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River Road Foreshore, Shoalhaven Heads: Assessment of Coastal Management Options

WRL Technical Report 2016/21
August 2017

FINAL DRAFT

By M J Blacka and I R Coghlan

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1. Introduction

1.1 Preamble

The estuary foreshore adjacent to River Road in Shoalhaven Heads suffered localised but significant erosion following a series of storm events that culminated with the East Coast Low of early June 2016. The localised erosion of some sections of the beach resulted in land-slips on the steep back-beach embankment, the loss of a range of dune vegetation including some mature trees, and left the embankment in a potentially unstable form with further trees at risk of toppling. Figure 1.1 shows the location of the River Road foreshore area.

In response to community concerns, Shoalhaven City Council (SCC, Council) engaged the Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney, to undertake an assessment of conceptual coastal management options for the eroded foreshore area. WRL was assisted by JK Geotechnics who provided expert geotechnical engineering analysis. This erosion assessment took into consideration broader options for managing the entrance of the Shoalhaven River tabled in a separate project by WRL *Management Options for Improving Flows of the Shoalhaven River at Shoalhaven Heads,* (Glamore et al. 2015).

This report presents the results of our assessment of foreshore management options along the River Road area. Results of the geotechnical inspection and risk analysis have been provided in a separate report by JK Geotechnics, reproduced as Appendix E.

1.2 Study Area

The investigation area comprised the section of foreshore between the public jetty opposite Jerry Bailey Road at the South Western extent, extending to the boat ramp at the North Eastern extent (Figure 1.2). A reference line has been established along the foreshore as shown in Figure 1.2 to maintain clarity and continuity when presenting the results of the field inspection and assessment of management options. The reference line starts at the southern end of the foreshore with chainage 0 m at the jetty, and proceeds to chainage 1060 m at the northern end of the foreshore adjacent to the boat ramp.

1.3 Objectives of Project

Following discussions with Council staff, the agreed objectives of the project include:

- Assessing the condition of the site;
- Developing a better understanding of the processes impacting the foreshore;
- Understanding the risks from coastal and geotechnical hazards and how these vary along the foreshore;
- Scoping realistic/achievable concept management options for the most critical areas;
- Evaluating and selecting a preferred concept management option that addresses short-term risks while not impeding long-term management aspirations.

As well as addressing coastal erosion and geotechnical hazards, the management options were also required to consider stormwater impacts and the presence of asbestos containing materials (ACM) in foreshore debris.

The process followed during the project is presented in Figure 1.3.

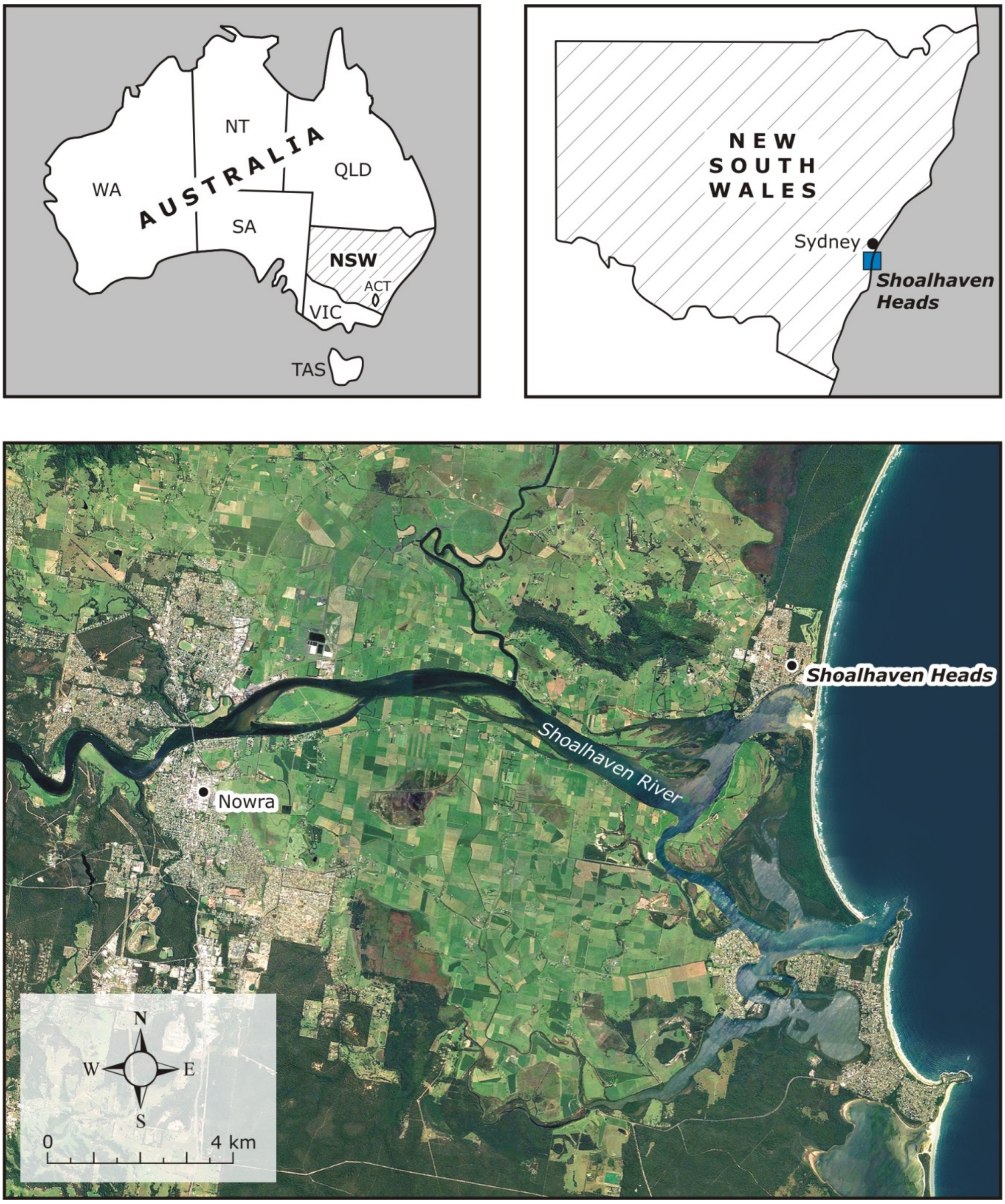


Figure 1.1: River Road Foreshore Location, Shoalhaven Heads

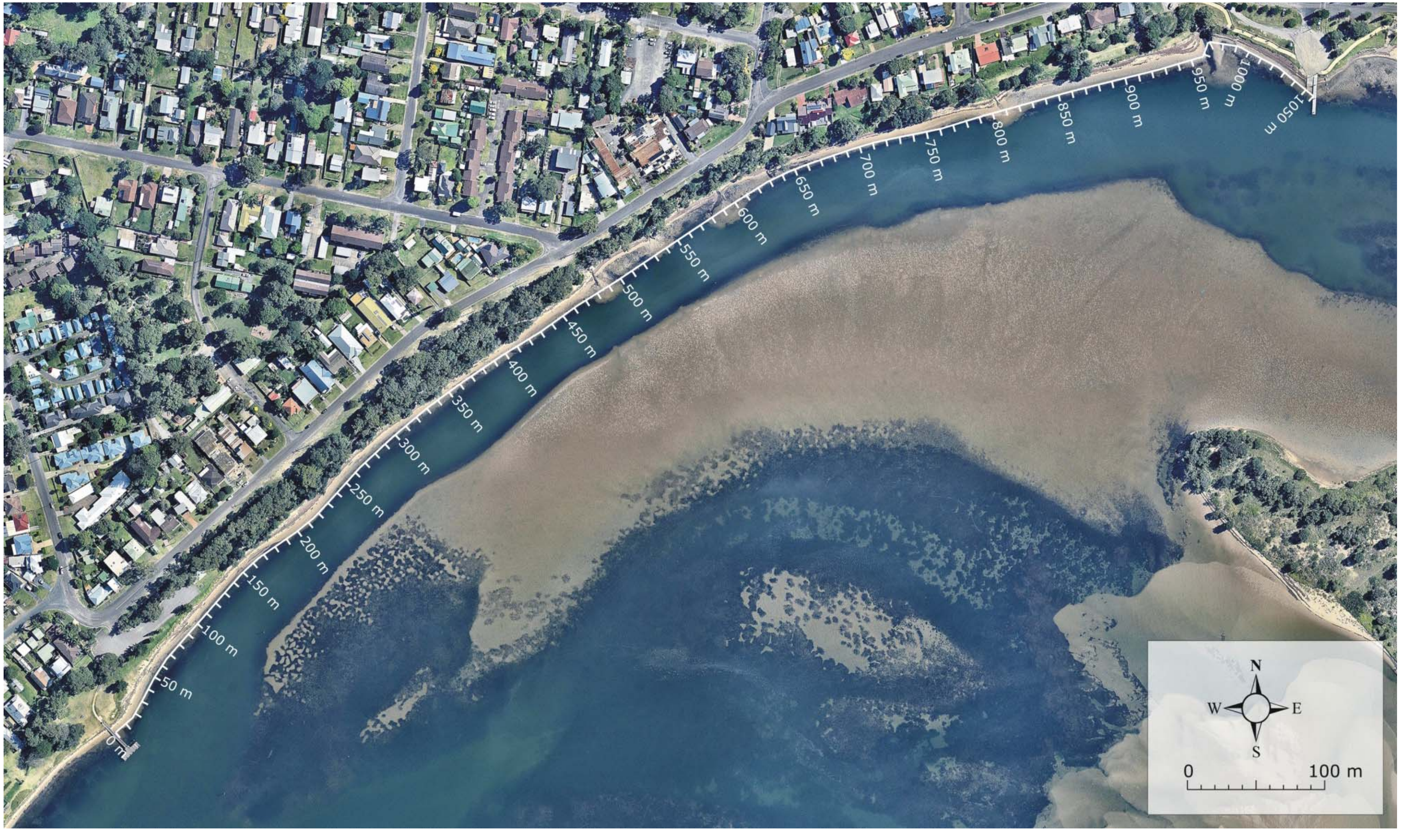


Figure 1.2: Investigation Area and Reference Line

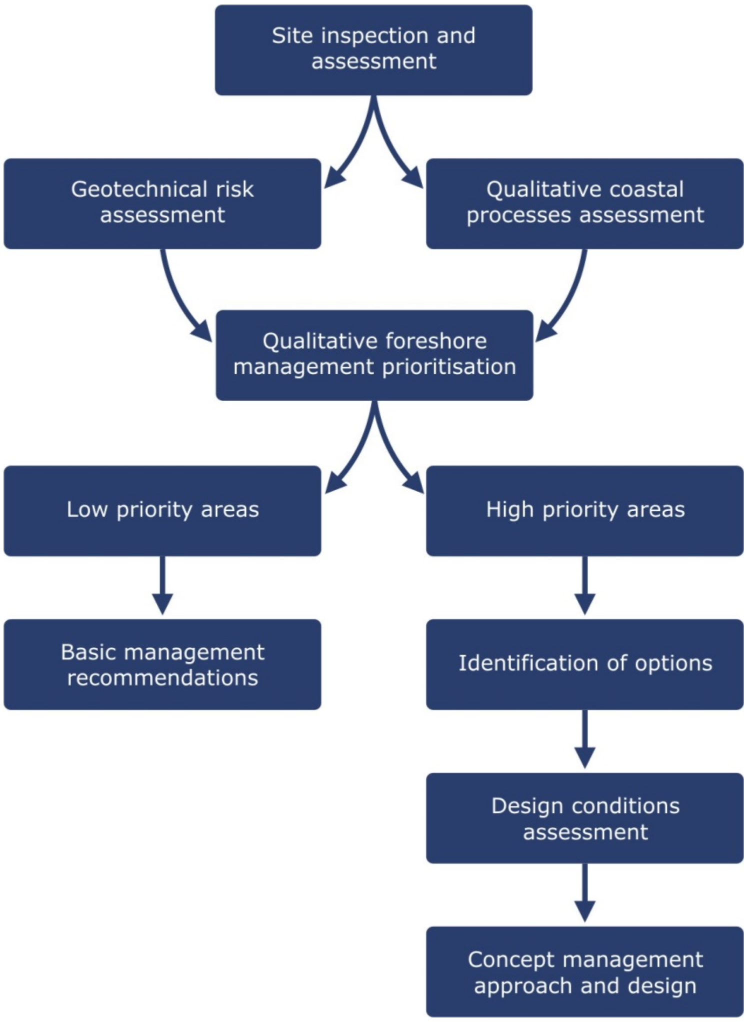


Figure 1.3: Project Process

2. Site Inspection

2.1 Overview of Site Inspection

Principal Coastal Engineer Matt Blacka from WRL and Engineering Geologist Paul Roberts from JK Geotechnics inspected the site and its immediate surrounds on 21st September 2016. The walk-over inspection included the collection of a range of information and observations relevant to developing a background understanding of the coastal processes and geotechnical risk such as:

- Relevant sediment movement pathways, and influence of natural and built structures on sediment transport;
- Impacts of previous storms on the foreshore and vegetation, including areas of localised or focussed damage;
- Impacts of previous mitigation/management works including type, location, materials, influence on processes etc.;
- Relative exposure to environmental processes (such as ocean swell waves from the estuary entrance);
- Photographic records of the site;
- Details regarding topographic and bathymetric form;
- Details of surface (and inferred subsurface) drainage;
- Details of geological conditions.

Relevant details from the site inspection are presented in Section 2.2 below (and in Appendix E for the geotechnical risk assessment).

2.2 Site Description

2.2.1 General Description

The foreshore has four zones with distinctly different characteristics along the 1 km length of study area. These zones are described below and presented in Figure 2.1 to Figure 2.3:

- Zone 1 (Chainage 0 to 140 m): South Western zone fronting the carpark and public toilet area opposite Jerry Bailey Road. Characterised by a relatively low flat area of fill that forms the car park, dropping down a steep (near vertical) bank of varying height (up to ~1 m), to the low gradient sandy beach.
- Zone 2 (Chainage 140 to 650 m): Southern central zone between the carpark/toilets and River Road intersection with Mathews Street. Characterised by a steep vegetated back-beach sand embankment (up to ~ 6 m height) between the low gradient sandy beach at the toe and a level grassed crest extending to the River Road pavement.
- Zone 3 (Chainage 650 to 970 m): Northern central zone between Mathews Street and the stormwater/creek outlet at the River Road Reserve. Characterised by elevated private properties with yards dropping to a level and grassed (in some areas) back-beach apron, fronted by a low gradient sandy beach.
- Zone 4 (Chainage 970 to 1060 m): North Eastern zone between the stormwater outlet at the River Road Reserve and the boat ramp. Characterised by a low-lying back beach area with car parking and footpaths, fronted by a low gradient sandy beach.

A detailed site description with photos of key features is provided in Section 2.2.

Figure 2.1: Foreshore Zone Map

Figure 2.2: Overview Photos of Foreshore Characteristics, Zone 1 (T) and Zone 2 (B)

Figure 2.3: Overview Photos of Foreshore Characteristics, Zone 3 (T) and Zone 4 (B)

2.2.2 Zone 1 Detailed Description (Chainage 0 m to 140 m)

The foreshore area in Zone 1 primarily comprises a levelled car park built on earth fill, with a public toilet building at the north eastern extent. Th e levelled area sits landward of a shallow sloping sandy beach, separated by a steep (near vertical) eroded bank (max height ~1.2 m) of fill material. The area between the top of the bank and the paved carpark area is surfaced with short grass, with a number of Norfolk Island pine trees also present. The trees and a row of treated pine parking bollards are set back from the top of the eroded bank by several metres. The toe of the eroded bank sits just above the typical high tide levels at the time of the inspection Figure 2.4 and Figure 2.5 show the foreshore characteristics typical of Zone 1.

Figure 2.4 Foreshore Zone 1 Looking South West

Figure 2.5 Foreshore Zone 1 Looking North East

2.2.3 Zone 2 Detailed Description (Chainage 140 m to 650 m)

Zone 2 extends from the end of the car park north to the intersection of Mathews Street with River Road. This zone is characterised by a narrow and eroded sandy beach, a very steep vegetated back beach sand embankment up to approximately 6 m in height, and a flat grassed shoulder between the top of the embankment and the edge of the paved surface of River Road. Power line poles are founded in the grassy shoulder adjacent to the top of the embankment along the length of the zone.

Along the south western half of Zone 2 (Zone 2A, Chainage 140 – 480 m), there is a low-lying back beach vegetated dune area that separates the sandy beach from the toe of the embankment by some 5 – 10 m (Figure 2.6 and Figure 2.7). The vegetation in this area comprises grasses, a range of understory shrubs, as well as semi-mature Banksias and Eucalypts. A number of the Banksias and Eucalypts had been toppled by erosion in recent storms, or had been removed by Council due to safety concerns. Within this zone the sand embankment at the rear of the beach was noted to be well vegetated, at a relatively stable slope, and showed no signs of impacts from wave runup or erosion. The grassed area between the crest of the embankment and the edge of River road is typically between 15 and 20 m wide in this area, representing a reasonable setback of the road and power poles from the top of the bank.

Figure 2.6: Foreshore Zone 2A (Taken at Chainage ~200 m, Looking South West)

Figure 2.7: Foreshore Zone 2A (Taken at Chainage ~450 m, Looking South West)

Along the north eastern half of Zone 2 (Zone 2B, Chainage 480 - 650 m), the toe of the back beach embankment is just above the high tide level, with the sandy beach and high tide water level reaching right to the toe of the embankment. In general the sand embankment in this area is at or beyond the gradient that would typically be considered stable for unconsolidated sands, however, is being held in most locations by vegetation.

At a number of locations in Zone 2B there has been localised land-slip occur on the embankment with sub-vertical scarps several metres in height, and debris consisting of fallen trees and root balls also present at the base of the scarps (Figure 2.8, Figure 2.9, Figure 2.10). It is evident along the length of Zone 2B that the base of the embankment is being directly impacted by waves during periods of high water level and/or larger wave conditions, exacerbating the potential for further land-slip on the embankment.

Figure 2.8 Back Beach Embankment in Zone 2B with extensive land-slip (~Chainage 500 m)

Figure 2.9 Back Beach Embankment in Zone 2B with extensive land-slip (~Chainage 530 m)

Figure 2.10 Back Beach Embankment in Zone 2B with extensive land-slip (~Chainage 550 m)

There are two sets of stairs in Zone 2B providing access onto the beach, the first opposite Renown Avenue (Chainage 480 m) and the second opposite the Shoalhaven Heads Hotel (Chainage 560 m). The Renown Avenue stairs are shown in Figure 2.11.

Figure 2.11 Beach Access Opposite Renown Avenue (~Chainage 480 m)

Zone 2B also has three stormwater outlets that discharge water across the beach (Chainage 490 m, 535 m and 590 m). A row of geotextile sand containers (likely 0.75 m^3 Elcorocks) run across the beach at the location of each stormwater outlet. At all three outlets, there was evidence that sediment from the beach had been pushed into the estuary channel by stormwater flows to form a significant fan/accumulation (Figure 2.1).

The first stormwater drain (two concrete pipes) discharges at the top of the back beach embankment adjacent to the Renown Street stairs (Chainage 490 m, Figure 2.12), with the embankment treated in this area with a concreted rock boulder surface, presumably to prevent the stormwater from scouring the embankment as it cascades to the beach. Stormwater passes down the face of the concreted rock wall and runs directly across the beach, guided by geotextile containers. The geotextile containers appear in poor condition.

Figure 2.12 Stormwater Outlet and Concrete Cascade at Chainage 490 m

The second stormwater drain (Chainage 535 m, Figure 2.13) emerges at the toe of the embankment, and has a concrete headwall fitted. This drain discharges water directly across the beach. There are a number of damaged geotextile sand containers to guide the stormwater across the beach, however, these appear to have lost most of their contents and are now at the same level as the beach.

The third stormwater drain (Chainage 590 m, Figure 2.14) emerges approximately 1 m seaward of the embankment toe, and is a spiral corrugated metal pipe. There is a significant amount of debris (stone boulders and other building rubble) cast around this stormwater outlet. This outlet also discharges water directly across the beach.

Figure 2.13 Stormwater Outlet and Damaged Geotextile Bag at Chainage 535 m

Figure 2.14 Stormwater Outlet and Rubble Debris at Chainage 590 m

2.2.4 Zone 3 Detailed Description (Chainage 650 m to 980 m)

Zone 3 extends from the intersection of River Road and Mathews Street north-east to the stormwater outlet/creek at the River Road Reserve (Figure 2.1). Along this zone there are 13 private properties from 62 to 86 River Road with back yards directly adjoining the foreshore reserve. This zone is generally characterised by the elevated properties/yards that drop down to a level grassed back beach apron, which drops to the sandy beach (Figure 2.3).

Along the Western half of this zone (Zone 3A, Chainage 650 to 720 m), the toe of the back beach embankment is protected by an uncertified rock seawall at the rear of the beach (Figure 2.15, Figure 2.16). The rock wall extends to a maximum height of approximately 1.6 m above the beach, and is on a near vertical slope. The boulders in the wall are of variable size, with dimensions typically of the order of 400 – 700 mm, and estimated mass in the range of 100 – 500 kg.

Figure 2.15 Uncertified Rock Seawall in Zone 3A

Figure 2.16 Uncertified Rock Seawall in Zone 3A

Along the eastern half of this zone (Zone 3B, Chainage 720 to 970 m), the backyards of the private properties drop down a retained and gardened slope to a level and grassed apron, which drops down to the sandy beach (Figure 2.17, Figure 2.18). There are a number of mature Banksias along this section of foreshore, typically located at the interface between the sandy beach and the grassed back-beach apron. Between Chainage 720 m and 820 m, this interface at the back of the beach has suffered minor erosion, with a crumbling scarp formed into the earth fill of the back-beach apron.

At Chainage 810 m there is an open stormwater drain that discharges onto the rear of the beach (Figure 2.19). There is a small amount of rock rubble placed around the end of the drain. It is understood that there is a public access that passes from River Road to the beach at this location, running between properties at 74 and 76 River Road.

Figure 2.17 Grassed Apron and Beach, Typical of Zone 3B, Looking West

Figure 2.18 Grassed Apron and Beach, Typical of Zone 3B, Looking East

Figure 2.19 Open Stormwater Drain and Rock Rubble at Chainage 810 m

2.2.5 Zone 4 Detailed Description (Chainage 970 m to 1060 m)

Foreshore Zone 4 extends from the stormwater drain/creek at the River Road Reserve (Chainage 970 m) east to the public boat ramp (Chainage 1060 m). For the most part, this section comprises a low-lying level car parking area and footpath separated from the beach by a narrow grassy strip (Figure 2.20). At the western end of Zone 4, there is a gated, dual pipe stormwater outlet that discharges at the rear of the beach, forming a creek/channel across the beach (Figure 2.21).

At approximately Chainage 990 m there is a short groyne comprising three geotextile sand-filled bags (presumably 0.75 m^3 Elcorock units), which remain intact but are relatively loosely filled (Figure 2.22). The groyne has retained a fillet of sand on the western side and is at its sediment retaining capacity. The beach to the east of the groyne appears starved of sand, with a lower lying profile and gravely surface. There has been minor erosion of the back-beach area to the east of the groyne (Chainage 990 to 1040 m), presumably from recent storms (Figure 2.23). The erosion scarp is typically low (less than 0.5 m height).

Figure 2.20 Zone 4 Foreshore Profile at Chainage 1000 m, Looking East

Figure 2.21 Gated Stormwater Outlet at Chainage 970 m

Figure 2.22 Geotextile Bag Groyne at Chainage 990 m

Figure 2.23 Minor Erosion, Typical of Chainage 990 m to 1040 m

3. Coastal Processes and Hazards Assessment

3.1 Preamble

The River Road foreshore area and beach at Shoalhaven Heads is typically a sheltered estuarine environment, with exposure to short period and low-energy wind seas and tidal currents. On occasions the estuary entrance is broken open to the sea during flooding events, scouring the entrance area and allowing longer period ocean swell waves to cross the lower estuary to the River Road foreshore area. Exposure of the foreshore to these more energetic and erosive conditions is therefore episodic, and requires the combination of:

- Elevated estuary water levels through either terrestrial flooding or ocean surge;
- The ocean entrance of the river to be open;
- Large ocean waves or very strong winds blowing across the estuary.

Typically these events are not independent (for example the June 2016 storm event), and therefore understanding the probability and risk of future erosion events/episodes is complex and has not been analysed in detail. The risk is further complicated by the fact that the entrance stays open for a variable period of time before shoaling up, and in theory it is possible for ongoing erosion to occur after initial opening with only elevated ocean levels and big swell (i.e. without the need for further terrestrial flooding).

While a large range of coastal and estuarine processes occur at the River Road foreshore, for the purposes of understanding the erosion of the foreshore, this section of the report focuses on the Shoalhaven Heads entrance processes as well as stormwater drainage as these processes appear to have most influenced the observed erosion.

3.2 Entrance Opening

In the *Shoalhaven River Entrance Study* (Nittim and Cox, 1986), the conditions for natural opening of the entrance were described as follows:

> "Once the water level had reached 2 metres above MSL in the Bay [Shoalhaven Heads] and 3 metres on the Nowra gauge, a break-out could be successfully initiated".

Webb McKeown & Associates (2006) states that the entrance will open naturally in floods equal to or greater than 20% Annual Exceedance Probability (AEP). Note that the August 1974 flood, which resulted in the widest open entrance condition on record, has been reported as having an AEP of 5% (Posford et al. 1977) and later an AEP 8.3% (Webb McKeown & Associates, 2006).

The status of the Shoalhaven Heads entrance between 1936 and 2016 has been tabulated by WRL in Table 3.1, based on a variety of sources. Over this time, the entrance was open approximately two-thirds of the time (67%) and closed one-third of the time (33%). Chafer (1998) asserted that there was a "quasi-septennial cycle in the entrance condition since 1936, with open and closed regimes lasting for 6-9 years". However, this simple rule-of-thumb is cautioned by Glamore et al. (2015), who note that:

> "a review of flood frequencies suggests that reduced flooding over the past 16-24 years may have curtailed the cyclical scour-infilling dynamics. This is further complicated by the expansion of dune vegetation towards the entrance and the expansion of Berry's Canal to a quasi-equilibrium state."

Table 3.1: History of Shoalhaven Heads Entrance Condition (1936-2016)

Date	Entrance Condition	Reference
??/??/1936	Open	Posford et al. (1977): Observation
??/05/1948	Open	Posford et al. (1977): Observation
04/04/1949	Closed	Chafer (1998): Aerial Photograph
1950-1960	Open	Posford et al. (1977): Observation
??/07/1961	Closed	Posford et al. (1977): Observation
21/09/1961	Open	AWACS (1999): Aerial Photograph
??/11/1961	Open	Posford et al. (1977): Observation
??/??/1963	Open	Posford et al. (1977): Observation
??/??1965	Closed	Posford et al. (1977): Observation
02/04/1966	Closed	AWACS (1999): Aerial Photograph
??/??1967	Open	Posford et al. (1977): Observation
1968-1969	Open/Closing	Posford et al. (1977): Observation
08/01/1969	Closed	AWACS (1999): Aerial Photograph
24/05/1969	Closed	AWACS (1999): Aerial Photograph
16/04/1970	Closed	Chafer (1998): Aerial Photograph
23/05/1970	Closed	AWACS (1999): Aerial Photograph
17/07/1970	Closed	AWACS (1999): Aerial Photograph
"Beginning" of 1971	Open	Posford et al. (1977): Observation
??/06/1972	Closed	Posford et al. (1977): Aerial Photograph
??/??/1973	Closed	Posford et al. (1977): Observation
??/05/1974	Closed	Posford et al. (1977): Observation
??/06/1974	Open	Posford et al. (1977): Observation
??/07/1974	Open/Closing	Posford et al. (1977): Aerial Photograph
29/08/1974	Open	Posford et al. (1977): Aerial Photograph
29/12/1974	Open	Chafer (1998): Aerial Photograph
20/05/1975	Open	AWACS (1999): Hydrosurvey
??/06/1975	Open	PWD (1990b): Observation
10/07/1975	Open	Posford et al. (1977): Observation
15/08/1975	Open	AWACS (1999): Hydrosurvey
12/09/1975	Open	AWACS (1999): Hydrosurvey
01/03/1976	Open	Posford et al. (1977): Observation
??/10/1976	Open	PWD (1990a): Observation
28/01/1977	Open	AWACS (1999): Hydrosurvey
14/03/1977	Open	AWACS (1999): Aerial Photograph
08/08/1977	Open	AWACS (1999): Hydrosurvey
23/08/1977	Open	Chafer (1998): Aerial Photograph
21/03/1978	Open	PWD (1990a): Observation
17/04/1978	Open	AWACS (1999): Aerial Photograph
30/07/1978	Open	AWACS (1999): Aerial Photograph
04/10/1978	Open	AWACS (1999): Hydrosurvey
26/6/1979	Open	Chafer (1998): Aerial Photograph
06/07/1979	Open	AWACS (1999): Aerial Photograph
28/02/1980	Closing	Chafer (1998): Aerial Photograph
12/02/1981	Closing	Chafer (1998): Aerial Photograph

Table 3.1: History of Shoalhaven Heads Entrance Condition (1936-2016) (Cont.)

Date	Entrance Condition	Reference
18/03/1981	Closed	AWACS (1999): Aerial Photograph
01/08/1981	Closed	AWACS (1999): Hydrosurvey
01/10/1981	Closed	AWACS (1999): Hydrosurvey
09/01/1982	Closed	AWACS (1999): Aerial Photograph
26/01/1983	Closed	AWACS (1999): Hydrosurvey
13/05/1984	Closed	Chafer (1998): Aerial Photograph
01/08/1986	Closed	Chafer (1998): Aerial Photograph
??/04/1988	Open	PWD (1990a): Observation
06/04/1989	Closed	Chafer (1998): Aerial Photograph
17/04/1989	Open	AWACS (1999): Hydrosurvey
22/04/1989	Open	Chafer (1998): Aerial Photograph
01/08/1990	Open	PWD (1990b): Observation
16/08/1990	Open	PWD (1990b): Aerial Photograph
22/04/1991	Open	Chafer (1998): Aerial Photograph
??/06/1991	Open	Webb, McKeown & Associates (2008): Aerial Photograph
01/07/1991-30/06/1992	Open	AWACS (1999): Observation
04/02/1993	Closing	Chafer (1998): Aerial Photograph
01/07/1994-30/06/1995	Closed	AWACS (1999): Observation
01/08/1995-31/08/1995	Closed	AWACS (1999): Observation
15/01/1996	Closed	Chafer (1998): Aerial Photograph
??/06/1997	Open	SCC (2006): Observation
08/08/1998	Open	SCC (2006): Record of Manual Opening
24/10/1999	Open	SCC (2006): Record of Manual Opening
30/11/1999	Closing	Glamore et al. (2015): Aerial Photograph
13/09/2005	Closed	Google Earth: Aerial Photograph
30/01/2006	Closed	Google Earth: Aerial Photograph
15/08/2009	Closed	Google Earth: Aerial Photograph
12/11/2010	Closed	NearMap: Aerial Photograph
01/02/2011	Closed	NearMap: Aerial Photograph
07/06/2011	Closed	NearMap: Aerial Photograph
14/09/2011	Closed	NearMap: Aerial Photograph
05/11/2011	Closed	NearMap: Aerial Photograph
27/11/2011	Closed	NearMap: Aerial Photograph
14/04/2012	Closed	NearMap: Aerial Photograph
17/07/2012	Closed	NearMap: Aerial Photograph
17/05/2013	Closed	Google Earth: Aerial Photograph
05/07/2013	Open	NearMap: Aerial Photograph
19/11/2013	Open	Google Earth: Aerial Photograph
02/02/2014	Open	Google Earth: Aerial Photograph
16/06/2014	Closed	NearMap: Aerial Photograph
21/06/2014	Closed	NearMap: Aerial Photograph
03/08/2014	Closed	NearMap: Aerial Photograph
08/05/2015	Closed	NearMap: Aerial Photograph
20/07/2015	Closed	Google Earth: Aerial Photograph
10/08/2015	Closed	NearMap: Aerial Photograph

Table 3.1: History of Shoalhaven Heads Entrance Condition (1936-2016) (Cont.)

Date	Entrance Condition	Reference
29/12/2015	Open	Google Earth: Aerial Photograph
17/01/2016	Open	Google Earth: Aerial Photograph
13/02/2016	Open	Google Earth: Aerial Photograph
24/02/2016	Open	Google Earth: Aerial Photograph
04/03/2016	Closing	NearMap: Aerial Photograph
12/03/2016	Closing	Google Earth: Aerial Photograph
19/04/2016	Closed	Google Earth: Aerial Photograph
29/05/2016	Closed	NearMap: Aerial Photograph
06/06/2016	Open	SCC Facebook Page
24/06/2016	Open	WRL Site Inspection
03/07/2016	Open	WRL UAV Aerial Photographs
21/09/2016	Open	WRL Site Inspection

Previous analysis of river entrance opening frequency (Glamore et al. 2016; Chafer, 1998) suggest that on average the entrance breaks open to the sea approximately every 7 years, and stays open for varying lengths of time. Therefore, a simplistic estimate of the risk of an erosion episode occurring would 13% AEP. It is difficult to know whether this is conservative or un-conservative, as:

- It is possible to have an entrance opening (flood event) without large ocean swell, and therefore erosion along the River Road foreshore may not occur and assuming the 13% AEP would be conservative;
- After the entrance is open, an erosion event may occur with only large ocean swell/tides and not further terrestrial flooding so assuming the 13% AEP would be un-conservative.

For the purposes of this investigation, it is sufficient to say that every time the entrance breaks open (13% AEP), there will be a risk of further erosion of the embankment and any geotechnical hazards that this might induce. The risk is further elevated by the potential of erosion caused by wind driven waves across the estuary at times of high water level.

3.3 Stormwater Processes

As identified in the inspection of the site, there are a number of stormwater drains opening onto the back of the beach in Zones 2 and 3 of the foreshore (Figure 2.12, Figure 2.13, Figure 2.14 and Figure 2.19). A single row of geotextile sandbags has previously been installed at each stormwater drain, presumably to assist in guiding stormwater flows across the beach to the estuary channel.

While no quantitative analysis of stormwater flows or their influence on sediment transport has been undertaken, observations made during the site inspection have led to the development of a conceptual model of the influence of stormwater drains on foreshore processes. Key processes include (Figure 3.1):

- At low tide there are significant 'deltas' or 'lobes' of sediment evident, which extend into the estuary channel directly adjacent to each of the stormwater outlets;

- The sediment lobes are likely to be the result of stormwater flows transporting sediment off the beach towards the estuary channel;
- Analysis of aerial photos suggests that the lobes of sand appear to have stabilised in aerial extent over recent years, suggesting that there is the potential that sand is also being lost off the lobes and into the estuary channel.

Figure 3.1: Influence of Stormwater on Sediment Transport

4. Summary of Geotechnical Hazard Assessment

A geotechnical risk assessment was undertaken by JK Geotechnics for the foreshore areas approximately corresponding with Zones 2 and 3a, with the assessment report provided in Appendix E and the results of the assessment summarised here. Due to the nature of the potential geotechnical risks and the assets perceived to be at risk, the assessment was undertaken using the RMS "Guide to Slope Risk Assessment", version 4, 2014.

Following the walkover site inspection, JK Geotechnics identified that potential geotechnical hazards at the site were associated with:

- Hazard pathway 1: Regression of the existing landslip/erosion back scarps (Figure 4.1);
- Hazard pathway 2: Instability caused by future coastal erosion events (Figure 4.2);
- Hazard pathway 3: On-going creep of the over-steep foreshore slope (Figure 4.3).

An assessment of the likelihood and consequences of the hazards was undertaken consistent with RMS (2014), to determine Assessed Risk Levels (ARLs) for each hazard pathway and section of the foreshore. Table 4.1 provides a summary of Consequence Classes, Likelihoods and Assessed Risk Levels for this risk assessment method.

Table 4.1 Assessed Risk Level Matrix

Increasing Consequences →

Increasing Likelihood ↑

	Consequence Class				
Likelihood	**C5**	**C4**	**C3**	**C2**	**C1**
L1	ARL3	ARL2	ARL1	ARL1	ARL1
L2	ARL4	ARL3	ARL2	ARL1	ARL1
L3	ARL5	ARL4	ARL3	ARL2	ARL1
L4	ARL5	ARL5	ARL4	ARL3	ARL2
L5	ARL5	ARL5	ARL5	ARL4	ARL3
L6	ARL5	ARL5	ARL5	ARL5	ARL4

Based on the risk assessment, the Assessed Risk Levels were:

- For Foreshore Zone 2B:
  - Potential Geotechnical Hazard 1: ARL4, assuming on-going recession, further landslips and further recession impacting future landslip back scarps over the next 50 to 100 years.
  - Potential Geotechnical Hazard 2: ARL3, assuming additional erosion events of a similar magnitude occurring over the next 50 to 100 years.
  - Potential Geotechnical Hazard 3: ARL5, assuming on-going creep occurring over the next 50 to 100 years.

- For Foreshore Zones 2A and 3A:
  - Potential Geotechnical Hazard 1: ARL5, assuming on-going recession, further landslips and further recession impacting future landslip back scarps over the next 50 to 100 years.

- Potential Geotechnical Hazard 2: ARL5 for additional erosion events of a similar magnitude occurring over the next 50 to 100 years.
- Potential Geotechnical Hazard 3: ARL5, assuming on-going creep occurring over the next 50 to 100 years.

The risk assessment concluded that:

- Current levels of geotechnical risk are considered acceptable, with the exception of future erosion events causing ongoing landslip (hazard pathway 2) within Foreshore Zone 2B (between Renown Avenue and Mathews Street intersections with River Road).
- "……construction of foreshore erosion protection measures would reduce the risk to 'acceptable' levels".
- Council should monitor the foreshore slope in order to assess existing conditions and any indications of deterioration such as tension cracks along the crest area of the foreshore slope, further evidence of landslips, damage to timber steps, drainage culverts etc.:
  - on an annual basis;
  - after periods of prolonged or heavy rainfall;
  - during periods of predicted peak tidal levels and/or wave conditions.

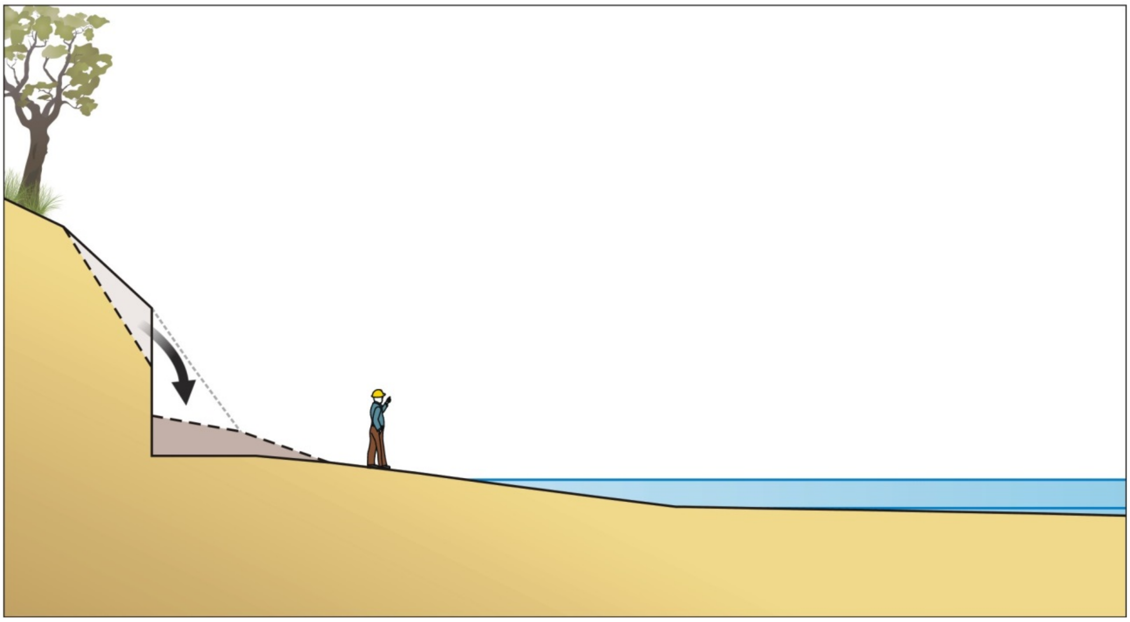


Figure 4.1: Hazard Pathway 1 - Regression of Existing Erosion Scarps

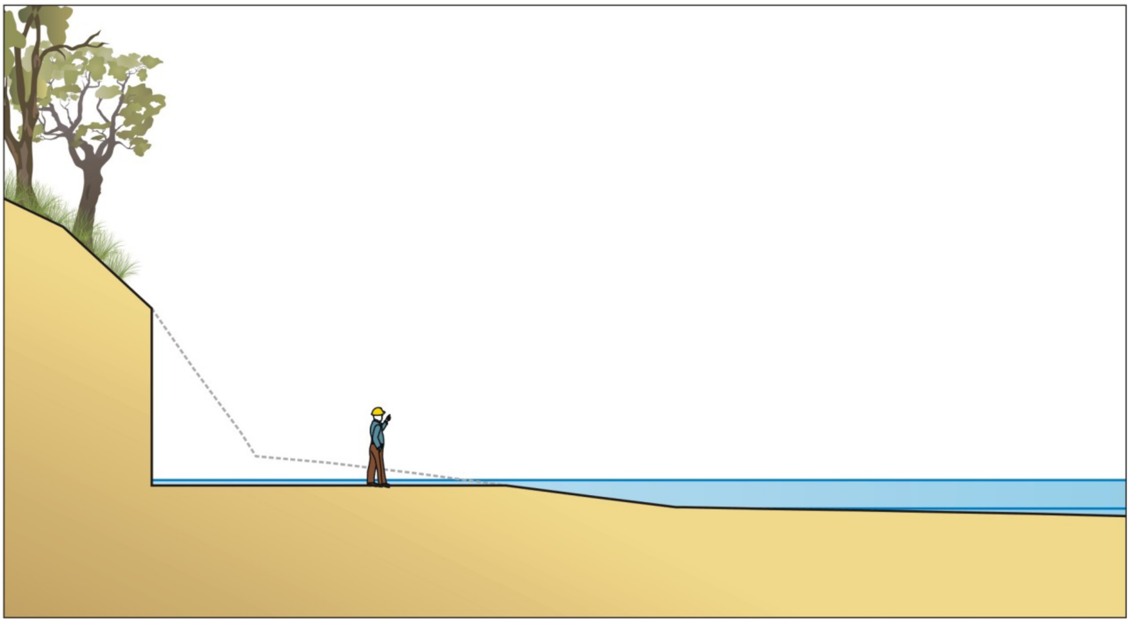


Figure 4.2: Hazard Pathway 2 – Additional Erosion and Landslip

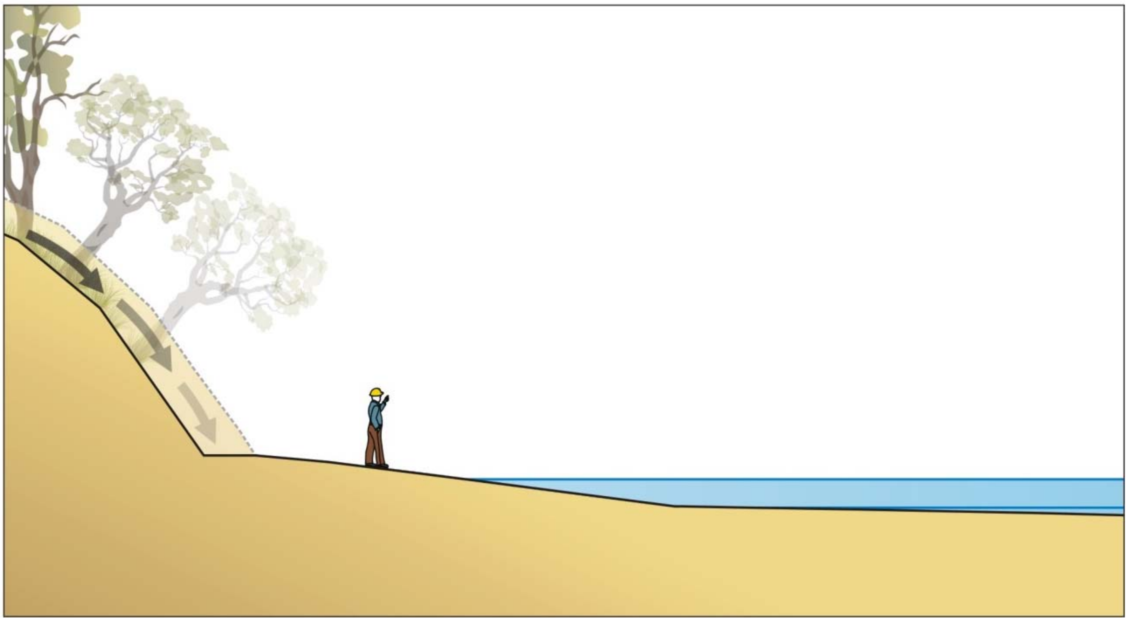


Figure 4.3: Hazard Pathway 3 – Creep of Foreshore Slope

5. Foreshore Management Prioritisation

A qualitative assessment of management prioritisation along the foreshore has been undertaken on the basis of:

- Potential for exposure to coastal processes causing hazard;
  - Erosion;
  - Recession;
  - Stormwater.
- Assessed Risk Levels for geotechnical hazards:
  - Regression of existing land-slip scarps;
  - Additional instability from ongoing erosion;
  - Ongoing creep of embankment slope surface.
- Current site condition and characteristics;
  - Embankment toe setback distance from water;
  - Asset setback distance from embankment crest;
  - Steepness of embankment slope.

The results of this prioritisation are provided in Table 5.1 and Figure 5.1.

Table 5.1: Qualitative Prioritisation of Management Works

Foreshore Zone	Description	Management Priority
1	South Western zone fronting the carpark and public toilet area opposite Jerry Bailey Road	Medium-High
2A	Southern central zone between the carpark and Renown Avenue	Medium
2B	Southern central zone between Renown Avenue and Mathews Street	Very High
3A	Northern central zone seaward of properties at 62-66 River Road	High
3B	Northern central zone seaward of properties at 62-66 River Road	Medium
4	North Eastern zone between the stormwater outlet at the River Road Reserve and boat ramp	Low

Figure 5.1: Qualitative Prioritisation of Management Works

6. Foreshore Management Options

A number of different management approaches are possible along the foreshore, with varying levels of impact/benefit to the environment and amenity of the site, costs, and implementation timeframes. These options include (but are not limited to):

- Do nothing;
- Monitoring with no active management works;
- Monitoring in combination with management works;
- Relocating existing sand located within the beach areas (beach scraping and dredging);
- Stabilisation of erosion scarps and revegetation;
- Protection structures (rock or geotextile bag revetment);
- Repairs and improvements to stormwater outlets on beach;
- Improvements to stormwater control across the beach;
- Nourishment of the beach with sand dredged from within the estuary shoals or excavated from the entrance area.

Based on the potential of each of the management options to address the identified hazard types and current risk, the suitability of the various management options for each different foreshore zone is presented in Table 6.1 and the recommended management options for each foreshore zone presented in Table 6.2. Note that the indicative costs estimates presented in Table 6.2 for beach nourishment and embankment protection works with rock or geotextile bags are later detailed in Sections 7.3.4 and 7.5.3.

It should be emphasised that the suggested management approach has been selected with a **focus on addressing the immediate coastal hazards in the short term** (as per the scope of WRL's project), **while also not compromising the ability to implement a longer term management plan for this section of the estuary at a later date**. It is recognised that alternative management approaches may provide longer term improvements in amenity of the foreshore (such as significant dredging of the estuary sand shoals and mass scale nourishment of the beach – one potential approach suggested by the Shoalhaven Heads Community Forum), however, these would require a range of additional investigations and funding beyond that presently available, and are therefore not well suited to addressing the immediate engineering risks.

Table 6.1: Suitability of Management Options for Foreshore Zones

	Foreshore Management Zone					
Management Option	**Zone 1**	**Zone 2A**	**Zone 2B**	**Zone 3A**	**Zone 3B**	**Zone 4**
Do nothing	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable
Monitor, without management works	Not suitable	Not suited	Not suitable	Not suitable	Some areas	May be suitable
Monitor, with management works	Suitable	Suitable	Not suitable	Not suitable	Suitable	Suitable
Beach scraping	Not required	Not required	With other management works	Not required	With other management works	With other management works
Stabilisation and revegetation of scarps	Suitable	Suitable	Not suitable	Not suitable	Suitable	Suitable
Protection revetment	May be option in future	May be option in future	Suitable	Suitable	Not required	Not required
Improvements to stormwater outlets	Not applicable	Not applicable	Suitable	Not applicable	Suitable	Suitable
Improvements to stormwater control across beach	Not applicable	Not applicable	Suitable	Not applicable	Suitable	Suitable
Nourishment of beach with sand from estuary	Suitable	Suitable	Suitable with additional protection [1]	Suitable with additional protection [1]	Suitable	Suitable

1. Based on an achievable/affordable modest extent of beach nourishment that could be applied in the short to medium term, as opposed to mass dredging of the estuary sand shoals and extensive nourishment of the whole foreshore profile.

Table 6.2: Recommended Foreshore Management Approach

Foreshore Management Zone	Suggested Management Approach
Zone 1	Now: Re-profile erosion scarp, stabilise erosion surface, revegetate, consider improved public access. Short Term Future: Nourish beach ($13,000-$30,000).
Zone 2A	Now: Remove/cover tree stumps, revegetate, monitor tree safety. Short Term Future: Nourish beach ($32,000-$73,000), monitor beach width/volume, monitor embankment (if impacted by erosion).
Zone 2B	Now: Remove debris, improve stormwater outlets, protect embankment toe with rock ($280,000) or geotextile bag ($580,000) revetment (additional costs for optional crest boardwalk), train stormwater across beach, monitor embankment and crest area. Short Term Future: Nourish beach ($16,000-$37,000), monitor beach width/volume.
Zone 3A	Now: Remove debris, improve stormwater outlets, upgrade existing protection to embankment toe with rock ($115,000) or geotextile bag ($240,000) revetment (additional costs for optional crest boardwalk), train stormwater across beach, monitor embankment. Short Term Future: Nourish beach ($7,000-$15,000), monitor beach width/volume.
Zone 3B	Now: Re-profile erosion scarp, stabilise erosion surface, revegetate, consider improved access. Short Term Future: Nourish beach ($24,000-$54,000).
Zone 4	Short Term Future: stabilise erosion scarps, revegetate, nourish opportunistically ($8,000-$19,000).

7. Concept Design of Foreshore Management Works

7.1 Overview

As outlined in Section 1.3 and WRL's scope for this project, the development of the concept management design addresses only the most critical areas of foreshore (high priority) which have been identified to include Zones 2B and 3A (see Table 5.1). The concept design presented for these areas includes consideration of:

- Embankment toe protection works (rock or geotextile bag revetment);
- Improvements to stormwater drainage across beach;
- Moderate beach nourishment.

A detailed analysis of environmental conditions and design details has been undertaken, with the results presented in Appendices B, C and D. Sections 7.1 to 7.5 of the report provide a summary of information for the concept design of management works which are also captured in Figure 7.1 to Figure 7.10.

Recommended foreshore management works for Zone 1 (medium-high priority) are also briefly outlined in Section 7.6.

7.2 Summary of Environmental Design Conditions

Environmental design conditions for the embankment toe protection works include the condition of Shoalhaven Heads entrance, local wind conditions and ocean wave and water level conditions. If Shoalhaven Heads is closed, the site is only exposed to short period, local wind waves. However, if the entrance is opened by a flood, the site may exposed to diffracted, long period, ocean swell waves for a period following the flood. A detailed analysis of environmental design conditions is presented in Appendix B. Three different entrance configurations have been explored in the concept design of embankment toe protection works as follows:

- Shoalhaven Heads closed;
- Shoalhaven Heads having a small opening; and
- Shoalhaven Heads having a large opening.

The design wave and water level conditions at the protection works affect the hydraulic performance (wave runup and overtopping) and stability of the structure, which in turn have a direct effect on the capital and maintenance costs protection works. After considering the three different potential entrance configurations and the resulting exposure of the site, through discussions with Council staff it was decided that the concept protection works would be designed for a small opening of the Shoalhaven Heads entrance (as has occurred during openings of recent decades).

7.3 Evaluation of Embankment Toe Protection Works

7.3.1 Toe Protection Armour Sizing

The hydraulic stability of sand-filled geotextile containers and rock boulders placed as a protection revetment to the embankment toe was assessed for each entrance condition.

The stability of two different standard sand-filled geotextile container sizes (0.75 m^3 and 2.5 m^3) was considered for both single and double layer arrangements. For the double layer arrangements, the design significant wave height (H_S) at the coastal protection works was compared against the H_S initiating damage in standard geotextile container guidelines (Coghlan et al. 2009). For the 0.75 m^3, single layer arrangement, the design $H_{0.1\%}$ at the coastal protection works was compared against the monochromatic wave height causing failure as noted by Coghlan et al. (2009). For the 2.5 m^3, single layer arrangement, the design $H_{0.1\%}$ at the coastal protection works was compared against the monochromatic wave height causing failure in Appendix C (previously unpublished data). For the geotextile container groyne arrangements, the design $H_{0.1\%}$ at the coastal protection works was compared against the monochromatic wave height initiating damage as noted by Carley et al. (2011). The behaviour of geotextile containers subject to lateral velocities is unknown. Therefore, WRL did not assess the hydraulic stability under freshwater flood flow velocities.

The stability of two different rock types, basalt (density ≈ 2650 kg/m^3) and sandstone (density ≈ 2300 kg/m^3) was considered in both seawall and groyne configurations, composed of two layers of graded primary armour stones overlying another two layers of graded secondary armour stones. The rock armour sizing to withstand wave attack for the seawall configuration was undertaken using several different empirical methods as detailed in CIRIA (2007): Hudson (SPM, 1977), Hudson (SPM, 1984), Van der Meer (deep water) and Van der Meer (shallow water). Armour sizing for the groyne configuration was undertaken using the methods of Hudson (SPM, 1977) and Hudson (SPM, 1984) only. The results of this analysis are presented in detail in Appendix B for structure slopes of 1V:1.5H. The rock masses to withstand wave attack adopted for each configuration have also been assessed for stability under the 5% AEP freshwater flood velocity using the stone blanket stability design equation from USACE (1994), reproduced in Equation VI-5-134 of Part VI CEM (2006). For each configuration, the armour mass required to withstand wave attack was higher than the mass required to withstand flood velocities.

A summary of the hydraulic stability assessment for sand-filled geotextile containers and rock in both seawall and groyne configurations is presented in Table 7.1 (for Shoalhaven Heads entrance closed or only a small opening) and Table 7.2 (large entrance opening). The geometry for the entrance closed and small entrance opening scenarios have been included on a single table since the design wave heights are almost identical (despite large differences in design wave period).

Table 7.1: Hydraulic Stability of Rock and Geotextile Bag Armouring Options (Entrance Closed or Small Opening)

Structure Type	Construction Material	Geometry	Stability
Seawall	Sand-Filled Geotextile Container	1V:1.5H Slope, Single Layer, 0.75 m^3 Units	Marginal
	Sand-Filled Geotextile Container	1V:1.5H Slope, Single Layer, 2.5 m^3 Units	Adequate
	Sand-Filled Geotextile Container	1V:1.5H Slope, Double Layer, 0.75 m^3 Units	Adequate
	Sand-Filled Geotextile Container	1V:1.5H Slope, Double Layer, 2.5 m^3 Units	Adequate
	Basalt Rock (2,650 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 150 kg	Adequate
	Sandstone Rock (2,300 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 250 kg	Adequate
Groyne	Sand-Filled Geotextile Container	1V:1.5H Side Slopes, 0.75 m^3 Units	Adequate
	Sand-Filled Geotextile Container	1V:1.5H Side Slopes, 2.5 m^3 Units	Adequate
	Basalt Rock (2,650 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 150 kg	Adequate
	Sandstone Rock (2,300 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 250 kg	Adequate

Table 7.2: Hydraulic Stability of Rock or Geotextile Bag Armouring Options (Large Entrance Opening)

Structure Type	Construction Material	Geometry	Stability
Seawall	Sand-Filled Geotextile Container	1V:1.5H Slope, Single Layer, 0.75 m^3 Units	Unsuitable
	Sand-Filled Geotextile Container	1V:1.5H Slope, Single Layer, 2.5 m^3 Units	Unsuitable
	Sand-Filled Geotextile Container	1V:1.5H Slope, Double Layer, 0.75 m^3 Units	Marginal
	Sand-Filled Geotextile Container	1V:1.5H Slope, Double Layer, 2.5 m^3 Units	Adequate
	Basalt Rock (2,650 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 750 kg	Adequate
	Sandstone Rock (2,300 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 1,300 kg	Adequate
Groyne	Sand-Filled Geotextile Container	1V:1.5H Side Slopes, 0.75 m^3 Units	Unsuitable
	Sand-Filled Geotextile Container	1V:1.5H Side Slopes, 2.5 m^3 Units	Marginal
	Basalt Rock (2,650 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 750 kg	Adequate
	Sandstone Rock (2,300 kg/m^3)	1V:1.5H Slope, Double Layer, M_{50} = 1,300 kg	Adequate

7.3.2 Design Crest Elevation

An empirical analysis has been undertaken to examine wave runup and overtopping under the assessed design conditions, to estimate a suitable design crest level for the embankment toe protection works. While empirical estimates of overtopping for coastal structures have improved significantly over the past decade, the available methods are still only useable to provide order of magnitude estimates or for relative comparison purposes. The state-of-the-art empirical technique for estimating overtopping is the EurOtop (2016) "Overtopping Manual". However, where more precise estimates are required, site specific physical modelling is typically undertaken.

The Overtopping Manual provides equations for runup (Equation 6.2 in EurOtop, 2016) and overtopping (Equations 6.6 and 6.7 in EurOtop, 2016) calculations on simplified structures. These methods were used to estimate theoretical runup levels and average overtopping rates for various potential crest level options (2.5 to 5.0 AHD) for each entrance condition. The results of this analysis are shown in Table 7.3 with markedly different hydraulic conditions for each entrance condition. Wave runup and overtopping from wind waves is shown to be negligible. Wave runup from swell waves (small and large entrance opening) exceeds almost all crest levels examined for 5% AEP conditions and so the proposed seawall will be an overtopped structure (under design conditions). Acceptable overtopping rates are therefore required to be established.

Table 7.3: Comparison of Estimated Relative Runup Levels and Overtopping Rates for a range of Crest Levels for three Entrance Conditions (for 5% AEP event)

Parameter	Crest Level (m AHD)	Entrance Condition		
		Closed	Small Opening*	Large Opening*
2% Runup, $R_{U2\%}$ (m AHD)		2.7	4.6	6.7
Mean Wave Overtopping Rate for Crest Elevations (L/s/m)	2.5	0.3	140.1	430.5
	3.0	<0.1	43.8	221.0
	3.5	<0.1	10.3	96.7
	4.0	0.0	2.0	38.0
	4.5	0.0	0.3	13.7
	5.0	0.0	0.1	4.6

*EurOtop (2016) recommends that wave setup be excluded from the input water levels as its empirical equations are based on physical model test results which implicitly reproduced wave setup against the test structures. However, WRL has included wave setup in the input water levels for the small and large entrance opening conditions in the inner Shoalhaven Heads bay as this super-elevation is due to wave breaking outside the entrance rather than directly against the seawall.

Since the purpose of the coastal protection works is primarily to prevent ongoing erosion to the River Road foreshore embankment, an acceptable design overtopping rate was estimated which would not result in crest armour damage or erosion of the natural embankment above the armouring crest. On this basis, and assuming that the embankment above the seawall crest will be revegetated shortly following construction, WRL has adopted an acceptable design overtopping rate of 5-10 L/s/m (tolerable rate for grass covered slope; H_{m0} < 1 m, EurOtop, 2016). With a small entrance opening as the adopted design entrance condition, a crest elevation of 4.0 m AHD is suggested for conceptual design. This would see the rock toe protection extending approximately 1/2 of the height of the existing embankment slope with natural vegetation protecting and reinforcing the upper sections.

7.3.3 Design Scour Level

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 (open coast) beach area. 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 the River Road, for seawalls constructed on the NSW Roads and Maritime Services' 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).

Since undertaking detailed erosion modelling at the toe of the proposed seawall was outside the scope of works, WRL adopted a design scour level of -1 m AHD. This elevation determines the required penetration of the structure to prevent undermining. However, it does not determine the maximum depth limited breaking wave height that can reach the structure as this is dictated by the assumed sand bar (flood tide delta) level of 0 m AHD. The justifications for the -1 m AHD scour level are:

- The less frequent potential for exposure to ocean swells (due to entrance opening regime);
- Consistency with engineering 'rule of thumb' for NSW; and

- It is much lower than many historical structures in NSW.

Should monitoring indicate that continued toe erosion or channel migration are occurring, additional scour protection could be provided in the form of:

- Beach nourishment and/or scraping;
- Additional rock armour.

7.3.4 Embankment Toe Protection Works – Concept Design Details

From the assessment summarised in Sections 7.3.1 to 7.3.3 (and detailed in Appendices B-D), the concept designs for embankment toe protection works are shown in Figure 7.1 and Figure 7.2. It is proposed that the embankment toe protection would be constructed as new works through section 2B of the foreshore (~170 m of armouring), and would replace the existing un-certified rock wall in Section 3A of the foreshore (~70 m of armouring).

For the purpose of estimating costs of the works, prices for supply and delivery of basalt rock armour from Schmidt Quarries were obtained, as well as prices for supply of geotextile bags from Geofabrics Australasia and fill sand from Cleary Bros quarry at Gerroa:

- Basalt primary armour: $50/tonne;
- Basalt secondary armour: $37/tonne;
- Vandal deterrent 2.5 m^3 geotextile bags: $307 each;
- Vandal deterrent 0.75 m^3 geotextile bags: $123 each;
- Fill sand for bags: $40/tonne.

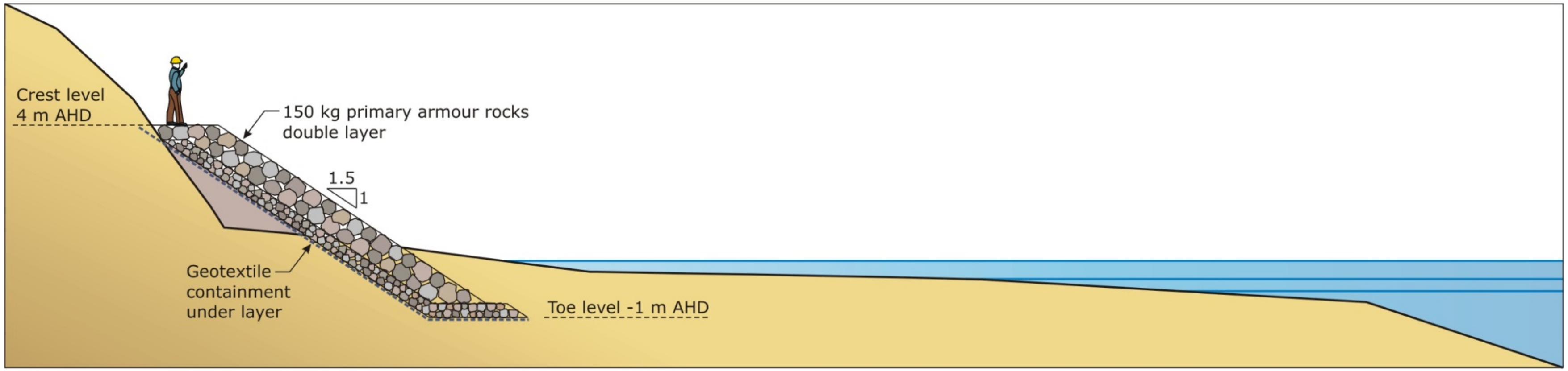
Costs for supply of other construction materials and for construction of the armouring were estimated on the basis of construction costing handbooks and experience from similar previous projects.

Overall cost estimates for constructing toe protection works are estimated to be:

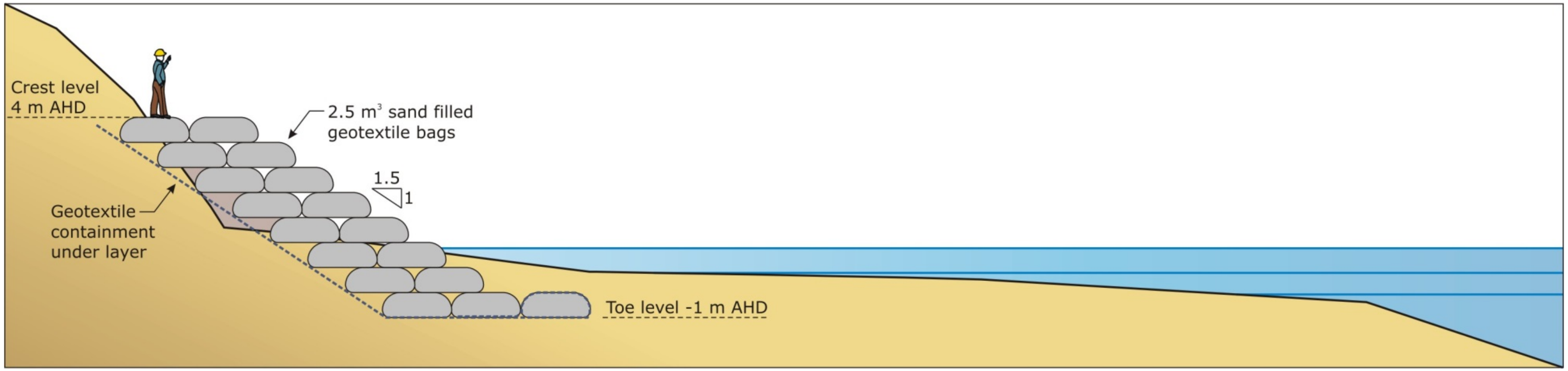
- Rock armouring (150 kg primary armour) ~$395,000 (~$1,650 per m of shoreline);
- Geotextile bag armouring (2.5 m^3 bags) ~$820,000 (~$3,380 per m of shoreline).

The costs of geotextile bag toe protection works could be significantly reduced if a single layer structure was used, however, this would be unconventional when compared to applications at other locations in NSW. A single layer design offers significantly less redundancy in the case that the structure suffers some storm damage.

Optional works that would improve access and amenity of the site could include a boardwalk (~1800 mm wide) along the crest of the embankment toe protection, and could be combined with an observation platform. Indicative details for this arrangement are shown in Figure 7.4 to Figure 7.6.



Rock Armouring Option



Geotextile Bag Armouring Option

Figure 7.1: Cross Section Details for Concept Embankment Toe Protection Options

Figure 7.2: Concept Layout and Minimum Access Requirements

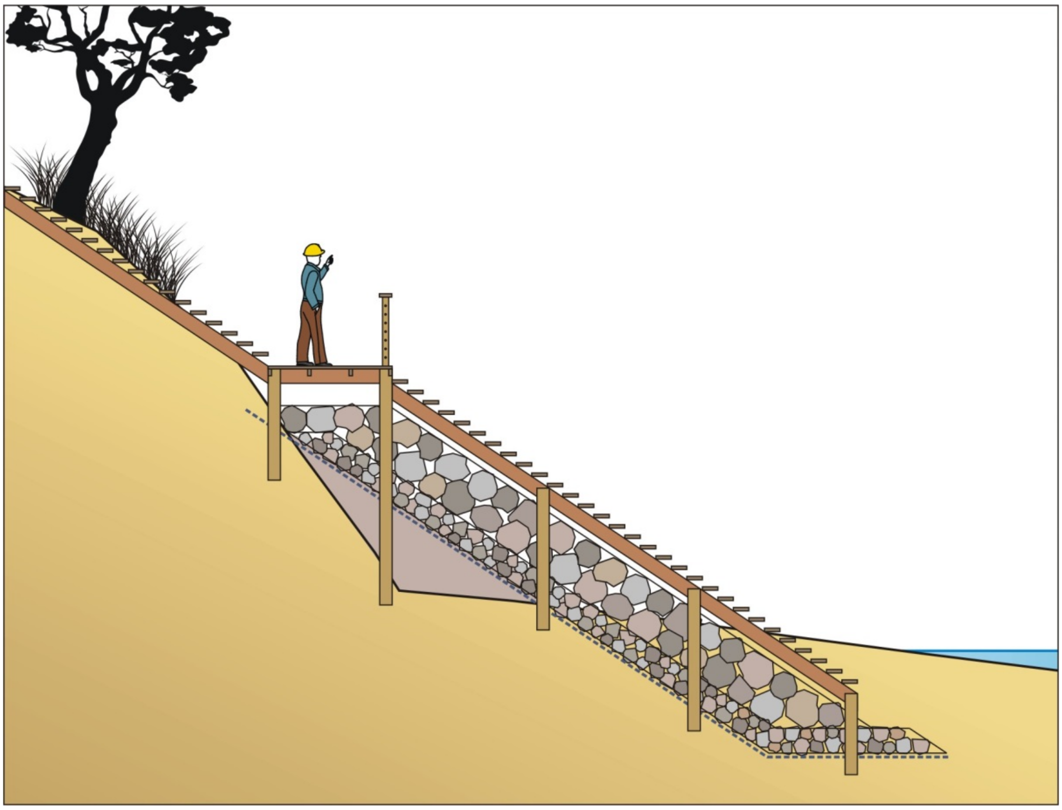


Figure 7.3: Indicative Arrangement of Access Stairs from River Road

Figure 7.4: Concept Layout with Optional Improved Amenity and Access Arrangements

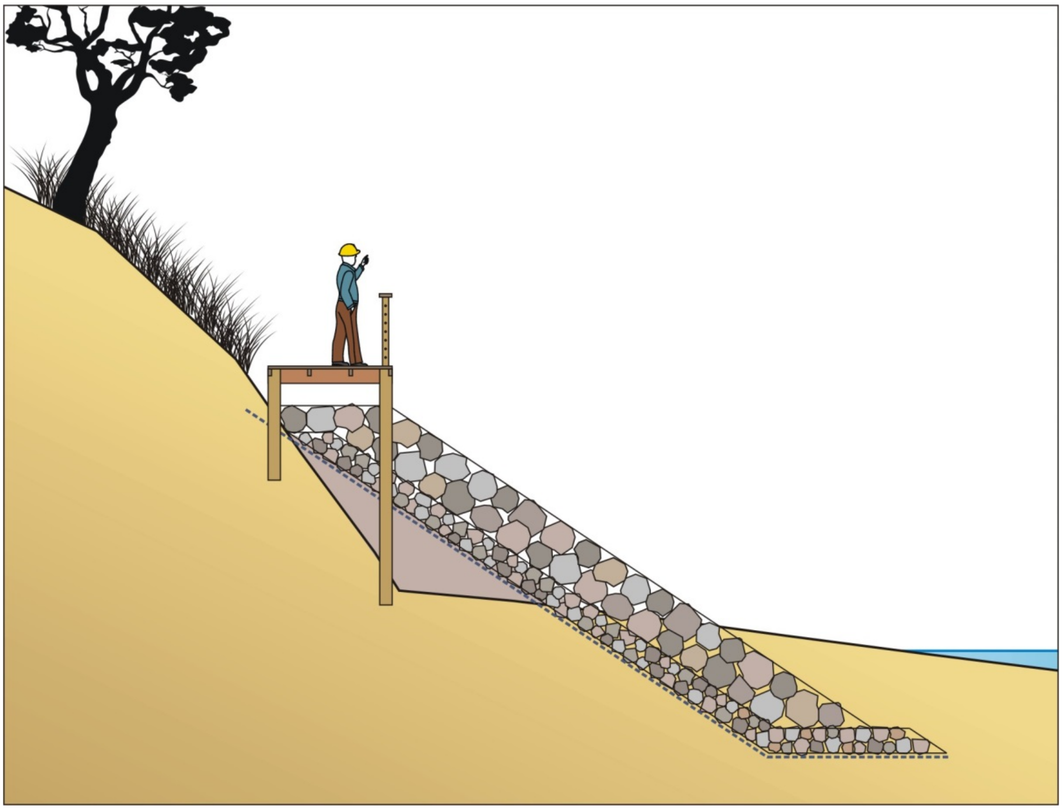


Figure 7.5: Indicative Section Through Boardwalk along Armour Crest

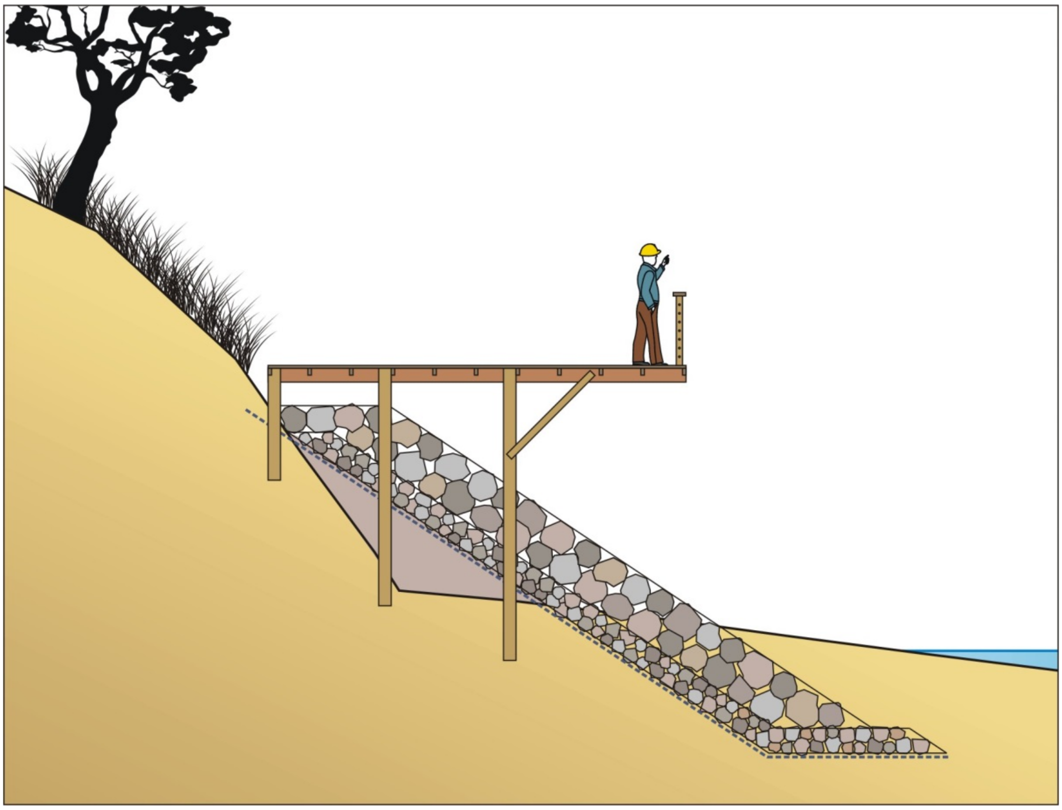


Figure 7.6: Indicative Section Through Observation Platform on Armour Crest

7.4 Stormwater Drainage Concept Improvement Works

7.4.1 Stormwater Outlet Improvement Works

As presented in the results of the site inspection (Section 2.2), there are a number of stormwater outlets that presently discharge stormwater at the back of the beach in foreshore zones 2B and 3B. Several of the outlets are in a relatively poor state and improvements to the outlet works are recommended as part of the foreshore management activities. At present the stormwater pipes diminish the visual amenity of the site primarily due to the ad-hoc nature of their implementation and their condition. As reported in Section 3.3, the stormwater flows across the beach also appear to be resulting in the removal of sediment from the beach and exacerbating beach recession. Ideally the stormwater drainage would be re-routed to an alternative discharge point so as to minimise long term impacts on the beach, however, this would require significant infrastructure works and may also not be technically feasible.

An alternative suggestion from community members was to continue the drains sub-surface under the beach and discharge the stormwater directly into the estuary channel. While this kind of management approach has been adopted at a range of metropolitan beaches on the NSW coast, it is not advised for the River Road foreshore for a number of reasons including:

- The transient/mobile nature of the estuary foreshore and beach sand would mean that foundations for pipework may need to be extensive;
- Additional recession of the beach would potentially result in exposure of and damage to the stormwater pipes;
- The pipes would become a navigation and safety hazard in the channel;
- Maintenance of submerged pipes to remove marine growth would be an expensive and difficult task;
- The pipes could become blocked if the foreshore accreted or the navigation channel migrated.

In the short term the proposed treatment for each of the drains includes the following works during construction of the embankment toe protection rock armouring:

- Terminating the outlet pipes to end flush with the alignment of the rock armour protection or slightly seaward of it;
- Fitting a concrete headwall to the end of the pipes;
- Integrating the rock armouring around the headwall to create a uniform and tidy finish.

These works are shown graphically in Figure 7.7.

The stormwater outlet at chainage 490 m will require additional piping works to continue the drainage from the crest of the embankment to the toe area. This piping could be integrated within the underlayer or fill within the embankment armouring.

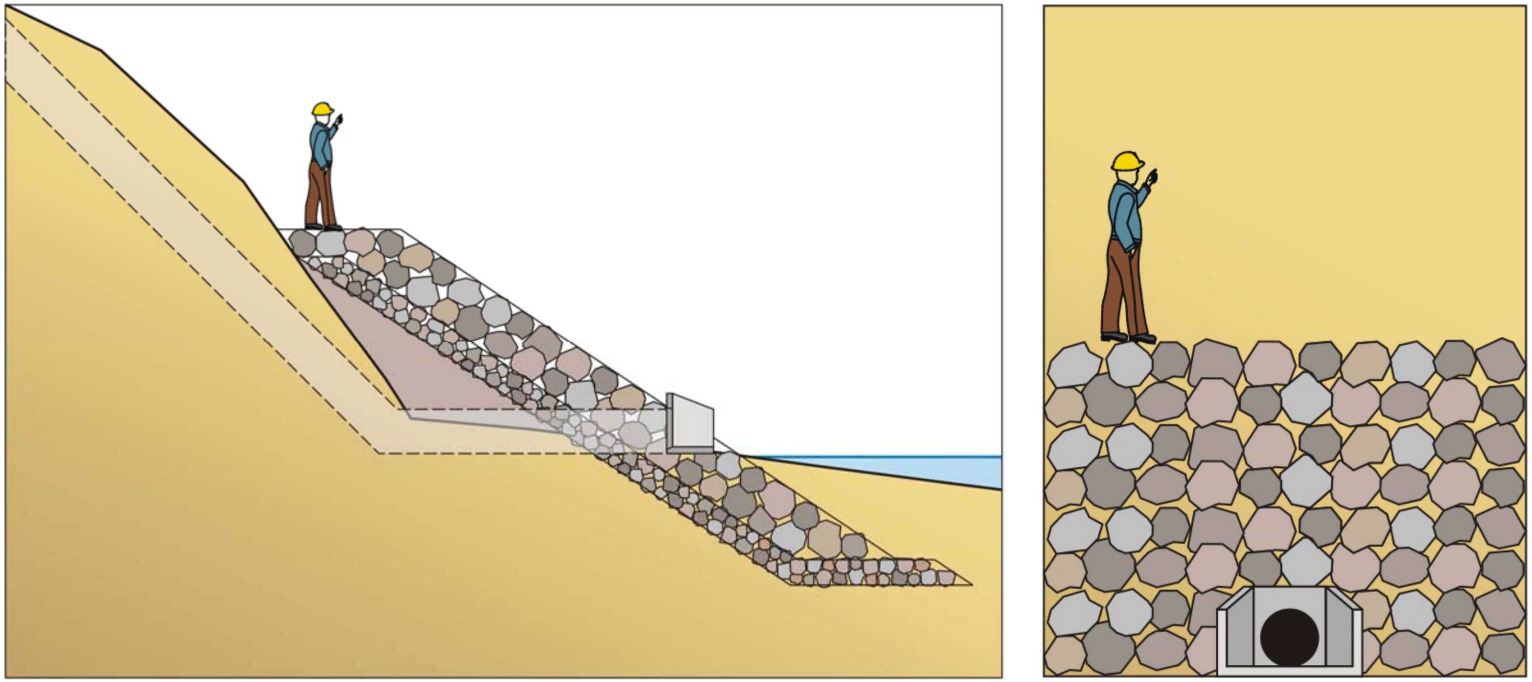


Figure 7.7: Treatment of Stormwater Outlets
(Left: Section View, Right: Elevation View)

7.4.2 Management of Stormwater Drainage Across Beach

Any approach to managing the stormwater discharges across the beach will impact on the amenity of the beach to a certain degree. If left completely untrained then stormwater flows from the rear of the beach result in uncontrolled scour of the beach during heavy rainfall, removing sediment from the beach system. To reduce the impacts of the stormwater flows across the beach while minimising impacts on the amenity of the beach, two potential approaches are suggested:

- Training of the stormwater flows and containment of scour using 0.75 m^3 sand filled geotextile bags; or
- Maintenance of the beach by an excavator to bring scoured sand back from the estuary channel and onto the beach.

Previous attempts have been made to "train" stormwater drainage across the beach using a row of 0.75 m^3 sand filled geotextile bags placed from the stormwater pipe outlet toward the low tide level on the beach. The majority of these bags are now in a relatively poor state with many damaged. While it appears that this system cannot be expected to have a long lifetime due to vandalism and wear and tear of the geotextile bags, it does allow for stormwater flows and the associated scour to be somewhat controlled without having a significant impact on beach amenity.

An alternative non-structural option for managing the impacts of stormwater scour on the beach would be for Council to trial beach scraping/rebuilding as a beach maintenance activity, whereby sand can be pulled back from the depositions on the estuary channel bank and redistributed onto the beach. It is unknown how often this kind of beach maintenance activity may be required, however, over the medium term it may be a more financially viable method to minimising sand losses and will also have the lowest impact on beach amenity.

7.5 Beach Nourishment Improvement Works

7.5.1 Overview of Beach Nourishment Management works

In recognising the importance of the recreational amenity of the beach to the local residents and tourism, it is recommended that a moderate extent of beach nourishment be undertaken in the short to medium term, following construction of the embankment protection works. The aim of the nourishment would be to establish a suitable width of usable beach adjacent to the proposed embankment protection works, and to assist in providing a healthier sand buffer for revegetation works along unprotected sections of the foreshore.

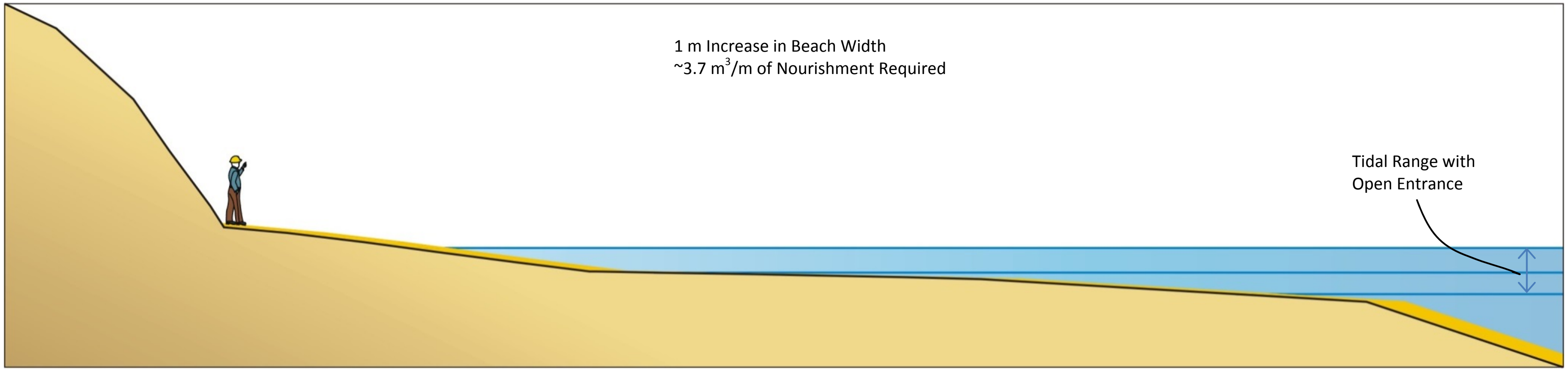
Nourishment sand for supplying the beach would likely come from one of two sources, being either the dry-notch maintenance activities at the Shoalhaven Heads entrance or dredging and maintenance of the navigation channel in the estuary immediately in front of the foreshore area. For the purpose of estimating costs of sand supply, it has been assumed that the sand would be dredged from the estuary as this is likely to have a higher cost rate compared with trucked sand from the dry-notch maintenance, and therefore provides conservative initial estimates.

Major nourishment of the entire foreshore profile (embankment, dry beach and submerged beach) would be required if nourishment was to be considered as a stand-alone management solution to reduce erosion risk along the higher risk sections of foreshore. For a range of reasons (cost, environmental, timing), this hasn't been considered as a viable short term management option as previously discussed.

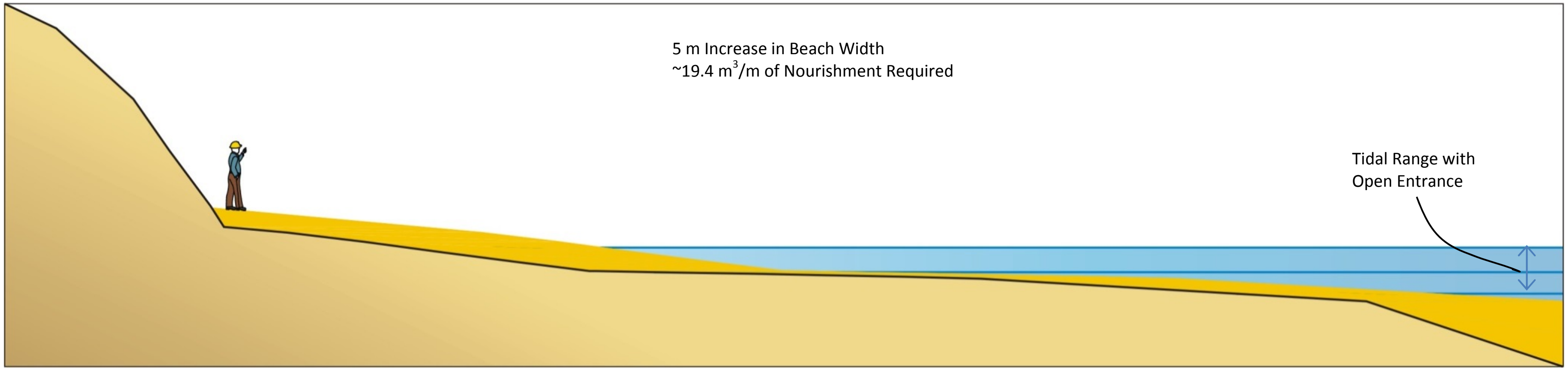
7.5.2 Nourishment Concept Design

It is recognised that nourishment of the beach may take place opportunistically or as budget becomes available, and so a range of nourishment options have been considered in this report so as to provide a more general idea of relative costs. As a minimum it is expected that the beach profile would need to be widened by 2 - 3 m along the area where embankment protection works are constructed (~240 m of beach length), so as to maintain an area of dry beach at most stages of the tide cycle. A significant improvement in beach amenity would require a 4-5 m widening of the beach along the entire foreshore (~1000 m of beach length). These two plausible nourishment scenarios have been considered as the lower and upper limits of nourishment that may be considered in the short to medium term for beach amenity purposes and are shown graphically in Figure 7.8.

Table 7.4 provides an indication of the nourishment sand volumes that would be required to increase beach width for various lengths of beach.



1 m Increase in Beach Width



5 m Increase in Beach Width

Figure 7.8: Lower and Upper Limit Beach Nourishment Profiles

Table 7.4: Estimated Beach Nourishment Volumes Required

Shoreline Length Nourished	Estimated Nourishment Volume Required (m^3) for Given Seaward Profile Movement				
Beach Width Gained	1 m	2 m	3 m	4 m	5 m
200	738	1494	2270	3066	3882
400	1476	2988	4540	6132	7764
600	2214	4482	6810	9198	11646
800	2952	5976	9080	12264	15528
1000	3690	7470	11350	15330	19410

Previously the sand shoals within the estuary have been used as a borrow source for sand supply, with maintenance of the navigation channel to the boat ramp providing some quantities of nourishment sand for the beach. Figure 7.9 shows an indicative area of sand shoal that could be dredged to provide sand for beach nourishment. This process would essentially result in a re-alignment of the navigation channel to the southeast of its present alignment, shifting the channel away from the beach. If the sand shoal within this area was lowered from ~0 m AHD to ~ -2 m AHD (similar level as the bed of the existing channel), approximately 15,000-20,000 m^3 of sand could be sourced. This would require a range of geotechnical and environmental investigations to assess suitability of the sediment and to understand the potential environmental impacts.

Figure 7.9: Indicative Entrance Channel Dredging Area

7.5.3 Estimated Beach Nourishment Costs

During the preparation of this report WRL sought cost estimates for dredging works at Shoalhaven Heads from a relatively local dredging company with suitable amphibious plant for extracting sand from the estuary shoals (Dredging Systems). Total costs for dredging ~15,000 m^3 of sand and pumping a distance of ~200 m to the beach were estimated to be ~$150,000, or an average rate of ~$10 per m^3 (including mobilisation, demobilisation and site management costs).

Including an allowance for initial environmental and geotechnical investigations, and costs to distribute and shape the sand on the beach, Figure 7.10 provides indicative costs for a range of beach nourishment options (based on sand dredged from the adjacent estuary shoals). From discussions with Council it is understood that sand could likely be excavated and trucked to the beach from the entrance dry notch for a lower unit cost, and so these indicative costs could be considered as slightly conservative.

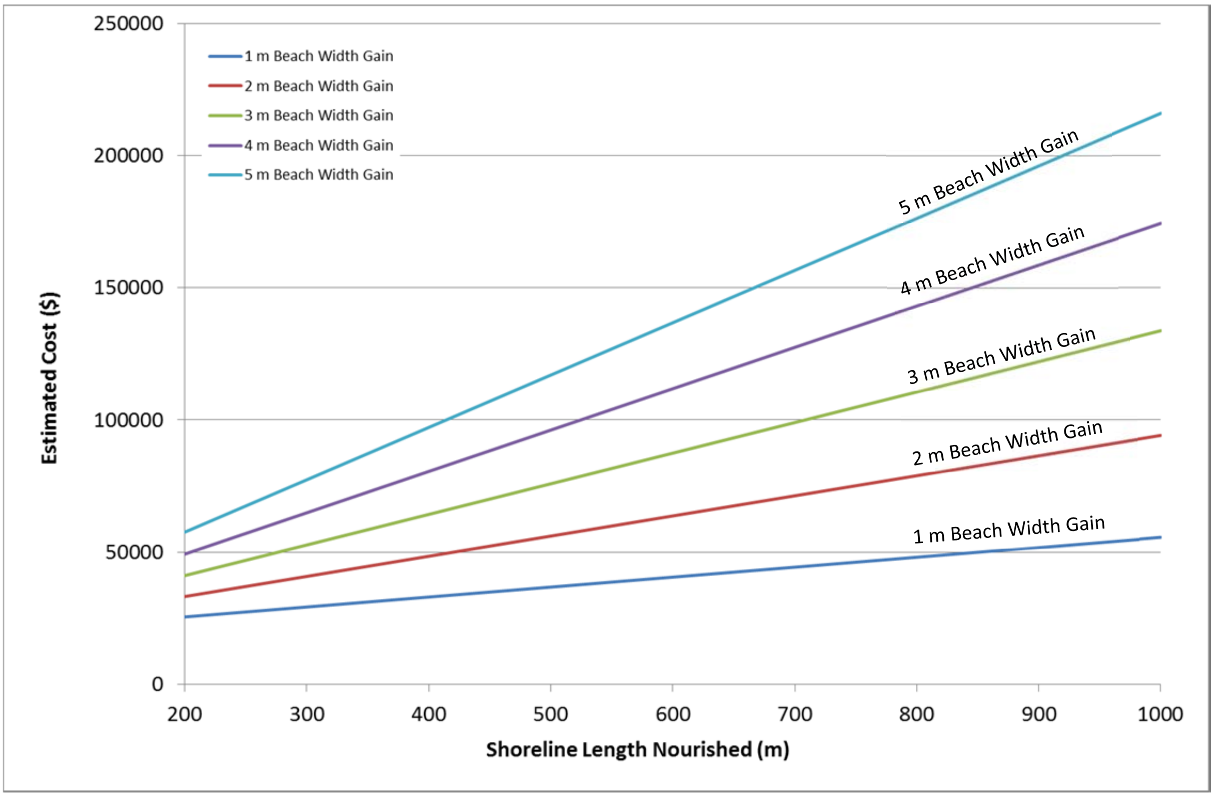


Figure 7.10: Indicative Costs for Beach Nourishment by Dredging from Estuary Shoals

7.6 Foreshore Management Works for Zone 1

As presented in the results of the site inspection (Section 2.2), there is a steep (near vertical) eroded bank landward of the sloping sandy beach in Zone 1. While management in this area is not as critical as in Zones 2B and 3A, WRL recommends that the eroded scarp in Zone 1 be re-profiled and stabilised. It is suggested that this stabilisation be provided through low-impact measures such as a temporary erosion matting (e.g. coir/jute mesh) and revegetation with typical dune species of grasses and shrubs.

If erosion progresses significantly under further exposure to high water levels and/or waves prior to vegetation becoming established, then a more engineered stabilisation approach may be required.

8. Debris and Waste

In general, there are localised areas of building waste, cut trees and other debris at a number of locations along the beach (in particular within foreshore zone 2B). Some of the waste hardboard has been identified as an asbestos containing material (ACM).

Prior to initiation of foreshore management works it is advised that specialist contractors be engaged to remove as much of the debris from the beach as possible, without undue disturbance to debris presently contained within the embankment. The remainder of the debris within the embankment should be properly contained below the rock armouring using a layer of suitably specified geotextile. This may require particular detailing during the design-for-construction of the management works, and a specialist asbestos consultant may be required during construction of some sections of the protection works to monitor and certify.

9. Summary

This project has investigated the current condition of the foreshore area along River Road at Shoalhaven Heads, and established recommended management actions to address immediate erosion concerns. The site has been impacted by recession/erosion of the beach over recent years during large storm events, with the June 2016 storm resulting in significant erosion of the beach, slipping of the foreshore embankment and loss of mature trees and vegetation.

An inspection of the site was undertaken to consider the coastal and geotechnical engineering risks, and the results used to prioritise management actions along the foreshore. Approximately 240 m of foreshore was identified as having a level of risk from geotechnical hazards which requires implementation of remediation works in the immediate short term. Primarily this risk stems from an over-steep foreshore embankment profile which has the potential for ongoing localised land slips that would threaten road, electrical and stormwater infrastructure located immediately landward. Ongoing erosion of the beach and embankment toe during storm and high water level events will exacerbate these risks.

A concept design for an embankment toe protection structure has been developed for this area, with options for rock or geotextile bag armouring. It is envisaged that this structure would be implemented in combination with medium-term management works to nourish the beach and revegetate the back-beach areas along the complete 1050 m stretch of foreshore to improve and restore the amenity of the site.

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Appendix A: Consultation Meetings

During this assessment and report preparation there were several meetings with WRL, Council staff, community representatives and the broader community, including:

- Informal meeting/discussion with community representatives at Shoalhaven Heads Hotel immediately following site inspections (21/09/2016);
- Meeting between Matt Blacka (WRL) and Isabelle Ghetti/Ray Massie (SCC) (29/11/2016);
- Public information session/presentation at the Shoalhaven Heads Bowling Club (7/12/2016); and
- Public "drop in" day discussions at the Shoalhaven Heads Community Centre (9/04/2017).

Appendix B: Revetment Design Information and Environmental Conditions

B.1 General

Design parameters for the coastal protection works include the condition of Shoalhaven Heads entrance, local wind conditions, ocean wave and water level conditions and geotechnical conditions. If Shoalhaven Heads is closed, the site is only exposed to short period, local wind waves. However, if the entrance is opened by a flood, the site may exposed to diffracted, long period, swell waves following the fresh water flood. While ambient wave energy will ultimately narrow and close the entrance over time, the Shoalhaven Heads foreshore is most vulnerable to oceanic swell wave attack immediately following a flood which opens or widens the entrance. The geotechnical conditions at the site determine the adequacy of existing foundation conditions for the proposed design. The design wave and water level conditions at the coastal protection works 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 assumptions inherent in the preliminary coastal protection works design are discussed in the following sub-sections.

B.2 Entrance Conditions

Three different entrance conditions have been considered in the preliminary design as follows:

- Shoalhaven Heads closed (Figure B.1);
- Shoalhaven Heads small opening (Figure B.2); and
- Shoalhaven Heads large opening (Figure B.3 and B.4).

The small opening has been defined as an entrance width of 160 m at an elevation of 0 m AHD. This is approximately the widest that the entrance has been since it opened in July 2013 after being closed for approximately 14 years.

The large opening has been defined as an entrance width of 600 m at an elevation of 0 m AHD. This is approximately equivalent to the widest recorded entrance condition after a flood in August 1974.

Each entrance condition was considered as a scenario with associated wave conditions at the proposed coastal protection works. It was outside the scope of works to develop average recurrence intervals (ARIs) for each of the three entrance widths considered.

The wave conditions, required protection works geometry and capital costs increase with the width of Shoalhaven Heads entrance.

Figure B.1: Entrance Closed, 14 September 2011 (Source: NearMap)

Figure B.2: Small Entrance Opening, 5 July 2013 (Source: NearMap)

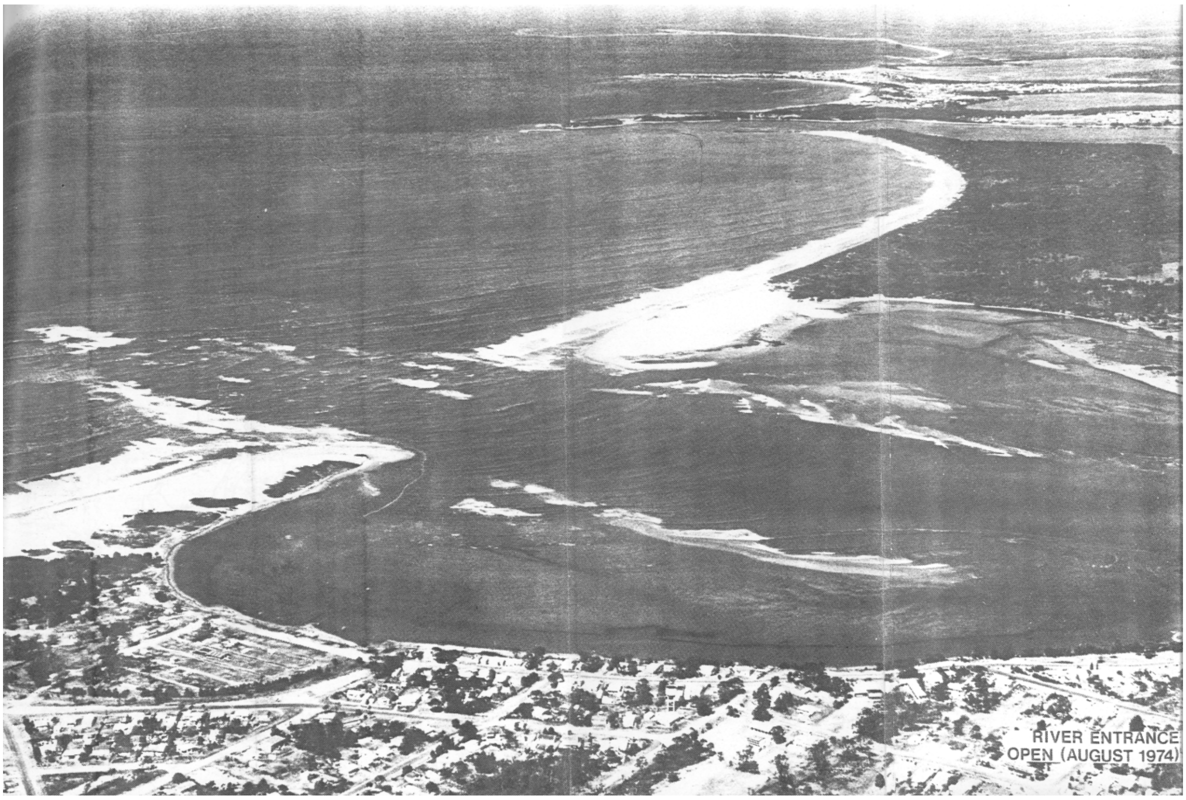

Figure B.3: Large Entrance Opening, August 1974
(Source: Posford et al. 1977)

Figure B.4: Large Entrance Opening, 29 December 1974 (Source: Chafer, 1998)

B.3 Basis of Design

B.3.1 Design Working Life

Establishing the design working life of the coastal protection works is critical for determination of subsequent design parameters. Following preliminary discussions with SCC, WRL has adopted a nominal design life of 10 years for the coastal protection works. The coastal protection works design discussed in the following sections has been prepared for this working life and would require revision if this planning horizon were modified (either increased or decreased).

B.3.2 Design Event and Encounter Probability

Having established the design life of the coastal protection works, it is prudent to select an appropriate level of design risk and assign design waves and water levels. An annual probability of exceedance for significant wave height and still water level forms the design "event" or design conditions. While there can be some technical/economic basis for risk and design event, 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 coastal protection works. 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.

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 6.1:

$$P = 1 - e^{(-N/ARI)} \qquad \text{Equation B.1}$$

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 6.1. Conversely, the appropriate ARI for failure can be derived by applying an acceptable encounter probability.

Australian Standard (AS) 4997 recommends design significant wave heights based on the function and design life of the structure as reproduced in Table B.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." (including the coastal protection works design at Shoalhaven Heads). 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. AS 4997 recommends that the design water levels accompanying these waves should not be below Mean High Water Springs (MHWS).

Table B.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

(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%.

WRL considers that the coastal protection works for Shoalhaven Heads may be regarded as either a structure "presenting a low degree of hazard to life or property" (Function Category 1) or a "normal" maritime structure (Function Category 2). The encounter probability included in AS 4997-2005 is 20% and 10%, respectively for these function categories. While a design working life of 10 years is not explicitly specified in AS 4997-2005, using Equation 6.1, the design event associated with this design life would be 45 or 90 year ARI, respectively. The guideline gives no further direction on the recommended design water level.

A further site-specific consideration is that the when Shoalhaven Heads is closed, it is not exposed to diffracted, long period, ocean swell waves (the determinant conditions). That is, the design events stipulated by Equation 6.1 and AS 4997-2005 are only directly applicable for structures located on the open coast or in deep water that are potentially exposed to wave attack throughout their whole working life. On this basis, WRL considers it reasonable to adopt a more frequent, 20 year ARI design event. If the entrance was to remain open for the full 10 year working life of the coastal protection works, this would represent an encounter probability of 39%. This probability would reduce in proportion to the time that Shoalhaven Heads entrance was closed over its working life.

A further consideration is that the maximum significant wave height that can reach the coastal protection works is a function of design water level and sand bar (flood tide delta) level due to depth limited wave conditions. WRL has selected the 20 year ARI event for deep water wave conditions (height, period and direction) and water level conditions (tide plus anomaly). Use of this combination of conditions is considered robust for preliminary design as the design water level is considerably higher than MHWS, resulting in a conservative depth limited significant wave height reaching the coastal protection works, even though reduced (i.e. not 45 or 90 year ARI) offshore design wave conditions were selected. WRL adopted a sand bar (flood tide delta) level of 0 m AHD, which was the lowest elevation maxima in any transect of available bathymetric datasets between the entrance and the inner foreshore.

WRL, in conjunction with OEH (formerly DECCW) have 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. When sufficient data is available, a full joint probability analysis is recommended to obtain curves of joint occurrence. Where such data is unavailable, marginal extremes should be combined assuming complete dependence of the variables (Shand et al. 2012). A full joint probability analysis at Shoalhaven Heads was outside the scope of this study and so complete dependence was assumed for 20 year ARI wave and water level conditions. It is acknowledged that, while the coincidence (phasing) of worst cases of these two variables may not occur simultaneously, there are insufficient studies to fully consider different phasing of each variable. This approach is also acknowledged to be conservative, however, well accepted (less conservative) alternative methodologies are not available.

Further discussion on the sensitivity of the design to changes in the assumed risk and design event are outlined in Section B.8.

B.4 Design Water Levels and Wave Conditions – Entrance Closed

B.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. The astronomical tidal planes, based on the Shoalhaven Heads tide gauge record, vary depending upon the condition of the entrance. Even when the Shoalhaven Heads entrance is closed, tidal action and anomalies still occur via the permanent Shoalhaven River entrance at Crookhaven Heads, although there is a reduction in the tidal range when the entrance is closed. Astronomical tidal planes based on the Shoalhaven Heads tide gauge record, are shown in Table B.2 from AWACS (1999) for both entrance closed and entrance open periods. This tide gauge is located adjacent to the River Road Reserve Boat Ramp in a water depth of approximately 1 m.

Table B.2: Astronomical Tidal Planes for Shoalhaven Heads (Source: AWACS, 1999)

Tide	Level (m AHD)	
	Entrance Closed (a)	**Entrance Open (b)**
High High Water Solstices Springs (HHWSS)	0.738	0.947
Mean High Water Springs (MHWS)	0.434	0.594
Mean High Water (MHW)	0.375	0.502
Mean High Water Neaps (MHWN)	0.315	0.410
Mean Sea Level (MSL)	0.067	0.090
Mean Low Water Neaps (MLWN)	-0.181	-0.231
Mean Low Water (MLW)	-0.240	-0.323
Mean Low Water Springs (MLWS)	-0.299	-0.415
Indian Spring Low Water (ISLW)	-0.516	-0.667

(a) Based on tidal measurements between 1 July 1994 and 30 June 1995 when the entrance was closed.

(b) Based on tidal measurements between 1 July 1991 to 30 June 1992 when the entrance was open.

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 top 10 recorded anomalies nearby at the Crookhaven Heads tidal gauge are reproduced in Table B.3 (MHL, 2010). This gauge is located approximately 1 km from the Shoalhaven River entrance.

Table B.3: Ranking of Highest Recorded Anomalies (1987-1990) for Crookhaven Heads (Source: MHL, 2010)

Rank (on Anomaly)	**Peak Anomaly** (m)	**Date**	**Anomaly ARI** (years)
1	0.85*	08/08/1998	17.4
2	0.83*	25/10/1999	8.7
3	0.46	28/06/1997	5.8
4	0.43	11/06/1991	4.4
5	0.41	27/06/1997	3.5
6	0.40	01/09/1996	2.9
7	0.39	12/06/1991	2.5
8	0.39	10/02/1992	2.2
9	0.39	26/09/1995	1.9
10	0.38	20/05/2002	1.7

* These anomalies are flood-affected events.

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 B.4. However, these levels are only applicable in the Newcastle - Sydney – Wollongong area and analysis of local tidal records on the NSW south coast is recommended.

Table B.4: Tidal Water Levels + Anomaly
Newcastle – Sydney – Wollongong
(source Watson and Lord, 2008 and DECCW, 2010)

Average Recurrence Interval ARI (year)	**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

The elevated water levels in Table B.4 can be supplemented with additional analyses for other tide gauges in southern NSW undertaken by MHL (2010) and BMT WBM (2009). 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 elevated water levels for Crookhaven Heads, Jervis Bay and Bermagui (from central estimates in Appendix B of MHL, 2010) and Batemans Bay BMT WBM (2009), in addition to Fort Denison, are reproduced in Table B.5. The default advice for elevated water levels at NSW sites south of Crowdy Head in "Coincidence of Catchment and Ocean Flooding Stage 2 – Recommendations and Guidance" (Smith et al. 2003) is also shown for reference.

Table B.5: Extreme Oceanic Water Levels for Southern NSW Tide Gauges

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)
South of Crowdy Head (Smith et al. 2013)			1.40		1.45
Fort Denison (Watson and Lord, 2008)	1.24	1.35	1.38	1.41	1.44
Crookhaven Heads (MHL, 2010)	1.21	1.27	1.28	1.29	1.31
Jervis Bay (MHL, 2010)	1.27	1.33	1.35	1.36	1.37
Batemans Bay (BMT WBM, 2009)		1.31	1.34	1.38	1.40
Bermagui (MHL, 2010)	1.21	1.27	1.28	1.29	1.30
Adopted for this study			1.35		1.40

The adopted extreme oceanic water level conditions (excluding wave setup) for the design are:

- 20 year ARI: 1.35 m AHD; and
- 100 year ARI: 1.40 m AHD.

These storm tide values are considered representative when the Shoalhaven Heads entrance is open. Since the astronomical tide contribution to elevated water levels at Shoalhaven Heads is reduced by approximately 0.15 m (Table B.2) when the entrance is closed, the adopted storm tide values for the entrance closed condition are:

- 20 year ARI: 1.20 m AHD; and
- 100 year ARI: 1.25 m AHD.

B.4.2 Flooding

While the 20 year ARI design water level accompanying the 20 year ARI wind-waves has been adopted as 1.20 m AHD, peak fresh water flood levels and velocities at Shoalhaven Heads are reproduced for reference in Table B.6. For each of these flood events, including 10 and 20 year ARI, modelled by Webb, McKeown and Associates (2008), the Shoalhaven Heads entrance was assumed to be closed at the start of the flood event and then assumed to scour out with the passage of floodwaters.

Table B.6: Shoalhaven River Peak Flood Levels and Velocities Location Shoalhaven Heads at Wharf Road (source Webb, McKeown & Associates, 2008)

Average Recurrence Interval ARI (year)	**Flood Level** (m AHD)	**Flood Velocity** (m/s)
10	2.5	1.2
20	2.7	1.6
50	2.9	2.0
100	3.3	2.2
200	3.6	
500	3.9	
Extreme	4.2	

Note: The numerical modelling for these peak flood levels and velocities assumed that the Shoalhaven Heads entrance was closed at the start of the flood event and then scoured out with the passage of floodwaters.

B.4.3 Design Wind Waves

When the Shoalhaven Heads entrance is closed, the greatest exposure to wave attack is from wind-waves generated upstream (from the SW). The wind conditions which generate wind waves 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) applicable to coastal engineering assessments are given for average recurrence intervals of 1 to 1,000 years. Site wind speeds ($V_{sit,\beta}$), are calculated according to Equation B.2 using multipliers for direction (M_d), terrain ($M_{z,cat}$), shielding (M_s) and topography (M_t).

$$V_{sit,\beta} = V_r M_d (M_{z,cat} M_s M_t) \qquad \text{Equation B.2}$$

The 20 year ARI regional wind gust speed (V_r) is 37 m/s (Shoalhaven Heads foreshore falls within Region A2). The direction multiplier (M_d) for the SW is 0.95. For Terrain Category 1 (enclosed, limited-sized water surfaces at serviceability and ultimate wind speeds), $M_{z,cat}$ is 1.12 at 10 m elevation (z). The shielding and topography multipliers were both 1.0. On this basis, WRL estimates that the 20 year ARI, 0.2 s wind gust speed is 39.4 m/s.

Wind waves generated by winds blowing along the Shoalhaven River are the result of sustained winds rather than extreme gusts. Equivalent sustained 30 minute 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 30 minute duration was selected based on the approach set out in Figure II-2-3 of the same document (USACE, 2006) which describes duration as a function of fetch and wind speed. The selected duration relates to the 2.5 km fetch to the SW of the Shoalhaven Heads foreshore. The 20 year ARI sustained (30 minute) wind speeds from the SW is estimated as 25.5 m/s.

The Simulating WAves Nearshore (SWAN) numerical wave model (Booij et al. 1999) was used to quantify the generation of local wind waves within the Shoalhaven River. SWAN (version 41.10) is a third-generation wave model that was developed at Delft University of Technology (2016). A 10 m resolution SWAN model was prepared to establish the 20 year ARI wind waves for the study area. Example model output for 20 year ARI wave conditions from the longest possible fetch (South-West by South) is presented in Figure B.5.

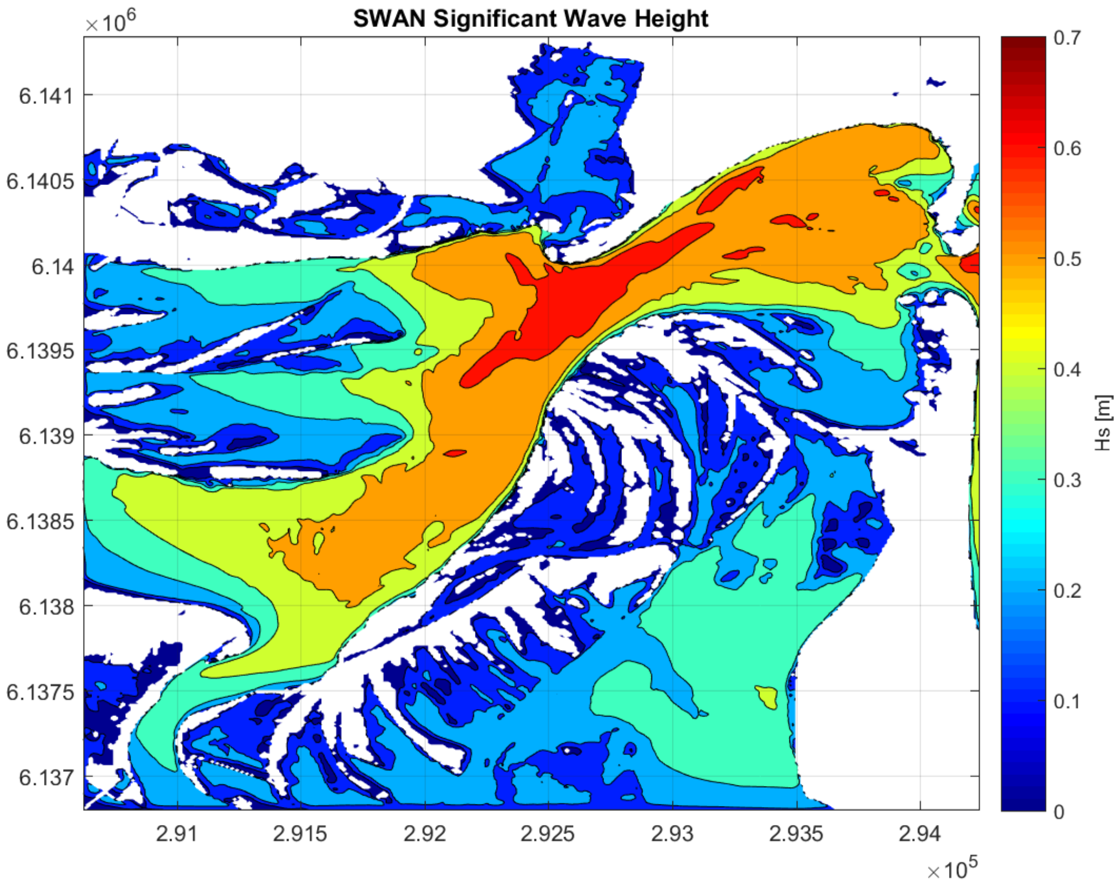

Figure B.5: Example SWAN Wind-Wave Model Output for Shoalhaven Heads - 20 year ARI – South-West by South (SWbS) Winds – 213.75°TN

The adopted 20 year ARI wave conditions when the Shoalhaven Heads entrance is closed are:

- Significant Wave Height (H_S): 0.67 m; and
- Peak Spectral Wave Period (T_P): 2.9 s.

B.4.4 Wave Setup

Wave setup associated with the 20 year ARI design wind waves (in conjunction with the 20 year ARI oceanic water level) was assessed by WRL using the Dally, Dean and Dalrymple (1984) two-dimensional surf zone model and found to be negligible.

B.4.5 Sea Level Rise

Mean sea level on the NSW coast is presently rising at between 1 and 4 mm/year (DECCW, 2010; Watson, 2011; Whitehead & Associates, 2014). Depending upon the scenario adopted, mean sea level is projected to increase by up to 0.9 m by 2100 by which time it would be rising at 13 mm/year (NCCOE, 2012).

Due to the relatively short initial design life of the coastal protection works, sea level rise does not need to be considered for this project, however, it would need to be considered if the project life is to be extended beyond approximately 20 years.

B.5 Design Water Levels and Wave Conditions – Small Entrance Opening

B.5.1 Storm Tide (Astronomical Tide + Anomaly)

For the entrance condition with a small opening, WRL adopted an extreme oceanic water level (excluding wave setup) of 1.35 m AHD as outlined in Section B.4.1.

B.5.2 Offshore Design Wave Conditions – Wave Height

A non-directional wave buoy operated offshore of Port Kembla (approximately 45 km NNE of Shoalhaven Heads) from 1974 to 2012 and was upgraded to measure wave direction in 2012. 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 design storm events for annual recurrence intervals of 1 to 100 years (Shand et al. 2010). The results from the study for the wave buoy at Port Kembla are tabulated for all wave directions in Table B.7.

Table B.7: Port Kembla - Extreme Offshore Wave Conditions (All Directions) (Source: Shand et al. 2010)

Average Recurrence Interval ARI (year)	**One Hour Exceedance H_S** (m)
1	5.4
10	7.1
20	7.6*
50	8.3
100	8.8

* Note that the estimated 20 year ARI value has been inferred by WRL for this study.

B.5.3 Offshore Design Wave Conditions – 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 Shoalhaven Heads where this analysis was undertaken was Botany Bay. Using the methodology outlined in (Shand et al. 2011), the coefficients determined for Botany Bay and the significant wave height from Port Kembla, the peak spectral wave period associated with the 20 year ARI offshore significant wave height was adopted as 12.2 s for all this study.

B.5.4 Offshore Design Wave Conditions – Wave Direction

Detailed wave refraction modelling has not been undertaken for this study. Such a study may better define waves at the seaward end of the surfzone offshore of Shoalhaven Heads, however, due to the depth limitation of waves inside the surf zone and greatest exposure to waves from the ESE, a design condition of full, unrefracted waves from the ESE has been assumed.

B.5.5 Design Wave Conditions at the Coastal Protection Works

The bathymetric profile coincident with the offshore design wave condition was extracted along the transect shown in Figure B.6. This profile is also plotted in Figure B.7 for reference.

Figure B.6: Bathymetry Transect for Determination of Design Wave Conditions at the Coastal Protection Works *(Aerial Photo 5 July 2013, Source: NearMap)*

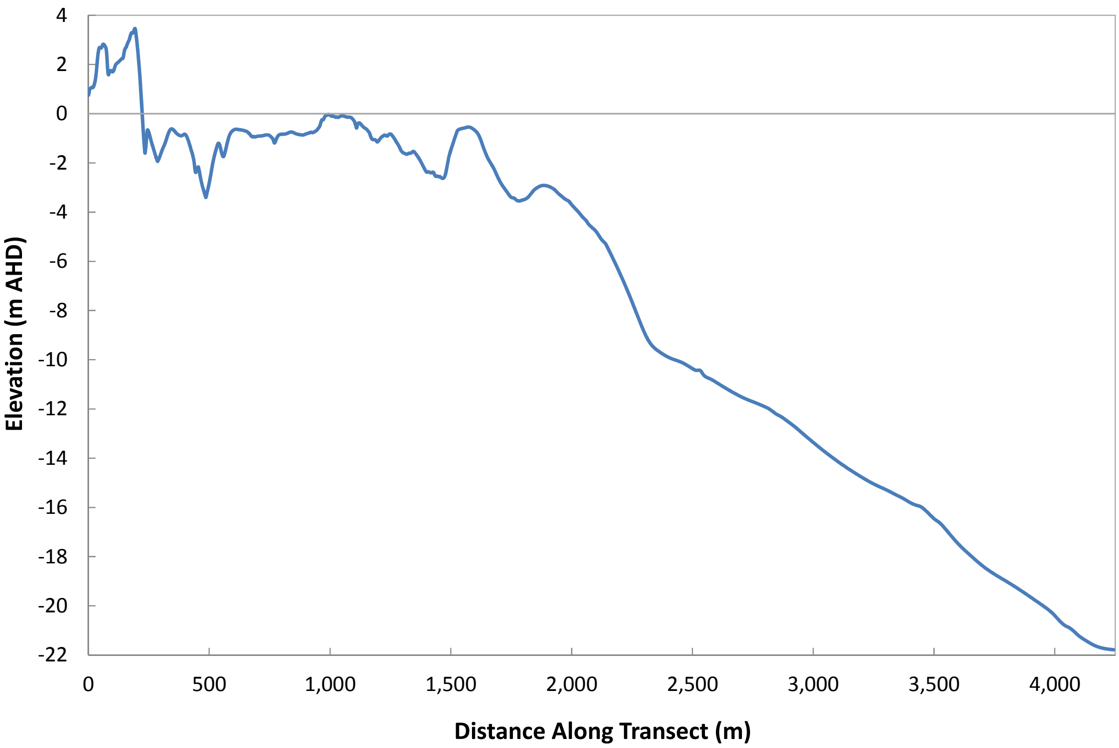

Figure B.7: Profile for Determination of Design Wave Conditions at the Coastal Protection Works

To derive design wave conditions at the coastal protection works when Shoalhaven Heads entrance is open, a series of desktop techniques were used to assess the dissipative processes of wave breaking and diffraction from the seaward end of the surfzone offshore, through the small entrance opening and into the bay:

- Use the Dally, Dean and Dalrymple (1984) surf zone model to estimate wave setup within the entrance;
- Estimate the depth-limited significant wave height within the entrance using the empirical technique of Goda (2007);
- Estimate the diffracted wave height at the inner foreshore using the irregular wave diffraction diagrams for waves passing through a structure gap developed by Goda (2000) and reproduced in Figure II-7-15 of Part II CEM (2006); and
- Estimate wave setup inside the bay at the coastal protection works.

Since the dominant dissipative processes for this entrance condition are diffraction and refraction, desktop methods were preferred to using the SWAN wave model whose diffraction approximation does not properly handle diffraction into harbours (Delft University of Technology, 2016).

Wave Setup Within the Entrance

Based on a bathymetry survey of the entrance undertaken on 8 September 2015 (the widest measured entrance condition since July 2013), the entrance cross-section was idealised as having a width of 160 m with an effective invert elevation of -1 m AHD (Figure B.8 and Figure B.9). While it is acknowledged that there were isolated spots surveyed in the entrance with elevations as low as -3 m AHD, the higher elevation of -1 m AHD is considered representative for considering diffraction across the full width of the entrance. To determine the wave setup within the entrance, H_{RMS} (m) corresponding to the adopted 20 year ARI wave conditions were first calculated according to CIRIA (2007) in Equation B.3.

$$H_{RMS} = 0.706 \times H_S \qquad \text{Equation B.3}$$

This wave height was applied as a boundary condition to the Dally, Dean and Dalrymple (1984) model. The 20 year ARI peak spectral wave period and storm tide water level were also applied. The wave setup at the -1 m AHD contour was estimated to be 0.6 m.

Figure B.8: Bathymetry Transect for Determination of Design Wave Conditions Within the Entrance *(Aerial Photo 5 July 2013, Source: NearMap)*

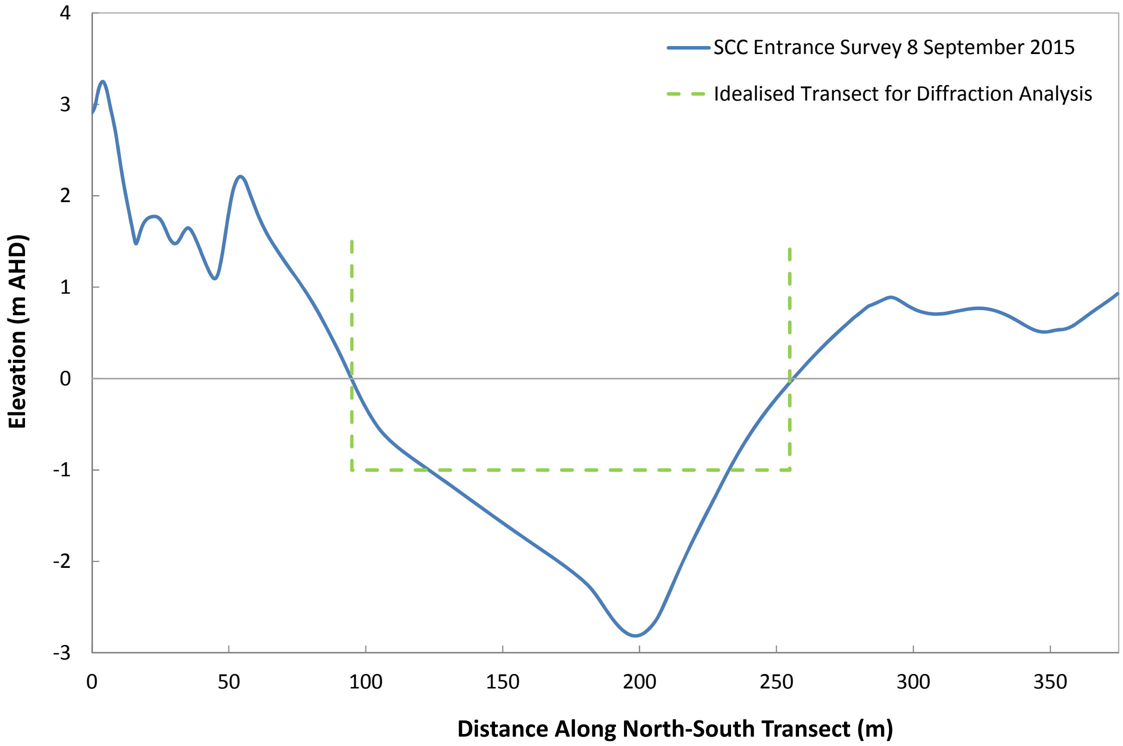

Figure B.9: Alongshore Entrance Profiles (Measured and Idealised) Looking Seaward for Wave Setup and Diffraction Analysis

Wave Height Within the Entrance

To establish the design depth limited H_S in the entrance, the 20 year ARI storm tide and wave setup water level conditions were combined. On this basis, the water depth at the -1 m AHD contour is 2.95 m. The breaker depth index (γ) is generally defined (Equation B.4) by the ratio of the breaker significant wave height (H_S) to the break point water depth (d_b). Note that wave setup has been included in all calculations involving d_b.

$$\xi_b = \frac{\tan\alpha}{\sqrt{H_b / L_0}} \qquad \text{Equation B.4}$$

where:
- α: bed slope seaward of seawall
- H_b: breaking wave height at the edge of the surf zone
- L_0: deep-water wave length

An empirical technique for estimating the breaker depth index was derived from laboratory experiments by Goda (2007) on slopes between 1V:9H and horizontal. These experiments indicated ratios of H_S/d_b = 0.51 to 0.60 (generally). The best estimate of the breaker depth index within the entrance (nearshore slope of 1V:120H) based on this technique is 0.56. For the assumed invert elevation of -1 m AHD, this results in a depth limited H_S of 1.66 m.

Diffracted Wave Height at the Inner Foreshore

To estimate the diffracted wave height at the inner foreshore, 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 B.10.

Figure B.10: Irregular Wave Diffraction Coefficients for Small Entrance Opening
(Aerial Photo 5 July 2013, Source: NearMap)

The best estimate of the diffraction coefficient based on this technique is 0.4; this results in an H_S at the inner foreshore of 0.66 m (note that this wave height statistic is not depth limited). The peak spectral wave period is assumed to be unchanged at 12.2 s.

Wave Setup at the Inner Foreshore

Estimating wave setup inside a waterway entrance is not readily possible using desktop techniques. Three options were canvassed regarding wave setup at the inner foreshore as indicated below:

- Historical measurements at the Shoalhaven Heads tide gauge;
- General guidance based on waterway classification; and
- Maximum wave setup on the exposed Seven Mile Beach.

As part of the *Shoalhaven River Estuary Data Compilation Study* (AWACS, 1999), major ocean storm events between 1 January 1991 and 31 December 1995 were analysed. The wave height, period and direction (direction understood to be estimated by an experienced observer) at the non-directional Port Kembla wave buoy were tabulated. Wave setup at the tide gauges at Shoalhaven Heads, Wharf Road and Crookhaven heads was recorded. To consider if freshwater flooding affected the wave setup measurements, the rainfall recorded at the Nowra RAN Air Station were also considered. A sub-set of this information, sorted by decreasing maximum wave setup measured at Shoalhaven Heads is reproduced in Table 4.8. Events where the 72 hour rainfall at Nowra RAN Air Station exceeded 12 mm have been excluded due to the influence of freshwater flooding component on water levels. Where possible, WRL has noted the condition of the entrance for each storm event based on notes in AWACS (1999) and Chafer (1998). Note that AWACS (1999) estimated the wave setup at each tide gauge by subtracting the actual water level from the predicted tide. It is acknowledged that this may result in an overestimation of wave setup if there was also a significant tidal anomaly (i.e. regional wind setup, barometric effects, long waves) coincident with each storm event. The maximum wave setup (within the limitations acknowledged above) measured at Shoalhaven Heads between 1991 and 1995 was 0.65 m (or 12.7% of the offshore H_S of 5.1 m).

Table B.8: Wave Climate and Wave Setup Measurements 1991-1995 (after AWACS, 1999)

Storm Date	Port Kembla Wave Climate			Entrance Condition	Wave Setup at Shoalhaven Heads Tide Gauge			
	Max H_S (m)	Mean T_{P1} (s)	Direction		Max (m)	Max (% of H_S)	RMS (m)	RMS (% of H_S)
12-15/4/94	5.1	10.4	SSE	Unknown	0.65	12.7	0.26	5.1
22-25/7/91	4.5	14.9	SSE	Open	0.34	7.6	0.14	3.1
15-16/7/92	4.2	10.2	SE	Open	0.28	6.7	0.12	2.9
12-14/11/92	4.3	10.8	SSE	Open	0.25	5.8	0.12	2.8
9-11/6/94	4.6	11.6	SSE	Unknown	0.26	5.7	0.16	3.5
13/06/1993	4.3	14.5	S	Unknown	0.22	5.1	0.09	2.1
30-31/3/93	4.5	11.5	SSE	Unknown	0.22	4.9	0.10	2.2
12-15/3/94	6.2	12.4	S	Unknown	0.29	4.7	0.15	2.4
8/04/1992	4.8	10.4	S	Open	0.13	2.7	0.06	1.3
4-5/5/1993	4.4	10.8	S	Unknown	0.10	2.3	0.05	1.1
20-21/10/94	4.5	9.4	SE	Closed	0.08	1.8	0.10	2.2
24-26/8/92	4.6	10.1	SSE	Open	0.08	1.7	0.05	1.1

WRL, in conjunction with OEH, have prepared general recommendations for establishing an ocean boundary for assessing estuary flooding depending on the waterway type Smith et al. (2013). According to this approach, for an untrained, wave dominated estuary or ICOLL (Ocean Boundary C), the recommended wave setup value is 10-15% (nominally 12%) of the offshore significant wave height.

Using a cross-section on Seven Mile Beach, immediately south of the entrance to Shoalhaven Heads, the offshore H_{RMS} was applied as a boundary condition to the Dally, Dean and Dalrymple (1984) model. The maximum wave setup on the open coast was estimated to be 1.0 m (13.5% of the 7.6 m offshore H_S). This value should be considered as an upper limit for maximum wave setup within the Shoalhaven Heads bay.

Considering the historical measurements of wave setup, the recommendations prepared for OEH based on waterway type and the upper limit estimate of wave setup on the open coast, WRL estimates that the maximum wave setup at the inner Shoalhaven Heads foreshore to be 0.9 m (12% of the 7.6 m offshore H_S). The resulting 20 year ARI total design water level (including wave setup) is estimated to be 2.25 m AHD.

B.5.6 Flooding

While it is acknowledged that the 20 year ARI total oceanic design water level (2.25 m AHD) is lower than the 20 year ARI peak flood level (2.7 m AHD), WRL considers that, for coastal protection works with a design life of 10 years, it is overly conservative to assume that a 20 year ARI ocean storm event would coincide with a 20 year ARI fresh water flood event. Were this to occur, it is unlikely that the wave climate at the inner Shoalhaven Heads foreshore would increase, since it is largely controlled by diffraction rather than depth limitations. Furthermore, according to the methodology of Herchenroder (1981; reproduced in Figure II-6-34 of Part II CEM, 2006); WRL estimates that design swell waves propagating across the Shoalhaven Heads inner bay against an opposing current of 1.6 m/s would only increase in height by approximately 5-10%. However, since wave energy is conserved in this interaction, there would be a corresponding reduction in wave period.

B.6 Design Water Levels and Wave Conditions – Large Entrance Opening

B.6.1 Storm Tide (Astronomical Tide + Anomaly) and Offshore Design Wave Conditions

For the entrance condition with a large opening, WRL adopted the same extreme oceanic water level conditions (excluding wave setup) and offshore design wave conditions as with the entrance with a small opening scenario.

B.7.2 Design Wave Conditions at the Coastal Protection Works

Wave Setup Within the Entrance

Based on the widest recorded entrance condition (August 1974), the entrance cross-section was idealised as having a width of 600 m with an effective invert elevation of -2 m AHD. This invert elevation is the same as that adopted in the PWD (1990a) *Lower Shoalhaven Flood Study*, albeit with a 400 m entrance prior to a flood event of at least 20 year ARI occurring. While it is acknowledged that the maximum depth in the entrance following the August 1974 flood has been reported as being between 10 and 20 m (Posford et al. 1977), the higher elevation of -2 m AHD is considered representative for considering diffraction across the full width of the entrance. Applying H_{RMS} as a boundary condition to the Dally, Dean and Dalrymple (1984) model, the wave setup at the -2 AHD contour was estimated to be 0.5 m.

Wave Height Within the Entrance

On the basis of a water depth at the -2 m AHD contour of 3.85 m and a breaker depth index within the entrance of 0.55, the 20 year ARI depth limited H_S was estimated to be 2.14 m.

Diffracted Wave Height at the Inner Foreshore

The irregular wave diffraction diagram for waves passing through a structure gap with B/L = 8.0 and S_{MAX} = 75 was again overlain on the study area as shown in Figure B.11.

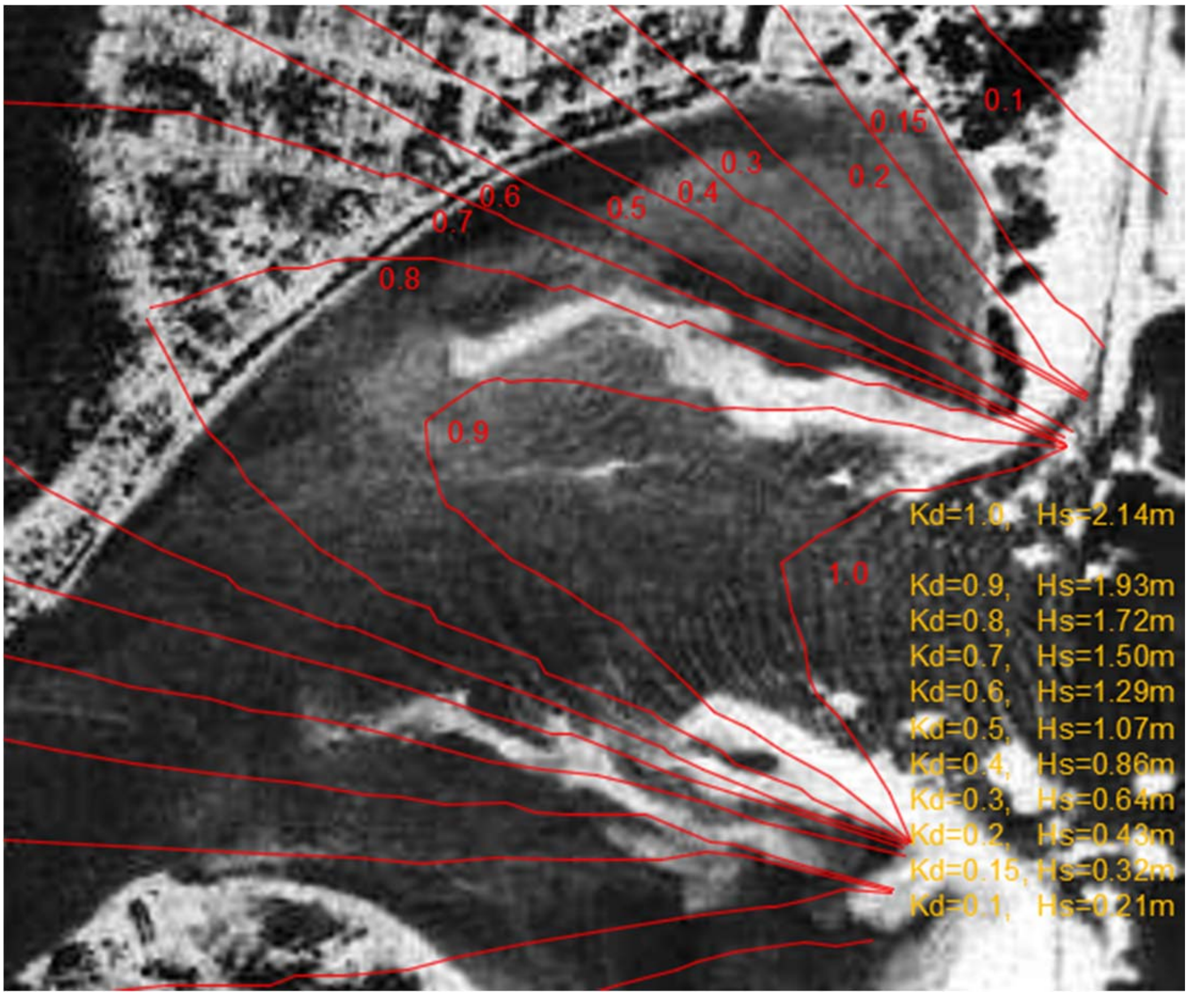

Figure B.11: Irregular Wave Diffraction Coefficients for Large Entrance Opening

(Aerial Photo 29 December 1974, Source: Chafer, 1998)

The best estimate of the diffraction coefficient for this entrance condition is 0.85; resulting in a diffracted H_S at the inner foreshore of 1.82 m (without allowance for depth limitations).

Wave Setup at the Inner Foreshore

WRL adopted the same estimate of wave setup (0.9 m) at the inner Shoalhaven Heads foreshore as with the small entrance opening scenario.

Depth-Limited Wave Height at the Inner Foreshore

Since the diffracted wave height with a large opening was much higher than with a small opening, it was also necessary to further reduce this wave height due to depth limitations across the Shoalhaven Heads inner bay. As discussed earlier, the lowest elevation maxima in any transect of available bathymetric datasets between the entrance and the inner foreshore was 0 m AHD. On the basis of a water depth over a sand bar (flood tide delta) elevation of 0 m AHD of 2.25 m and a breaker depth index at the coastal protection works of 0.55, the 20 year ARI depth limited H_S was estimated to be 1.25 m.

B.6.3 Flooding

Again, while it is acknowledged that the 20 year ARI total oceanic design water level (2.25 m AHD) is lower than the 20 year ARI peak flood level (2.7 m AHD), it is overly conservative to assume that a 20 year ARI ocean storm event would coincide with a 20 year ARI fresh water flood event. Since wave dissipation is less effected by diffraction with a large entrance opening, wave heights at the inner foreshore would increase by 15-25% due to reduced depth limitations and interaction with the opposing freshwater flooding current.

B.7 Summary of Preliminary Design Conditions

The adopted design conditions presented in the preceding sections are summarised in Table B.9.

The significant wave heights at the proposed coastal protection works for each entrance condition were used to derive additional depth limited wave statistics ($H_{1/10}$, $H_{2\%}$, $H_{0.1\%}$) according to Battjes and Groenendijk (2000). All depth limited wave calculations were undertaken assuming that the lowest sand level between the entrance and the coastal protection works is 0 m AHD.

One time-domain wave parameter, mean wave period (T_m) at the proposed coastal protection works was calculated according to Equation 4.4 (CIRIA, 2007). Note that CIRIA (2007) identifies that, for a JONSWAP spectrum, the exponent in Equation B.5 has a range between 0.79 and 0.87. WRL adopted the mid-point of this range in estimating the mean wave period.

$$T_m = 0.83 \times T_P \qquad \text{Equation B.5}$$

Two spectral wave parameters, spectral significant wave height (H_{m0}) at the proposed coastal protection works and nearshore spectral mean energy wave period ($T_{m-1,0}$) were also calculated according to Equation B.6 (USACE, 2006) and Equation B.7 (USACE, 2006), respectively.

$$H_{m0} = 1.1 \times H_S \qquad \text{Equation B.6}$$

$$T_{m-1,0} = \frac{T_P}{1.1} \qquad \text{Equation B.7}$$

Detailed wave height and period estimates at the proposed coastal protection works for each of the statistics discussed are included in the overall summary table (Table B.9).

Table B.9: Summary of Preliminary Design Conditions Estimated for 20 year ARI for Present Day

Variable	Entrance Condition		
	Closed	**Small Opening**	**Large Opening**
Design offshore significant wave height (H_{SO})	n/a	7.6 m	7.6 m
Design offshore spectral peak wave period T_P	n/a	12.2 s	12.2 s
Design offshore wave direction	n/a	East-South-East	East-South-East
Design still water level (excluding wave setup)	1.20 m AHD	1.35 m AHD	1.35 m AHD
Elevation of Entrance Invert (m AHD)	n/a	-1.0 m AHD	-2.0 m AHD
Offshore root mean square wave height (H_{RMS})	n/a	5.37 m	5.37 m
Wave setup within Entrance	n/a	0.6 m	0.5 m
Water depth within Entrance	n/a	2.95 m	3.85
Breaker index for Hs within Entrance (Goda, 2007)	n/a	0.55	0.55
Hs within Entrance	n/a	1.65 m	2.13 m
Sand bar (flood tide delta) level within the Bay	0.0 m AHD	0.0 m AHD	0.0 m AHD
Wave setup at the Inner Foreshore	0.0 m	0.9 m	0.9 m
Total design water level (including wave setup)	1.20 m AHD	2.25 m AHD	2.25 m AHD
Design Hs at coastal protection works	0.67 m	0.66 m	1.25 m
Design spectral peak wave period T_P at coastal protection works	2.9 s	12.2 s	12.2 s
Design time-domain mean energy wave period T_{m-}	2.4 s	10.1 s	10.1 s
Design spectral mean energy wave period $T_{m-1,0}$	2.6 s	11.1 s	11.1 s
Design spectral significant wave height (H_{m0}) at coastal protection works	0.74 m	0.73 m	1.39 m
$H_{1/10}$ at coastal protection works (Battjes and Groenendijk, 2000)	0.78 m	0.84 m	1.46 m
$H_{2\%}$ at coastal protection works (Battjes and Groenendijk, 2000)	0.82 m	0.90 m	1.55 m
$H_{0.1\%}$ at coastal protection works (Battjes and Groenendijk, 2000)	0.96 m	1.05 m	**1.82 m**

B.8 Sensitivity of Design Conditions

As outlined in the preceding discussion, it was necessary to make a series of assumptions to develop the preliminary design wave conditions at the proposed coastal protections works. WRL considers that the values developed for each entrance condition are conservative, under what would be extremely complex wave propagation and dissipation processes.

For purposes of simplifying the design loading parameters, the maximum diffracted wave height (0.66 m) was assumed for the coastal protection works. For the small entrance opening condition, the calculated significant wave height actually varied from 0.33 to 0.66 m along the proposed structure footprint.

If the design event is changed during detailed design (either because the design life is extended or the acceptable risk is reduced) from the 20 year ARI to event to, say, the 100 year ARI event, the wave and water level conditions at the proposed seawall along the inner foreshore are not expected to increase significantly. For example, for the small entrance opening condition, WRL estimates that the significant wave height would vary between 0.35 to 0.69 m along the proposed structure footprint with a corresponding increase in water level of 0.20 m at the structure. The peak spectral wave period would increase from 12.2 s to 12.8 s. While increased overtopping would be expected as a result of these changes, overall there is not a great deal of difference in the wave conditions at the River Road foreshore due to the dissipative processes of depth-limited wave breaking and diffraction.

For the rock-based and double layer sand-filled geotextile container seawall options later discussed in Appendix D, a tolerable degree of damage not causing structural failure has been assumed to occur under the 20 year ARI design conditions. As such, these structures would be expected to have some redundancy in their protective capacity should overload conditions occur (i.e. a 100 year ARI event) without changing the design entrance condition.

The required geometry of the coastal protection works is much more sensitive to the selection of entrance condition used for subsequent design development (rather than the ARI of the offshore wave and water level conditions). As discussed in Section 5, the future condition of the entrance is primarily dictated by fresh water flooding of the Shoalhaven River, which is encouraged to breach for the purposes of flood mitigation by maintenance of a “dry notch” weir and management of associated sand dune vegetation.

Appendix C: Hydraulic Stability of Single Layer Sand-Filled Geotextile Container Seawalls

Physical modelling results for 0.75 m^3, single layer, sand-filled geotextile container seawalls with a 1V:1.5H structure slope are presented in Table 6 of Coghlan et al. (2009). However, results for 2.5 m^3 containers in the same paper were omitted. To assist in the development of management options for Shoalhaven Heads, the single layer monochromatic wave heights causing failure of 0.75 m^3 containers were increased ("scaled up") by 35% (in accordance with the scaling relationship in Coghlan et al. 2009) to obtain wave heights causing failure for 2.5 m^3 containers. WRL has reproduced this adjustment in Table C.1.

Table C.1: Summary of Monochromatic Wave Tests (Adapted from Coghlan et al. 2009)

Still Water Level (m MSL)	Mono. Wave Period (s)	Offshore Mono. Failure Wave Height (m)	
		Single Layer 0.75 m^3 Units	Single Layer 2.5 m^3 Units
0.0	5	0.8	1.1
	10	1.1	1.5
	15	0.9	1.2
1.5	5	2.0	2.7
	10	2.5	3.4
	15	2.0	2.7
3.0	5	1.9	2.6
	10	1.8	2.4
	15	1.7	2.3

As noted in Coghlan et al. (2009), total failure of a single layer, sand-filled geotextile container seawall occurs suddenly rather than the gradual, progressive incursion of damage observed for double layer, geotextile container seawalls.

Appendix D: Rock Armour Sizing for Preliminary Seawall and Groyne Designs in Basalt and Sandstone

Table D.1: Armour Sizing for Basalt Seawall (Entrance Closed or Small Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, trunk density ≈ 2650 kg/m^3)
Hudson (SPM, 1977)	36	239	H_S = 0.66 m, K_D = 3.5 (rough, angular, random, n=2)
Hudson (SPM, 1984)	131	367	$H_{1/10}$ = 0.84 m, K_D = 2.0 (rough, angular, random, n=2)
Van der Meer (deep water)	42	251	H_S = 0.66 m, T_m = 10.1 s, S_2 = 2, N= 7500, P=0.4
Van der Meer (shallow water)	43	254	$H_{2\%}$ = 0.90 m, $T_{m-1,0}$ = 11.1 s, S_2 = 2, N= 7500, P=0.4
Adopted for this study	**150**	**380**	

Note: It is acknowledged that the design conditions at the coastal protection works fall approximately in between the ranges of validity of the Van der Meer deep water and shallow water formulae. However, both are included in absence of an equation with a validity range inclusive of the design conditions.

Table D.2: Armour Sizing for Basalt Seawall (Large Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, trunk density ≈ 2650 kg/m^3)
Hudson (SPM, 1977)	245	452	H_S = 1.25 m, K_D = 3.5 (rough, angular, random, n=2)
Hudson (SPM, 1984)	690	639	$H_{1/10}$ = 1.46 m, K_D = 2.0 (rough, angular, random, n=2)
Van der Meer (deep water)	414	539	H_S = 1.25 m, T_m = 10.1 s, S_2 = 2, N= 7500, P=0.4
Van der Meer (shallow water)	326	794	$H_{2\%}$ = 1.55 m, $T_{m-1,0}$ = 11.1 s, S_2 = 2, N= 7500, P=0.4
Adopted for this study	**750**	**660**	

Table D.3: Armour Sizing for Basalt Groyne (Entrance Closed or Small Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, head density ≈ 2650 kg/m^3)
Hudson (SPM, 1977)	44	255	H_S = 0.66 m, K_D = 2.9 (rough, angular, random, n=2)
Hudson (SPM, 1984)	138	373	$H_{1/10}$ = 0.84 m, K_D = 1.9 (rough, angular, random, n=2)
Adopted for this study	**150**	**380**	

Table D.4: Armour Sizing for Basalt Groyne (Large Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, head density ≈ 2650 kg/m^3)
Hudson (SPM, 1977)	295	481	H_S = 1.25 m, K_D = 2.9 (rough, angular, random, n=2)
Hudson (SPM, 1984)	726	650	$H_{1/10}$ = 1.46 m, K_D = 1.9 (rough, angular, random, n=2)
Adopted for this study	**750**	**660**	

Table D.5: Armour Sizing for Sandstone Seawall (Entrance Closed or Small Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, trunk density ≈ 2300 kg/m^3)
Hudson (SPM, 1977)	65	305	H_S = 0.66 m, K_D = 3.5 (rough, angular, random, n=2)
Hudson (SPM, 1984)	235	467	$H_{1/10}$ = 0.84 m, K_D = 2.0 (rough, angular, random, n=2)
Van der Meer (deep water)	75	320	H_S = 0.66 m, T_m = 10.1 s, S_2 = 2, N= 7500, P=0.4
Van der Meer (shallow water)	78	324	$H_{2\%}$ = 0.90 m, $T_{m-1,0}$ = 11.1 s, S_2 = 2, N= 7500, P=0.4
Adopted for this study	**250**	**480**	

Note: It is acknowledged that the design conditions at the coastal protection works fall approximately in between the ranges of validity of the Van der Meer deep water and shallow water formulae. However, both are included in absence of an equation with a validity range inclusive of the design conditions.

Table D.6: Armour Sizing for Sandstone Seawall (Large Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, trunk density ≈ 2300 kg/m^3)
Hudson (SPM, 1977)	439	576	H_S = 1.25 m, K_D = 3.5 (rough, angular, random, n=2)
Hudson (SPM, 1984)	1,239	814	$H_{1/10}$ = 1.46 m, K_D = 2.0 (rough, angular, random, n=2)
Van der Meer (deep water)	744	686	H_S = 1.25 m, T_m = 10.1 s, S_2 = 2, N= 7500, P=0.4
Van der Meer (shallow water)	586	634	$H_{2\%}$ = 1.55 m, $T_{m-1,0}$ = 11.1 s, S_2 = 2, N= 7500, P=0.4
Adopted for this study	**1,300**	**830**	

Table D.7: Armour Sizing for Sandstone Groyne (Entrance Closed or Small Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, head density ≈ 2300 kg/m^3)
Hudson (SPM, 1977)	79	324	H_S = 0.66 m, K_D = 2.9 (rough, angular, random, n=2)
Hudson (SPM, 1984)	247	475	$H_{1/10}$ = 0.84 m, K_D = 1.9 (rough, angular, random, n=2)
Adopted for this study	**250**	**480**	

Table D.8: Armour Sizing for Sandstone Groyne (Large Opening)

Armour Sizing Technique	**Mass** m_{50} (kg)	**Equiv. Cube Side** D_{n50} (mm)	**Notes** (structure slope 1V:1.5H, head density ≈ 2300 kg/m^3)
Hudson (SPM, 1977)	530	613	H_S = 1.25 m, K_D = 2.9 (rough, angular, random, n=2)
Hudson (SPM, 1984)	1,305	828	$H_{1/10}$ = 1.46 m, K_D = 1.9 (rough, angular, random, n=2)
Adopted for this study	**1,300**	**830**	

Appendix E: Geotechnical Assessment

REPORT

TO
WATER RESEARCH LABORATORY

ON
GEOTECHNICAL ASSESSMENT

OF
EXISTING FORESHORE AREA

AT
RIVER ROAD, SHOALHAVEN HEADS, NSW

8 November 2016
Ref: 29770ZRrpt

dlcs AS/NZS ISO 9001 certified

AS/NZS 4801 certified

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Date: 8 November 2016
Report No: 29770ZRrpt
Revision No: 0

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For and on behalf of
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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a geotechnical assessment of the existing foreshore area adjacent to the southern side of River Road, Shoalhaven Heads, NSW. The assessment was commissioned by Warwick Dawson of Water Research Laboratory (WRL) by signed WRL Contractor Agreement dated 8 September 2016. The commission was on the basis of our fee proposal (Ref. P42891ZR) dated 12 July 2016.

Shoalhaven City Council have commissioned WRL to assess coastal management and protection options for the section of foreshore adjacent to River Road that has recently been impacted by erosion following the severe storms in early June 2016. In this regard, Shoalhaven City Council have completed surveys of the north-eastern portion of the foreshore on 28 August 2008 and 30 June 2016. As part of the assessment, JK Geotechnics were requested to complete a preliminary risk assessment for the subject length of foreshore and provide risk management options. At this stage, we understand that WRL are considering toe protection works for the eastern end of the foreshore area which are expected to comprise rock armour and/or sand filled geotextile bags (geobags).

Based on the above, the purpose of the assessment was to complete a walkover inspection of the site as a basis for assessing the stability of the foreshore area and completing a risk assessment under existing conditions.

2 ASSESSMENT PROCEDURE

The assessment was completed by a Senior Associate level engineering geologist on 21 September 2016, and comprised a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs.

The geotechnical features described in Section 3 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to any proposed remediation measures, we recommend they be located more accurately using instrument survey techniques. The features observed were compared to those of other similar portions of foreshore in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting this portion of road.

The risk assessment has been completed generally in accordance with the RMS "Guide to Slope Risk Analysis" Version 4, dated April 2014. We note that the traffic volume information used in our risk analyses has based estimates from our site observations of traffic during our site inspection, as there was no relevant local information available on the RMS website. We have also completed an assessment of risk to life for users of the beach and reserve area in accordance with the AGS 2007c risk assessment guidelines (see Reference 1)

3 SITE OBSERVATIONS

The site is located on the flat coastal plain lining a portion of the north-western foreshore of the Shoalhaven River immediately landward of Shoalhaven Heads.

The foreshore area lined the south-eastern side of the asphaltic concrete (AC) paved River Road. The subject length of the foreshore was about 460m long and extended north-east from the intersection with Celia Place, to immediately to the north-east of the intersection with Mathews Street and included the foreshore area seaward of the rear yards of the immediately adjacent private properties (62 and 64 River Road); see Plate 1.

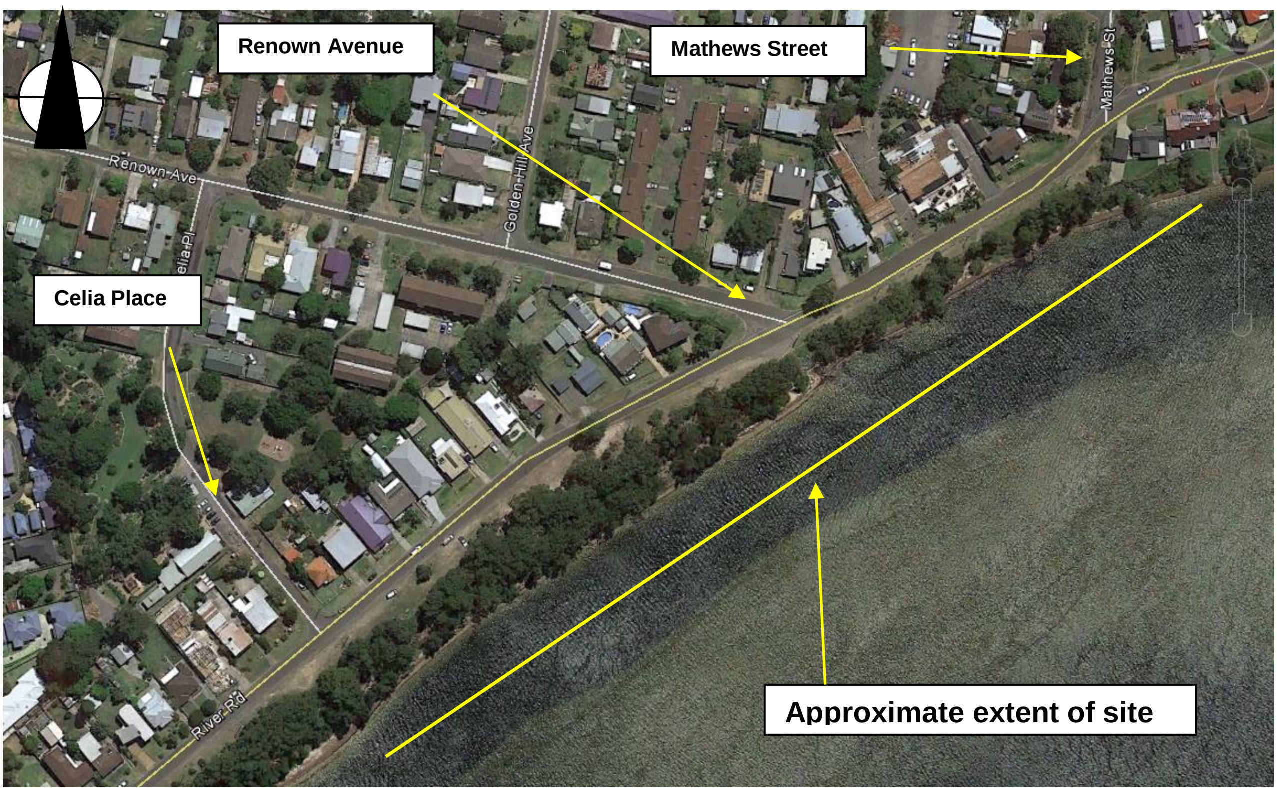

Plate 1: Site Location

The foreshore area comprised a vegetated sand slope (maximum height about 6m) which typically sloped down to the south-east at between about 35° and 45° to the gently sloping sandy beach. The crest area of the foreshore slope was lined by a relatively flat grass surfaced reserve area which was a minimum of 5.2m wide (towards the north-eastern end of the site) and a maximum of between about 17m and 22m wide towards the south-western end of the site. A number of medium to large sized trees were present along the road reserve and lined the crest of the foreshore slope. Cars were occasionally parked within the reserve area and a number of benches were also located along the length of the reserve. The roadway was immediately landward of the grass surfaced reserve area (see Plates 2 and 3).

Plates 2 and 3: Crest areas

North-eastern end of site **South-western end of site**

Timber power poles located within the grass surfaced reserve area were set-back between about 3.7m and 18m from the crest of the foreshore slope (see Plates 2 and 3).

The minimum set-back distance of the power poles and minimum width of the grass surfaced reserve were situated towards the centre of the site, opposite the intersection with Renown Avenue. In this area, the seaward margin of an AC surfaced car parking bay was set-back about 2.5m from the crest of the foreshore slope.

No tension cracks were observed within the grass surfaced area landward of the crest of the foreshore slope.

The pertinent features of the foreshore slope were as follows:

- Below the seaward side of the rear yards of 62 and 64 River Road, the foreshore area was overgrown and sloped down to the south-east at between about 25° and 30°. The toe of the slope below 64 River Road was lined by a stacked boulder wall, between about 1.5m and 2m high (see Plate 4). The boulder wall may extend south-west below 62 River Road but was unable to be confirmed as observations were limited due to the overgrown nature of the slope. There were voids behind the face of the boulder wall which extended back landward a maximum distance of about 1m.

Plate 4: Boulder wall at 64 River Road

- South-west from 62 River Road to the timber steps (Access 1), the toe of the slope was characterised by 0.5m to 1m high sub-vertical sections with traces of sandstone boulders. A dilapidated concrete surface was also present at the toe of the slope behind the timber steps (see Plate 5).

Plate 5: Timber steps (Access 1)

- Immediately to the north-east of Access 1, a 3m high sub-vertical back scarp was present and extended over a length of about 10 (see Plate 6).

Plate 6: Landslip area north-east of Access 1

- South-west from Access 1 to the timber steps opposite Renown Avenue (Access 2), there was an approximately 20m length of slope impacted by instability and characterised by a maximum 4.5m high sub-vertical back scarp. The landslip debris included the remnants of collapsed trees and tree root balls formed overhanging sections along the crest of the back scarp which extended back a maximum ‘depth’ of about 0.5m (see Plate 7).

Plate 7: Landslip area

- The north-eastern margin of Access 2 was lined by a 6m high boulder wall with two concrete stormwater pipes (about 0.5m diameter) discharging through the face of the upper portion of the boulder wall (see Plate 8).

Plate 8: Boulder wall adjacent to Access 2

- The high water mark (HWM) along the beach was inferred from the line of seaweed debris present on the beach. Between 64 River Road and Access 2, the HWM was typically off-set between about 1m and 3m from the toe of the foreshore slope and/or boulder walls.

- A number of the trees over the vegetated slope immediately to the south-west of Access 2 were leaning over from vertical and/or had curved bases (see Plate 9)

Plate 9: Curved, leaning trees immediately south-west of Access 2

- To the south-west of Access 2 the vegetated foreshore slope appeared to be free of back scarp features and the toe of the slope was set-back between about 5m and 8.5m from the inferred HWM described above. A number of the trees covering the slope (particularly the younger [smaller] trees) had curved bases and/or were leaning over.

- The toe of the slope was characterised by a flat to gently sloping beach and tree covered area with localised erosion scarps (maximum about 0.5m high) exposing tree roots (see Plate 10).

Plate 10: Relatively flat toe area south-western end of the site

- A number of trees had been felled in this area and we assume that these trees were leaning over or had been damaged by the erosion during early June 2016.

- The foreshore slope reduced in height to about 3m to 4m towards the south-western end of the site and continued to reduce in height beyond the south-western end of the site (see Plate 11).

Plate 11: South-western end of the site

4 GEOTECHNICAL ASSESSMENT

4.1 Overview

We understand from WRL that the coastal engineering setting of the River Road foreshore area is typically a sheltered estuarine environment, with exposure to short period and low-energy wind seas and tidal currents. Occasionally, the estuary entrance (Shoalhaven Heads) is broken open to the sea during flooding events, leading to scouring of the entrance area and allowing longer period ocean swell waves to cross the lower estuary to the River Road foreshore area. Exposure of the foreshore to these more energetic and erosive conditions is therefore episodic, and requires a combination of:

- Elevated estuary water levels through either terrestrial flooding or ocean surge;
- The ocean entrance of the river being open; and
- Large ocean waves.

These events are not statistically independent (for example the June 2016 storm event), and therefore understanding the likelihood of future erosion events/episodes occurring is complex and has not been analysed in detail. The likelihood of these events occurring is further complicated by the fact that the entrance stays open for a variable period of time before shoaling up, and in theory it is possible for ongoing erosion to occur after initial opening with only elevated ocean levels and big swell (i.e. without the need for further terrestrial flooding).

Previous analysis by WRL of the frequency of the river entrance opening suggest that on average the entrance breaks open to the sea at seven yearly intervals. On this basis, a likelihood of approximately 1 in 10 (expressed as an annual probability) of such an event occurring may be assumed. This may be regarded as a simplistic estimate of likelihood of an erosion episode occurring due to the following:

- It is possible to have an entrance opening (flood event) without large ocean swell, and therefore erosion may not occur; a 1 in 10 likelihood would be a conservative estimate.
- After the entrance is open, an erosion event may occur with only large ocean swells/tides and without further terrestrial flooding; a 1 in 10 likelihood would be an un-conservative estimate.

However, for the purposes of this assessment, we consider that a 1 in 10 year likelihood of an erosion event occurring to be a reasonable estimate.

Based on a review of the survey information provided by Council, the survey data from the Renown Avenue intersection with River Road (Access 2), north-east to the Mathews Street intersection with River Road has indicated the following:

- The toe of the foreshore slope has receded landwards between about 1.2m and 2.5m from 28 August 2008 to 30 June 2016.
- The beach surface level at the toe of the slope has increased between about 0.3m and 0.9m from 28 August 2008 to 30 June 2016.

The above landward recessions of the foreshore toe correspond to the recently observed areas of slope instability. However, it is difficult to assess how much of the recorded recession occurred as a result of the June 2016 storms and any other storm erosion events between 28 August 2008 and June 2016. It is probable that the majority of the recorded recession is a result of the June 2016 storm erosion. In addition, the increase in beach surface level is considered to be associated with the accumulation of landslip debris along the toe of the slope.

The erosion of the toe of the foreshore sand slope leads to over steepening and localised instability. Terrestrial flooding and high tidal levels also introduce increased quantities of water into the soil profile. This can lead to short duration elevated hydrostatic pressures within the foreshore slope immediately following the storm and/or high tidal level event (i.e. similar to rapid drawdown conditions) as the water drains from the slope. This can lead to a reduction in shear strength and increase the likelihood of instability occurring.

Such instability has been indicated by the presence of landslips within the foreshore slope between Access 2 and the south-western boundary of 62 River Road. Over the same area of the foreshore, erosion of the toe of the slope has formed sub-vertical faces (maximum height 1m) and damaged concrete faces around Access 1. The sandstone boulder wall and traces of sandstone boulders north-east of Access 1 to the north-eastern end of the site (62 and 64 River Road) indicates that some attempts have been made to mitigate the erosion of the toe of the slope. Over this portion of the site, the inferred HWM was estimated to be between 1m and 3m from the toe of the slope, compared to between 5m and 8.5m towards the south-western end of the site. This suggests a greater likelihood of storm swells and high tide events impacting the foreshore slope over the north-eastern end of the site. This is corroborated by the presence of the above described landslip features over this portion of the site and the recession recorded by the Council survey information.

In addition, the foreshore slopes are typically quite steep, ranging between about 35^{o} and 45^{o}. Such slope angles are regarded as over-steep for the silty sands exposed in the foreshore beach slope faces. The action of roots (trees, shrubs etc) and some soil suction effects has been inferred to be increasing the shear strength of the sands such that they stand at these steeper angles. However, particularly as the sands dry out, soil suction effects reduce and some slumping can occur. Further, on-going creep of the over–steep soil slopes can also be expected and was indicated by the curved and leaning trees.

The current slope instability appears to be impacting the near surface of the foreshore slopes and deeper seated rotational failures extending back landward to dot appear to be occurring. As noted in Section 3 above, no obvious signs of tension cracks in the grass surfaced reserve area were noted, which would indicate the traces of the rear of such rotational failures.

4.2 Risk Assessment

Based on the above, we consider that the potential geotechnical hazards at the site are associated with:

1. Regression of the existing landslip back scarps.
2. Additional instability caused by coastal erosion processes.
3. On-going creep of the over-steep foreshore slope.

In our opinion, the elements most at risk as a result of the above potential geotechnical hazards are:

- Members of the public in the reserve area, on the beach and occupants of vehicles on the road.
- Stormwater infrastructure and power poles adjacent to, and within, the foreshore slope.
- Parked vehicles and benches within the reserve area.

In relation to hazard 1, we note that the current erosion back scarp slopes are over steep and, at best, marginally stable. Assuming on-going regression with erosion of the slumped sand from the toe of the steep dune face during 'normal' tidal action, over the short term (within the next 12 months) we would expect the sand slope to regress back to between about an approximately 35^o and 45^o, i.e. similar to the current slope angles. We note that these angles are steeper than the angle of repose or internal friction angle of the sand, but are similar to observed slope angles, where root action has been assessed to increase the shear strength of the sand. This has the potential to result in additional landward recession of the crest of the slope of the order of approximately 1m.

In relation to hazard 2, we have assumed that a similar storm event that causes the entrance to be eroded and become open to the sea to have an annual probability of 1 in 10.

In relation to hazard 3, this has been assumed to be an active process that is currently occurring.

The posted road speed is 50KPH. Based on our site observations, we have assumed a traffic volume ranging between 270 and 2,600 vehicles/lane/day (i.e. T3) for the site. If Council have more accurate traffic volume information then this should be provided, and our risk assessment revised as necessary.

We have assessed that a vehicle would be lost into the void created by the landslip (Hazard 2) or recession of the back scarp (Hazard 1) if either hazard reached the road.

Based on the above, the Assessed Risk Levels (ARLs) for the site are as follows:

- For the portion of foreshore between the River Road intersections with Renown Avenue and Mathews Street.
    - Potential Geotechnical Hazard 1: ARL4, assuming on-going recession, further landslips and further recession impacting future landslip back scarps over the next 50 to 100 years.
    - Potential Geotechnical Hazard 2: ARL3, assuming additional erosion events of a similar magnitude occurring over the next 50 to 100 years.
    - Potential Geotechnical Hazard 3: ARL5, assuming on-going creep occurring over the next 50 to 100 years.

- For the portion of foreshore south-west of the River Road intersection with Renown Avenue and north-east of the Mathews Street intersection (62 and 64 River Road)
    - Potential Geotechnical Hazard 1: ARL5, assuming on-going recession, further landslips and further recession impacting future landslip back scarps over the next 50 to 100 years.
    - Potential Geotechnical Hazard 2: ARL5 for additional erosion events of a similar magnitude occurring over the next 50 to 100 years.
    - Potential Geotechnical Hazard 3: ARL5, assuming on-going creep occurring over the next 50 to 100 years.

Based on our past experience with RMS and application of the RMS risk analysis procedures for a wide variety of sites within various Council areas, we have assumed a risk level of ARL3 will generally be considered to be 'tolerable', if monitored.

In addition, we have completed an assessment of risk to life in accordance with the criteria given in the AGS 2007c risk assessment guidelines (see Reference 1). Using the indicative probability associated with the assessed likelihood of instability outlined above, and assuming typical temporal, vulnerability, evacuation and spatial factors for this type of site, we consider the levels of risk to life to be less than $1x10^{-6}$. This would be considered an 'acceptable' level in relation to the criteria given in Reference 1.

The above risk assessment has indicated that current levels of risk are generally at 'acceptable' levels, with the exception of future erosion events causing landslips within the foreshore slope

(Hazard 2) between the River Road intersections with Renown Avenue and Mathews Street and impacting the road and road users.

On this basis, we consider that on-going monitoring would be an appropriate landslide risk management option. However, with regard to the portion of foreshore slope between the River Road intersections with Renown Avenue and Mathews Street, construction of foreshore erosion protection measures would reduce risk to ‘acceptable’ levels. This assumes they are designed and constructed in accordance with the advice presented in Section 5, below and the requirements of the drawings to be prepared by WRL.

It should be recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners be made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, risk cannot be completely removed, only reduced, as removing risk is not currently scientifically achievable.

In preparing our recommendations given below we have assumed that no activities on surrounding land which may affect the risk on the subject sites would be carried out. We have further assumed that all Council buried services and other buried services within the sites are, and will be regularly maintained to remain, in good condition.

5 GEOTECHNICAL ADVICE

5.1 Landslide Risk Management Strategy

Based on the results of our assessment, we consider on-going monitoring to be a reasonable short term landslide risk management strategy, i.e. within the next five years following issue of this report. However, this does assume that the current coastal engineering overview outlined by WRL (and summarised in Section 4.1, above) is an accurate assessment of current conditions. If it becomes apparent that storm events leading to an opening of the entrance to the sea are more frequent than assumed, and are leading to erosion at a greater rate than assumed, then the landslide risk management strategy will need to be reviewed.

In addition, our observations and review of the provided Council survey monitoring results have indicated that the portion of foreshore slope between the River Road intersections with Renown Avenue and Mathews Street are characterised by the following:

- A reduced width of grass surfaced reserve behind the crest of the foreshore slope, and
- Current evidence of erosion and landslips.

As such, on-going erosion of this portion of the foreshore has the potential to impact the timber power poles and the road at an earlier future date than for the remainder of the foreshore slope. We understand that WRL are considering toe protection works for the north-eastern end of the foreshore area which are expected to comprise rock armour and/or sand filled geotextile bags (geobags). As noted above, the provision of such toe protection works would reduce risk to 'acceptable' levels for this portion of the site. To maintain the amenity of the foreshore area and significantly reduce the rates of erosion over this portion of the foreshore area, we recommend that these toe protection works be implemented.

5.2 Monitoring

Council should monitor the foreshore slope on an annual basis, after periods of prolonged or heavy rainfall and during periods of predicted peak tidal levels, in order to assess existing conditions and any indications of deterioration such as tension cracks along the crest area of the foreshore slope, further evidence of landslips, damage to timber steps, drainage culverts etc. We recommend that the owners of 62 and 64 River Road also be made aware of the contents of this report and undertake similar monitoring of their foreshore area and present the results to Council.

It is imperative that such monitoring be formally documented and that the required frequency of reporting (and to whom) is clearly defined. Where incidents of instability have occurred within the monitoring period, then where possible we suggest that Council/private property owners provide relevant details within the monitoring reports. These details would include the date of the incident, the weather conditions on the day and leading up to the incident, a location plan, photographs and dimensions of the specific feature (width and length of landslip features, leaning power poles, collapsed trees, crack widths etc would also need to be recorded). The monitoring reports should be provided to the geotechnical and coastal engineers so that if there are any causes for concern, further advice can be provided. The need for site specific stabilisation measures can then be better assessed.

If during the monitoring period, if erosion occurs such that power poles are impacted and/or the landslip back scarp encroaches to within 3m of the south-eastern (seaward) side of the road, then the following measures will need to be immediately implemented:

- Close the affected section of foreshore beach and reserve area at the crest of the area of instability.
- Notify the utility company.
- Close the adjacent section of road.
- Contact the coastal and geotechnical engineers and arrange an immediate site meeting to determine risk levels and appropriate risk management measures.

On a five yearly basis, consideration should be given to completion of detailed assessment by experienced geotechnical and coastal engineers to assess current conditions with regard to the on-going inspection monitoring reports. Depending on the future stability of the foreshore area and the results of the monitoring reports, the interval between geotechnical and coastal engineering assessments may need to be reduced or increased with respect to the nominated five yearly interval.

5.3 Stormwater Drainage

We recommend that the stormwater drains (and any other water carrying pipelines) crossing under the road and/or discharging onto the foreshore area, be checked for leaks and damage by a plumber or similarly qualified professional. Erosion protection measures (e.g. 'rip rap') should be provided, where necessary, below the pipe discharge points.

Based on the above checks, appropriate maintenance and repairs should be completed without delay and may also need to include improvement of the existing stormwater system (and possibly other water carrying pipelines).

5.4 Tree Planting

We note that over the south-western portion of the site and within the areas of recent instability, a number of trees have been removed by Council. In some instances (the areas of instability), we assume that the trees had collapsed and/or were leaning and posing a hazard to beach users. We note that tree roots acting to bind the sands together, improving their shear strength and reducing the impact of wave and, wind and surface run-off erosion. We recommend that where possible, trees that have been removed should be replaced with appropriate species suitable for this foreshore environment. Appropriate advice should be sought from a specialist arborist.

5.5 Erosion Protection Measures

The design of any adopted erosion protection measures (rock armour protection of geobags) will require additional geotechnical input as outlined in Section 2.2 of our proposal (Ref. P42891ZR) dated 12 July 2016, namely

- A limited scope geotechnical investigation, and
- Stability analyses for the proposed foreshore protection measures using the SLOPE/W soft ware package.

The alignment of the any erosion protection will need to be carefully detailed and constructed with regard to providing a smooth transition with the adjacent sections of unprotected foreshore slopes and the interfaces with existing stormwater infrastructure. In this regard, we note that at the interface between 'hard' and 'soft' foreshore slope areas, additional erosion can occur due to turbulence effects.

5.6 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Limited scope geotechnical investigation.
- Stability analyses to confirm the suitability of the preferred erosion protection measures.
- Review of monitoring reports completed by Council and private residents.
- Completion of five yearly assessments of the foreshore slopes.

6 GENERAL COMMENTS

It is possible that the subsurface soil or groundwater conditions may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

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Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.