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North Wonboyn NSW
Foreshore Protection Study

WRL Technical Report 2013/29
May 2015

By M J Blacka, A Mariani and J T Carley

Water Research Laboratory
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1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Australia was commissioned by Bega Valley Shire Council (BVSC) to provide advice in regards to seawall protection options for an eroding foreshore at North Wonboyn, NSW. This report presents background information and details of the site inspection undertaken by WRL, as well as suggested foreshore management options and the advantages and disadvantages of each.

WRL have undertaken an inspection of the site (**Section 3**) and presented a brief assessment of the relevant estuary processes and historical changes in the study area (**Section 4**). Coastal protection options and potential impacts have been presented (**Section 5**), along with considerations for project design life (**Section 6**) and establishment of initial design criteria (**Section 7**). Finally, WRL developed a concept design and preliminary costing for the preferred option (**Section 8** and **Section 9**) and briefly presented less expensive alternative options (**Section 10)**.

2. Location and Site Overview

2.1 Location

Wonboyn Village, Wonboyn Lake and the Wonboyn River are situated on the Far South Coast of New South Wales, approximately 30 km south of Eden. The North Wonboyn settlement is located on the northern foreshore of the Wonboyn River entrance (Figure 2.1). Along with Wonboyn Village, North Wonboyn is the only notable development in the area.

2.2 Site Overview

The untrained Wonboyn River entrance is dominated by extensive sand shoals, with a narrow channel typically flanking the northern section of foreshore in the study area. From the limited data available, channel depths at mean tide vary from 1 to 2 metres. As such, the channel is used by recreational boaters travelling between the Wonboyn Lake and Wonboyn Village, and Disaster Bay Beach. The entrance conditions vary in relation to the incidence of floods and large wave events (WBM, 2002) associated with scouring and deposition. Anecdotal reports and analysis of aerial photography confirm that the entrance is effectively open most or all of the time. It is almost certain that the entrance has been located on the southern side of the headland at times in the geological past, although WRL has not observed this in any historical aerial photos. Future beach erosion or recession and/or a large river flood could cause the entrance to break through on the southern side of the headland in the future.

In recent years the foreshore at North Wonboyn has eroded/receded, creating a number of problems which include:

- Loss of recreational use of the beach;
- Loss of private property;
- Increased vulnerability to storm wave conditions.

Figure 2.2 shows the North Wonboyn foreshore during a recent storm event.

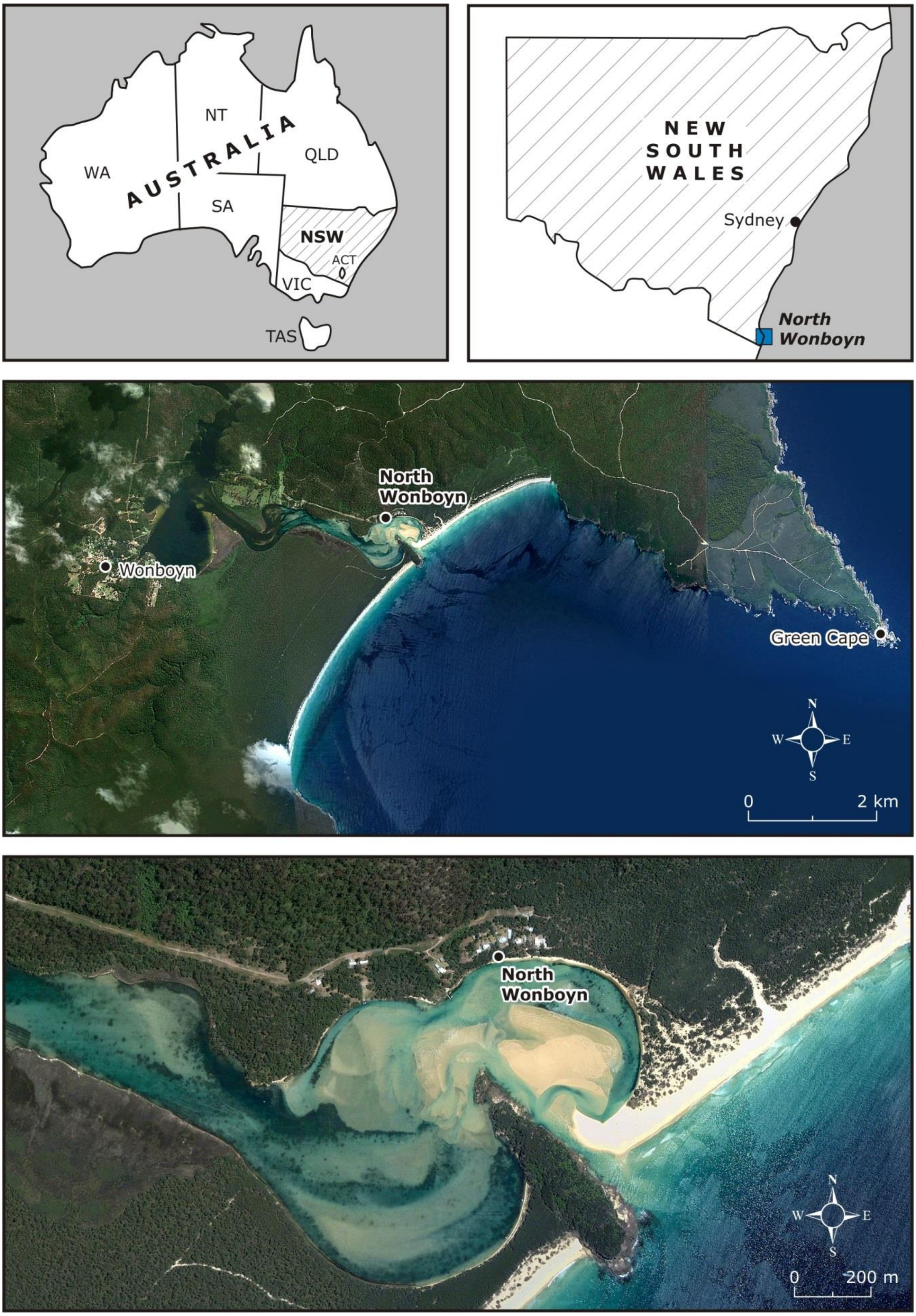

Figure 2.1: North Wonboyn Location

Figure 2.2: Images of Recent Storm at North Wonboyn
(Source: Robert Smith)

Figure 2.2 Cont'd: Images of Recent Storm at North Wonboyn (Source: Robert Smith)

3. Site Inspection and Description

WRL Principal Coastal Engineer Matt Blacka inspected the site on Wednesday 12/02/2014, with access provided by local resident Mr Robert Smith. Photographs of the site are provided in Figure 3.1 to Figure 3.10. Temporary dune protection at the site is currently being provided by a timber retaining wall structure, though the structure has become undermined along most of its length, with only geotextile retaining the dune sand. The retaining wall has also become outflanked at the western end with significant erosion of backfill sand from behind the wall. The crest of the dune and retaining wall forms the seaward edge of private property yards, and in most places is landscaped with mature vegetation (Figure 3.8 to Figure 3.10).

The foreshore along this stretch of the estuary is predominantly used for recreational purposes by the local residents of North Wonboyn, with several small private boats moored on the beach. Access to the beach for people is via timber stairs that form a part of the seawall (Figure 3.5) and for vehicles/boats is via timber slatted boat ramps (Figure 3.6).

The estuary foreshore fronting several of the North Wonboyn properties is currently impacted by beach recession with loss of subaerial beach, dune and established vegetation (as can be seen in photographs of the site). There is a narrow strip of intertidal beach that currently forms the northern edge of the primary tidal channel within the estuary.

Figure 3.1 Site Foreshore Looking East

Figure 3.2 Site Foreshore Looking West

Figure 3.3: General View of Foreshore (Looking West)

Figure 3.4: Existing Treated Pine Sleeper Seawall (Looking West)

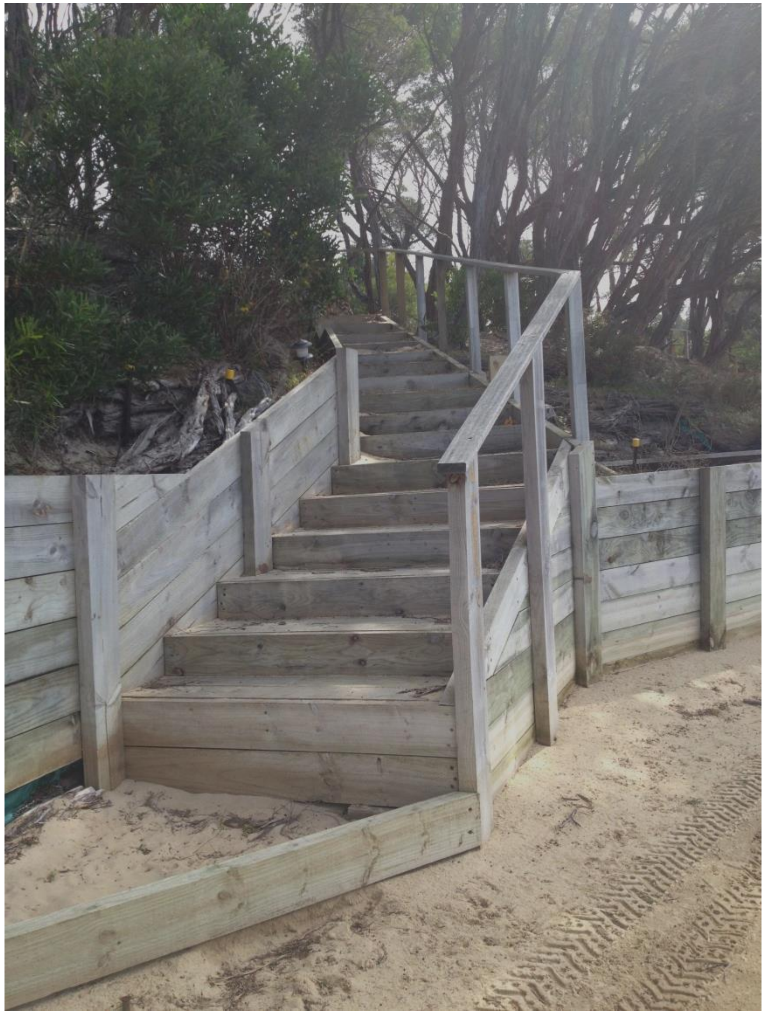

Figure 3.5: Stepped Access to Beach

Figure 3.6: Damaged Timber Slatted Boat Ramp

Figure 3.7: Eroded Foreshore at Western End of Seawall

Figure 3.8: Vegetation and Landscaping at Top of Seawall

Figure 3.9: Vegetation and Landscaping at Top of Seawall

Figure 3.10: Vegetation and Landscaping at Top of Seawall

4. Estuary Processes Considerations and Shoreline Analysis

4.1 Overview of Estuary Processes in Study Area

The lower reaches of the Wonboyn Estuary are dominated by large sand shoals, with a natural channel meandering through the shoals to the untrained entrance on Disaster Bay Beach. A collection of aerial photographs of the estuary were collated for this investigation, with the available images summarised in Table 4.1.

Table 4.1: Summary of Historical Aerial Photos

Name	Date	Source
Accessed online at http://maps.six.nsw.gov.au/	2013	NSW Land and Property Information
wonboyn001.tif	27/06/2011	Bega Valley Shire Council
wonboyn002.tif	27/03/2007	Bega Valley Shire Council
wonboyn003.tif	06/03/2005	Bega Valley Shire Council
wonboyn004.tif	01/04/2001	Bega Valley Shire Council
wonboyn005.tif	28/02/1989	Bega Valley Shire Council
wonboyn006.tif	40/06/1972	Bega Valley Shire Council
img-Y28135731-0001	06/1962	Bega Valley Shire Council

Note: the 1962 and 1972 images were not included in the analysis due to low resolution.

The aerial photographs have been analysed in two ways, firstly the photographs have been used to qualitatively consider how the characteristics of the estuary have changed over the past five (5) decades, in particular the position of the entrance, sand shoals and dominant tidal channel. Secondly, where possible the photographs were used to analyse how the position of the shoreline at the North Wonboyn site has changed through time.

Not all of the photographs clearly show the location of tidal channels between/through the sand shoals, which makes it somewhat difficult to ascertain definite changes in the estuary through time. While the North Wonboyn foreshore appears to have always been fronted by a deeper channel throughout the last 50 years, this channel has, up until the early 2000s, always been either a secondary tidal channel with an indirect connection to the entrance (1962, 1989, 2005, 2007), or has been a backwater within the estuary with no direct connection to the entrance (1972, 2001). Photographs from 2011 and 2013 show that the North Wonboyn channel appears to have now become the primary tidal channel of the lower estuary, carrying the majority of the tidal flows. While this is a discreet change in the lower estuary form, it may be hydraulically significant to the recession that has been experienced along the North Wonboyn foreshore in recent years.

As noted previously, it is almost certain that the entrance has been located on the southern side of the headland at times in the geological past, although WRL has not observed this in any historical aerial photos. Future beach erosion or recession and/or a large river flood could cause the entrance to break through on the southern side of the headland in the future.

No definite or obvious cause of the foreshore recession at North Wonboyn can be identified, but it is likely that the recession is a natural adjustment of the shoreline position, within a dynamic section of the estuary. These adjustments in the foreshore are more than likely linked to adjustments in the surrounding estuary shoals and channels, and are driven by a number of processes including:

- Time periods between large freshwater flooding events that scour the lower estuary sand shoals;
- Wave climate on the open coast which drives the infilling of sand within the lower estuary shoals;
- The extent to which the estuary entrance is open to the sea, which dictates the tidal exchange; and
- Ongoing evolution of the estuary due to a shift of the entrance from the southern side of the headland to the present northern side.

Based on observations at the site, the eroded dune profile now forms the subaerial extension of the estuary channel bank. That is to say that the active estuary channel bank profile now extends continuously from the deeper submerged base of the channel to the top of the eroded dune, and includes a flatter intertidal section of beach at the toe of the existing seawall. It is likely that the channel has attempted to meander further north over the past five (5) years, such that the once extensive subaerial beach at North Wonboyn has now completely receded.

Much of the area of the private residences at North Wonboyn is founded on relic marine sand. If the North Wonboyn area is considered in a historical geological context, it is likely that the private properties are located in an area that was once a part of the active and dynamic estuary sand shoals. While the foreshore recession at the site may be the worst in living memory, it is likely that the northern edge of the estuary has been located further north than its present position at some point in history, and that the recession at the site is a long-term natural fluctuation within the estuary.

4.2 Shoreline Mapping Through Analysis of Historical Aerial Images

An assessment of shoreline evolution at North Wonboyn was undertaken by analysing the available aerial and satellite images. The vegetation line was chosen as the morphological indicator of shoreline change. For this analysis the vegetation line was defined as the boundary between stable, dense vegetation and the sandy beach. Among other shoreline indicators (e.g. high water mark, wet/dry boundary, etc.), the vegetation line was the physical feature most distinctively identifiable from the available images and is less prone to short term (days or weeks) variations. While 3D photogrammetry provides superior data, it was not available for this study. The vegetation line was therefore considered to be the best available indicator of shoreline change at North Wonboyn. The aerial photographs from 1962 and 1972 were not of suitable quality to accurately identify the vegetation line, as such the analysis covered the period from 1989 to 2013 (24 years).

The aerial photographs used in the analysis are reproduced in Appendix A. The images were geo-referenced and rectified and a study foreshore section at North Wonboyn of approximately 200 m fronting six (6) dwellings and Crown Reserve was analysed. The vegetation line over the various years was identified, superimposed and compared. Figure 4.1 shows the 1989 and 2011 vegetation lines superimposed on the geo-referenced 1989 aerial photo.

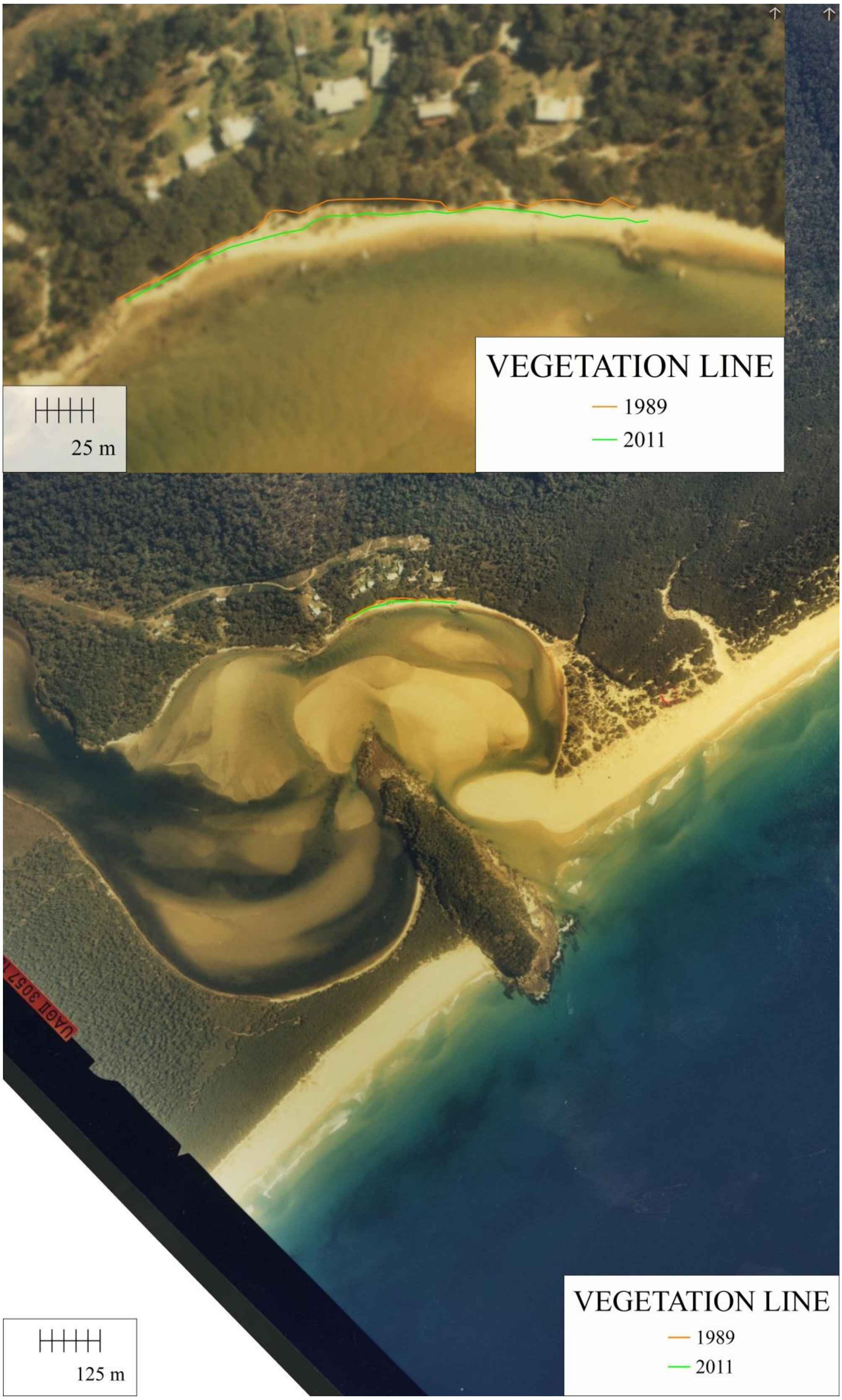

Figure 4.1: Aerial Photo 28/02/1989 and 1989 and 2011 Vegetation Line Mapping

4.3 Shorelines Changes from 1989 to 2013

Significant variation in the shoreline was detectable from this analysis, with the most eroded state occurring in 1989 followed by a period of general accretion evident from the subsequent aerial images until 2011. The present condition of the shoreline (2013) is the second most eroded state after 1989. Shoreline variations were more significant in the middle and eastern sections of the study area with variations of up to 10 m. Figure 4.2 shows a plot of vegetation line position for different years.

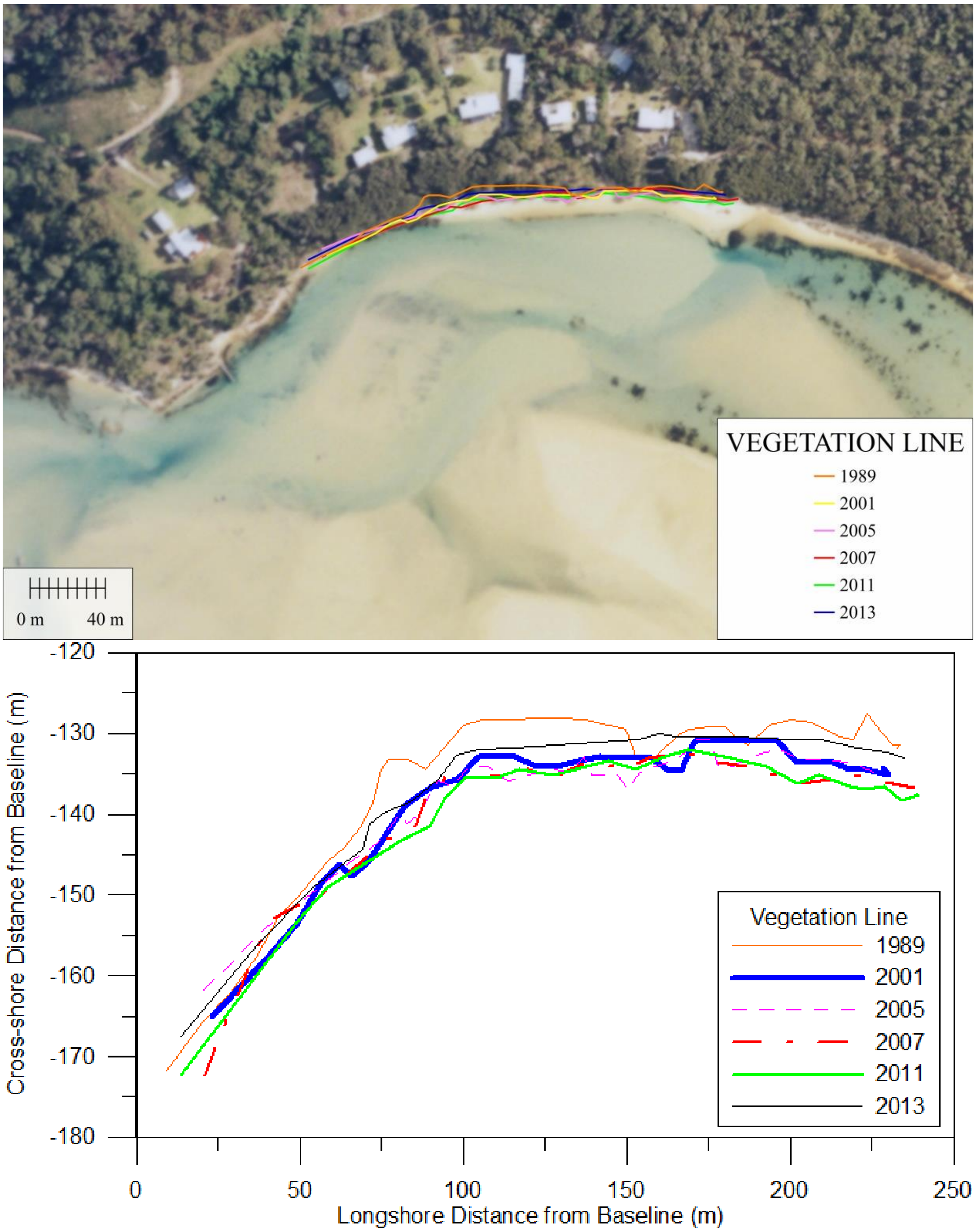

Figure 4.2: Aerial Photo 6/03/2005 and 1989 to 2013 Vegetation Lines (distorted scale)

The data does not show any clear long term trend which could be extrapolated, however, the following qualitative observations can be drawn from the historical image analysis:

- In 1989, the foreshore at North Wonboyn appeared to be in a more eroded state than the present, based on the position of the vegetation line;
- The large rainfall event reported by WBM (2002) which occurred early 1989 could explain the eroded state observed from the 1989 aerial image;
- From 1989 until 2011, a trend of general accretion could be observed;
- From 2011 to 2013, a trend of general recession could be observed;
- The most impacted areas are the middle and eastern portions;
- A heavily shoaled entrance generally coincided with a more accreted state of the foreshore; and
- A well-defined and deep channel adjacent to the study area generally coincided with more eroded state of the foreshore.

4.4 Transects Analysis

Three transects were analysed in terms of changes of vegetation line position over the years. The location of the transects is shown in Figure 4.3. Plots of changes in vegetation line position relative to the 1989 and 2001 positions are shown in Figure 4.4 and Figure 4.5 respectively.

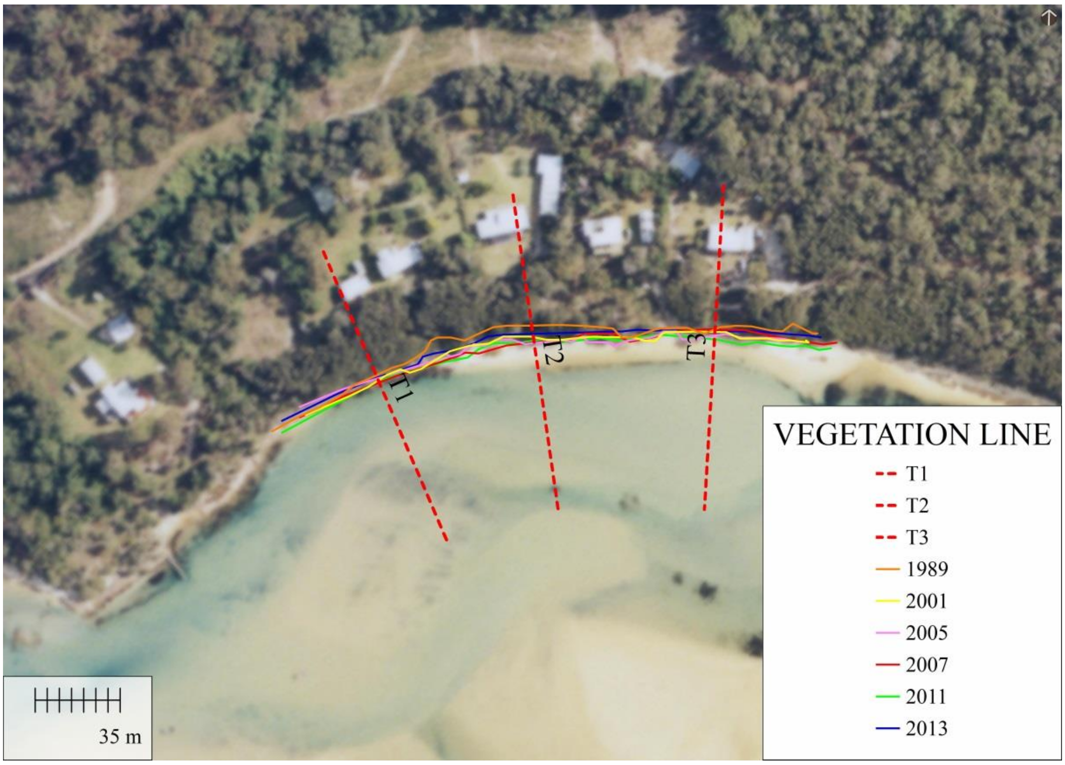

Figure 4.3: Transect Location for Shoreline Changes Analysis

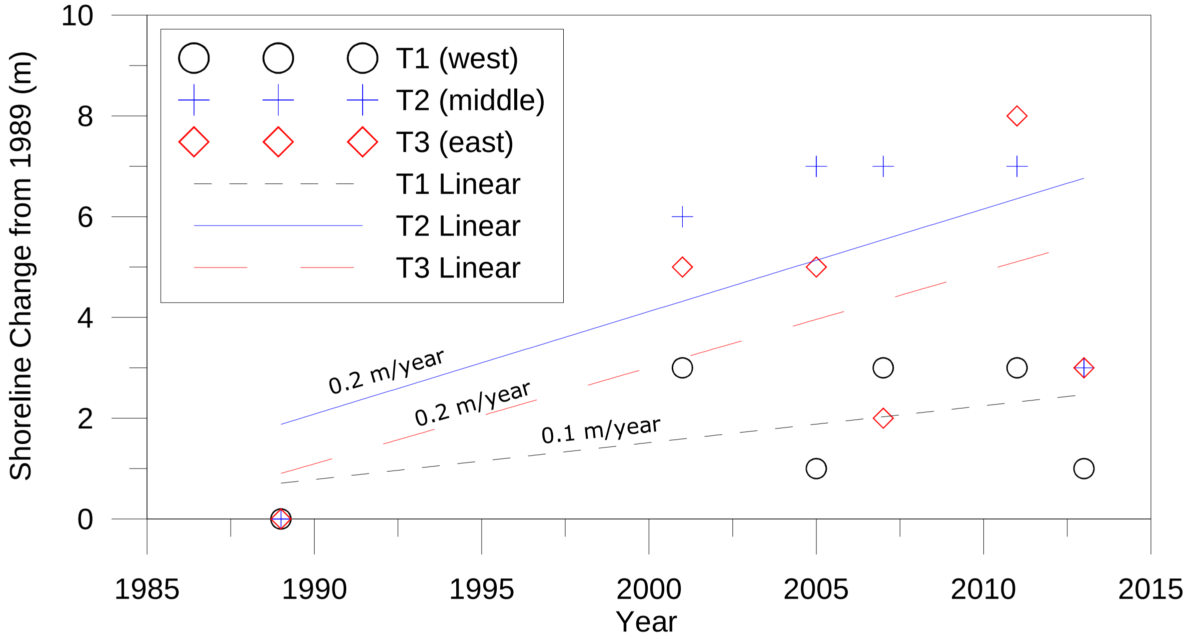

Figure 4.4: Shoreline Changes from 1989 Position

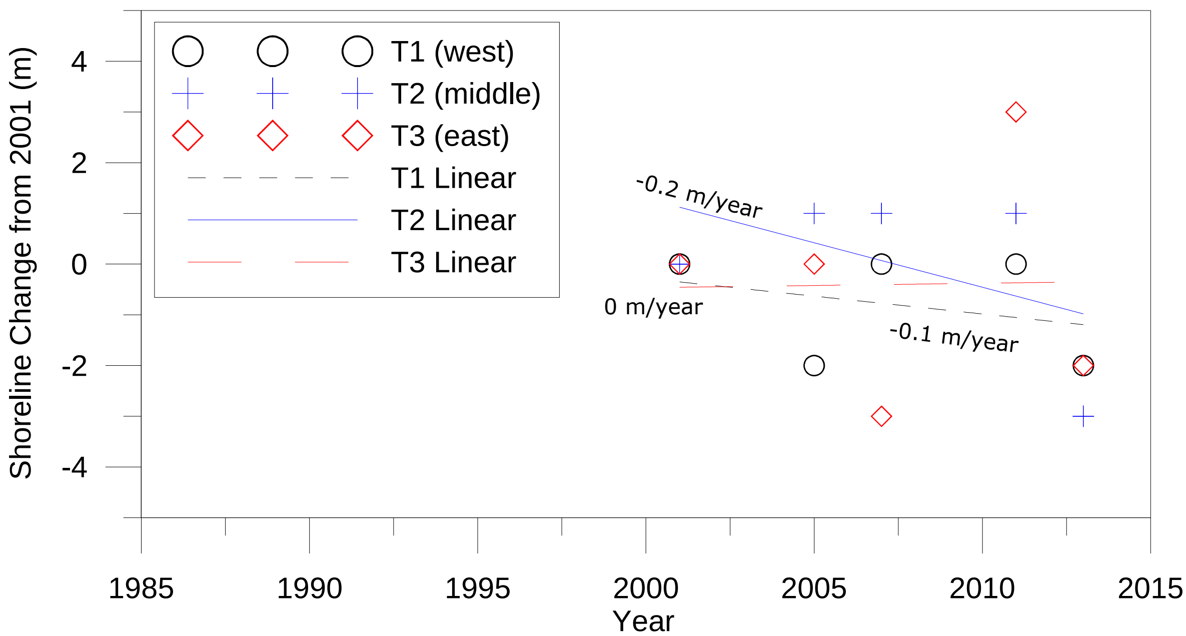

Figure 4.5: Shoreline Changes from 2001 Position

The analysis of the transects showed there were no clear long term trends. Nevertheless, fitting an average linear trend showed that on average the shoreline accreted at the three transect locations from 1989 to 2013, at average rates of 0.1 to 0.2 m/year. However from 2001 to 2013, the shoreline receded at locations T1 and T2, i.e. the western and middle section of the study area, with recession rates of -0.1 to -0.2 m/year. The eastern section of the study area (transect T3) did not show a trend of either accretion or recession. However, the most significant factor in the latest (2013) shoreline position is the erosional change since the 2011 aerial photo.

4.5 Summary of Coastline Change

Clear long term linear trends of recession could not be inferred from the analysis of the historical aerial images. However, shoreline variability of up to 10 m was evident from the mapping of the vegetation line. This variability appears to be correlated with the conditions of the river entrance and the channel flanking the study area which in turn are dependent on a range of other estuarine processes.

5. Foreshore Protection Options

5.1 Overview of Foreshore Protection Considerations

Options for protection of the foreshore at North Wonboyn are limited due to a number of reasons including:

- Only a small number of private properties will benefit, and there is no immediate risk of damage to assets of significant value on these properties;
- The location is remote and haulage of construction materials to the site would be costly due to the travel times;
- The dune is well vegetated, which would restrict access of construction plant along the top of a seawall or revetment structure;
- The available working area at the toe of the existing seawall is narrow and predominantly intertidal, which may restrict construction times to periods of lower tide level or working in a wet zone; and
- NSW coastal protection policies.

Due to the transient and dynamic nature of the sand shoals within the lower Wonboyn Estuary, at this stage it is recommended that any protection strategy for the site be targeted at providing temporary (short to medium term) coastal protection only. Unlike recession on some open coast beaches, the recession experienced at North Wonboyn may reduce or even reverse in years to come, depending on the natural evolution of the lower estuary and the environmental events that are experienced.

Based on discussions with Council and the local resident representative, it is understood that residents are considering options to protect the foreshore under the NSW Government Temporary Coastal Protection Policy. This policy effectively restricts protection options to either softer alternatives such as nourishment of the beach or harder protection through the use of large sand filled geotextile containers. Both of these protection options require a source of marine sand, which is likely to be a limitation and cost for coastal protection.

5.2 Potential Seawall Impacts

Any structure built within the active area of the coastal zone has the potential to impact the surrounding coastline. This section of the report discusses potential impacts of coastal protection on the surrounding stretch of foreshore and estuary shoreline.

Seawall impacts can be broadly classified into three categories:

(1) Physical impacts
(2) Ecological impacts
(3) Socio-economic impacts.

5.2.1 Physical Impacts

The relevant potential physical impacts related to the proposed seawall in North Wonboyn are:

- Altered erosion and accretion seaward of the wall;
- Altered erosion and accretion either side (alongshore) from the wall (seawall end effects);

- Altered longer term recession and progradation alongshore from the wall; and
- Changes to wave runup.

While a substantial amount of research has been undertaken investigating the structure-beach interaction and documenting cases of beach response (summarised in Kraus, 1988; Kraus and McDougal, 1996), due to the complexity of the processes, robust and widely-accepted methods for predicting the magnitude and extent of beach response to seawalls are not available. Figure 5.1 shows an example of seawall end effects at Kingscliff, NSW. Differences in erosion "end effects" between different seawall types are not currently known.

Figure 5.1: Kingcliff NSW Example of Excess Seawall End Erosion Depth, r and Length, s

5.2.2 Ecological Impacts

Ecological impacts may include loss and alteration of beach habitat in the vicinity of the seawall, potentially adversely affecting a range coastal flora and fauna. Subject to its porosity, a seawall may also provide new ecological habitat (however, this is less relevant to sandbag seawalls).

5.2.3 Socio-Economic Impacts

The construction of seawalls may cause a range of socio-economic impacts. Positive impacts may include:

- Provision of additional, improved or more secure public recreational space;
- Improved security to landowners; and
- Changes to property values.

Negative impacts may include:

- Loss of recreational beach amenity;

- Erosion and/or recession at adjacent beach due to alongshore impacts of structure;
- Increased wave runup and overtopping due to the introduction of hard wall; and
- Injuries due to collisions/trips/falls on seawall by beach users.

5.2.4 Summary of Seawall Impacts

In the context of the North Wonboyn foreshore, it is likely that impacts associated with a temporary sandbag seawall structure would be similar or less than the impacts that would be experienced with the existing temporary vertical timber retaining wall at the site. A well-engineered and constructed sandbag seawall would present less of a hazard to local residents using the beach when compared with the existing structure, which will continue to increase in safety risk as its condition deteriorates with ongoing foreshore erosion.

Due to the relatively restricted access to the site for construction plant, it is likely that some damage to the existing dune vegetation would be experienced during construction of a sandbag seawall at North Wonboyn. Some consideration would be required for rehabilitation of the site following construction, so as to minimise the long term impacts of the seawall construction process.

6. Design Considerations for Coastal Protection

6.1 Design Life and Balance between Risk and Capital Cost

The design life of any coastal structure needs to be considered as a component of the overall risk within a project. Structures which are designed for a short/frequent Average Recurrence Interval (ARI) event, or which are retained in excess of their design life will incur substantial costs, which may be in the form of maintenance, repairs, consequential damage or political consequences. Structures which are designed for high/rare ARI events will have low maintenance costs and/or costs due to the risk of failure, but will involve high upfront capital costs. This is illustrated in Figure 6.1. While there can be some technical/economic basis for risk and design life, the final decision involves a degree of subjectivity.

Explicit formal guidance is not readily available for selection of an appropriate design event for maritime structures equivalent to the proposed interim structures. Conventional coastal engineering practice in Australia is to allocate a design ARI which may range from the design life of the project (e.g. a 1 year design life structure would use a minimum 1 year ARI design event) up to that suggested in Australian Standard AS 4997-2005.

By considering initial damage and failure, it is possible to design a structure for which initial damage (serviceability) occurs at an ARI approximating the design life. Failure (limit state) would occur at a higher ARI (than the design life) which complies with the standards cited below.

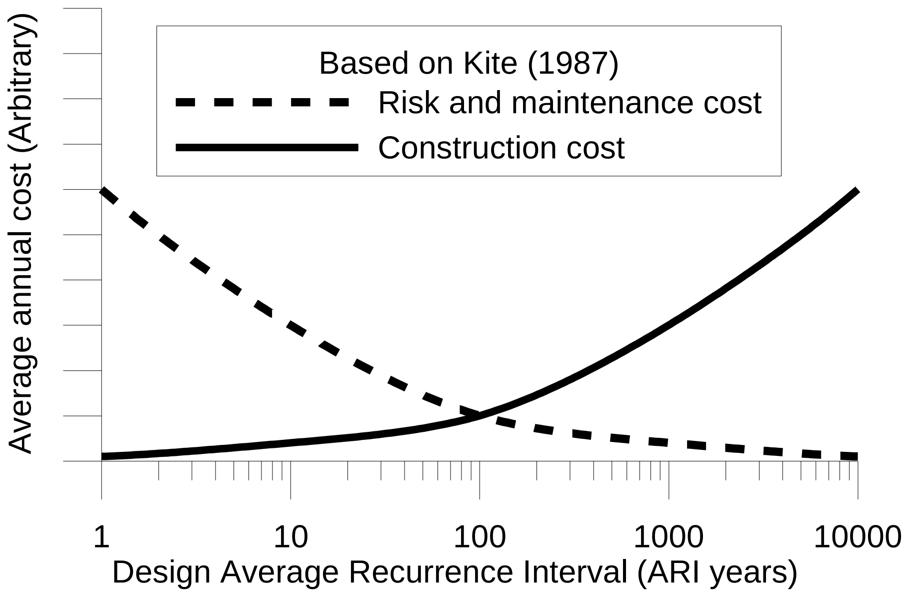

Figure 6.1: Balance between Risk, Maintenance and Capital Cost

6.2 Encounter Probability

Encounter probability is defined as the probability that an event will be equalled or exceeded over the design life of a project. Encounter probabilities and design life are related in Equation 4.1:

$$P = 1 - e^{(-N/ARI)} \qquad (4.1)$$

Where P = Encounter probability (0 to 1 or 0% to 100%)
N = Design working life (years)
ARI = Average recurrence interval (years)

The probability that a structure will fail over its design life can be calculated by applying an appropriate ARI for failure in Equation 4.1. Conversely, the appropriate ARI for failure can be derived by applying an acceptable encounter probability.

6.3 Australian Standard AS 4997-2005

Australian Standard (AS) 4997-2005 *Guidelines for the Design of Maritime Structures* recommends design wave heights based on the function and design life of the structure as reproduced in Table 6.1. Note that while this standard covers rigid maritime structures (e.g. wharves and concrete seawalls), it specifically excludes the design of flexible "coastal engineering structures such as rock armoured walls, groynes, etc.". However, in the absence of any other relevant Australian Standard, it is commonly considered in the assessment of probability in contemporary Australian coastal engineering practice.

Table 6.1: Annual Probability of Exceedance of Design Wave Events (Source AS 4997-2005)

Function Category	Structure Description	Encounter Probability (a, b)	Design Working Life (Years)			
			5 or less (temporary works)	**25** (small craft facilities)	**50** (normal maritime structures)	**100 or more** (special structures/ residential developments)
1	Structures presenting a low degree of hazard to life or property	~20%(c)	1/20	1/50	1/200	1/500
2	Normal structures	10%	1/50	1/200	1/500	1/1000
3	High property value or high risk to people	5%	1/100	1/500	1/1000	1/2000

(a) Apart from the column "Encounter Probability" (calculated by WRL), the table is a direct quote from AS 4997-2005.

(b) Inferred by WRL based on Equation 4.1.

(c) The encounter probability for temporary works, normal maritime structures and special structures in Function Category 1 is ~20%. However, the encounter probability for small craft facilities in Function Category 1 is 39%.

While there is a degree of subjective interpretation regarding function category and structure type within AS 4997-2005, in WRL's opinion proposed foreshore protection works at the North Wonboyn site are most likely "Function Category 1 – Structures presenting a low degree of hazard to life or property". It can be seen from Table 6.1 that AS 4997-2005 suggests a working life of 5 years or less for temporary works.

6.4 International Standard ISO 21650:2007

International Standard ISO 21650:2007 "Actions from Waves and Currents on Coastal Structures" contains some guidance on design life and probability. It provides guidance for a range of "safety classes" as shown in Table 6.2.

ISO 21650:2007 provides the following commentary: "*Temporary and small coastal structures would belong to the very low safety class. Larger coastal structures such as … exposed seawalls protecting infrastructure would belong to the low safety class. Breakwaters protecting an LNG terminal or a power station would belong to the normal safety class whereas a sea dyke protecting populated low land would belong to the high safety class*."

Based on the above guidance, WRL considers the proposed interim works at the North Wonboyn site as being either Very Low or Low safety class according to ISO 21650:2007.

ISO 21650:2007 suggests the following design working life of coastal structures:

- Temporary coastal structure: 1 to 5 years; and
- Permanent coastal structure: 50 to 100 years.

ISO 21650:2007 quotes two tentative methods for specifying a probability of failure, namely the Spanish ROM 0.0 method and that of Burcharth (1999). Both of these methods provide a probability for "serviceability" (performance under commonly encountered conditions) and "limit state" (ultimate failure) which are shown in Table 6.2. ISO 21650:2007 only provides the extreme range for the probability of failure in Table 6.2, however, intermediate values for the Burcharth method are presented in Ram *et al*. (2003).

Table 6.2: Example of Safety Classes for Coastal Structures (Source: ISO 21650:2007)

Safety Class	**Consequence of Failure**	**Probability of Failure (Encounter Probability)**	
		ROM 0.0	**Burcharth (a)**
Very low	No risk of human injury. Small environmental and economic consequences	Serviceability 20% Limit state 20%	Serviceability 40% Limit state 20%
Low	No risk of human injury. Some environmental and economic consequences	Not provided	Serviceability 20% Limit state 10%
Normal	Risk of human injury and/or significant environmental pollution or high economic or political consequences	Not provided	Serviceability 10% Limit state 5%
High	Risk of human injury and/or significant environmental pollution or very high economic or political consequences	Serviceability 7% Limit state 0.01%	Serviceability 5% Limit state 1%

(a) as quoted in Ram *et al.* (2003)

6.5 Adopted Design Life and ARI

Based on consideration of AS 4997-2005 and ISO 21650:2007, and WRL's previous experience with existing sandbag works in NSW, a 20 year design life has been adopted for the foreshore stabilisation works at North Wonboyn.

A range of design ARIs which could be considered for a 20 year project life and encounter probability as recommended in AS4997-2005 and ISO 21650:2007 is presented in Table 6.3. These values were developed by using Equation 4.1.

Table 6.3: Design ARI Events for Consideration

Design Life (years)	Function Category/Safety Class (from AS 4997- 2005 and ISO 21650:2007)	Encounter Probability (recommended)	Design ARI Event (years)
20	1/Very Low	20%	90
20	1/Very Low	40%	39

The values adopted for the design of the foreshore protection works at North Wonboyn are:

- **20 year design life; and**
- **100 year ARI event**;

This has a 20% encounter probability and complies with ISO 21650:2007 "Very Low" safety class and AS 4997-2005 "Low Degree of Hazard" structure.

7. Design Conditions for Site

7.1 General

Design parameters for the proposed foreshore protection include ocean wave and water level conditions and the expected scour level at the toe of the structure. The toe scour level determines the required penetration of the structure to prevent undermining. The design water level and bathymetry of the estuary shoals influence the maximum depth limited breaking wave height that can physically impact the structure. In turn, the design wave and water level conditions at the structure affect the hydraulic performance (wave runup and overtopping) and stability of the structure which have a direct effect on the capital and maintenance costs. The geotechnical conditions at the site were not included in this assessment. The site appears to be predominantly marine sand. Potential acid sulphate soils have been noted further up the estuary (WBM, 2002). They are less likely in the vicinity of the proposed works, however, excavation should cease if they are encountered during construction.

7.2 Coincidence of Extreme Waves and Water Levels

Extreme conditions used for designing coastal structures arise from the combination of large waves, high water levels and eroded sand levels. Detailed studies on the joint coincidence of these factors are not available for the study site. Shand *et al* (2012) examined the joint probability of waves and tidal anomalies (but not eroded sand levels) for Sydney, with an example shown in Figure 7.1. It can be seen that for 100 year ARI conditions, the offshore significant wave height (for Sydney) varies by less than 1 m for 100 year ARI conditions, whether the tidal anomaly is 0.0 m or 0.4 m.

For the NSW South Coast, intense low pressure systems such as east coast lows or large extratropical low pressure systems cause the largest waves and most elevated oceanic water levels. Sand levels also erode in response to such storms. While the coincidence (phasing) of worst cases of these three variables may not occur simultaneously, there are insufficient studies to fully consider different phasing of each variable. Further complicating the situation is the possible influence and phasing of increased estuarine water levels from catchment runoff.

Therefore, as a conservative estimate, it has been assumed that for the ARI considered, the same ARI be applied to each component. That is, it has been assumed that the 100 year ARI (1 hour duration) wave height and water level coincide, together with the 100 year ARI beach erosion level. This is acknowledged to be conservative, however, well accepted (less conservative) alternative methodologies are not available.

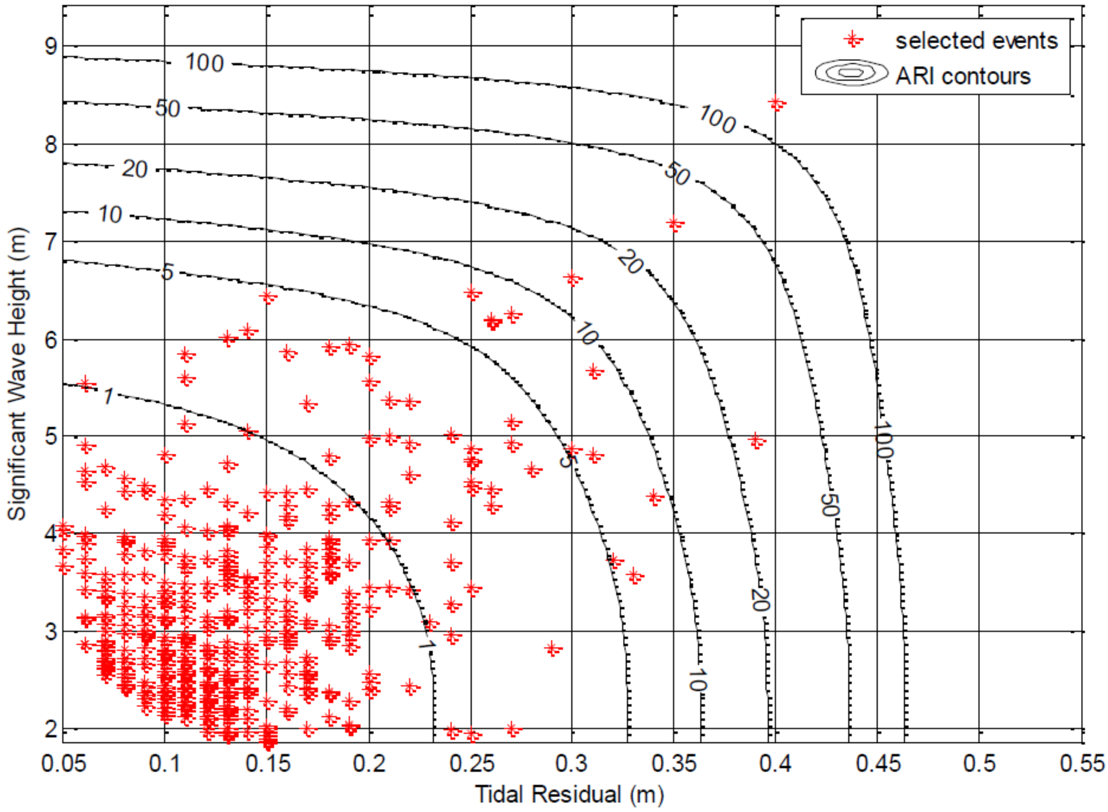

Figure 7.1: Joint Probability of Waves and Tidal Residuals for Sydney (Source: Shand *et al.*, 2012)

7.3 Adopted Offshore Design Wave Conditions

7.3.1 Wave Height

Waves reaching the foreshore at North Wonboyn may be modified by the processes of refraction, wave-wave interaction, dissipation by bed friction and wave breaking.

WRL, in conjunction with New South Wales Office of Environment and Heritage (OEH, formerly DECCW) have completed an assessment of coastal storms and extreme wave conditions for NSW which involved the identification of all measured coastal storms during the period 1971 – 2009 and derivation of directional design storm events for annual recurrence intervals of 1 to 100 years (Shand *et al.* 2010a). The results from the study for the Eden wave buoy and two wave buoys further north at Batemans Bay and Port Kembla are shown in Table 7.1.

Table 7.1: Extreme Offshore Wave Conditions (All Directions)
(Source Shand *et al.,* 2010a)

Average Recurrence Interval ARI (year)	**One Hour Exceedance H_S** (m)		
	Eden	**Batemans Bay**	**Port Kembla**
1	5.4	4.9	5.4
10	7.0	6.3	7.1
50	8.1	7.3	8.3
100	8.5	7.7	8.8

The capture rates for the three wave buoys are 83.5% (Eden), 89.7% (Batemans Bay) and 85.1% (Port Kembla). WRL has adopted the offshore significant wave heights from the Eden wave buoy for estimating the design waves for the North Wonboyn site. Note that this assumption does not have a substantial outcome on the design wave conditions at the structure, due to the depth limited nature of waves as they cross the lower estuary sand shoals.

7.3.2 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 subject site where this analysis was undertaken was Eden, with results presented in Table 7.2. WRL has adopted the peak spectral wave periods from the Eden wave buoy for this study.

Table 7.2: Associated Wave Period for Extreme Wave Events at Eden (Source: Shand *et al.,* 2011)

Average Recurrence Interval ARI (Years)	Peak T_P (s)
1	11.6
10	12.5
50	13.2
100	13.4

7.3.3 Wave Direction

The closest directional wave buoy to the study site (with long records) is Batemans Bay. It is noted that all NSW wave buoys are now directional, but do not yet have sufficient data for detailed analysis. In the aforementioned study by WRL (Shand *et al.,* 2010a), WRL also examined the influence of wave direction on extreme storm wave height along the NSW coast. Results showed that for wave events arriving from north of 90°, the extreme values were approximately 75% of the *'all direction'* values, wave events from the east to south-east were approximately 5% lower than the 'all direction' values and waves arriving from south to south-east were typically 100% of the 'all direction' values. Considering the direction of the Disaster Bay embayment, WRL adopted the south-east direction as the design direction for ocean waves approaching the Wonboyn River entrance.

7.3.4 Wave Transformation

Waves travelling from offshore to the subject site are influenced by the processes of refraction, shoaling, friction and breaking. Prior to the breakpoint, wave transformation can be represented by the equation:

$$H_{s\ nearshore} = K\ H_{os} \quad (1)$$

where $H_{s\ nearshore}$ is the nearshore significant wave height (prior to breaking)
K is a combined coefficient of refraction, diffraction friction and shoaling
H_{os} is deepwater significant wave height

In the absence of a comprehensive numerical wave modelling study for the area, WRL adopted a K value of 1. This is considered realistic for large waves from the south-east. Offshore wave heights are used as deep water input into the Dally *et al.* (1984) surf zone model to estimate wave setup and wave height at the North Wonboyn foreshore.

7.4 Design Water Levels

7.4.1 Storm Tide (Astronomical Tide + Anomaly)

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". Additional anomalies occur due to "trapped" long waves propagating along the coast. Water levels within the surf zone are also subject to wave setup and wave runup.

Design storm surge levels (astronomical tide + anomaly) are recommended in the Coastal Risk Management Guide (DECCW, 2010) based on data from the Fort Denison tide gauge in Sydney and reproduced in Table 7.3. This is based on approximately 100 years of data at the Fort Denison tide gauge which is not subject to wave setup or river flow effects. However, these levels are primarily applicable in the Newcastle - Sydney – Wollongong area and analysis of local tidal records on the NSW South Coast is recommended.

Table 7.3: Design Water Levels (Tide + Storm Surge) – Newcastle, Sydney, Wollongong (Source DECCW, 2010)

Average Recurrence Interval ARI (Years)	Water Level Excl. Wave Setup and Runup (m AHD)
1	1.24
10	1.35
50	1.41
100	1.44

The elevated water levels in Table 7.3 can be supplemented with additional analyses for other tide gauges in NSW South Coast undertaken by MHL (2010). However, it should be noted that these are generally based only on approximately 20 years of data and many of the southern NSW tide gauges are subject to river flow effects. The nearest tide gauge sites with negligible freshwater and bathymetric effects are Eden, Batemans Bay (Offshore) and Ulladulla. However, both the Eden and Ulladulla sites have insufficient data to allow extreme water level analysis. The elevated water levels for Batemans Bay Offshore and Eden (from central estimates in Appendix B of MHL, 2010) are reproduced in Table 7.4.

WBM (2002) noted that (from a limited data collection program and limited numerical modelling) tides within the Wonboyn estuary were reduced in amplitude compared with the open coast at Eden. They noted that tides within the estuary were closer to the open ocean range during spring tides and when the entrance was more open. Due to the North Wonboyn site being close to the entrance, the need to design for high spring tide conditions and the absence of long term measured data in the vicinity of the site, the open coast water levels for Batemans Bay were adopted for design.

Table 7.4: Extreme Water Levels for Northern NSW Tide Gauges (based on MHL, 2010)

Location	1 year ARI (m AHD)	10 year ARI (m AHD)	20 year ARI (m AHD)	50 year ARI (m AHD)	100 year ARI (m AHD)
Batemans Bay Offshore	1.19	1.28	1.30	1.32	1.33
Eden	1.15	insuf	insuf	insuf	insuf
Adopted for this study	1.15	1.28	1.30	1.32	1.33

7.4.2 Wave Setup

Wave setup may be reduced at river entrances and tidal inlets. Field measurements by Nielsen *et al.* (1989) and Hanslow *et al.* (1996) show that in a trained river (like Bermagui and Narooma) waves would generate no measurable wave setup (at least up to 4 m H_s). The Wonboyn River entrance is untrained and, when fully opened, the entrance can be up to 200 m wide. Additionally, the study section is located in proximity to the entrance.

Considering that the 100 year ARI design storm event would be associated with larger waves (H_s of 8.5 m), WRL adopted the precautionary assumption that wave setup would fully propagate to the study section, and estimated wave setup for the 100 year ARI storm event using the Dally *et al.* (1984) two-dimensional surf zone model.

Wave setup and runup are intrinsically dependent on the determination of the nearshore wave conditions.

To determine the wave setup at the study area, the effective offshore significant wave height H_s was adjusted to the root mean square wave height H_{RMS} according to CIRIA (2007) in Equation (1).

$$H_{RMS} = 0.706 \times H_S \qquad (2)$$

This wave height was applied as a boundary condition to the Dally *et al.* (1984) two-dimensional surf zone model. The bathymetric profile for the model was obtained from bathymetric data provided by the OEH and Council (Section 7.5). The corresponding 100 year ARI peak spectral wave period and storm tide water level were also applied.

The corresponding 100 year ARI wave setup and setup water surface level at the foreshore study area was 0.9 m and 2.2 m AHD respectively.

7.4.3 Sea Level Rise

Sea level rise is not expected to have significant impacts on the proposed works due to the short term design life. Therefore, no allowance for sea level rise was included in the structure design.

7.5 Reference Profile

The reference profile used is shown in Figure 7.2. The profile was reconstructed in a piecemeal fashion using the datasets shown in Table 7.5.

Table 7.5: Summary of Bathymetric and Topographic Data

Dataset	Source
Terrestrial LiDAR data (DEM dated 2012-12-04)	BVSC
Wonboyn River Hydrographic Survey (Oct. 1997)	NSW OEH
Offshore Bathymetry	Geoscience Australia

A Dean (1977) equilibrium profile for a median sand size of 0.25 mm (Surf Life Saving Australia database) was used to join the offshore bathymetry to the estuary survey data (a distance of approximately 100 m required infilling). WBM (2002) assumed scoured entrance conditions with water depths at mean tide of approximately 1 to 1.5 metres. Accordingly, WRL assumed scoured entrance conditions with the representative profile chosen along the scoured section of the river entrance and corresponding water depths at mean tide of approximately 1 m.

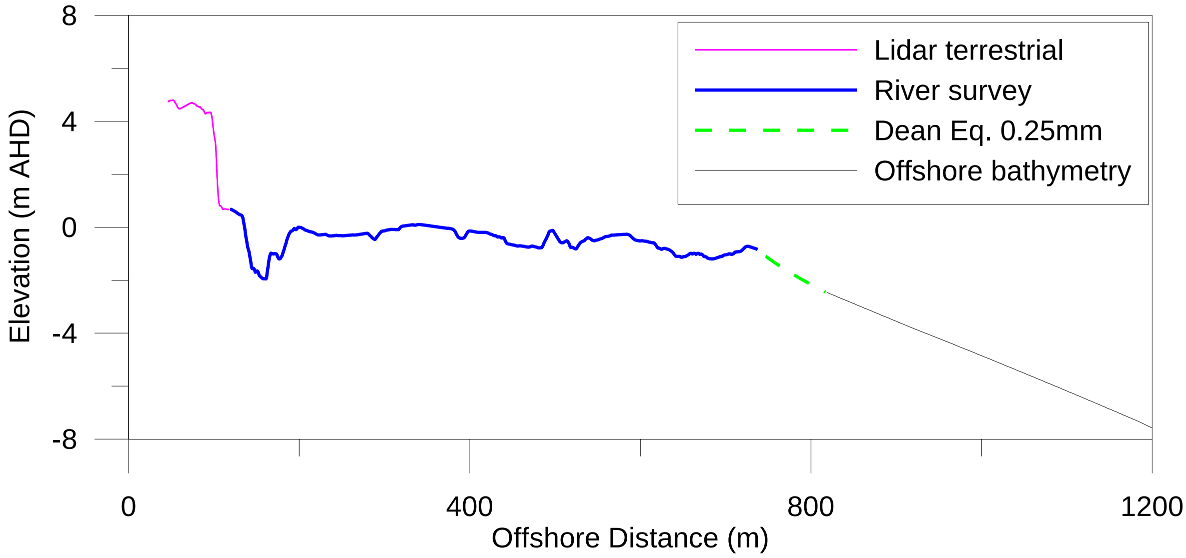

Figure 7.2: Reference Profile

7.6 Design Scour Levels

A range of options were canvassed regarding determination of the design scour level. These are indicated below:

- Engineering "rules of thumb";
- Photogrammetry;
- Erosion modelling;
- Published data on profile change such as Gordon (1987) and Chapman and Smith (1983).
- Other allowances using a Dean (1977) equilibrium profile.

While these methods were primarily developed for open coast beaches, their application to the relatively protected study area provides a useful range of scour level estimates. Additionally, it is expected that scour levels at the proposed structure will be primarily dominated by river and estuarine processes.

7.6.1 Rules of Thumb

In NSW, the scour level of approximately -1.0 m AHD is commonly adopted as an engineering rule of thumb for rigid coastal structures located at the back of the active beach area (i.e. not applicable to North Wonboyn), with -2 m AHD frequently adopted for vertical coastal structures due to increased wave reflections. This is based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms in front of existing permeable and non-permeable seawalls along the NSW coast (Nielsen *et al*. 1992; Foster *et al*. 1975). While not directly applicable to North Wonboyn, for seawalls constructed on the NSW Maritime Authority's land a minimum allowance of 0.6 m for scour from the seaward face of the seawall is required unless the seawall is founded on rock (NSW Maritime, 2005).

7.6.2 Photogrammetry

Photogrammetry, depending on the water level at the time of the aerial photograph, generally does not extend out to levels below approximately 0 m AHD, so cannot be used to determine extreme historical scour levels.

7.6.3 SBEACH Modelling

In addition to wave setup modelling, WRL undertook two-dimensional modelling of beach erosion using SBEACH (version 4.03). The SBEACH model is a two-dimensional numerical cross-shore sediment transport and profile change model developed by the United States Army Corps of Engineers, Coastal Engineering Research Center. 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.

The process for confirming the design scour level for each structure using SBEACH is outlined in the following discourse.

Firstly, the design erosion volume (storm demand/storm bite) for the study section without a foreshore protection structure in place was estimated by modelling a time series of three consecutive, synthetic storm events (Shand et al., 2011). A design erosion volume of 60 m^3/m above AHD was established for the 100 year ARI storm event at the study section. This value is in accordance within the range used in coastal hazard studies of sheltered entrance and estuary beaches in the South NSW Coast (Coghlan et al. 2012).

Secondly, a structure was introduced such that erosion of the dune is prevented. Finally, the time series of storm events (which resulted in the adopted storm demand without a structure in place) was modelled in SBEACH with a structure in place to estimate the scour level at the toe of each structure design.

The SBEACH modelling found scour levels fronting the structures around -0.2 m AHD. This value is considered un-conservative because, while taking into account scouring from wave processes, it excludes potential scour from river processes.

7.6.4 Published Profile Change

While the following methods are generally applied to open coast environments (therefore not directly applicable to the study area), they provide useful range of plausible scour levels for more protected low energy environments.

Gordon (1987) published the expected range of vertical change on the NSW coast as a function of average sand levels. Chapman and Smith (1983) introduced the concept of "swept prism" based on approximately 9 years of ongoing measurements on the Gold Coast. Results from these methods are shown in Table 7.6. For the study section in North Wonboyn, assuming an average sand level against the structure of +1 m AHD, the minimum expected sand level at the structure from interpolating these methods is -1.75 m AHD.

Table 7.6: Vertical Change of Reference Elevations from Field Measurements

	Vertical Change from Reference			
Average sand level (m AHD)	**Gordon (1987) High Demand** (m)	**Gordon (1987) Low Demand** (m)	**Chapman and Smith (1983)** (m)	**Minimum estimated sand level from these references** (m AHD)
+4	± 2.75	± 2.0	± 2.25	1.25
+2	± 2.5	± 1.9	± 2.75	-0.75
0	± 2.25	± 1.8	± 2.75	-2.75

7.6.5 Adopted Scour Depth Fronting Structure

The estimated scour level from a range of techniques is shown in Table 7.7. Taking into consideration the potential for scour from both wave and river processes, WRL adopted a design scour level of -1.5 m AHD. It should be noted that scour due to a large river flood may extend below this level, however, detailed modelling of the river would be needed and construction below the adopted scour level would be impractical within the scope of this project. The adopted toe design could adjust to limited scour below -1.5 m AHD.

Table 7.7: Estimate of Scour Levels at Toe of Structure

Method	**Scour Level** (m AHD)
SBEACH modelling	-0.2
*Chapman and Smith (1983)	-1.75
*Gordon (1987)	-0.9
Rule of thumb	-1.0
Adopted	-1.5

Notes: *values are presented with minor rounding and assuming +1 m AHD average sand level against structure.

7.7 Nearshore Wave Heights

For the 100 year ARI wave, water level and eroded profile condition, depth limited nearshore wave heights were determined using the Dally et al. (1984) surf zone model for significant wave heights and Battjes and Groenendijk (2000) for $H_{10\%}$ and $H_{2\%}$. Results are summarised in Table 7.8.

7.8 Summary of Adopted Design Conditions

The adopted design conditions presented above are summarised in Table 7.8.

Table 7.8: Summary of Design Conditions Estimated for 100 year ARI

Variable	Design Conditions
Design offshore significant wave height (Hso)	8.5 m
Design offshore significant wave direction	South-east
Wave transformation coefficient (K)	1.0
Design still water level (excluding wave setup)	1.33 m AHD
Design spectral peak wave period Tp	13.4 s
Inshore wave setup at foreshore	0.9 m
Design nearshore water level	2.2 m AHD
Design scour level at structure	-1.5 m AHD
Design Hs at structure (Dally et al., 1984)	1.5 m

7.9 Design Crest Level

In order to establish a design crest level for the structure, empirical analysis has been undertaken to examine wave runup, overtopping, stability and safety under design conditions. Tolerable overtopping discharges from USACE (2006) are shown in Table 7.9. Tolerable overtopping discharges from EurOtop (2008) and CIRIA (2007) are shown in Table 7.10.

Table 7.9: Tolerable Overtopping Discharges for Pedestrians (USACE, 2006)

Qualitative Overtopping Hazard for Pedestrians	Mean Overtopping (L/s/m)
Wet, but not uncomfortable	0.0001 - 0.0040
Uncomfortable but not dangerous	0.0040 – 0.0300
Dangerous on vertical wall breakwaters	0.0300 – 1.0000
Very dangerous	> 1.0000

Table 7.10: Tolerable Overtopping Discharges
(Source: EurOtop, 2008 and CIRIA, 2007)

	Hazard Type and Reason	**Tolerable Mean Overtopping Discharge q** (L/s/m)	**Tolerable Maximum Overtopping Volume V_{max}** (L/m)
Pedestrians	Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10	500 at low level
	Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	0.1	20-50 at high level or velocity
	Unusual conditions where pedestrians have no clear view of incoming waves, may be easily upset or frightened, are not dressed to get wet, on a narrow walkway, or in close proximity to trip or fall hazards	0.03	-
Vehicles	Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed	10-50	100-1000
	Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01-0.05	5-50 at high level or velocity
Buildings	No damage	0.001	-
	Minor damage to building structure elements such as fittings etc.	0.001-0.03	-
	Structure damage	0.03	-
	Damage to equipment set back 5-10 m	0.4	-
Promenade or Revetment Seawall	Damage to paved or armoured promenade behind seawall	200	-
	Damage to grassed or lightly protected promenade or reclamation cover	50-200	-
	No damage	50	

Analysis of overtopping is presented in Table 7.11 using input from Table 7.8. By reconciling Table 7.11 with Table 7.10, it can be seen that a crest between of +4 and +5 m AHD would experience overtopping within the limits for pedestrians - "trained staff". It should be noted that these overtopping calculations utilise the best contemporary desktop techniques, but are approximate only. Improved estimates can be obtained from physical modelling, but these need to be corrected for strong onshore winds.

Due to existing ground elevations at the sites (dune crest of 4 to 5 m AHD) and low public use of the foreshore, a crest of +4 m AHD has been adopted.

Table 7.11: Estimated Mean Overtopping Rates for a Range of Crest Levels (Source: EurOtop, 2007)

ARI	Mean Overtopping rates (L/s/m) for Crest Elevation			
	+3 m AHD	+4 m AHD	+5 m AHD	+6 m AHD
100 year ARI	182	17	2	0

8. Concept Design

8.1 General

Sand-filled geotextile container revetments generally have a recommended slope of 1V:1.5H. Figure 20 shows a typical engineered geotextile container seawall acting as a revetment at Portsea, Victoria.

Figure 8.1: Typical Engineered Geotextile Container Revetment, Portsea, Victoria

A preliminary design cross-section for geotextile containers was prepared with a design scour level of -1.5 m AHD. A revetment slope of 1V:1.5H was adopted. A two layer 0.75 m^3 design was recommended with an additional third geotextile container at the toe for "self-healing" purposes.

8.2 Hydraulic Stability

To confirm that such a design is hydraulically stable, standard geotextile container guidelines were considered (Coghlan et al. 2009). These guidelines indicated that for the 0.75 m^3 geotextile containers, the significant wave height initiating damage to the structure is 1.2 to 1.3 m. The guidelines indicated that for the 2.5 m^3 geotextile containers, the significant wave height initiating damage to the structure is 1.6 m.

The estimated significant wave height for 0.75 m^3 containers of 1.2 to 1.3 m is slightly below the estimated design depth limited significant wave height of 1.5 m for the site, which may indicate the need for 2.5 m^3 geotextile containers. However, 0.75 m^3 geotextile containers have been adopted (over 2.5 m^3) for the following reasons:

- Built assets are not under direct threat;
- The wave modelling technique used is conservative for the subject site, since it is inside the river mouth; and
- The site is remote with difficult access – the use of smaller containers (0.75 m^3) would allow smaller machinery to be used.

The behaviour of geotextile containers subject to lateral velocities is unknown. Therefore WRL did not assess the structure stability under river flow velocities.

8.3 Preliminary Design Cross Section

Figure 8.2 shows a preliminary design cross-section for the proposed terminal seawall with geotextile containers, a design scour level of -1.5 m AHD and a design crest level of +4 m AHD. The geotextile container design comprises two layers of 0.75 m^3 containers placed in a "stretcher-bond" fashion, with the long axis of the containers perpendicular to the direction of wave attack (i.e. long axis parallel to wave crest). The outer layer of containers are to be fabricated from vandal deterrent material with the inner layer to be the standard type. A heavy grade geotextile is required between the in-situ material and the geotextile containers to prevent loss of fines.

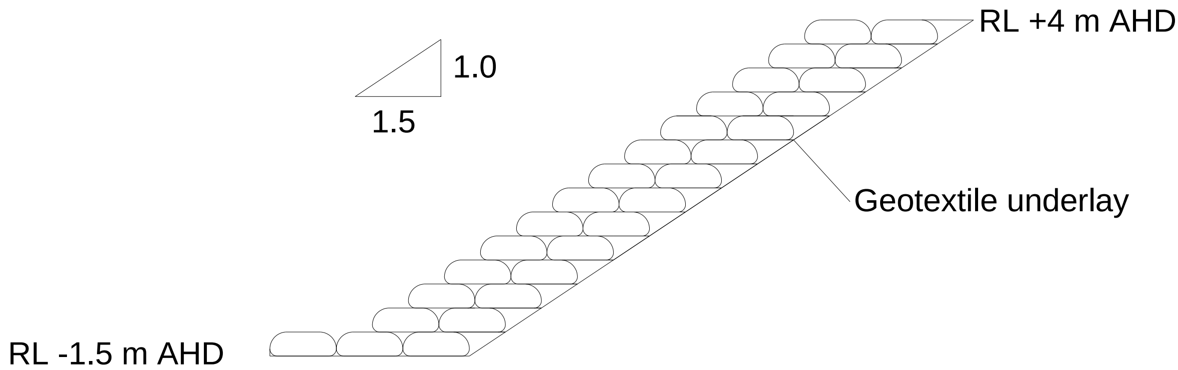

Figure 8.2: Preliminary Sketch of Seawall Cross Section

8.4 Alignment

A detailed site survey would be needed to document the alignment. The cross section could be constructed on the following alignments:

- Remove the existing timber wall and construct wall on existing bank;
- Backfill the existing timber wall and construct new wall seaward of it.

8.5 Construction Machinery

The geotextile containers would be filled using an 8 t excavator and a filling frame. These would be placed on the geotextile underlayer with a 20 t excavator. A front end loader would assist with sand excavation, stockpiling and replacement. Dewatering pumps would also be required to enable geotextile container placement below the water table.

9. Preliminary Costing

9.1 Capital Cost Estimate

The following costs are based on the ELCOROCK® brand of geotextile containers or equivalent. Costs are based on two layers, with an outer layer of 0.75 m^3 containers constructed from sand coloured vandal deterrent geotextile and an inner layer constructed from standard non-woven geotextile.

The total capital cost estimate derived for the preliminary design geotextile container seawall with a crest length of 180 m is approximately **$750,000**.

Substantial cost savings could be made if a single layer structure was adopted, however, this would involve a much higher probability of sudden and catastrophic failure and is not recommended. If a higher level of risk or wave overtopping can be accepted and/or additional design studies are undertaken, a less expensive design with a reduced crest elevation could be considered. A reduction of the crest to +3 m AHD would allow a cost saving of approximately $100,000. This is the minimum crest elevation which could be considered.

9.2 Durability

Geotextile damage due to wave impacts (excluding displacement) and abrasion is considered to be minor. The major concern with regards to durability is vandalism (primarily knife cuts) and damage from driftwood. WRL has recommended the use of vandal deterrent geotextile containers on the outer layers to mitigate this concern. This geotextile allows sand to be trapped within the geotextile and this trapped sand provides further protection from knife cuts. Durability under ultra-violet light exposure is a secondary concern over the design life of the structure. Since use of these structures is relatively modern, their durability in the field when fully exposed to ultra-violet light would only be considered reasonable for up to 25 years.

9.3 Maintenance Cost Estimate

Structural maintenance costs cover the displacement of one or more geotextile containers by wave action and damage to one or more geotextile containers through incidental damage, vandalism, abrasion and ultra-violet light degradation. Considering the relatively sheltered and remote location of the proposed seawall, the average annual cost for maintenance of the geotextile container option has been estimated to be approximately 1.5% of the initial capital cost. This equates to a maintenance cost of approximately $10,000 per year. This estimate would reduce if the seawall remains largely buried and not impacted by waves.

The estimated net present cost for maintenance of the terminal seawall with a discount rate of 7% over 20 years, is approximately $100,000.

10. Design Alternatives

The engineered geotextile container design is relatively expensive. In response to this, three alternative designs have been briefly considered. These could be further assessed if considered feasible. None of these alternatives have been costed, but the works could likely be undertaken for a lower cost than the engineered geotextile container design.

10.1 Alternative 1: Repair and Retain Existing Timber Wall

The following actions could be undertaken to rehabilitate the existing timber walls:

1. Place geotextile behind eroded portions of the timber wall and backfill with sand.
2. Excavate a trench in front of the timber wall and place several courses of geotextile containers in this trench;
3. Construct a low geotextile container wall to above the lowest timber sleepers.

10.2 Alternative 2: Spur Groynes

Two (or more) spur groynes (20 to 50 m long) could be constructed at North Wonboyn from geotextile containers, with one to the east and one to the west of the present area of concern. These would force the channel further south allowing sand to accrete between them. This would allow a greater buffer of sand to be seaward of the private properties to offset erosion when ocean waves penetrate the entrance.

This would need to be viewed as a full scale experimental trial, which would require monitoring and may require future alterations.

10.3 Alternative 3: Managed Channel Realignment

As stated previously, erosion at North Wonboyn is a combination of estuarine and coastal processes.

In recent times, the main estuarine channel has migrated to the northern bank adjacent to the private properties, whereas historically it has often been located further south. Subject to further assessment, it may be feasible to occasionally excavate a new main channel further south which would allow a greater sand buffer to form in front of the private properties. This would require ongoing monitoring and may require future repeat campaigns. If pursued, it would be best undertaken as a series of monitored trials. Such programs have been technically and economically feasible elsewhere.

11. Conclusions

A concept design of seawall for foreshore protection has been developed for North Wonboyn. The seawall would be constructed using geotextile containers filled with sand directly sourced from excess sand from on-site earthworks for batter slope preparation and seawall trenching.

Geotextile materials were chosen because of the relatively low extent of development at the site and its remote location. Typically, geotextile containers are perceived as a softer solution compared to rock or concrete seawall and have low freight requirements.

The seawall design prepared by WRL is for a high quality engineered structure consisting of two layers, which complies with contemporary coastal engineering standards and practice. The total estimated cost for construction of the seawall would be approximately $750,000.

Substantial cost savings could be made if a single layer structure was adopted, however, this would involve a much higher probability of sudden and catastrophic failure and is not recommended. If a higher level of risk or wave overtopping can be accepted and/or additional design studies are undertaken, a less expensive design with a reduced crest elevation could be considered. A reduction of the crest to +3 m AHD would allow a cost saving of approximately $100,000. This is the minimum crest elevation which could be considered.

The engineered geotextile container design is relatively expensive. In response to this, three alternative designs have been briefly considered, namely:

- Alternative 1: Repair and Retain Existing Timber Wall;
- Alternative 2: Spur Groynes; and
- Alternative 3: Managed Channel Realignment.

These could be further assessed if considered feasible. Neither of these alternatives have been costed, but the works could likely be undertaken for a lower cost than the engineered geotextile container design.

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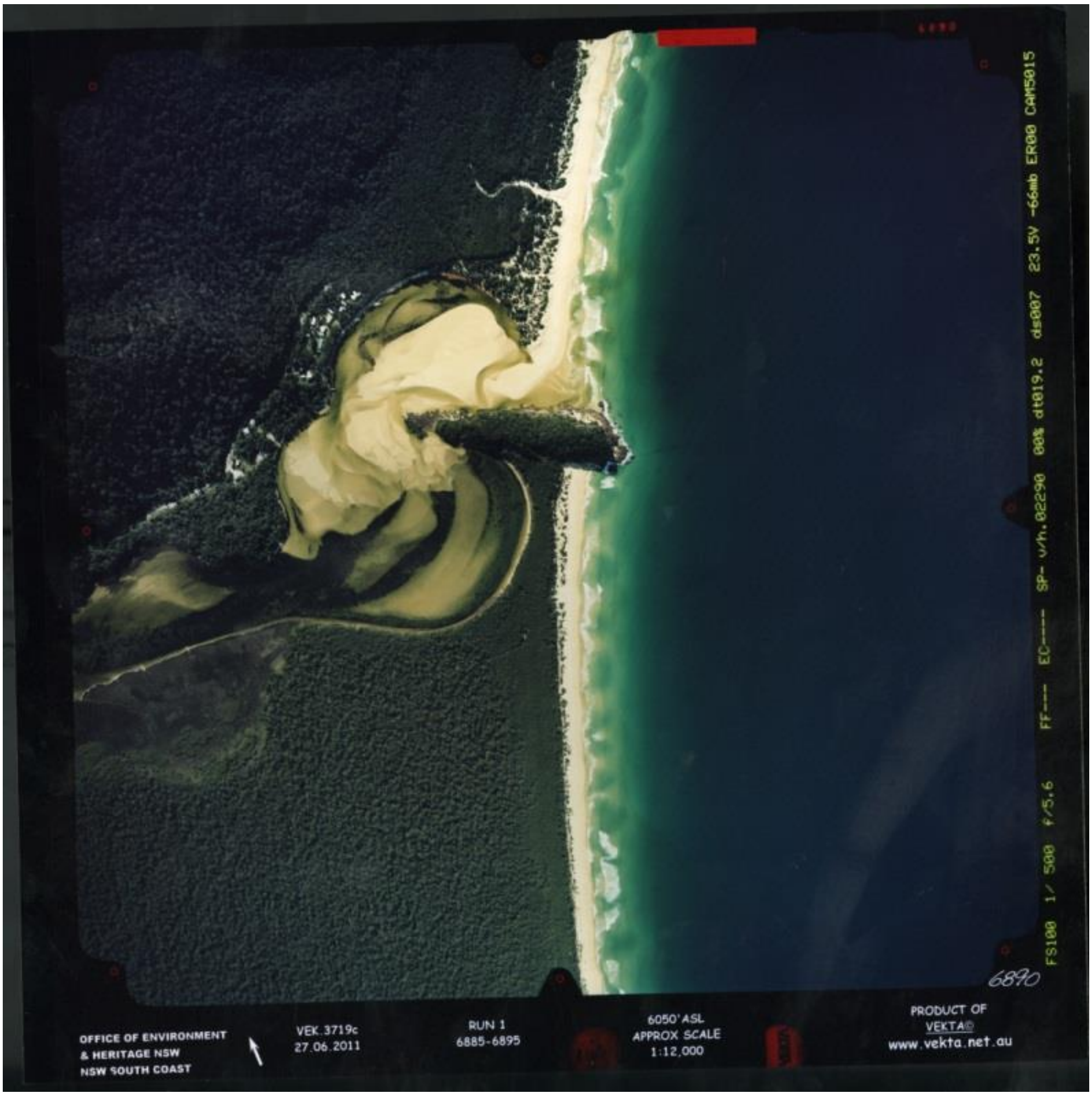
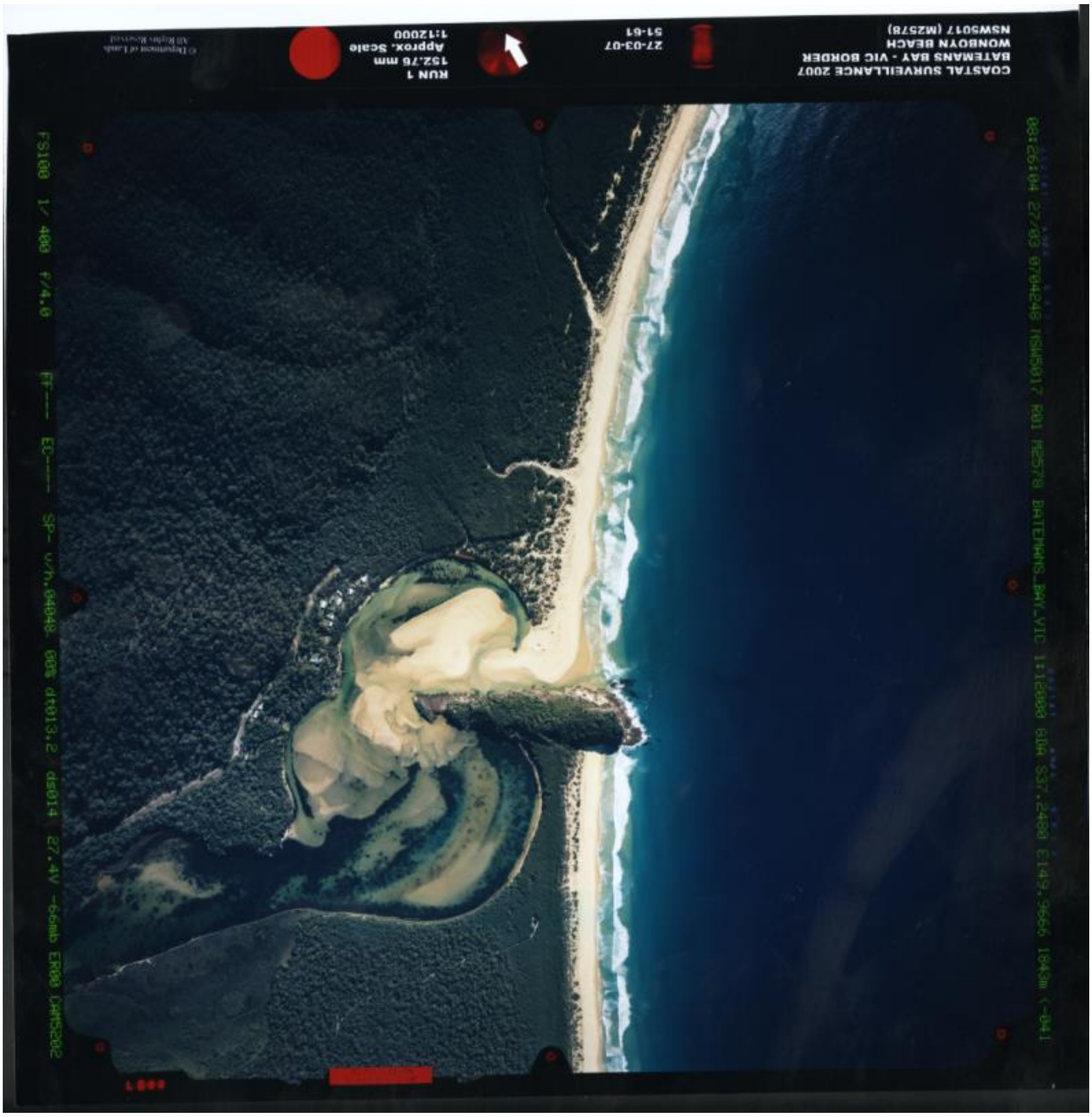
13. Appendix A : Historical Aerial Images

2013


OFFICE OF ENVIRONMENT
& HERITAGE NSW
NSW SOUTH COAST
VEK 3719c
27.06.2011
RUN 1
6885-6895
6050'ASL
APPROX SCALE
1:12,000
PRODUCT OF
VEKTA©
www.vekta.net.au
6890

27/06/2011



27/03/2007


COASTAL SURVEILLANCE 2005
BATEMANS BAY - VICTORIAN BORD.
WONBOYN BEACH
NSW4896 (M2465)
06-03-05
192-202
RUN 1
152.76 mm
Approx. Scale
1:12000

06/03/2005


COASTAL SURVEILLANCE 2001
BATEMANS BAY - VICT. BORDER
WONBOYN BEACH
NSW4549 (M2265)
01-04-01
01-11
RUN 1
152.76 mm
Approx. Scale
1:12000
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01/04/2001

28/02/1989


NSW
2016
5188
CROWN
COPY N.S.W
LANDS PHOTO
LOG-E
NSW COASTLINE
NSW 2016
RUN 15
4·6·72
20,000'A.S.L
152·36MM

04/06/1972

June 1962