. (1992) method assumes a swash elevation of 2 m AHD. Some of the beaches exposed to lower energy wave climates did not fit this model, so a swash elevation of 1 m AHD was adopted for the following sites:

- Maloneys Beach;
- Surfside Beach (east and west);
- Tomakin Cove; and
- Broulee Beach (southern section only).

The following values were assumed for all calculations:

• angle of repose of the dune sand:	34° (1V:1.48H)
• scour level:	-1 m AHD
• eroded beach face slope:	5.7° (1V:10H)
• factor of safety:	1.5

Appendix H: Broulee Island Connectivity

Broulee Island and tombolo are located at the southern end of Broulee Beach.

A tombolo is a salient (foreshore widening) which extends sufficiently to connect dry sand (i.e. above mean sea level) to an offshore feature (such as an island). Where a feature is located sufficiently close to shore, sand will accumulate in the lee to form a tombolo during periods of low wave energy. During high wave energy, tombolos may be severed from the feature, resulting in a salient. Once connected, a tombolo will starve downdrift beaches of normal longshore sediment supply. The effect of periodic tombolos is the temporary storage and release of a "slug" of sediment to the downdrift region (Chasten et al., 1993).

The periodic or ephemeral tombolo at Broulee Island has been historically breached during large swells, separating the island from the mainland temporarily, although the most recently recorded breach occurred sometime between May 1984 and May 1987.

Ballard (1982) provides an extensive history of the salient/tombolo at Broulee Island between 1828 and 1981. A variety of data sources were used including maps, photographs, illustrations, documented observations and NSW legislative assembly proceedings. Ballard found that the tombolo was severed rapidly from waves originating on its southern side (Bengello Beach) but then took a longer period of time to re-connect to the island. Analysis of a series of aerial photographs between 1961 and 1981 clearly showed the transport of sediment from the tombolo to the north into Broulee Bay when it was breached.

The status of the Broulee Island tombolo between 1828 and 1901 has been tabulated by WRL in Table H-1, based on findings by Ballard (1982). In 1873, shortly before it was severed, vegetation (including root systems) on the tombolo was removed to widen a track which existed between Broulee Island and the mainland. From 1920 to 1930, shell-grit was mined from within Broulee Bay which may have resulted in a depleted supply of sediment to maintain the tombolo (Ballard, 1982). Unfortunately there are large gaps in the record when the tombolo may have been severed which have gone unrecorded, particularly between 1901 and the first aerial photograph in 1961.

Table H-1: History of Broulee Island Salient/Tombolo Condition (1828-1901)

Date	Salient/Tombolo Condition	Reference
1828	Connected	Ballard (1982)
1837	Connected	Ballard (1982)
1839	Connected	Ballard (1982)
1841	Disconnected (Possible)	Ballard (1982)
1843	Disconnected	Ballard (1982)
1845	Connected	Ballard (1982)
1869	Disconnected	Ballard (1982)
1873	Disconnected	Ballard (1982)
1891	Connected	Ballard (1982)
1892	Connected	Ballard (1982)
1901	Disconnected (Possible)	Ballard (1982)

Ballard's aerial photography analysis has been extended by WRL through examination of historical aerial images provided by the Office of Environment and Heritage (OEH) between 1961 and 2011 (reproduced at the end of this appendix as Figure H-2 to Figure H-19). Additional photographs from other sources were also collected and included in the analysis with the status of the Broulee Island tombolo between 1961 and 2017 tabulated in Table H-2.

Table H-2: History of Broulee Island Salient/Tombolo Condition (1961-2017)

Date	Salient/Tombolo Condition	Vegetation Status	Reference
1/08/1961	Connected	No vegetation	OEH Aerial Photograph
??/03/1963	Connected	No vegetation	Oblique Photograph
??/02/1694	Connected	No vegetation	OEH Aerial Photograph
3/02/1965	Connected	?	Ballard (1982)
15/05/1966	Disconnected	N/A	Ballard (1982)
??/??/1967	Disconnected	N/A	Moruya & District Historical Society Observation
7/01/1969	Disconnected	N/A	OEH Aerial Photograph
9/05/1971	*Connecting*	?	Ballard (1982)
4/06/1972	Connected	No vegetation	OEH Aerial Photograph
??/06/1974	Disconnected	N/A	Ballard (1982)
10/09/1975	Disconnected	N/A	OEH Aerial Photograph
11/03/1977	Disconnected	N/A	OEH Aerial Photograph
28/07/1977	Disconnected	N/A	Ballard (1982)
26/11/1977	Disconnected	N/A	OEH Aerial Photograph
28/11/1977	Disconnected	N/A	OEH Aerial Photograph
??/12/1979	Disconnected	N/A	WRL Site Inspection
21/12/1980	*Connecting*	No vegetation	OEH Aerial Photograph
27/06/1981	Connected	No vegetation	OEH Aerial Photograph
11/04/1984	Connected	Thinly vegetated	OEH Aerial Photograph
29/05/1984	Connected	Thinly vegetated	WRL Site Inspection
22/05/1987	Disconnected	N/A	Landsat Satellite Image
25/10/1988	Disconnected	N/A	OEH Aerial Photograph
19/01/1989	Connected	No vegetation	Landsat Satellite Image
22/11/1991	Connected	No vegetation	OEH Aerial Photograph
15/04/1993	Connected	Thinly vegetated	OEH Aerial Photograph
6/03/1996	Connected	Vegetated	DLWC Oblique Aerial Photograph
6/02/1999	Connected	Vegetated	OEH Aerial Photograph
7/03/2005	Connected	Vegetated	OEH Aerial Photograph
28/03/2007	Connected	Vegetated	OEH Aerial Photograph
15/05/2011	Connected	Vegetated	OEH Aerial Photograph
8/12/2012	Connected	Vegetated	WRL Site Inspection
24/02/2017	Connected	Vegetated	WRL Site Inspection

Based on the histories presented in Table H-1 and Table H-2, Broulee Island has been disconnected three to five (3-5) times between 1828 and 1901 (73 years) and three (3) times between 1961 and 2017 (56 years). While significant gaps in the dataset are acknowledged, on average, the tombolo has been severed approximately every 15-25 years. At the time of writing, the island has remained connected for at least 28 years. There is not enough evidence to confidently comment on the varying length of time that the island may be disconnected for, following a breach, but it is noted that, following the breach in May/June 1974, a tombolo did not reform until late 1980/ mid 1981; a six to seven (6-7) year duration. Note that WRL has made no attempt to estimate the varying volume of sand released into Broulee Bay when severing of the tombolo has occurred in the past.

On 24 February 2017, WRL undertook a cross-sectional survey at the approximate narrowest point of the tombolo. Ground surface elevations were measured using a Trimble R10 RTK-GPS and offset using the NSW CorsNET network. The transect had a volume of 159 m^3/m above 0 m AHD and is shown in Figure H-1.

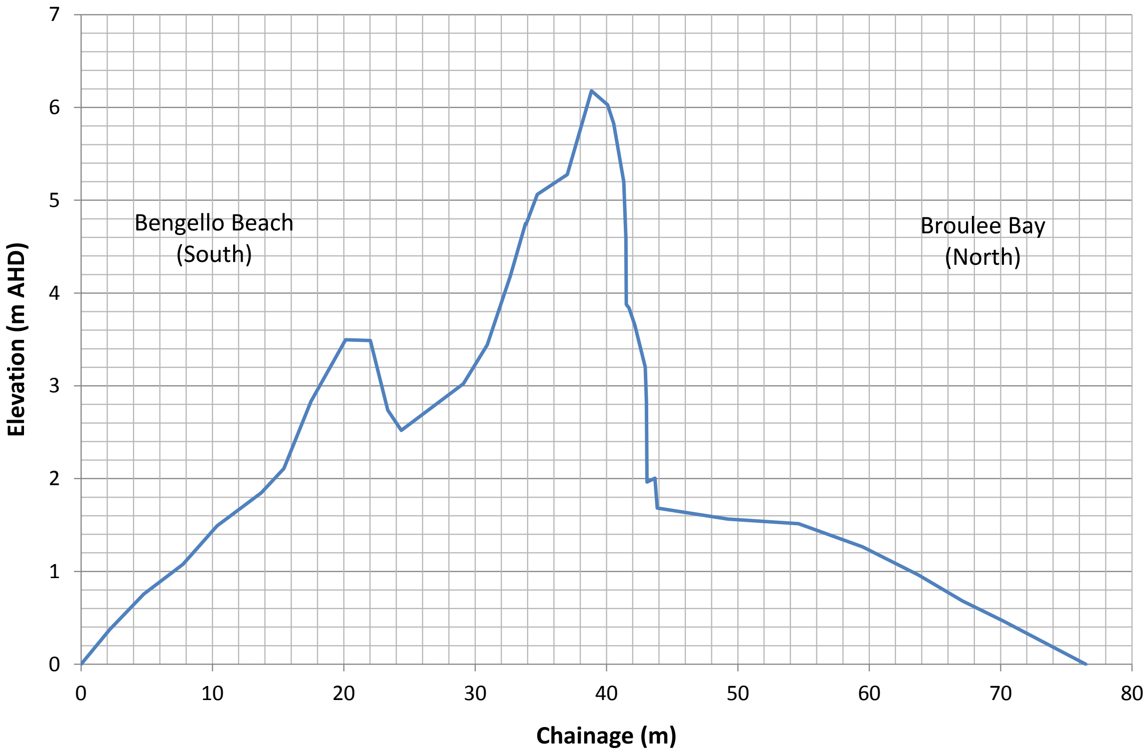


Figure H-1: WRL Survey of Broulee Island Tombolo (Facing West)- 28/02/2017

In addition to the present connected state, WRL has also considered the scenario of erosion/recession of Broulee Beach with Broulee Island disconnected in this report. Such a breach would almost certainly be initiated from the southern side (Bengello Beach). WRL has not undertaken an assessment of the potential for a breach to occur on the present profile. Detailed modelling of the potential for a breach would be complex as it involves interactions between wave runup, wave overtopping, cross shore erosion, longshore processes and vegetation. It is noted that the nominal design storm demand for the centre of Bengello Beach (170 m^3/m above 0 m AHD, based on erosion measured during May/June 1974) is slightly larger than the volume currently in the tombolo transect. However, the present profile appears to have more volume and is heavily vegetated in contrast to the un-vegetated state of the tombolo prior to the May/June 1974 storm sequence (Figure H-5). Indeed, the tombolo is now in its most heavily vegetated state since aerial photograph records began in 1961, which may contribute to the lack of breaches in the last 28 years. However, WRL considers that it is likely that the tombolo will be severed again at some stage in the future.

Figure H-2:OEH Photogrammetry - Broulee Island 1/8/1961

Figure H-3: OEH Photogrammetry - Broulee Island February 1964

Figure H-4: OEH Photogrammetry - Broulee Island 7/1/1969

Figure H-5: OEH Photogrammetry - Broulee Island 6/4/1972

Figure H-6: OEH Photogrammetry - Broulee Island 10/9/1975

Figure H-7:OEH Photogrammetry - Broulee Island 11/3/1977

Figure H-8: OEH Photogrammetry - Broulee Island 28/7/1977

Figure H-9: OEH Photogrammetry - Broulee Island 26/11/1977

Figure H-10: OEH Photogrammetry - Broulee Island 21/12/1980

Figure H-11: OEH Photogrammetry - Broulee Island 27/8/1981

Figure H-12: OEH Photogrammetry - Broulee Island 12/4/1984

Figure H-13: OEH Photogrammetry - Broulee Island 25/10/1988

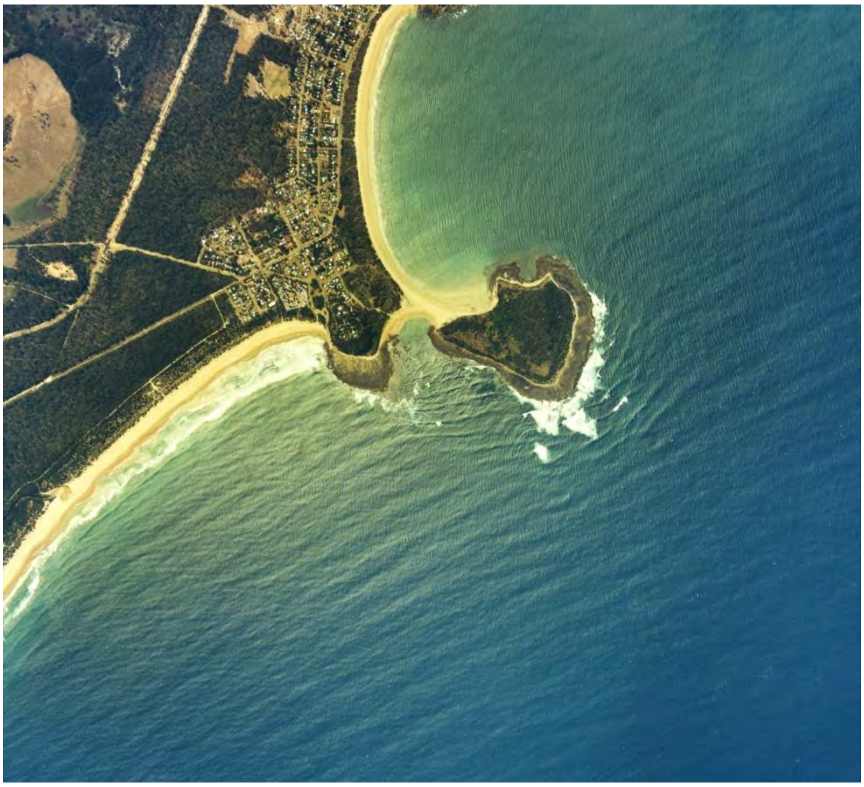


Figure H-14: OEH Photogrammetry - Broulee Island 22/11/1991

Figure H-15: OEH Photogrammetry - Broulee Island 15/4/1993

Figure H-16:OEH Photogrammetry - Broulee Island 6/2/1999

Figure H-17: OEH Photogrammetry - Broulee Island 7/3/2005

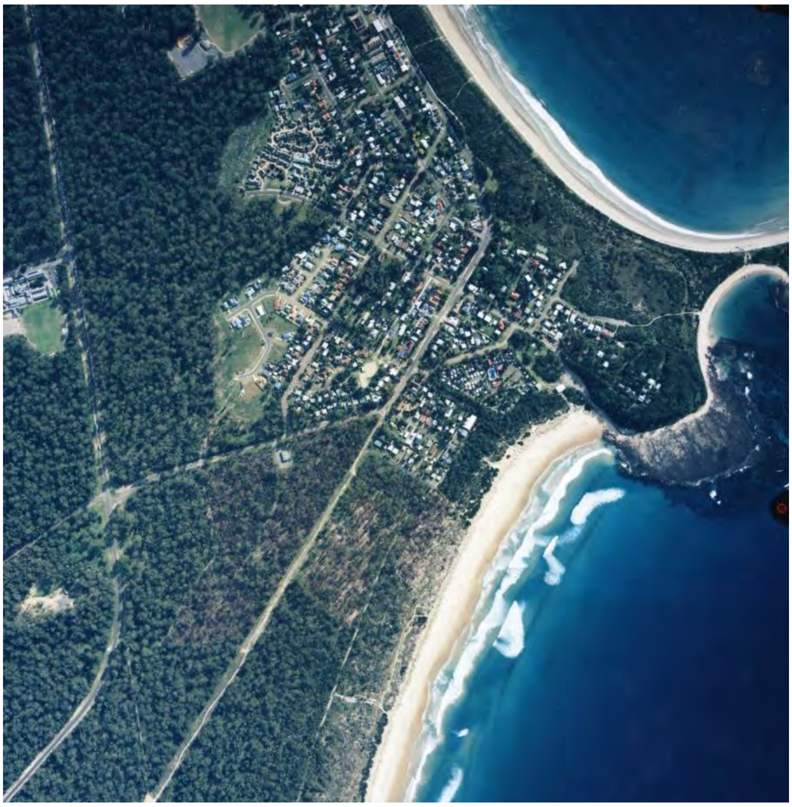


Figure H-18: OEH Photogrammetry - Broulee Island 28/3/2007

Figure H-19:OEH Photogrammetry - Broulee Island 15/5/2011

Appendix I: Erosion/Recession Hazard Maps

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Maloneys Beach
Deterministic erosion/recession hazard lines

- 2017
- 2050
- 2065
- 2100
- Watercourse instability region

Figure I.1

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: 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.
Note 4: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Long Beach existing seawall
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region
Bedrock (non-erodible)
Seawall

Figure I.2

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: 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.
Note 4: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Long Beach existing seawall (east)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Bedrock (non-erodible)
Seawall

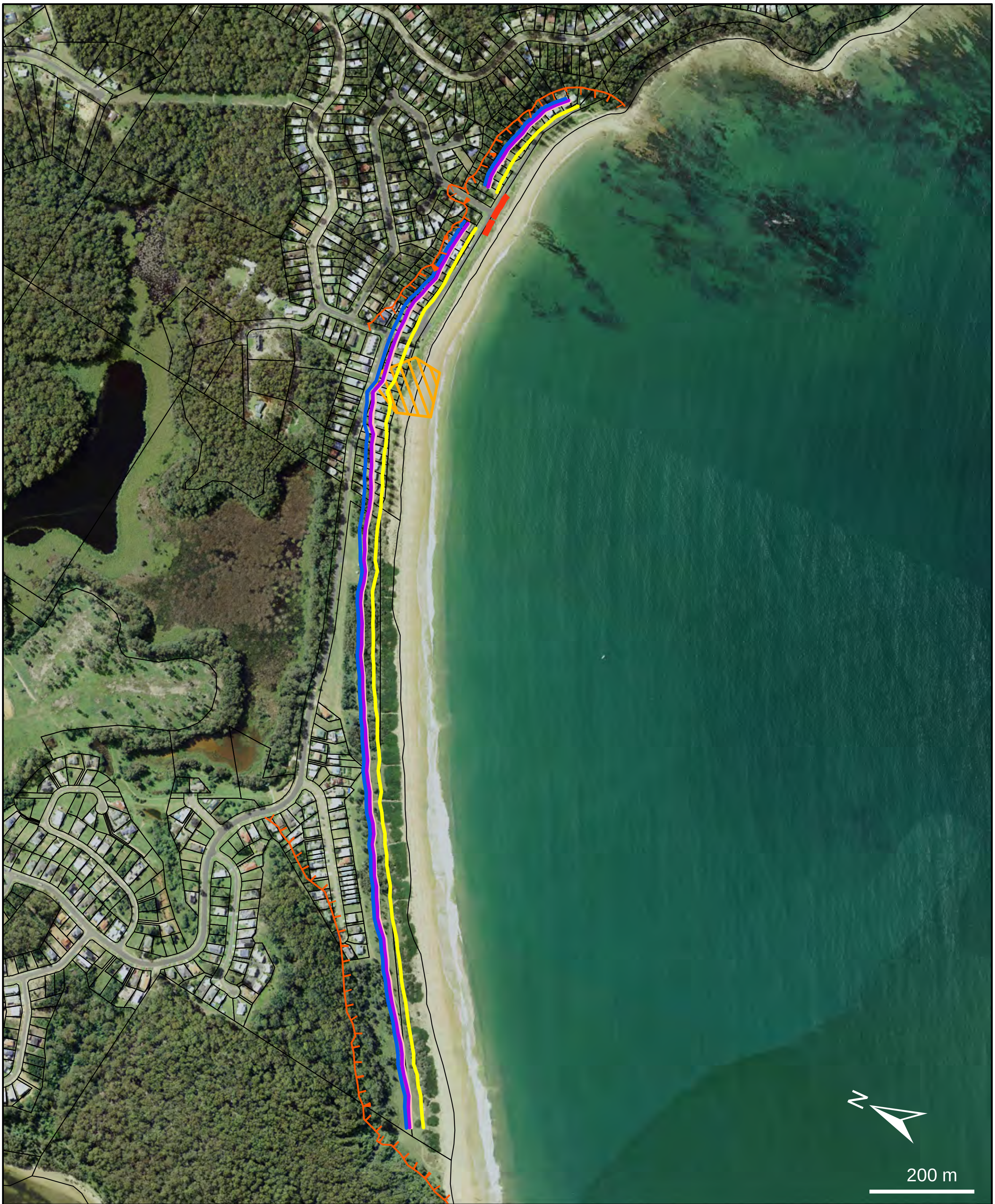
Figure I.3

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Long Beach existing seawall (west)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Bedrock (non-erodible)

Figure I.4



Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: 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.
Note 4: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Long Beach existing seawall
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Bedrock (non-erodible)
Seawall

Figure I.5

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: 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.
Note 4: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Long Beach existing seawall (east)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region
Bedrock (non-erodible)
Seawall

Figure I.6

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.

Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Long Beach existing seawall (west)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017	Bedrock (non-erodible)
2050	
2065	
2100	

Figure I.7

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: 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.

Long Beach no seawall (east)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Bedrock (non-erodible)

Figure I.8

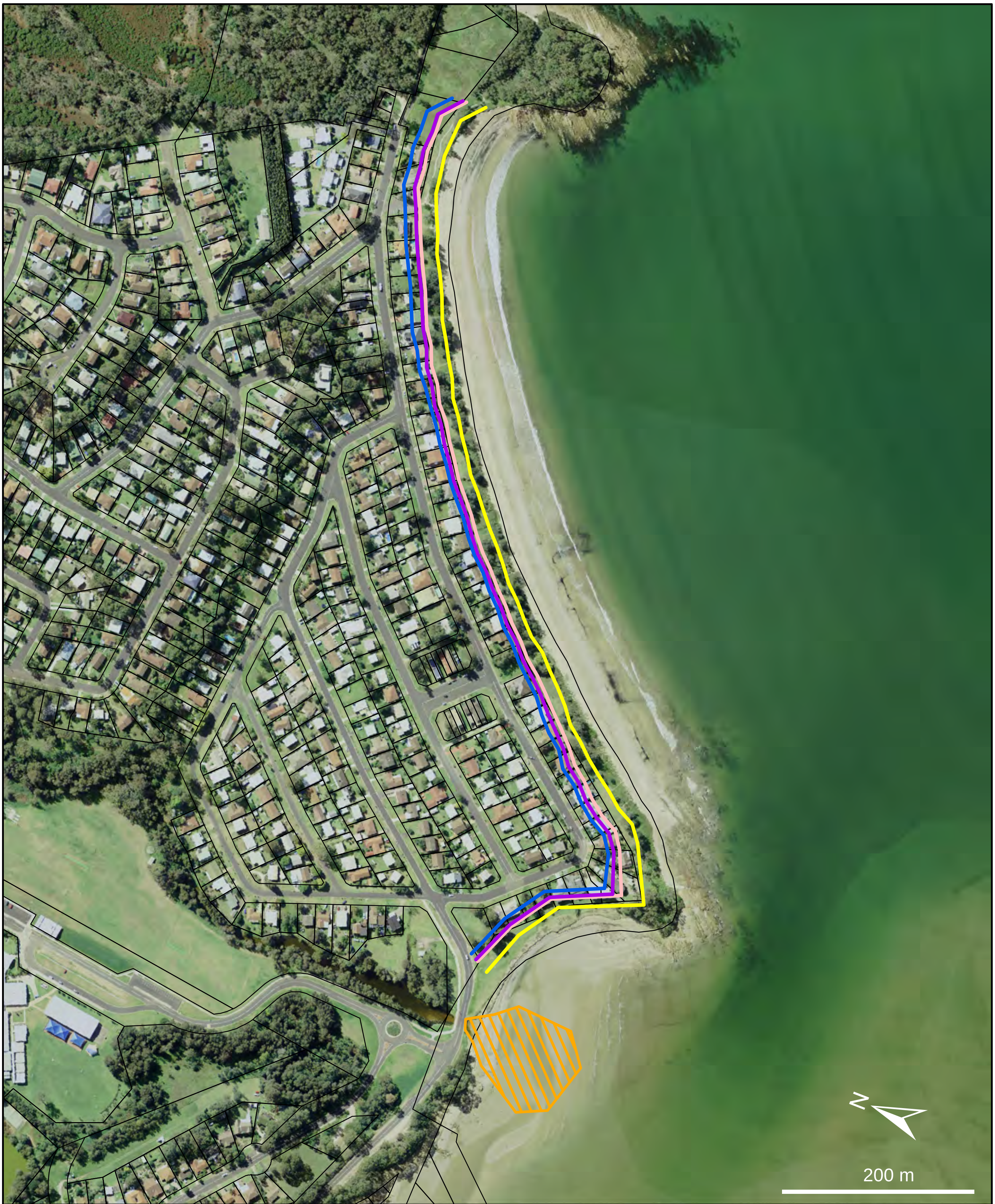
Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: Areas landward of the bedrock (non-erodible) line could be subject to coastal cliff or slope instability hazards which are beyond the scope of this study.

Long Beach no seawall (east)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region
Bedrock (non-erodible)

Figure I.9

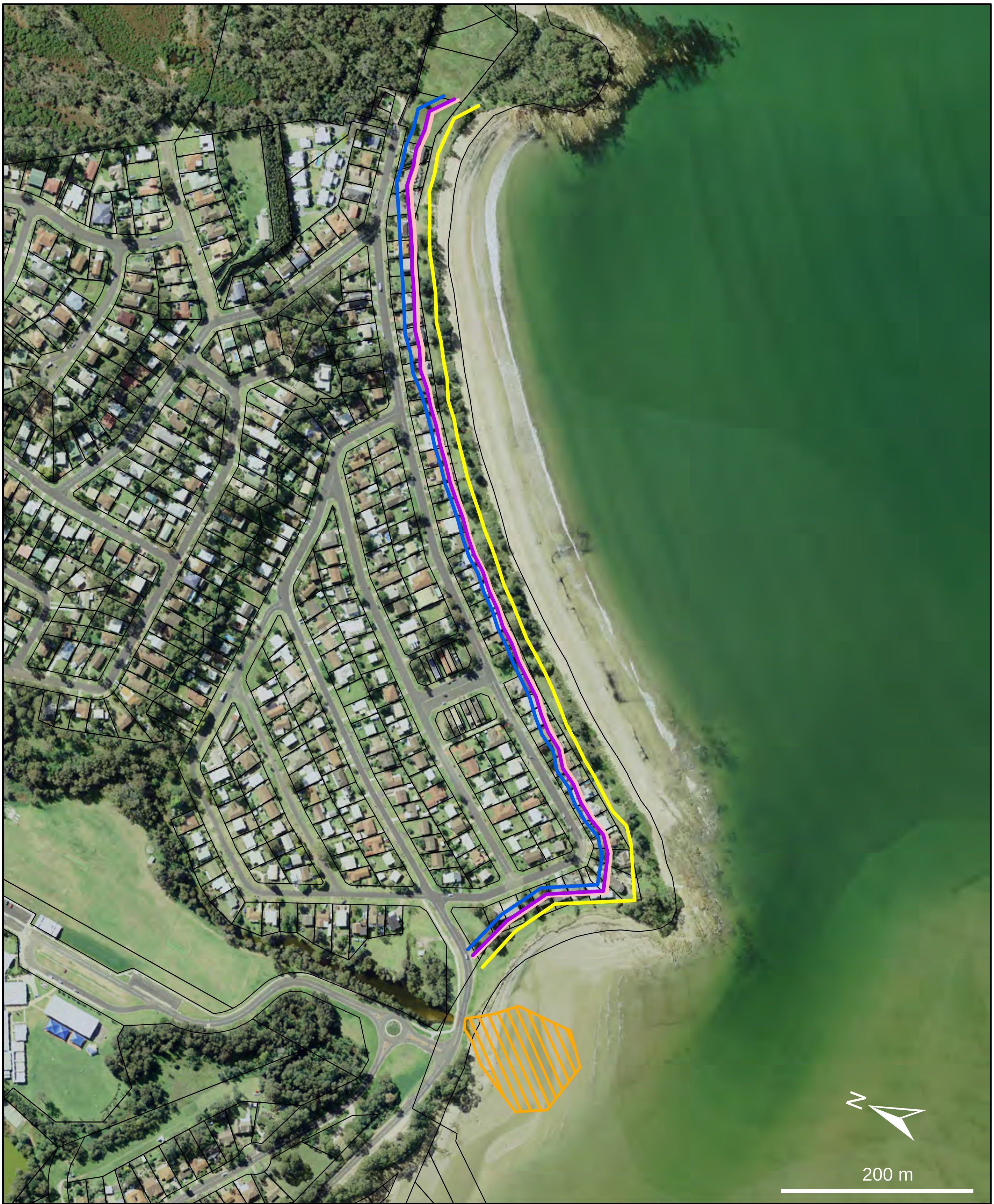


Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Surfside Beach
5% encounter probability
Probabilistic erosion/recession hazard lines

- 2017
- 2050
- 2065
- 2100
- Watercourse instability region

Figure I.10



Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Surfside Beach
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.11

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: 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.

Sunshine Bay
Deterministic erosion/recession hazard lines

2017
2050
2065
2100

Bedrock (non-erodible)

Figure I.12

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Malua Bay existing seawall
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Seawall

Figure I.13

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: The shape of hazard lines not located at the seawall is hypothetical only and requires further detailed assessment beyond the scope of this study.

Malua Bay existing seawall
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Seawall

Figure I.14

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Malua Bay no seawall
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.15

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Malua Bay no seawall
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.16

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Guerilla Bay (south)
Deterministic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region

Figure I.17

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: Hazard lines do not extend to the western end of the beach as this is the limit of available photgrammetry.

Barlings Beach
Deterministic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region

Figure I.18

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The watercourse entrance instability region is potentially subject to breakthrough from the Tomaga River and modified sand transport processes.
Note 3: Potential for salient loss cannot be fully quantified with contemporary desktop engineering techniques. The shoreline could potentially move landward of the hazard lines in this region due to sea level rise or other coastal processes.

Tomakin Cove
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region
Potential salient loss region

Figure I.19

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The watercourse entrance instability region is potentially subject to breakthrough from the Tomaga River and modified sand transport processes.
Note 3: Potential for salient loss cannot be fully quantified with contemporary desktop engineering techniques. The shoreline could potentially move landward of the hazard lines in this region due to sea level rise or other coastal processes.

Tomakin Cove
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region
Potential salient loss region

Figure I.20

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Watercourse instability region

Figure I.21

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached (north)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.22

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached (south)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100

Figure I.23

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.24

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached (north)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.25

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.

Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island attached (south)
1% encounter probability
Probabilistic erosion/recession hazard lines

- 2017
- 2050
- 2065
- 2100

Figure I.26

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.

Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Note 3: Ephemeral tombolo region is considered to be temporary land subject to erosion when/if the Broulee Island reverts to a detached state.

Broulee Beach island detached
5% encounter probability
Probabilistic erosion/recession hazard lines

2017	Watercourse instability region
2050	Ephemeral tombolo region
2065	
2100	

Figure I.27

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island detached (north)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.28

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: Ephemeral tombolo region is considered to be temporary land subject to erosion when/if the Broulee Island reverts to a detached state.

Broulee Beach island detached (south)
5% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Ephemeral tombolo region

Figure I.29

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: Ephemeral tombolo region is considered to be temporary land subject to erosion when/if the Broulee Island reverts to a detached state.

Broulee Beach island detached
1% encounter probability
Probabilistic erosion/recession hazard lines

2017	Watercourse instability region
2050	Ephemeral tombolo region
2065	
2100	

Figure I.30

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.

Broulee Beach island detached (north)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Watercourse instability region

Figure I.31

Note 1: Landward movement of the shoreline could be limited by the presence of bedrock.
Note 2: The shoreline could potentially move landward of the hazard lines in the watercourse entrance instability region due to lowering of the beach profile from entrance scouring.
Note 3: Ephemeral tombolo region is considered to be temporary land subject to erosion when/if the Broulee Island reverts to a detached state.

Broulee Beach island detached (south)
1% encounter probability
Probabilistic erosion/recession hazard lines

2017
2050
2065
2100
Ephemeral tombolo region

Figure I.32

Appendix J: Width of Zone of Reduced Foundation Capacity (ZRFC), in metres

Table J-1: Maloneys Beach

Block	Profile	2017	2050	2065	2100
E	1	6.2	6.2	6.2	6.2
E	2	6.6	6.6	6.6	6.6
E	3	7.5	7.5	7.5	7.5
E	4	7.9	7.9	7.9	7.9
E	5	8.3	8.3	8.3	8.3
E	6	9.4	9.4	9.4	9.4
E	7	9.3	9.3	9.3	9.3
E	8	10.0	10.0	10.0	10.0
E	9	9.9	9.9	9.9	9.9
E	10	10.1	10.1	10.1	10.1
E	11	10.3	10.3	10.3	10.3
E	12	10.6	10.6	10.6	10.6
E	13	10.2	10.2	10.2	10.2
E	14	10.4	10.4	10.4	10.4
E	15	10.2	10.2	10.2	10.2
E	16	10.2	10.2	10.2	10.2
E	17	10.3	10.3	10.3	10.3
E	18	9.5	9.5	9.5	9.5
E	19	9.9	9.9	9.9	9.9
E	20	9.7	9.7	9.7	9.7
E	21	9.8	9.8	9.8	9.8
E	22	9.8	9.8	9.8	9.8
E	23	9.4	9.4	9.4	9.4
E	24	9.1	9.1	9.1	9.1
E	25	8.7	8.7	8.7	8.7
E	26	9.3	9.3	9.3	9.3
E	27	8.9	8.9	8.9	8.9
E	28	8.5	8.5	8.5	8.5
E	29	8.9	8.9	8.9	8.9
E	30	8.9	8.9	8.9	8.9
E	31	8.6	8.6	8.6	8.6
E	32	8.4	8.4	8.4	8.4
E	33	7.9	7.9	7.9	7.9
E	34	8.0	8.0	8.0	8.0
E	35	7.4	7.4	7.4	7.4
E	36	5.2	5.2	5.2	5.2
E	37	6.9	6.9	6.9	6.9
E	38	6.4	6.4	6.4	6.4

Table J-2: Long Beach

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
C	1	8.6	8.8	8.8	8.8	9.7	9.0	9.0	8.9
C	2	8.6	8.3	8.4	8.6	9.9	8.2	8.9	8.8
C	3	8.9	7.7	8.0	8.2	9.5	9.1	8.8	8.4
C	4	9.2	7.6	7.7	7.9	10.7	9.1	8.9	8.4
C	5	9.2	7.6	7.8	8.3	10.4	8.9	8.5	8.3
C	6	8.4	8.1	8.1	8.3	9.3	8.2	8.0	8.2
C	7	10.2	8.2	8.3	9.1	9.0	8.3	8.1	8.5
C	8	9.7	8.3	8.5	9.0	9.9	8.6	8.5	8.7
C	9	9.8	8.1	8.3	9.2	10.3	8.8	8.6	8.8
C	10	9.5	8.3	9.1	9.2	10.1	8.6	8.5	8.6
C	11	9.8	9.1	9.5	9.4	10.0	8.6	8.4	9.0
C	12	9.7	9.9	9.7	9.6	9.4	8.4	8.4	9.2
C	13	8.7	9.6	9.4	9.7	8.7	8.1	8.2	9.1
C	14	9.0	9.3	9.3	9.9	10.1	8.2	8.6	9.1
C	15	9.0	9.3	9.3	9.3	10.3	8.6	9.0	9.1
C	16	8.6	10.4	10.5	10.1	9.2	8.0	8.4	9.2
C	17	8.9	10.1	9.7	9.5	9.9	7.8	7.9	9.0
C	18	9.0	10.2	9.7	9.4	10.4	7.9	7.9	9.2
C	19	9.1	9.5	9.2	9.1	9.9	7.9	7.9	8.9
C	20	8.5	9.5	9.3	9.9	10.1	7.3	7.7	9.2
C	21	8.2	9.5	9.6	9.6	9.5	8.1	8.7	9.0
C	22	8.2	9.3	9.3	9.6	9.9	10.3	10.1	9.4
C	23	7.9	9.6	9.5	9.7	9.9	9.9	9.9	9.5
C	24	8.4	10.1	10.3	10.3	10.2	9.3	9.5	9.6
C	25	8.3	10.4	10.4	10.5	9.7	10.3	10.2	9.9
C	26	9.4	10.0	9.7	9.4	9.5	9.6	9.8	9.9
C	27	8.8	10.3	10.0	9.6	10.2	9.4	9.4	9.8
C	28	8.9	9.7	9.4	9.0	10.0	9.2	8.9	9.4
C	29	10.0	9.8	9.8	9.3	10.5	8.6	8.7	9.4
C	30	11.0	9.9	10.0	10.3	11.2	9.6	9.6	10.1
C	31	9.8	9.9	10.0	10.2	9.3	10.2	10.1	10.1
C	32	10.6	10.0	10.0	10.2	11.7	10.5	10.4	10.5
C	33	9.6	9.9	9.8	10.0	9.8	10.6	10.5	10.1
C	34	9.6	9.6	9.7	9.8	9.5	10.3	10.1	9.8
C	35	10.1	9.8	9.9	10.3	9.6	11.2	10.8	10.2
C	36	10.0	10.0	10.0	10.2	9.9	10.8	10.8	10.2
C	37	9.5	10.3	10.4	10.3	10.3	10.5	10.5	10.2
C	38	10.1	11.2	11.2	10.9	10.9	10.6	10.7	10.8
C	39	9.6	10.9	10.8	10.5	9.9	9.7	10.0	10.3

Table J-3: Long Beach (continued)

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
C	40	9.1	10.6	10.6	10.6	9.5	10.9	10.7	10.5
C	41	11.0	10.7	10.7	10.7	9.9	10.7	10.7	10.7
C	42	10.0	10.0	10.0	10.0	10.2	9.9	9.9	10.0
C	43	9.7	10.0	10.0	9.8	12.2	9.3	9.4	9.8
C	44	11.0	10.6	10.6	10.6	11.3	10.6	10.6	10.7
C	45	10.1	10.8	10.8	10.8	9.4	11.1	11.0	10.8
C	46	9.8	10.1	10.0	10.0	9.8	9.9	10.1	10.0
C	47	9.9	9.6	9.6	9.6	10.3	9.7	9.7	9.7
C	48	10.2	10.2	10.2	10.2	10.3	10.2	10.2	10.2
C	49	11.4	11.1	11.1	11.1	10.7	11.1	11.1	11.1
C	50	10.9	12.4	12.4	12.2	10.7	12.0	12.2	12.0
C	51	10.3	10.8	10.8	10.8	9.4	10.7	10.8	10.7
C	52	11.1	9.9	9.9	10.2	10.5	10.4	10.3	10.3
C	53	10.7	10.0	9.8	10.1	10.3	10.8	10.7	10.4
C	54	10.2	10.2	10.1	10.1	10.8	11.0	10.8	10.5
C	55	9.3	8.8	8.7	9.2	10.9	10.8	10.3	9.6
C	56	10.4	9.5	9.4	9.6	10.7	10.1	10.0	9.9
C	57	10.1	10.0	9.9	9.9	10.9	10.2	10.2	10.1
C	58	10.5	10.0	10.0	10.2	9.0	10.6	10.4	10.3
C	59	10.3	10.2	10.1	10.3	9.8	10.6	10.5	10.4
C	60	10.2	10.8	10.7	10.4	12.6	10.6	10.6	10.5
C	61	10.5	10.3	10.3	10.3	11.7	10.5	10.4	10.4
C	62	9.8	9.2	9.1	9.2	12.1	8.9	9.0	9.5
C	63	9.9	10.1	10.1	9.9	11.0	9.7	9.8	10.0
C	64	9.4	9.1	9.1	9.2	11.4	9.8	9.7	9.5
C	65	9.5	9.1	9.0	9.1	10.7	9.3	9.3	9.3
C	66	8.5	9.1	8.6	8.1	9.9	7.4	7.8	8.3
C	67	8.8	7.9	7.9	8.1	9.1	8.3	8.2	8.4
C	68	8.0	7.1	7.1	7.4	7.9	7.5	7.4	7.5
C	69	7.0	7.4	7.4	7.4	8.0	7.3	7.3	7.3
C	70	7.6	7.6	7.7	7.7	8.3	7.5	7.4	7.5
C	71	8.6	8.4	8.2	8.1	9.6	8.4	8.4	8.4
C	72	9.7	9.1	9.1	8.8	9.7	8.8	8.9	9.2
D	1	9.8	9.5	9.5	9.5	10.2	9.6	9.6	9.6
D	2	10.8	10.2	10.1	10.2	8.7	10.4	10.4	10.3
D	3	9.1	9.1	9.1	9.2	9.1	9.4	9.3	9.3
D	4	9.3	9.5	9.5	9.4	8.9	9.5	9.5	9.4
D	5	8.9	8.6	8.6	8.6	8.7	8.7	8.6	8.7
D	6	8.6	8.5	8.5	8.6	9.0	8.7	8.7	8.7

Table J-4: Long Beach (continued)

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
D	7	8.9	8.8	8.8	8.8	8.3	8.7	8.7	8.7
D	8	8.8	8.3	8.3	8.4	8.2	8.5	8.5	8.4
D	9	8.4	8.1	8.2	8.2	7.9	8.2	8.3	8.1
D	10	7.7	8.2	8.2	8.1	7.4	8.2	8.2	8.1
D	11	7.9	7.9	7.9	8.0	7.5	8.1	8.1	8.0
D	12	7.7	7.9	8.0	8.0	7.4	8.0	8.0	7.9
D	13	8.1	8.2	8.2	8.1	7.1	8.0	8.1	8.0
D	14	8.1	8.7	8.8	8.7	7.9	8.6	8.6	8.5
D	15	7.8	8.4	8.2	8.1	7.4	8.3	8.3	8.2
D	16	8.5	8.3	8.4	8.4	8.0	8.3	8.3	8.3
D	17	8.3	8.1	8.1	8.1	8.5	8.0	8.0	8.1
D	18	8.4	7.9	8.0	8.1	8.2	8.0	8.0	8.2
D	19	8.4	8.2	8.2	8.3	8.1	8.3	8.3	8.3
D	20	8.2	7.7	7.9	8.1	7.4	8.0	7.9	8.0
D	21	8.0	7.6	7.6	7.7	7.4	7.8	7.6	7.7
D	22	7.0	6.9	7.0	7.1	7.1	7.6	7.4	7.1
D	23	6.9	7.5	7.4	7.4	7.6	7.4	7.5	7.3
D	24	7.4	7.2	7.3	7.3	7.7	7.3	7.3	7.3
D	25	7.5	8.0	8.0	8.1	7.5	8.1	8.1	8.0
D	26	7.4	7.8	7.7	7.9	7.2	8.2	8.1	7.8
D	27	7.7	7.1	7.1	7.3	7.0	7.5	7.6	7.4
D	28	7.4	8.1	8.3	8.1	7.2	7.8	7.8	7.8
D	29	7.1	7.6	7.6	7.7	7.3	7.6	7.6	7.6
D	30	7.1	7.8	7.8	8.0	7.5	7.7	7.7	7.7
D	31	7.7	7.7	7.8	7.7	7.1	6.8	7.1	7.3
D	32	8.1	7.0	6.9	7.0	6.6	6.9	6.9	7.1

Table J-5: Surfside Beach (east)

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
A	1	5.1	5.1	5.1	5.1	5.4	5.1	5.1	5.1
A	2	5.1	5.0	5.0	5.1	5.3	4.9	5.1	5.0
A	3	5.6	5.2	5.1	5.1	5.5	5.6	5.6	5.2
A	4	5.5	6.0	5.8	5.4	5.6	6.0	6.0	5.9
A	5	5.7	5.7	5.7	5.7	5.9	5.8	5.7	5.7
A	6	5.9	6.4	6.5	6.6	5.5	6.2	6.3	6.5
A	7	5.5	5.8	5.8	5.7	5.9	6.0	5.9	5.8
A	8	5.3	6.2	6.2	6.3	6.5	6.0	6.1	6.2
A	9	6.4	6.2	6.2	6.2	6.7	6.4	6.3	6.2

Table J-5: Surfside Beach (east) (continued)

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
A	10	6.0	6.0	6.1	6.7	8.5	5.1	5.3	6.4
A	11	6.5	6.2	6.3	6.4	6.8	5.5	5.8	6.2
A	12	6.5	5.2	5.6	5.8	7.4	5.9	5.6	5.6
A	13	6.5	6.5	6.6	6.5	7.3	6.2	6.1	6.6
A	14	7.4	5.7	5.7	5.6	5.9	6.2	5.9	5.6
A	15	7.1	5.4	5.6	5.7	7.9	5.7	5.4	5.4
A	16	6.8	5.9	6.0	5.9	7.3	6.2	6.0	6.0
A	17	6.9	5.7	5.5	5.5	7.4	6.3	6.0	5.6
A	18	7.1	5.9	6.2	6.3	7.5	6.5	6.2	6.0
A	19	7.1	6.5	6.3	6.3	7.1	7.1	6.9	6.4
A	20	6.7	7.1	6.8	6.8	6.4	7.2	7.1	7.0
A	21	6.8	6.5	6.5	6.5	6.2	6.9	6.7	6.5
A	22	7.1	6.6	6.6	6.7	7.4	6.7	6.6	6.6
A	23	7.0	6.9	6.7	6.5	7.1	6.7	6.7	7.0
A	24	6.5	6.5	6.3	6.3	7.0	7.1	6.9	6.4
A	25	6.9	6.9	6.7	6.7	7.0	7.0	7.0	6.8
A	26	6.9	6.6	6.5	6.5	6.9	6.6	6.7	6.5
A	27	6.4	6.5	6.5	6.7	7.2	6.3	6.3	6.6
A	28	6.8	6.5	6.3	6.3	6.7	6.6	6.5	6.4
A	29	5.8	6.4	6.3	6.3	6.1	6.1	6.4	6.2
A	30	5.2	6.3	6.3	6.4	5.7	6.1	6.1	6.1
A	31	5.4	6.2	6.2	6.1	6.8	5.8	5.9	5.9
A	32	5.7	6.1	6.1	6.0	6.8	5.4	5.6	5.7
B	1	6.3	5.9	5.9	5.9	6.9	6.0	6.0	6.1
B	2	5.8	6.1	6.1	6.0	6.8	5.7	5.7	5.8
B	3	5.9	6.0	6.0	5.9	6.5	5.6	5.7	5.8
B	4	6.7	5.8	5.9	6.0	7.1	5.8	5.8	6.2
B	5	6.6	6.1	6.2	6.2	6.4	6.0	6.1	6.3
B	6	5.7	5.9	5.8	5.8	6.5	5.7	5.7	5.7
B	7	5.6	6.0	6.0	5.9	6.6	5.8	5.9	5.8
B	8	5.5	5.9	5.9	5.9	7.4	5.8	5.8	5.8
B	9	5.9	5.7	5.7	5.7	6.7	5.7	5.7	5.7
B	10	5.7	5.3	5.3	5.3	7.1	5.5	5.5	5.6
B	11	4.9	5.2	5.2	5.1	5.4	4.9	4.9	5.1
B	12	5.2	5.2	5.2	5.2	6.1	5.2	5.2	5.2

Table J-6: Surfside Beach (west)

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
1	4	5.0	5.0	5.0	5.0	4.8	5.1	5.1	5.0
1	5	5.0	4.9	4.9	4.9	5.1	4.9	4.9	5.0
1	6	5.0	5.0	5.0	5.1	5.1	5.1	5.1	5.0
1	7	5.3	5.2	5.2	5.3	5.7	5.4	5.3	5.3

Table J-7: Sunshine Bay

Block	Profile	2017	2050	2065	2100
1	4	11.6	11.6	11.6	11.6
2	1	8.7	8.7	8.7	8.7

Table J-8: Malua Bay

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
1	1	7.4	8.1	8.2	8.1	7.5	7.6	7.6	7.7
1	2	8.0	7.3	7.3	7.3	7.3	7.1	7.2	7.4
1	3	9.4	8.7	9.0	9.1	9.2	8.9	8.8	9.0
1	4	9.3	9.9	9.9	9.8	8.3	9.5	9.6	9.5
1	5	9.9	9.3	9.6	9.6	8.9	9.3	9.2	9.5
1	6	11.0	9.9	9.9	10.2	8.8	10.3	10.3	10.4
2	1	10.0	9.8	9.8	10.1	8.9	10.2	10.2	10.2
2	2	10.2	10.8	10.8	10.8	11.3	10.7	10.7	10.7
2	3	10.0	10.2	10.3	10.3	9.6	10.3	10.3	10.2
2	4	9.6	10.3	10.3	10.3	11.5	10.4	10.3	10.4
2	5	10.5	11.9	11.8	11.6	10.7	11.7	11.7	11.4
2	6	6.8	4.4	6.7	6.7	6.6	7.2	6.4	6.2

Table J-9: Guerilla Bay (south)

Block	Profile	2017	2050	2065	2100
5	1	10.0	10.0	10.0	10.0
6	1	8.0	8.0	8.0	8.0
6	2	7.9	7.9	7.9	7.9
6	3	6.1	6.1	6.1	6.1

Table J-10: Barlings Beach

Block	Profile	2017	2050	2065	2100
1	1	11.8	11.8	11.8	11.8
1	2	10.8	10.8	10.8	10.8
1	3	9.7	9.7	9.7	9.7
1	4	9.1	9.1	9.1	9.1
1	5	9.5	9.5	9.5	9.5
1	6	10.2	10.2	10.2	10.2
2	1	14.9	14.9	14.9	14.9
2	2	14.1	14.1	14.1	14.1
2	3	10.4	10.4	10.4	10.4
2	4	12.1	12.1	12.1	12.1
2	5	13.0	13.0	13.0	13.0
2	6	11.3	11.3	11.3	11.3
2	7	11.4	11.4	11.4	11.4
2	8	9.7	9.7	9.7	9.7
2	9	6.7	6.7	6.7	6.7
2	10	8.4	8.4	8.4	8.4
3	1	13.8	13.8	13.8	13.8
3	2	11.6	11.6	11.6	11.6
3	3	9.4	9.4	9.4	9.4
3	4	8.6	8.6	8.6	8.6

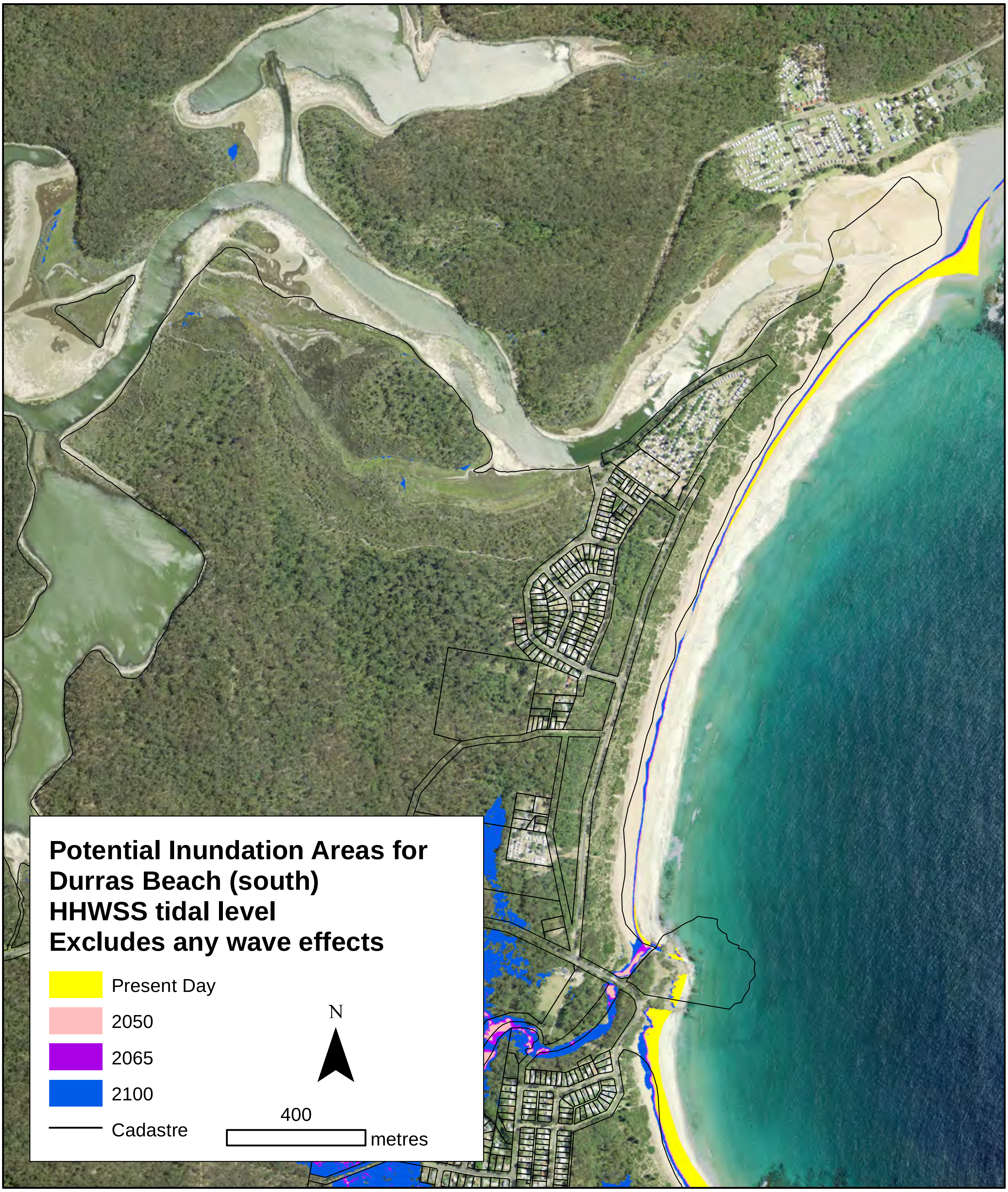
Table J-11: Tomakin Cove

Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
1	1	6.6	8.7	8.5	8.6	4.9	8.5	8.6	8.4
1	2	8.8	9.2	9.1	9.1	10.0	9.1	9.0	9.2
1	3	9.2	9.3	9.3	9.2	9.1	9.2	9.2	9.3
1	4	10.9	9.8	10.2	10.7	10.8	10.6	10.5	10.7
1	5	9.8	10.6	10.4	10.0	9.5	10.0	10.1	9.9
1	6	7.0	9.7	9.8	9.6	6.7	9.4	9.5	9.0
2	1	10.7	9.8	9.8	9.8	8.0	9.7	9.8	9.6
2	2	9.8	9.7	9.7	9.9	9.8	10.0	9.9	10.0
2	3	9.2	9.3	9.2	9.1	8.8	9.1	9.1	9.2
2	4	9.6	9.1	9.1	9.3	9.3	9.3	9.3	9.4
2	5	8.3	8.8	8.9	8.7	8.6	8.5	8.5	8.6
2	6	8.4	7.9	7.9	7.9	7.8	7.8	7.8	8.0
3	1	7.9	8.8	8.9	8.7	9.0	8.6	8.6	8.7
3	2	9.8	9.6	9.6	9.6	8.7	10.2	10.0	9.6
3	3	9.2	10.2	9.9	10.1	10.7	9.6	9.9	10.3
3	4	9.9	11.8	11.3	10.7	10.5	11.1	10.9	10.7
3	5	11.0	13.5	13.5	11.8	9.5	12.5	12.2	11.4

Table J-12: Broulee Beach

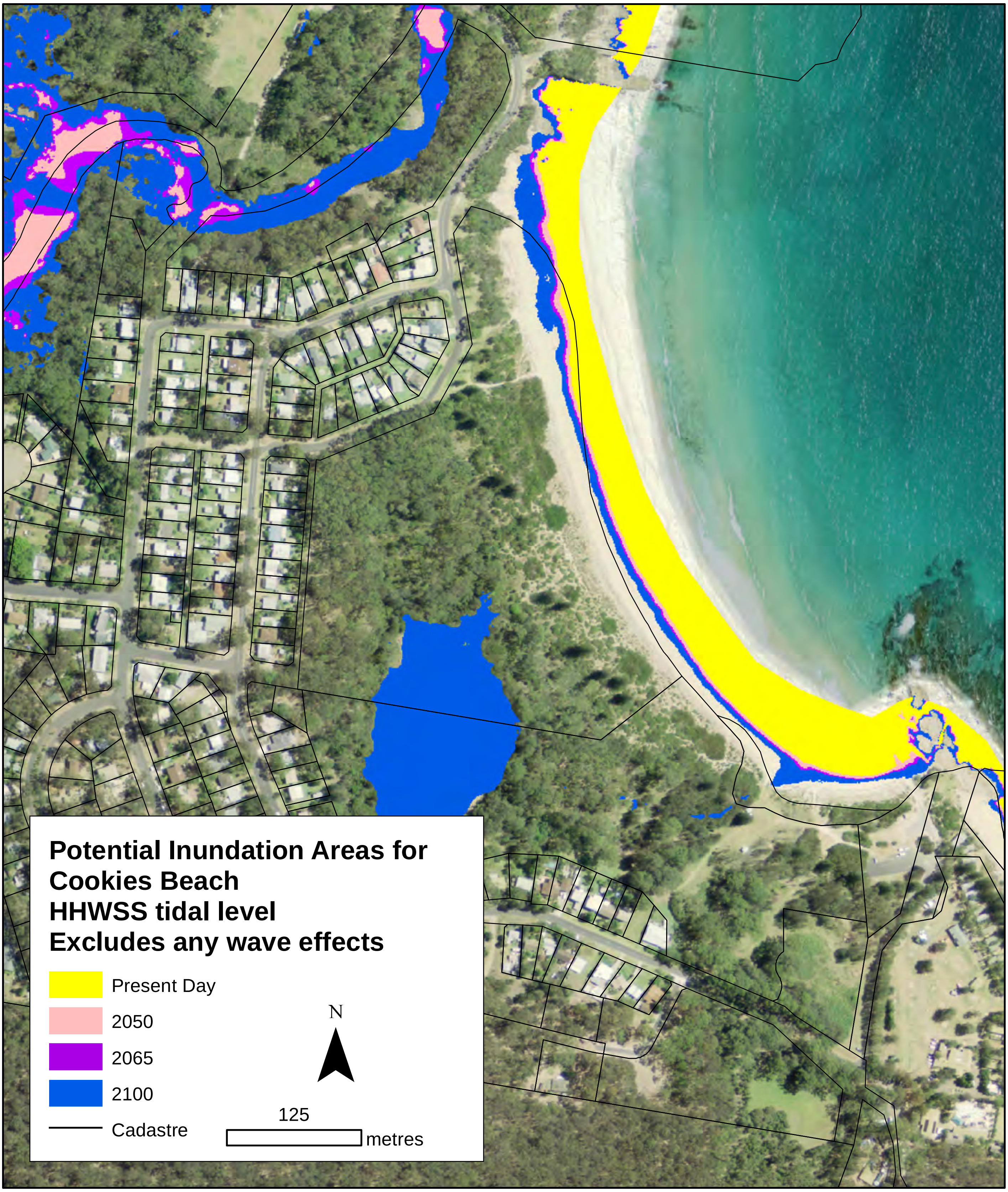
Block	Profile	1% encounter probability				5% encounter probability			
		2017	2050	2065	2100	2017	2050	2065	2100
M	1	5.9	7.2	7.0	6.6	5.8	8.4	8.3	7.7
M	2	5.9	9.8	9.6	9.8	8.8	5.9	8.1	10.1
M	3	6.1	6.9	7.0	7.3	5.4	7.6	7.3	7.1
M	4	5.5	5.4	5.3	5.3	6.3	7.1	6.9	6.2
M	5	6.1	5.5	5.3	5.4	5.3	6.2	6.1	6.0
M	6	6.4	7.9	7.8	7.1	5.8	6.8	6.9	7.0
M	7	6.4	7.1	7.1	7.1	6.0	8.6	8.4	7.7
M	8	5.7	8.9	8.7	8.1	5.2	8.1	8.1	8.2
M	9	6.8	7.1	7.1	7.6	5.9	8.7	8.6	7.9
M	10	6.3	7.4	7.6	7.5	6.9	7.6	7.5	7.1
N	1	8.0	8.9	9.2	9.0	7.7	7.3	7.6	8.0
N	2	7.7	10.1	10.3	10.4	7.4	10.1	10.0	9.6
N	3	15.4	17.7	17.8	16.6	6.1	15.0	15.5	16.0
N	4	6.5	9.5	9.9	9.6	8.4	6.7	7.0	7.6
N	5	12.6	11.6	11.6	12.0	7.9	12.6	12.4	11.9
N	6	11.1	11.0	11.0	11.0	8.2	11.3	11.3	11.0
N	7	10.4	13.1	13.1	12.1	9.2	12.3	12.3	12.2
N	8	13.3	11.6	11.5	11.8	8.2	11.9	11.9	11.8
N	9	12.3	13.6	13.7	13.5	12.7	12.8	13.0	13.1
N	10	12.2	12.7	12.4	12.1	10.5	13.2	13.1	12.6
N	11	12.7	11.1	11.4	12.1	13.9	11.1	11.1	12.0
N	12	10.9	10.6	10.7	10.9	13.9	10.1	10.2	10.9
N	13	11.7	12.6	13.3	11.9	17.4	9.9	10.3	12.0
N	14	12.2	14.1	14.4	13.6	13.9	11.8	12.3	13.1
N	15	15.1	11.1	11.4	11.8	9.2	10.3	10.3	11.9
N	16	12.0	11.8	11.7	11.7	8.8	11.7	11.7	11.7
N	17	13.0	12.1	12.0	12.4	15.4	12.0	12.0	12.7
N	18	13.9	12.2	12.4	12.8	9.9	12.1	12.2	12.7
N	19	11.9	11.2	11.3	11.6	11.2	11.9	11.8	11.7
N	20	13.0	11.9	11.8	12.4	14.2	12.4	12.3	12.6
O	1	12.0	11.8	12.0	12.2	11.1	12.8	12.5	12.1
O	2	12.0	10.0	10.1	11.4	14.1	10.7	10.8	11.8
O	3	11.8	9.3	9.7	11.3	14.8	10.0	10.4	11.9
O	4	11.5	9.8	10.1	11.7	15.2	10.4	10.7	12.2
O	5	12.2	11.5	11.4	12.3	15.2	11.6	11.7	12.7
O	6	14.0	12.0	12.4	13.5	16.0	13.1	13.1	13.8
O	7	18.5	14.4	14.4	15.3	8.2	15.2	15.2	15.5
O	8	4.5	5.7	6.2	7.7	5.2	8.1	7.3	7.2

Appendix K: Tidal Inundation Hazard Maps (Excludes Wave Effects)



Inundation areas are mapped based on the most recent year of LIDAR data available (2011). The mapping has been based on the ground elevation (the "all ground" LIDAR layer) and does not consider flow paths, flow velocities or loss of flow momentum. It does not include allowance for future landward recession of the beach face and assumes that the crest level of the seawall (if present) and the topography remain as they were from the 2011 LIDAR data. By 2050, 2065 or 2100 both of these assumptions may not be valid. Should the seawall/dune be allowed to fail then the landward extent of inundation may increase. WRL is not responsible for the accuracy of the LIDAR data. Local surveys by a registered surveyor are recommended to determine local inundation extents.

Figure K.1



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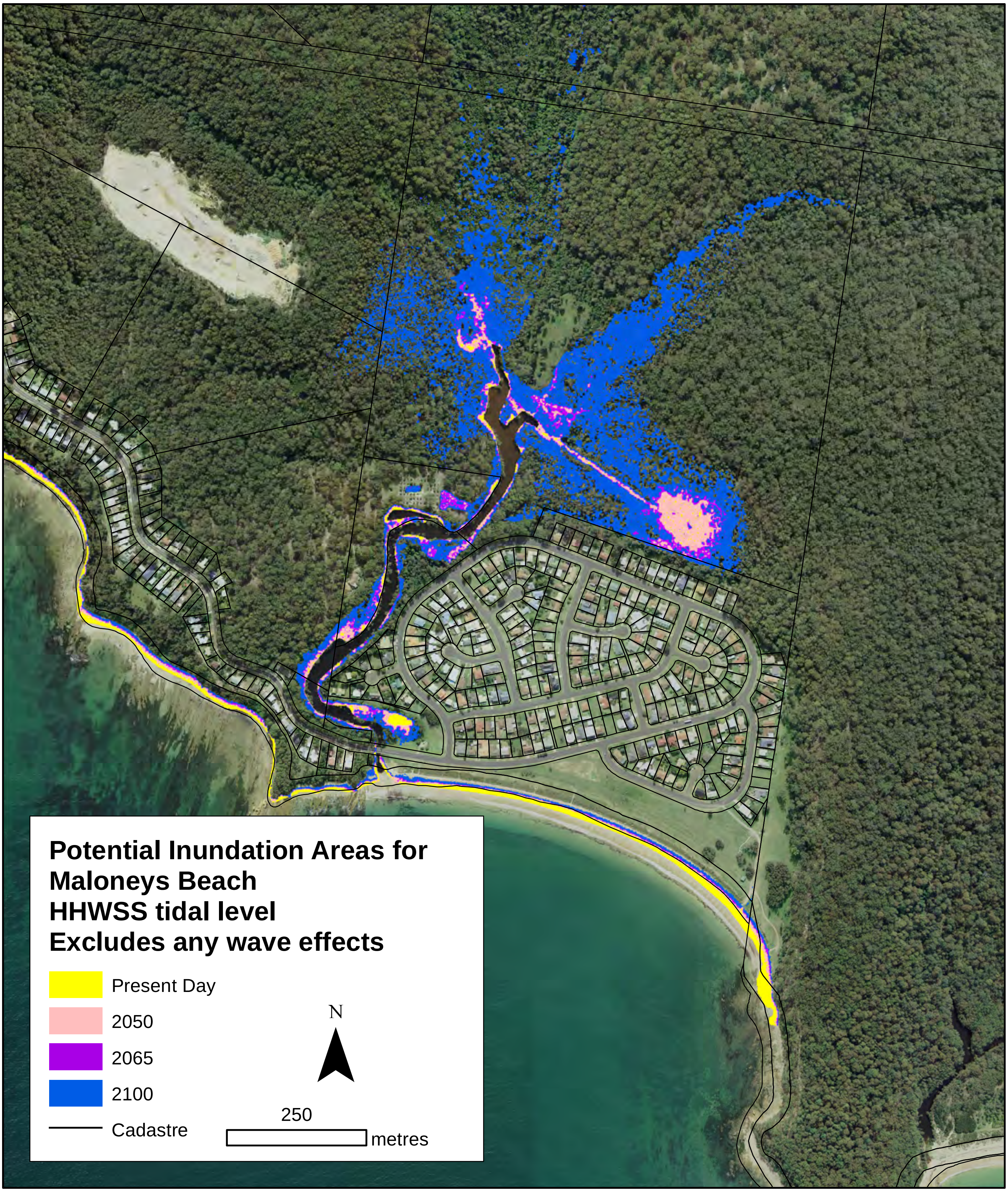

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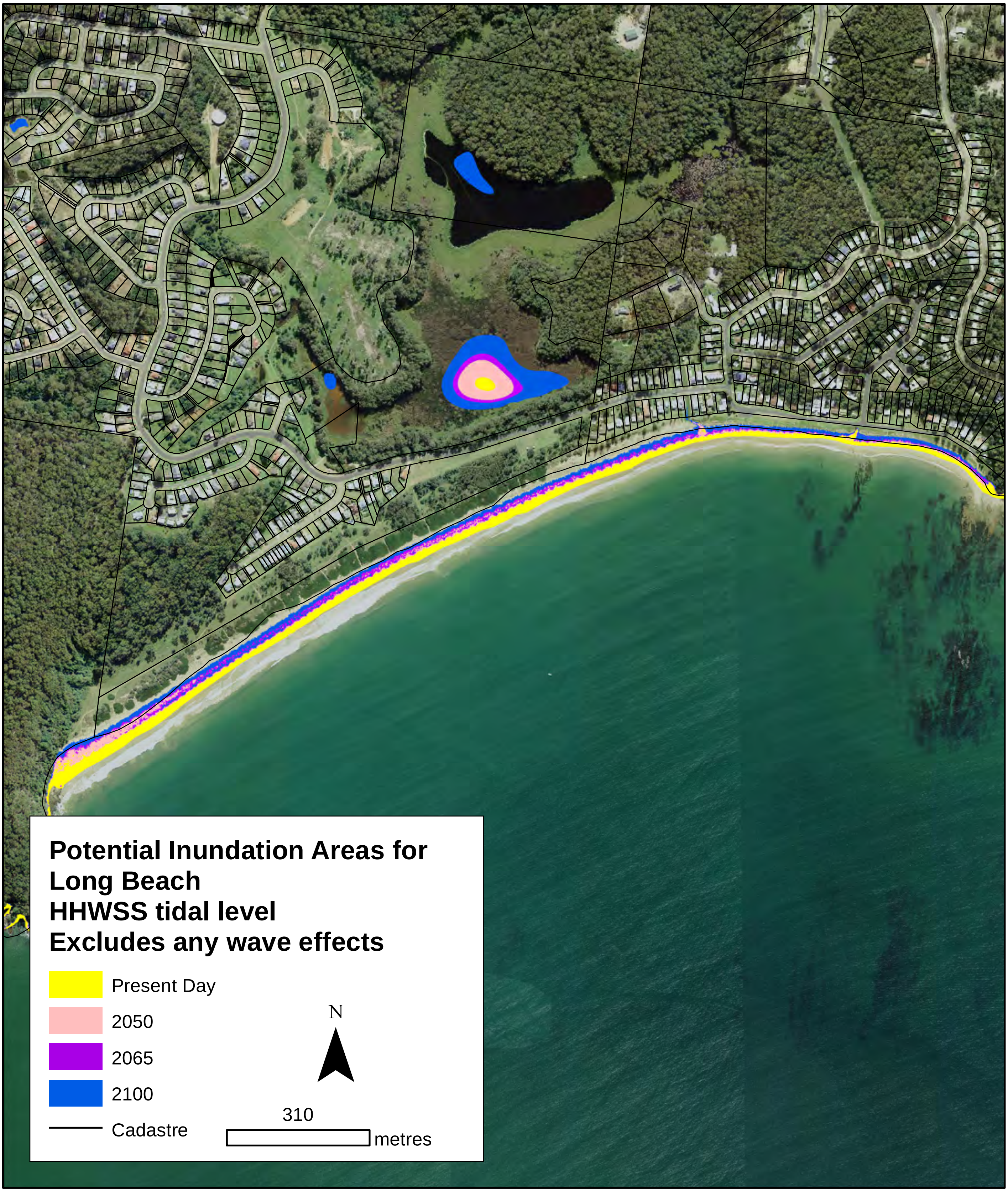

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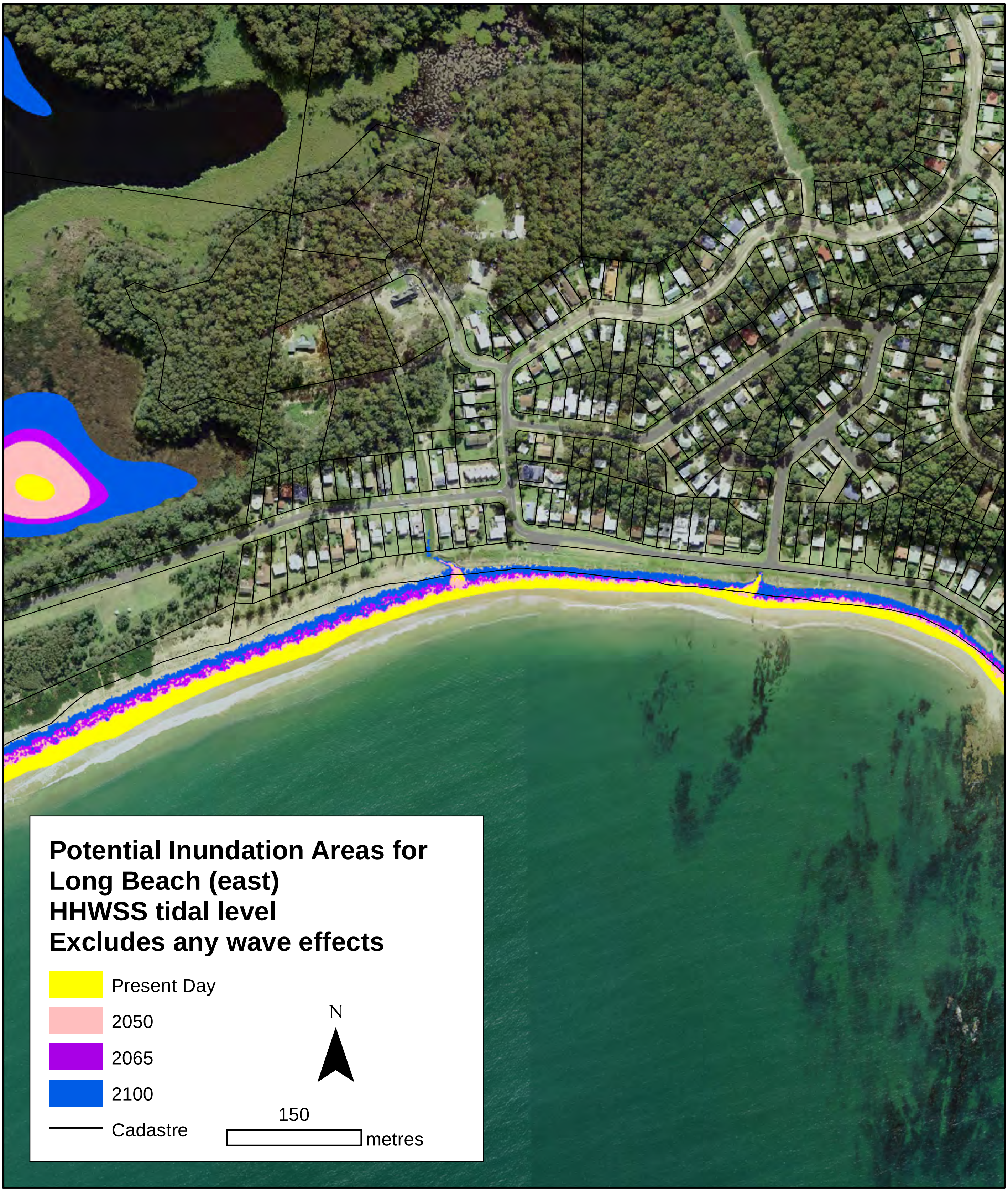

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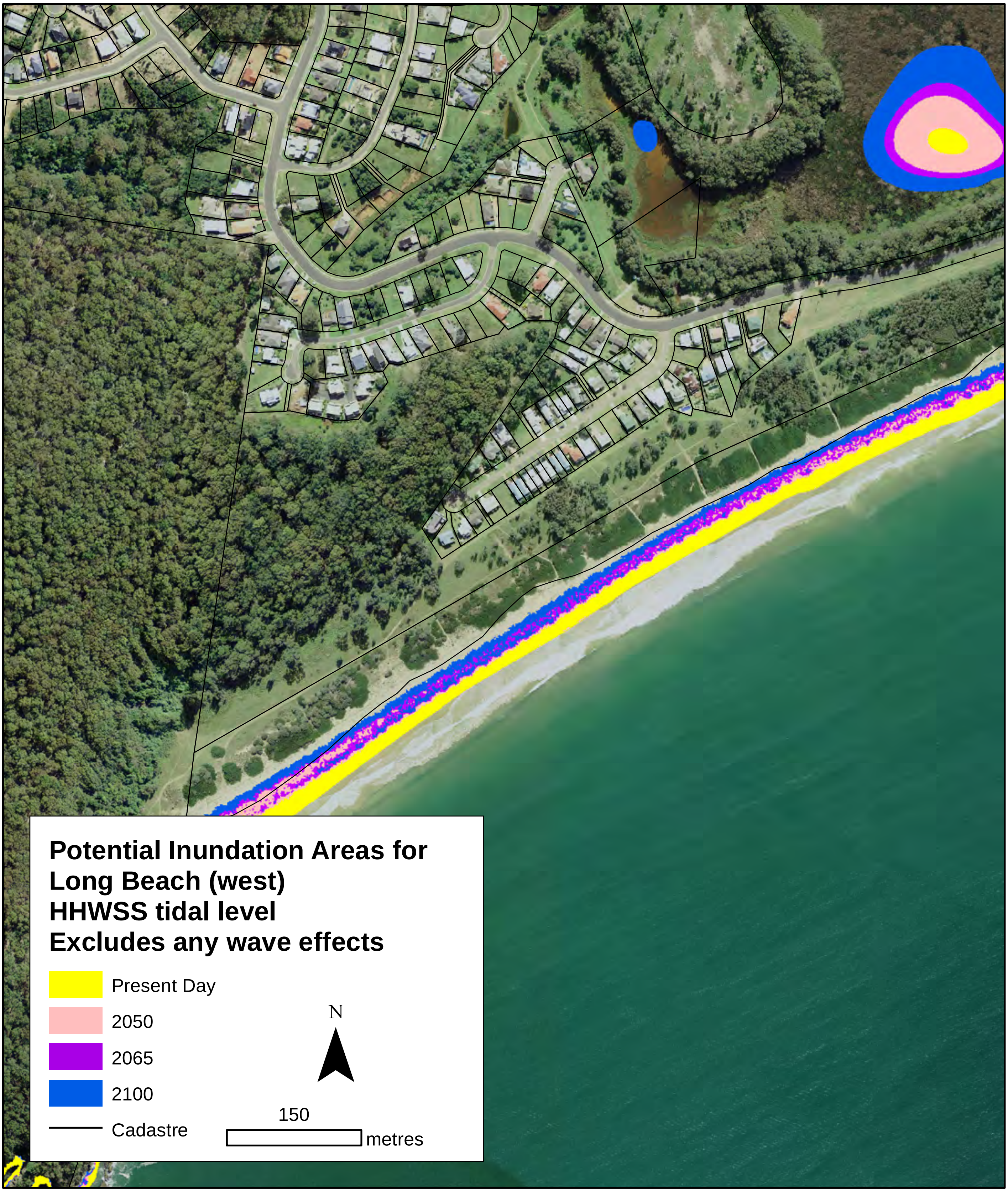

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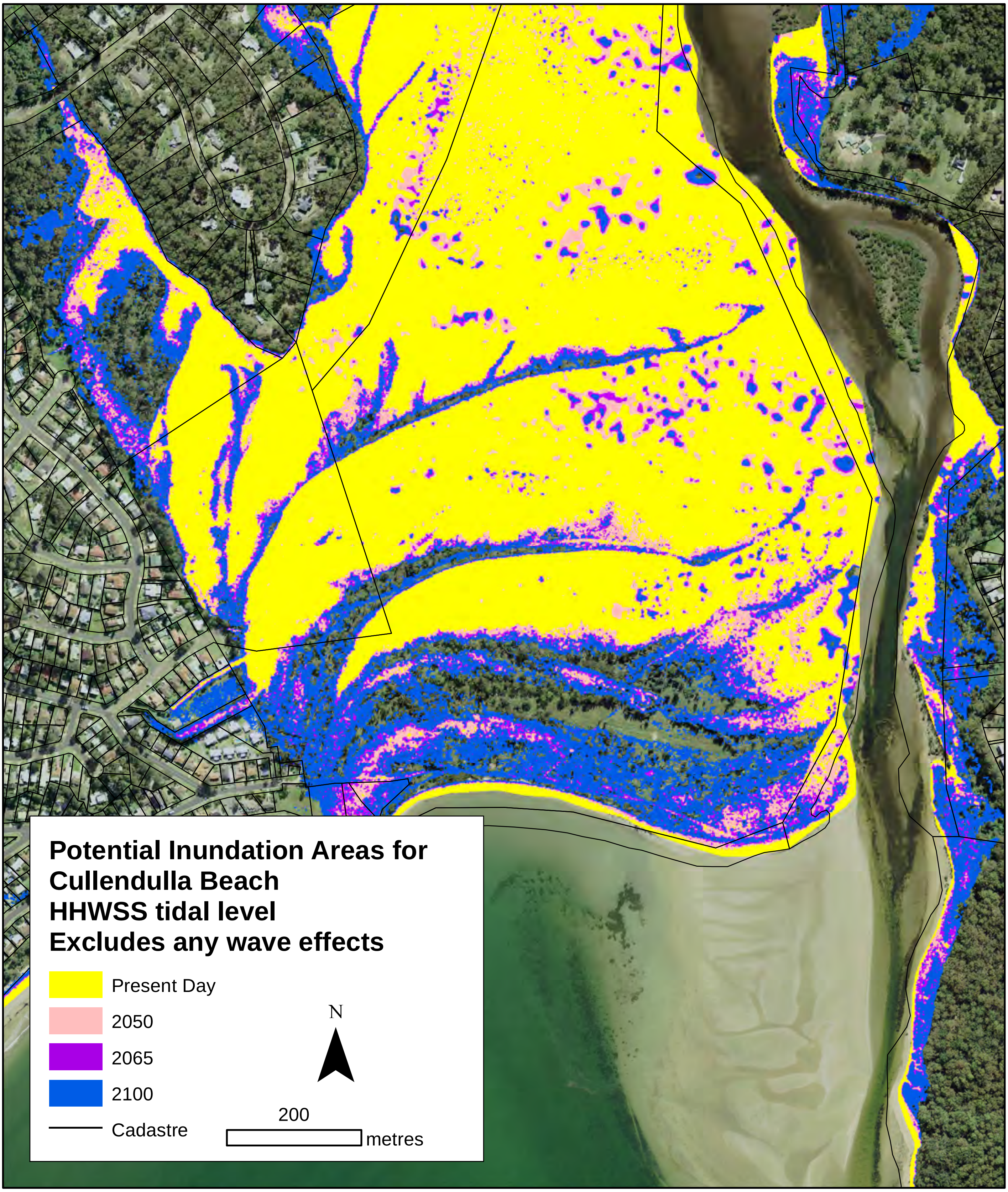

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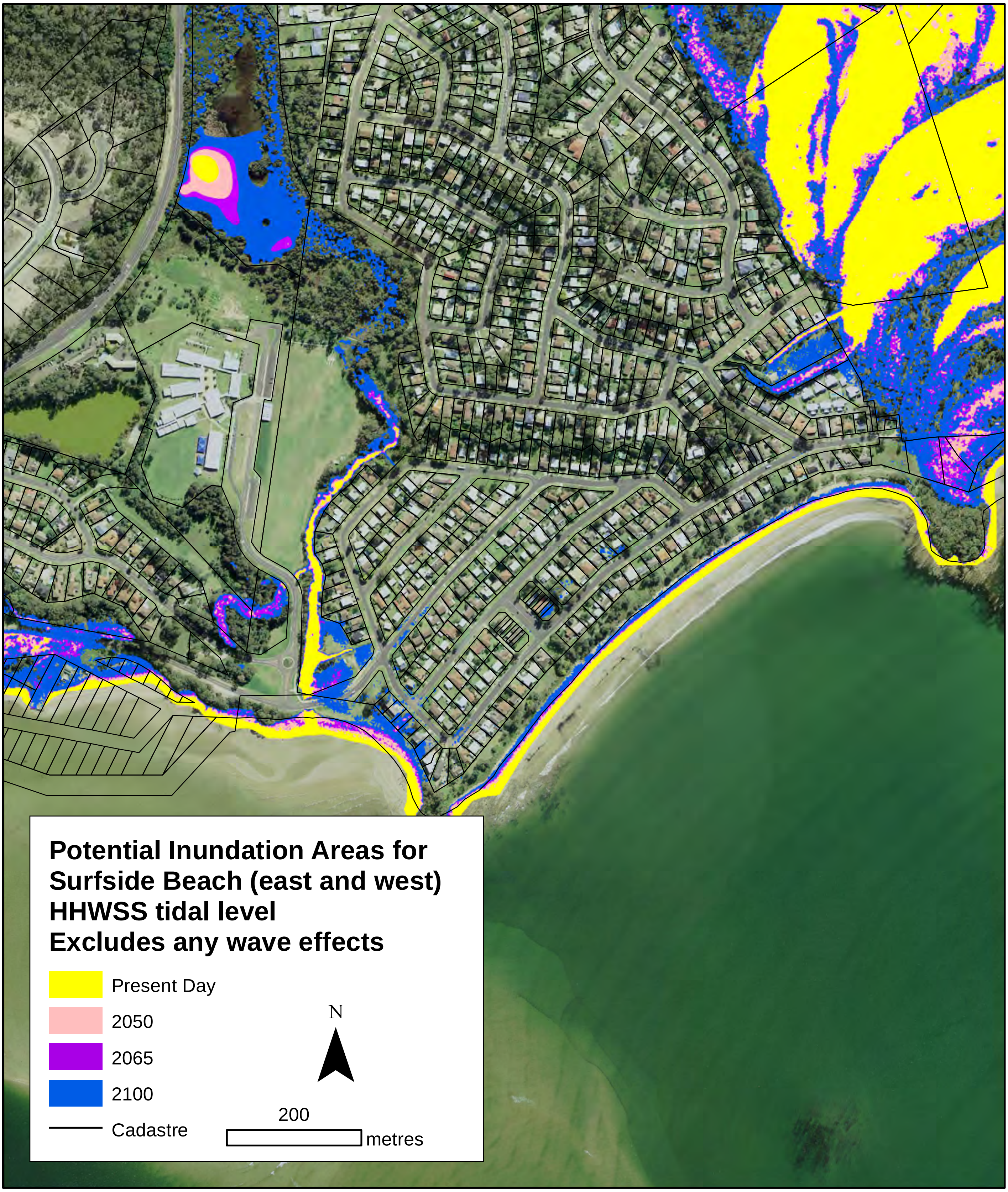

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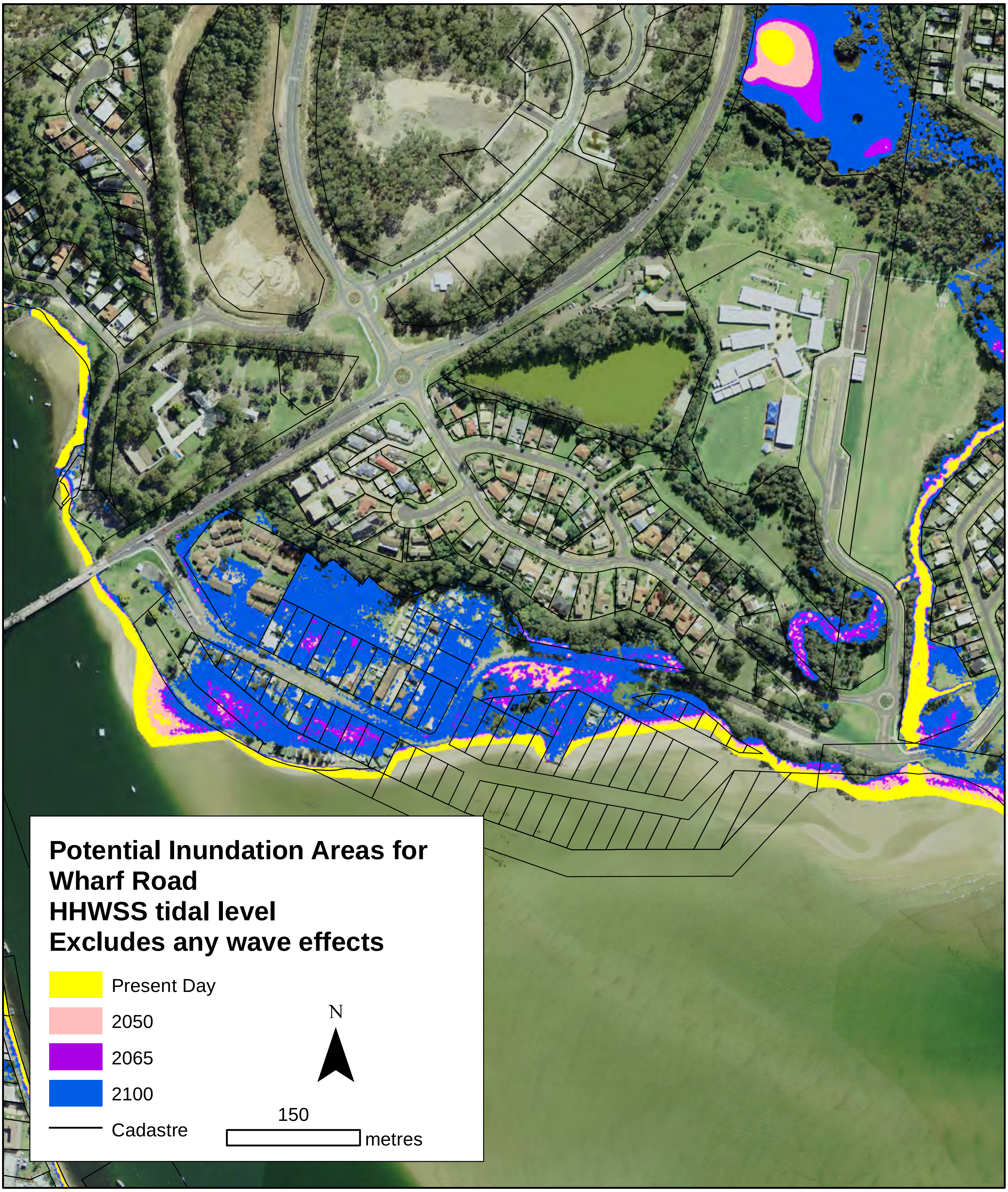

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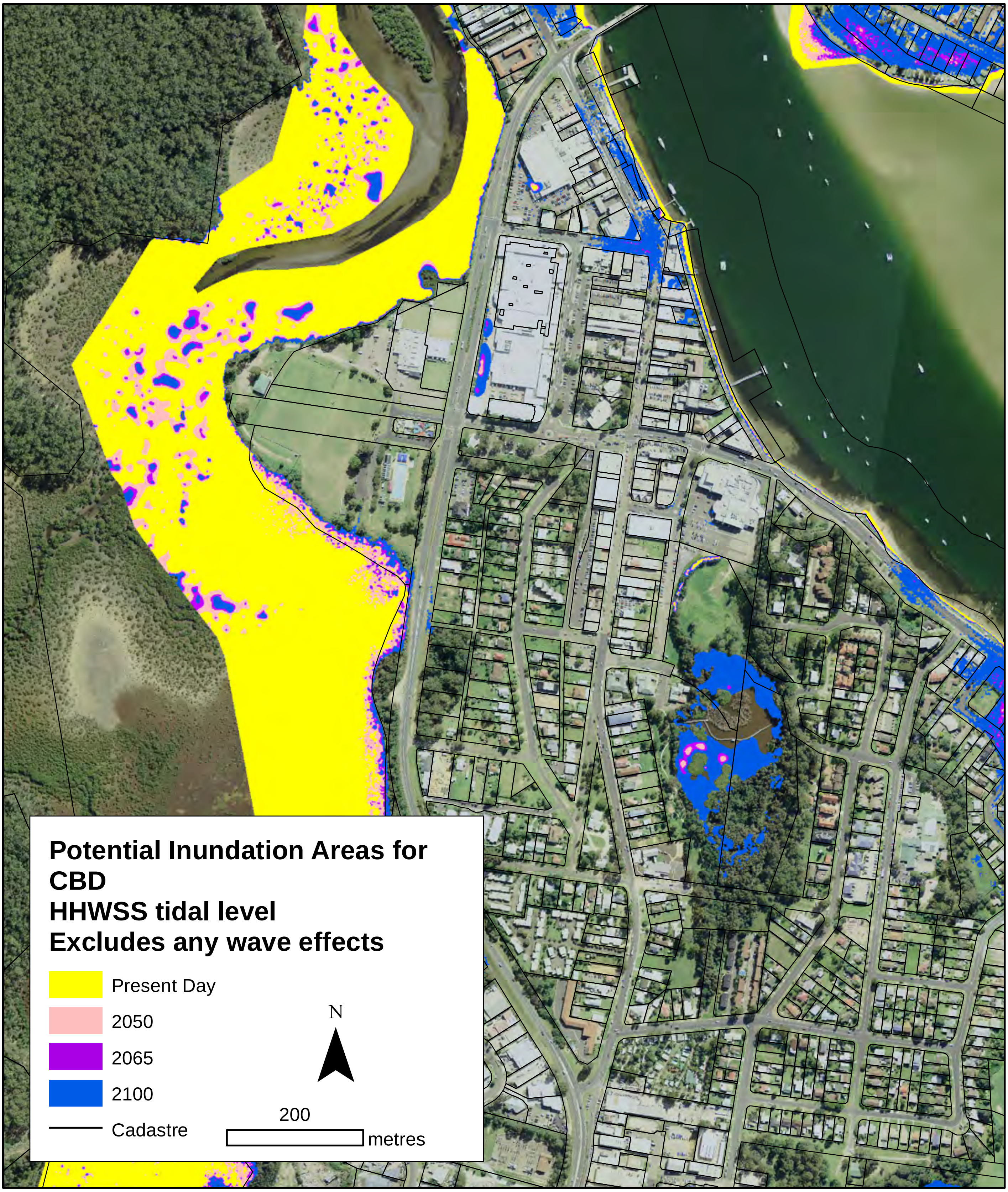

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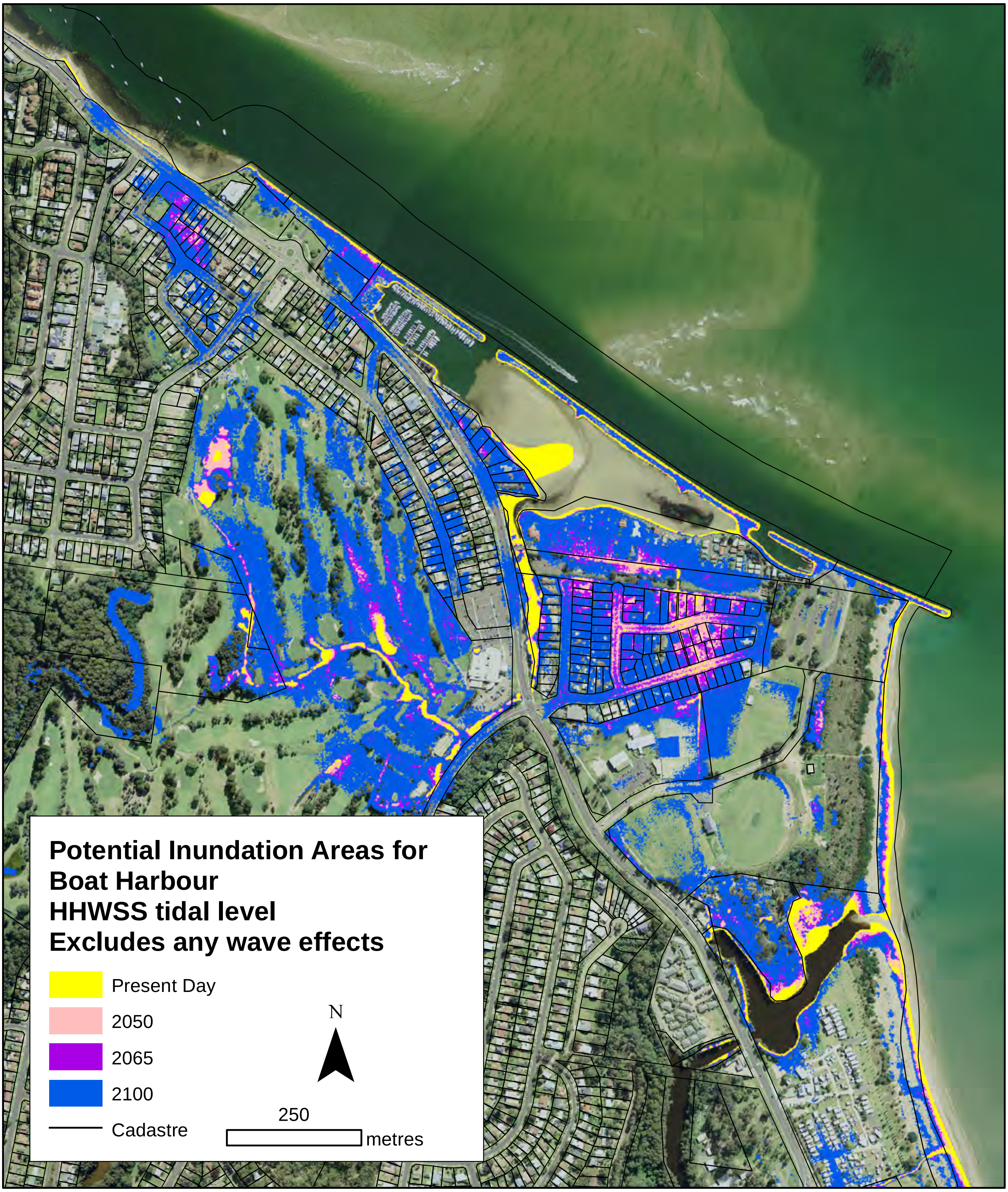

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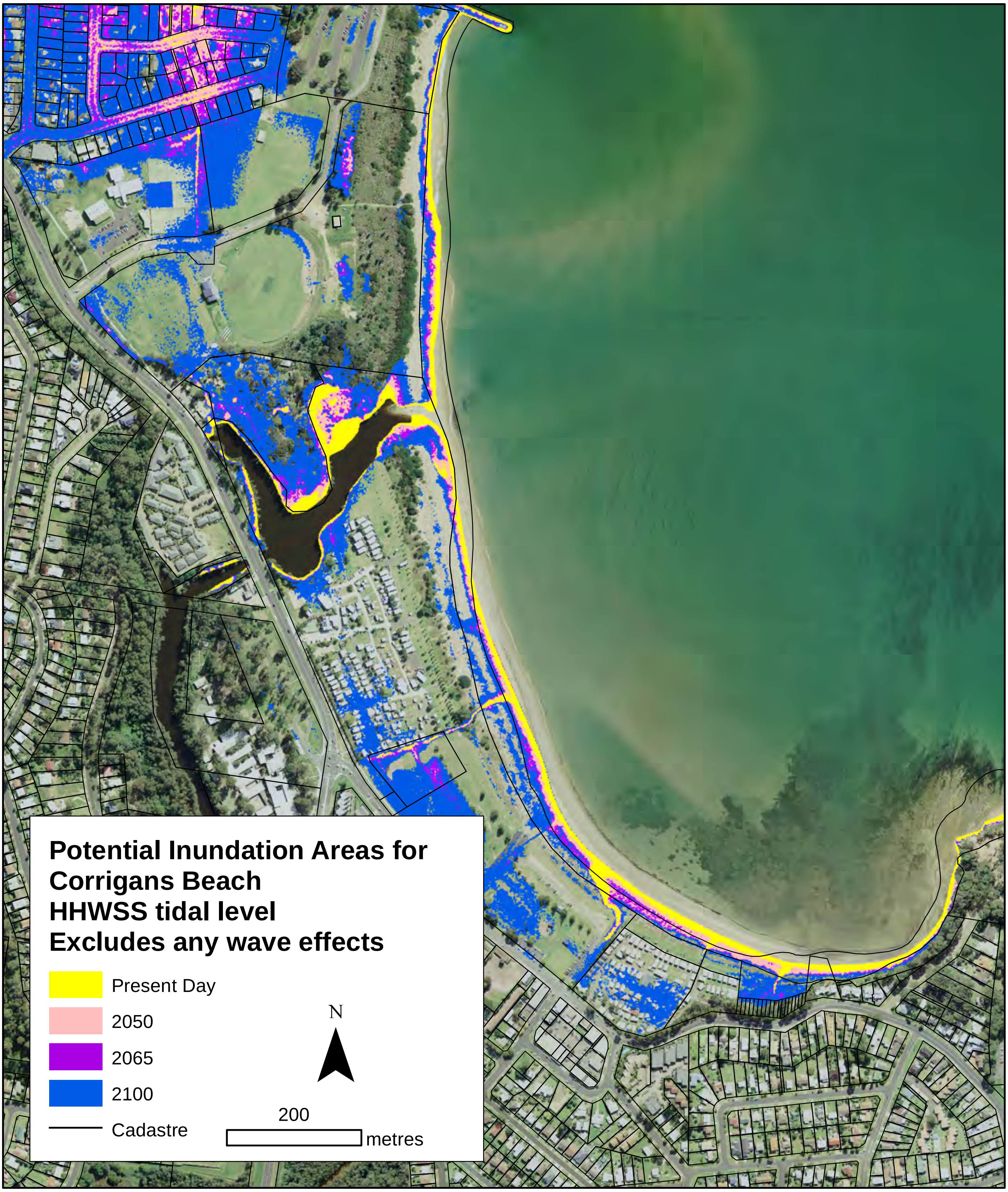

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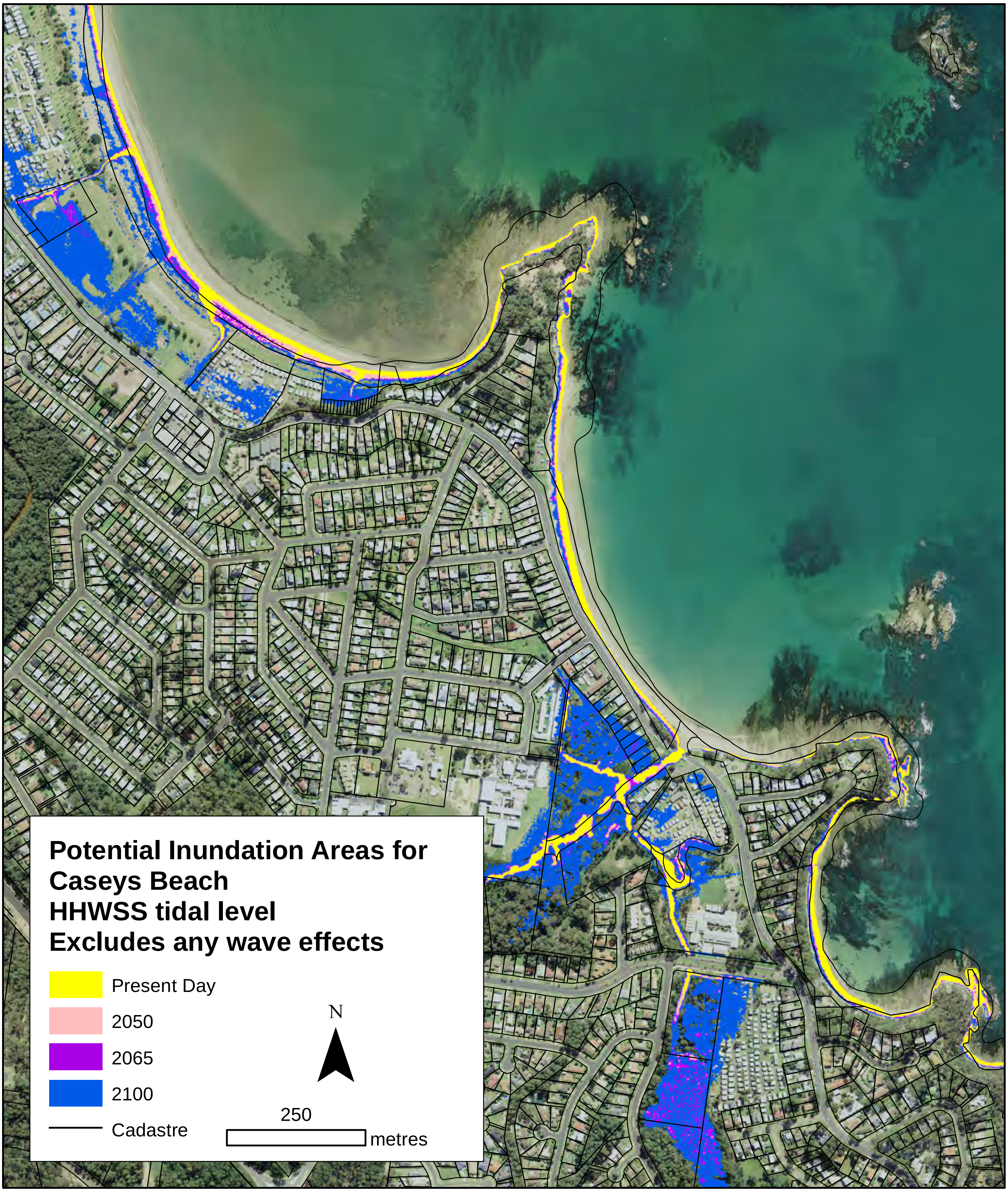

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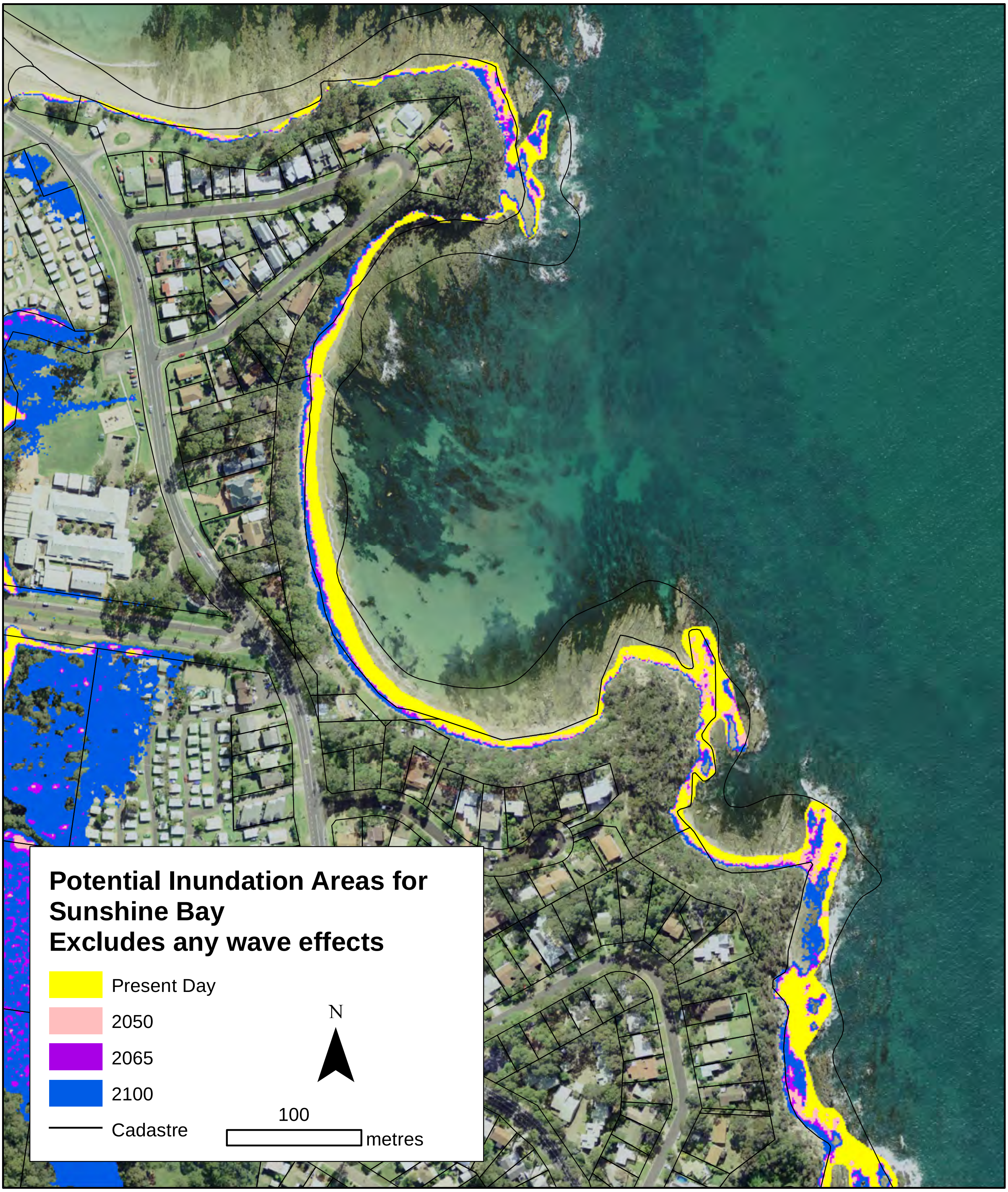

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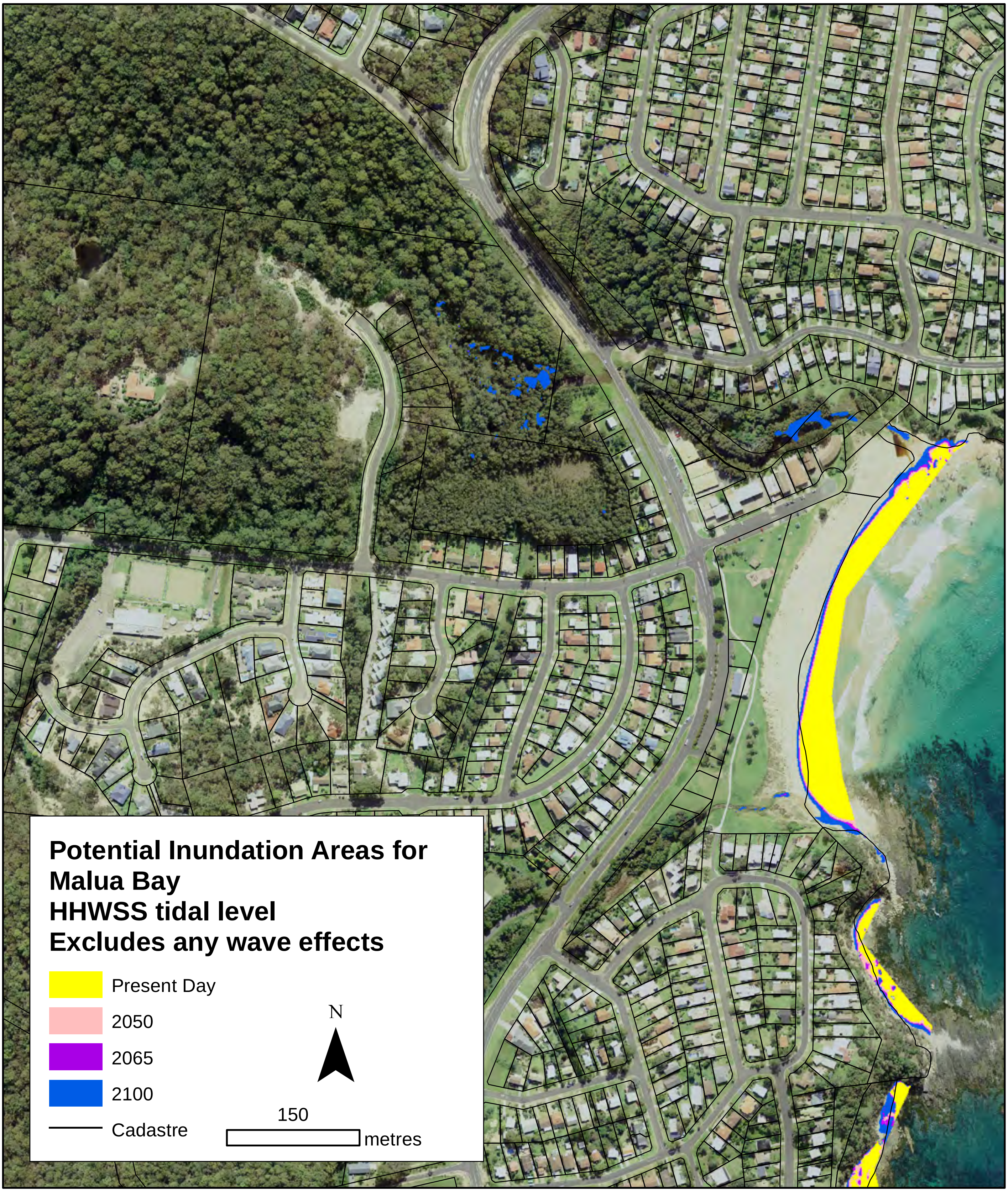

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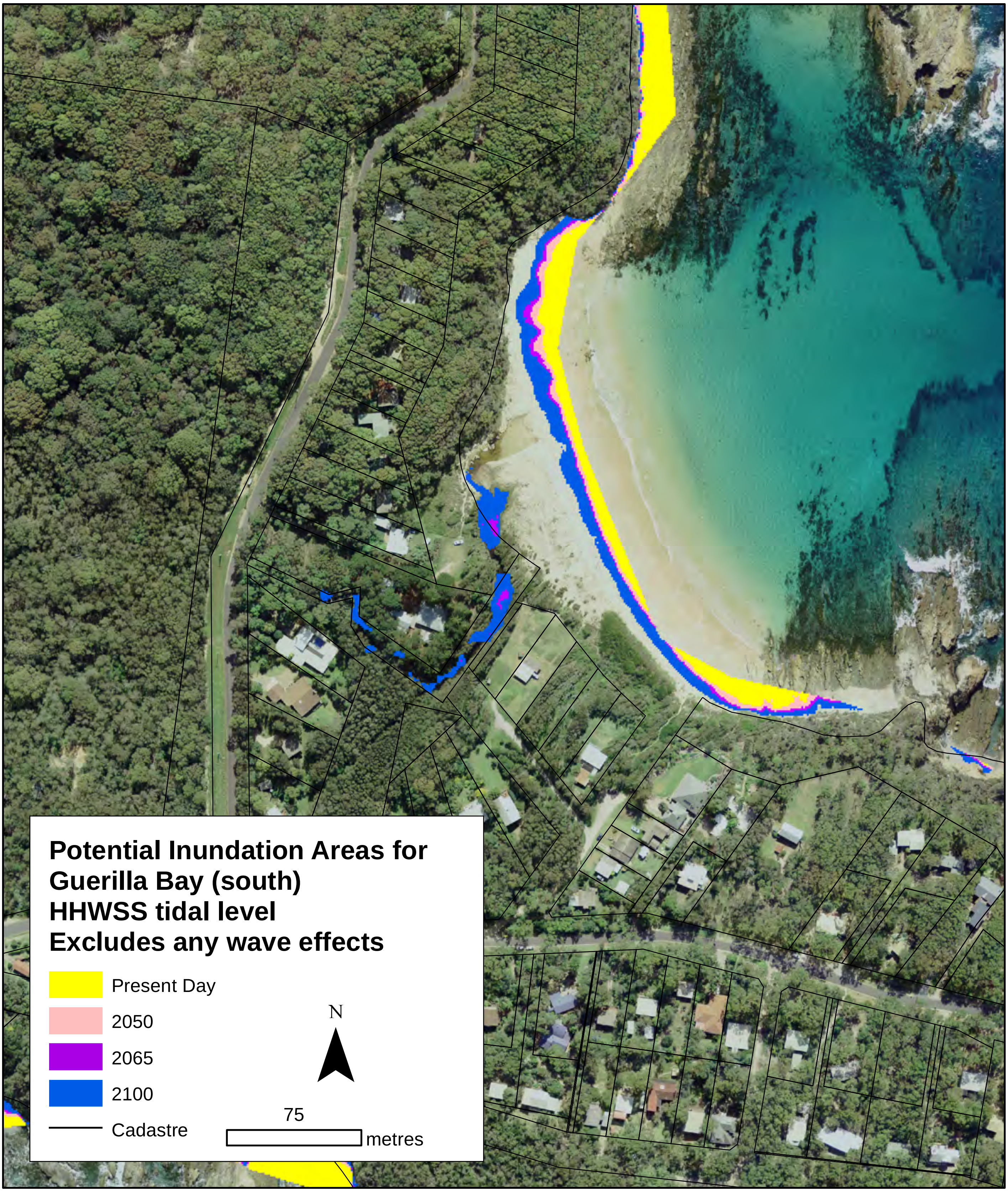

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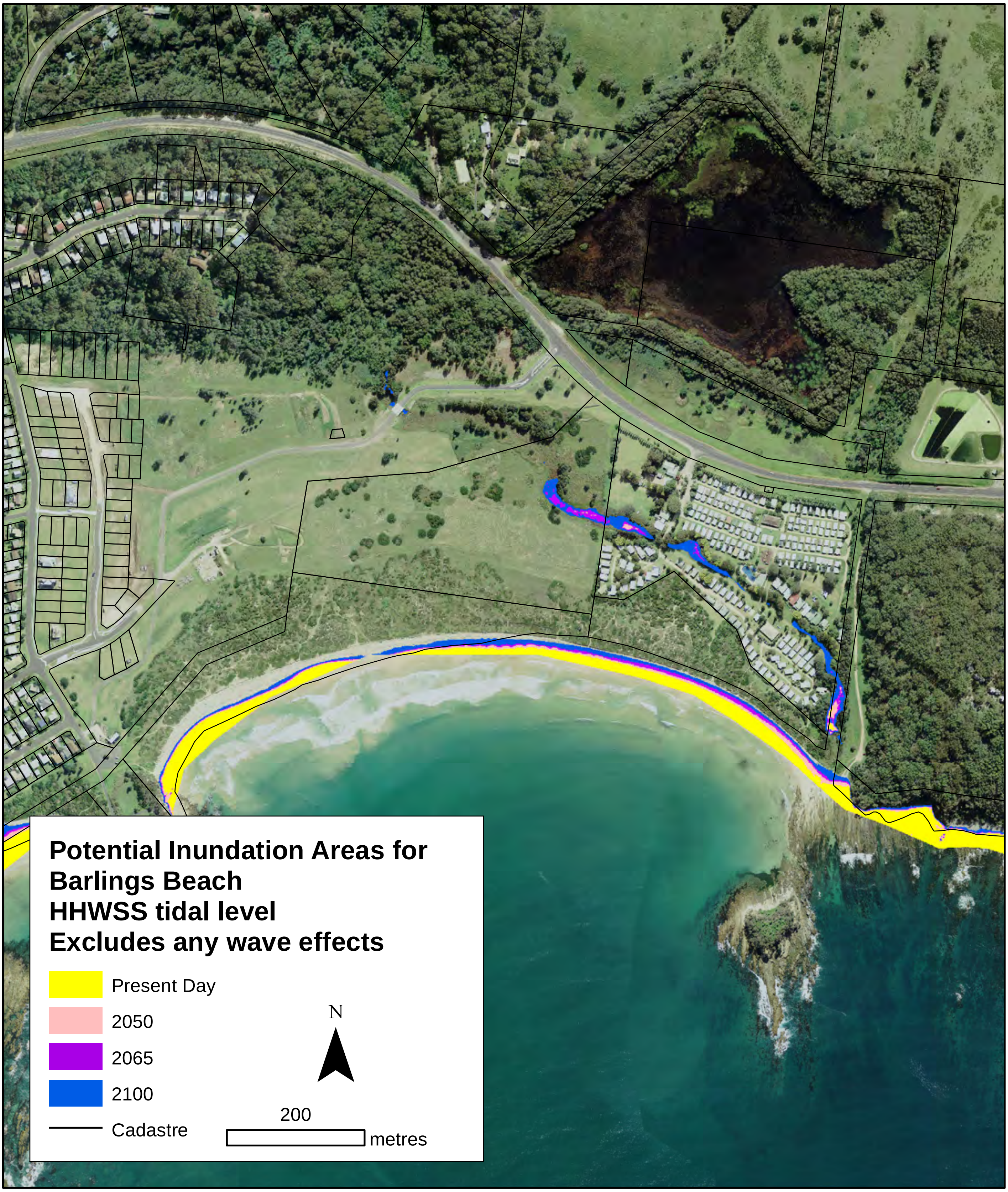

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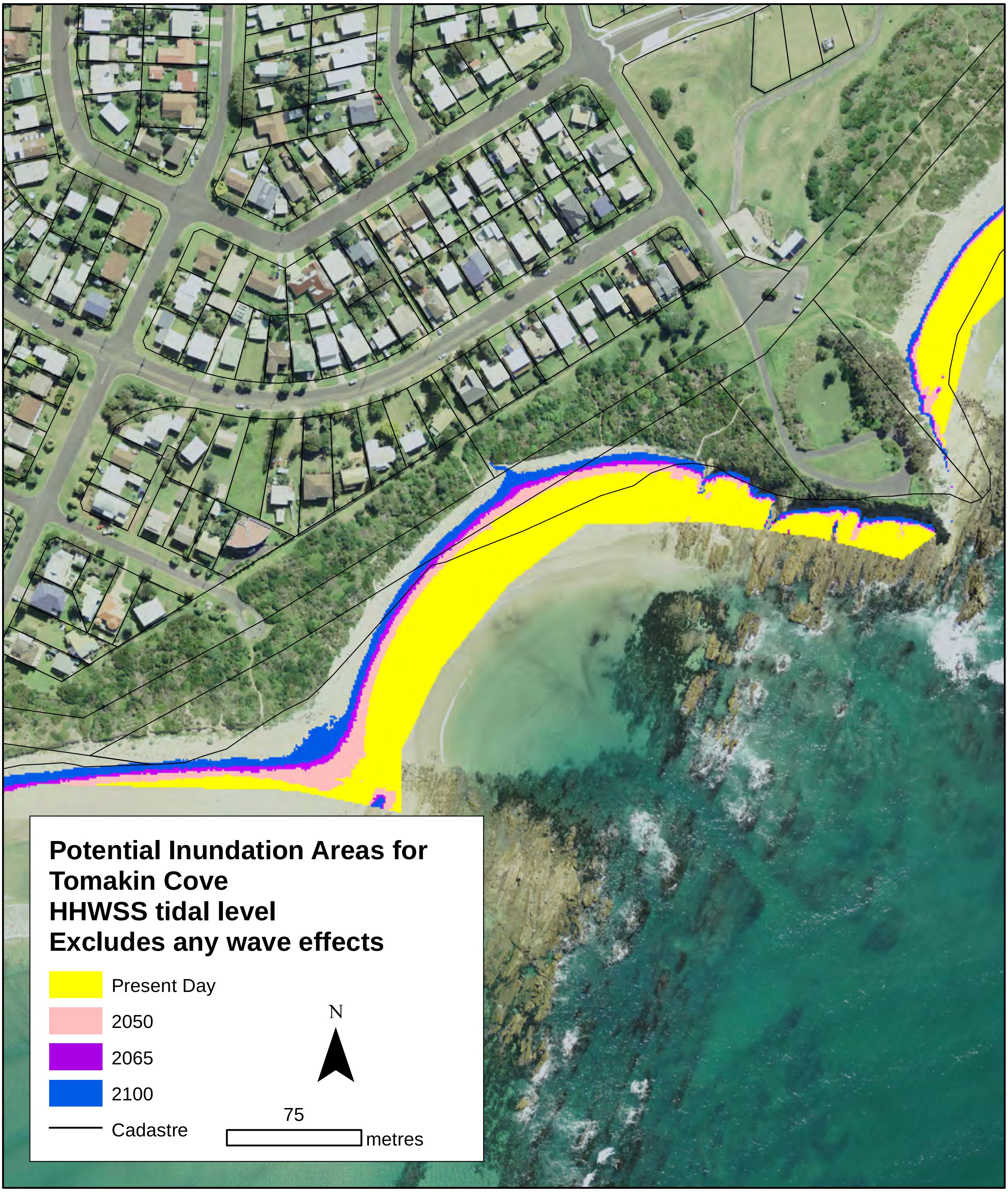

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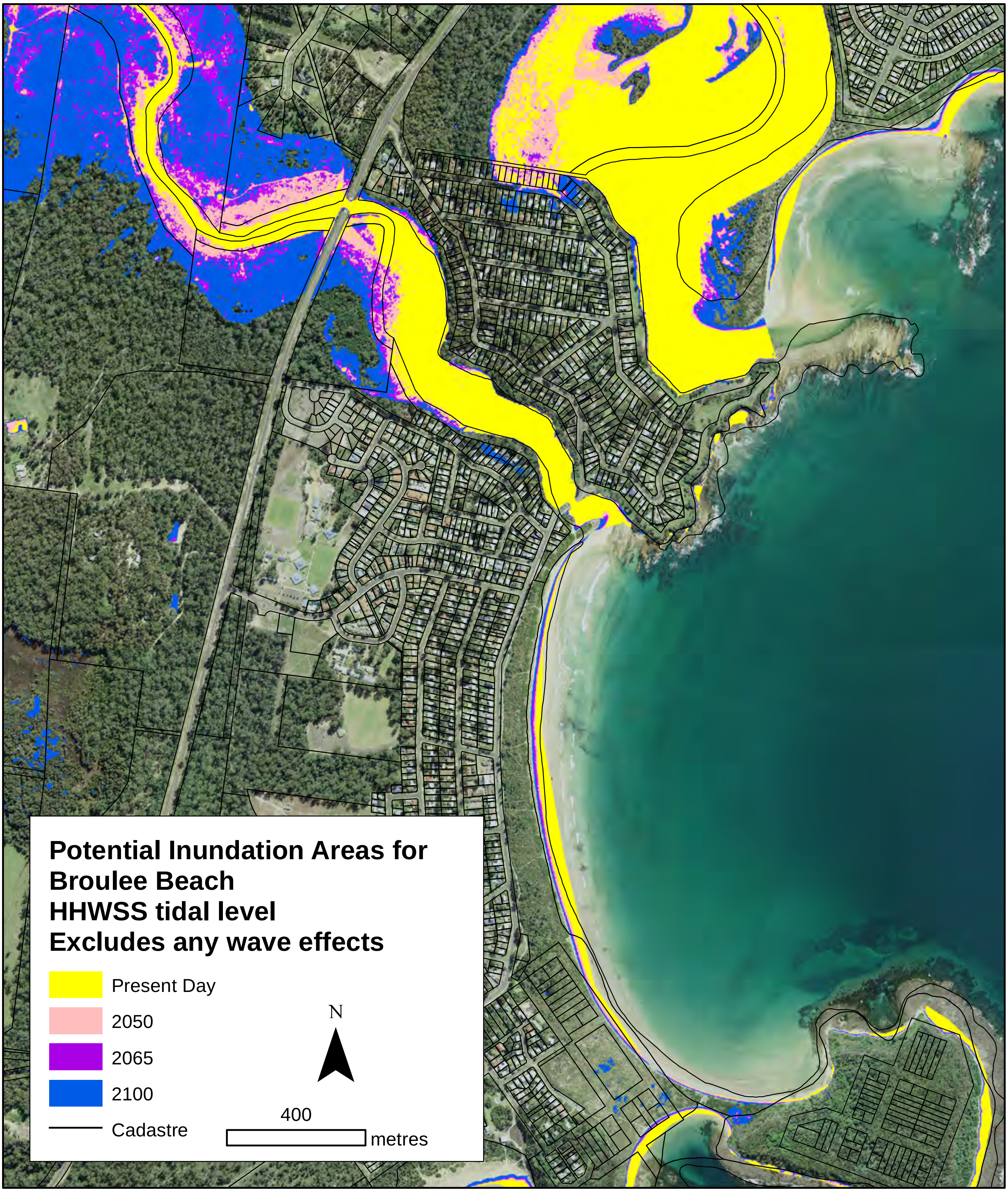

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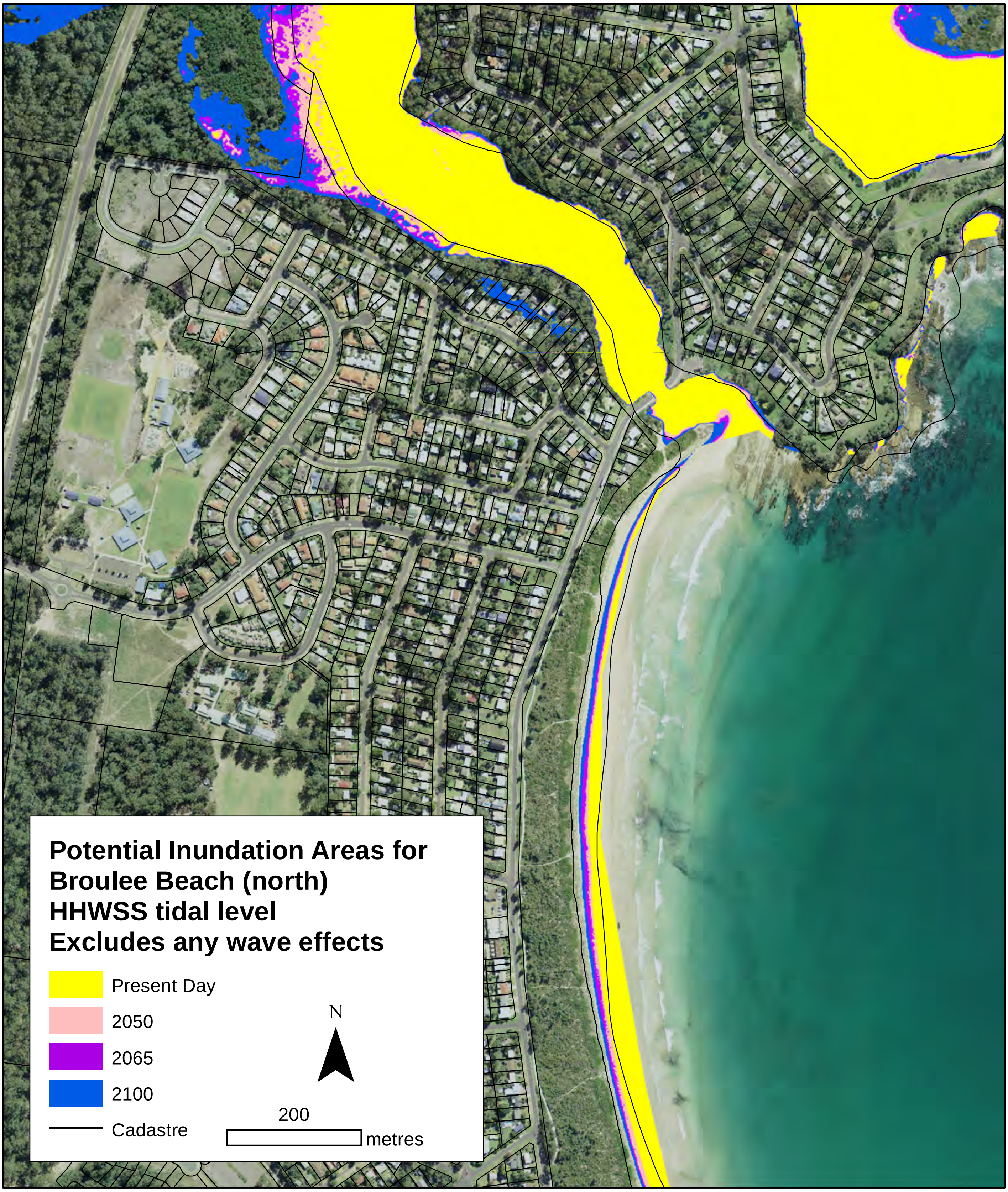

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Figure K.20

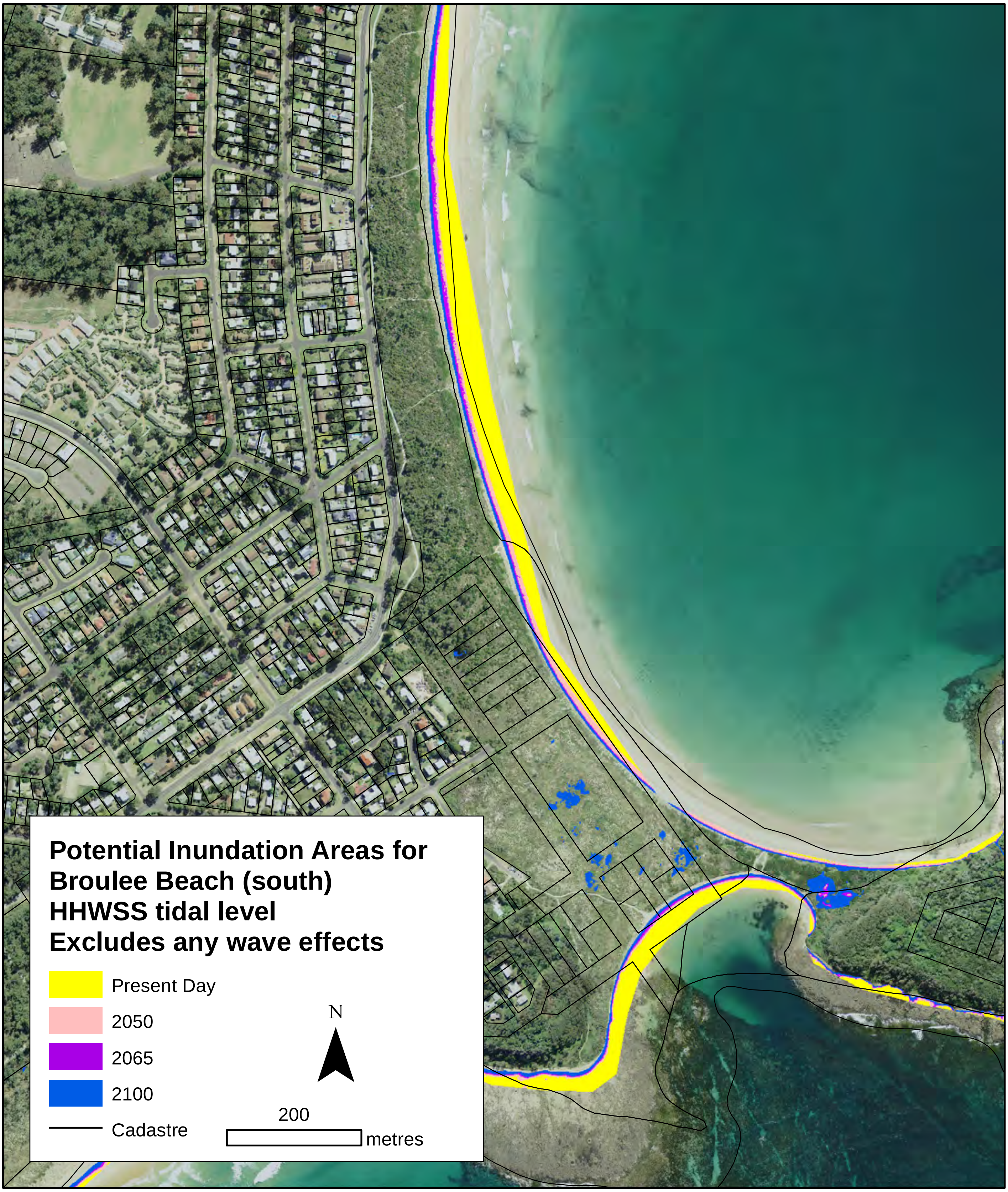

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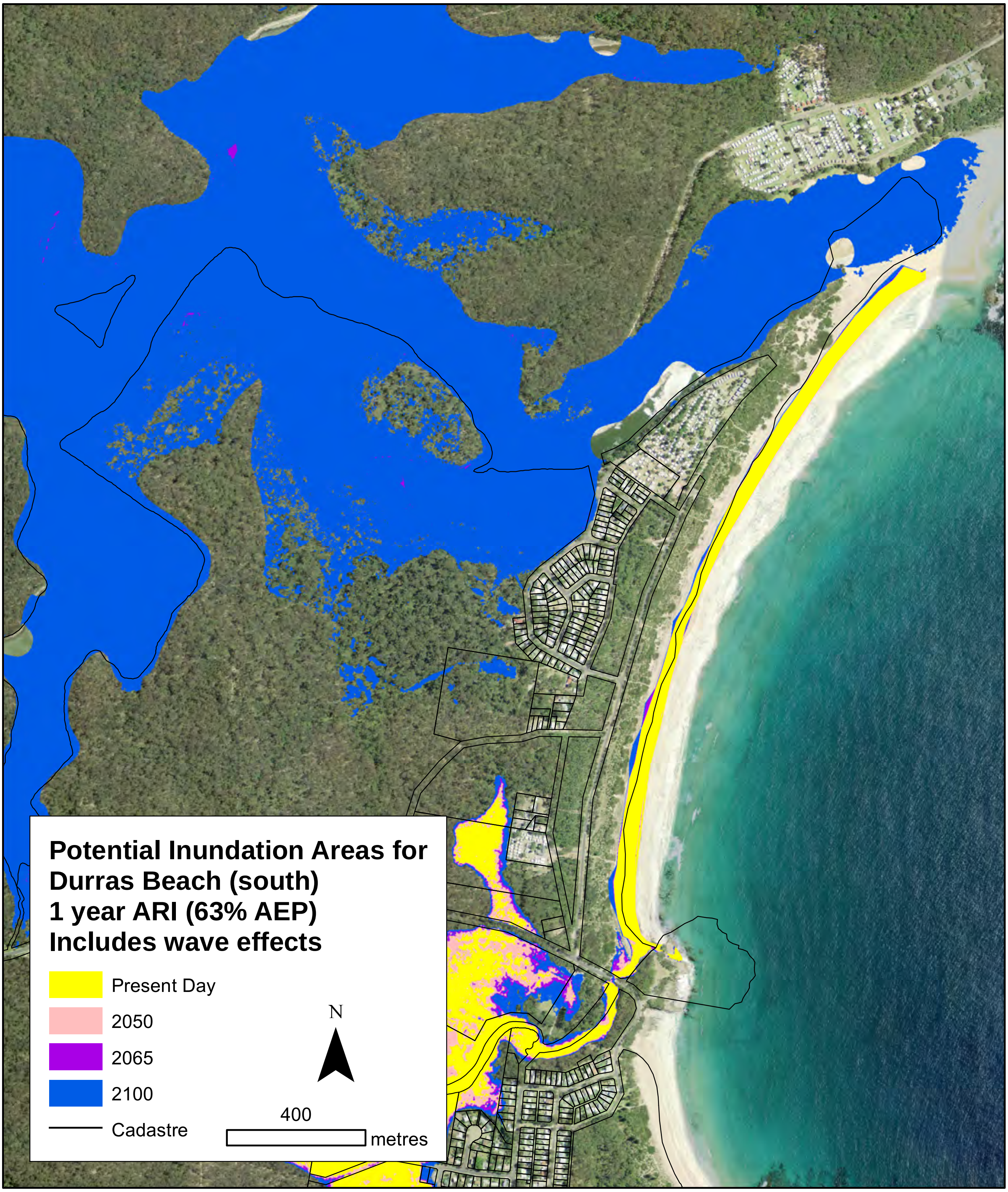
Figure K.21

Appendix L: Coastal Inundation Hazard Maps (Includes Wave Effects)



Inundation of the beachface and the area immediately landward of the dune crest is based on the most recent year of LIDAR data available (2011) and is in accordance with ESC's sea level rise policy and planning framework. It does not include allowance for future landward recession of the beach face and assumes that the crest level of the seawall (if present) and the topography remain as they were from the 2011 LIDAR data. By 2050, 2065 or 2100 both of these assumptions may not be valid. Should the seawall/dune be allowed to fail then the landward extent of inundation may increase. Inundation of low lying areas behind the beach is based on the most recent year of LIDAR data available (2011). The low lying inundation areas behind the beach are mapped based on the ground elevation (the "all ground" LIDAR layer) and do not consider flow paths, flow velocities, loss of flow momentum or wave propagation into creek areas. WRL is not responsible for the accuracy of the photogrammetry or LIDAR data. Local surveys by a registered surveyor are recommended to determine local inundation extents.

Figure L.1

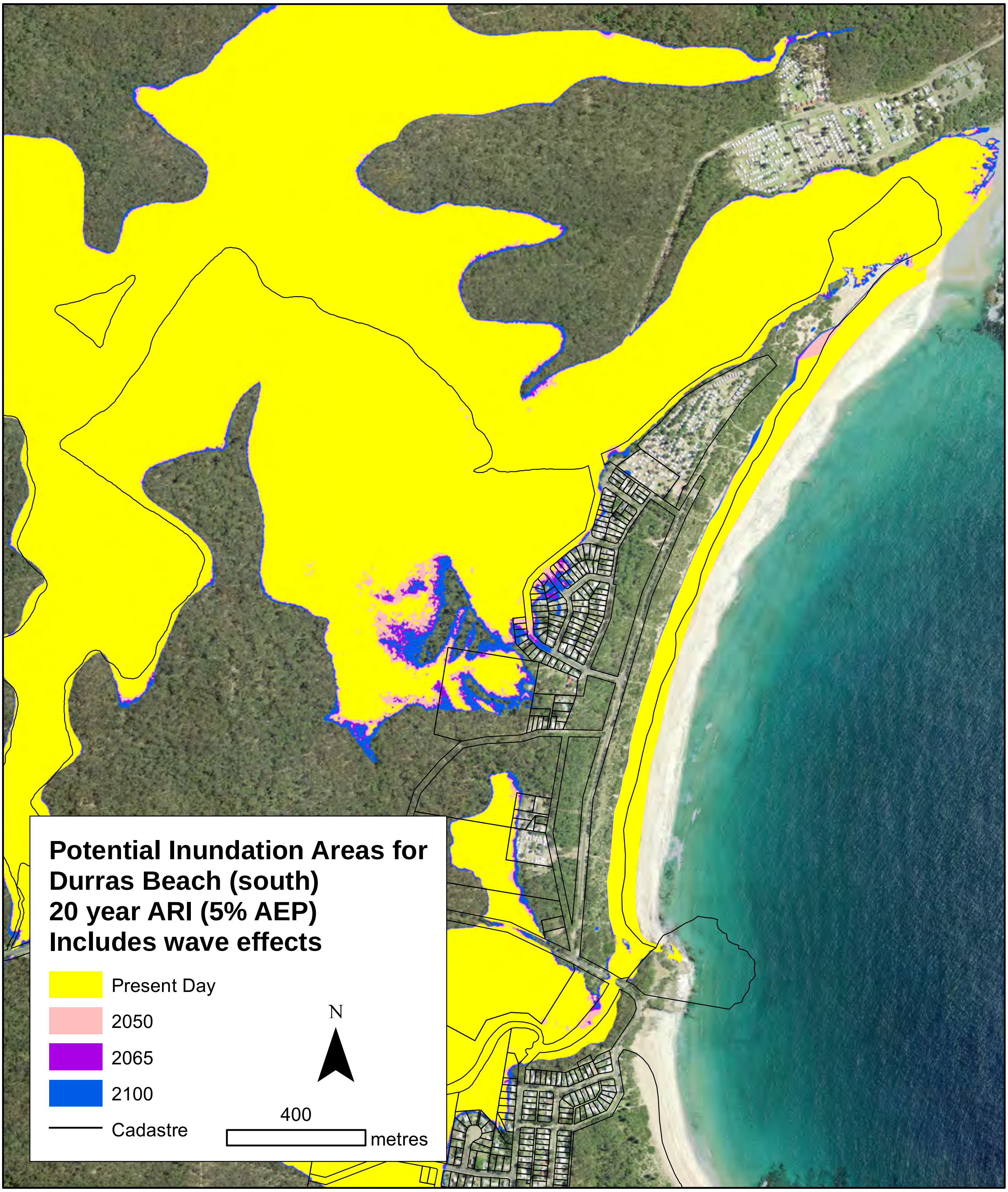


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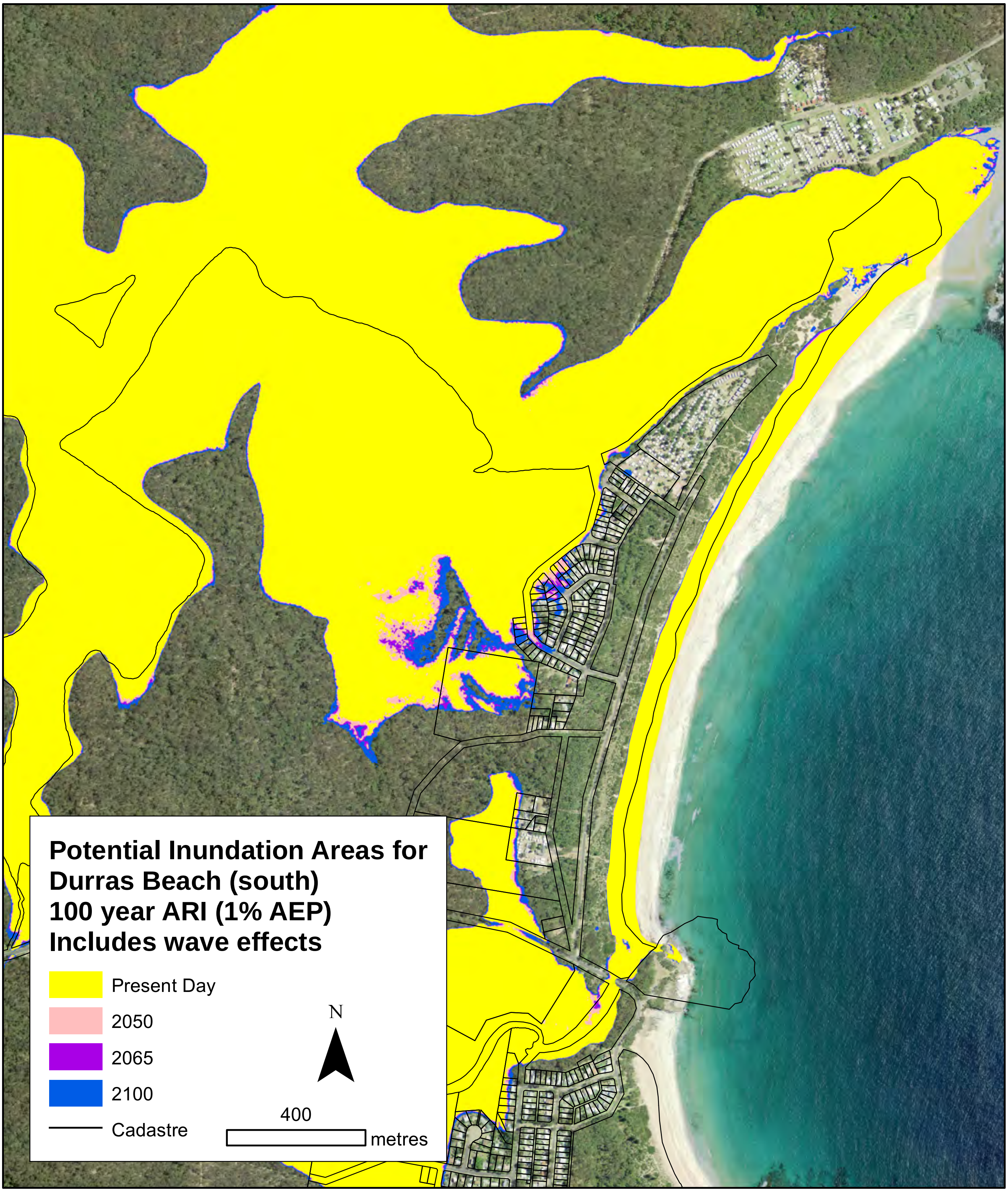
Figure L.2



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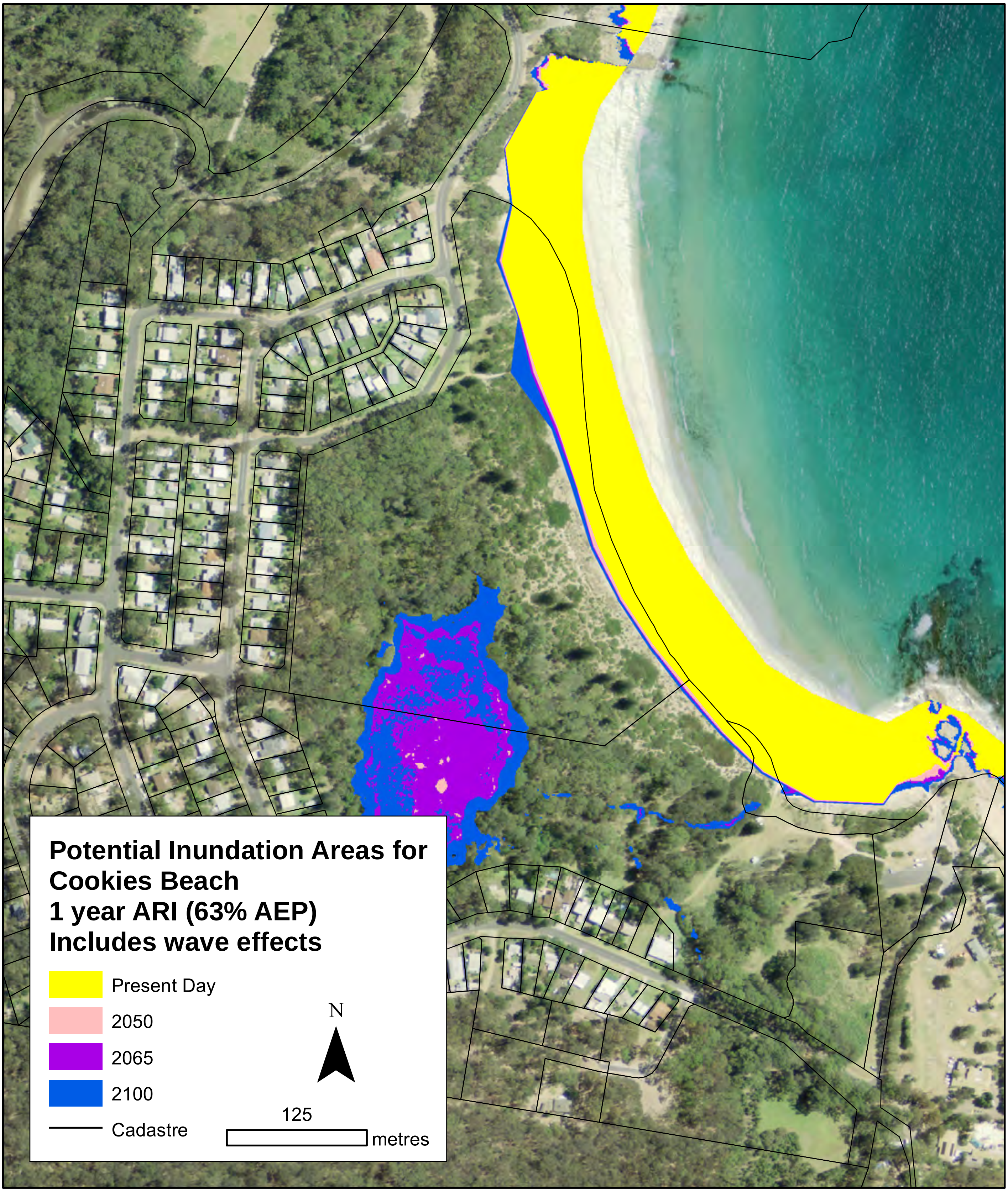

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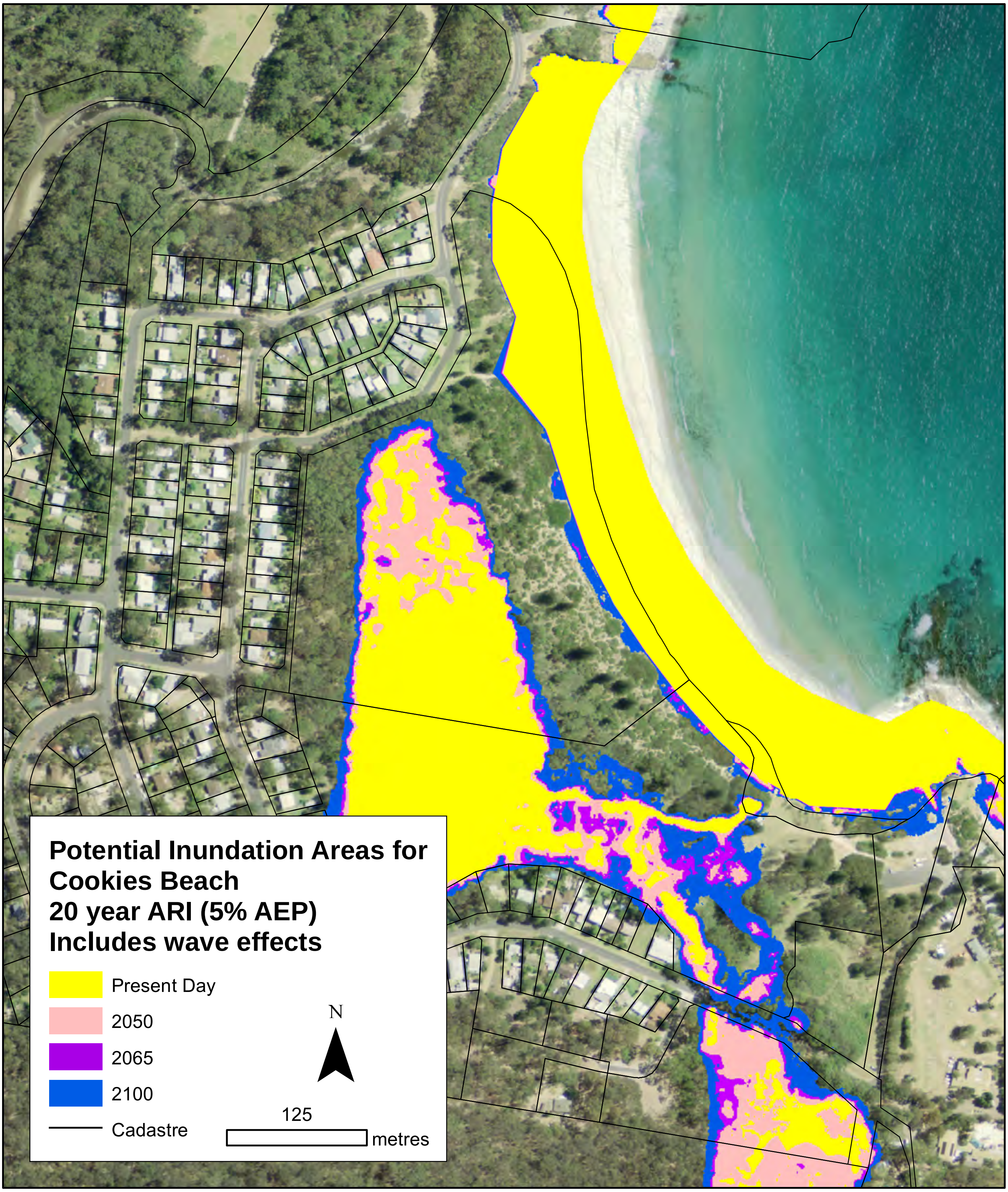
Figure L.4



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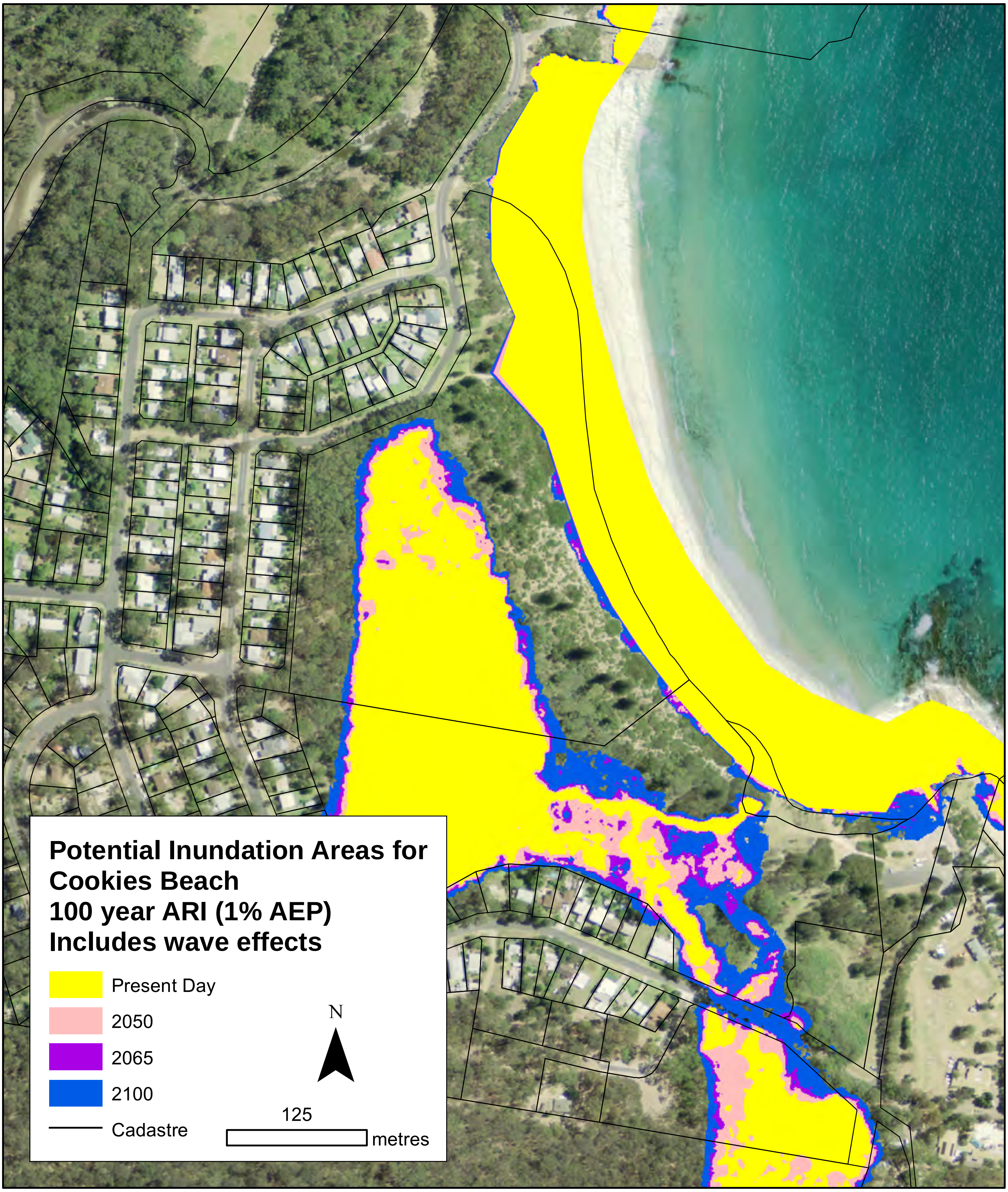

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Figure L.6

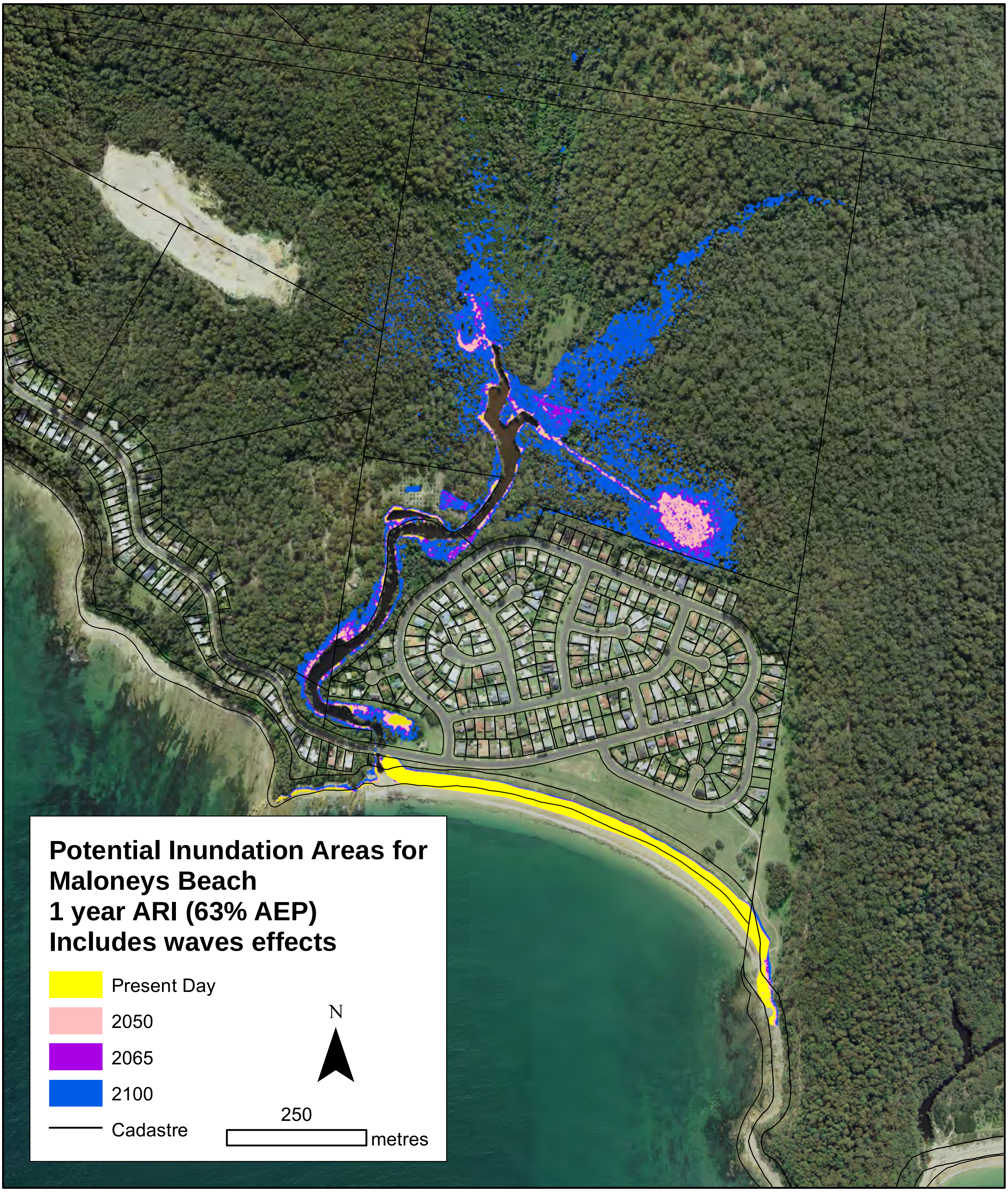

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Figure L.7

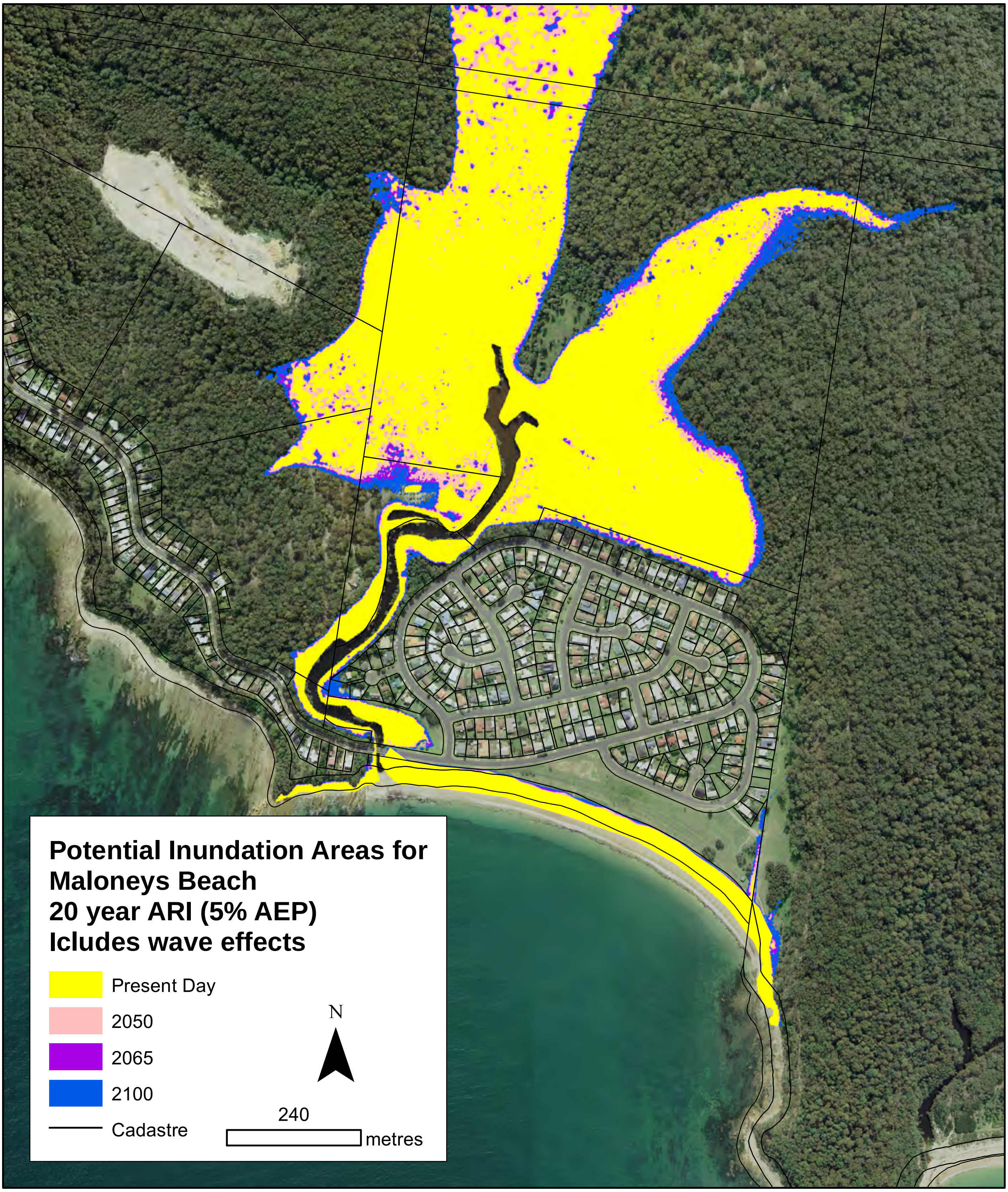

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Figure L.8

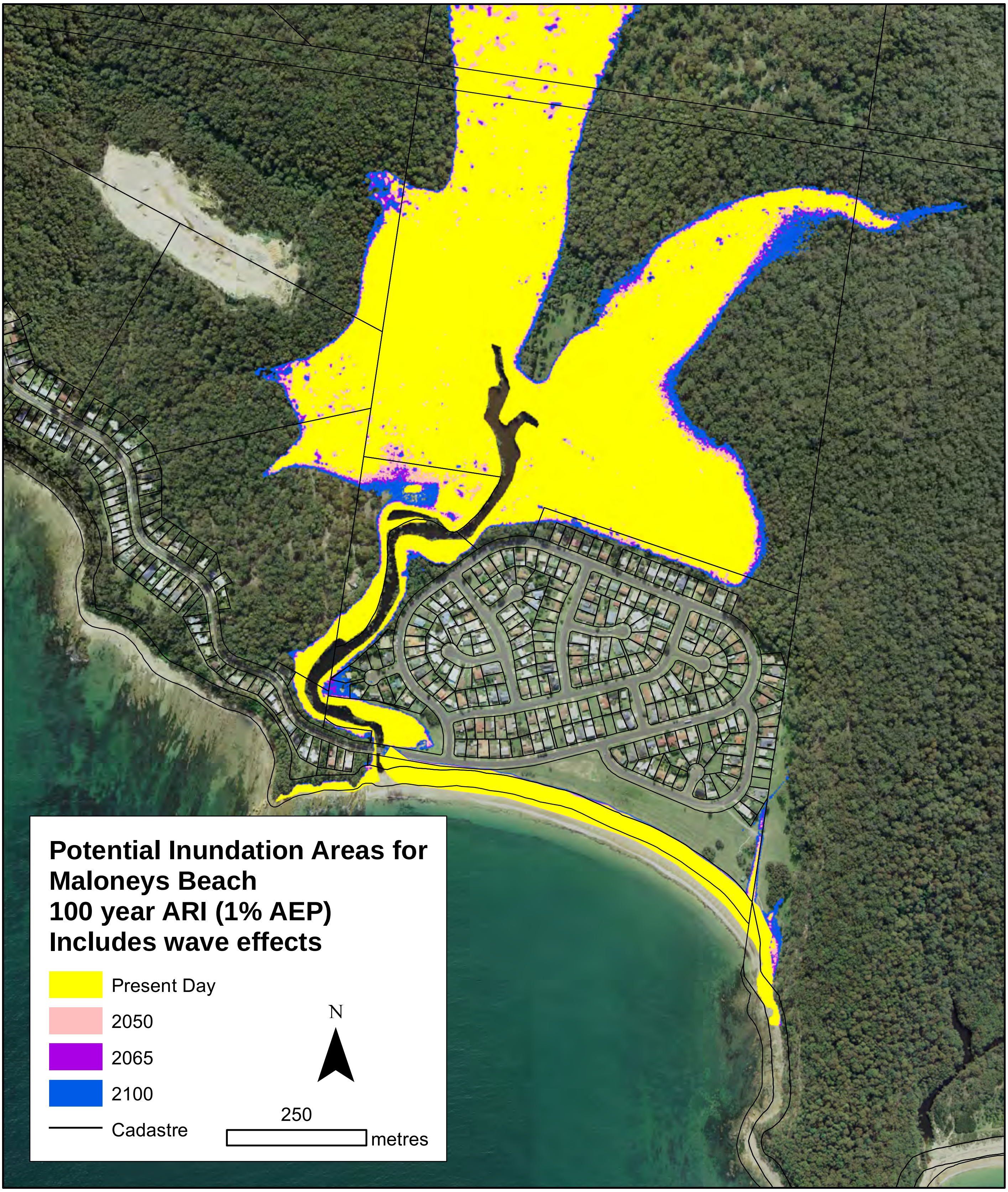

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Figure L.9

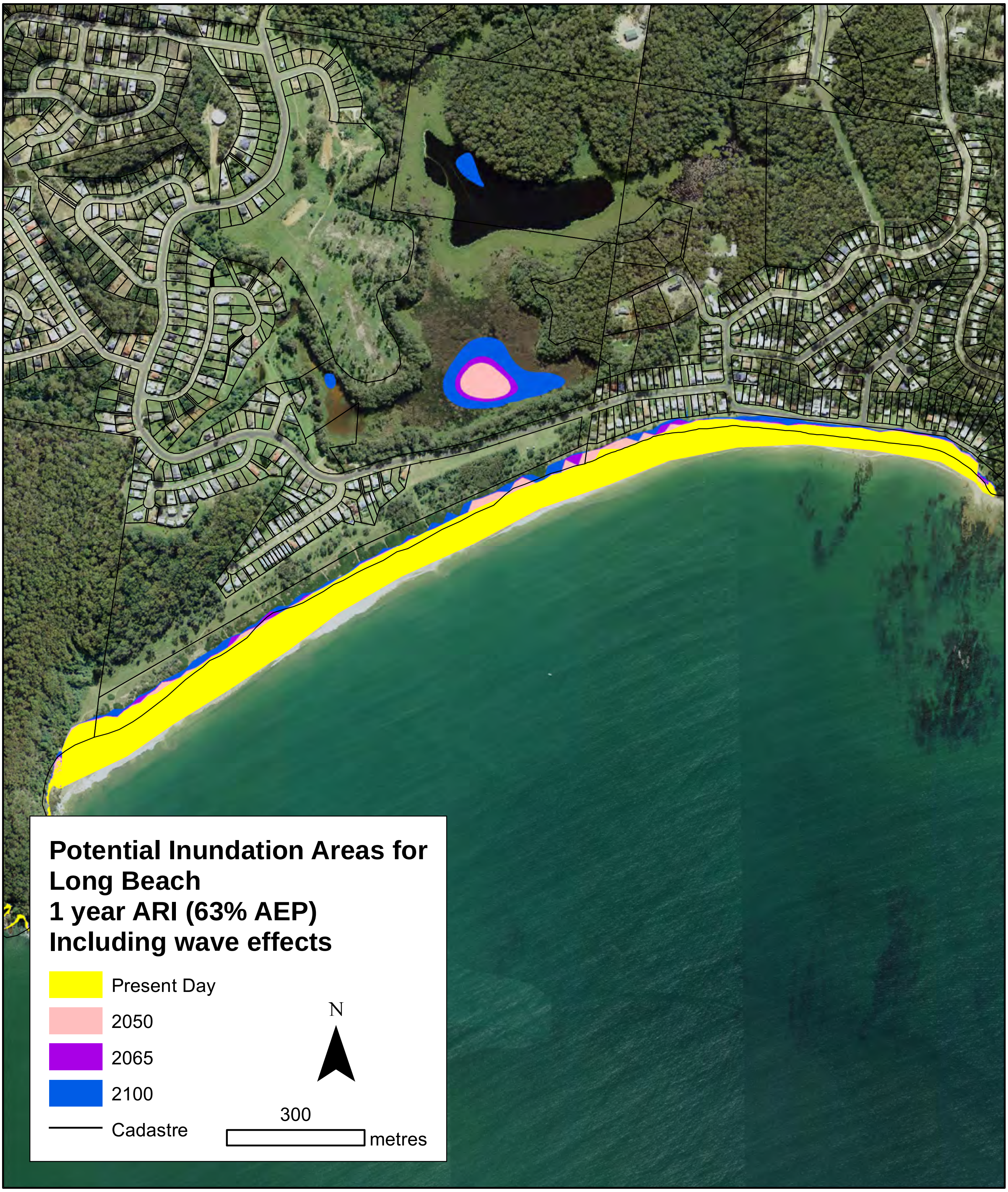

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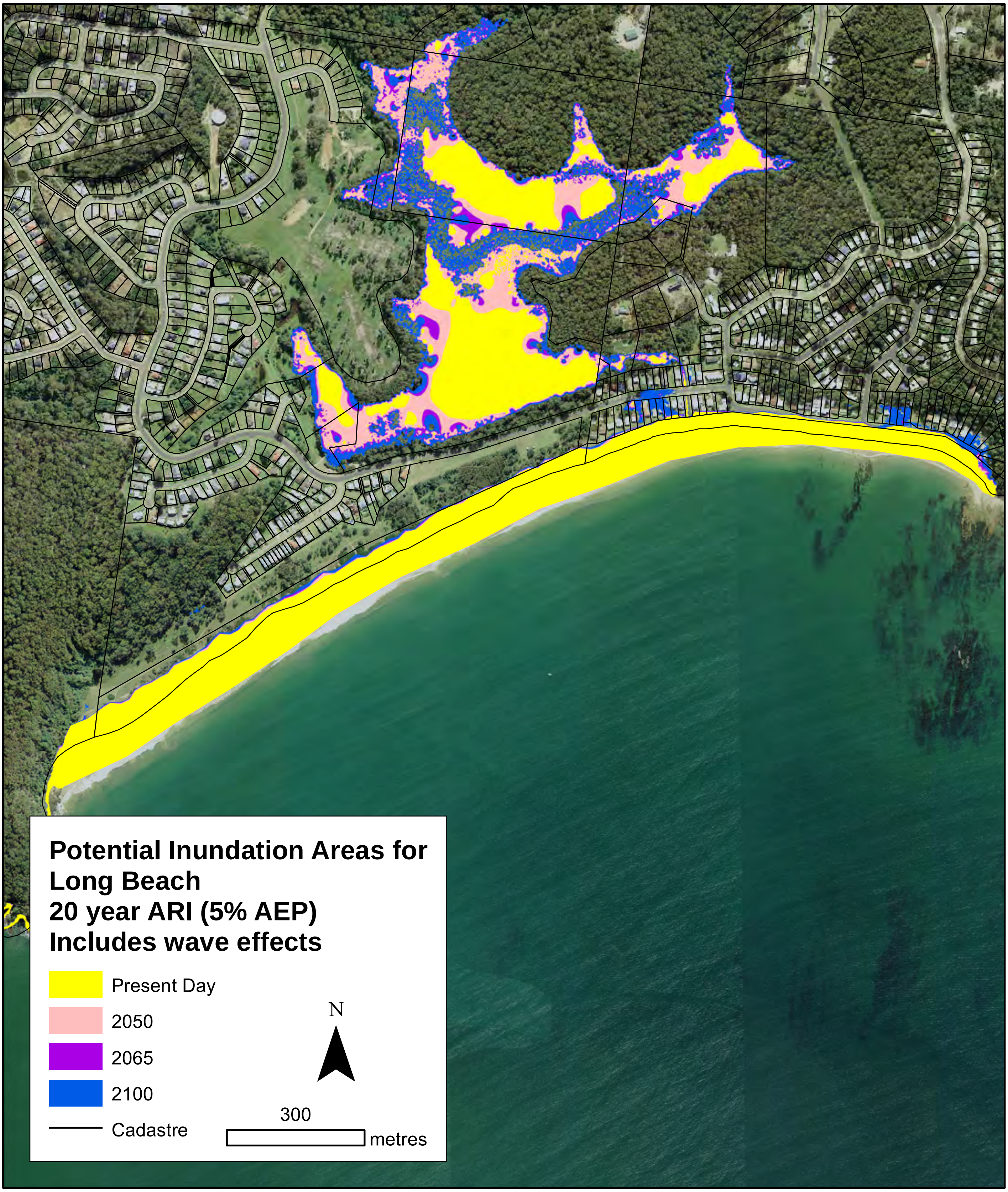
Figure L.10



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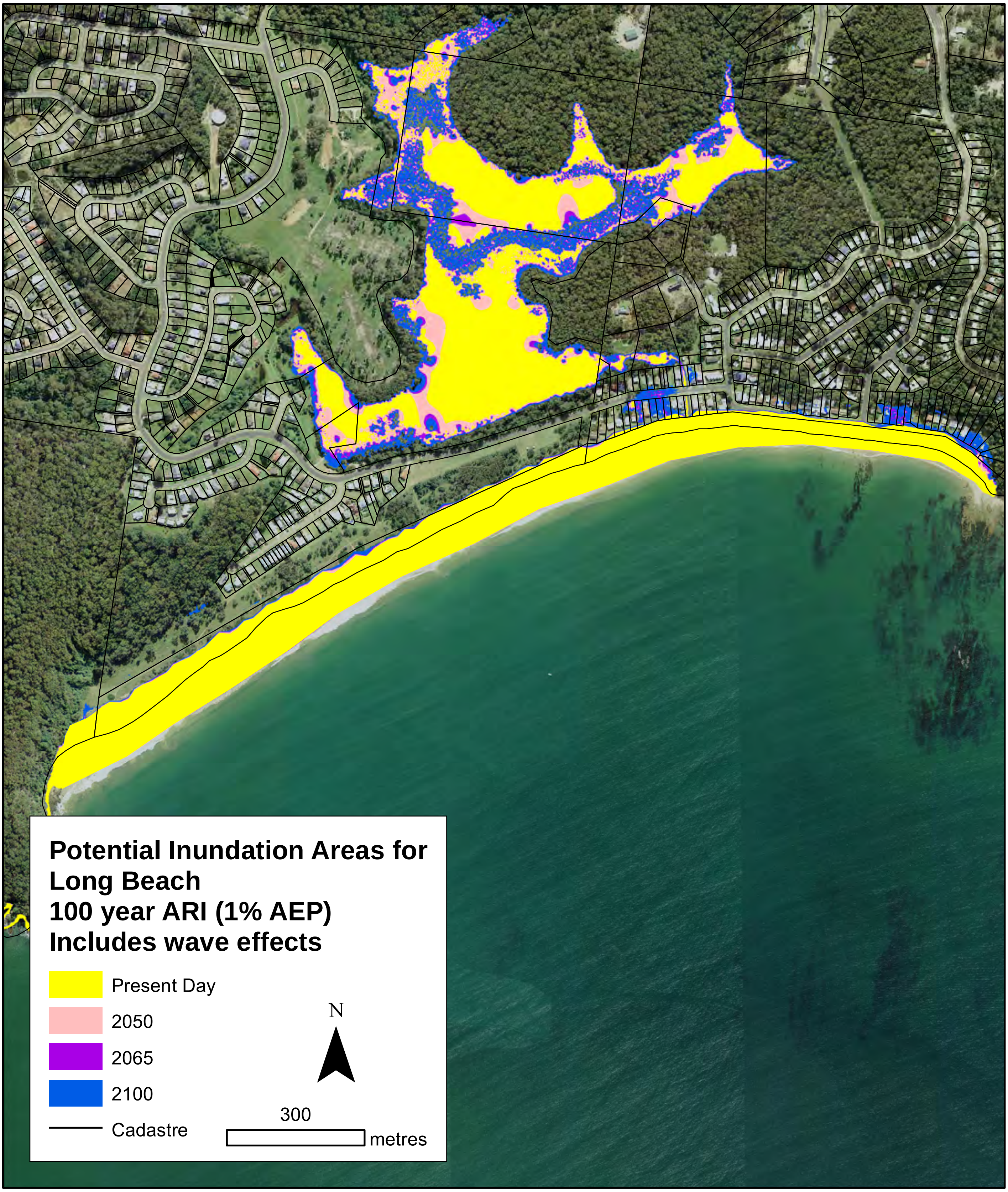

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Figure L.12

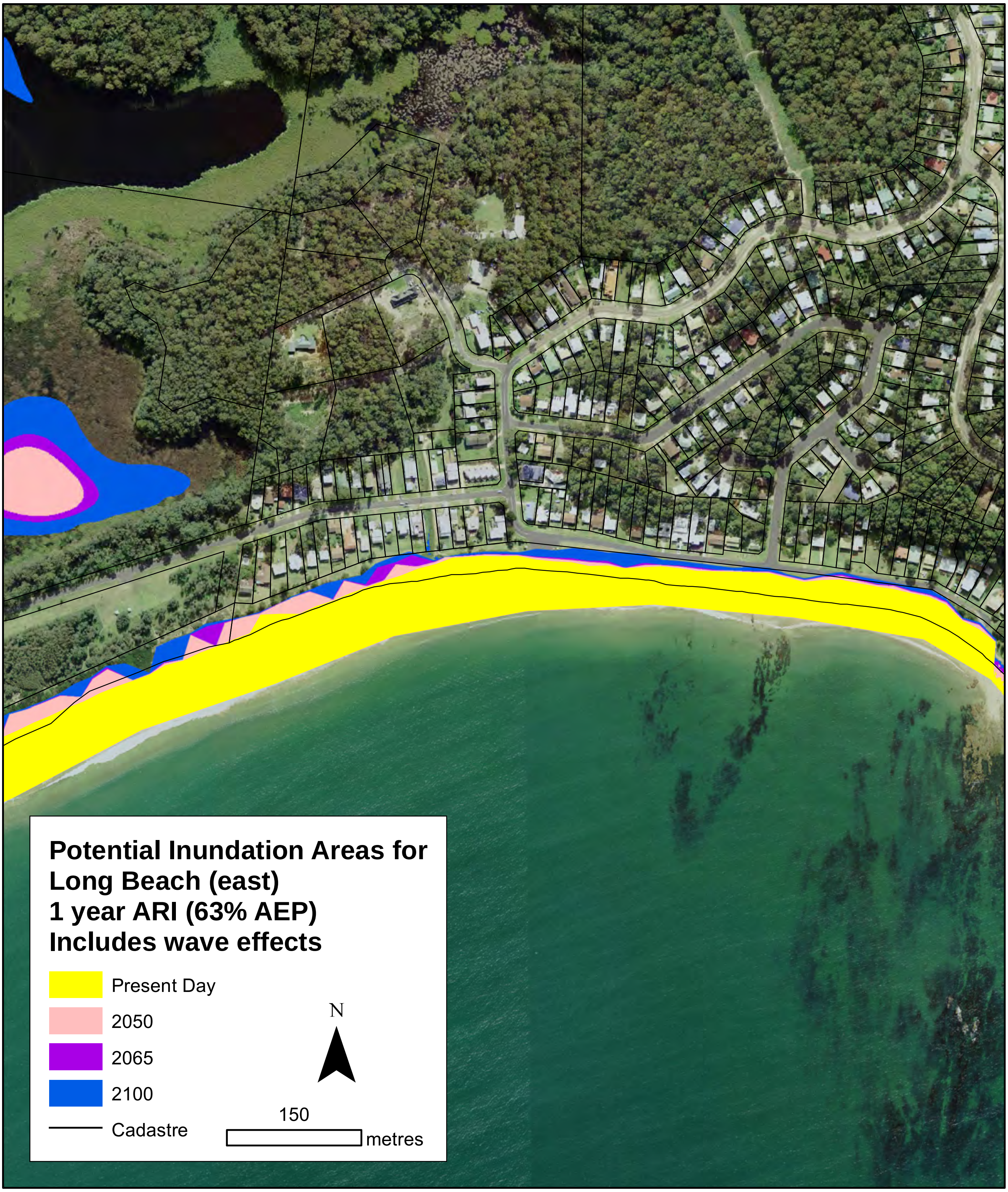

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Figure L.13

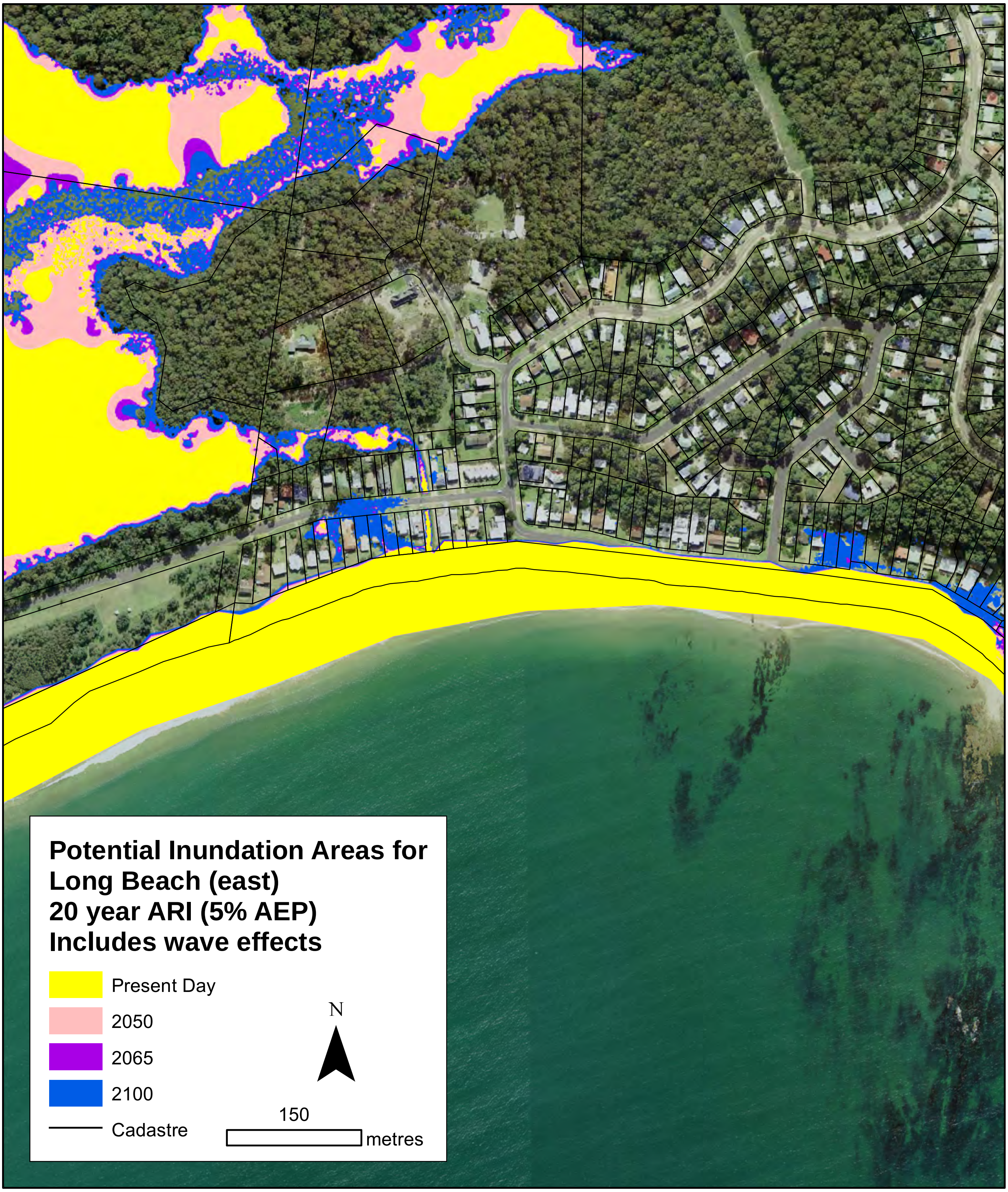

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Figure L.14

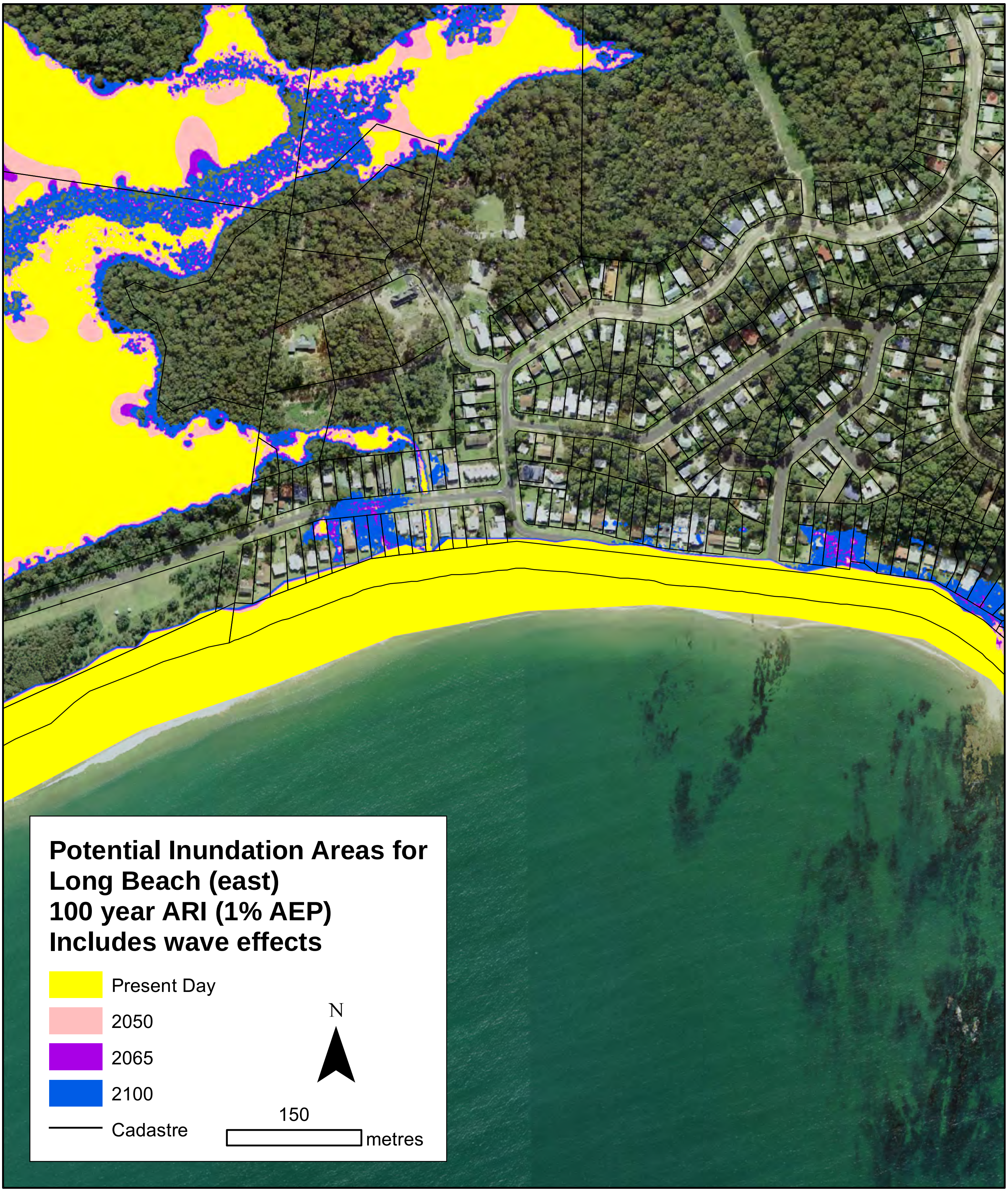

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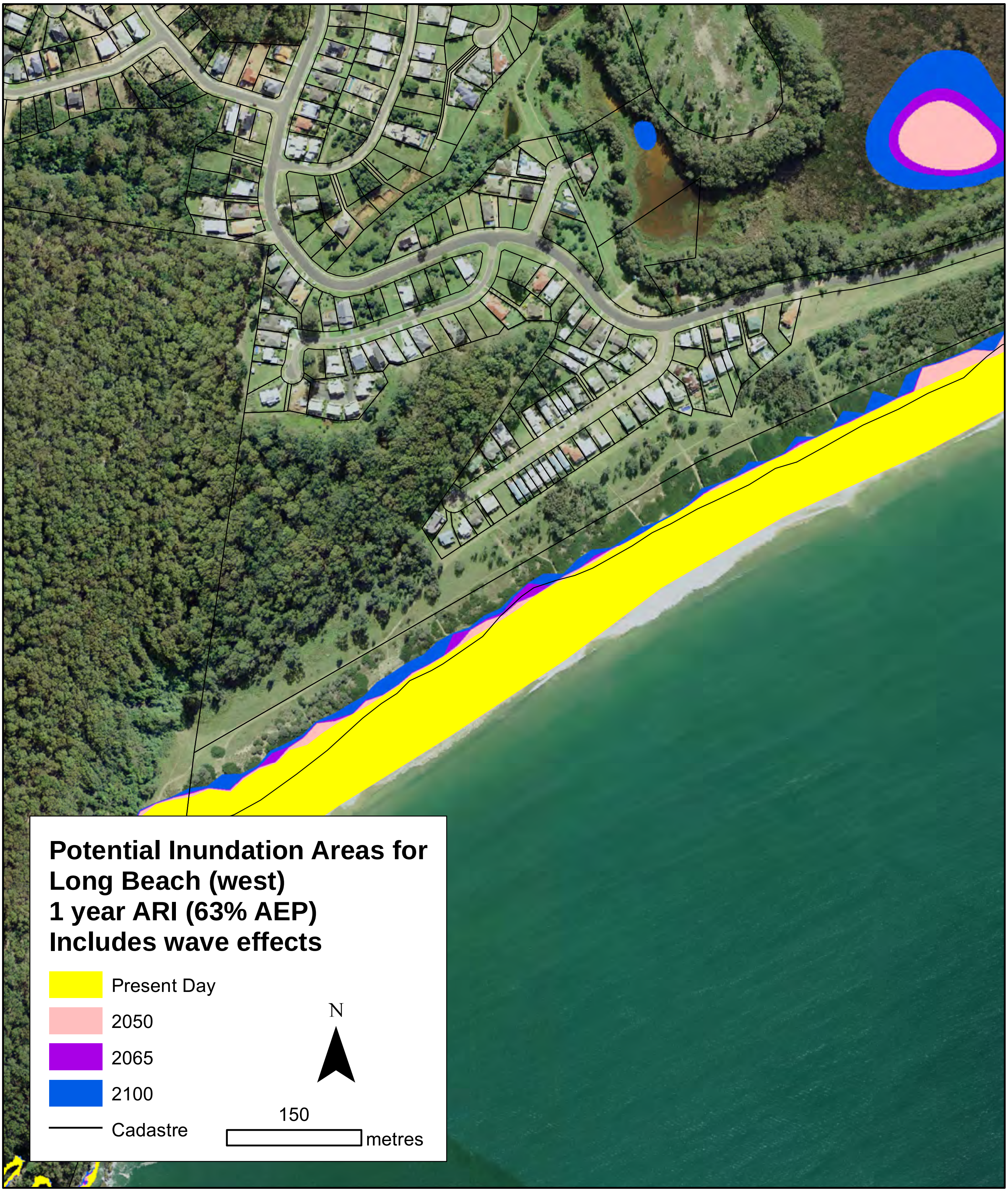
Figure L.15



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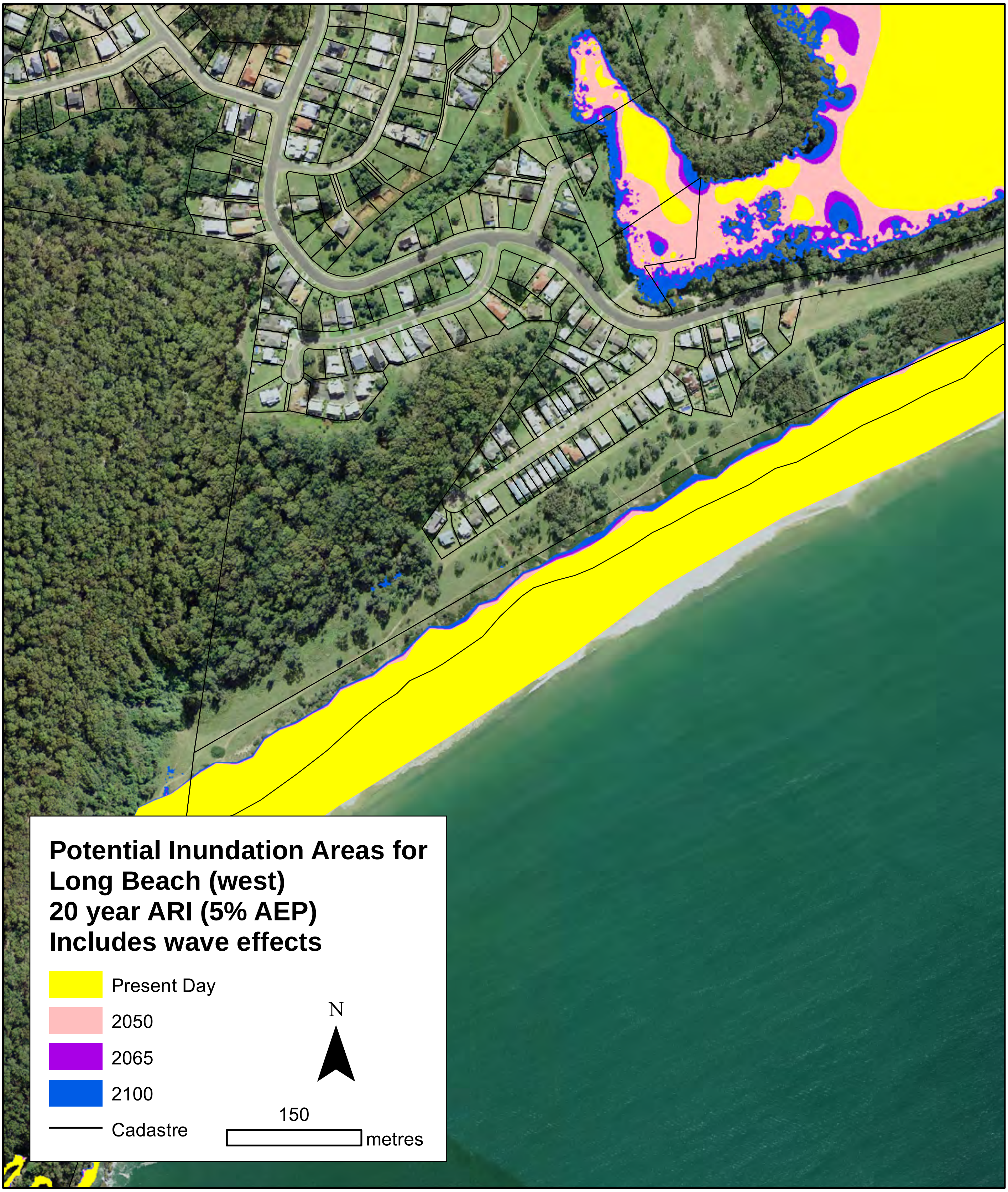

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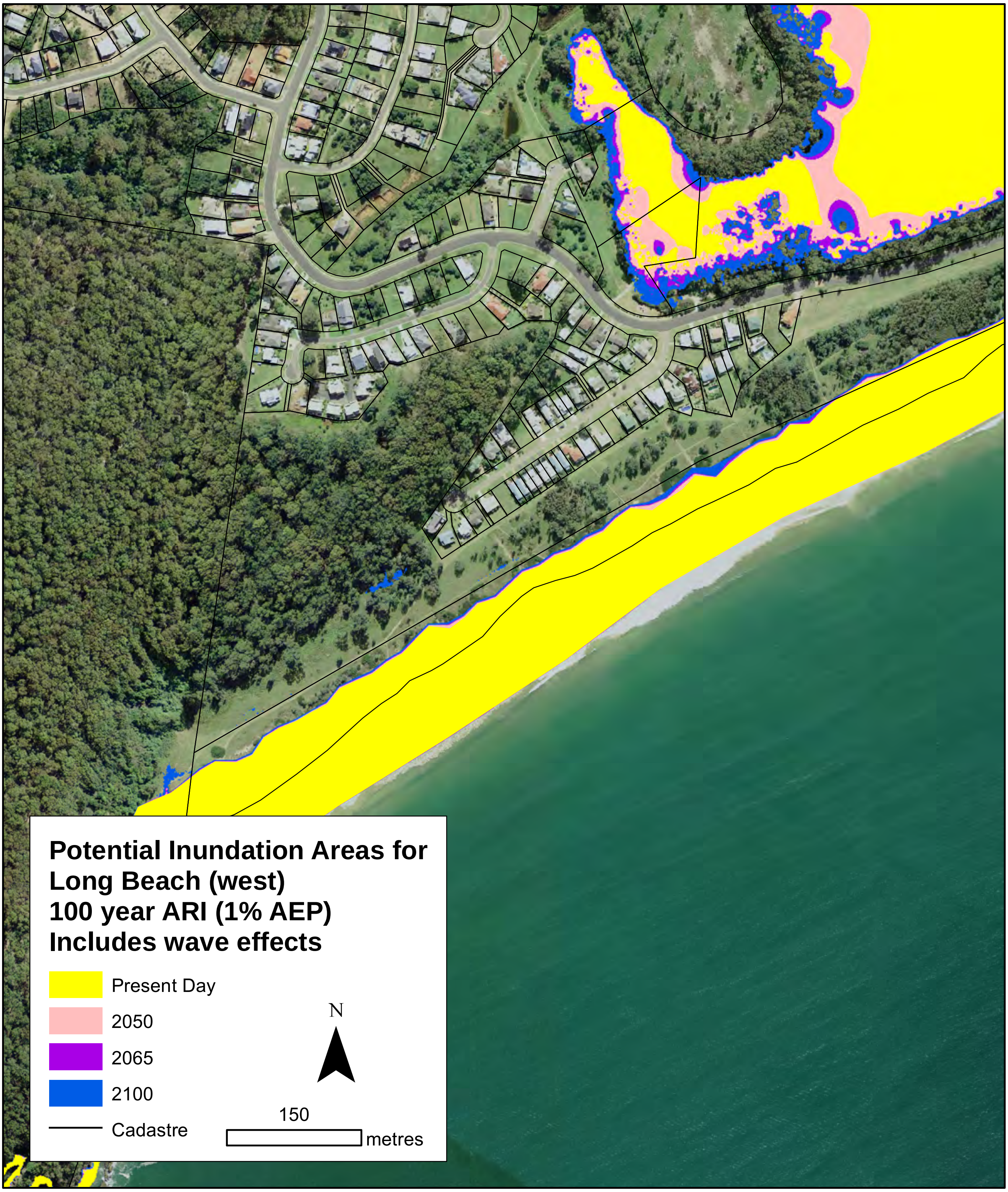

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Figure L.18

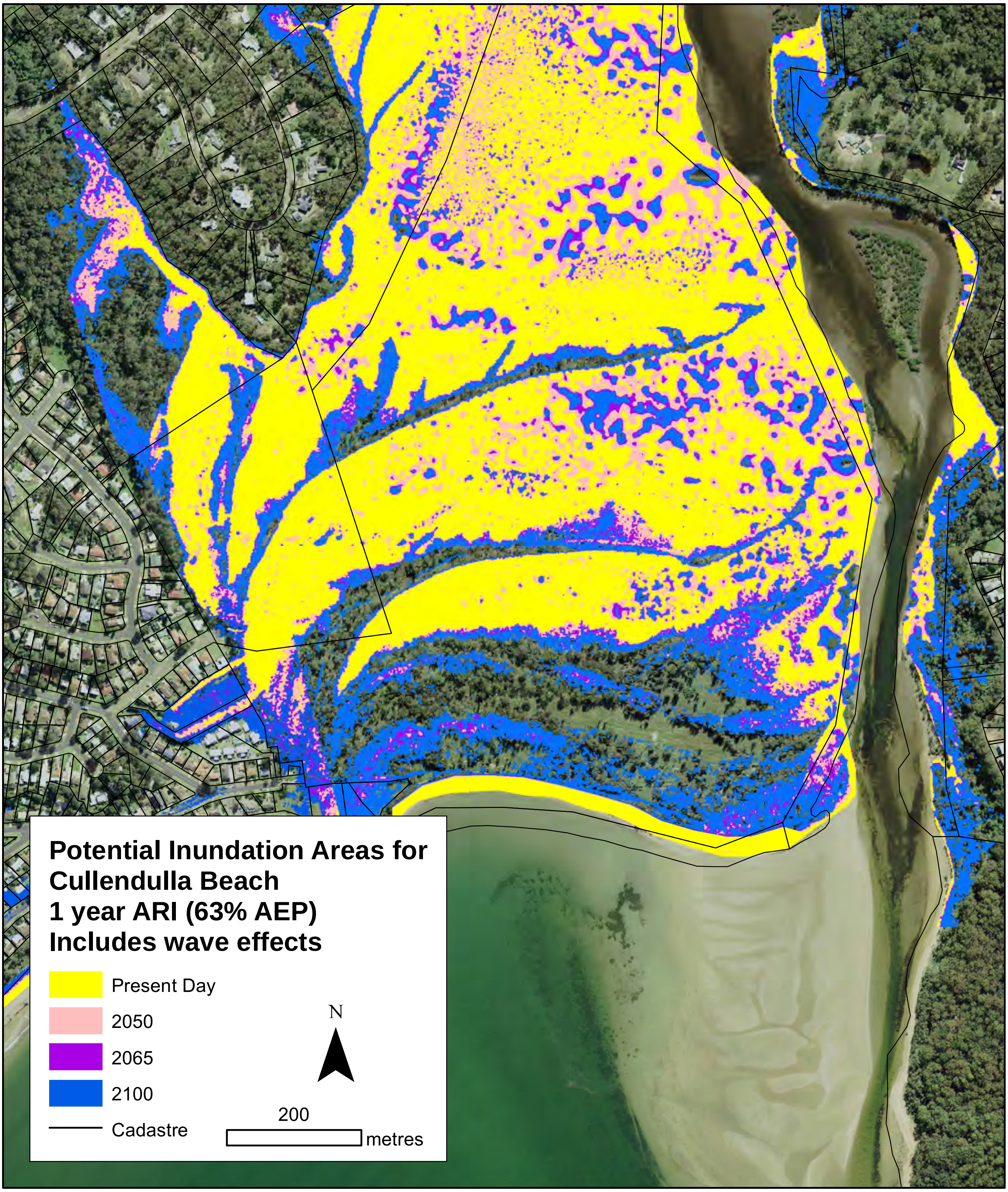

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Figure L.19

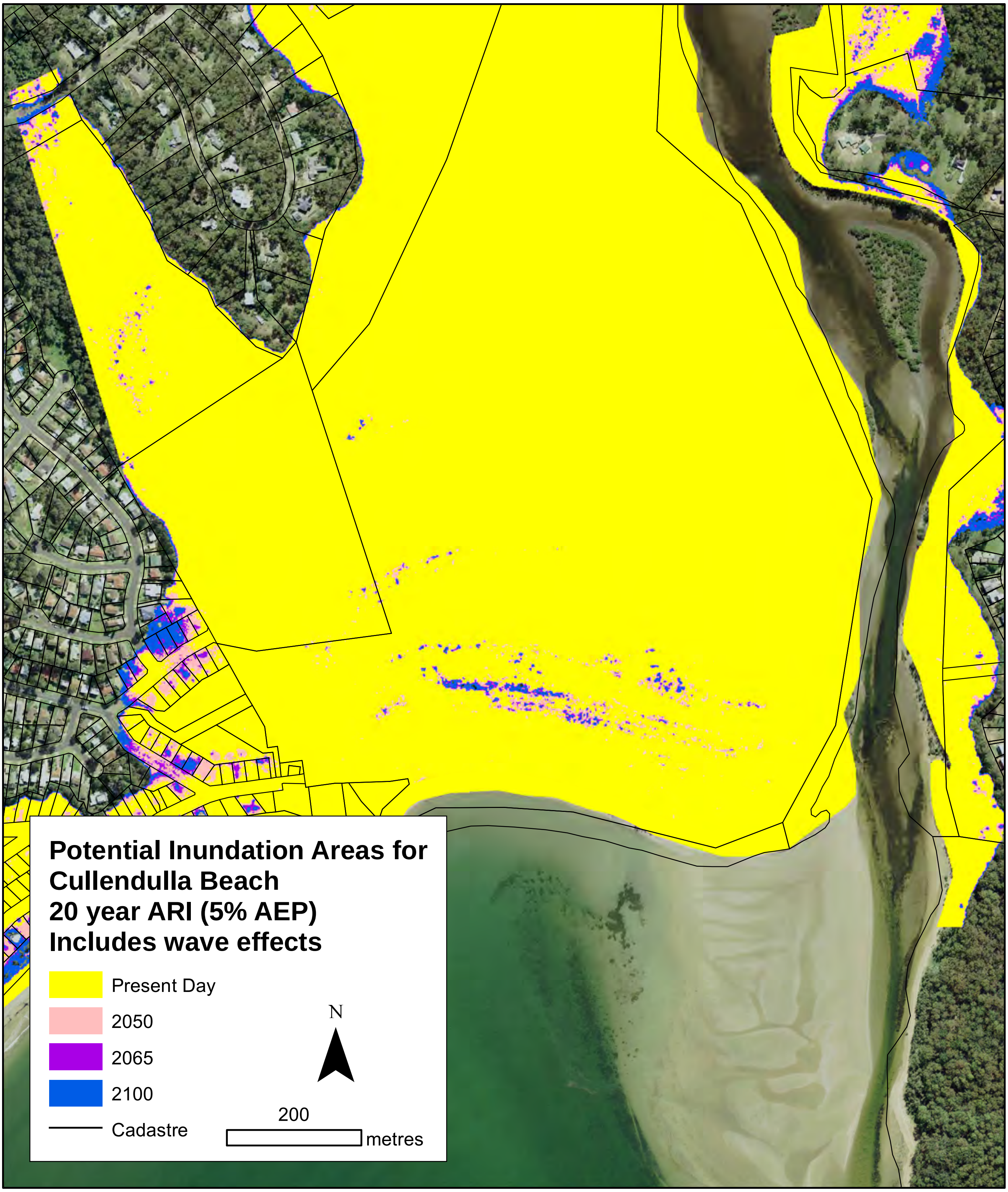

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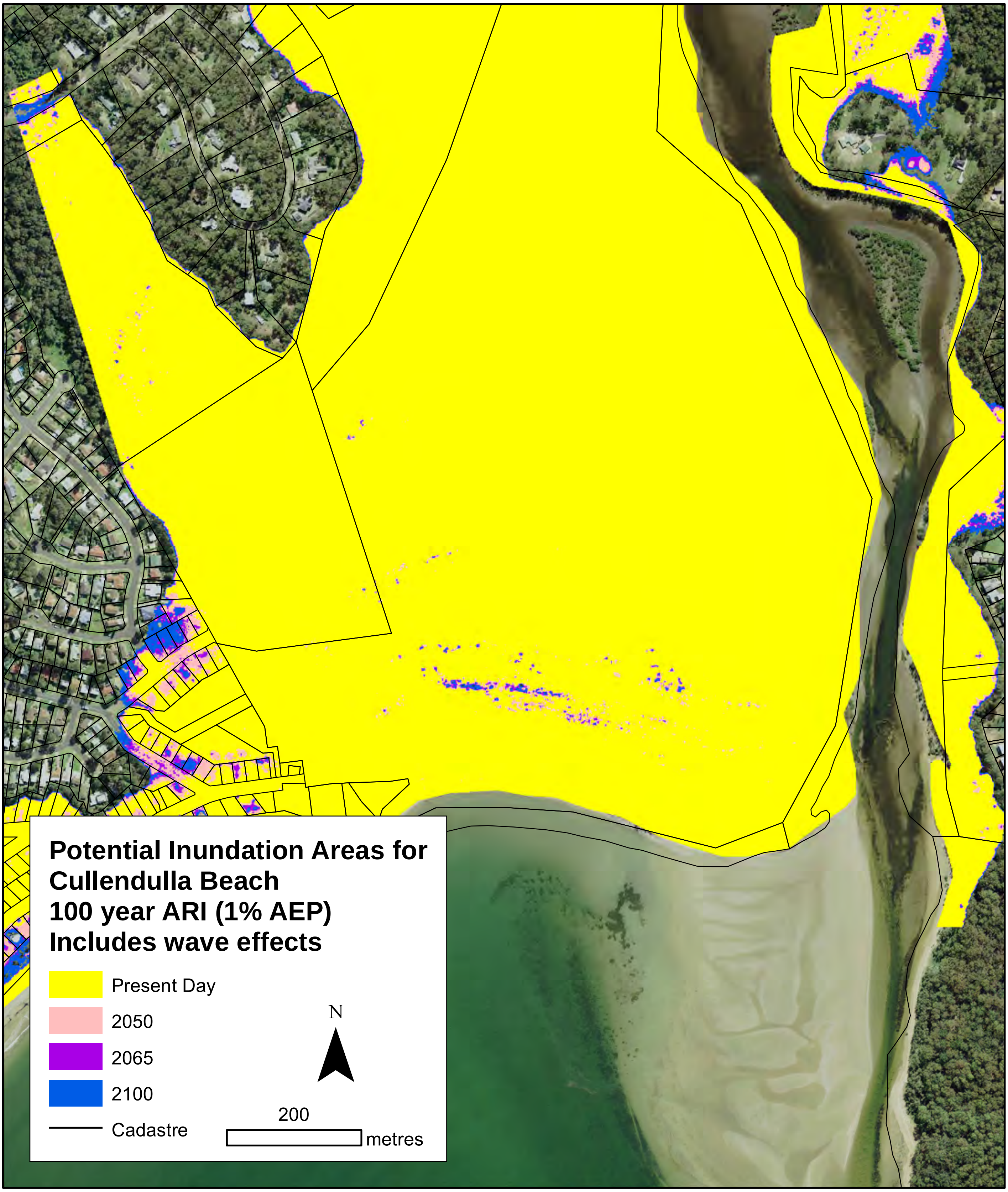
Figure L.20



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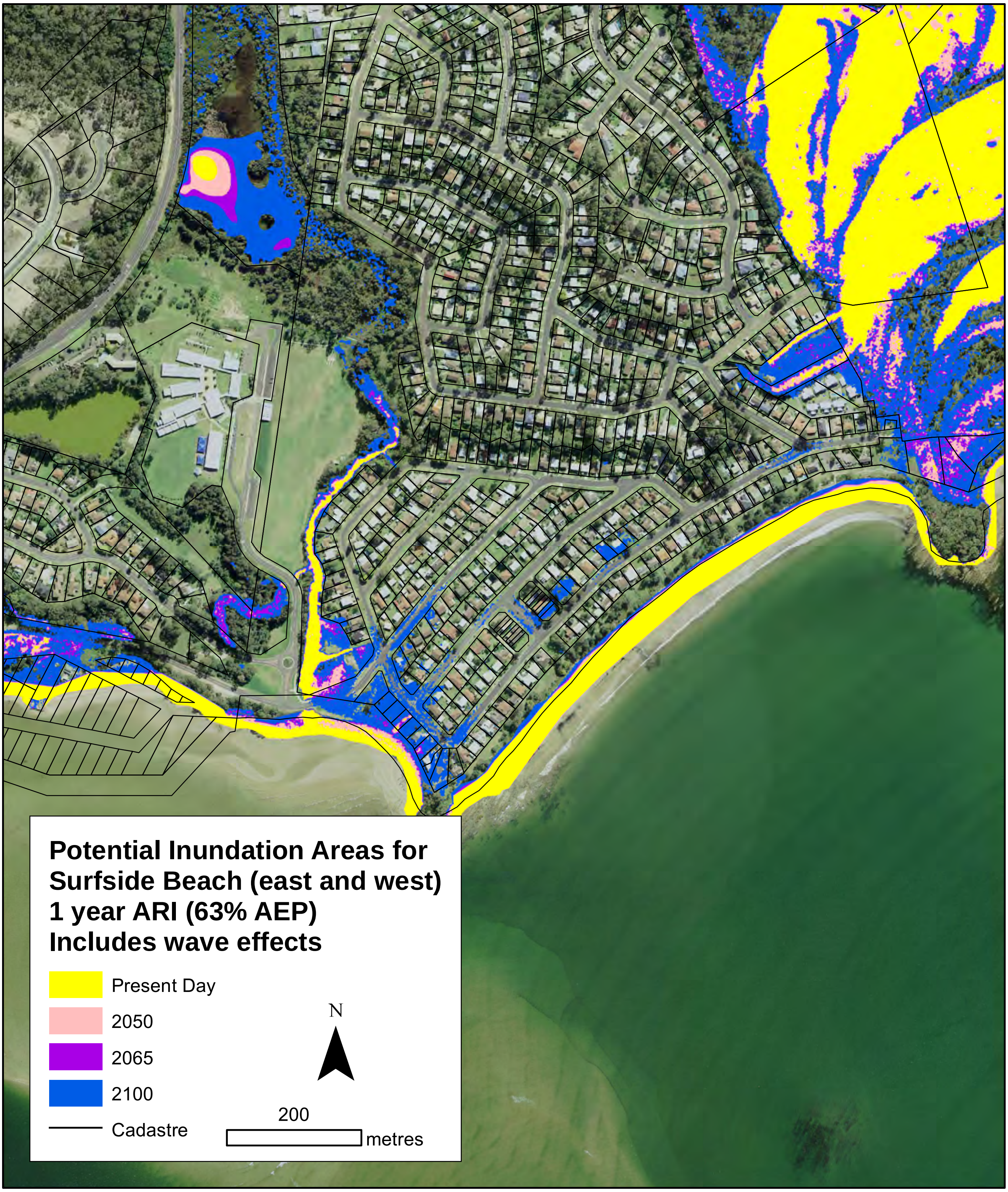

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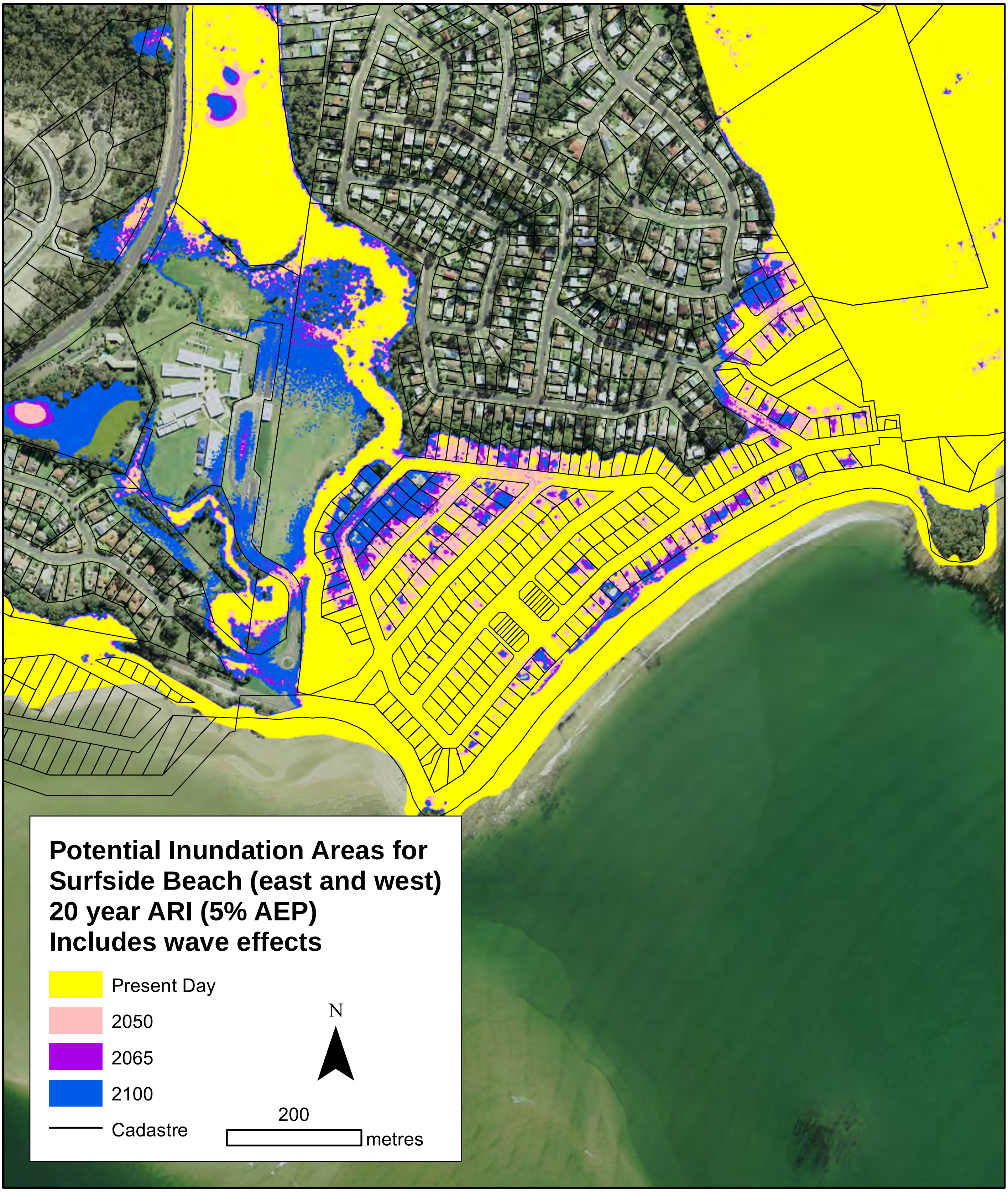
Figure L.22



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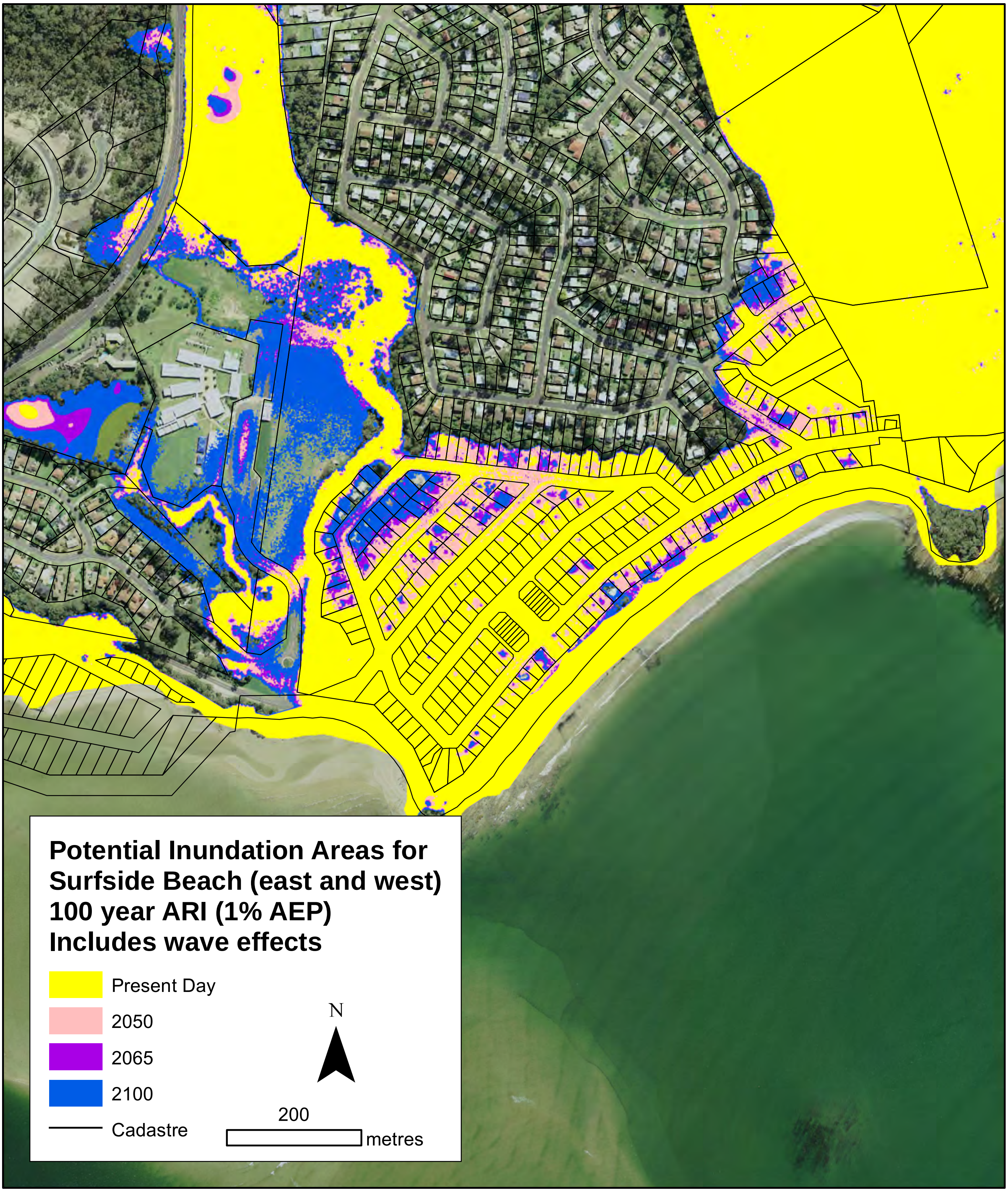

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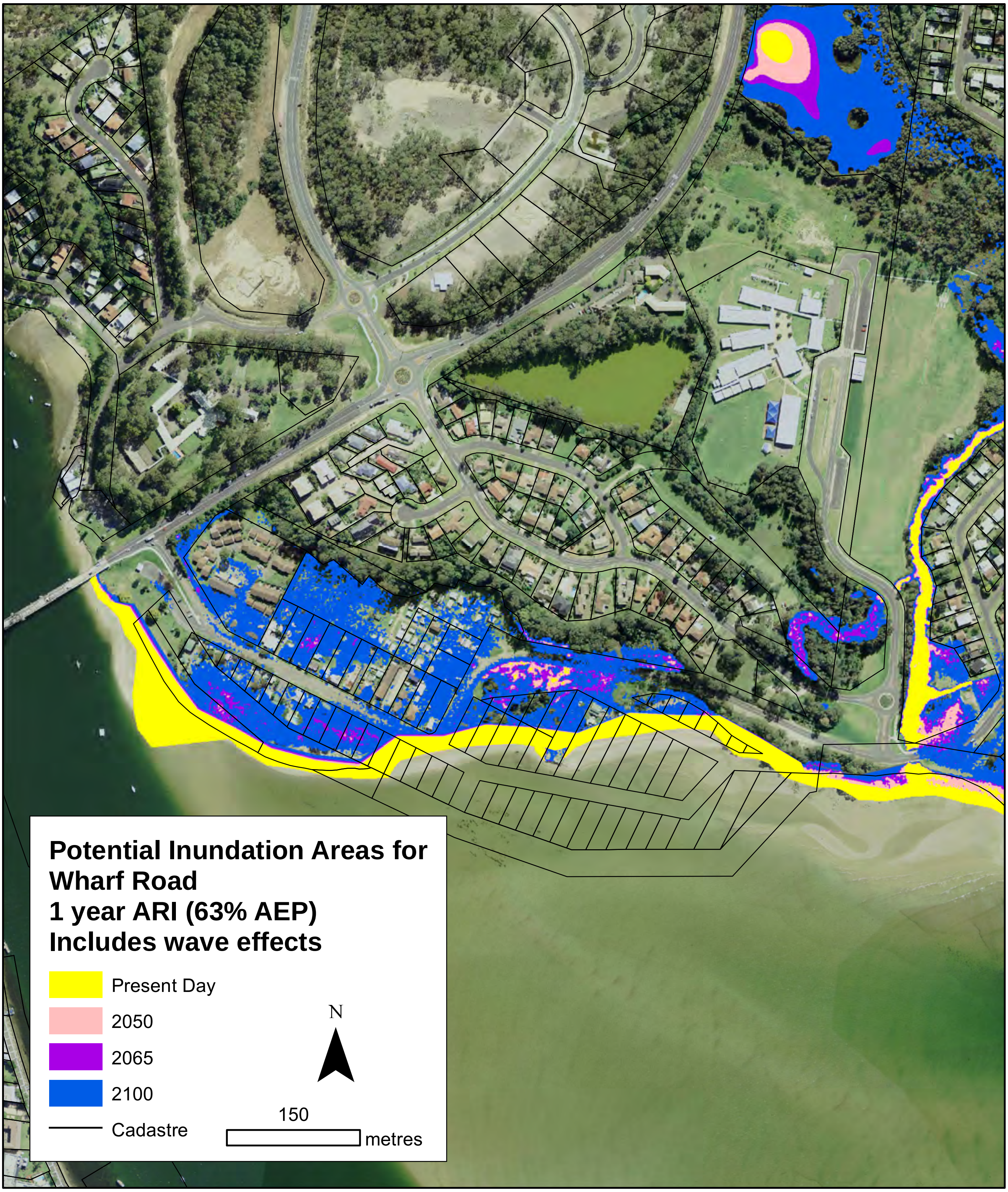
Figure L.24



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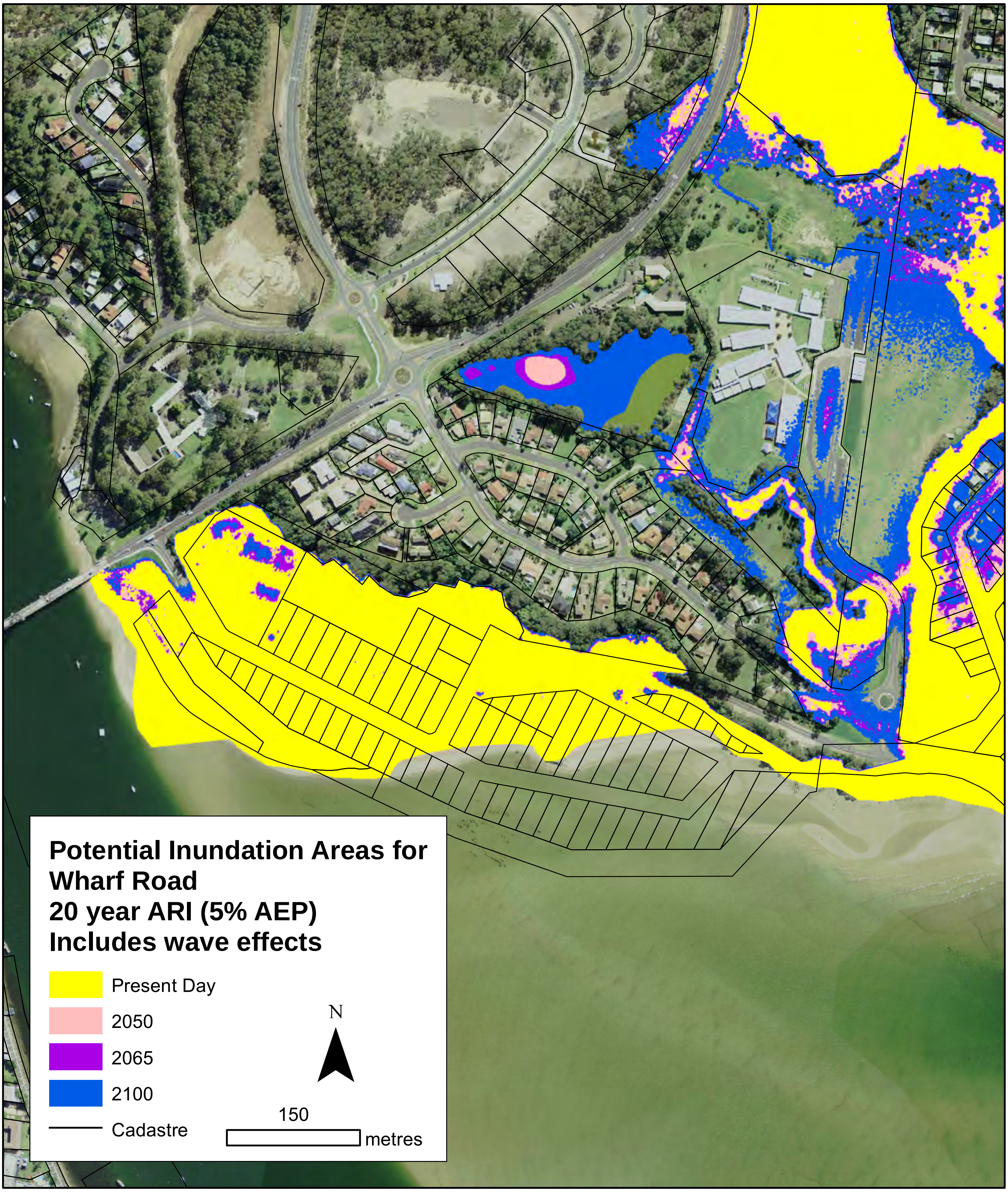

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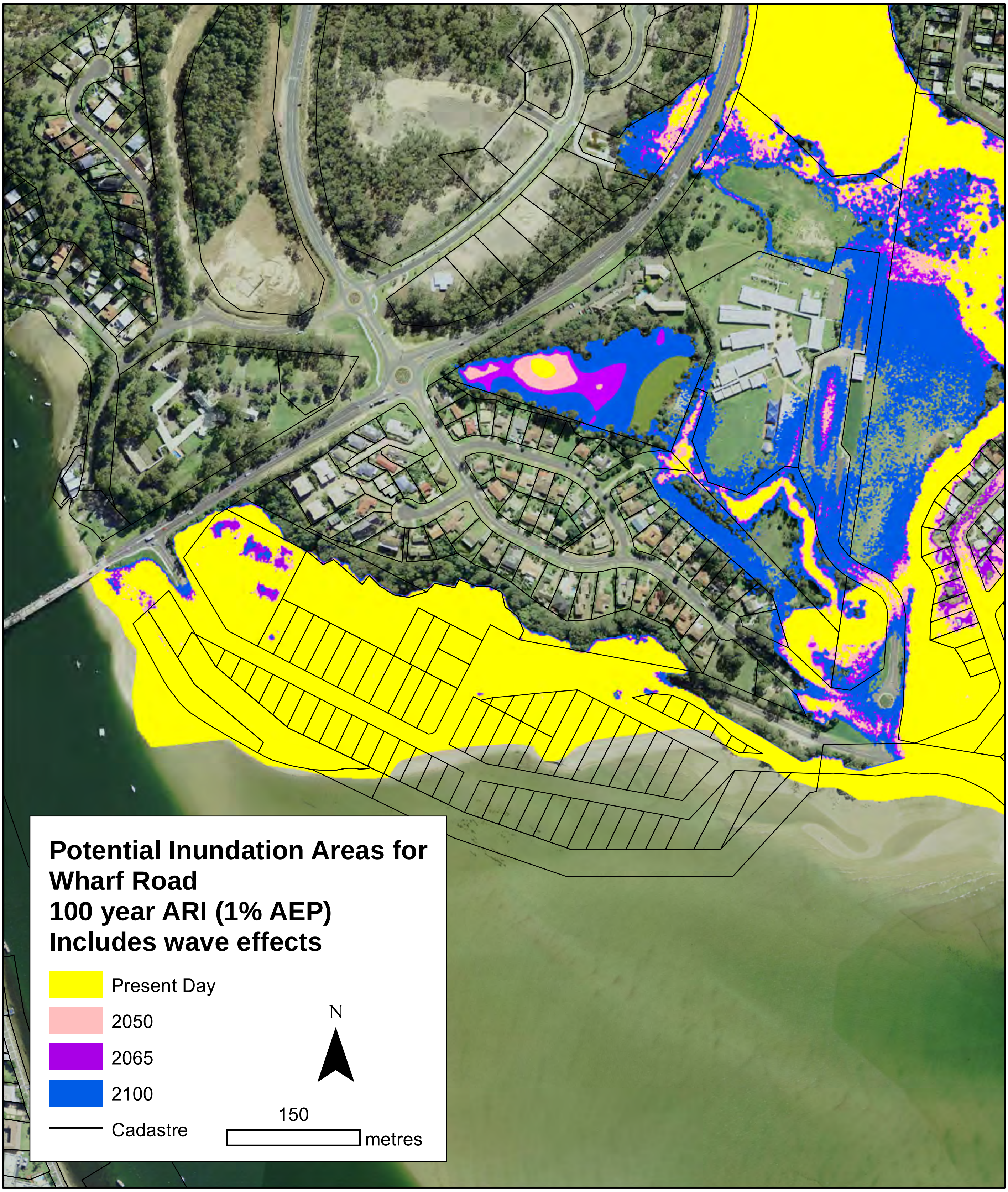
Figure L.26



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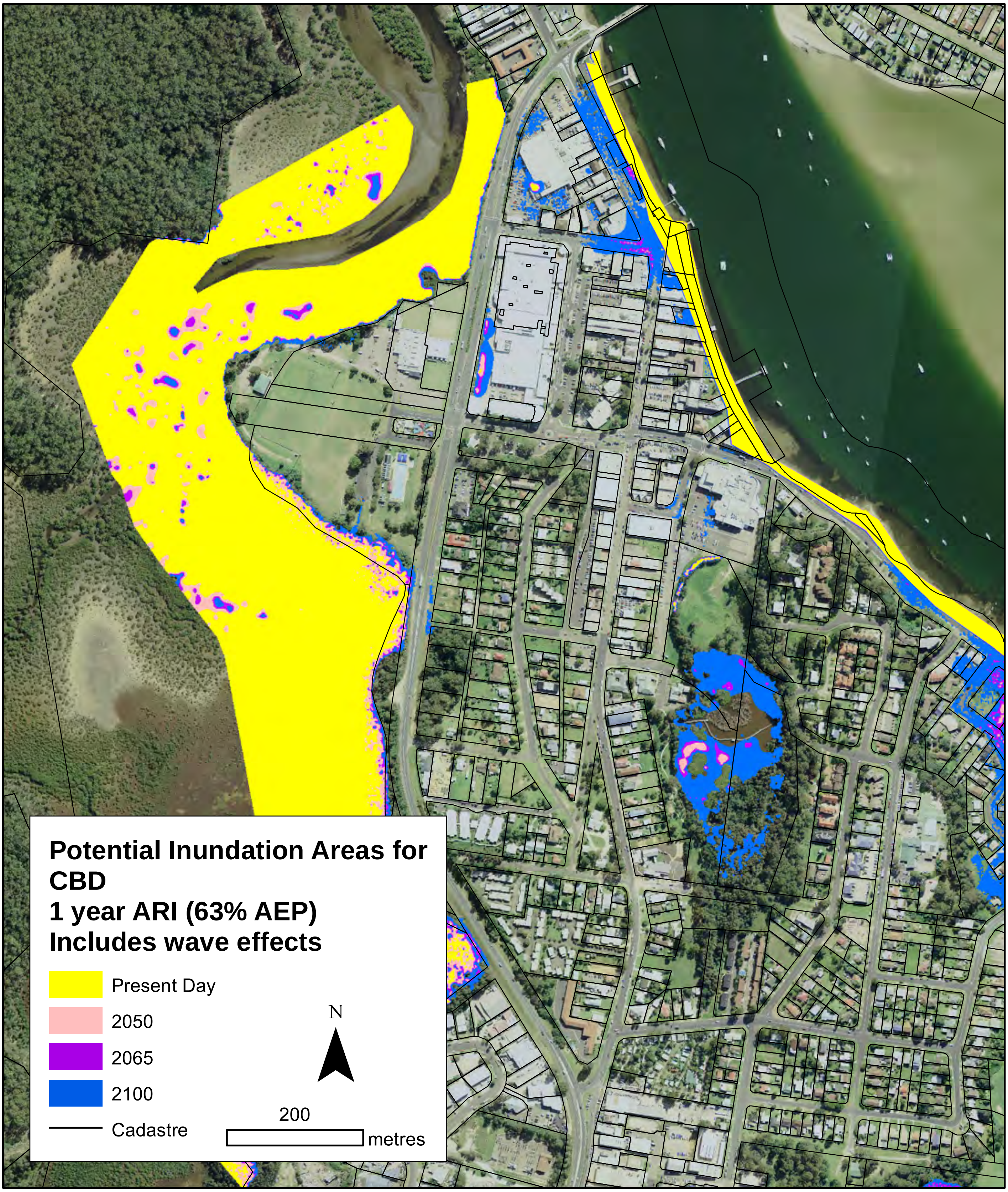

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Figure L.28

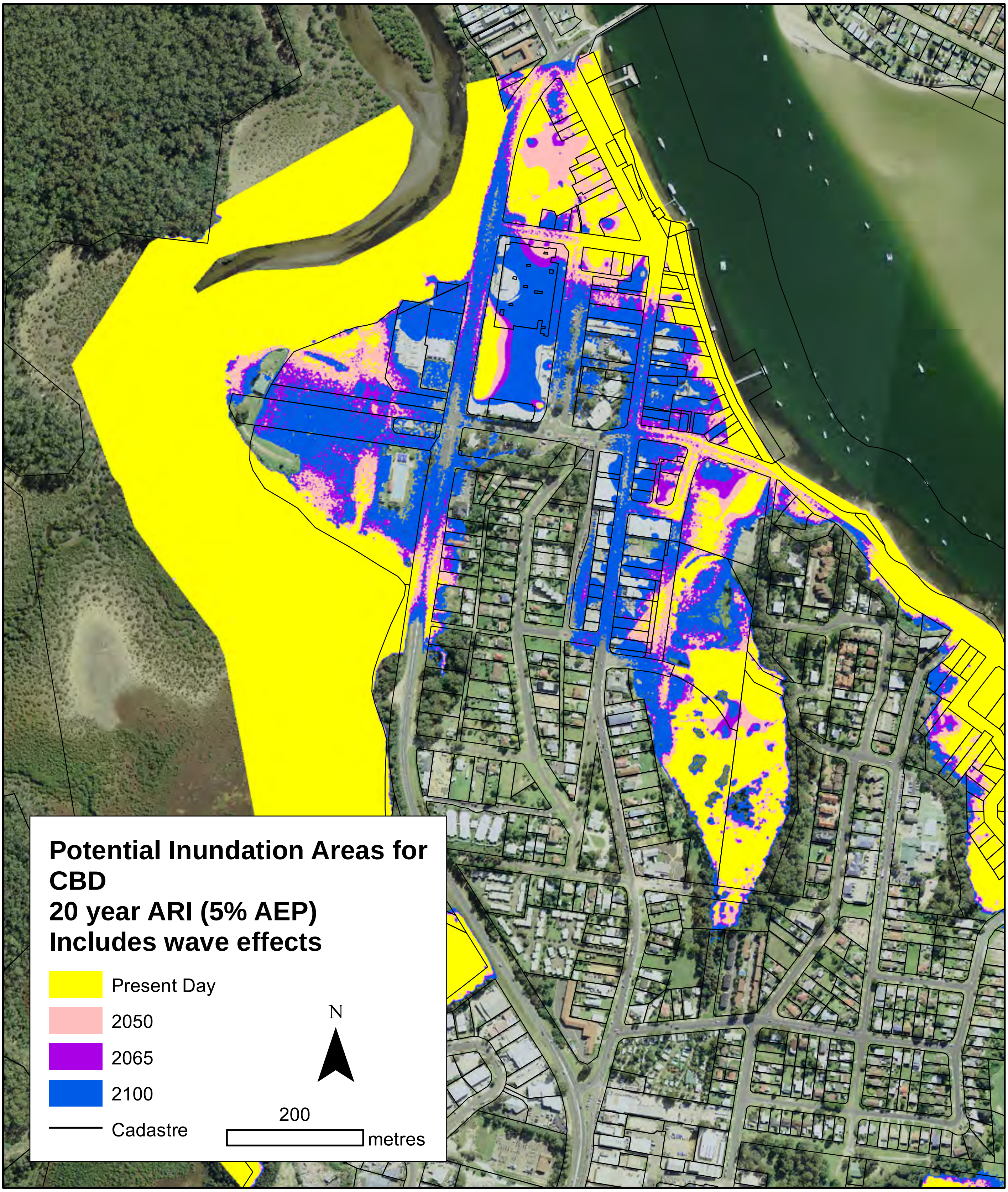

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Figure L.29

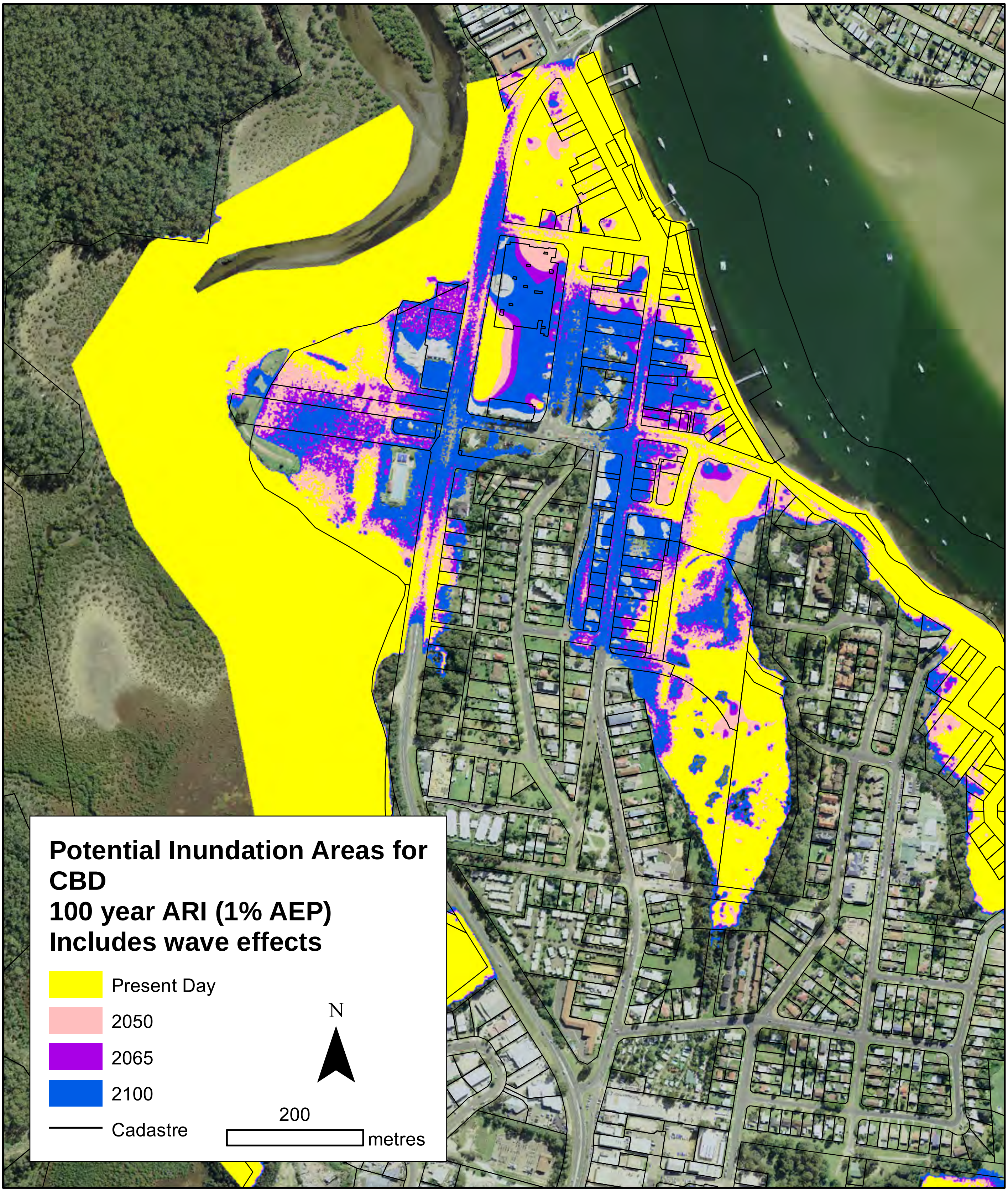

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Figure L.30

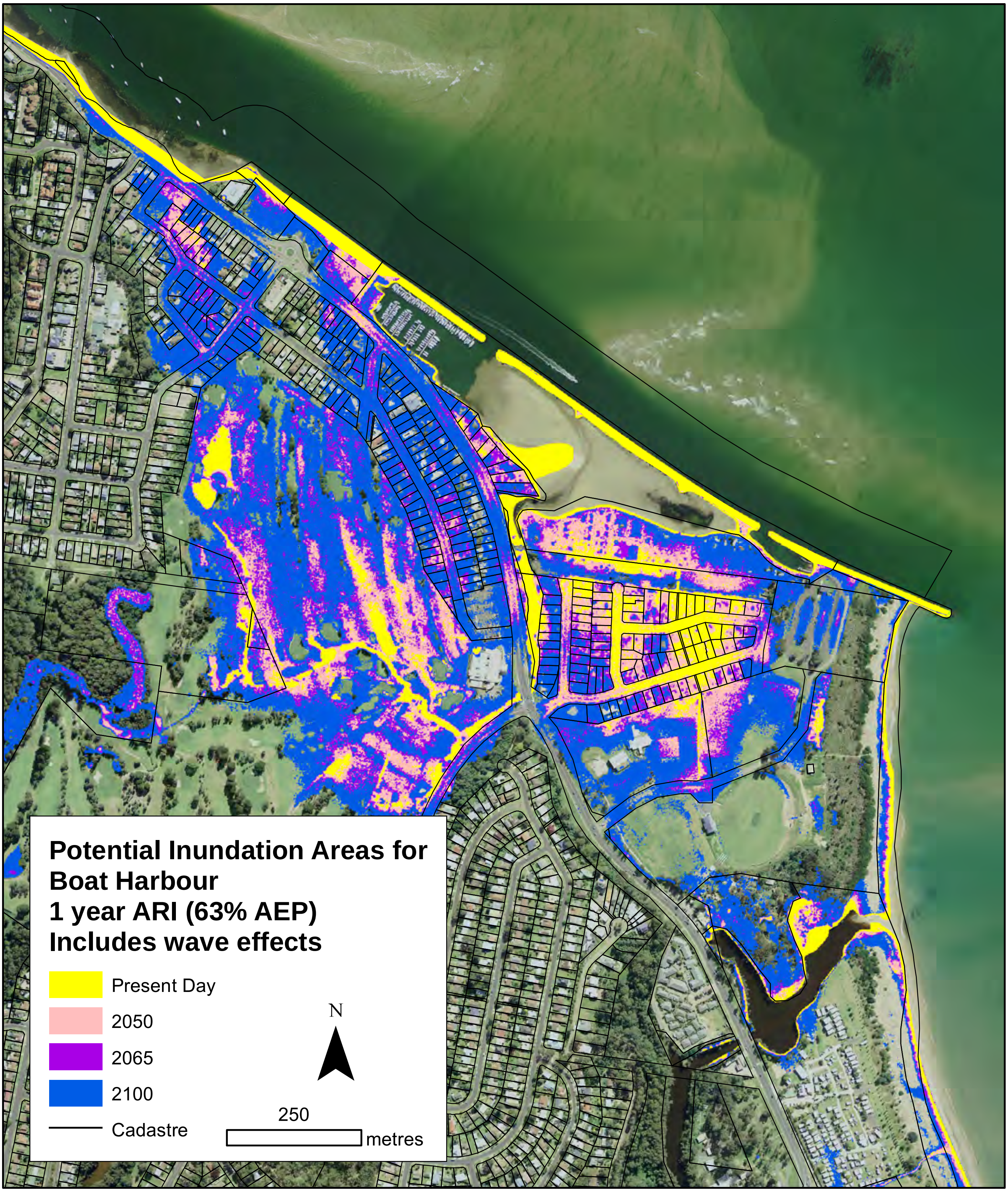

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Figure L.31

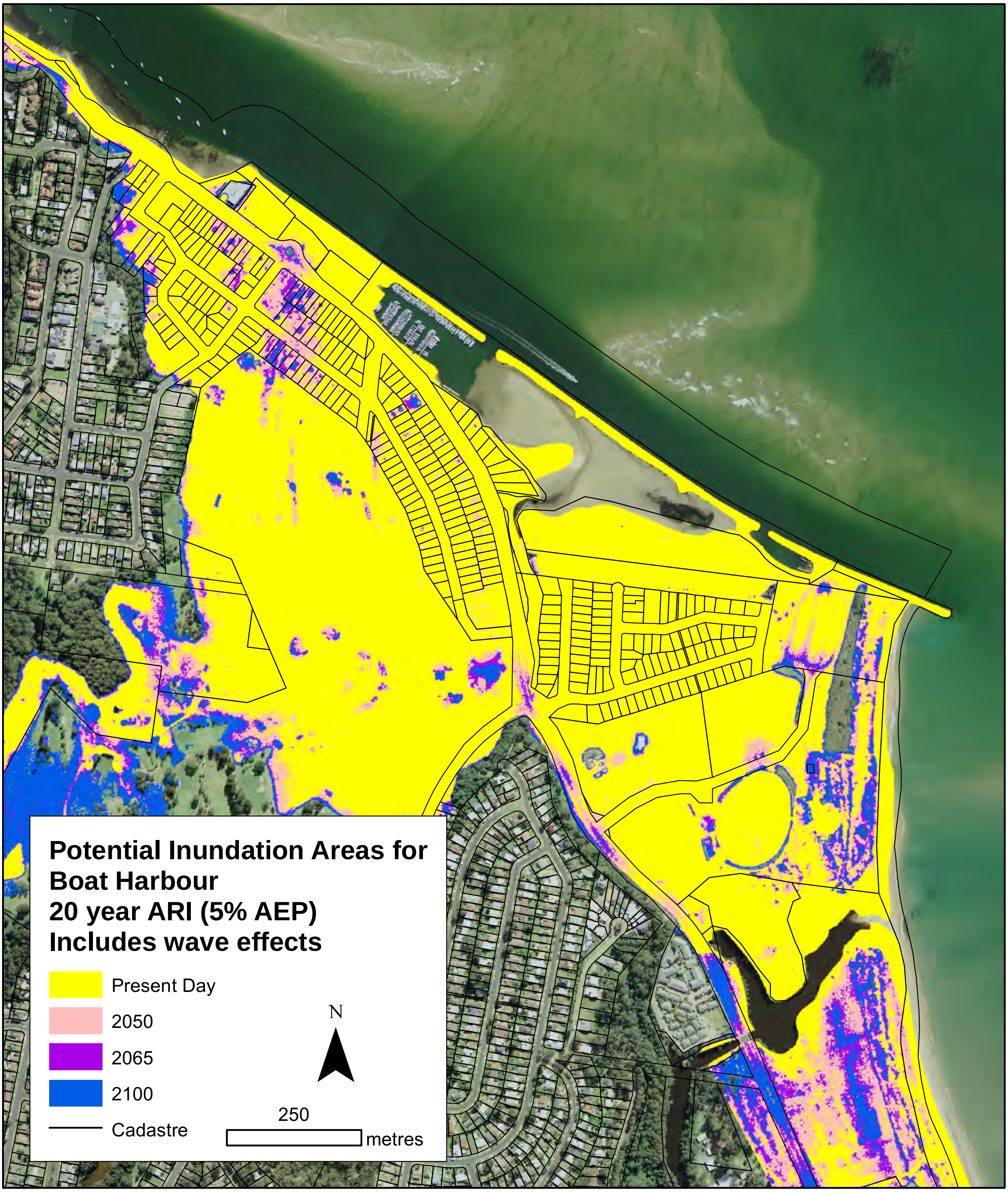

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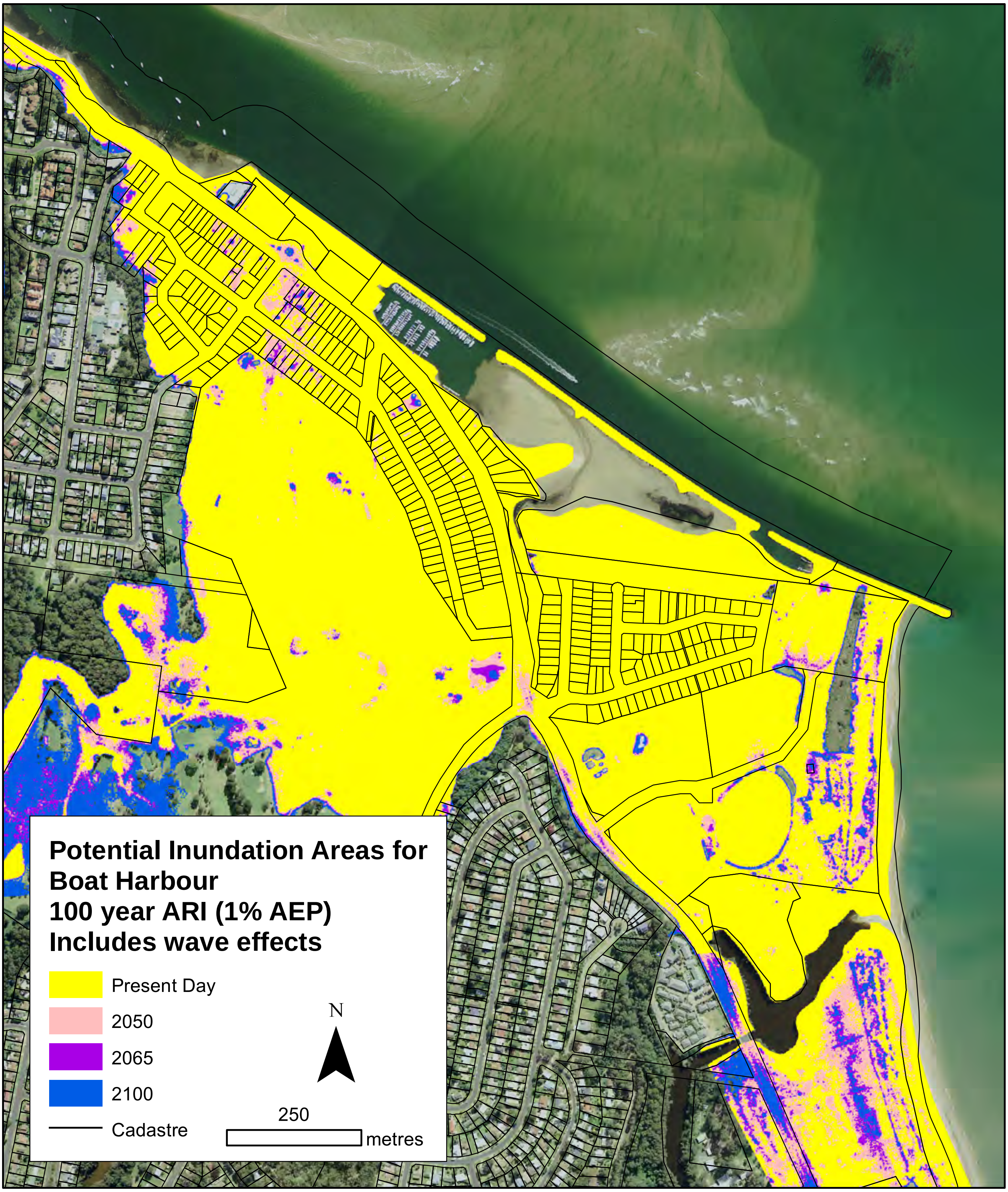
Figure L.32



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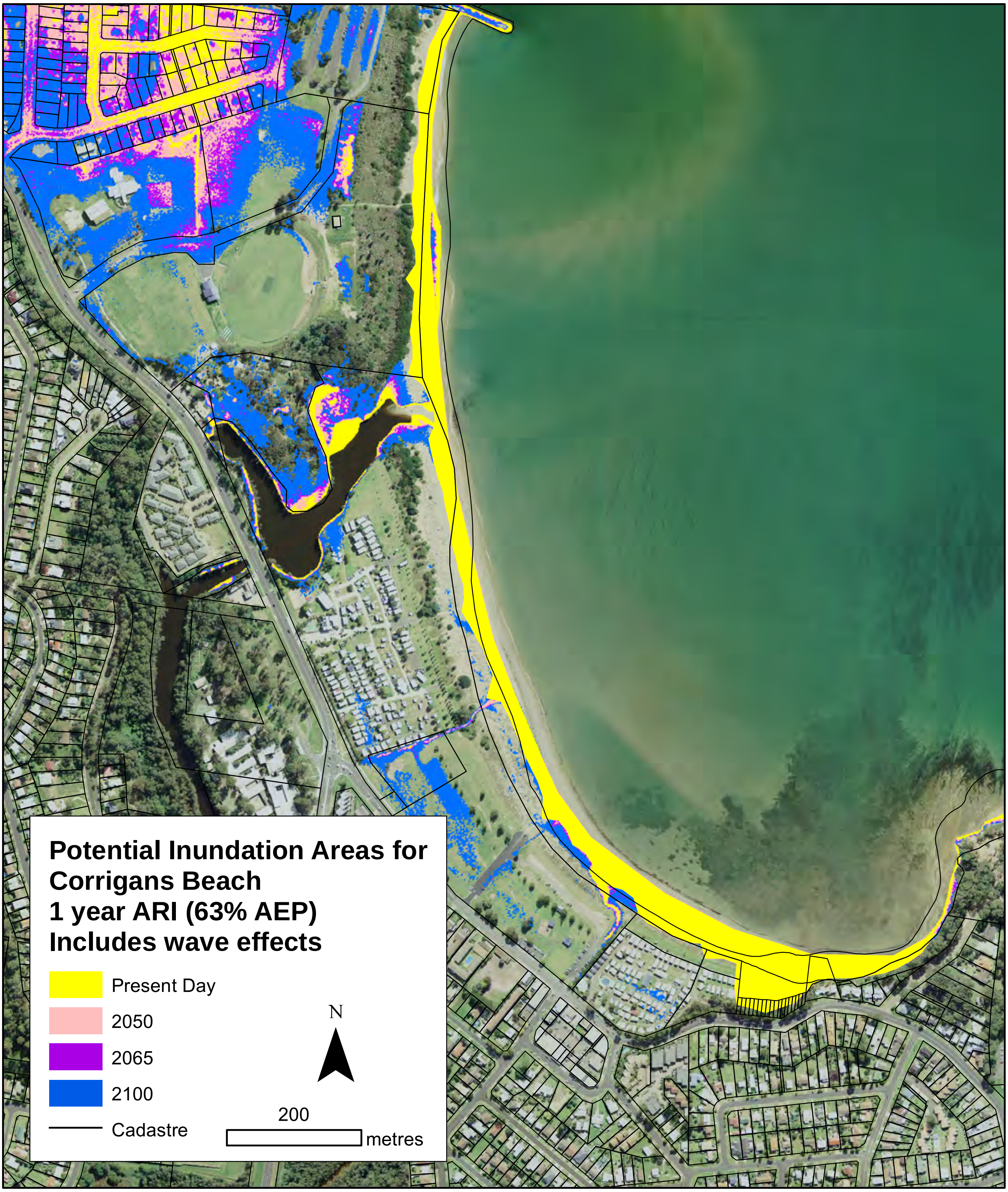

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Figure L.34

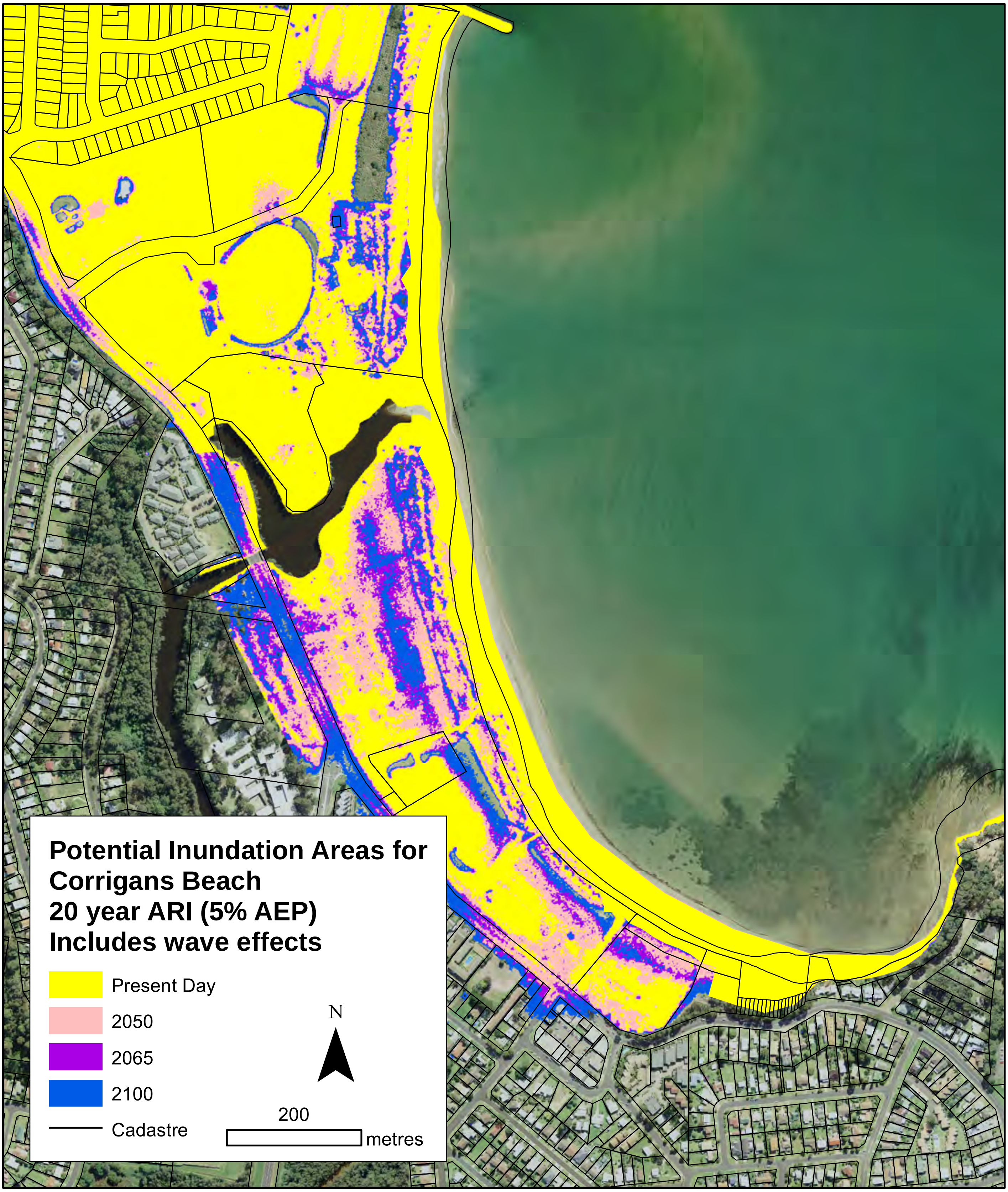

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Figure L.35

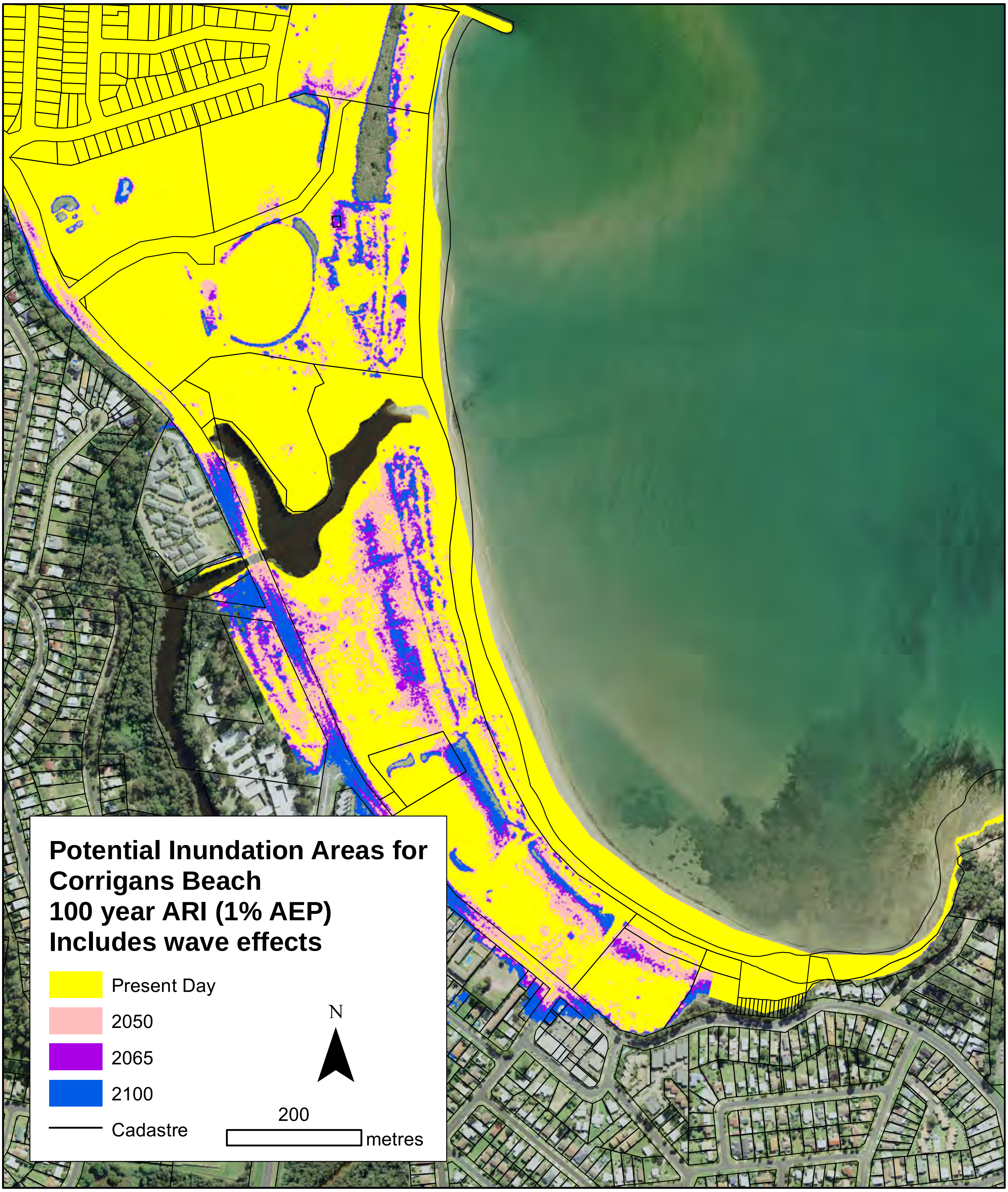

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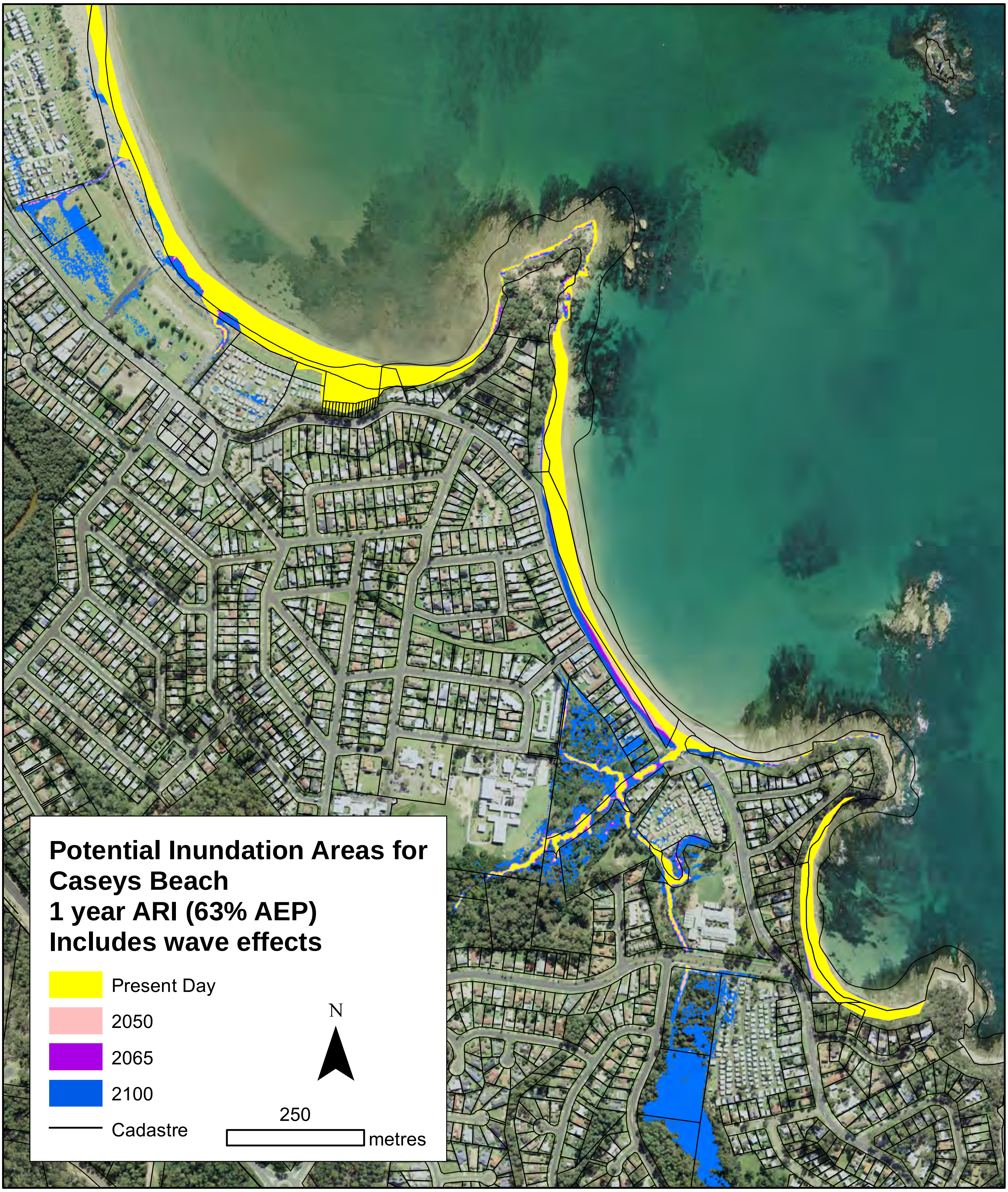
Figure L.36



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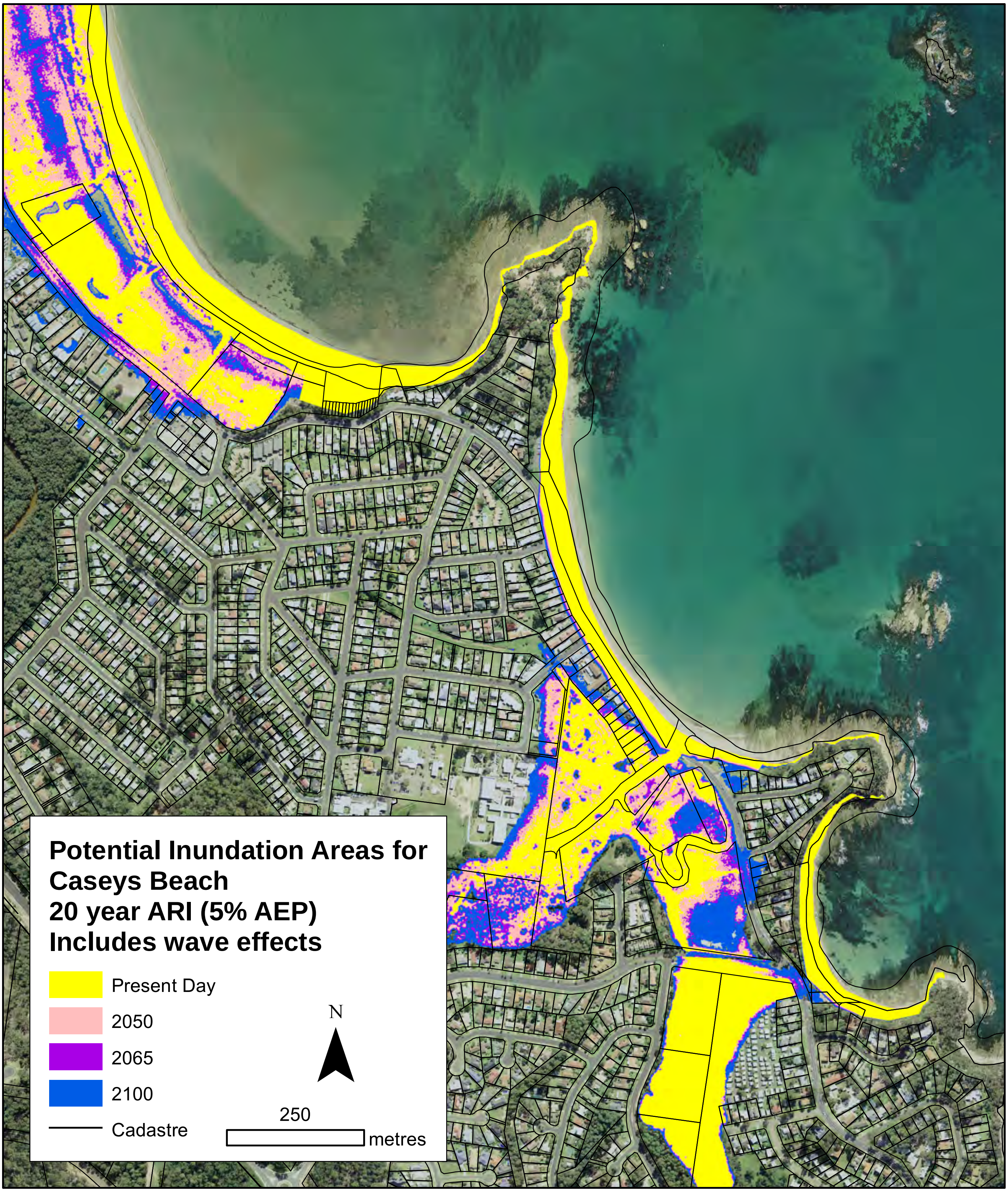

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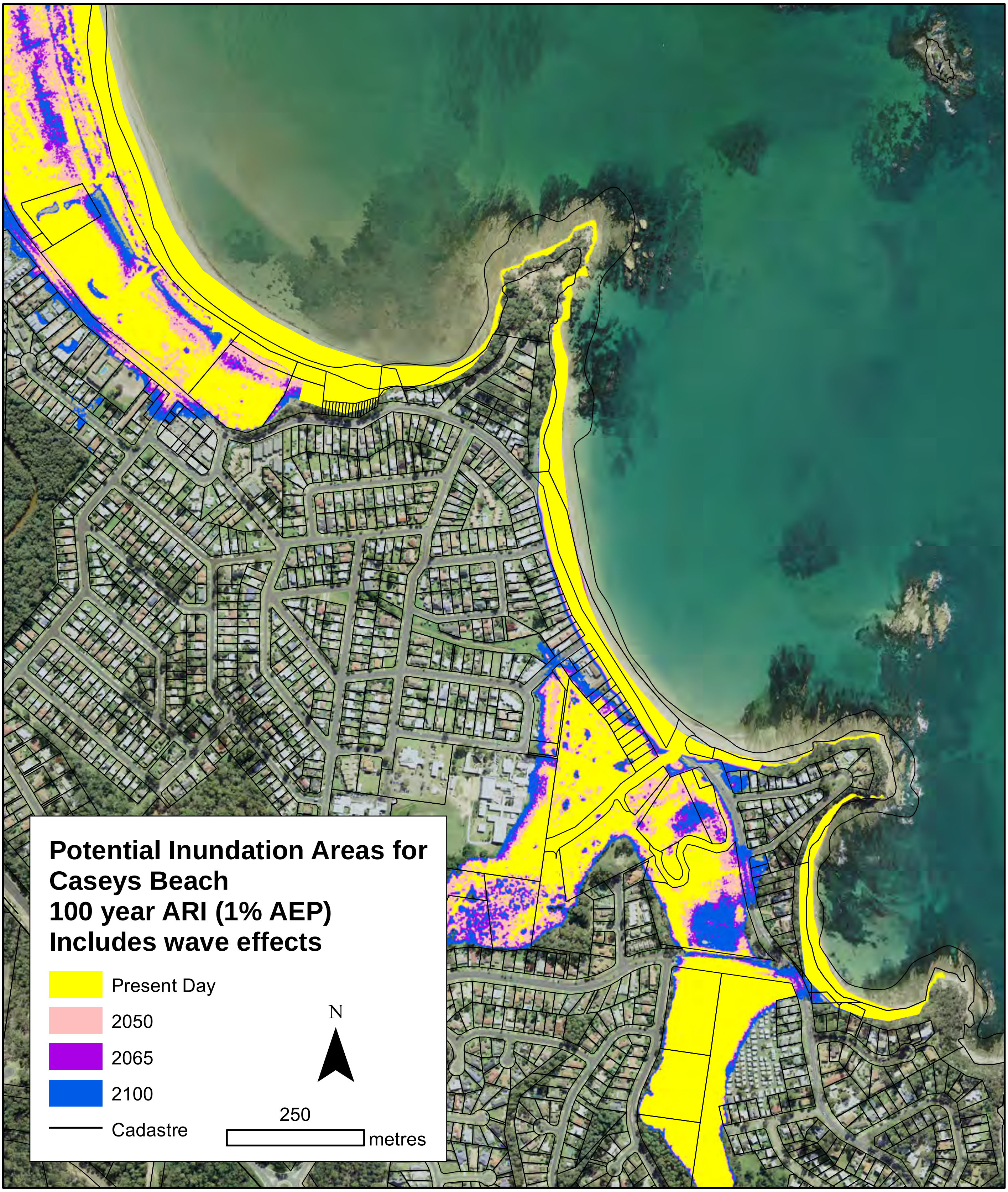
Figure L.38



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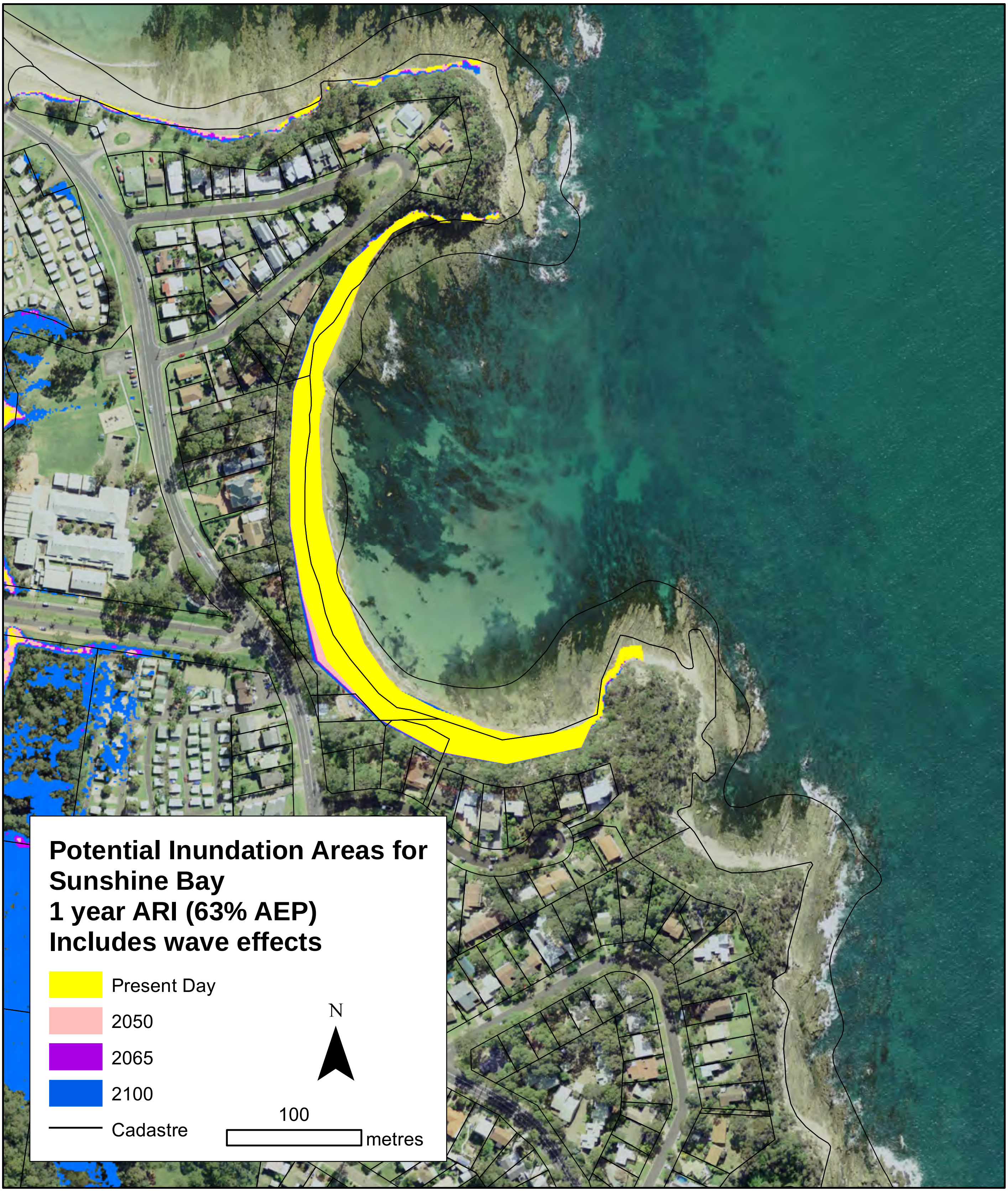

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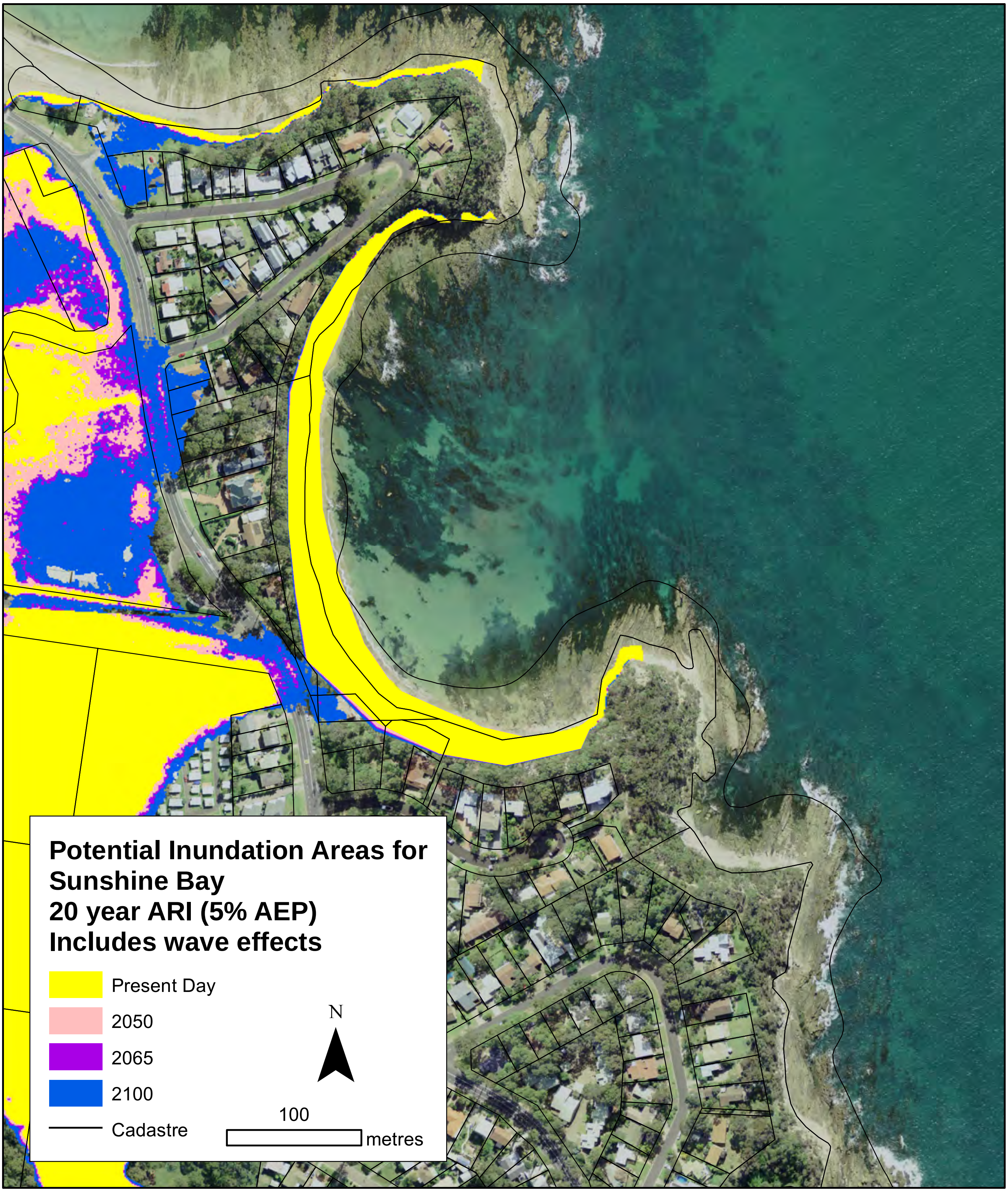
Figure L.40



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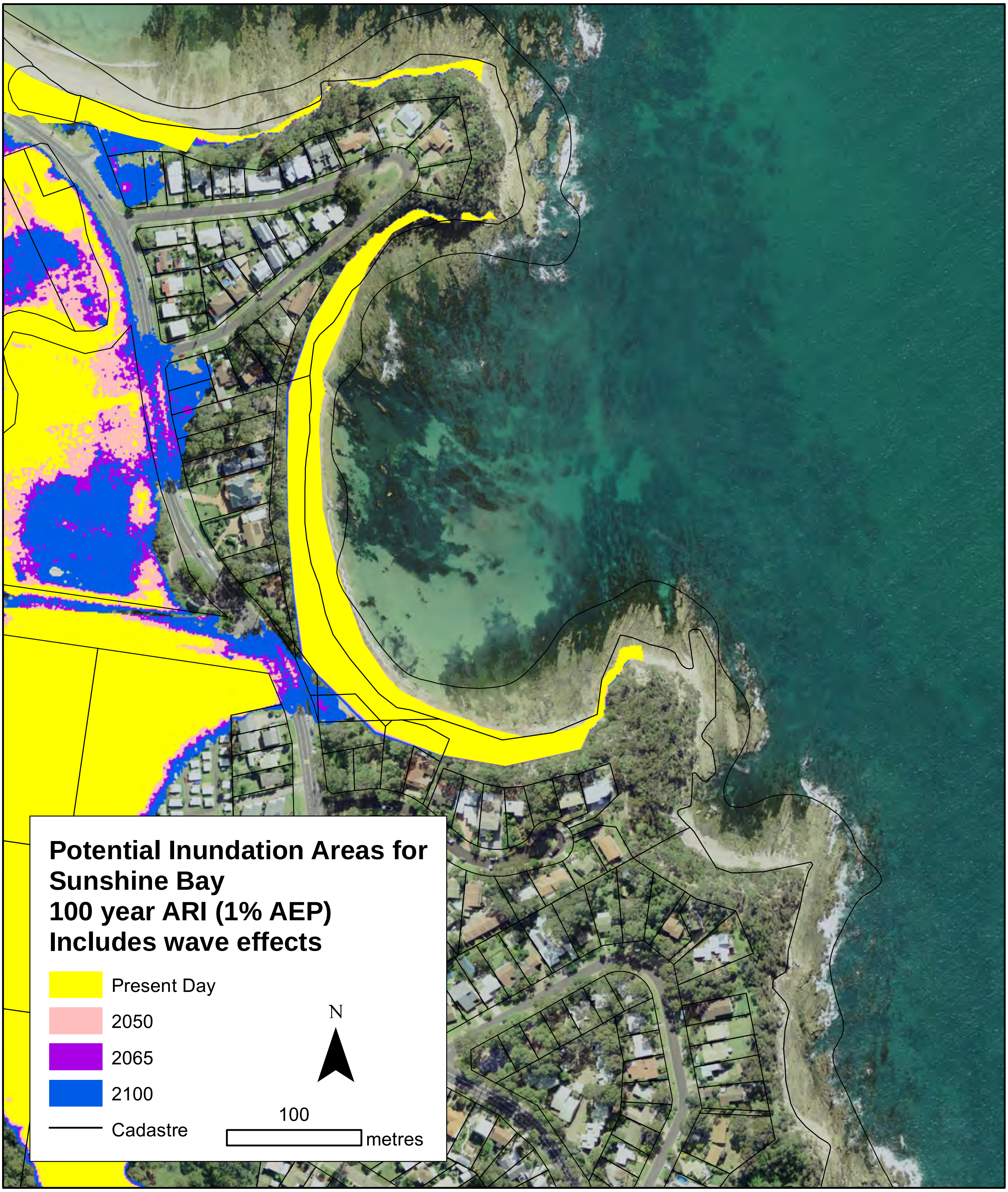

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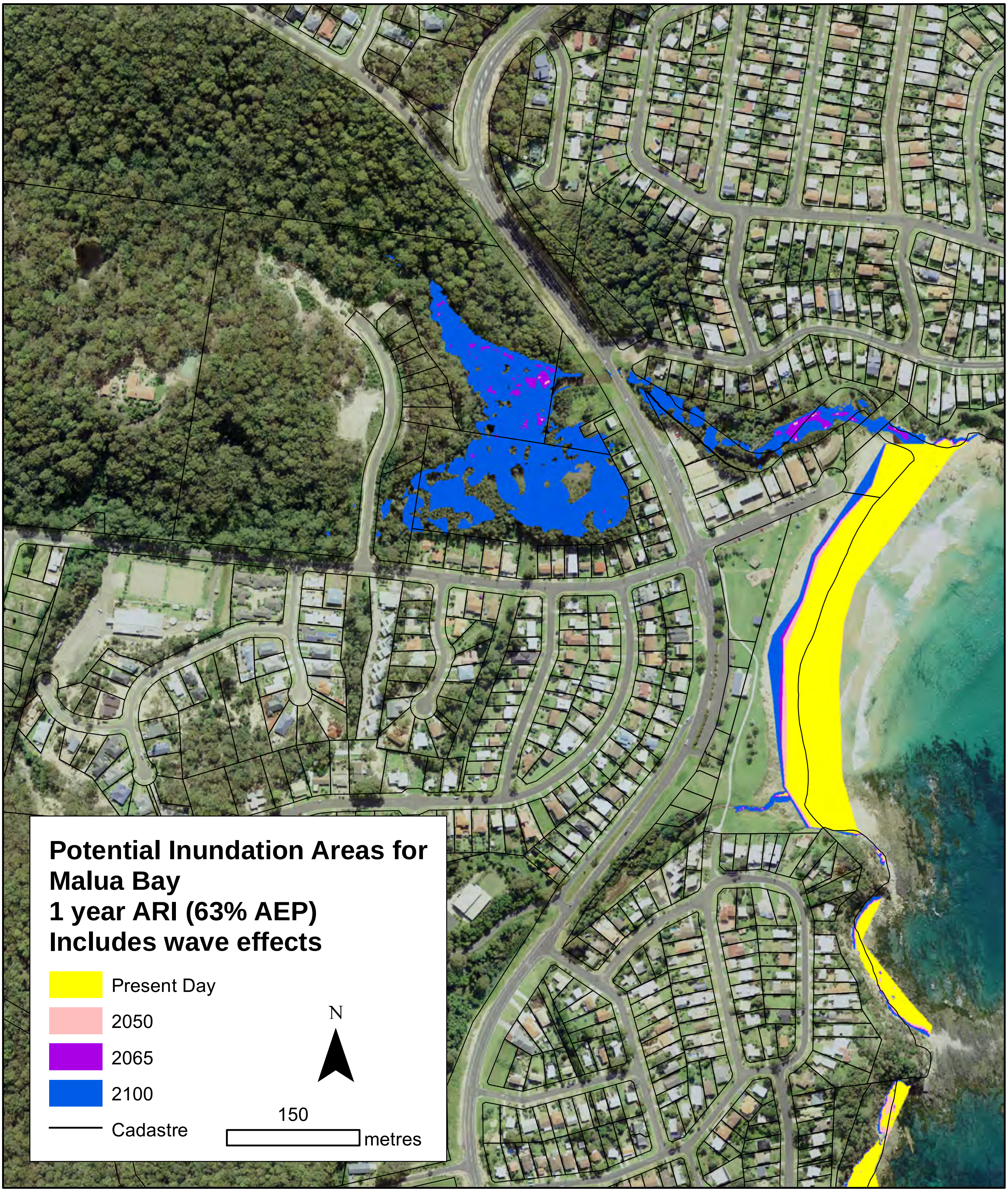
Figure L.42



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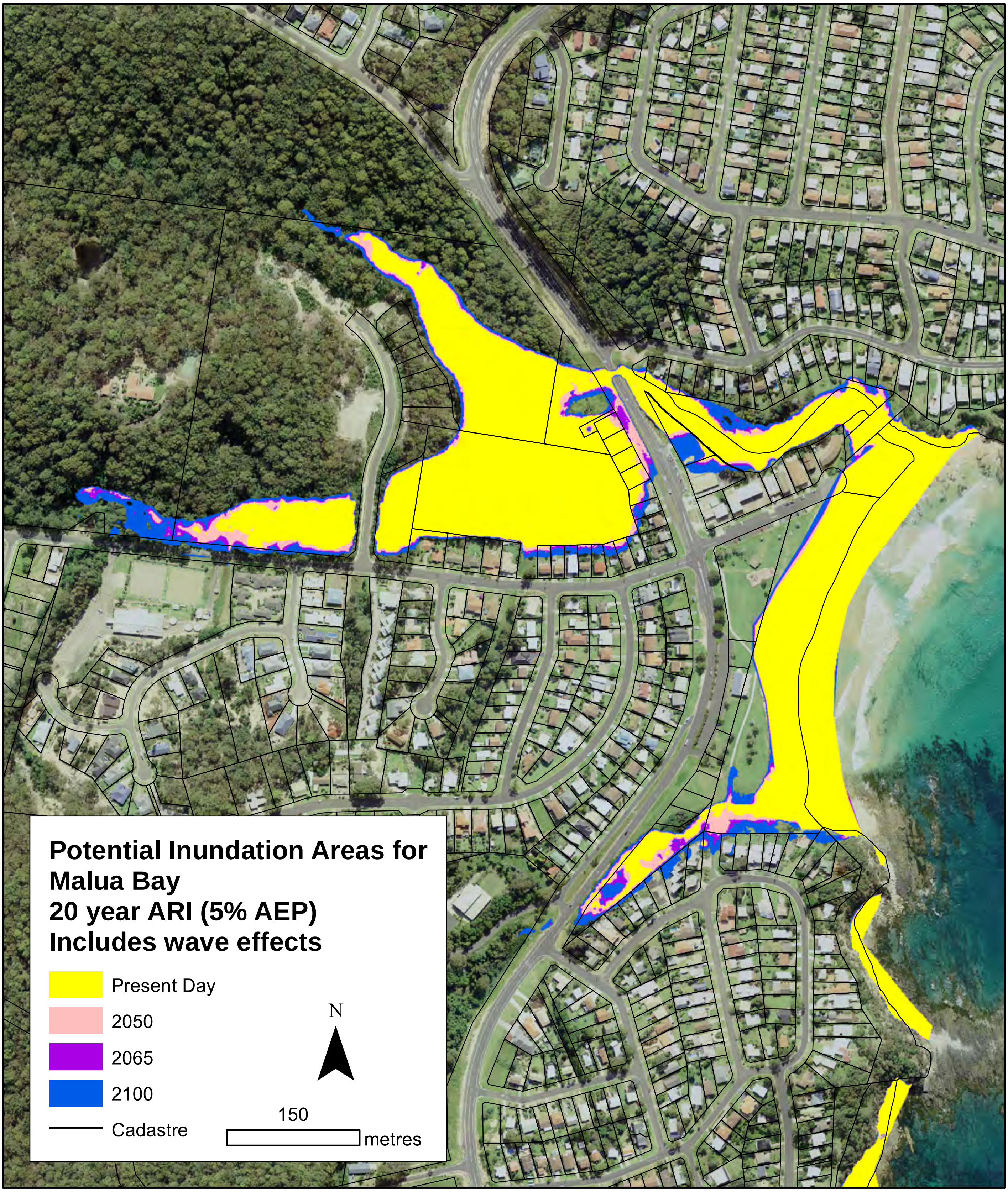

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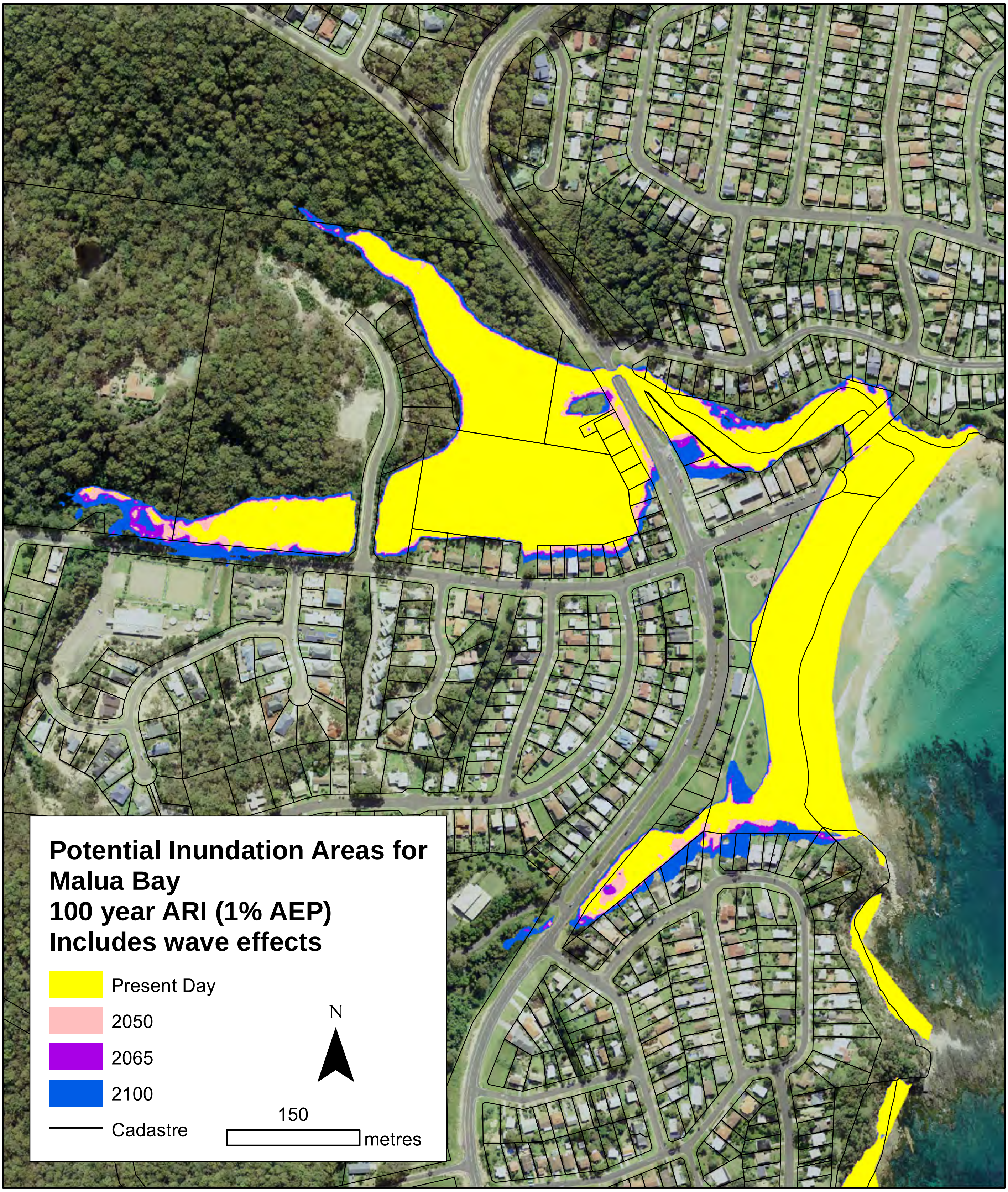

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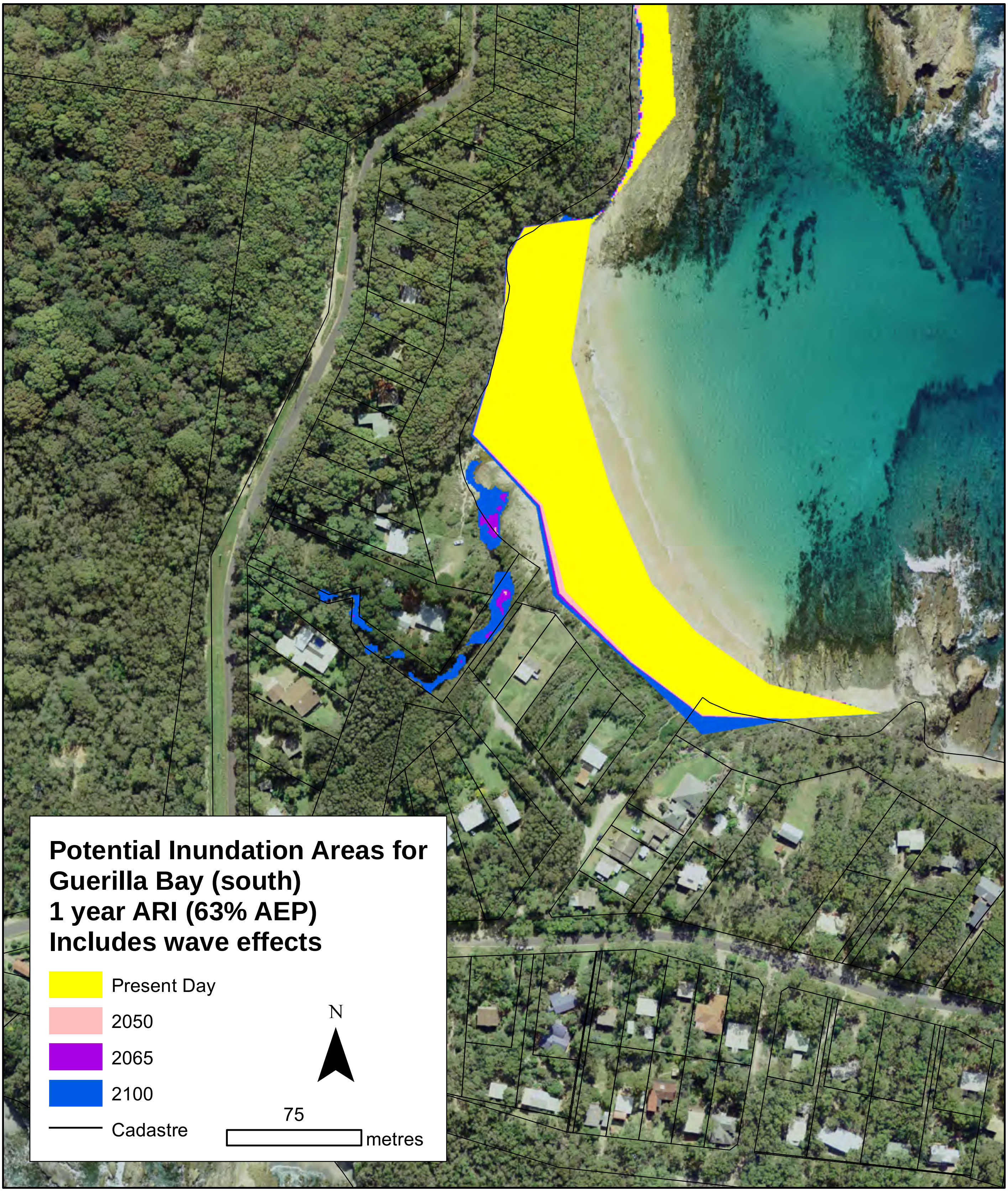

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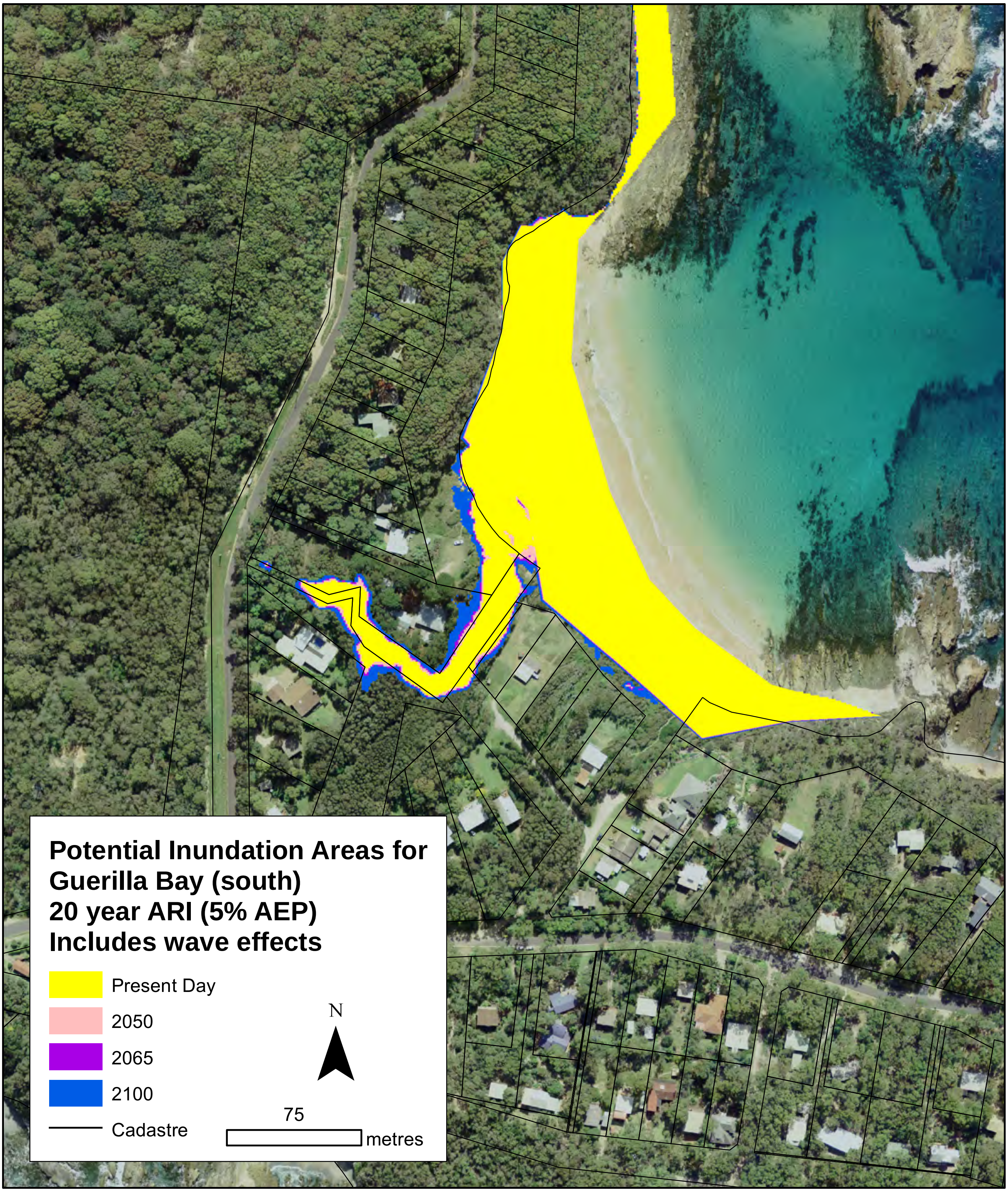
Figure L.46



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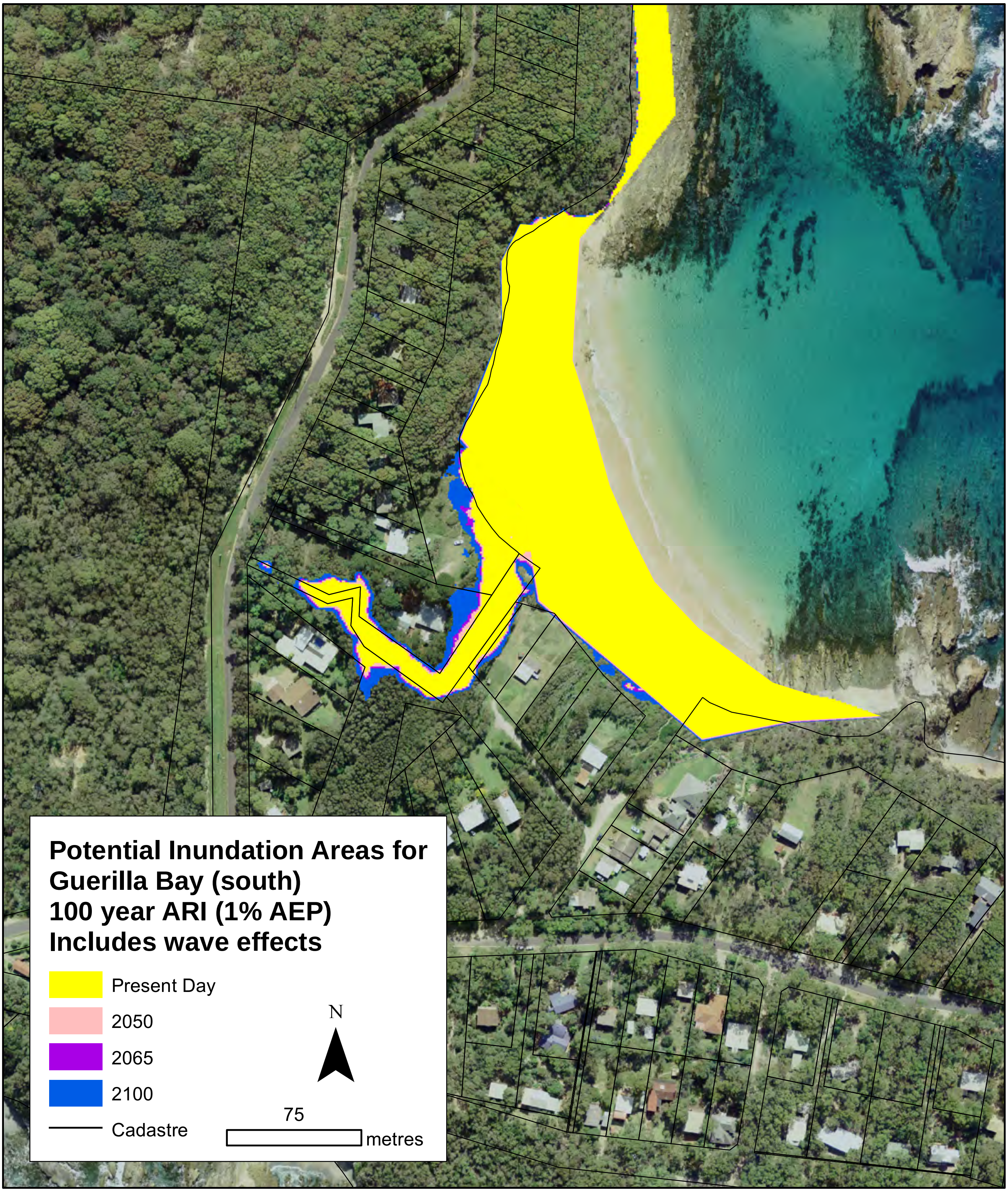

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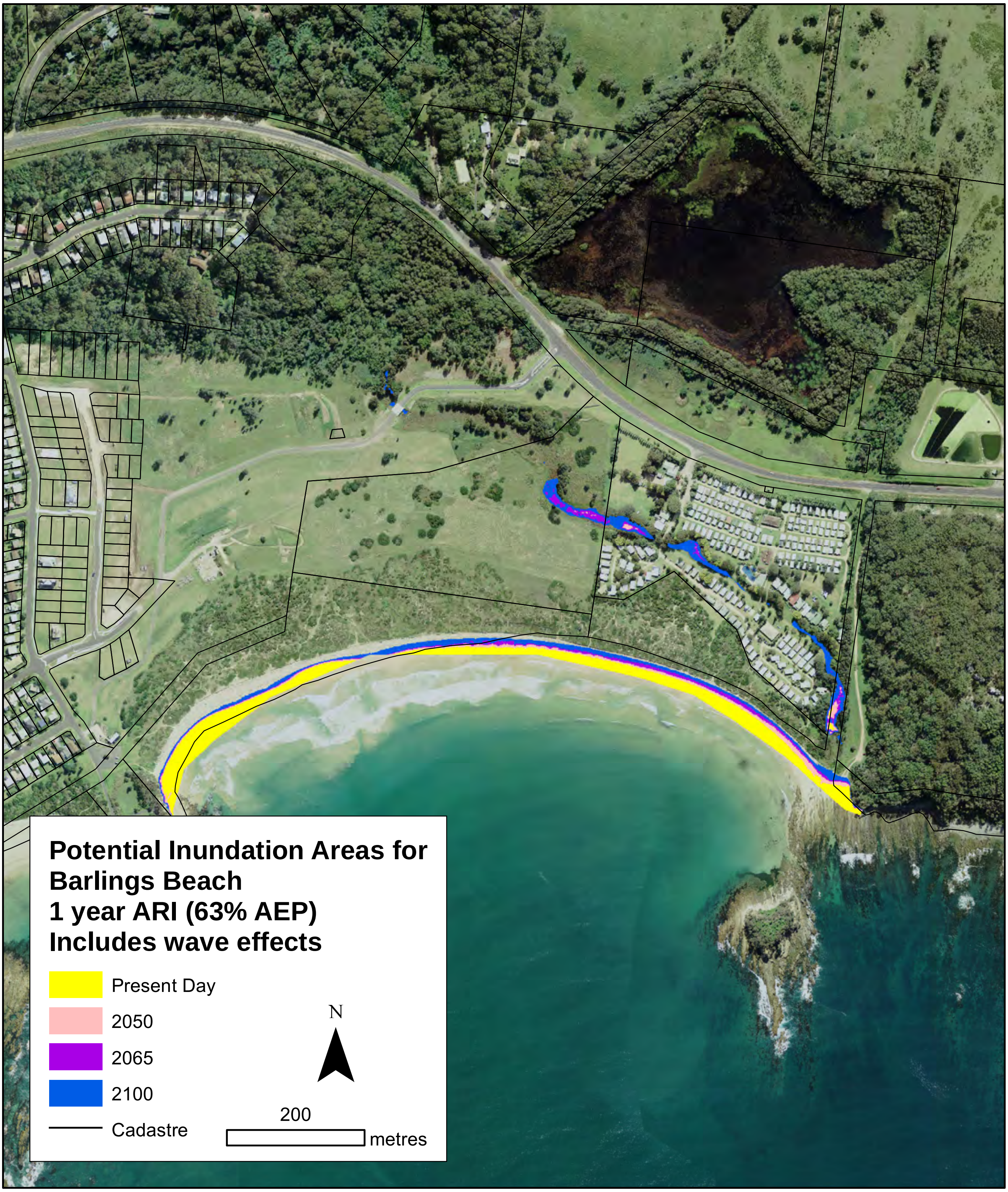
Figure L.48



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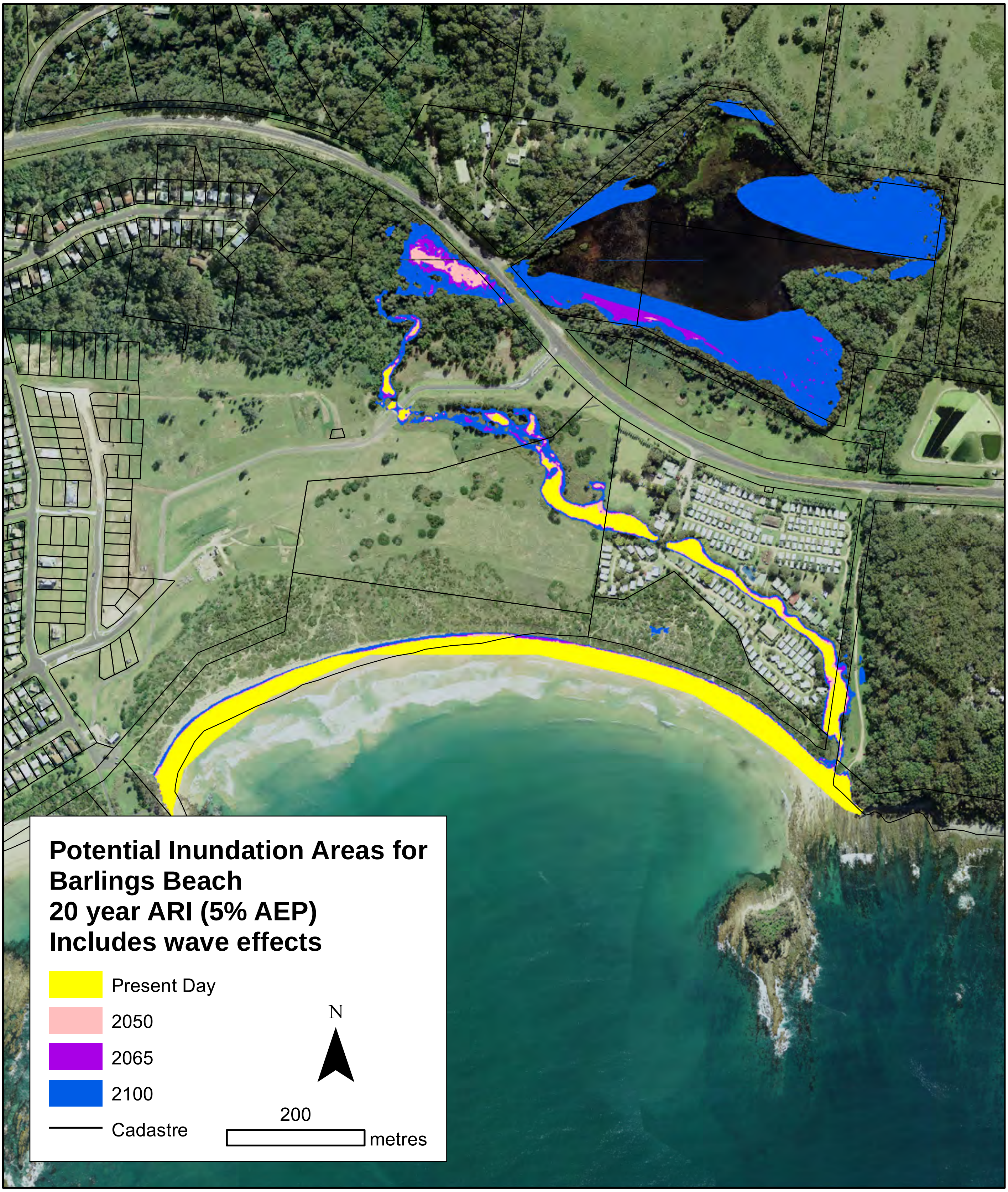

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Figure L.50

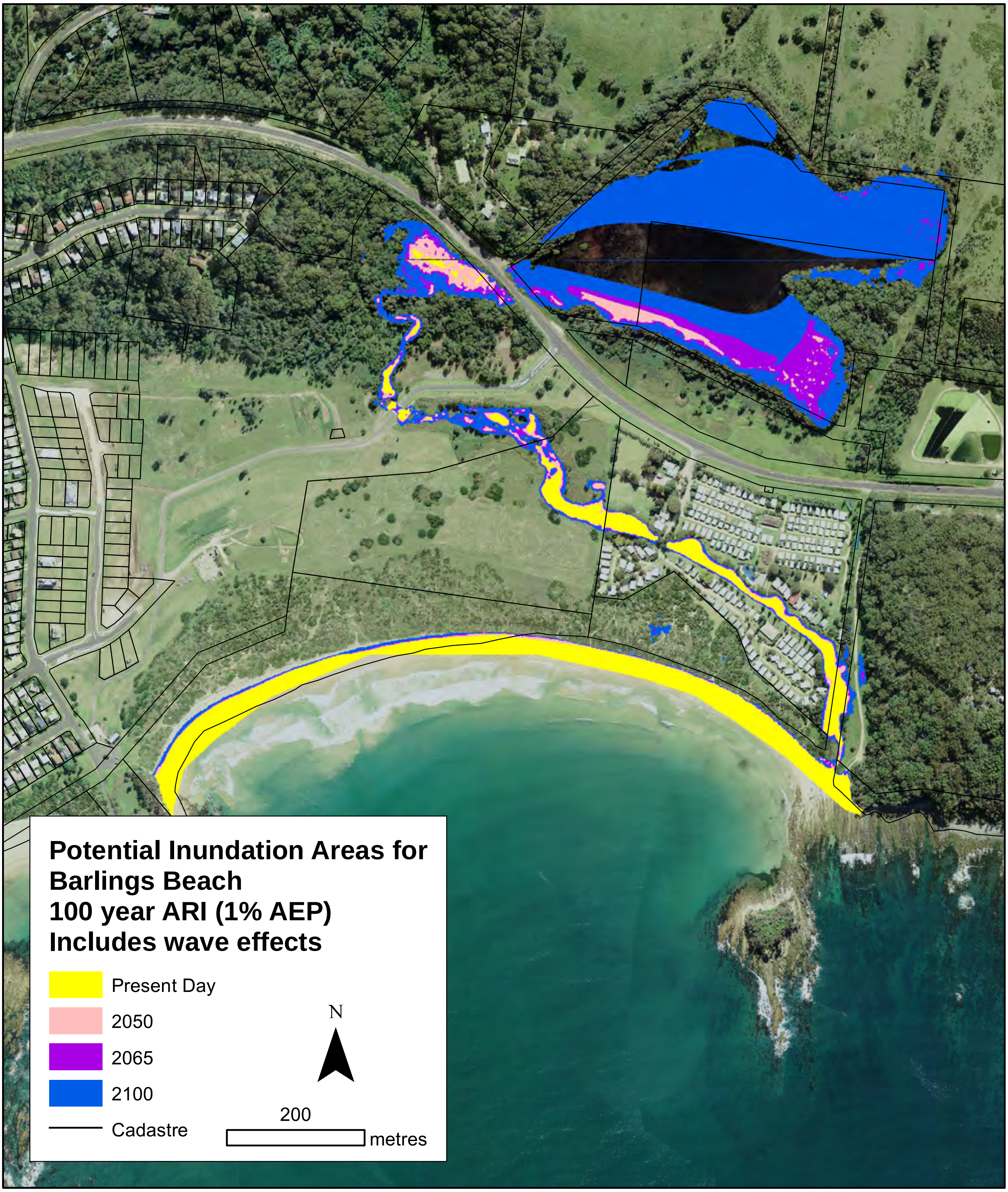

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Figure L.51

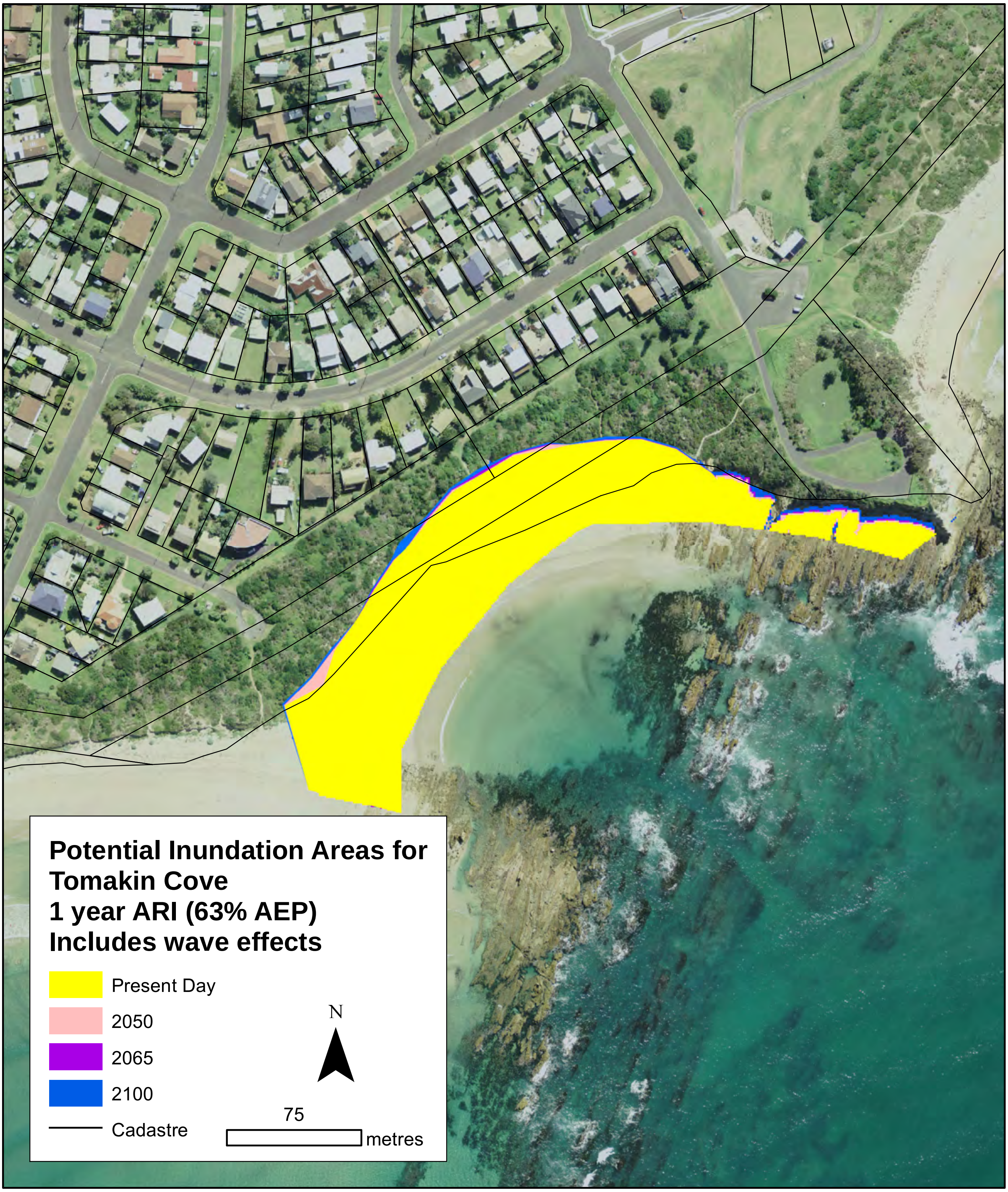

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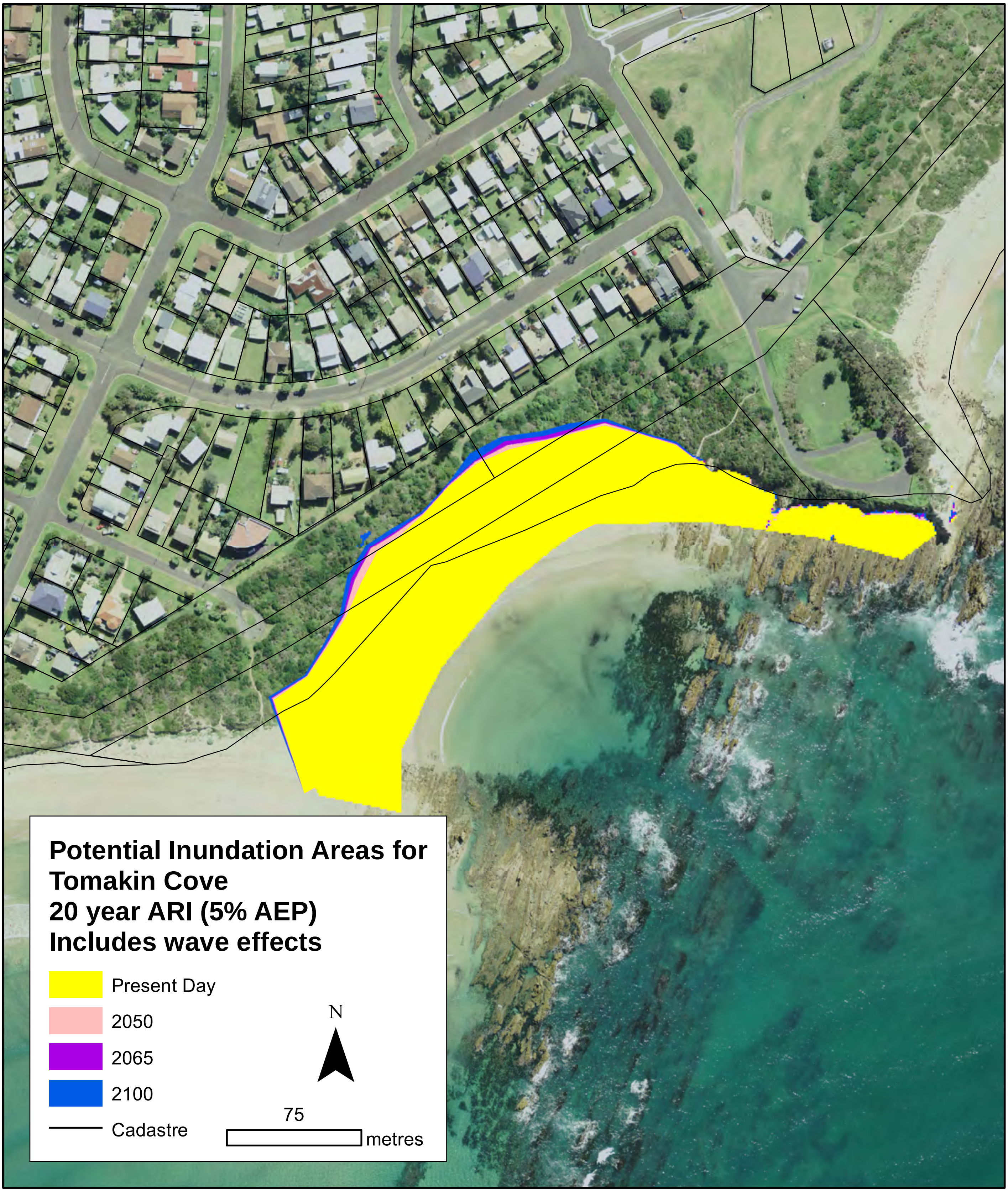
Figure L.52



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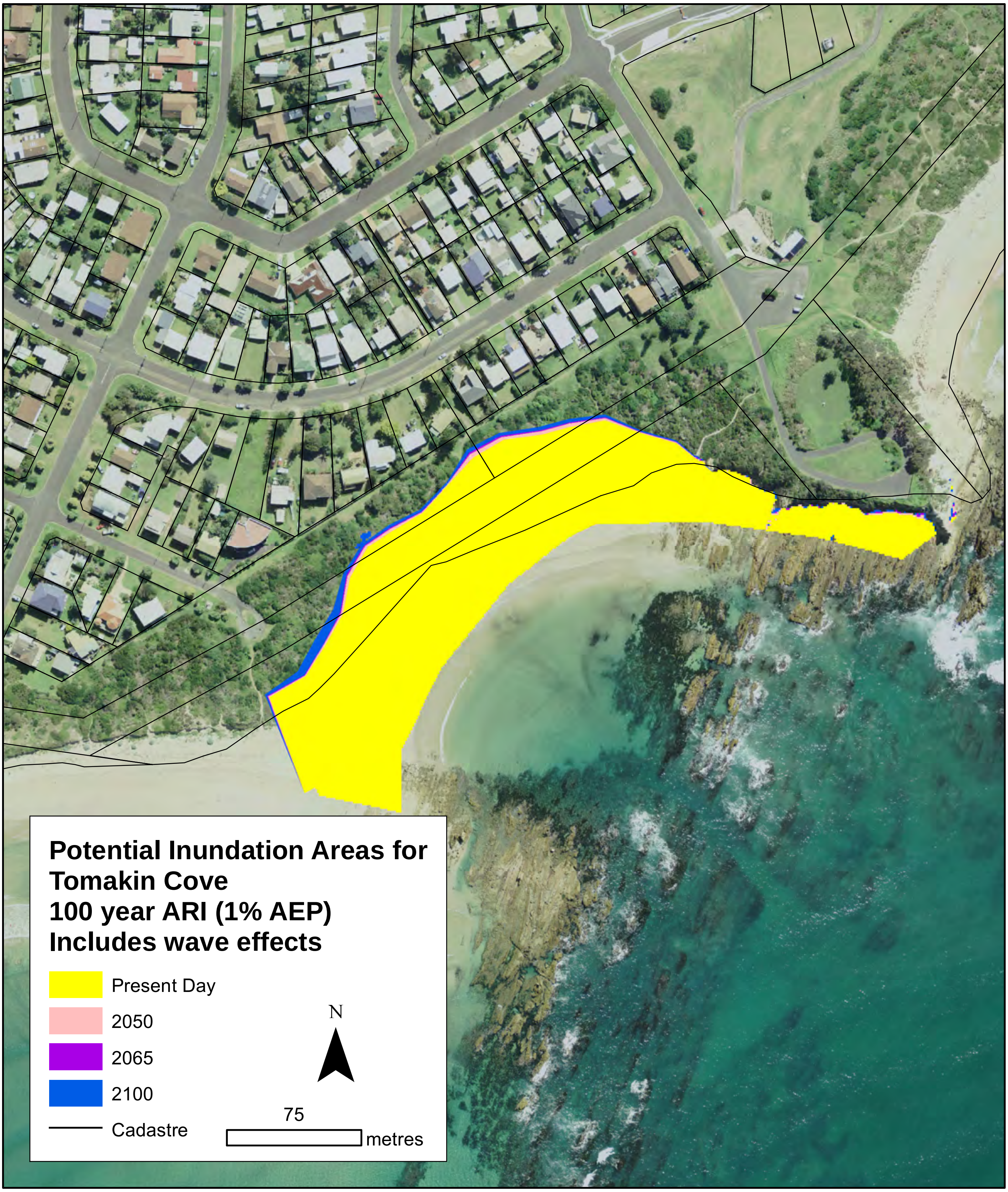

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Figure L.54

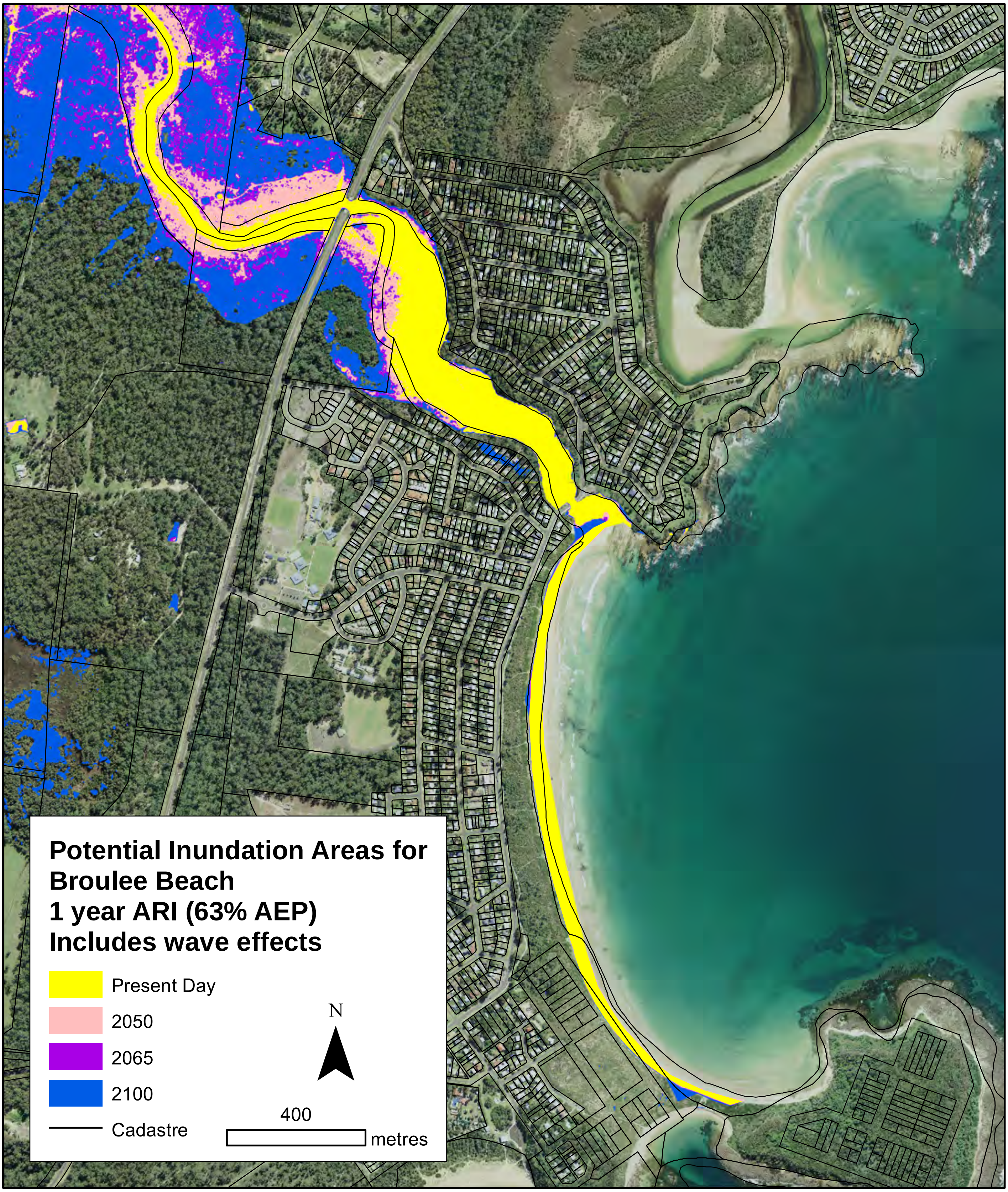

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Figure L.55

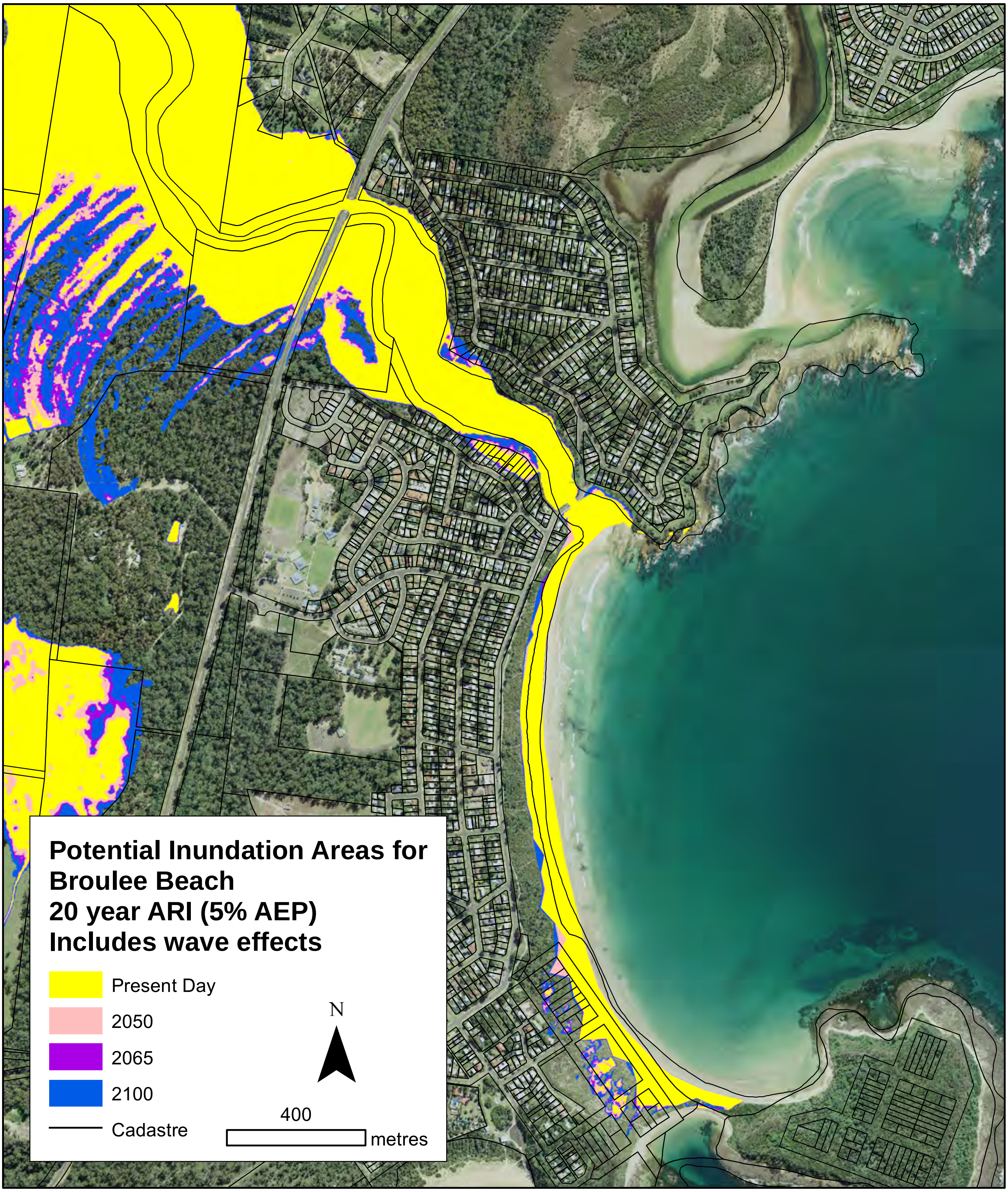

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Figure L.56

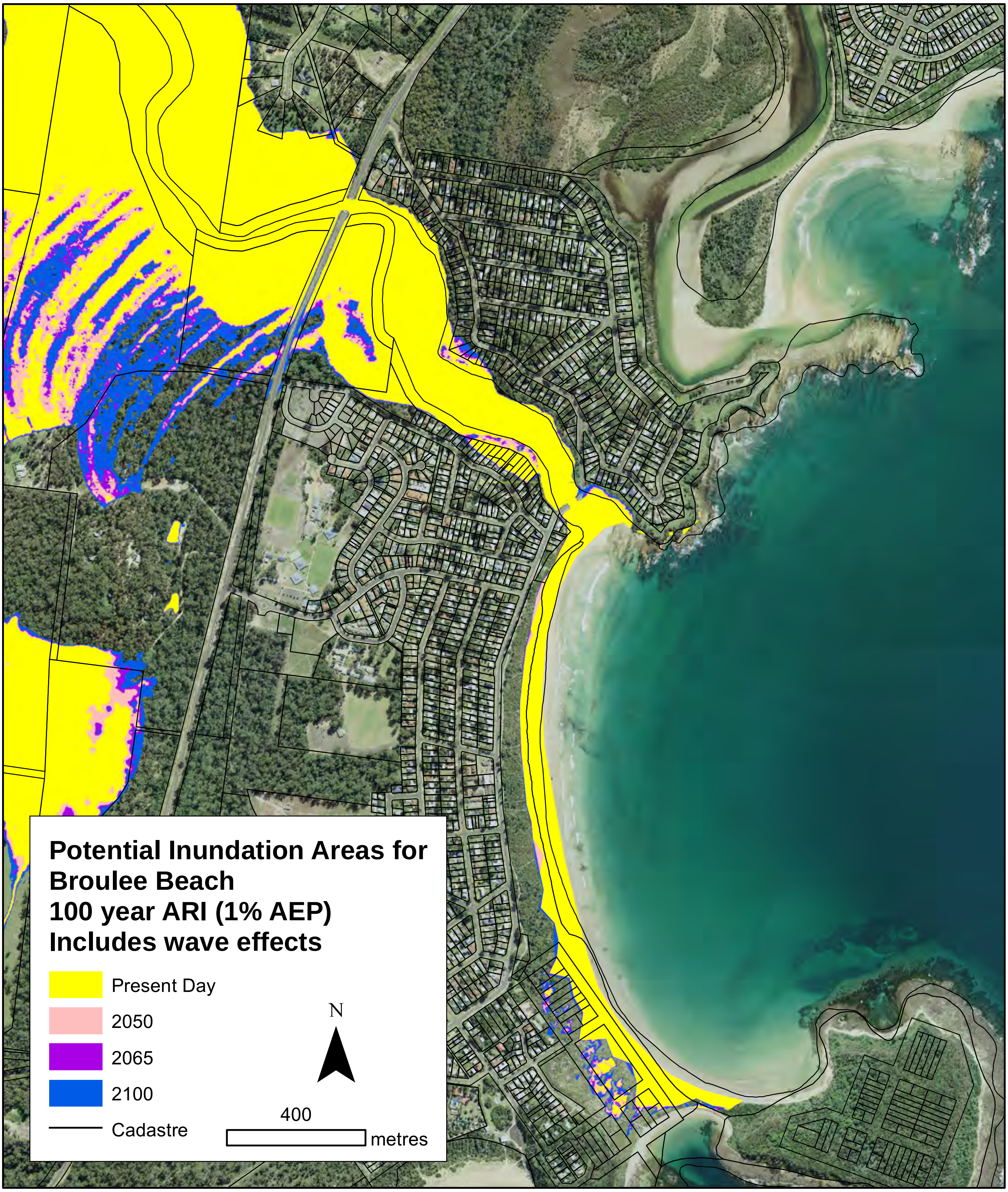

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Figure L.57

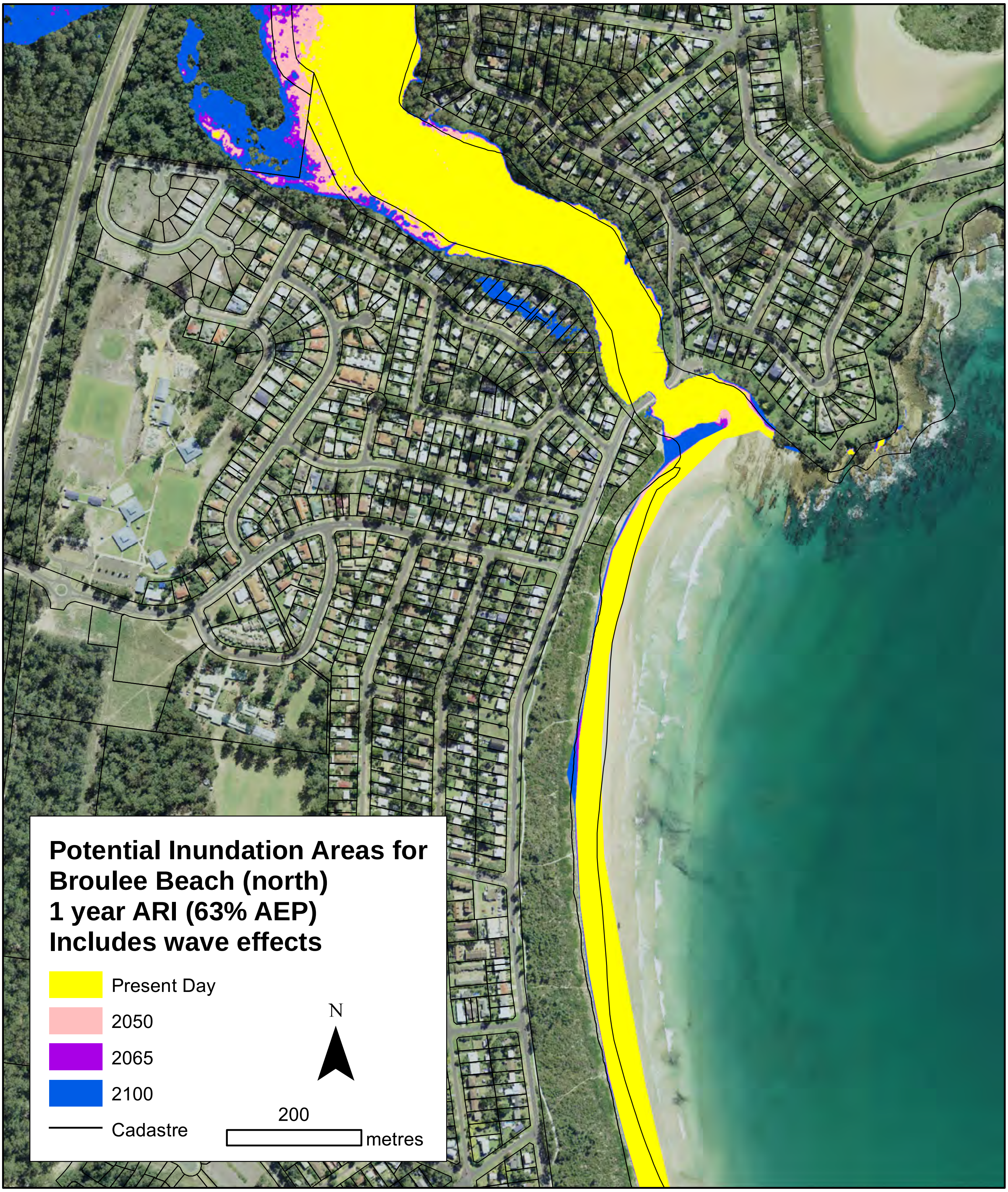

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Figure L.58

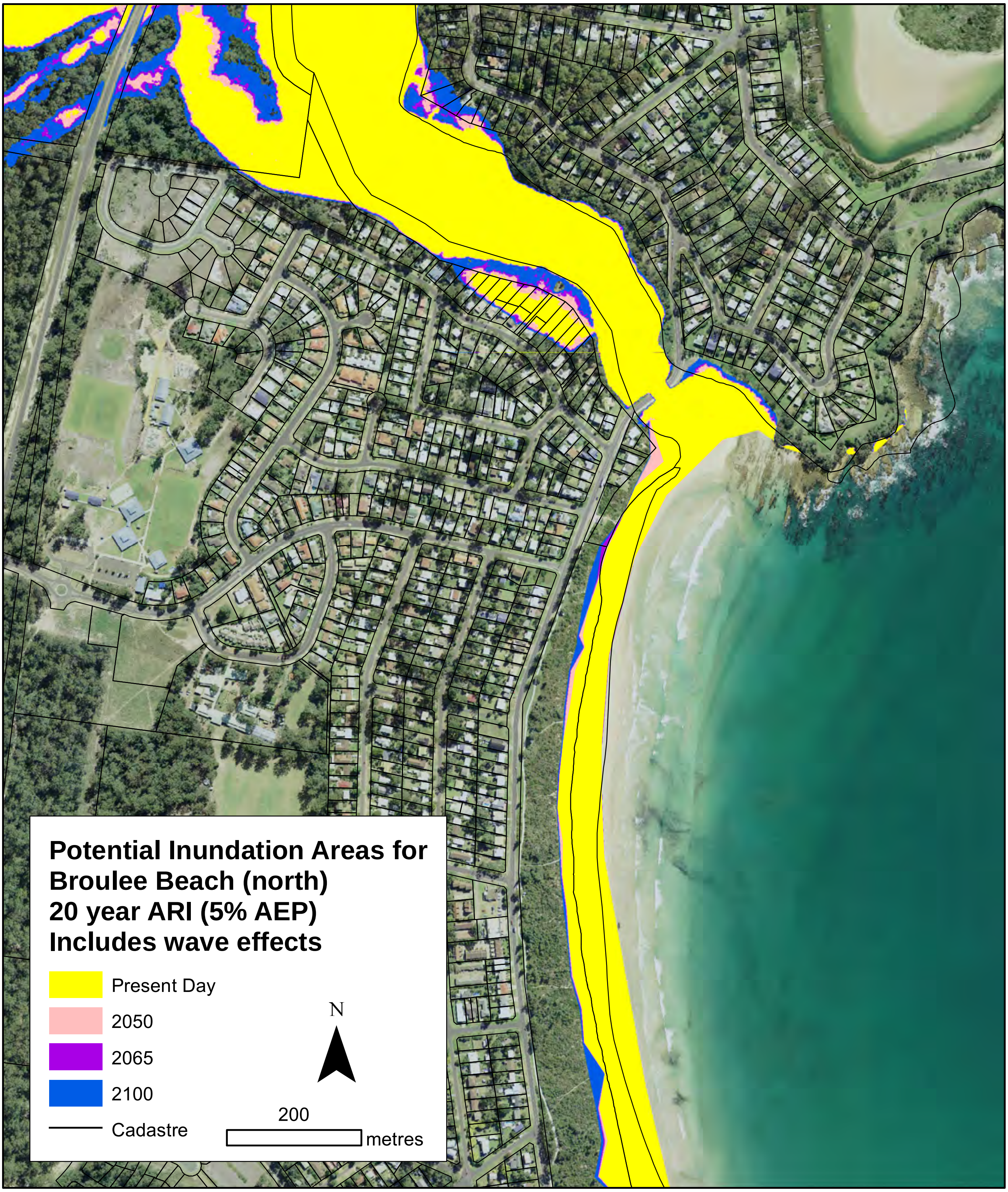

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Figure L.59

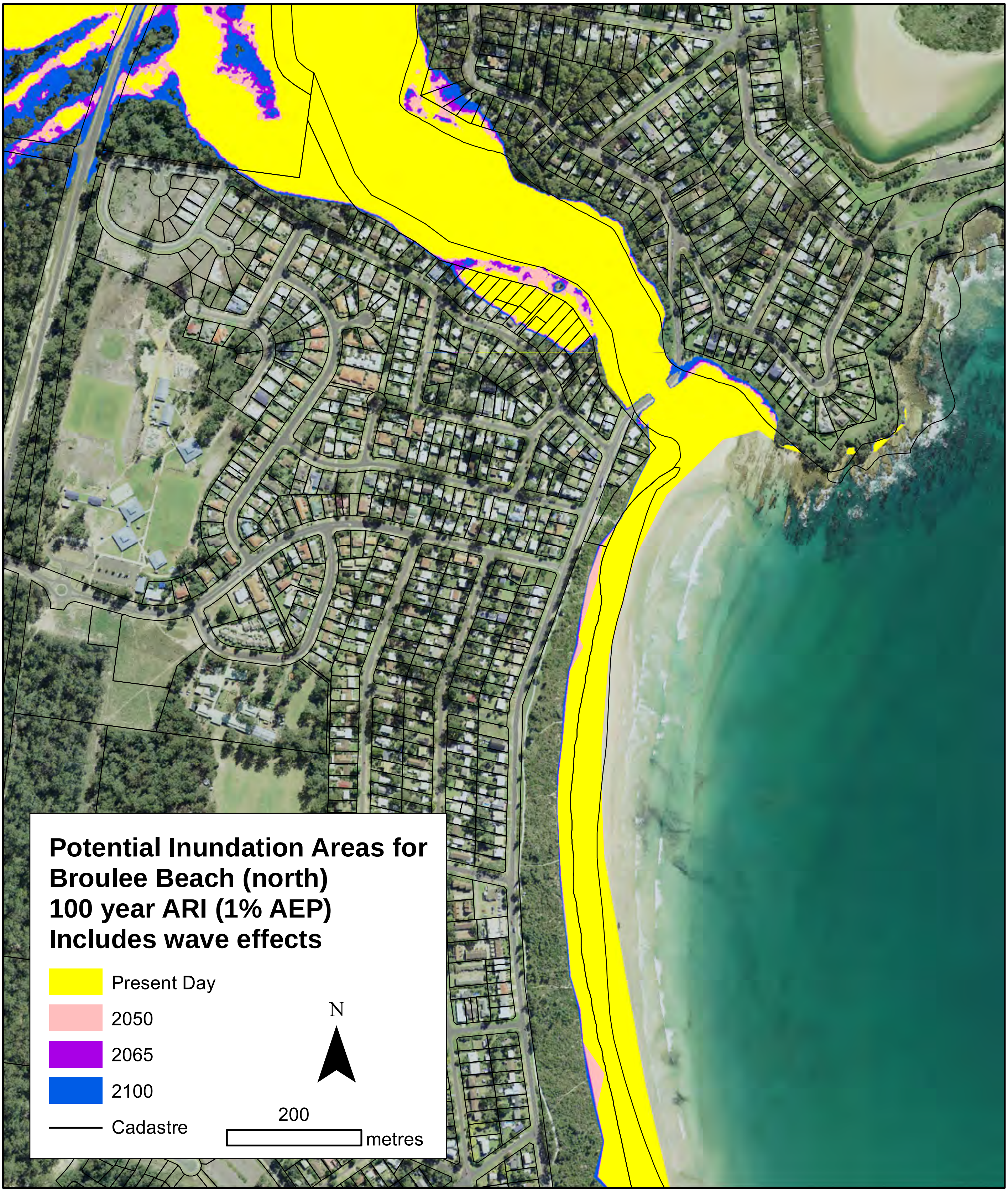

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Figure L.62

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Figure L.63

Appendix M: Durras Lake Tailwater Conditions

M.1 Preamble

WRL has completed a tailwater condition assessment for the entrance to Durras Lake, to be used as an ocean boundary condition for a future flood study of this estuary for the 63%, 5% and 1% AEP (1, 20 and 100 year ARI) storm events. This appendix outlines the methodology used to establish the site specific water level conditions for this site.

Durras Lake is as an ICOLL (Intermittently Closed and Open Lake or Lagoon) which is classified as a Type C Waterway Entrance in OEH's *Floodplain Risk Management Guide* (OEH, 2015). Since the entrance is likely to be exposed to open ocean waves, the maximum set-up equivalent to the set-up on an open ocean beach is relevant. WRL has undertaken site-specific, detailed analysis of wave setup for the Durras Lake entrance. However, as discussed in Section 8.3.2, the quality of nearshore bathymetry used in the analysis is unknown. WRL recommends that this analysis be repeated when a bathymetry survey is undertaken offshore of Durras Beach (south).

Example photos of the entrance shortly after being mechanically opened after heavy rainfall on 26 August 2014 are reproduced in Figure M-1.

M.2 Astronomical Tides

Using harmonic analysis, a synthetic tide was generated for Batemans Bay, (based on tidal constituents for Princess Jetty. Two tides were chosen for modelling of the Durras Lake tailwater conditions:

- 5% and 1% AEP modelling: 14/06/2011 – 20/06/2011, maximum water level 0.82 m AHD; and
- 63% AEP modelling: 19/09/2014 – 24/09/2014, maximum water level 0.502 m AHD.

The 63% AEP tidal series has been chosen to best represent a MHW tidal condition, while the rarer events have been chosen to coincide with a higher tidal condition.

M.3 Tidal Anomalies

As discussed in Section 3.3.2, adopted offshore extreme water levels for the study area are reproduced in Table M-1. These levels do not include wave setup or wave runup.

Table M-1: Adopted Extreme Water Levels (excluding wave setup and wave runup)

AEP %	ARI (years)	Water Level (m AHD)
63	1	1.22
5	50	1.37
1	100	1.43
Mean High Water		0.508

Figure M-1: Durras Lake Entrance Shortly After Mechanical Opening 26 August 2014 (Source: Durras Lake North Holiday Park, 2014)

M.4 Design Wave Conditions

Significant wave heights were extracted from the SWAN wave modelling (as discussed in Appendix D) at the point where approximately 1% of waves were breaking for storm durations between 1 hour and 144 hour for each appropriate recurrence interval. The wave periods and duration of the design storm event is based on the extreme wave analysis in Shand et al. (2011) for the Eden NSW wave buoy, the closest site available to Durras Lake. However the wave statistics from Batemans Bay wave buoy have been used (generally 5 – 15% smaller than the wave climate at Eden). The synthetic 1%, 5% and 63% AEP storm (H_{RMS} and T_p) timeseries is shown in Figure M-2. To investigate the wave setup expect to occur at Durras Beach, the root mean square wave height H_{RMS} (m) corresponding to the significant wave heights extracted from the SWAN model was first calculated according to CIRIA (2007) in Equation M.1.

$$H_{RMS} = 0.706 \times H_S \tag{M.1}$$

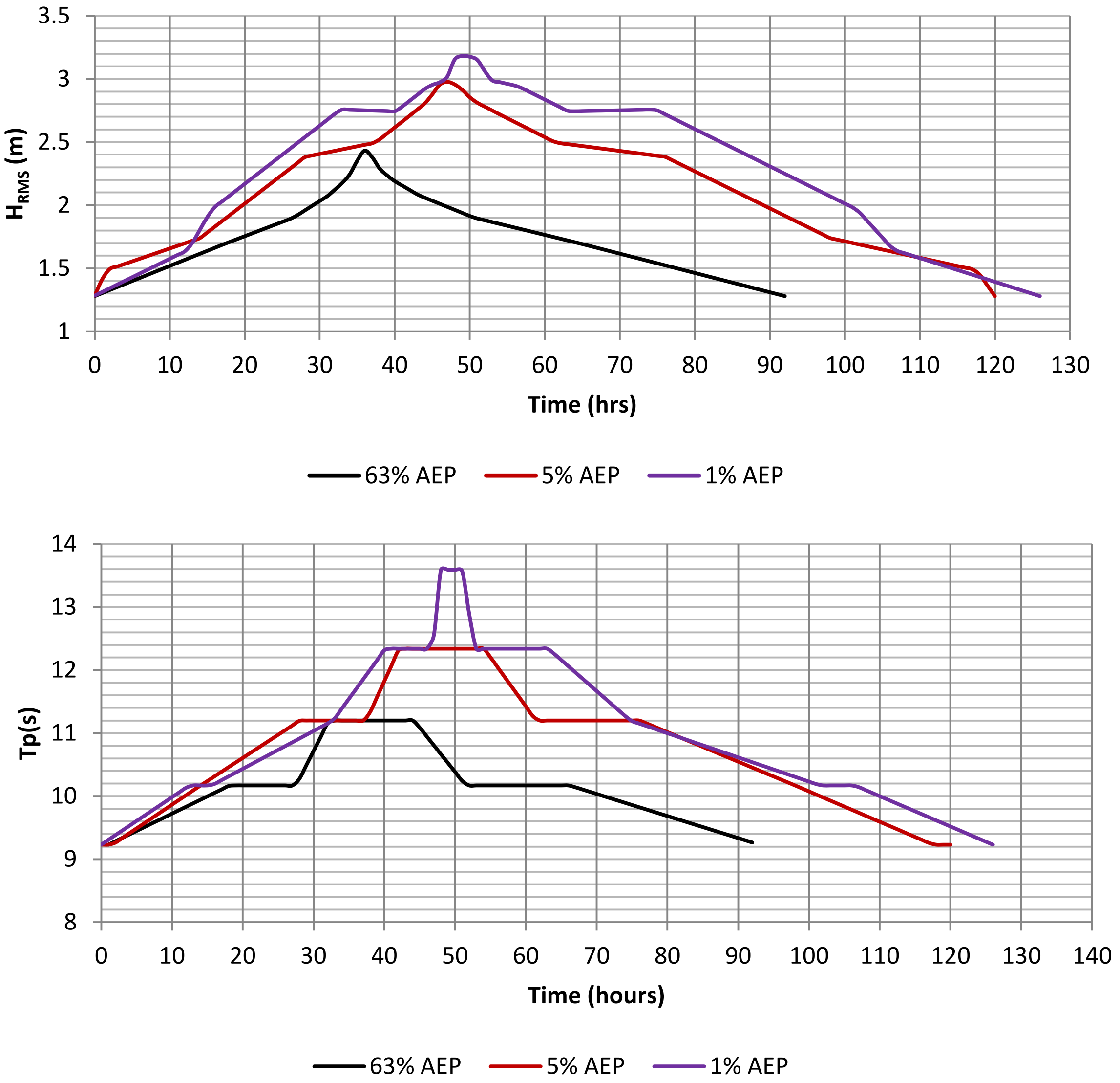

Figure M-2: Synthetic Design Storms (H_{RMS} and T_P) used for Durras Lake Tailwater levels

The Durras Lake tailwater modelling has been based on a profile immediately south of the lake entrance (shown in Figure M-3). Topographic information was extracted of the subaerial portion of the profile from the 2011 LIDAR. As no recent near shore bathymetric data was available for Durras Beach, an Dean Equilibrium Profile (Dean, 1977) was assumed based on a grain size of 0.37 mm, as per the methodology in Section 8.3.2.

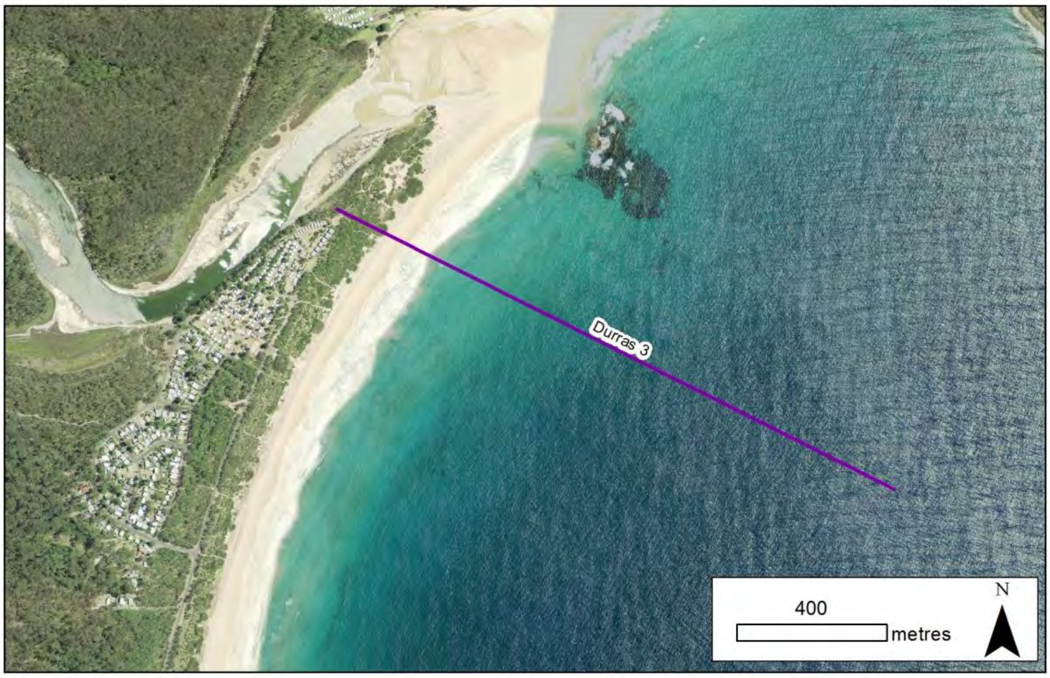

Figure M-3: Location of tailwater conditions analysis for Durras Lake

M.4.1 Phasing of Extreme Ocean Water Levels and Design Wave Conditions

While the 20 and 100 year ARI events wave conditions have been combined with the 20 and 100 year ARI events for water level conditions (tide plus anomaly), respectively, 1 year ARI wave conditions have been combined with the Mean High Water (MHW) level (0.51 m AHD) as previously agreed with OEH. This wave and water level combination is considered more representative of that which would result in 1 year ARI coastal inundation rather than assuming complete dependence of the variables (i.e. 1 year ARI waves and 1 year ARI water level).

M.4.2 Wave Induced Water Level Components

Breaking waves cause an elevated water level in the surf zone due to the radiation stress of the breaking waves being balanced by a gradient in the water surface. The storm waves shown in Figure M-2 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). From this model, total water levels, including wave setup, were extracted at 15 minute time intervals to define the design tailwater conditions discussed in the following sub-section.

M.5 Summary of Design Water Level Conditions and Constructed Tidal Signals

Elevated design peak water level conditions (excluding wave runup) are summarised in Table M-2. Constructed synthetic water level time series are provided in Figure M-6, Figure M-5 and Figure M-4 for the 63%, 5% and 1% AEP (1, 20 and 100 year ARI) storm events respectively. The length of the provided timeseries corresponds directly with the length of the appropriate coastal storm event, as per Figure M-2.

Table M-2: Summary of Design Water Level Conditions

AEP %	**ARI** (years)	**Peak Tide Level** (m AHD)	**Peak Anomaly** (m)	**Wave Setup at Tide/Anomaly Peak** (m)	**Peak Water Level (including setup)** (m AHD)
63	1	0.50	0	0.80	1.30
5	50	0.82	0.55	1.09	2.46
1	100	0.82	0.61	1.15	2.57

Note that the site specific 5% and 1% AEP values compare well to the default values on OEH (2015) for sites south of Crowdy Head (2.35 m AHD and 2.55 m AHD respectively).

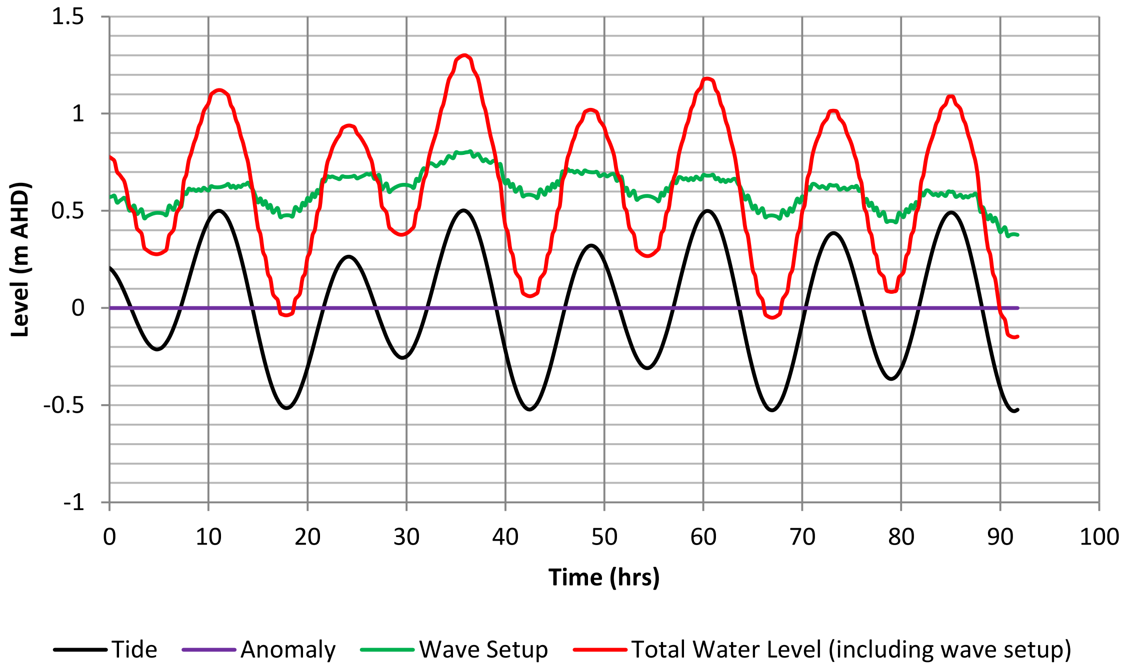

Figure M-4: Durras Lake tailwater conditions: 63% AEP

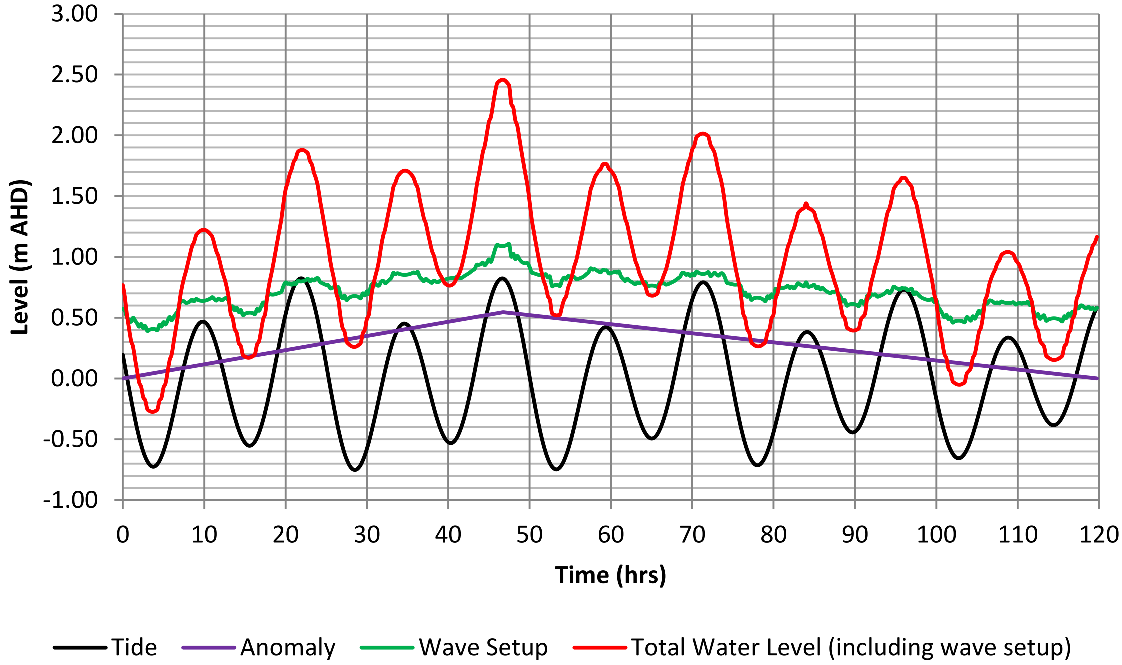

Figure M-5: Durras Lake tailwater conditions: 5% AEP

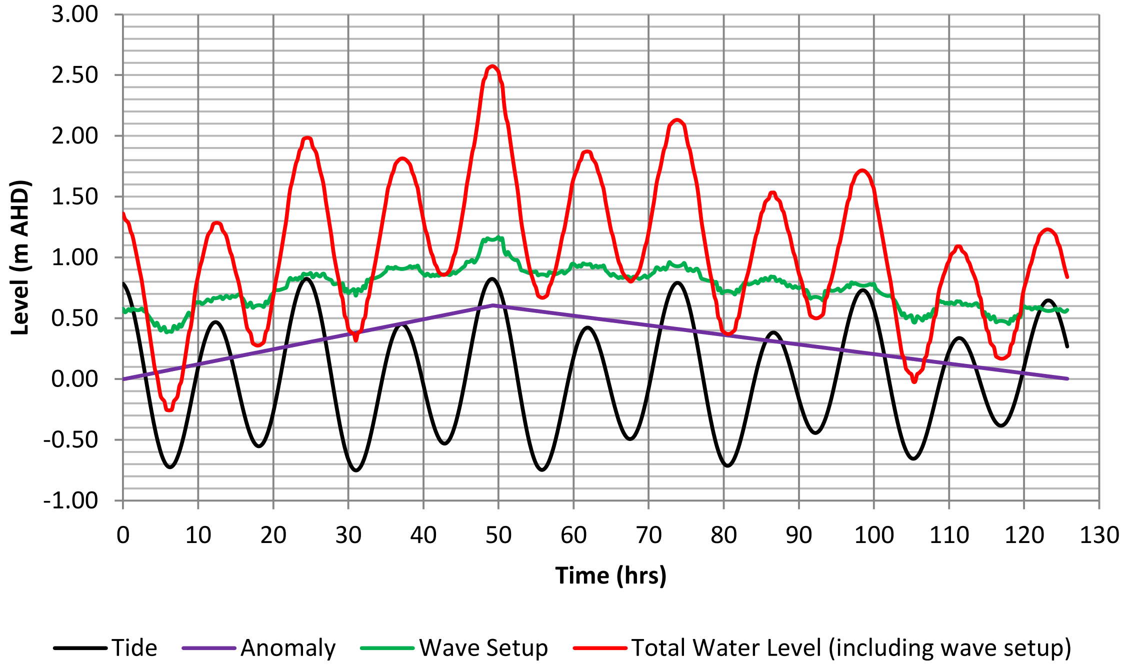

Figure M-6: Durras Lake tailwater conditions: 1% AEP