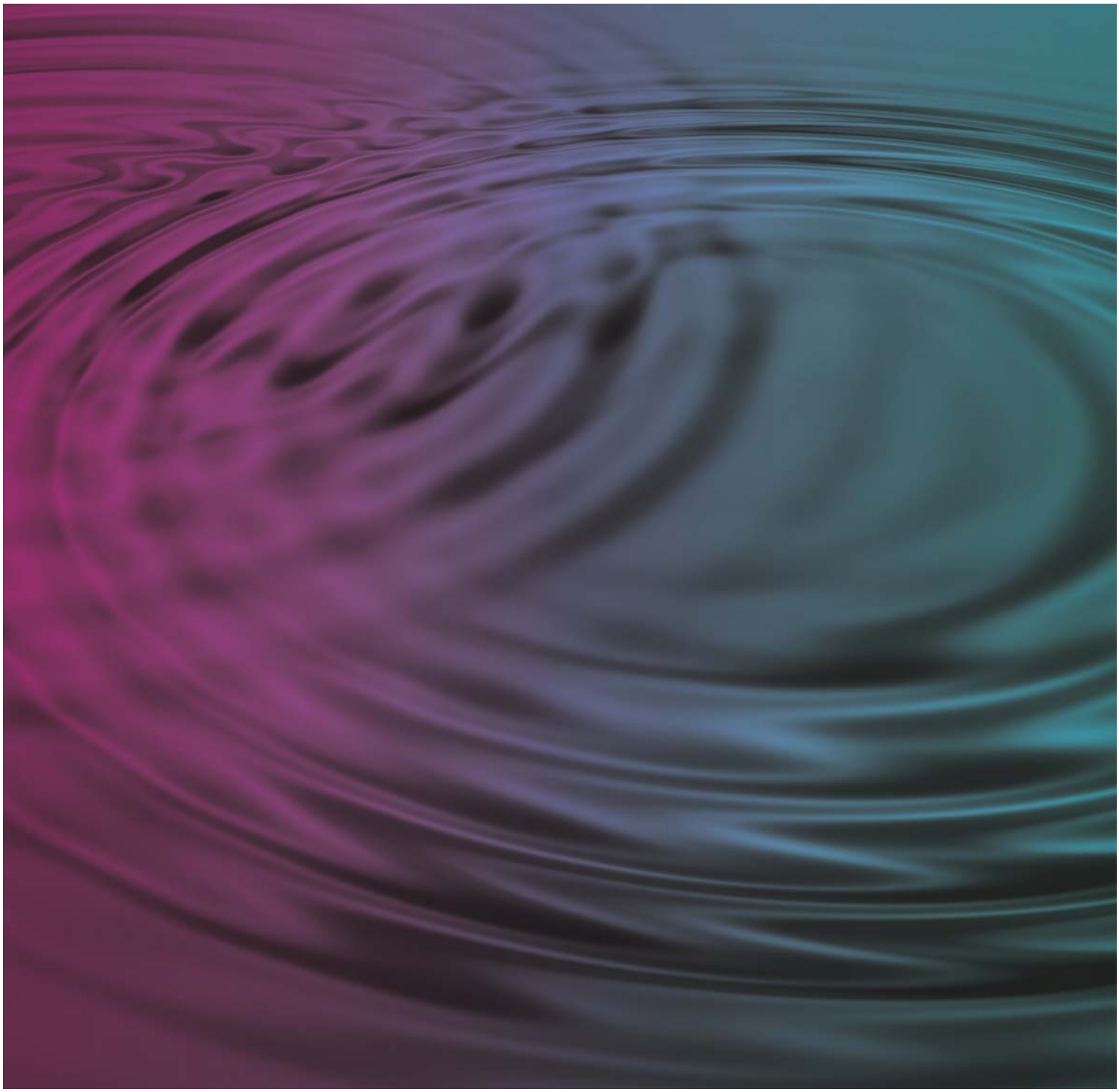
District Council of Tumby Bay
28-Feb-2014

Tumby Bay Foreshore Protection Consultancy Services

Final Concept Design Report



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Final Concept Design Report

Client: District Council of Tumby Bay

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Revision History

Revision	Revision Date	Details	Authorised	
			Name/Position	Signature
A	31-Oct-2013	Draft Report	Doug Bowers Principal Civil Engineer	
B	12-Dec-2013	Final Concept Design Report	Doug Bowers Principal Civil Engineer	
C	28-Feb-2014	Final Concept Design Report	Doug Bowers Principal Civil Engineer	

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Executive Summary

The precast concrete wall installed in 1999 along the foreshore within the Tumby Bay main township has progressively suffered a series of failures since its installation after storm events.

Council in 2003 developed a proposal to provide a rock seawall as a protection treatment along this main foreshore area to replace the concrete wall but this did not proceed due to community concerns on the impacts of such a proposal on the foreshore infrastructure and available beach width.

Since 2003 Council has upgraded infrastructure on the foreshore and sections of the wall failed in 2005 and in June 2013 following major storms.

Council is now seeking a solution or solutions that address the current problems and projected problems arising from climate change and sea level rise that are affordable and likely to be more acceptable to the local community.

The existing seawall is being overtopped under current wave and sea level conditions and is failing due to saturation and erosion of the sand behind and under the wall during wave overtopping events and the foundation level being too shallow.

The long fetch in Spencer Gulf under strong easterly and north easterly winds is considered to be the primary factor for generating large damaging waves along the Tumby Bay coast which have the potential to reach 3.5-4m height.

The height of the current sea wall and foreshore is below the 1 in100 year ARI storm level and under further sea level rise the foreshore and Tumby Terrace will suffer increased inundation and erosion placing existing infrastructure and assets at risk.

Current sea level rise along the Adelaide coast is being measured at 7mm/pa.

If no protection is provided, the coastal edge could recede as much as 26m and 46m by 2050 and 2100 respectively under IPCC 0.3m and 1m sea level rise projections and 1 in100 year ARI storm surge conditions.

The foreshore must be protected by a sea wall that is robust enough to withstand the expected larger wave forces and accelerated erosion rates arising from sea level rise and climate change and the crest of the wall must be located considerably higher than the existing foreshore levels to minimise flooding and erosion impacts on the foreshore and abutting properties.

A sloping rock or geofabric seawall installed with a crest height of 3.3m AHD is considered an acceptable and affordable solution up until 2050; however a rock seawall would be more durable.

Existing foreshore infrastructure will need to be modified to accommodate the new seawall.

1.0 Background and Scope of Services

1.1 Background

The town of Tumby Bay is located in a bay approximately 50km north of Port Lincoln on the Eyre Peninsula abutting the western coastline of Spencer Gulf.

The foreshore and beach abutting the Town Centre prior to the 1970's was a natural low lying vegetated sand dune shoreline with playground equipment, car parking and a café.

In the 1970's lawn was established by the Progress Association and vegetation cleared to improve the usage of the foreshore area.

In the late 1980's a major storm surge and high tide event caused severe erosion of the beach and foreshore abutting the main township resulting in damage and loss of car parking playgrounds and other foreshore infrastructure. This event prompted the District Council of Tumby Bay (Council) in 1999 to install 1.8m high interlocking L shaped precast vertical concrete wall panelling to the south of the main jetty to the Lions Park and to several hundred metres north of the Ritz Café. The top of the wall was installed at approximately 2.4-2.6m AHD.

Since 1986 the Coastal and River Murray Unit of the Department of Environment, Water and Natural Resources (DEWNR) has been monitoring erosion and sand movement at five profile locations along the township coastline and they have identified this location as an erosion hot spot.

A short section of the wall to the north of the Ritz Café experienced complete failure and clockwise overtopping in December 2001 and by 2003after several subsequent failures the entire wall section north of the Ritz Café was removed and the section north of the Ritz car park replaced with imported sand and revegetated. The creation of these revegetated dunes appears to have shown success in withstanding high tide and storm surge events since.

In 2001 a Foreshore Committee comprising community and Council representatives was formed to develop potential foreshore protection solutions which included trial sand bag groynes and a rock revetment wall.

In around 2003 Council, in conjunction with Tonkin Consulting prepared plans for the foreshore area, with the Council adopting a position of seeking to establish a rock revetment wall for much of the foreshore area and installing three lateral sand groynes south and north of the jetty and at the north end of the township. Despite the preparation of preliminary plans to this effect, the rock wall was never constructed due to community concerns about the loss of beach width and impacts on foreshore infrastructure, and accordingly over a period of time alternative approaches to foreshore stabilisation have been adopted.

The trial sand groynes were installed at this time in an attempt to minimise south to north littoral drift.

A localised 45 m section of the southern section of wall abutting the Lions Park was severally damaged in March 2005 and was subsequently repaired and extended further south with additional toe protection provided in the form of Elcorock sandbags. A concrete footpath and additional beach access ramps were also provided at this time.

Council installed a trial 'Elcorock' bag wall in the central section north of the jetty north of the playground in 2005 in lieu of rock revetment and provided revegetation above which has held well since.

During a high tide and storm surge event on 23 June 2013, the central wall section north of the jetty abutting the Ritz café was overtopped, which lead to the outwards rotation of approximately 100m of vertical concrete wall and slumping of the backfill material. This wall has been temporarily protected and supported through the installation of 'Elcorock' sandbags. Rapid sand loss from the beach in front of the wall was also observed.

The portion of the foreshore adjoining the Town Centre area (The study area) is approximately 350 m long and currently comprises developments including the RITZ café, new playground areas, a picnic/BBQ area, a public convenience, historic rotunda, jetty abutments, beach access ramps, car parking, lawned and paved public areas, a war memorial a since removed clock tower, and the Lions Park. Much of this development is sited on land reclaimed from the sea, or in areas historically occupied by a low dune system.

The study area as shown in Figure 1 is heavily used by the public particularly in the summer months and extends from approximately 80m south of the jetty to the north edge of the Ritz car park.

Figure 1 Aerial Photo

In areas to the north of the Town Centre area, the implementation of the soft engineered solutions required the loss of some width of previously maintained grassed foreshore area, which while being possible in this area, is less desirable in the study area.

Given the above, Council has determined that Council's previously accepted option of a rock revetment wall is considered to require further and more critical assessment and other potential softer solutions require investigation.

With this history of storm surge erosion and wall failure events Council in conjunction with the Coast and River Murray Unit of the Department of Environment Water Natural Resources (DEWNR) determined that an investigation was required into alternate protection options and the adopted solution should be designed.

Council was subsequently successful in obtaining funding from the South Australian Coast Protection Board to undertake such a study and design the adopted treatment.

1.2 Scope of Services and Methodology

The consultancy involved the following tasks in accordance with Council's brief.

1.2.1 Stage 1 - Concept Design (The subject of this report)

Development and investigation of concept designs for works to protect a 350 m length of the Tumby Bay Town Centre Foreshore area from erosion in conjunction with the local community and key stakeholders.

The concept designs considered the aesthetic and functional values of the foreshore area and to the extent possible must be compatible with existing development and public use of the space. Concept designs developed were practical, deliverable within indicative budgets and developed with Council staff, and were discussed with and consistent with Coastal and River Murray Unit of the Department of Environment, Water and Natural Resources (DEWNR). Consideration of sand retention measures, including reinstatement of groynes were also considered as part of the concepts.

Key tasks included:

- A Project start up meeting incorporating a site inspection with Council staff on 24 October 2013

- An initial meeting with Council staff and key community group representatives on 24 October 2013 to obtain critical data and historical information.
- Initial liaison with key stakeholders (primarily DEWNR) on 22 October 2013, including determining required parameters for sea level rise allowances, profile and historical photography and bathtub inundation models for Tumby Bay Township and presenting draft protection options.
- Assessing coastal process issues for the upgrade of the foreshore protection of Tumby Bay
- Development of draft concepts (minimum 4- including a retreat Non Protect option), clearly defining opportunities and constraints of each option.
- Development of a draft concept design report (This report) including:
    - Setting maximum crest levels for any protection works
    - Identification of foreshore pedestrian linkages and beach access arrangements for each concept
- Presentation of draft report outcomes to the community and elected members in Tumby Bay as well as internal and external stakeholders including indicative cost estimates (construction costs and whole of life considerations) and the positive and negative points of each option.
- Development of a final concept design report and a recommendation including documenting the findings of the study, the outcomes of the public consultation, the options considered, the preferred option, the construction strategy, design details, recommended order of construction staging and cost estimates. All the relevant organisations and required approvals should be identified and detailed within the report.

1.2.2 Stage 2 — Detailed Design

After Council's acceptance of the concept report, detailed design and documentation detailed design and documentation of the preferred protection treatments will be undertaken including:

- Design and documentation of the preferred option identified during Stage 1
- Liaison with relevant stakeholders to obtain design comments and approvals
- Draft drawings and specifications issued for Council comment
- Final Drawings and Specification
- Construction Cost Estimate

2.0 Data Review

2.1 Council Data

2.1.1 Historical photographs

Council provided a range of oblique historical photographs dating from 1939 to 2013 which confirmed that the coastal beach edge prior to placing the lawn and concrete panels was further west by several metres.

The photos also indicate the coastal edge adjacent to the jetty is the most westerly point of the bay

2.1.2 Tumby Bay Bathtub Inundation Map for 2050 Climatic Conditions

Council provided a coloured plan indicating inundation depths across the town indicating the foreshore and Tumby Terrace would be inundated by up to 300mm under the 2050 design level sea level rise scenario for a 100 year ARI event of 2.95m AHD.

2.2 DEWNR Data

2.2.1 Coastal Profiles

Coastal profiles as supplied by DEWNR were provided and examined for five profile locations abutting the township and are discussed in Section 4.

2.2.2 Aerial and oblique photographs

Aerial photographs for 1945 and 2010 and historical oblique photographs were examined for the purpose of observing any historical coastal changes.

2.3 Geology

An examination of PIRSA geology maps indicates that the local geology is part of the Gawler Craton formation comprising a 5-10m depth of Quaternary sands overlaying a 15-20m depth Gibber bed underlayed by the Spilsby Suit which is confined to the Spencer Gulf. It is understood that shallow groundwater is evident within the sands and extends under the township.

The top 400-500mm of material on the foreshore reserve comprises imported clay in lawned areas.

A more detailed description is contained in Appendix A Geotechnical investigations.

2.4 Aboriginal and European Heritage

2.4.1 Indigenous Heritage

An examination of the Government data base indicates there are no known aboriginal sites within the study area

2.4.2 European Heritage

Based on discussions with Council officers and local heritage records, sites of historical importance include the following and would be important assets to retain:

- Ritz Café-originally built 1903
- Jetty-originally built 1909

3.0 AECOM Observations

3.1 Site Observations 24 October 2013

Site observations were undertaken by Doug Bowers from AECOM from 8.30am-9am and from 12noon to 3.30pm and the following observations were made:

- Conditions were reasonably calm in the morning with an easterly breeze strengthening to an estimated 20km/hr by 3.30pm (Actual observed by BOM at 3pm 22km/hr). Wave heights did not exceed 0.25m and wave periods ranged from 1-3 seconds and approached at 90 degrees to the coast. Low tide was estimated at 12noon with high tide occurring around 4pm. Waves broke typically within 20m of the tide mark.
- The tide was running out quite swiftly from the harbor channel located at the south end of the beach at 8.30am and near shore reefs were evident on the seaward side of the adjacent point.
- The upper beach profile is steep throughout the study area. Sand is fine grained and soft under foot with some shelly portions adjacent to the sea wall.
- The three lateral sand groynes were evident approximately 50m south and 40m north of the jetty and at the north end of the township.
- The top of the two southern sand groynes were largely below the low water mark and well down the beach and are largely at the end of their useful life and ineffective, whereas the northern one was further out of the water and extended further up the beach, however no discernible change in sand levels on either side of the bag indicating the groynes in their current condition are ineffective in controlling littoral drift.
- The 'Elcorock' sand bag revetment wall adjacent to the playground was in good condition showing little signs of erosion and appears to have provided reasonable performance for its eight years of service.
- There are several stormwater outlets through the wall and coastal edge none with erosion control or headwalls many silted with sand.
- There is recycled water irrigation network within the lawned area of the foreshore.
- Stormwater from the northern end of the Ritz car park is discharged directly by overland flow into the unprotected coastal edge causing erosion.
- The vegetated coastal edge north of the Ritz car park appears stable
- There have been modifications to the foreshore infrastructure since the 2003 engineering survey including:
    - A new playground to the north of the jetty
    - New beach access ramps to the north of the jetty and playground and paved footpath at the top of the wall;
    - A new footpath and boardwalk ramp adjacent to the wall south of the jetty fronting the Lions park;
    - A new gravel car park and coastal path south of the Lions Park

Photographs highlighting the observations are shown in Appendix B.

3.2 Data Review Observations

Having reviewed the several historical photographs of previous coastal erosion and storm surge damage (Refer Appendix B), the 2003 engineering survey and the bath tub inundation models provided by Council, the community and DEWNR, the following observations can be made:

- A clockwise rotation of the top of the wall towards the beach and major slump in the ground behind the wall was evident near the Ritz car park in May 2001. By December 2001 a section of the wall had completely collapsed with the panels facing in a horizontal position towards the beach;
- By April 2002 only a short section of the northern wall was remaining;
- By Feb 2003 no wall north of the Ritz car park was evident and considerable further erosion of coast where the wall once stood, had occurred;

- On 24 June 2013 there was evidence of significant slump behind the central wall abutting the east side of the Ritz and horizontal clockwise displacement and seaward rotation of the top of the wall panel. There was evidence of sea grass at the top of the wall and the adjacent Elcorock sand back wall at this time indicating wave overtopping and also seagrass build up;
- Typical vertical erosion scars on the non-protected sections of the coast after a storm are 0.9 m high;
- In 1999, a very high tide was evident with the beach completely inundated and the water level sitting at 0.5 m from the top of the concrete wall- nominally 2.0 m AHD;
- The top of the vertical precast concrete wall has been installed at approximately 2.4-2.6 m AHD and the foundation is set at approximately 0.6 to 0.7 m AHD which is within the wave run up and erosion zone;
- The L shaped panels appear to have a 450-500 mm wide heel on the land side and have no vertical keys, which would offer negligible passive resistant to any active pressures or wave overtopping induced surcharge loads imposed on the wall;
- The foreshore along the study area including Tumby Terrace would be inundated for any coastal water levels above the current top of the sea wall;
- There is an inter-tidal reef located approximately 400 m offshore immediately to the east of the marina channel adjacent to the headland (1200 m south east of the jetty/ beach interface and further reefs on Tumby Island 4000m South east of the jetty/beach interface. Further islands are located 21000 m south east from Tumby bay. All of these reefs and islands would refract longer fetch offshore waves arising from the south east.

4.0 Coastal Processes

The main drivers of coastal processes at Tumby Bay are:

- Winds
- Waves
- Tides, storm surge, and sea level rise
- Tides (incl currents through estuary at south end of beach)

A detailed description of each of these meteorological and oceanographic (met-ocean) parameters at Tumby Bay and the surrounding region is given below.

4.1 Winds

The wind patterns in the Great Australian Bight show a distinct seasonal pattern where the dominant winds are from the south, south-south-west, and south-east in summer and north and west in winter (Figure 1).

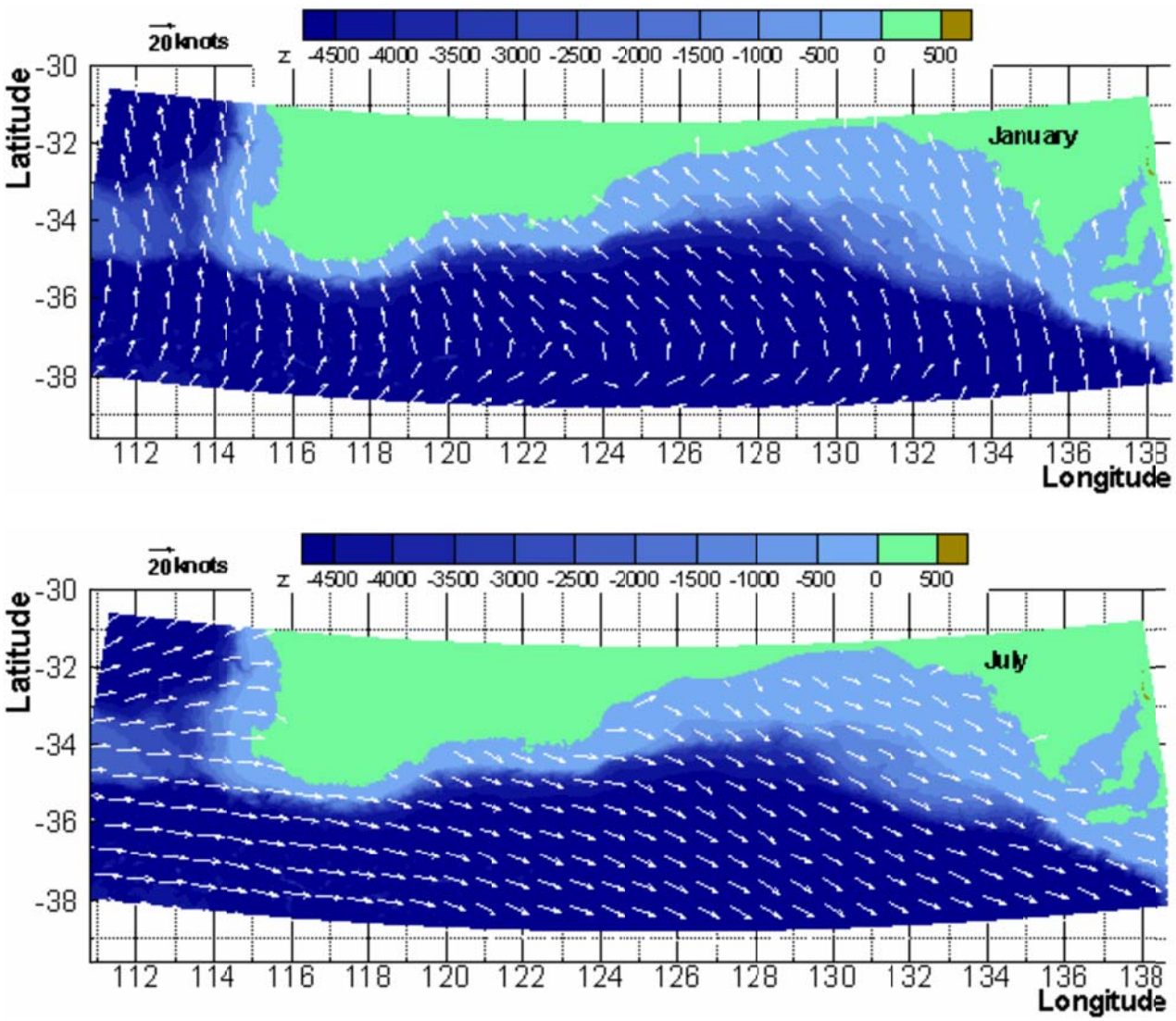

Figure 2 Wind patterns in summer (upper) and winter (lower).

The cold fronts in winter are associated with mid-latitude low-pressure systems. The westerlies and south-westerlies generate storm surges of at half a meter in height following the passage of cold fronts (Government of South Australia, 2013).

The annual wind distribution patterns at a location (North Shields) closest to the study site are shown in Figure 2 where the 9 am winds blow from virtually all directions but the east, south-east to south-west winds are dominant at 3 pm.

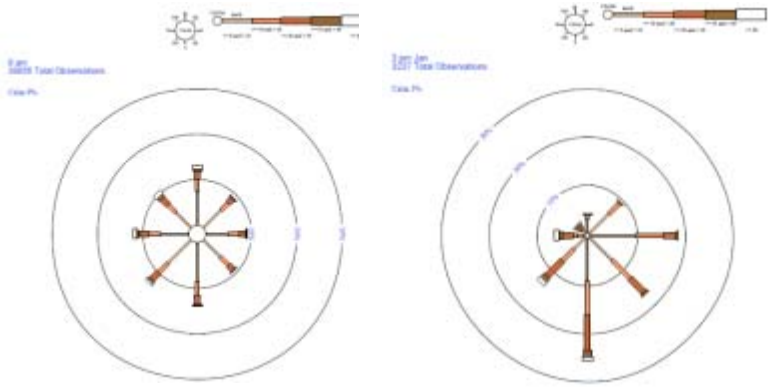

Figure 3 Annual wind roses at North Shields (Port Lincoln). Left panel shows winds (9 am) and the right panel shows winds at 3 pm (Source: Bureau of Meteorology).

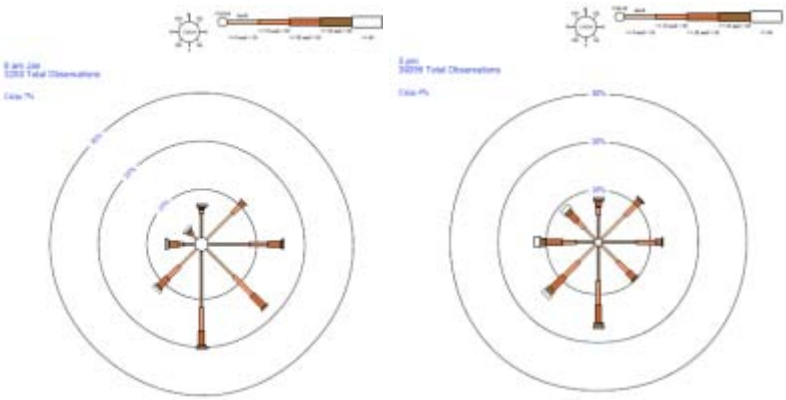

Figure 4 Summer wind roses at North Shields (Port Lincoln). Left panel shows winds (9 am) and the right panel shows winds at 3 pm (Source: Bureau of Meteorology).

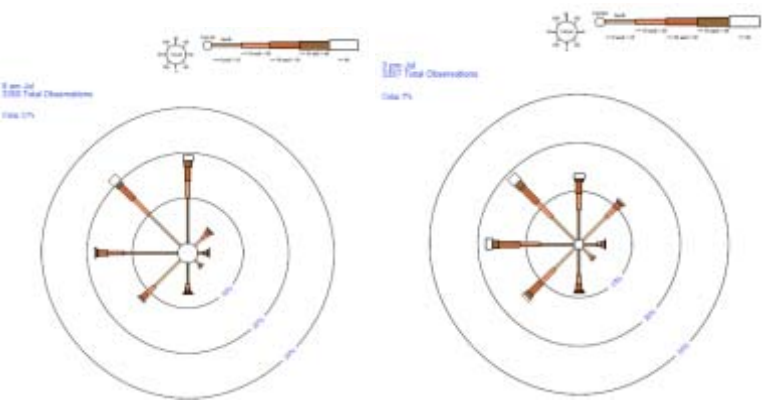

Figure 5 Winter wind roses at North Shields (Port Lincoln). Left panel shows winds (9 am) and the right panel shows winds at 3 pm (Source: Bureau of Meteorology).

Similar to the regional pattern of seasonal variations, the seasonality is also evident in the local winds as seen in (Figure 3) when the summer winds are pre-dominantly from the south, south-east, while the winter winds mainly approach from the south-west through to west, north-west and north (Figure 4).

The waves in the study area will be mainly generated by winds within the Spencer Gulf.

4.2 Wave Climate

The wave climate in Spencer Gulf is driven by the winds (CSIRO, 2007) with west to southwest winds and waves dominant in winter and south to southeast wind and waves dominant in summer. The enclosed configuration of Tumby Bay provides protection from the intense south-westerly winds resulting from the westerly cold fronts. The southerly waves will also dissipate most of their energy as they refract, diffract around the headlands; the islands immediately offshore of Tumby Bay; and the Gambier Islands near the mouth of the Gulf.

4.2.1 Local Fetch Waves

Whilst swell and wind waves from the Southern Ocean may not have a significant impact on Tumby Bay, the study site will experience large waves generated by strong winds in the Spencer Gulf. The study site is exposed to large fetches of up to 190 km from the northeast and about 130 km from the east and east-south-east as shown in Figure 6.

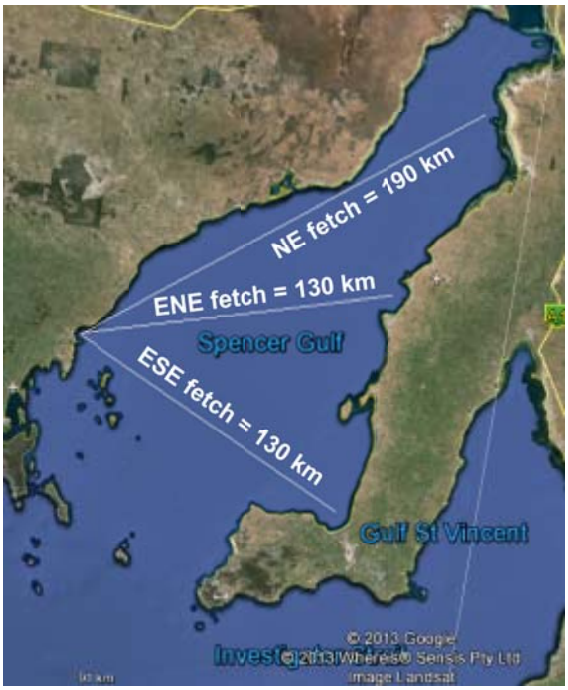

Figure 6 Available fetches at the study site

Using the fetches shown in Figure 6, and the extreme winds from the Australian Standard AS1172.2 wind code, the wind-waves generated by 50, 100, and 500 year ARI winds were calculated. The simple wind-growth functions (SPM, 1984) were employed to compute the wind-waves. The 3-s gust from AS1172.2 was converted to a mean-hourly wind speed required for wave generation. Directional factors (0.8 for NE and ENE) were applied to the wind speed. An average water depth across the fetch length was assumed. Table 1 presents the results of the wind-wave analysis.

Table 1: Wind-wave parameters in Tumby Bay

Storm event (year ARI)	Mean hourly wind speed (m/s)	NE		ENE/ESE	
		Hs (m)	T (s)	Hs (m)	T (s)
500	24	4.4	8.6	4.4	8.1
100	22	4.1	8.3	4.0	7.8
50	21	4.0	8.1	3.9	7.7

It should be noted that this method assumes that the wind is constant across the fetch and the waves are coincident with the winds. It does not take into account the complex bathymetry. This may result in overestimation of wave heights. In particular, the reefs offshore of the marina channel and around Tumby Island will dampen and dissipate wave energy. Long-term wave measurements or a numerical wave model set-up with representative bathymetry and realistic boundary conditions would provide an accurate estimate of wave parameters at the site.

4.2.2 Swell Waves

In the absence of site specific recorded or modelled wave data, offshore waves at Cape du Couedic (Kangaroo) Island were analysed to provide an indication of the offshore wave climate. Analysis of the eight-year recorded wave data (2000-2008) at Cape du Couedic shows that the offshore waves are significantly higher in winter than in summer. The average significant wave height is 2.6 m over the eight-year period while the maximum significant wave height is 8.5 m with an associated peak wave period of 18 s. Figure 7 shows the frequency distribution of significant wave heights at Cape du Couedic and Figure 8 presents time-history of significant wave heights at Cape du Couedic during the winter month of May in 2001. The 100 year ARI significant wave height at Cape du Couedic was computed as 9.4 m using extremal analysis. The offshore wave directions from hindcast global wave data are represented in Figure 9 and indicate that waves enter the Spencer Gulf mainly from the south-west sector.

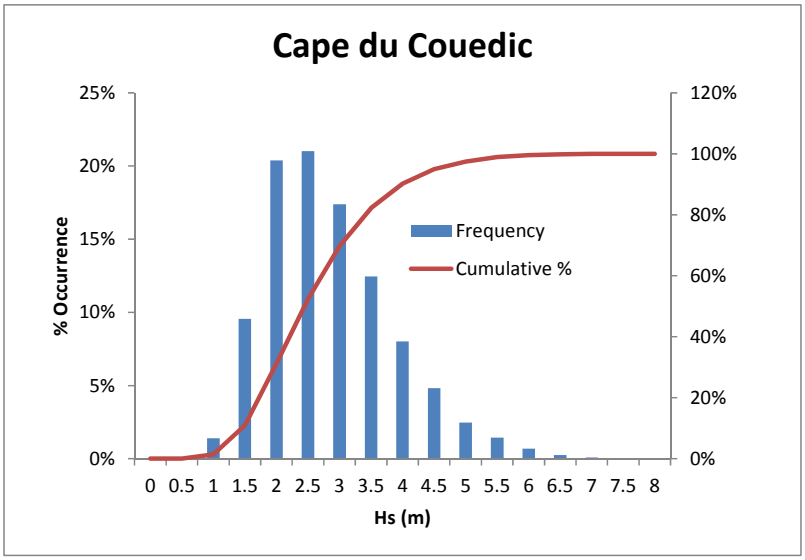

Figure 7 Histogram of wave height distribution at Cape du Couedic (Kangaroo Island).

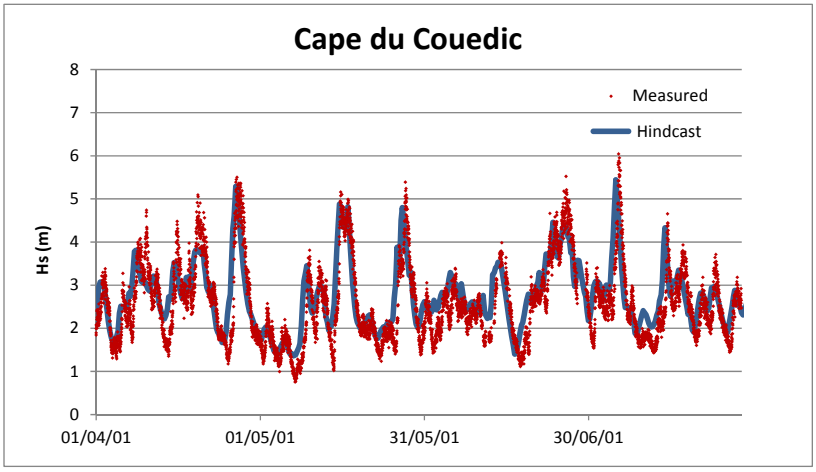

Figure 8 Recorded data (red) and hindcast (global model) at Cape du Couedic (Kangaroo Island).

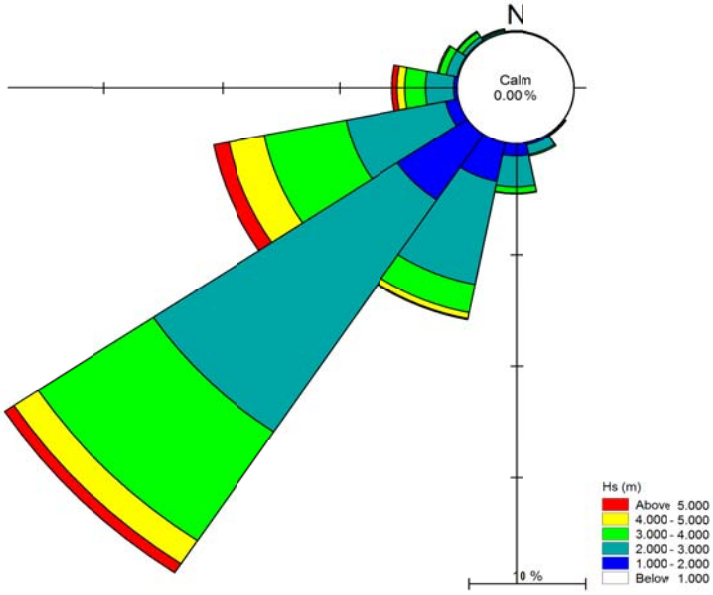

Figure 9 Wave rose using global hindcast data at Cape du Couedic (Kangaroo Island).

A wave model developed by Middleton et al (2012) shows that a southwest significant wave height of 5 m at the mouth will reduce to less than 1 m as it reaches Tumby Bay (Figure 10).

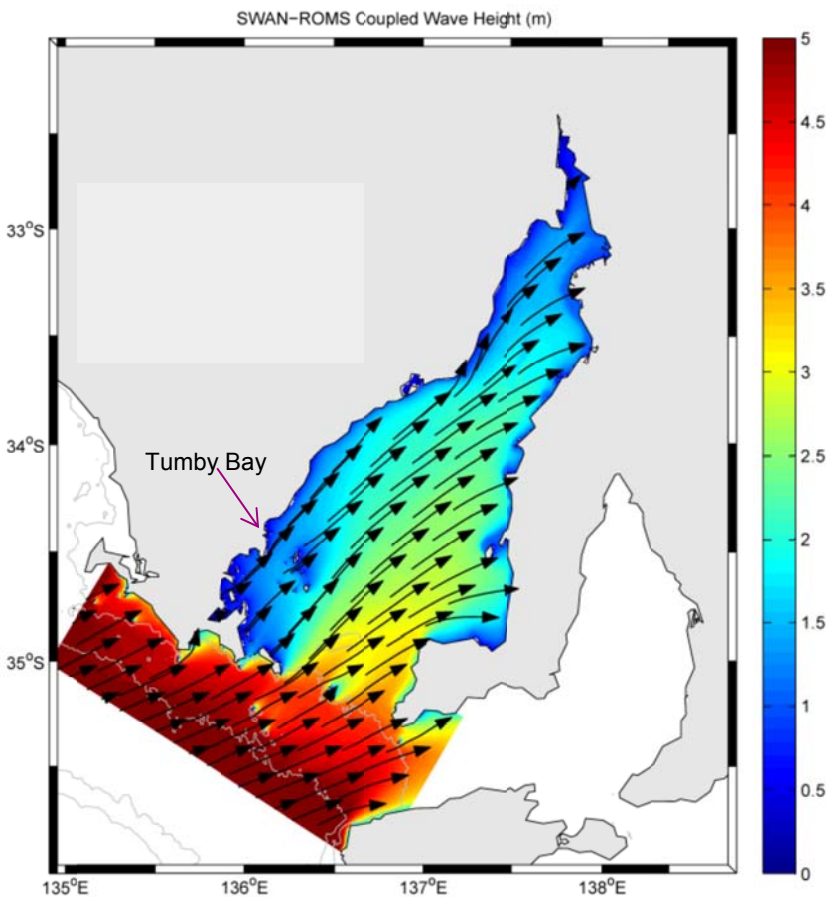

Figure 10 Wave model of Spencer Gulf

4.3 Sand Transport

The dynamics of near shore sediment movement is governed by the interplay of the key coastal processes, especially the wave climate that varies over timescales from hours to decades. An understanding of the ambient and extreme waves will be necessary for the evaluation of sediment transport patterns along the Tumby Bay foreshore. The study area is located about 75 km from the entrance from the Southern Ocean and is exposed to the ocean swell and sea states. The wind waves are dominant in the study area and hence longshore transport is likely driven by local wind-waves. In summer, the south and south-west winds are dominant, which would generate a northerly drift. However, the summer waves are generally smaller and the fetches from the south and south-east are smaller compared with the northeast. During winter, the northwest and northern winds prevail and these will generate a southerly drift.

The sediment transport along the study area appears to have influenced by the development of the foreshore, construction of groynes and the concrete sea walls. The beach at Barraud Street within the undisturbed coastline (125 m south of the Jetty) south of the study area appears to have lost up to 1-2m depth of sand since 2006 whereas there appears to be a net gain at Mortlake and Tennant Street indicating a south to north movement within the power beach zone from mean sea level to 50m offshore.

There was considerable loss of sand from the upper beach at Barraud Street between 2006 and 2013 as shown in Figure 11 (top panel). The changes at Mortlock Street (110 m north of the Jetty) are less dramatic than those at Barraud Street. The measured beach profiles (Figure 11, middle panel) show that the dune and upper beach face eroded between 2000 and 2006 but since then has accumulated some sand. There have been several changes in beach profile at Tennant Street (450 m north of the Jetty) indicating the movement of sand across and along the beach. The profile shows that the berm progressively receded by 5 m between 2000 and 2013. All three profiles generally show little sand movement below -4 m AHD.

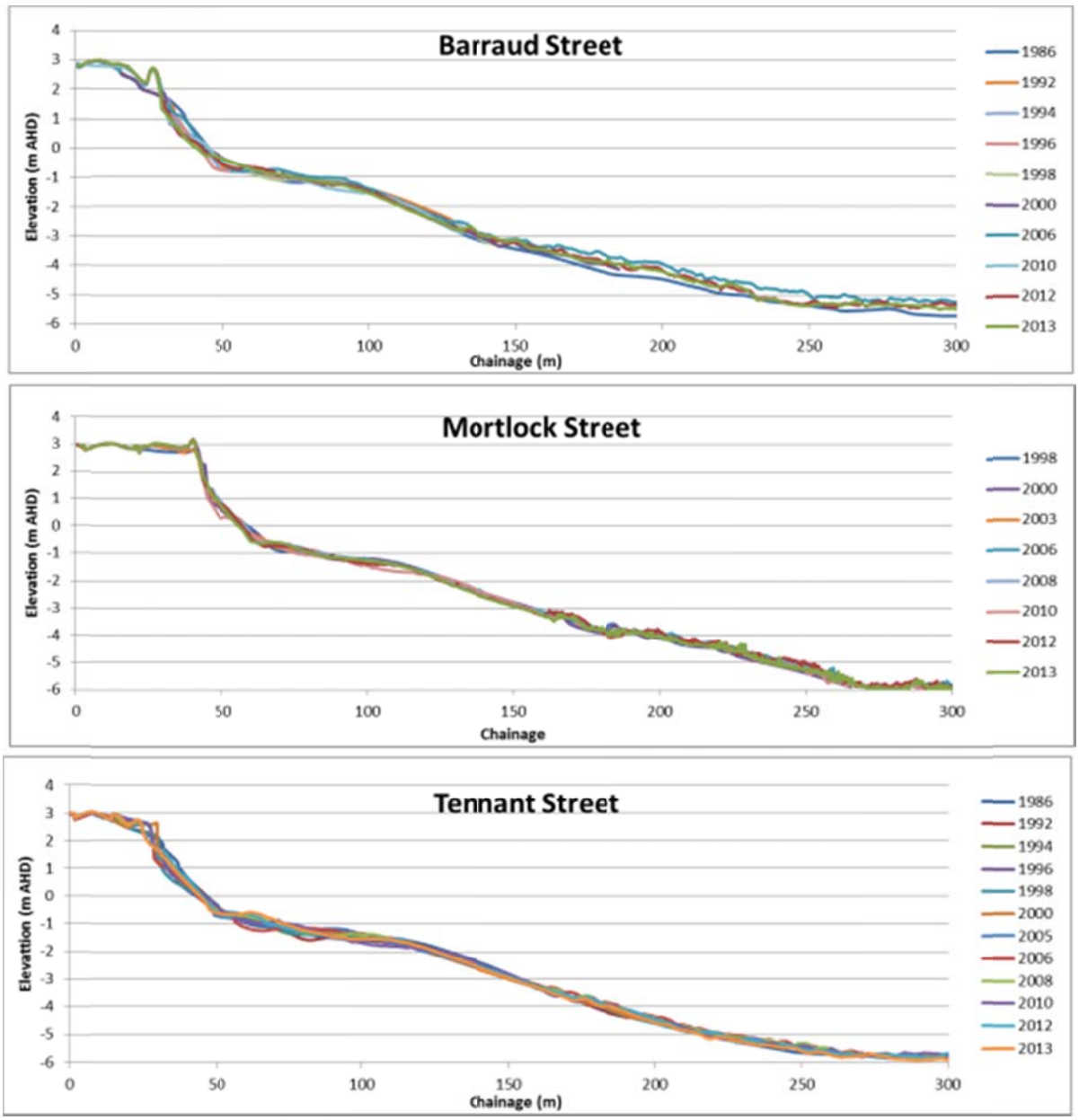

Figure 11 Measured beach profiles at Barraud Street (upper), Mortlock Street (middle), and Tennant Street (lower).

4.4 Tides, Storm Surge, and Sea Level Rise

Generally, the sea level comprises two components: 1) astronomical tide; and 2) storm surge. In addition to variations in sea level caused by the astronomical tides, the variations due to meteorological effects on the ocean surface are referred to as 'Storm Surge'. The wind blowing over the ocean and atmospheric pressure changes can also result in sea level variations, high pressure depresses the sea level and low pressure tends to result in elevated sea levels. The most damaging storm sea levels occur when the storm surges coincide with the high astronomical tide.

As illustrated in Figure 12, the most severe storm surge events typically occur when three factors combine:

- High astronomical tides
- Low pressure meteorological events (storm surge)
- Large waves generated by strong winds (wave set-up and wave run-up)

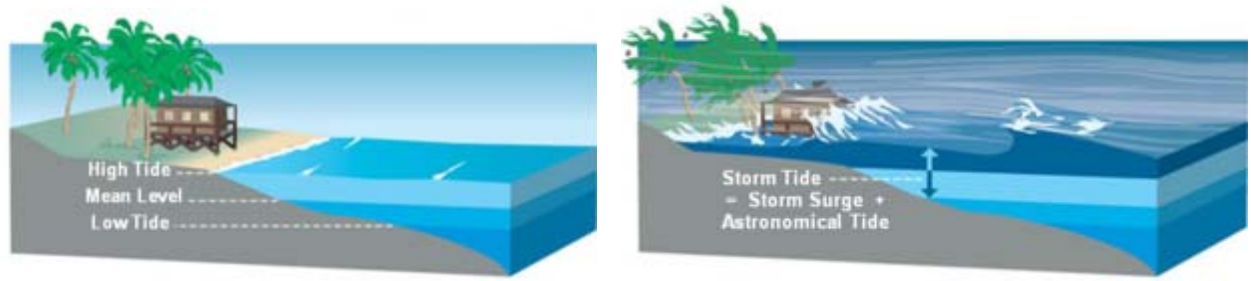

Figure 12 Illustration of how storm surges, astronomical tides and sea level rise influence coastal water levels.

A brief description of each of these phenomena in Spencer Gulf and at the study site is given below.

4.4.1 Tides (Ambient)

The flow in Spencer Gulf is mainly driven by tidal flow with tidal ranges of 2 m at Point Lincoln and increasing to 2.7 m in the northern Gulf. The tides in Spencer Gulf are remarkable in two major aspects. One is the large amplitude of the S_2 (diurnal) constituent and the other is the change in tidal character over the length of the gulf. Anomalous behaviour of the tides known as 'dodge tides' occur fortnightly where the contribution from M_2 (semi-diurnal) adds consecutively during a spring tide and then almost cancel each other during the following neap tide, thus leaving only the M_2 (diurnal) component producing only one tide per day. An example of the dodge tide occurrence at Adelaide Outer Harbour is shown in Figure 13. Extreme dodge tides occur in Spencer Gulf approximately every six months when both S_2 and M_2 cancel each other, resulting in extremely small tidal ranges (BHP, 2009).

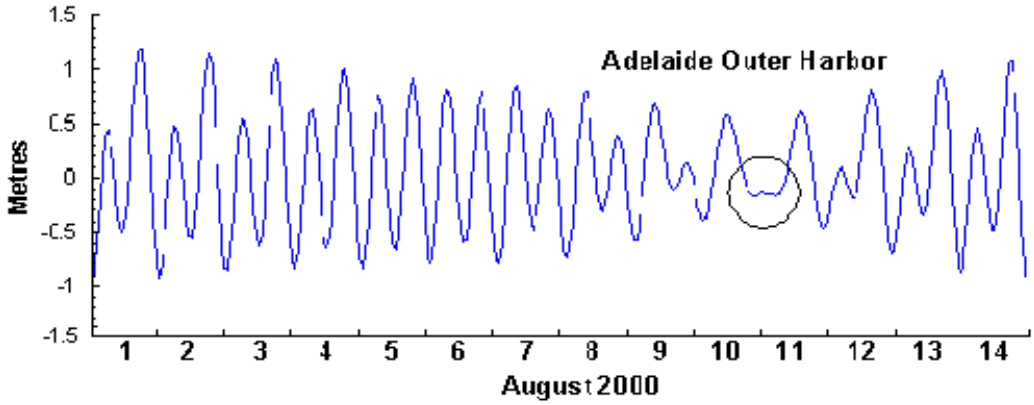

Figure 13: Dodge tides in South Australia

The tide gauge at Port Lincoln is the closest location (approximately 50 km south of Tumby Bay) from where tidal information is available. The tidal planes (Table 2) at Port Lincoln are similar to those at Tumby Bay.

Table 2: Tidal Planes at Port Lincoln (Australian National Tide Tables 2010).

Tidal Planes	m AHD	m CD
Highest recorded tide (source: Flinders Ports)	2.0	2.84
HAT	1.1	1.9
MHHW	0.7	1.5
MLHW	0.2	1.0
MSL	0.0	0.83
MHLW	-0.2	0.6
MLLW	-0.6	0.2
LAT	-0.8	0.0

During the storm of 23 June 2013, the highest tidal level was recorded as 1.37 m AHD (2.2 m CD) during high tide at the Port Lincoln tide gauge. The predicted high tide at Port Lincoln was 1.02 m AHD (1.82 m CD) on this day. This means that a storm surge of approximately 0.3 m was generated by this storm. This is significantly lower than the 100 year ARI water level of 1.95 m AHD. Also, the predicted tide did not exceed the HAT of 1.1 m AHD. The observed wind speed at 0900 on 23 June 2013 was 8.3 m/s (30 km/hr) from east reducing to 6.1 m/s (22 km/hr) by 3 pm turning east-south-east. Whilst this wind speed appears low, the 3-s gust on 23 June 2013 was recorded as 15.8 m/s (57 km/hr). The gust was higher at 18 m/s (65 km/hr) a day earlier on 22 June 2013. In comparison with extreme storm wind speeds of above 40 m/s for various average recurrence intervals (ARI) from the wind code (AS 1170.2), the wind speeds during the 23 June 2013 storm were relatively low but caused significant erosion and damage to infrastructure on the coast.

4.4.2 Extreme Value Analysis (storm tides)

Continuous recorded hourly sea level data over a period of several decades are required to estimate extreme sea levels. We obtained monthly tidal statistics derived from measured tidal data at Port Lincoln from the Flinders Port. The statistics consisted of mean, maximum, minimum, and standard deviation of recorded sea levels for each month from January 1964 until October 2013. The maximum recorded tide during this 48-year time period was 2.76 m CD or 1.93 m AHD in July 1964. This is less than 2.84 m CD recorded by Flinders Ports, which would have occurred prior to 1964.

Extreme value analysis using the maximum sea levels provided in the 48-year dataset was undertaken using the Coastal Engineering Design and Analysis software package. The software employs the Weibull distributions to extrapolate values to specified extreme events. Both the annual maximum and peak over threshold methods were used to identify the extreme storm events. The results from the analysis are presented in Table 3.

Table 3: Extreme sea levels (tide plus storm surge) at Port Lincoln

Extreme Event (year ARI)	Sea level (m AHD)
10	1.67 ± 0.05
25	1.77 ± 0.08
50	1.85 ± 0.10
100	1.95 ± 0.12

The information on sea levels provided by DEWNR is provided in Table 4.

Table 4: Estimates of current sea level components provided by DEWNR

Sea level components	m
100 year ARI sea level (tide plus storm surge)	1.95 m AHD
Wave set-up	0.2 m
Wave run-up	0.5 m
Total 100 year ARI water level today	2.65 m AHD

Flinders Ports records indicate that highest recorded tide of 2.01 m AHD (2.84 m CD) occurred when the predicted tide was 0.75 m AHD (1.58 m CD). This would indicate a storm surge of 1.26 m at Port Lincoln and Tumby Bay during this storm event.

4.4.3 Future Climatic Conditions

The sea level has been projected to rise by 1.0 m by 2100. However recent observations of faster than predicted melting of ice in the polar caps have led to increased sea level rise projections by 2100 to 1 -1.4 m. For this study, sea level rise of 0.3 m to 2050 and 1.0 m to 2100 has been adopted based on IPCC projections.

Whilst the changes in wind intensities and directions are not considered robust, CSIRO has undertaken studies that predict that extreme wind speeds will decrease in most parts of South Australia in summer and winter while these would increase in autumn in the north of the state (Government of South Australia, 2013).

The frequency of winter time low pressure systems is projected to decrease by about 20% in the vicinity of South Australia. We have made no allowance for a change in storm frequency or intensity.

2050 Projections

The 2050 projected sea level for the 100 year ARI storm event is computed as 2.95 m AHD. It includes the astronomical tide, storm surge, wave set-up, wave run-up, and sea level rise as shown in Table 5.

2100 Projections

The 2100 projected sea level for the 100 year ARI storm event is computed as 3.65 m AHD. It includes the astronomical tide, storm surge, wave set-up, wave run-up, and sea level rise as shown in Table 5

Adopted Design Water Levels

Table 5 100 year ARI Design Storm Tide Level

Time Horizon	Sea Level Rise (m)	Water Level (m AHD)
Today	0.0	2.65
2050	0.3	2.95
2100	1.0	3.65

4.5 Coastal Erosion and Recession

Coastal erosion refers to the erosion of beaches and cliffs due to waves, tides and storm surge; while shoreline recession is the long-term change in shoreline position due to waves, sea levels and sediment transport patterns. Both affect the safety of assets and the people living and working within the risk areas.

Coastal erosion can have both long and short term impacts. These include the loss of land as well as short term damage due to storm erosion. Coastal erosion can be caused by three different mechanisms:

1) Short-term storm erosion

   It is the combination of vertical erosion of the beach profile and the horizontal recession of the coastline that occurs during a storm event. During a large storm event, sand can be moved from the beach, to deeper water. Over time, the beach profile generally returns to the original configuration as sand is redistributed.

2) Long-term coastal erosion

   Long term coastal erosion refers to historical changes in the shoreline position where the shoreline is receding landward over time due to various natural and man-made processes. It includes longshore transport, the movement of sand parallel to the shore, induced by waves or currents running parallel to the coast line. This can be either erosion or deposition of sand from coastal processes such as tides, waves or currents.

3) Recession due to sea-level rise

   Increases in sea level can lead to erosion of unconsolidated sands. Bruun hypothesised that a beach assumes a profile that is in equilibrium with the incoming wave energy (Bruun, 1954) (Bruun, 1962) (Bruun, 1983), therefore, a rise in sea level would cause the beach and coastal profile to adjust.

$$R = \frac{S}{(h_c + B)/L}$$

$R = Shoreline\ recession\ due\ to\ sea\ level\ rise$

$S = Sea\ level\ rise$

4.5.1 Short-term Storm Erosion

Morphological response of the shoreline due to storm wave conditions occurs over relatively short periods of time (hours to days). This response primarily involves the erosion of the sub aerial beach face or storm cut through offshore transport and deposition near the storm wave break point to form an offshore bar. It is referred to as cross-shore or onshore-offshore transport. During a large storm event, sand can be moved from the beach, to deeper water. Over time, the beach profile generally returns to the original configuration as sand is redistributed.

To predict the erosion potential of the design profile as a result of extreme storm events, the computer modelling software SBEACH was used. SBEACH, short for Storm-induced Beach Change Model, was developed at the US Army Engineer Waterways Experiment Station (WES), Coastal and Hydraulics Laboratory, to calculate beach and dune erosion under storm water levels and wave action. SBEACH is an empirically based program that simulates beach profile change, including the formation and movement of major morphologic features such as longshore bars, troughs, and berms, under varying storm waves and water levels. The wave model is relatively sophisticated and computes shoaling, refraction, breaking, breaking wave re-formation, wave and wind induced set-up, set-down and run-up.

The SBEACH model requires the following data inputs to predict the short term storm erosion.

- Coastal elevation profile
- Sediment size
- Design storm, consisting of storm waves and water levels

The most recently measured coastal profiles of 2013 at Barraud and Mortlock Streets were selected as inputs into the model. The location of the profiles is shown in Figure 13 while the profiles are presented in Figure 10.

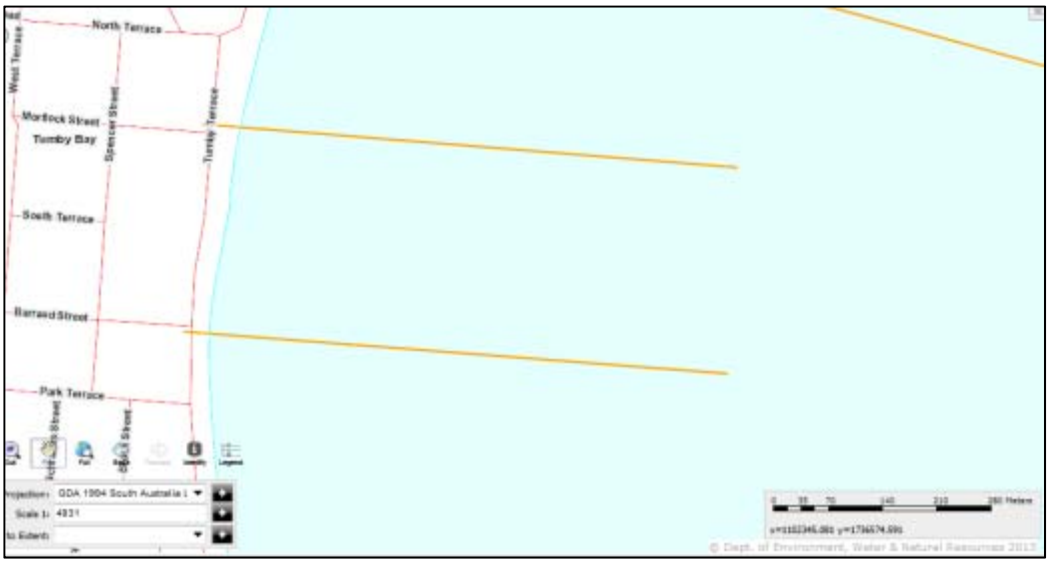

Figure 14 Map showing location of coastal profiles

The mean sediment size was obtained from Ozcoasts Smartline Maps (2013) as shown in **Table 6**. The mean for swash, berm, and dune were averaged and the resulting mean of 0.58 mm was used as the model input.

Table 6: Mean sediment sizes at Tumby Bay from Ozcoast Smartline Maps

Location	Mean sediment size (mm)
Swash (high energy)	0.97
Berm (lower energy)	0.48
Dune (windblown)	0.29
Average	0.58

A synthetic design storm representing a 100 year ARI storm was developed. A storm duration of 96 hours was assumed in which the wave height and wave period increase in a sinusoidal progression up to the peak storm design value after 48 hours and then reduce to non-storm conditions in the following 48 hours. The peak design wave and water level conditions for the 100 year ARI storm event from Table 1 and Table 5respectively were adopted. The design water level was composed of a spring tide superimposed with a design storm surge such that the maximum water level was the same as the 100 year ARI water level of 1.95 m AHD. The wave set-up and run-up was not included since the model computes these parameters.

Figure 14 shows the synthetic design storm.

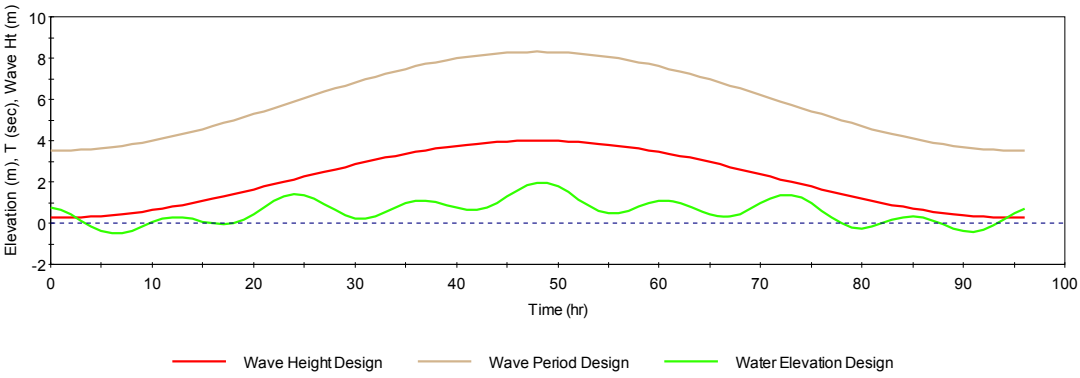

Figure 15 Synthetic 100 year ARI design storm

Whilst long-term recorded tide gauge data were available to estimate the 100-year ARI water level, the wave parameters for the 100 year ARI have been based on simple analytical functions. It is likely that the wave heights may be over-estimated because the effects of complex bathymetry or the offshore reef have not been considered in the model. Therefore, for sensitivity testing, the peak significant wave height of 4.1 m was reduced to 3.5 m and the SBEACH model was re-run to predict the storm erosion due to reduced wave heights.

Two different simulations for the following two scenarios were undertaken:

1) Assuming that there is no protection from an existing or proposed sea wall

2) Assuming that a seawall is present and does not fail under storm conditions

The maximum water surface elevations (including wave set-up) and the beach recession in response to the 96-hour 100 year ARI storm are presented in **Table 7** and **Figure 16** and **Figure 17**for the above two scenarios. The model predicts that the beach at the Barraud Street location may recede by up to 21 m if there is no seawall or if an existing sea wall affords no protection due to overtopping or damage. The beach recession is predicted to reduce to 6 m (Table 7) at this location if there is a seawall, which does not fail.

The maximum water level is predicted to reach 3.05 m AHD in response to the 100 year ARI storm. The model includes wave set-up and wave run-up. This estimate is higher by 0.4 m than the estimate of 2.65 m AHD provided by DEWNR. Both estimates use the same magnitude of storm surge and tide (1.95 m AHD). However, the wave set-up and run-up differs. The model computes the wave set-up and run-up based on 3.5 m and 4.0 m H_s and the measured beach profile.

Table 7: Maximum water surface elevations and beach recession due to 100 year ARI storm with no protection from seawall

	100-year ARI storm H_s = 4.0m		100-year ARI storm H_s = 3.5 m	
	Barraud Street	**Mortlock Street**	**Barraud Street**	**Mortlock Street**
Maximum water elevation	3.00 m AHD	3.05 m AHD	2.92 m AHD	3.05 m AHD
Recession at 2.5 m AHD	21 m	17 m	19 m	12 m
Recession at 2.0 m AHD	14 m	9 m	11 m	5 m

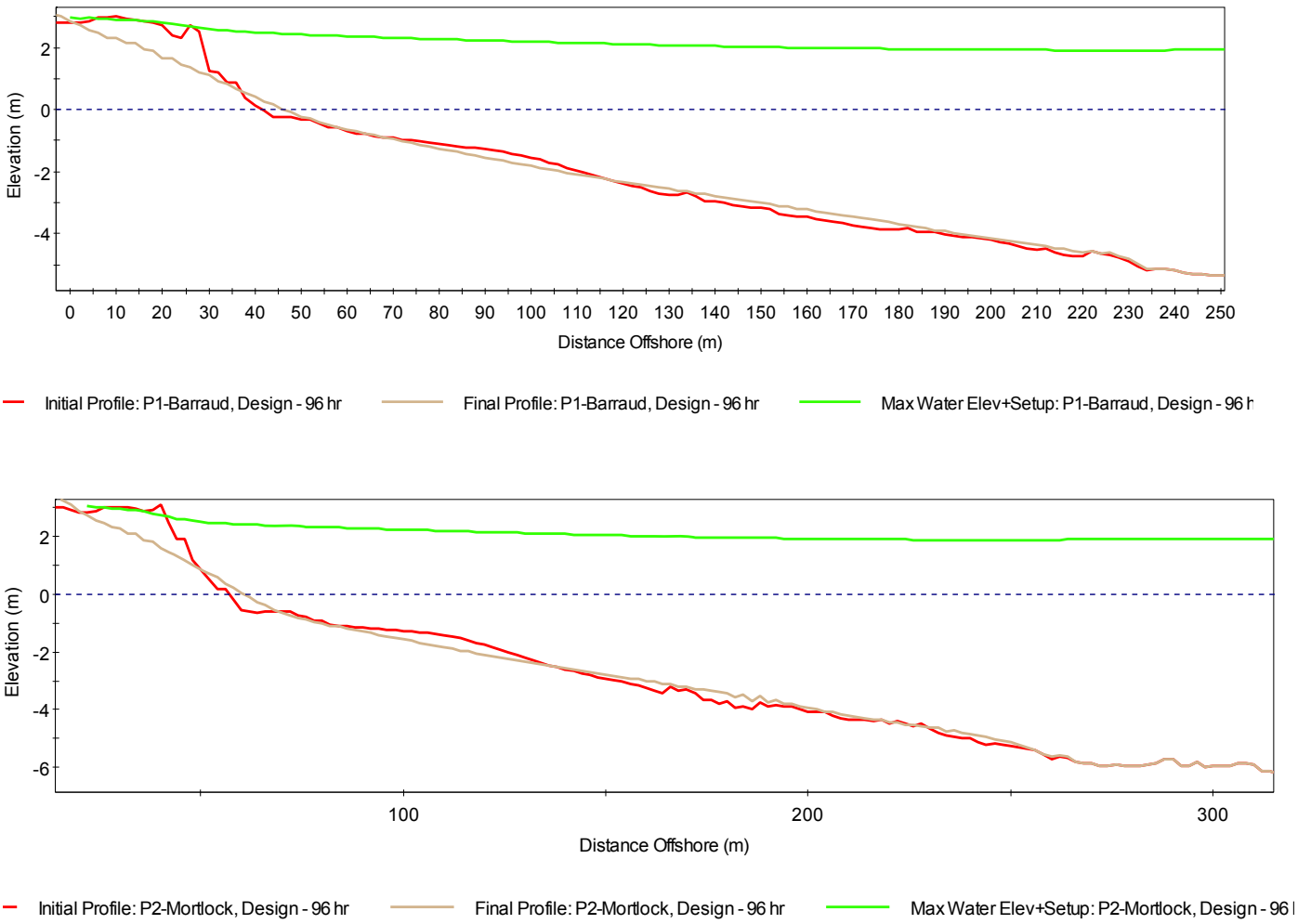

Figure 16 **Beach response due to 100 year ARI storm without protection from sea wall. Upper panel is profile at Barraud Street, and lower panel shows profile at Mortlock Street.**

Table 8: Maximum water surface elevations and beach recession due to 100 year ARI storm with protection from seawall

	100-year ARI storm H_s = 4.0m		100-year ARI storm H_s = 3.5 m	
	Barraud Street	Mortlock Street	Barraud Street	Mortlock Street
Maximum water elevation	2.60 m AHD	2.53 m AHD	2.50 m AHD	2.45 m AHD
Recession at 0.8 m AHD	6 m	2 m	6 m	2 m

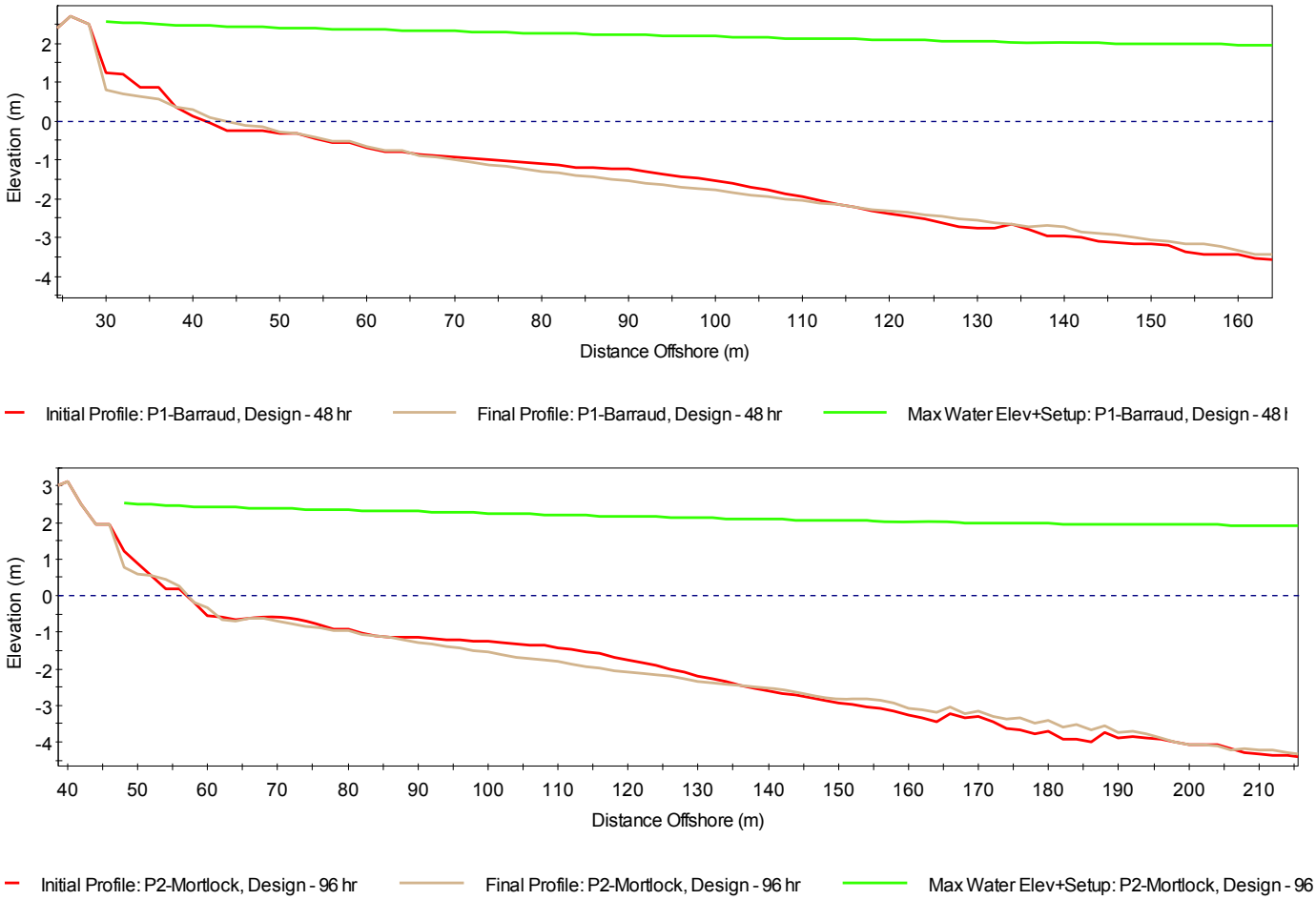

Figure 17 Beach response due to 100 year ARI storm with protection from sea wall. Upper panel is profile at Barraud Street, and lower panel shows profile at Mortlock Street.

4.5.2 Long Term Coastal Erosion (Historic Recession)

Historical changes in the position of shoreline indicate where the coast is receding and where it is accreting. Analysis of aerial images provide an indicative estimate of the historical movement where measured beach profiles are not available. Since measured beach profiles are available for the study area, these profiles were analysed to determine the historical recession.

Analysis of the beach profiles show that the shoreline position has fluctuated between -4 and 4 m from 1986 until 2013. The changes in the beach positions at MSL between successive years are shown in **Table 9** to **Table 12** for Barraud and Mortlock Streets. The average rate of change of shoreline indicates recession at MSL between 0.17 and 0.4 m per year. It should be noted that these recession rates have been caused by both natural processes and the human development works along the coast. Therefore, these rates of recession cannot be projected to the future.

Table 9: Changes in shoreline positions at MSL between successive Surveys at Barraud Street

	1992 – 1986	1994 – 1992	1996 – 1994	1998 – 1994	2006 – 1998	2010 – 2006	2012 – 2010	2013 – 2012
Change in shore-line position (m)	-1.5	-0.5	-1.4	+0.9	+1.0	+1.5	-2.2	-1.8
Rate of change/pa (m/a)	-0.25	-0.25	-0.70	0.45	0.13	0.38	-1.10	-1.80

Table 10: Changes in shoreline positions at MSL relative to 2013 at Barraud Street

	2013 – 1986	2013– 1992	2013 – 1994	2013 – 1996	2013 – 1998	2013 – 2006	2013 – 2010	2013 – 2012
Change in shore-line position (m)	-4.0	-2.5	-2.0	-0.6	-1.5	-2.5	-4.0	-1.8
Rate of change (m/a)	-0.15	-0.12	-0.11	-0.04	-0.10	-0.36	-1.33	-1.80

Table 11: Changes in shoreline positions at MSL between successive years at MortlockStreet

	2000 - 1998	2003 - 2000	2008 - 2003	2010 - 2008	2012 - 2010	2013 - 2012
Change in shore-line position (m)	-1.3	0.6	-0.3	-1.3	0.3	-1.5
Rate of change/pa (m/a)	-0.65	0.20	-0.06	-0.65	0.15	-1.50

Table 12: Changes in shoreline positions at MSL relative to 2013 at Mortlock Street

	2013 – 1998	2013– 2000	2013 – 2003	2013 – 2008	2013 – 2010	2013 – 2012
Change in shore-line position (m)	-3.5	-2.2	-2.8	-2.5	-1.2	-1.5
Rate of change (m/a)	-0.23	-0.17	-0.28	-0.50	-0.40	-1.50

From this data it can be gleaned that the typical long term rate of recession of the beaches in the study area is approximately 0.2 m/a.

4.5.3 Recession Due to Sea Level Rise

The recession due to sea level rise was computed using Brunn's Rule. Bruun's rule is defined as follows.

$$R = \frac{S}{(h_c + B)/L}$$

Where:

$R = Shoreline\ recession\ due\ to\ sea\ level\ rise$

$S = Sea\ level\ rise$

$B = Berm\ height$

$hc = Depth\ of\ closure$

$L = Length\ of\ active\ zone$

Utilising Bruun's Rule, it is possible to determine the potential erosion due to sea level rise. This takes into consideration the magnitude of the sea level rise and the profile of the beach. Application of Bruun's Rule is subject to certain assumptions and limitations, which include presence of unconsolidated sands, equilibrium profile and cross-shore transport only. Since the study area comprises of mainly sandy beaches, we have used Bruun's Rule to determine the estimation of recession due to future sea level rise.

The closure depth for all profiles was estimated from the measured beach profiles as well as from the SBEACH results. Both indicate that there is insignificant sediment movement below -4 to -5 m AHD. Therefore, -4 m was adopted as the closure depth (h_c). The berm height (B) at all profiles is 3 m, and the length of the active zone (L)

between the berm and the closure depth is 180 m for all three profiles at Barraud, Mortlock, and Tennant Streets. The recession due to sea level rise due to 2050 and 2100 conditions is given below.

4.5.3.1 2050 Climatic Conditions

The recession due to sea level rise (*S*) of 0.3 m and the profile conditions described above assuming no protection is provided, is 8 m.

4.5.3.2 2100 Climatic Conditions

The recession due to sea level rise (*S*) of 1.0 m and the profile conditions described above assuming no protection is provided, is 26 m.

4.5.4 Forecast Shoreline Movements

The measured rates of ongoing recession combined with forecast climate change induced recession and the impact of storm cut all need to be considered when estimating the beach condition for design purposes. The forecast horizontal shoreline movements can be assessed as:

$$\Delta = \Delta_{SC} + \Delta_R + \Delta_{SLR}$$

Where:

Δ_{SC} = Shoreline movement due to Storm Cut ≈ -10m

Δ_R = Shoreline movement due to ongoing Recession ≈ -0.2m/a → 8m in 2050 or 18m in 2100

Δ_{SLR} = Shoreline movement due to Sea Level Rise ≈ -8m in 2050 or 26m in 2100

4.5.4.1 Design Erosion Allowance Today (2013)

For today only storm cut needs to be considered. This equates to a horizontal movement in the upper profile of 10m. When imposed on the typical beach profile, with a beach level of +1m AHD at the seawall this equates to a peak storm beach level at the seawall of approximately +0.5m AHD

4.5.4.2 Forecast Design Erosion for 2050 (0.3m Sea Level Rise)

The design forecast erosion in 40 years from now is 26m. When imposed on the beach profile this equates to beach level at the seawall of approximately 0m AHD.

4.5.4.3 Forecast Erosion 2100

The design forecast erosion in 90 years from now is 46m. In estimating this value a simple combination of the sea level rise and ongoing recession is overly conservative, and as such their contribution to the total recession has been discounted by 20%. When imposed on the beach profile this equates to beach level at the seawall of approximately -1m AHD.

The impact of the future recession is shown in Figure SK004 Appendix C.

4.6 Projected Wave Spectrum and forces on coastal protection structures

The design wave for a 100 year ARI event is H_s of 4.1 m, T_p of 8.3s, at a water depth of 6 m. However, the wave climate at the foreshore and the seawall is controlled by the depth of water and bathymetry (slope) over which the waves approach. The adopted design beach levels are described above, which, when combined with a bed slope of 1 in 40 yields design wave spectrum (Beta-rayleigh) as set out in **Table 13**.

Table 13: Design Wave Spectrum at Foreshore (design 100 year ARI event)

	2100 Conditions (SWL 3.65m AHD)	2050 Conditions (SWL 2.95m AHD)	2013 Conditions (SWL 2.65m AHD)
Design Beach Level	-1.0m AHD	0.0m AHD	0.5m AHD
H_{mo}	2.61m	1.72m	1.27m
$H_{1/10}$	3.00m	1.97m	1.46m
$H_{2\%}$	3.17m	2.08m	1.54m
$H_{1\%}$	3.32m	2.18m	1.61m
$H_{0.1\%}$	3.71m	2.44m	1.81m

The design wave force (P1) for a vertical wall with the 2100 conditions is 219 kN/m

4.7 Predicted Wave Overtopping Rates

The predicted wave overtopping rates for various protection structures assuming crest heights of 2.6m AHD, 3.3 m AHD, 3.6 m AHD and 4.0 m AHD under present (today), 2050 and 2100 wave climate conditions are summarised in the **Table 14** to **Table 16**. The overtopping formulations (Owen, 1980) from the CIRIA Rock Manual have been used to compute the overtopping discharges. The same formula has been applied to all cases. This ensures consistency among the various structural options. There are several overtopping formulae available, which would yield varying results. The Owen formula is applicable mainly to sloping structures. However, for low crest heights and large discharges, the slope angle is not important (CIRIA, 2007). Tests have shown that the overtopping discharge for slopes of 1:1 and 1:2 are nearly the same.

Table 14 Wave Overtopping – Design Conditions Today

Wave climate	Q (m^3/s/m) 2.6m AHD Wall Crest	Q (m^3/s/m) 3.3m AHD Wall Crest	Q (m^3/s/m) 3.6m AHD Wall Crest	Q (m^3/s/m) 4.0m AHD Wall Crest
Rock or Geobag seawall 1V: 1.75H	Crest Height<SWL	0.230	0.137	0.069
Vertical concrete seawall	Crest Height<SWL	0.340	0.257	0.178
Sloping concrete seawall1V: 1.5H	Wall Crest Height<SWL	0.377	0.287	0.200

Table 15 Wave Overtopping - Projection Design Conditions 2050

Wave climate	Q (m^3/s/m) 2.6m AHD Wall Crest	Q (m^3/s/m) 3.3m AHD Wall Crest	Q (m^3/s/m) 3.6m AHD Wall Crest	Q (m^3/s/m) 4.0m AHD Wall Crest
Rock or Geobag seawall 1V: 1.75H	Crest Height<SWL	0.568	0.364	0.201
Vertical concrete seawall	Crest Height<SWL	0.630	0.497	0.362
Sloping concrete seawall1V: 1.5H	Crest Height<SWL	0.704	0.556	0.407

Table 16 Wave Overtopping - Projection Design Conditions 2100

Wave climate	Q (m^3/s/m) 2.6m AHD Wall Crest	Q (m^3/s/m) 3.3m AHD Wall Crest	Q (m^3/s/m) 3.6m AHD Wall Crest	Q (m^3/s/m) 4.0m AHD Wall Crest
Rock or Geobag seawall 1V: 1.75H	Crest Height<SWL	Crest Height <SWL	Crest Height<SWL	0.950
Vertical concrete seawall	Crest Height<SWL	Crest height <SWL	Crest height <SWL	1.000
Sloping concrete seawall1V: 1.5H	Crest Height<SWL	Crest height <SWL	Crest height <SWL	1.124

The analysis reveals that

- The overtopping exceeds limits of mean discharges of 0.05 to 0.2 m^3/s/m recommended by the EurOtop Manual (2007) when there is adequate protection to crest and rear slope for all cases presented above and hence confirms under current conditions damage to grassed areas and paving would occur. This is consistent with the observations which show that the current sea wall has been overtopped during severe storms and damage has occurred. The implications of these overtopping rates on existing infrastructure are indicated in Table 18.

The EurOtop recommendations are given in Table 17. The overtopping rates are indicative and should not be considered as the major design factor but taken into account in conjunction with the design of the crest, slope and type of structure.

Table 17 Limits for overtopping for damage to crest or rear slope (from EurOtop Manual, 2007)

Hazard Type and Reason	Mean Discharge Q (m3/s/m)
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	0.05-0.20
No damage to crest and rear face of grass covered embankment of clay	0.001-0.01

Hazard Type and Reason	Mean Discharge Q (m3/s/m)
No damage to crest and rear face of embankment if not protected	0.0001
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	0.20
Damage to grassed or lightly protected promenade or reclamation cover	0.05

- Setting a crest height at existing top of wall levels irrespective of the treatment will result in significant inundation and erosion of the foreshore reserve and Tumby Terrace and would be below the 2050 design level of 2.95m AHD and therefore the crest height of any protection structure would need to be set above existing levels;
- The greater the freeboard above the design wave level the lesser the wave overtopping rate;
- Setting crest heights at 3.3m AHD and 3.6m AHD still results in significant wave overtopping under 2050 climatic conditions and hence some rear wall protection (eg a footpath and drainage) would be required in order to provide long term durability and control rear wall surcharging;
- Setting crest heights above the 3.65m AHD 2100 design level would be more suitable however this would come at a higher capital cost.

4.8 Current Wall Failure Modes

Based on the data review photographs above analysis and eye witness accounts, the key failure made is considered to be excessive surcharge loads on the rear (land side) of the wall panels caused by saturation of the backfill material under wave overtopping, the absence of a vertical key and insufficient heel width to resist these lateral loads, damage to grassed and unpaved areas, and rapid loss of sand from beneath foundations from the saturated conditions leading to slumping of the backfill and outward rotation of the panels.

The height of the foundation nominally at+ 0.6 to 0.7m AHD and slope of the beach are also considered to be major factors as the foundation sits within the intertidal wave run up zone and waves and sand can retreat rapidly and hence the foundations should be located close to the historical medium low water level (MLWL) of -0.6mAHD

5.0 Community & Stakeholder Consultation

5.1 Initial Community Group Engagement

An initial discussion forum was held with representatives from key community groups at Council's municipal offices in Tumby Bay on 24 October 2013. The meeting was attended by Doug Bowers of AECOM and Damien Windsor and Gary King from Council.

Community Representatives in attendance included:

- David Dupree;
- John James-Tumby Bay Caravan Park & Progress Association
- Kym Mason-Tumby Bay Lions Club
- Bronte Gregurke-Tumby Bay Progress Association and Former Foreshore Committee member
- Sally Curtis-Long term resident

A summary of key comments is listed below.

- The foreshore and beach abutting the Town Centre prior to the 1970's was a natural low lying vegetated sand dune shoreline with playground equipment, car parking and a café.
- Major storms occurred in the 1930's,1940's, 1970's, 1989, 2001, 2012 and 2013
- In the 1970's lawn was established by the Progress Association and vegetation cleared to improve the usage of the foreshore area.
- Area north of the jetty and the Lions Park is heavily used by the community for swimming and passive recreation and dining activities
- The Norfolk Island pines and grassed areas are very important community assets
- The Ritz café and the associated car park probably the most important asset because of trade
- Beach width has declined since vertical wall panels installed
- Main jetty has never been overtopped
- The channel deepening for the boat harbor has altered sand movement at south end of beach abutting shacks during tide movements-suction effect moving sand back south.
- The coastal edge was more sloped and eroded more frequently before vertical walls installed typically back as far as the existing Norfolk Island Pines
- In 2001 a Foreshore Committee comprising community and Council representatives was formed to develop potential foreshore protection solutions which included trial sand bag groynes and a rock revetment wall. The committee was disbanded in 2003.
- Stormwater discharge is causing localised beach erosion
- Largest amount of recession has occurred around the jetty and further north
- Don't want to lose any more beach width within study area
- Former lower jetty adjacent to the Ritz broke some of the wave energy from the north east
- Reefs adjacent to Tumby Island and islands further south east impact on strength of waves from south east
- Vertical wall did not work
- Community division on whether rock revetment is suitable
- Natural vegetated solution similar to near hospital might be better
- No active Dune Care or Coast Care Groups in Tumby Bay
- Sand movement regularly blocks stormwater outlets

- Can get localised wind direction changes around Tumby Island when south westerly winds can swing around to the east
- There is a significant sand bar between Tumby Island and the main land and can be walked across in low tide, possibly trapping south to north offshore sand movement

5.2 DEWNR Engagement

5.2.1 Initial Engagement 20 October 2013

A meeting was held between Doug Bowers from AECOM and George Hadji, Murray Townsend, Guy Williams and Sharie Ditmar from the Coastal and River Murray Unit of the Department of Environment, Water and Natural Resources (DEWNR) at the offices of DEWNR 1 Richmond Road to discuss key issues receive historical and profile information and expectations in terms of documentation for review and approval.

The outcomes of the meeting are summarised below:

- The bay is located in a low wave energy section of coast
- North easterly winds would generate the largest waves due to the longer fetch.
- The reefs and islands to the south east would refract waves generated from the south and south east.
- In the 1970's and 1980's the Tumby Bay coast was identified as an erosion area and hence DEWNR began to monitor recession and accretion at five profile locations.
- The coast has been receding particularly south of the jetty estimated 10m in 35 years with a net movement of sand from south to north and sand loss from the beach a major community issue.
- Key design parameters
    - 1 in 100 year ARI storm surge-1.95m AHD
    - Wave set up-0.20m
    - Wave run up-0.50m
    - 2050 sea level rise- 0.3m
    - 2050 beach level - 0.0m AHD
    - 2050-design crest level-2.95mAHD
    - 2100 sea level rise-1.0m
    - 2100 beach level - -1.0m AHD
    - 2100 design crest level-3.65mAHD
- Consider using Port Lincoln & Whyalla tidal data
- Potential protection options to consider
    - Could try a variety of innovative treatments given a low energy site including geotextile sand walls
- DEWNR would like to:
    - Discuss treatment options developed in the concept design report and to review the report;
    - Review calculations assumptions and geotechnical investigations at detailed design stage

5.2.2 Further Engagement 12 December 2013

Discussions were held between Doug Bowers and Saima Aijaz from AECOM and George Hadji and Sharie Ditmar from the Coastal and River Murray Unit of the Department of Environment, Water and Natural Resources (DEWNR) at the offices of DEWNR 1 Richmond Road to discuss the draft report.

The findings and outputs were accepted and it was considered that a rock and /or Geobag wall installed at a height of 3.3m AHD designed for 2050 wave conditions would provide appropriate and cost effective affordable levels of protection.

It was also noted that sea level had risen by an average of 4.2mm/pa at the Port Stanvac tide gauge over the past 20 years and was now rising at 7mm pa.

5.3 Council Staff Engagement

Discussions were held with Damian Windsor-Works Manager and Gary King-Works Supervisor to ascertain the history of storm events and repair works undertaken.

Key comments are summarised below:

- Southfront are currently preparing a Stormwater Management Plan for the township which may impact on existing coastal stormwater outlets.
- SARDI are currently preparing a receding waters risk assessment which may inform this study
- Council commissioned Design Flow in 2003 to undertake a Coastal Management Plan
- The beach width was much wider and there has been a significant loss of sand over recent years.
- The vertical wall adjacent to the Ritz café was overtopped and moved during the 23 June storm surge event this year. Sand in front of the wall washed away within 24 hours and the backfill subsided. Council officers witnessed a significant amount of water flowing out of the precast panel weepholes and surcharging behind the wall coupled with loss of sand in front of the wall lead to the wall rotating.
- Council placed large sand bags on the seaward side of the panels to help stabilise the panels with funding assistance from the South Australian Coast Protection Board and technical assistance from DEWNR officers.
- Up to 1m of sand had accreted against the wall within 1 month indicating considerable sand movement in the winter months.
- Most of the storm surges and seagrass build up occur in the winter months primarily from the east and north east which is also when higher tides occur.
- A smaller lower level jetty existed adjacent to the Ritz café which was removed due to severe and regular storm damage and erosion.
- Council is looking for a long term solution that could be staged.
- The coastal edge was more gently sloped prior to the wall panels being installed and eroded frequently up to the current position of the Norfolk Island Pines. Some infrastructure including a playground and car park were lost due to coastal erosion prior to the wall panels being installed. Council estimates that up to 4m of erosion occurred over 20 years.
- A sand bar is located 20-30m from the mean water mark.
- Run off from Salt creek discharges into the bay to the north of the township and plumes travel south.
- Less sand available in the near shore region than in the past.
- Council considers the main reason the wall has failed is because the foundation was installed too shallow and hence is regularly undermined.
- There is a large granite quarry and local contractors in Pt Lincoln which can supply and install rock sea wall material if needed. A similar wall was constructed to 3m AHD in North Shields approx 5km south of Pt Lincoln airport
- Many of the lower levels stormwater outlets get silted up regularly with sand and is a major maintenance problem

5.4 DPTI Marine Facilities Branch

Discussions were held with Mr Mike Cooney from the Marine Facilities Branch to gain an understanding of any previous wave overtopping of the jetty.

The advice received was that DPTI has no knowledge of this jetty being overtopped or damaged by a storm event.

The deck level of the existing Tumby Bay Jetty (the" New Jetty" as it is known) is 3.21m AHD which would only slightly be overtopped in an extreme event with wave height exceeding 2.5m.

In comparison, the" Old Tumby Bay Jetty" (since demolished) was significantly damaged by storms on more than one occasion. (Deck level 2 29m AHD).

5.5 Final Community Engagement

Further community engagement was undertaken in the form of a public meeting and flyer in the local newspaper and council newsletter outlining the key findings options considered and recommendations in January and February 2014 to gauge community views.

A presentation was made to the community and elected members on 18 February 2014 in Tumby Bay and 16 people including 6 elected members attended.

A further briefing and discussions was held with elected members and council executive staff on the 19 February 2014.

All treatment options outlined in this report were presented together with plans showing the impact with two potential alignments.

An attendance sheet flyer and questionnaire was provided at the meeting for the public to respond with a preferred treatment option and additional comments.

The community views are summarised below:

- There was general acceptance that protection was required to reduce the threat of erosion and flooding;
- Views were mixed on which protection option was most preferred;
- One resident suggested that an extension of the existing marina breakwater further north and/or extending the offshore reef option for the entire length of the bay should be considered;
- One couple suggested a precast vertical concrete wall as a better option because of the reduced impact on foreshore infrastructure;

6.0 Potential Protection Options

Given the coastal processes assessment, existing foreshore infrastructure and likely wave energy under sea level rise, a vertical concrete wall if considered as a potential option would need to be installed with a vertical key within the foundation below the water mark to provide passive resistance and the toe would need to be protected with toe stones. Accordingly it would need to be precast in short <2.5m sections and trucked to site and significant dewatering would be required in order to construct such a structure and a higher crest level higher than a rock wall would be necessary to reduce wave overtopping to reasonable levels.

A concrete stepped sloping wall similar to Wallaroo is not considered appropriate at Tumby Bay due to the limited beach width available and the visual impact.

More permeable options that tolerate some movement minimise future erosion and can be increased in height in the future are considered more appropriate.

An assessment of the options based on indicative whole of life cost, infrastructure and erosion impacts and wave overtopping rates is summarised in Table 10.below.

Table 18 Potential Coastal Protection Options Comparison

Treatment Option	Discounted Whole of Life cost $/m	2050 wave conditions overtopping rates (m^3/s/m)	Infrastructure and erosion/recession impacts	Staging Potential
Option1-Rock sea wall (1V:1.75H) crest height 3.3m AHD-Long life solution >20 years	1415	0.57	Coastal edge of top of wall set at 3.5m inland or 0.5m seaward of the existing concrete wall alignment, 6.75m - 3.75m encroachment	Good

Treatment Option	Discounted Whole of Life cost $/m	2050 wave conditions overtopping rates (m^3/s/m)	Infrastructure and erosion/recession impacts	Staging Potential
			onto reserve or beach depending on wall position. Refer Figures SK001-SK006 Appendix C. Minimal storm erosion potential and low maintenance. Relocation of coastal paths and beach access ramps and fencing, part of playground, part of Lions park, and potential loss of Pines. Reduction and reworking of war memorial and Ritz car parks .Potential reduction in beach width. Stormwater outlets to be reconnected	
Option 2-Elcorock seawall 2.5m3 bags (1V:1.75H) crest height 3.3m AHD Medium life solution 8-10 years	3980	0.57	7.4m encroachment into reserve or beach depending on wall position. Some storm erosion potential and increased maintenance. Relocation of coastal paths and beach access ramps and fencing, part of playground, part of Lions park, and potential loss of Pines. Reduction and reworking of war memorial and Ritz car parks. Potential reduction in beach width. Stormwater outlets to be reconnected potential for erosion at headwalls. Bags would need to be replaced very 8-10 years	Good
Option 3-Sloping concrete sea wall (1V:1.5H) crest height 3.3m AHD. Long life solution >20 years	3050	0.70	5.0m potential encroachment onto reserve or beach depending on wall position. Minimal storm erosion potential and low maintenance. Potential partial loss or relocation of coastal paths, beach access ramps, part of playground, part of Lions park, Pines, and playground. Potential for damage to paths and lawn areas due to wave overtopping. Reduction and reworking of war memorial and Ritz car parks	Good

Treatment Option	Discounted Whole of Life cost $/m	2050 wave conditions overtopping rates (m^3/s/m)	Infrastructure and erosion/recession impacts	Staging Potential
Option 4-Sand Nourishment -sand and wrack bund wall to 3.3m AHD in front of wall Short life maintenance only solution	1260-1423		8m encroachment onto beach Sand would require replacement annually and seasonally 46m long term erosion by 2100.	Good
Option 5-Do Nothing-Retain existing precast panels with Elcorock bags in front up to Ritz Car park. No further treatment abutting Ritz car park Short to medium life solution 5-10 years.			21m storm erosion abutting Ritz car park. Erosion and path damage behind wall due to wave overtopping. Damage to beach access ramps. >26m recession by 2100 into reserve. All reserve infrastructure up to Tumby Terrace east kerb at risk in 1 in100 storm surge event after wall fails. All reserve and road infrastructure up to west edge of Tumby Terrace carriageway at risk by 2100.	N/A
Option 6-Vertical precast concrete wall 3.6m AHD crest height	1600	0.50	2.0m potential encroachment onto reserve or beach depending on wall position. Minimal storm erosion potential and low maintenance. Potential partial loss or relocation of coastal paths, beach access ramps, part of playground, part of Lions park, Pines, and playground. Potential for damage to paths and lawn areas due to wave overtopping.	Good
Option7-Offshore rock Reef constructed 200m offshore with crest at 3.0m AHD	3100	0.70	30m width loss of sea grass. Potential EIS required and may not be supported by the EPA. Rock reef would be visible at all times. Potential reduction in sand supply to the existing beach	Good

7.0 Conclusions and Recommendations

7.1 Conclusions

From the investigation the following conclusions can be drawn:

- The existing concrete seawall has failed on several occasions due to excessive wave overtopping causing saturation of the backfill damage to unpaved areas and horizontal displacement of the wall panels from surcharge loading and resultant loss of sand from the shallow foundation.
- The recent placement of Elcorock sand bags in front of the wall panels is likely to provide some lateral stability to the panels in the short term under wave forces but is unlikely to reduce storm based erosion behind the wall under wave overtopping conditions.
- The level of the existing top of wall level and foreshore reserve is below the 2050 design level of 2.95mAHD and hence the foreshore reserve and Tumby Terrace will experience an increased amount of inundation and erosion with sea level rise and hence any new protection structure would need to installed at a higher level.
- Total coastal recession is estimated at 26m by 2050 and 46m by 2100 if no protection is provided as shown in figure SK005 in Appendix C.
- If no protection is provided, a significant proportion of the existing above ground assets and underground services on the foreshore reserve, the beach and Tumby Terrace would be at risk of damage and inundation from wave overtopping and erosion.
- The Elcorock seawall installed in 2005 has provided reasonable protection from erosion and wave overtopping.
- There appears to have been a net movement of sand from south along the coast and the beach within the study area.
- The upper beach profile is quite steep which is exacerbating wave run up wave overtopping and sand loss on the upper beach zone.
- There is the potential for wave heights up to 3.5- 4m being generated in Spencer Gulf adjacent to Tumby Bay under easterly and north easterly winds due to the long fetch.
- The beach level adjacent to the existing wall could be potentially cut to -1m AHD under 2100 climatic conditions.
- Any engineered protection structure would need to install with a foundation level set at or below -0.7m AHD.
- A rock revetment wall or Elcorock revetment wall installed with a crest height of 3.3AHD a foundation level of -0.7m AHD and a sloping face of 1V: 1.5 1.75H would afford a reduced level of wave overtopping compared to a vertical or sloping concrete wall.
- Setting the alignment of the coastal edge of a rock revetment wall at 3.5m inland from the existing concrete wall position would have a significant impact on existing foreshore infrastructure and trees including the existing public toilet but would retain the current beach width.
- Setting the alignment of the coastal edge of a rock revetment wall at 0.5m seaward from the existing concrete wall position would have a moderate impact on existing foreshore infrastructure and trees and would retain the public toilet but would reduce the current beach width by approximately 3m.
- A rock revetment wall would be more durable then Elcorock and would potentially encourage sand accretion and vegetation (pig face and samphire) propagation.
- A rock revetment wall would have lower whole of life costs compared to an Elcorock wall due the need to replace the geotextile bags every 10 years.
- A rock seawall is estimated to cost $306,000 for the 340m length excluding the cost of the necessary modifications to paths fences beach access ramps and stormwater outlets based on using local contractors and rock sourced from a local granite quarry. It would be amenable to be constructed in stages to suit funding and to be constructed so that future height increases can be accommodated.

- A Elcorock seawall is estimated to cost $705,000 for the 340m length excluding the cost of the necessary modifications to paths fences beach access ramps and stormwater outlets based on using local contractors and sand sourced from a local borrow area or quarry.
- A sloping concrete wall is estimated to cost in excess of $1Million for the 340m length excluding the cost of the necessary modifications to paths fences beach access ramps and stormwater outlets based on using local contractors, and would need to be precast with a key but would be difficult to construct due to the required foundation level at -0.7mAHD and associated groundwater issues.
- A vertical precast concrete sea wall is estimated to cost $530,000 for the 340m length but would result in excessive wave overtopping rates even if installed with a crest height at 3.6m AHD, however is likely to have less impact and encroachment on foreshore and beach infrastructure.
- Sand nourishment through provision of a sand bund is likely to result in potentially lower whole of life costs than other structures however regular maintenance and importing of sand would be required after storm and seasonal spring tide events due to erosion and could result in reduced beach width particularly if placed in front of the existing concrete wall. It would be largely ineffective in resisting long term recession. Obtaining sand from the nearby coastal dunes may be difficult due to the potential impacts on the town CWMS irrigation area.
- An offshore rock reef placed 200m from the existing shoreline with a crest height of 3m AHD is estimated to cost $1,120,000 for the 340m length and may not be supported by the EPA due to the impact on a 30m width of seagrass and would also potentially reduce sand supply to the beach.
- Extending the marina access breakwater further north would have little impact on reducing wave energy generated from the damaging eastern and north easterly winds and may potentially reduce sand supply to the beach abutting this main foreshore.
- Additional hardstand (eg 1.5-2m wide path) and drainage infrastructure will be required on the land side of any protection structure in order to effectively drain any overtopping water and minimise the potential for surcharging the backfill material.
- Modifications to foreshore infrastructure including the playground picnic areas fencing carparks underground irrigation and stormwater would be required to accommodate a replacement sea wall structure.
- Protection of the coast is the only viable option to protect vital foreshore infrastructure and raising the level of any protection structure above existing wall levels is needed from an inundation perspective under sea level rise.

7.2 Recommendations

In consideration of the above analysis it is recommended that:

- The existing concrete panel seawall be removed;
- A replacement rock seawall with a slope of 1V:1.5H foundation level of -0.7m AHD crest level of 3.3m AHD crest width of 2m and 2Tonne toe stones be installed with the coastal edge of the crest located 0.5m seaward of the existing concrete wall alignment for the length of the study area as per Option 2 indicated on figures SK003 & SK004 in Appendix C;
- The seawall be constructed in two stages with Stage 1 comprising the section from the north edge of the Ritz carpark to the south edge of the existing Elcosol bag wall and Stage 2 south of this point;
- A 1.5-2m width concrete footpath with a longitudinal spoon drain be provided on the foreshore reserve behind the seawall together with grated sump pits and 150mm PVC stormwater outlets at regular intervals;
- A masterplan be prepared to inform the redevelopment of the foreshore area behind the new seawall;
- The number of beach access ramps north of the jetty be reduced from four to three with the one adjacent to the public toilet removed;
- New in-situ concrete beach access steps with handrails be installed down the proposed rock wall from the existing reserve to the beach at the following locations:
    - North edge of the playground;

  - South edge of the Ritz carpark.
- A DDA compliant hardwood timber ramp be provided from the north side the jetty to the beach;
- Stormwater drainage at the Ritz carpark be rationalised and improved and a Gross pollutant trap or Bio-filtration bed be installed;
- Tidal flaps headwalls and ’renomat’ erosion control be provided at stormwater outlets.

8.0 Glossary of Terms

Glossary

AHD	Australian height datum; 0.83 m above LAT at Tumby Bay
ARI	Average recurrence interval
CD	Chart datum. This is generally based on a local determination of Lowest Astronomical Tide (LAT) and ideally CD and LAT should coincide.
Fetch	Distance over which the wind blows over water
HAT	Highest Astronomical Tide. The highest level of tide that can be predicted to occur under average meteorological conditions and under any combination of astronomical tides.
H_b	Breaking wave height
h_c	Depth of closure is the depth below which the change in sediment transport is minimal.
H_{max}	Maximum wave height in a specified time period.
H_{mo}	Significant wave height based on the zeroth moment of the wave energy spectrum (rather than the time domain $H_{1/3}$ parameter)
H_s or $H_{1/3}$	Significant wave height is the average wave height of the highest third of a set of waves. It corresponds well with the visual wave heights observed by human observers.
$H_{1/10}$	Average of the highest one-tenth of the waves.
$H_{2\%}$	Wave height exceeded by 2% of the waves.
$H_{1\%}$	Wave height exceeded by 1% of the waves.
$H_{0.1\%}$	Wave height exceeded by 0.1% of the waves.
IPCC	Intergovernmental Panel on Climate Change
LAT	Low Astronomical Tide. The lowest level of tide that can be predicted to occur under average meteorological conditions and under any combination of astronomical tides.
MHHW	Mean higher high water. The mean of the higher of the two daily high waters over a long period of time (usually 18.6 years).
MLHW	Mean lower high water. The mean of the lower of the two daily high waters over a long period of time (usually 18.6 years).
MSL	Mean sea level. The average level of the sea surface over a long period, preferably 18.6 years.
MHLW	Mean higher low water. The mean of the higher of the two daily low waters over a long period of time (usually 18.6 years).
MLLW	Mean lower low water. The mean of the lower of the two daily low waters over a long period of time (usually 18.6 years).
Longshore Transport	The movement of sand along the coastline caused by waves arriving oblique to the coast and a wave-generated current running parallel to the beach.
SBEACH	Storm induced Beach Change model

SLR	Sea level rise
SWL	Still Water Level
T_p	Wave energy spectral peak period is the wave period related to the highest ordinate in the wave energy spectrum.
Wave Height	The height between the top of the crest and the bottom of the trough.
Wave Length	The distance between two wave crests.
Wave Set up	Wave set up is the increase in the mean water level shoreward of the region in which breakers form at the seashore, caused by the onshore flux of wave momentum against the beach.
Wave Run up	Wave run-up is the maximum vertical extent of wave uprush on a beach or structure above the still water level.
Wave Period	The time it takes for two successive wave crests to pass a given point.

9.0 References

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Appendix A

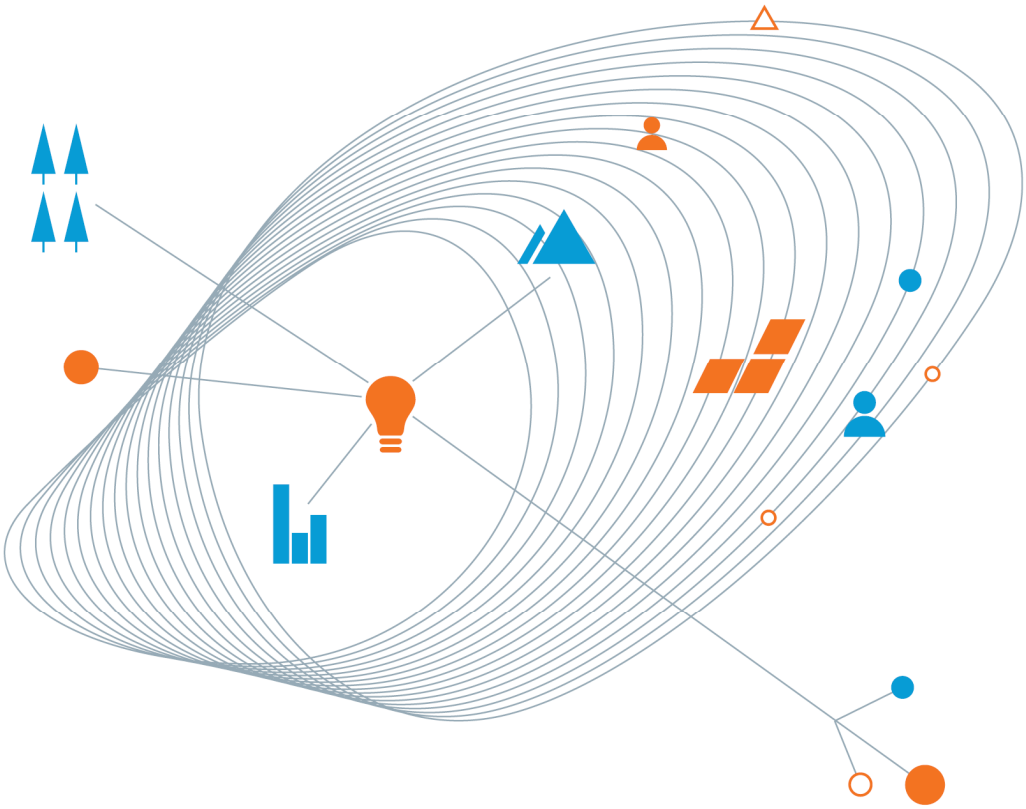
Geotechnical Report

Appendix A Geotechnical Report

AECOM

TUMBY BAY FORESHORE PROTECTION GEOTECHNICAL INVESTIGATION

GEOTMEND07441AA-AB
13 December 2013



Innovation is
finding answers
to questions
no one has
asked

13 December 2013

AECOM
Level 28, 91 King William Street
ADELAIDE SA 5000

Attention: Doug Bowers

Dear Doug

RE: TUMBY BAY FORESHORE PROTECTION

PRELIMINARY GEOTECHNICAL INVESTIGATION

Please find enclosed our report on the geotechnical investigation undertaken for the above project.

Your attention is drawn to the enclosed sheet titled "*Important Information About Your Coffey Report*", which outlines the limitations of this report.

Should you require any further information or clarification regarding our report, please contact Mark Argent or the undersigned.

For and on behalf of Coffey

John Slade

Adelaide Geotechnics Team Leader

Distribution: Original held by Coffey

1 electronic copy to AECOM

CONTENTS

1 INTRODUCTION

Coffey has undertaken a preliminary geotechnical assessment at the site of the proposed foreshore protection works at Tumby Bay, South Australia.

The aim of the geotechnical investigation was to assess the subsurface conditions along the existing seawall in order to provide recommendations for the proposed foreshore protection works. The approximately 350 m section of sea wall at the Tumby Bay foreshore is proposed to be upgraded due to progressive undermining and damage by wave erosion.

The investigation was commissioned by Mr Doug Bowers of AECOM via a sub-consultancy agreement dated 18 November 2013. The scope and extent of the investigation were consistent with a proposal prepared by Coffey dated 12 September 2013 (reference: GEOTMEND07441AAP).

This report describes the investigation undertaken and summarises the subsurface conditions encountered. Geotechnical recommendations for proposed foreshore protection works are presented in Section 4.

2 OUTLINE OF THE INVESTIGATION

2.1 Field Work

The field investigation was carried out on 23 November 2013 and comprised:

- Drilling three (3) boreholes (denoted BH01 to BH03) to depths ranging between 2.5 m and 3.2 m below the top of the existing sea wall. BH02 was drilled from behind the seawall (land side), whilst BH01 and BH03 were drilled in front of the seawall (beach side); and
- Undertaking six (6) Dynamic Cone Penetrometer (DCP) tests (denoted DCP01 to DCP06) at a spacing of approximately 60 m along the existing seawall to a depth of 3.6 m.

The boreholes were drilled using a hand auger, from a shallow excavation below the existing beach profile. PVC casing was driven into the ground prior to hand auguring to stabilise the borehole during drilling.

The DCP tests and boreholes were located in order to provide a broad coverage of the proposed seawall. The investigation locations are shown approximately on Figures 1A and 1B.

The field investigation was undertaken in the presence of a Geotechnical Engineer from Coffey who was responsible for locating the boreholes, logging the soil profile encountered and undertaking the DCP testing.

The soil profile encountered in the boreholes is described on the engineering logs contained in Appendix A. The logs are preceded by an explanation sheet that outlines the terms and symbols used in their preparation.

The results of DCP testing are presented graphically in Appendix A as plots of blow count per 100 mm penetration against depth.

2.2 Laboratory Testing

The laboratory testing undertaken on select samples comprised:

- Two (2) x Particle Size Distribution tests (AS1289.3.6.1); and
- Two (2) x Atterberg Limit and Linear Shrinkage tests (AS1289.3).

The laboratory testing was carried out in Coffey's NATA accredited laboratory in Adelaide. The results of the laboratory testing are summarised in Table 1. Laboratory test certificates are presented in Appendix B.

3 SITE CONDITIONS

3.1 Site Description

The Tumby Bay foreshore area is located approximately 50 km north of Port Lincoln on the eastern coastline of the Eyre Peninsula.

The foreshore area near the Tumby Bay Town Centre is approximately 350 m in length and includes recreational facilities (playgrounds, picnic areas, jetty), car parking and commercial properties (café, art gallery). The foreshore area is currently protected by either a concrete panel wall or 'ElcoRock' sandbag wall (Photograph 1A and 1B). Sand bags were present in front of the concrete panel wall. Outside of the Town Centre area, vegetated dunes offer protection from high tide and storm surge events.

The 'ElcoRock' wall system is present where sections of the concrete panel seawall have failed and required remediation. The remaining concrete panel seawall is in poor condition with rotation of approximately 10° from vertical at some locations (Photograph 1A). Significant spalling and cracking of the concrete wall was also observed.

Photograph 1A and 1B: The existing concrete seawall is in poor condition (left), and ElcoRock wall (right).

3.2 Regional Geology

The Geological Survey of South Australia (2012) Tumby Sheet indicates that the natural soil profile at the study area is likely to be underlain by the Holocene age deposits of Semaphore Sand and St Kilda

Formation. These materials are likely to comprise fore dune, dune and tidal sands consisting of quartz sands, shelly sands and skeletal carbonate sands. In some locations, the Semaphore Sand is overlain by modern intertidal and swamp deposits (soft clay and peat).

The sands are generally fine to coarse grained with abundant shells or shell fragments and are normally consolidated. The formation contains highly organic bands consisting of fibrous seaweed and seagrass. Peat or organic zones sand zones of up to about 500mm thickness have been previously encountered within this formation and thicker layers may be present along modern beach environments.

Coffey have previously undertaken geotechnical investigations about 1 km south of the Town Centre foreshore area where the subsurface conditions were in general agreement with the regional geology described above. Variably cemented calcrete was encountered at all test pit locations, and one test pit encountered high strength sandstone at about -3.2 mAHD. Peat was also encountered at this site in some investigation locations.

Groundwater along the alignment of the foreshore protection area would be expected to be reflective of the tide level.

3.3 Subsurface Conditions

The materials encountered in the geotechnical investigation were broadly consistent with the regional geology described in Section 3.2.

Based on the recovered hand auger samples, and DCP testing it is inferred that subsurface conditions over the depth range investigated comprised normally consolidated Holocene marine sands that increased in density with depth.

Groundwater was encountered at all investigation locations and broadly corresponded with the level of the tidal level at the time of investigation.

3.4 Laboratory Testing

The results of laboratory tests are summarised in Table 1.

Table 1: Summary of Laboratory Test Results

Borehole	Depth Range (m)[1]	Material Description	Atterberg Limits (W_L/I_p/LS)	Particle Size Distribution				
				Gravel (%)	Sand (%) (coarse, medium, fine)			Fines (< 75µm) (%)
BH02	1.2 – 1.4	SAND	NP/NP/0.0	1	55	42	1	1
BH02	2.2 – 2.4	SAND	NP/NP/0.0	3	52	35	8	2

[1] Depth measured from top of wall

Notes: W_L – Liquid Limit; I_p – Plasticity Index; LS = Linear Shrinkage

4 GEOTECHNICAL ASSESSMENT

4.1 Project Overview

Based on preliminary information, it is understood that the existing concrete seawall is founded at approximately 0.7 mAHD and the top of the wall is at approximately 2.4 to 2.6 mAHD. Any replacement structure is understood to be set at approximately -0.6 mAHD to ensure the base of the structure is well embedded in a saturated subgrade material.

At the time of writing this report, the type of retention structure proposed for the foreshore area had not been defined. A 2003 concept design documented preliminary plans for a rock revetment wall over much of the foreshore area. Other options may include an ElcoRock sandbag wall, geotextile wall, masonry block wall, or replacement concrete wall.

4.2 Retention System Design Parameters

An active earth pressure distribution (K_a = 0.33) may be used for the design of cantilevered or mass gravity retaining walls that can tolerate movement at the top of wall.

Retaining walls should be designed in accordance with the recommendations of AS4678-2002 *"Earth Retaining Structures"* and should not rely on the passive resistance in front of the wall for stability due to the wave action erosion and seasonal loss of sand in the bay.

Protection measures should be adopted to prevent undermining of the wall. Protection measures may include rip rap rock, sand bags at the base of the slope and also groynes, concrete revetment mattress, or potentially offshore protection measures such as a breakwater.

Where relevant, the compaction equipment used to compact backfill behind any replacement retaining wall must be carefully selected and preferably lightweight compaction equipment should be used. The load on the retaining wall due to compaction equipment may be estimated from Figure J5 in AS4678. For compacted site won sands, an active coefficient K_a of 0.33 and an at rest coefficient K_0 of 0.5 are recommended together with a bulk density of 18 kN/m^3.

4.3 Construction Considerations

4.3.1 Excavatability

The soil strength materials to a depth of at least 4 m below the top of the seawall are expected to be excavatable with conventional earth moving equipment such as large tracked excavators. Near vertical excavations are not stable and therefore the construction sequence of foreshore protection measures and any required trenching must be carefully planned.

Assessment of likely excavation characteristics has been based on the results of DCP testing and resistance to hand auger penetration.

4.3.2 Batter Slopes

Temporary batter slopes of 1V:4H are recommended for any excavations for the foreshore protection works due to the likely water seepage which will occur where the depth of excavation approaches tidal levels. Given the space that such an excavation would require, it is likely that any temporary excavation and backfilling activities will require shoring. Excavating and backfilling in slots at around low tide with

steeper batter slopes may be considered depending on the foreshore protection measure to be adopted. Dewatering of any excavation below sea level is likely to be impractical.

4.3.3 Trafficability

Trafficability on the normally consolidated sands for rubber-tyred vehicles and heavy tracked vehicles can be expected to be difficult. The provision of bog mats for light weight tracked construction plant may be required to improve trafficability.

4.3.4 Re-Use of Site Won Fill Material

Select fill behind the seawall may comprise site won sand materials (not including organic matter). The select fill should be placed in 200 mm thick loose layers and compacted to a dry density ratio of at least 95% based on Standard Compaction (AS1289.5.1.1) at optimum or above optimum moisture content.

5 ADDITIONAL SERVICES

It is recommended that an experienced Geotechnical Engineer review plans and specifications which affect or are affects by geotechnical issues to help assure proper interpretation of the geotechnical findings and recommendations.

We are pleased to offer our services for the review of designs, as well as earthworks quality control testing during construction.

6 LIMITATIONS

The findings contained within this report are the results of geotechnical investigations conducted in accordance with normal practices and standards. To the best of our knowledge, they represent a reasonable interpretation of the general condition of the site. Under no circumstances, however, can it be considered that these findings represent the actual state of the site at all points.

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

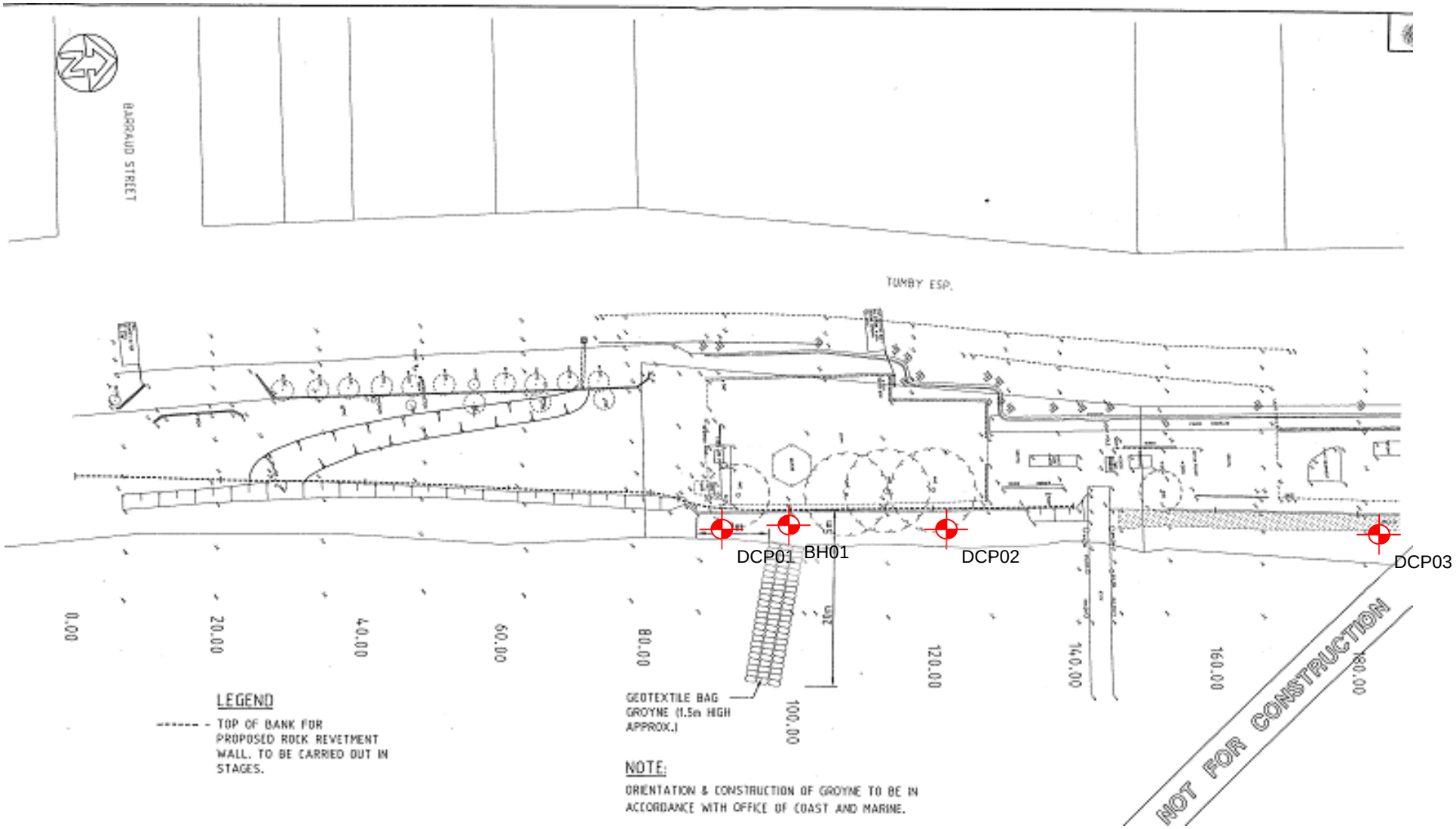
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

Figures

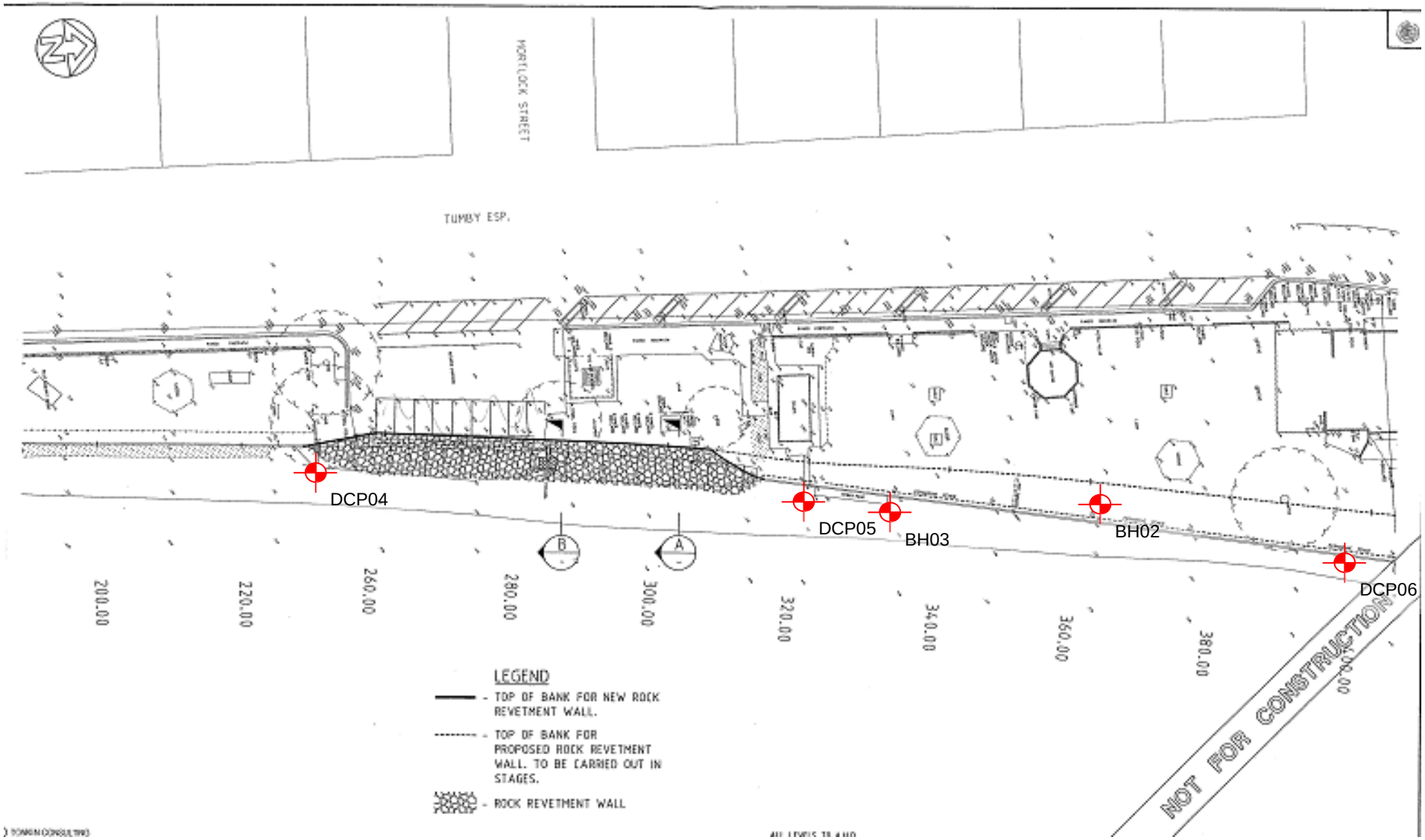


Legend

Approximate Investigation Location

IMAGE SOURCE: SUPPLIED BY AECOM

drawn	MA	coffey	client: AECOM PTY LTD
approved	JS		project: TUMBY BAY FORESHORE PROTECTION WORKS
date	16-12-13		
scale	NTS		title: INVESTIGATION LOCATION PLAN
original size	A4		project no: GEOTMEND07441AA-AB / figure no: FIGURE 1A



Legend

Approximate Investigation Location

IMAGE SOURCE: SUPPLIED BY AECOM

drawn	MA	client:	AECOM PTY LTD
approved	JS	project:	TUMBY BAY FORESHORE PROTECTION WORKS
date	16-12-13		
scale	NTS	title:	INVESTIGATION LOCATION PLAN
original size	A4	project no: GEOTMEND07441AA-AB	figure no: FIGURE 1B

Appendix A

Results of Field Investigation

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Soil Description Explanation Sheet (1 of 2)

DEFINITION:

In engineering terms soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

CLASSIFICATION SYMBOL & SOIL NAME

Soils are described in accordance with the Unified Soil Classification (UCS) as shown in the table on Sheet 2.

PARTICLE SIZE DESCRIPTIVE TERMS

NAME	SUBDIVISION	SIZE
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 μm to 2.36 mm
	medium	200 μm to 600 μm
	fine	75 μm to 200 μm

MOISTURE CONDITION

Dry Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist Soil feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet As for moist but with free water forming on hands when handled.

CONSISTENCY OF COHESIVE SOILS

TERM	UNDRAINED STRENGTH s_u (kPa)	FIELD GUIDE
Very Soft	<12	A finger can be pushed well into the soil with little effort.
Soft	12 - 25	A finger can be pushed into the soil to about 25mm depth.
Firm	25 - 50	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 - 100	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 - 200	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	>200	The surface of the soil can be marked only with the thumbnail.
Friable	–	Crumbles or powders when scraped by thumbnail.

DENSITY OF GRANULAR SOILS

TERM	DENSITY INDEX (%)
Very loose	Less than 15
Loose	15 - 35
Medium Dense	35 - 65
Dense	65 - 85
Very Dense	Greater than 85

MINOR COMPONENTS

TERM	ASSESSMENT GUIDE	PROPORTION OF MINOR COMPONENT IN:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: <5% Fine grained soils: <15%
With some	Presence easily detected by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 - 12% Fine grained soils: 15 - 30%

SOIL STRUCTURE

ZONING		CEMENTING	
Layers	Continuous across exposure or sample.	Weakly cemented	Easily broken up by hand in air or water.
Lenses	Discontinuous layers of lenticular shape.	Moderately cemented	Effort is required to break up the soil by hand in air or water.
Pockets	Irregular inclusions of different material.		

GEOLOGICAL ORIGIN

WEATHERED IN PLACE SOILS

Extremely weathered material	Structure and fabric of parent rock visible.
Residual soil	Structure and fabric of parent rock not visible.

TRANSPORTED SOILS

Aeolian soil	Deposited by wind.
Alluvial soil	Deposited by streams and rivers.
Colluvial soil	Deposited on slopes (transported downslope by gravity).
Fill	Man made deposit. Fill may be significantly more variable between tested locations than naturally occurring soils.
Lacustrine soil	Deposited by lakes.
Marine soil	Deposited in ocean basins, bays, beaches and estuaries.

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Soil Description Explanation Sheet (2 of 2)

SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 60 mm and basing fractions on estimated mass)						USC	PRIMARY NAME
COARSE GRAINED SOILS More than 50% of materials less than 63 mm is larger than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.36 mm	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.		GW	GRAVEL
				Predominantly one size or a range of sizes with more intermediate sizes missing.		GP	GRAVEL
			GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)		GM	SILTY GRAVEL
				Plastic fines (for identification procedures see CL below)		GC	CLAYEY GRAVEL
		SANDS More than half of coarse fraction is smaller than 2.36 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes		SW	SAND
				Predominantly one size or a range of sizes with some intermediate sizes missing.		SP	SAND
			SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).		SM	SILTY SAND
				Plastic fines (for identification procedures see CL below).		SC	CLAYEY SAND
FINE GRAINED SOILS More than 50% of material less than 63 mm is smaller than 0.075 mm			IDENTIFICATION PROCEDURES ON FRACTIONS <0.2 mm.				
		SILTS & CLAYS Liquid limit less than 50	**DRY STRENGTH**	**DILATANCY**	**TOUGHNESS**		
			None to Low	Quick to slow	None	ML	SILT
			Medium to High	None	Medium	CL	CLAY
			Low to medium	Slow to very slow	Low	OL	ORGANIC SILT
		SILTS & CLAYS Liquid limit greater than 50	Low to medium	Slow to very slow	Low to medium	MH	SILT
			High	None	High	CH	CLAY
			Medium to High	None	Low to medium	OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS			Readily identified by colour, odour, spongy feel and frequently by fibrous texture.			Pt	PEAT

• Low plasticity – Liquid Limit w_L less than 35%. • Medium plasticity – w_L between 35% and 50%. • High plasticity – w_L greater than 50%.

COMMON DEFECTS IN SOIL

TERM	DEFINITION	DIAGRAM
PARTING	A surface or crack across which the soil has little or no tensile strength. Parallel or sub parallel to layering (eg bedding). May be open or closed.	
JOINT	A surface or crack across which the soil has little or no tensile strength but which is not parallel or sub parallel to layering. May be open or closed. The term 'fissure' may be used for irregular joints <0.2 m in length.	
SHEARED ZONE	Zone in clayey soil with roughly parallel near planar, curved or undulating boundaries containing closely spaced, smooth or slickensided, curved intersecting joints which divide the mass into lenticular or wedge shaped blocks.	
SHEARED SURFACE	A near planar curved or undulating, smooth, polished or slickensided surface in clayey soil. The polished or slickensided surface indicates that movement (in many cases very little) has occurred along the defect.	

TERM	DEFINITION	DIAGRAM
SOFTENED ZONE	A zone in clayey soil, usually adjacent to a defect in which the soil has a higher moisture content than elsewhere.	
TUBE	Tubular cavity. May occur singly or as one of a large number of separate or inter-connected tubes. Walls often coated with clay or strengthened by denser packing of grains. May contain organic matter	
TUBE CAST	Roughly cylindrical elongated body of soil different from the soil mass in which it occurs. In some cases the soil which makes up the tube cast is cemented.	
INFILLED SEAM	Sheet or wall like body of soil substance or mass with roughly planar to irregular near parallel boundaries which cuts through a soil mass. Formed by infilling of open joints.	

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Engineering Log - Borehole

Borehole ID.	***BH01***		
sheet:	1 of 1		
project no.	***GEOTMEND07441AA***		
client:	***AECOM***	date started:	***23 Nov 2013***
principal:		date completed:	***23 Nov 2013***
project:	***TUMBY BAY FORESHORE PROJECT***	logged by:	***MA:ldr***
location:	***REFER TO FIGURE 1***	checked by:	***JS***

position: E: 601435; N: 6195362 (Datum Not Specified) surface elevation : Not Specified angle from horizontal: 90°

drill model: HAND GEAR mounting: hole diameter : 70 mm

drilling information | **material substance**

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	hand penetrometer (kPa)	structure and additional observations
					0.5			**SAND**: excavated to expose sand bag at base of wall.				Depth measured from top of wall
HA, C		23/11/13	D, D		1.0, 1.5, 2.0, 2.5, 3.0		SP	**SAND**: fine to coarse grained, pale grey, some orange brown, trace fine grained gravel (carbonate shell). fibrous matting (seaweed) between 1.0-1.3 m.	M W	VL / L MD		**NATURAL** sandbags at 900 mm, below top of wall
					3.5			Borehole BH01 terminated at 3.2 m Collapse				

NEW GEOTECHNICS TEMPLATE MELB.GLB Log COF BOREHOLE: NON CORED 07441AA BORELOGS.GPJ <<DrawingFile>> 13/12/2013 16:43

method
- AD auger drilling*
- AS auger screwing*
- RR roller/tricone
- W washbore
- CT cable tool
- HA hand auger
- DT diatube
- B blank bit
- V V bit
- T TC bit
- * bit shown by suffix
- e.g. AD/T

support
- M mud
- N nil
- C casing

penetration
- 1 2 3 — no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shear peak/remouded (uncorrected kPa)
- R refusal

classification symbol & soil description based on Unified Classification System

moisture
- D dry
- M moist
- W wet

consistency / relative density
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

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Engineering Log - Borehole

Borehole ID.	***BH02***
sheet:	1 of 1
project no.	***GEOTMEND07441AA***
date started:	***23 Nov 2013***
date completed:	***23 Nov 2013***
logged by:	***MA:ldr***
checked by:	***JS***

client: ***AECOM***

principal:

project: ***TUMBY BAY FORESHORE PROJECT***

location: ***REFER TO FIGURE 1***

position: E: 601472; N: 6195607 (Datum Not Specified) surface elevation : Not Specified angle from horizontal: 90°

drill model: HAND GEAR mounting: hole diameter : 70 mm

drilling information | **material substance**

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	hand penetrometer (kPa)	structure and additional observations
HA, N								**FILL: Silty SAND**: fine to medium grained, brown, organic matter, low plasticity fines (topsoil).	D	VL / L		**FILL**
					0.5			**FILL: SAND**: fine to coarse grained, predominantly medium grained, yellow white, minor orange, trace fine grained gravel.				
					1.0							
					1.5							
					2.0		SP	**SAND**: fine to coarse grained, pale grey, minor orange.	W	L		**PROBABLE NATURAL**
		23/11/13			2.5			Borehole BH02 terminated at 2.5 m Collapse				
					3.0							
					3.5							

NEW GEOTECHNICS TEMPLATE MELB.GLB Log COF BOREHOLE: NON CORED 07441AA BORELOGS.GPJ <<DrawingFile>> 13/12/2013 17:15

method
- AD auger drilling*
- AS auger screwing*
- RR roller/tricone
- W washbore
- CT cable tool
- HA hand auger
- DT diatube
- B blank bit
- V V bit
- T TC bit
- * bit shown by suffix
- e.g. AD/T

support
- M mud
- N nil
- C casing

penetration
- 1 2 3 — no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- VS vane shear peak/remouded (uncorrected kPa)
- R refusal

classification symbol & soil description based on Unified Classification System

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- W wet

consistency / relative density
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- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

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Engineering Log - Borehole

Borehole ID.	***BH03***		
sheet:	1 of 1		
project no.	***GEOTMEND07441AA***		
client:	***AECOM***	date started:	***22 Nov 2013***
principal:		date completed:	***22 Nov 2013***
project:	***TUMBY BAY FORESHORE PROJECT***	logged by:	***MA:ldr***
location:	***REFER TO FIGURE 1***	checked by:	***JS***

position: E: 601471; N: 6195583 (Datum Not Specified) surface elevation : Not Specified angle from horizontal: 90°

drill model: HAND GEAR mounting: hole diameter : 70 mm

drilling information | **material substance**

method & support	penetration	water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	hand penetrometer (kPa)	structure and additional observations
					0.5			**SAND**: excavated to expose sand bags at base of wall.				Depth measured from top of wall
HA, N		22/11/13			1.0, 1.5, 2.0, 2.5		SP	**SAND**: fine to coarse grained, white yellow, with fine grained gravel (shell).	D	VL / L		**NATURAL**
								fibrous seaweed (1.8-2.1 m).				sandstone cobble 200 mm
								trace sandstone, fine grained, pink, medium grained gravel.	W	MD		
					3.0, 3.5			Borehole BH03 terminated at 2.9 m Collapse				

NEW GEOTECHNICS TEMPLATE MELB.GLB Log COF BOREHOLE: NON CORED 07441AA BORELOGS.GPJ <<DrawingFile>> 13/12/2013 16:43

method
- AD auger drilling*
- AS auger screwing*
- RR roller/tricone
- W washbore
- CT cable tool
- HA hand auger
- DT diatube
- B blank bit
- V V bit
- T TC bit
- * bit shown by suffix
- e.g. AD/T

support
- M mud
- N nil
- C casing

penetration
- 1 2 3 — no resistance ranging to refusal

water
- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests
- U## undisturbed sample ##mm diameter
- D disturbed sample
- B bulk disturbed sample
- E environmental sample
- HP hand penetrometer (kPa)
- N standard penetration test (SPT)
- N* SPT - sample recovered
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- R refusal

classification symbol & soil description based on Unified Classification System

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- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

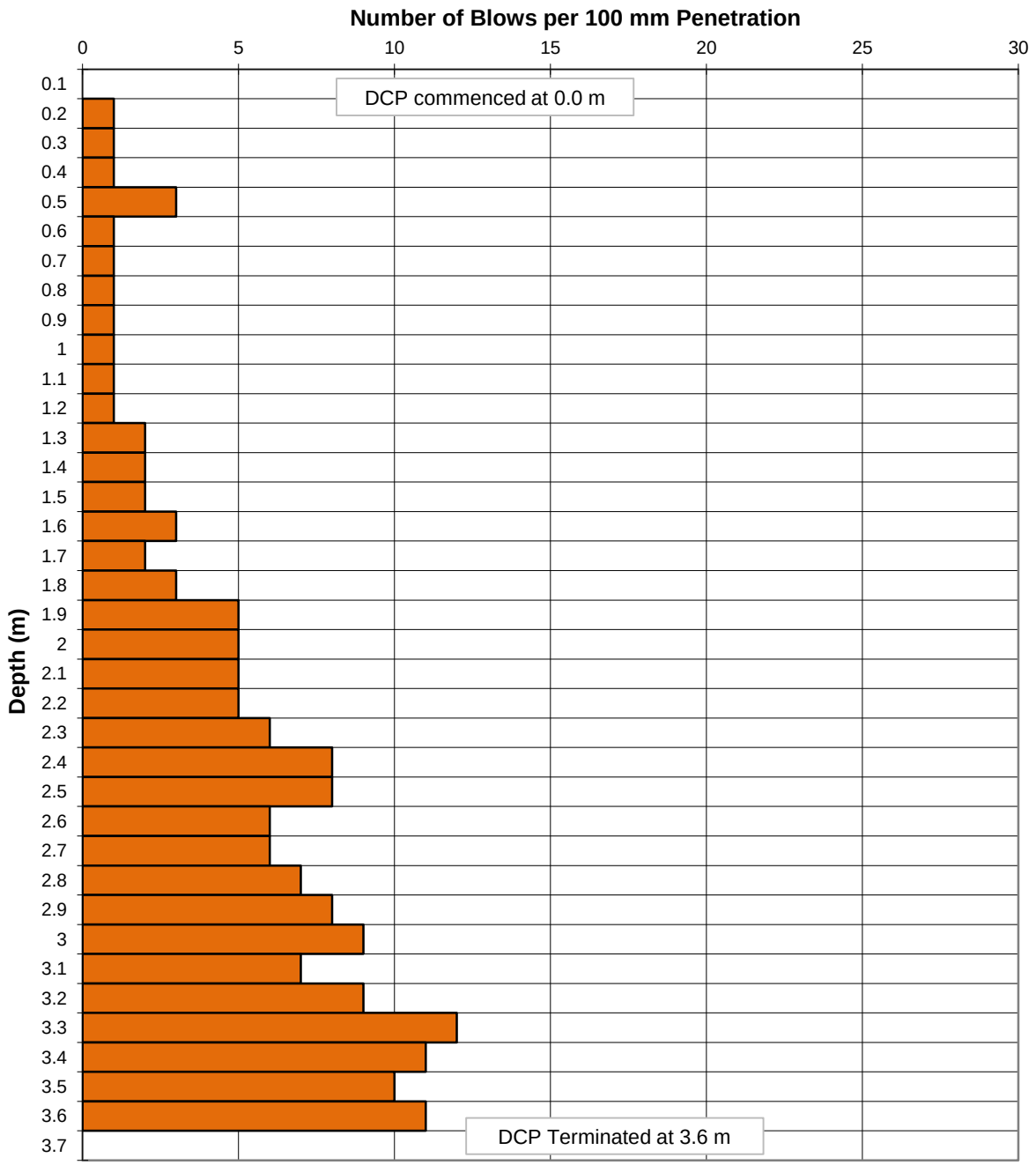
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP01**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



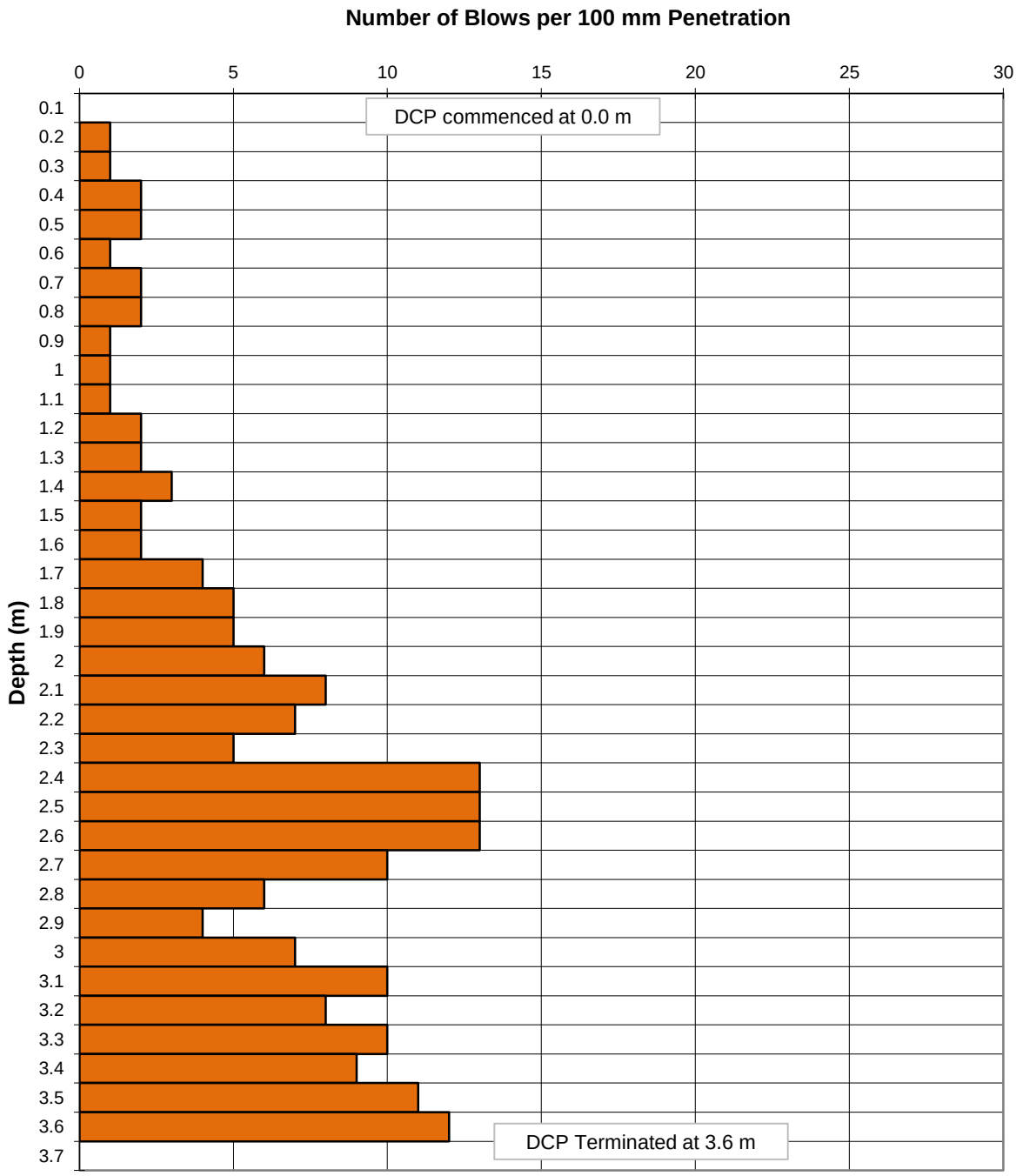
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP02**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



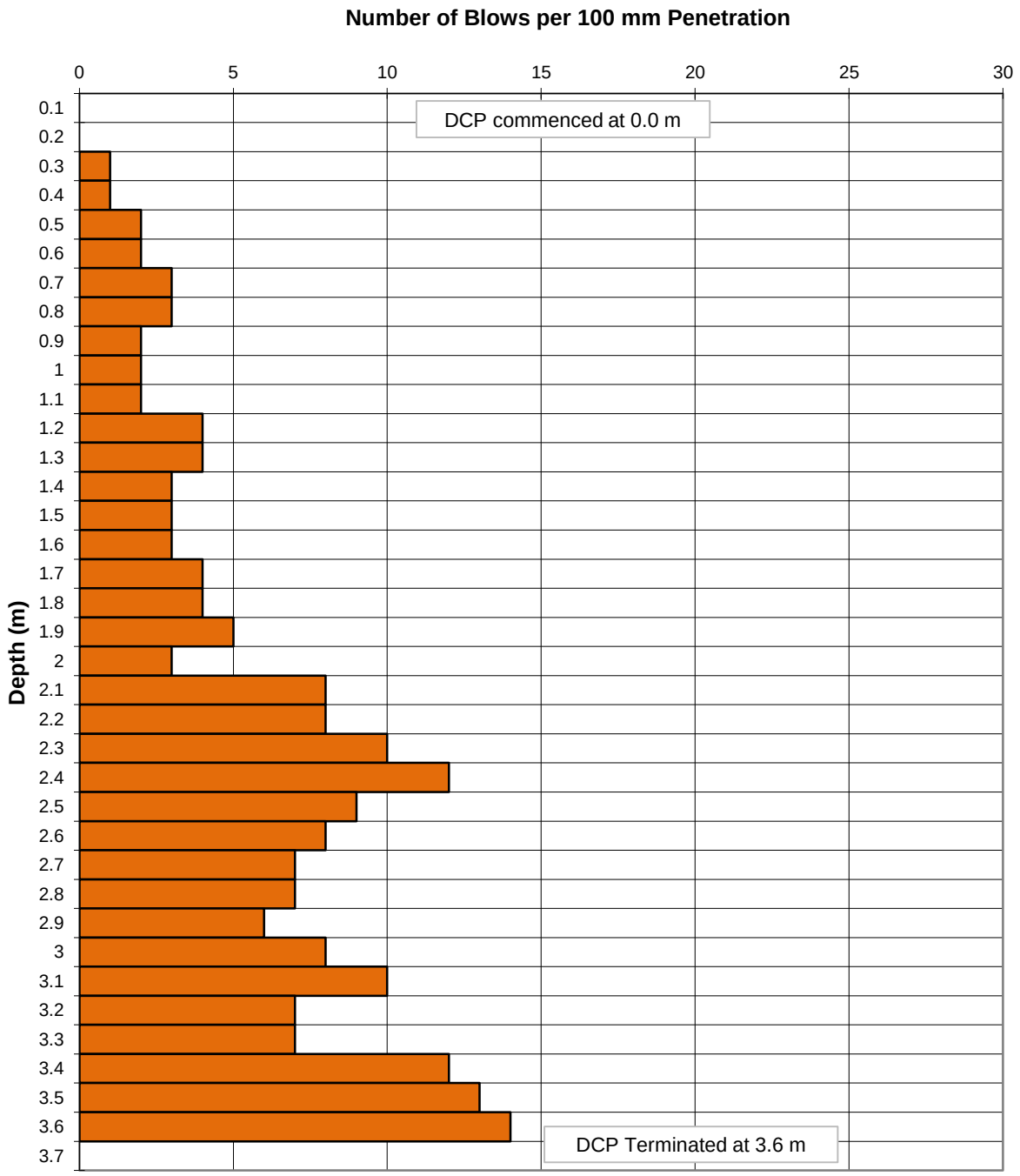
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP03**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



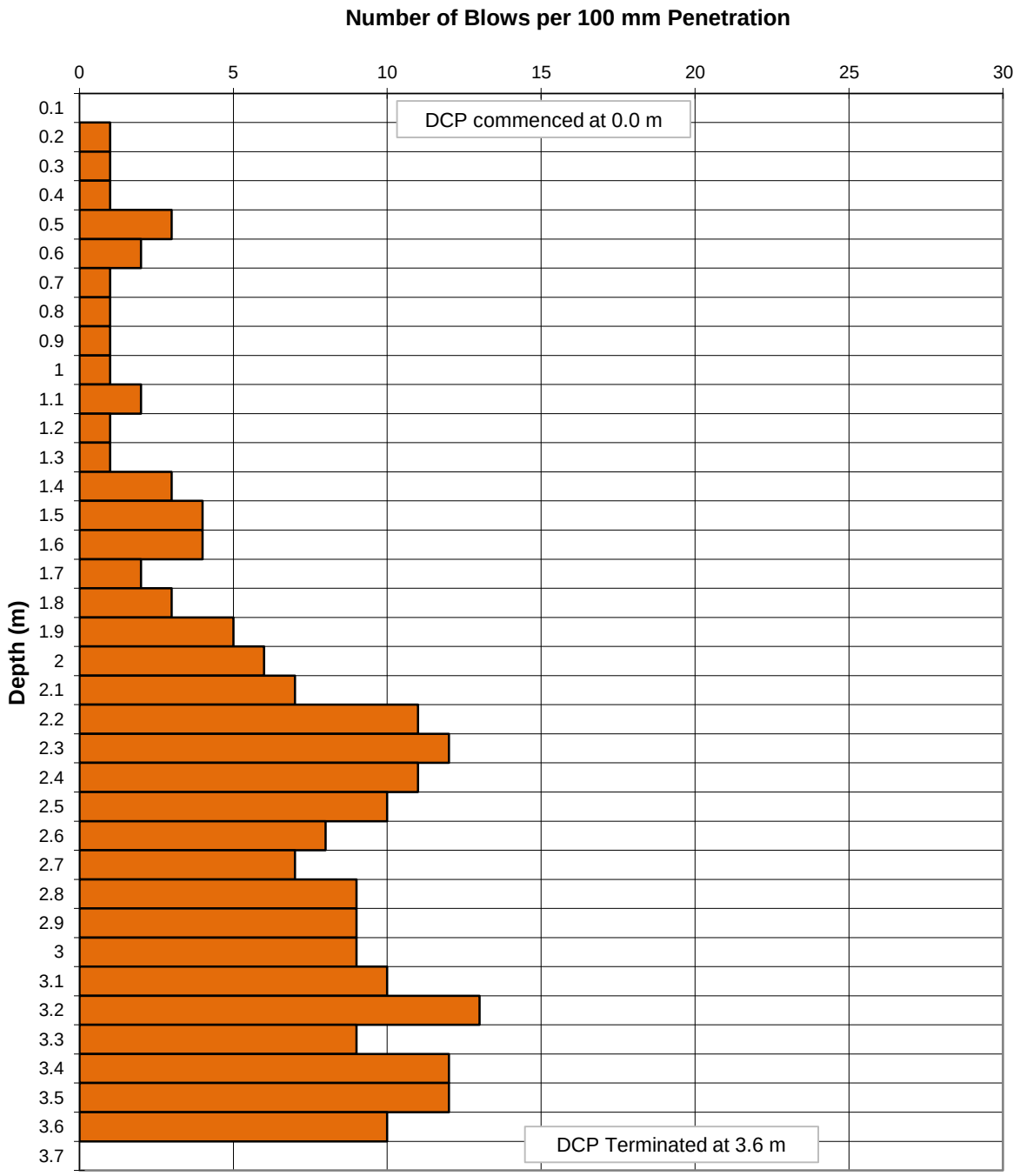
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP04**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



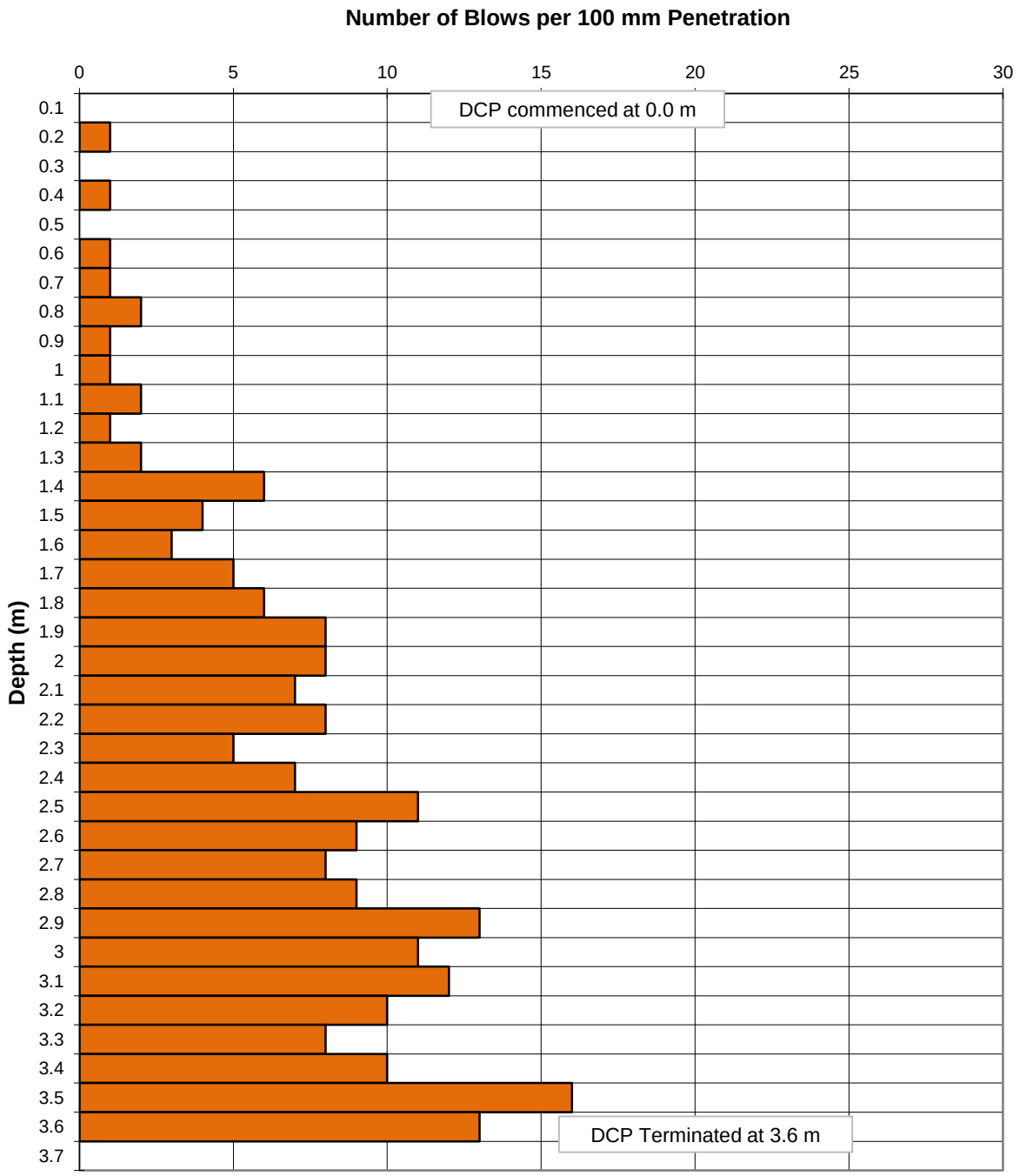
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP05**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



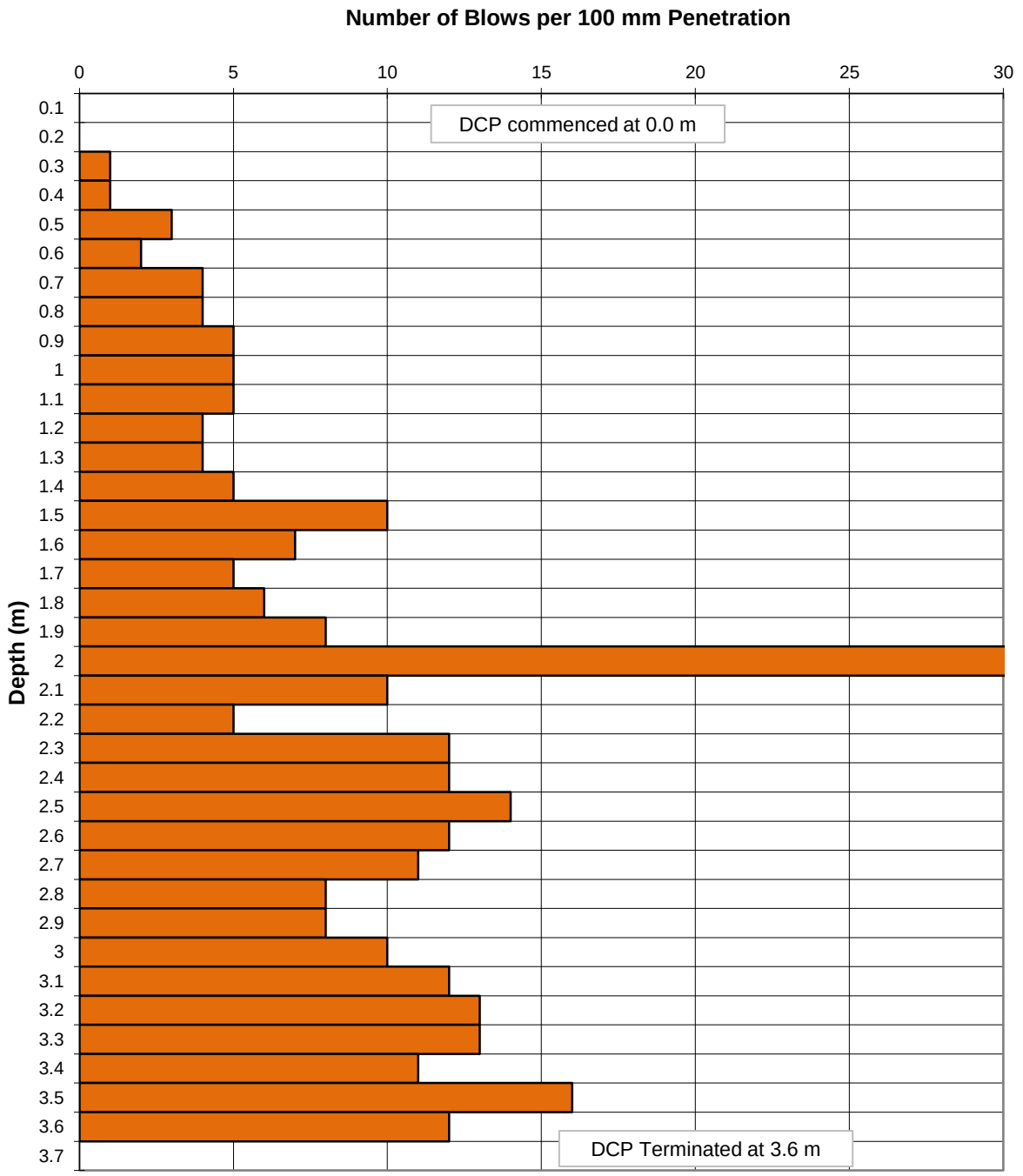
Worldpark, 33 Richmond Road, Keswick SA 5035
Ph: (08) 8375 4400 Fax: (08) 8375 4499

Report of Dynamic Cone Penetrometer (DCP) Test

Client:	AECOM	**Job No.**	GEOTMEND07441AA
Principal:	-	**DCP Test No.**	**DCP06**
Project:	TUMBY BAY SEAWALL	**Tested On:**	22-Nov-13
Location:	TUMBY BAY	**Date:**	22-Nov-13

Test Method: AS 1289 . 6.3.2 - 1997 Determination of the Penetration Resistance of a Soil using 9 kg Dynamic Cone Penetrometer

Hammer Mass: 9.06 kg **Hammer Drop:** 508 mm



Appendix B

Results of Laboratory Testing

Keswick, Adelaide Laboratory

Coffey Testing Pty Ltd
ABN 92 114 364 046
33 Richmond Road
Keswick ADELAIDE SA 5035

Phone: +61 8 8375 4400
Fax: +61 8 8375 4499

Material Test Report

Report No: KESW13S-03813-1
Issue No: 1

Client:	Coffey Geotechnics Pty Ltd (Keswick) 33-39 Richmond Road Keswick SA 5031
Principal:	
Project No.:	INFOKESW01481AA
Project Name:	GEOTMEND07441AA - Aecom, Tumby Bay Foreshore Protection
Lot No.:	TRN:

Accredited for compliance with ISO/IEC 17025.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

Approved Signatory: Ross Dingle
(Technical Manager.)
NATA Accredited Laboratory Number:431
Date of Issue: 6/12/2013

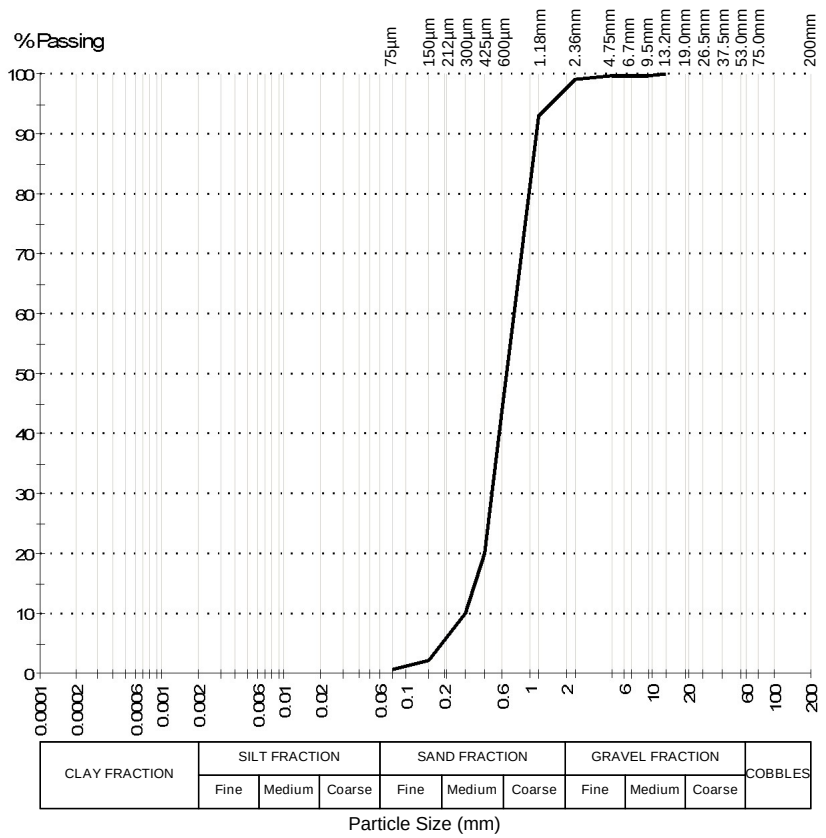
Sample Details

Sample ID:	KESW13S-03813
Client Sample:	
Date Sampled:	22/11/2013
Source:	BORE HOLE
Material:	
Specification:	AS Grading
Sampling Method:	Submitted by client
Project Location:	TUMBY BAY, SOUTH AUSTRALIA
Sample Location:	BH03 1.2-1.4m

Other Test Results

Description	Method	Result	Limits
Sample History	AS 1289.1.1	Oven-dried	
Preparation	AS 1289.1.1	Dry Sieved	
Linear Shrinkage (%)	AS 1289.3.4.1	0.0	
Mould Length (mm)		127	
Crumbling		No	
Curling		No	
Cracking		No	
Liquid Limit (%)	AS 1289.3.1.2	N/A	
Method		One Point	
Plastic Limit (%)	AS 1289.3.2.1	NP	
Plasticity Index (%)	AS 1289.3.3.1	NP	

Particle Size Distribution



Method: AS 1289.3.6.1
Drying by: Oven
Date Tested:
Note: Sample Washed

Sieve Size	% Passing	Limits
13.2mm	100	
9.5mm	100	
6.7mm	100	
4.75mm	100	
2.36mm	99	
1.18mm	93	
600µm	44	
425µm	20	
300µm	10	
150µm	2	
75µm	1	

Comments

NP = Non Plastic

Keswick, Adelaide Laboratory

Coffey Testing Pty Ltd
ABN 92 114 364 046
33 Richmond Road
Keswick ADELAIDE SA 5035

Phone: +61 8 8375 4400
Fax: +61 8 8375 4499

Material Test Report

Report No: KESW13S-03814-1
Issue No: 1

Client:	Coffey Geotechnics Pty Ltd (Keswick) 33-39 Richmond Road Keswick SA 5031
Principal:	
Project No.:	INFOKESW01481AA
Project Name:	GEOTMEND07441AA - Aecom, Tumby Bay Foreshore Protection
Lot No.:	TRN:

Accredited for compliance with ISO/IEC 17025.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

Approved Signatory: Ross Dingle
(Technical Manager.)
NATA Accredited Laboratory Number:431
Date of Issue: 6/12/2013

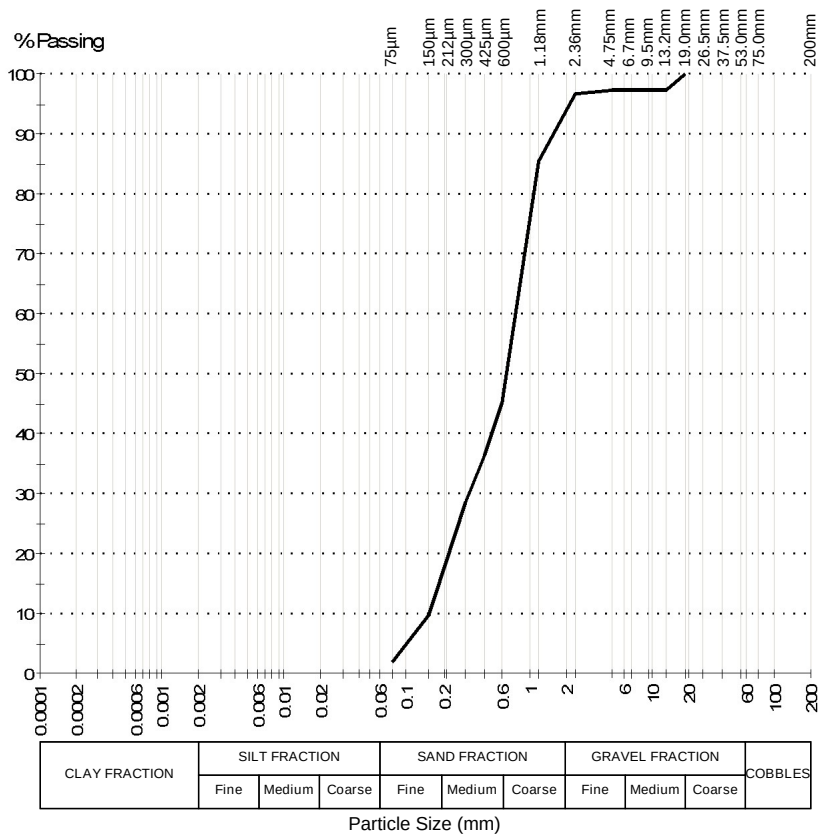
Sample Details

Sample ID:	KESW13S-03814
Client Sample:	
Date Sampled:	22/11/2013
Source:	BORE HOLE
Material:	
Specification:	AS Grading
Sampling Method:	Submitted by client
Project Location:	TUMBY BAY, SOUTH AUSTRALIA
Sample Location:	BH03 2.2-2.4m

Other Test Results

Description	Method	Result	Limits
Sample History	AS 1289.1.1	Oven-dried	
Preparation	AS 1289.1.1	Dry Sieved	
Linear Shrinkage (%)	AS 1289.3.4.1	0.0	
Mould Length (mm)		250	
Crumbling		No	
Curling		No	
Cracking		No	
Liquid Limit (%)	AS 1289.3.1.2	N/A	
Method		One Point	
Plastic Limit (%)	AS 1289.3.2.1	NP	
Plasticity Index (%)	AS 1289.3.3.1	NP	

Particle Size Distribution



Method: AS 1289.3.6.1
Drying by: Oven
Date Tested:
Note: Sample Washed

Sieve Size	% Passing	Limits
19.0mm	100	
13.2mm	97	
9.5mm	97	
6.7mm	97	
4.75mm	97	
2.36mm	97	
1.18mm	86	
600µm	45	
425µm	36	
300µm	28	
150µm	10	
75µm	2	

Comments

NP = Non Plastic

Appendix B

Photographs of Previous Wall Failures and Site Issues

Appendix B Photographs of Previous Wall Failures and Site Issues

Figure 18 Existing Sand Groyne at north end of beach

Figure 19 Existing Elcorock Sea Wall Showing Revegetation

Figure 20 Existing sand groynes North and South of Jetty

Figure 21 Existing path and infrastructure in Lions Park

Figure 22 Example of upper beach profile

Figure 23 Example of silted up stormwater outlet

Figure 24 Drainage erosion scar at Ritz car park

Figure 25 Existing Elcorock wall after June 2013 storm

Figure 26 Damaged precinct wall and subsidence of backfill June 2013 Storm Note seagrass from wave overtopping

Figure 27 Damaged precast wall and subsidence at Lions Park March 2005

Figure 28 Highest recorded tide 1999 at 2.01m AHD

Figure 29 Failed precast wall May 2001

Appendix C

Sketches SK001 - SK006

Appendix C Sketches SK001 - SK006

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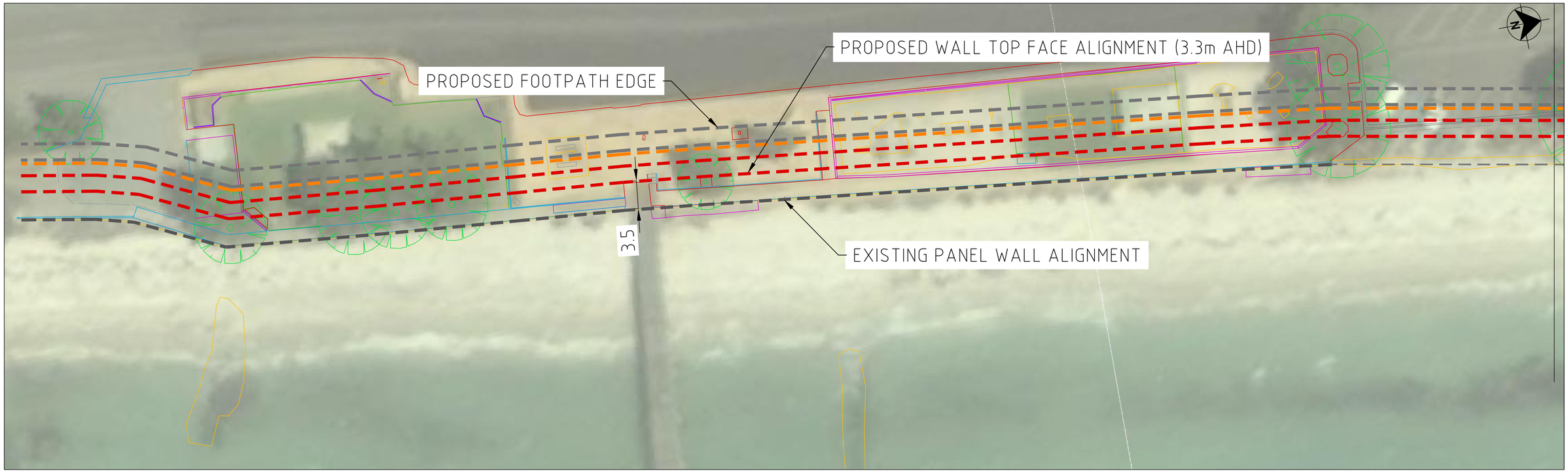

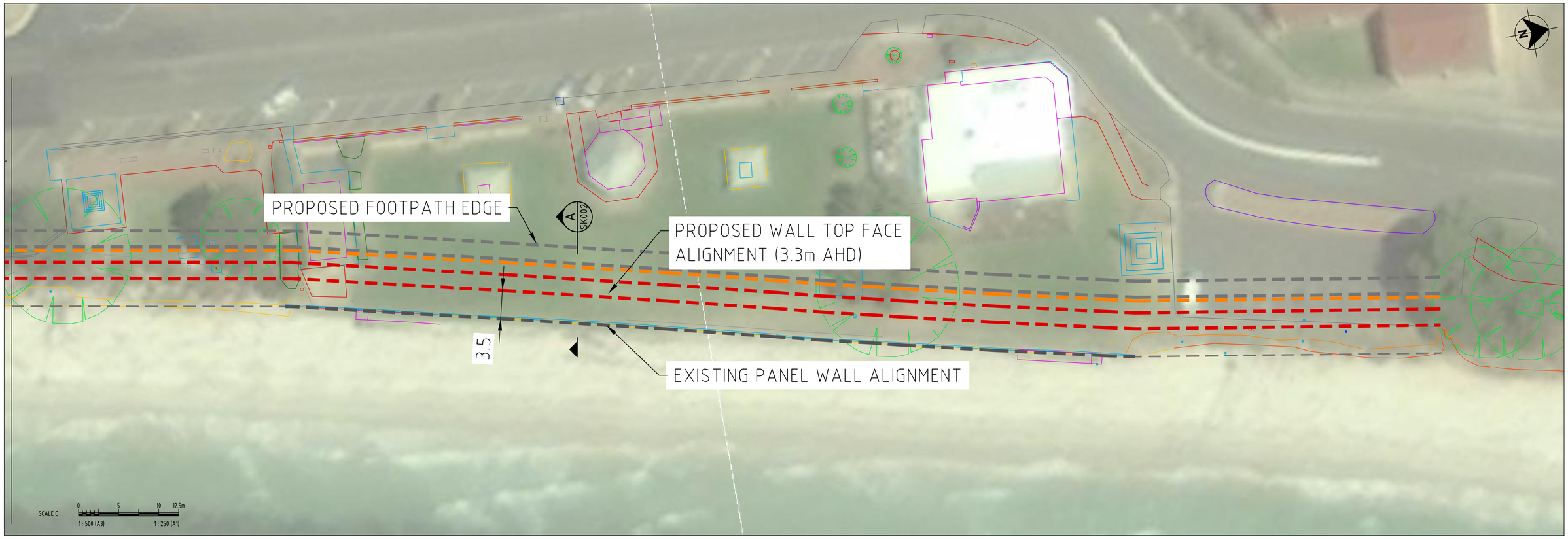

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AECOM

AECOM Australia Pty Ltd
A.B.N. 20 093 846 925

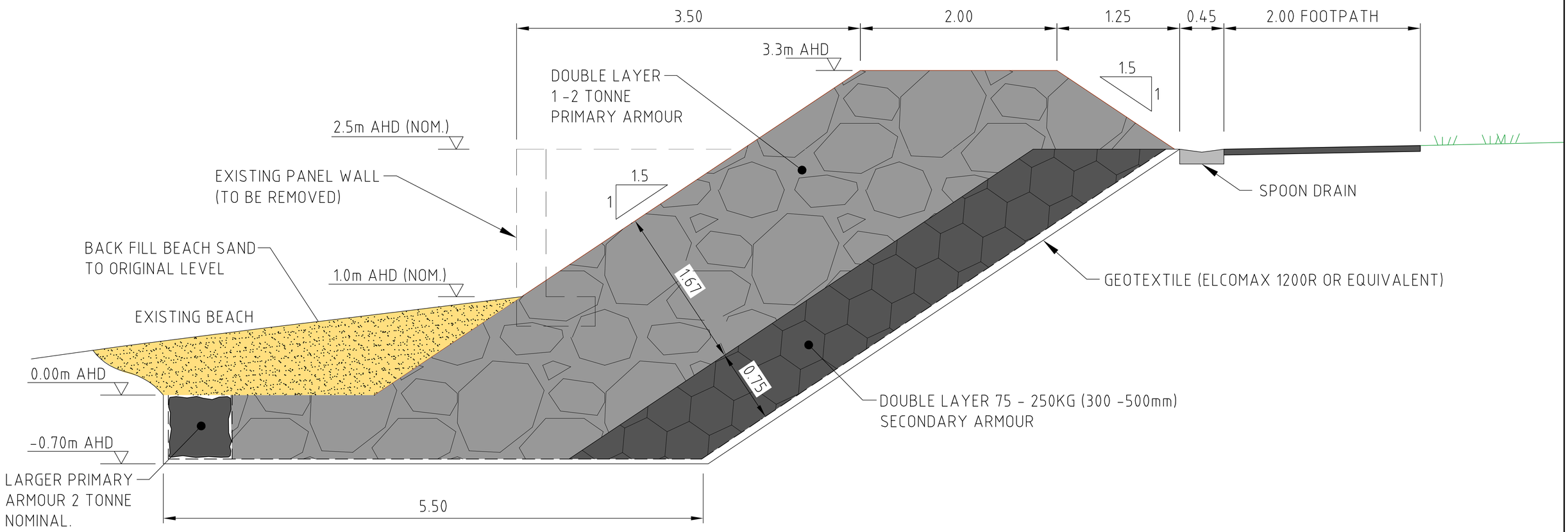
Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL CONCEPT PLAN
OPTION 1

Status: PRELIMINARY

Drg No.: 60310334-SK001

Rev: A



SECTION A
NTS SK001

TUMBY BAY
TYPICAL SEAWALL CROSS SECTION
AT FORESHORE RESERVE
(OPTION 1)

NTS

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A.B.N. 20 093 846 925

Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL TYPICAL SECTION
OPTION 1

Status: PRELIMINARY

Drg No.: 60310334-SK002

Rev.: B

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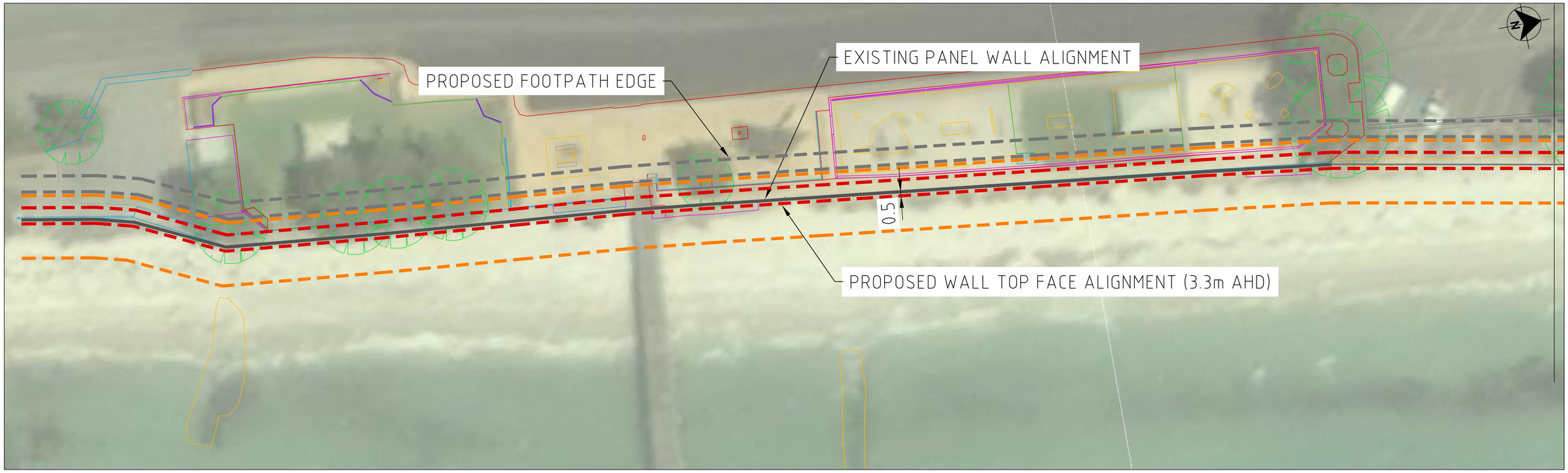

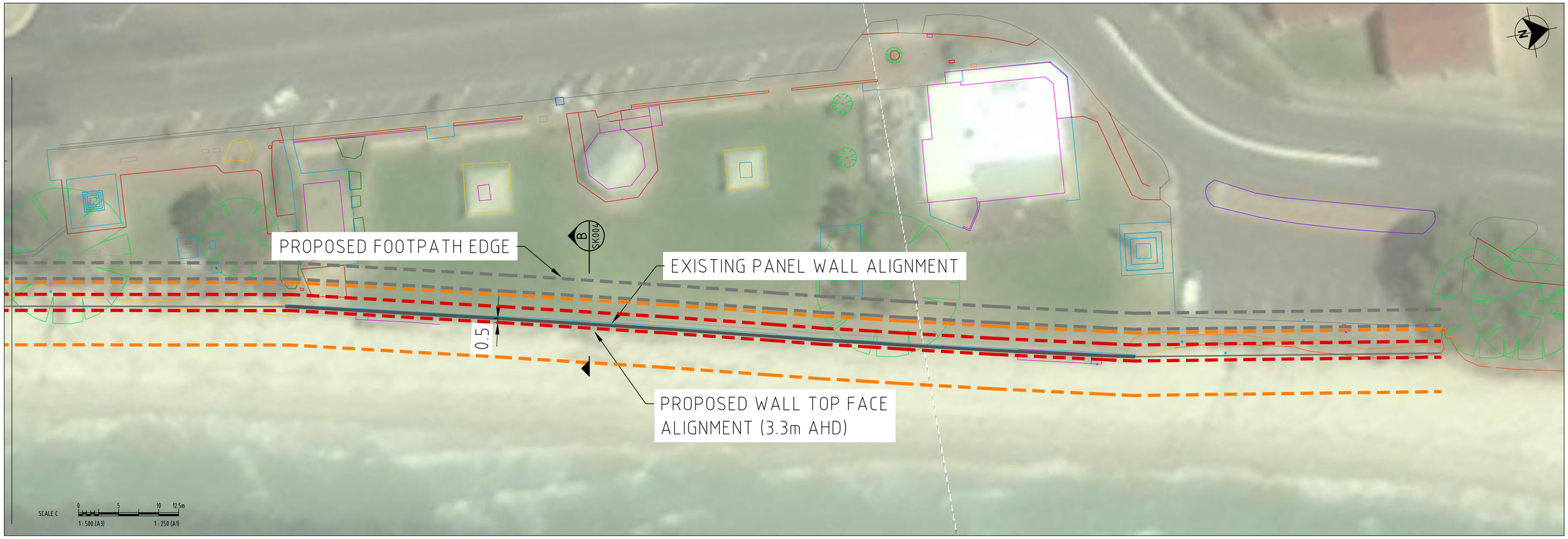

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Client :

Designer :

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AECOM Australia Pty Ltd
A.B.N. 20 093 846 925

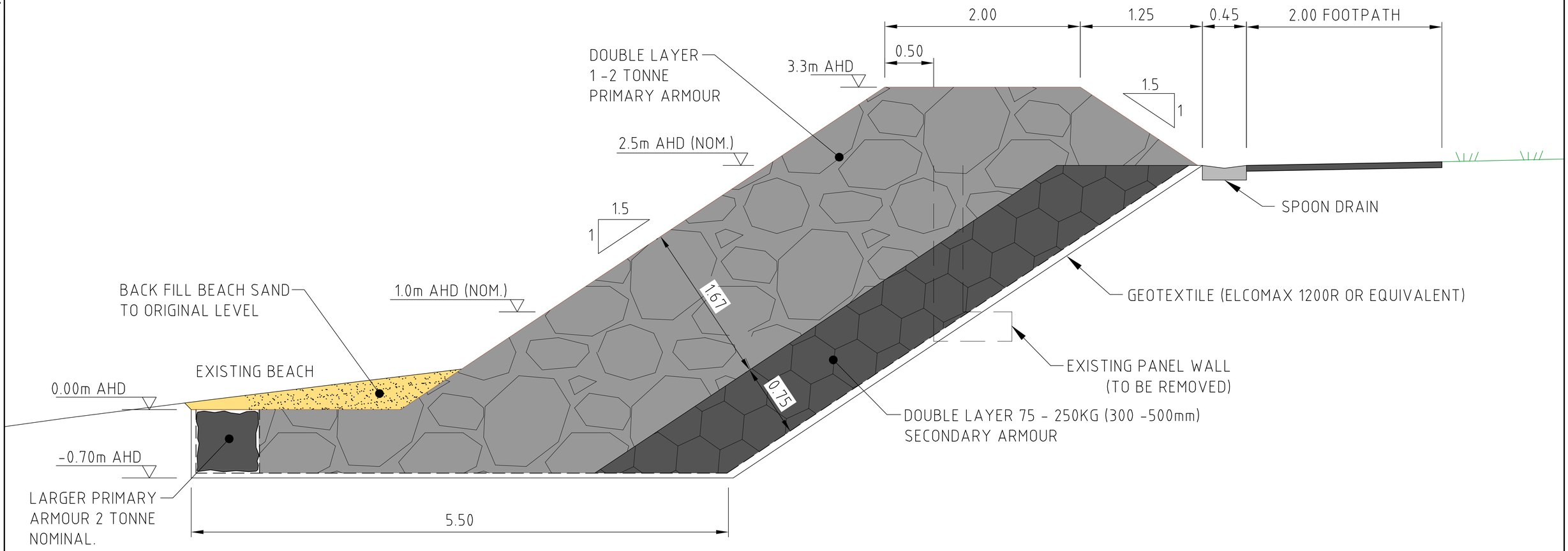
Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL CONCEPT PLAN
OPTION 2

Status: PRELIMINARY

Drg No.: 60310334-SK003

Rev: A



SECTION B
NTS SK003

TUMBY BAY
TYPICAL SEAWALL CROSS SECTION
AT FORESHORE RESERVE
(OPTION 2)

NTS

REVISIONS

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A	SD	20.01.14	ISSUED FOR INFORMATION	DB

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A.B.N. 20 093 846 925

Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL TYPICAL SECTION
OPTION 2

Status: PRELIMINARY

Drg No.: 60310334-SK004

Rev.: B

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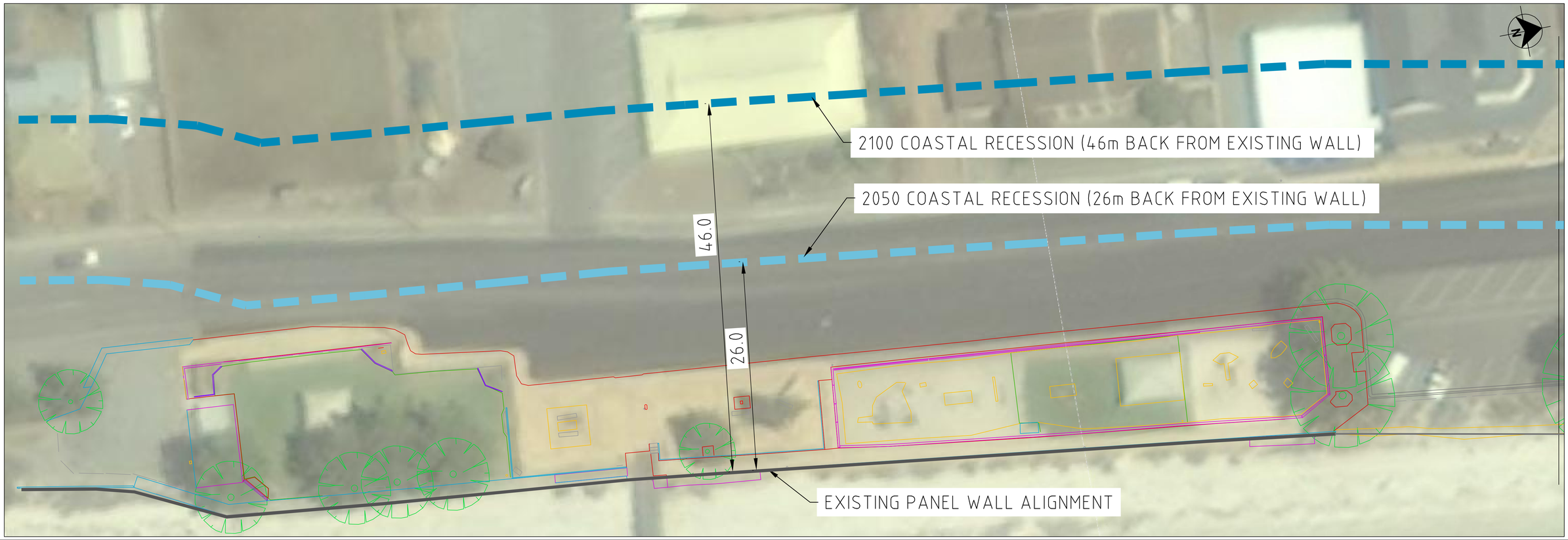

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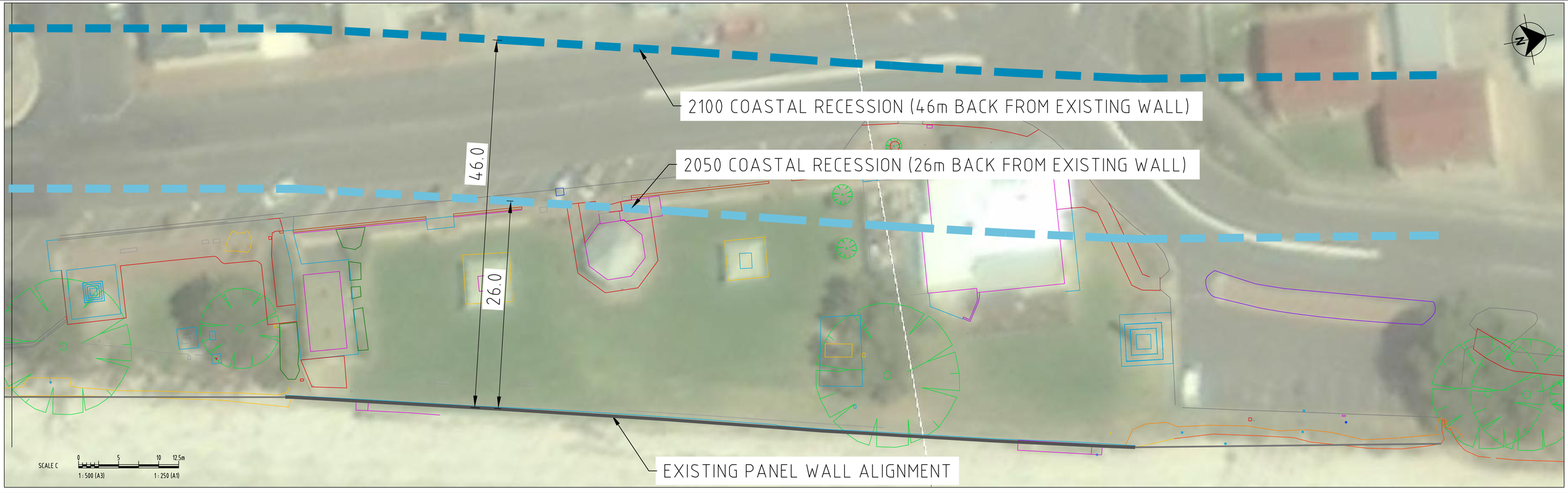
FOR CONTINUATION REFER ABOVE RIGHT



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Designer :

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A.B.N. 20 093 846 925

Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL CONCEPT PLAN
COASTAL RECESSION

Status: PRELIMINARY

Drg No.: 60310334-SK005

Rev.: A

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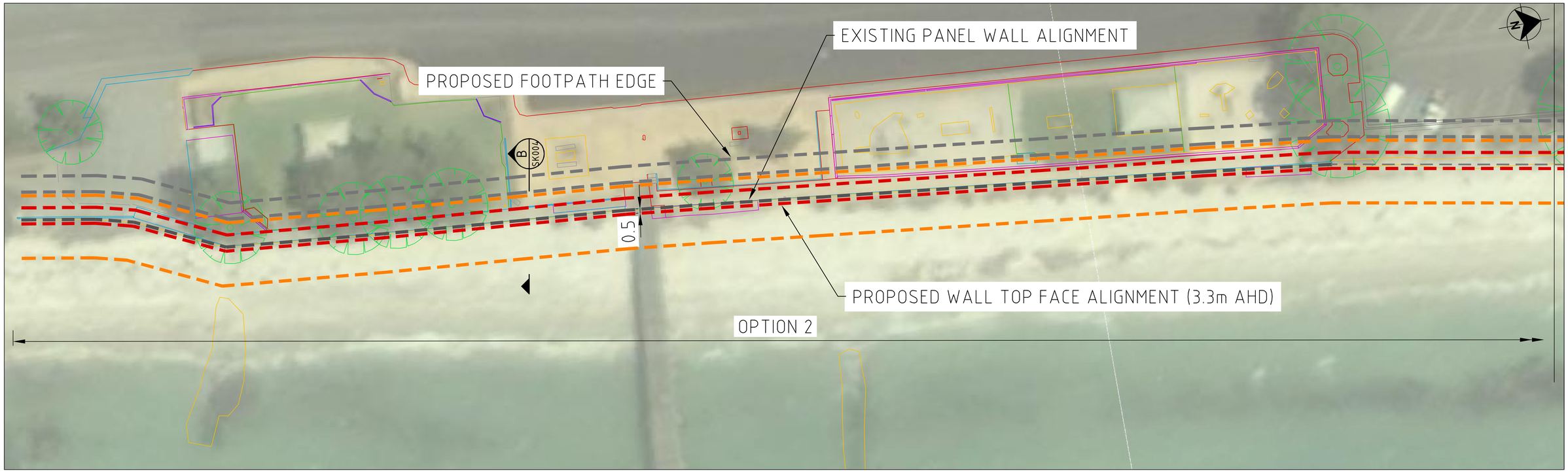

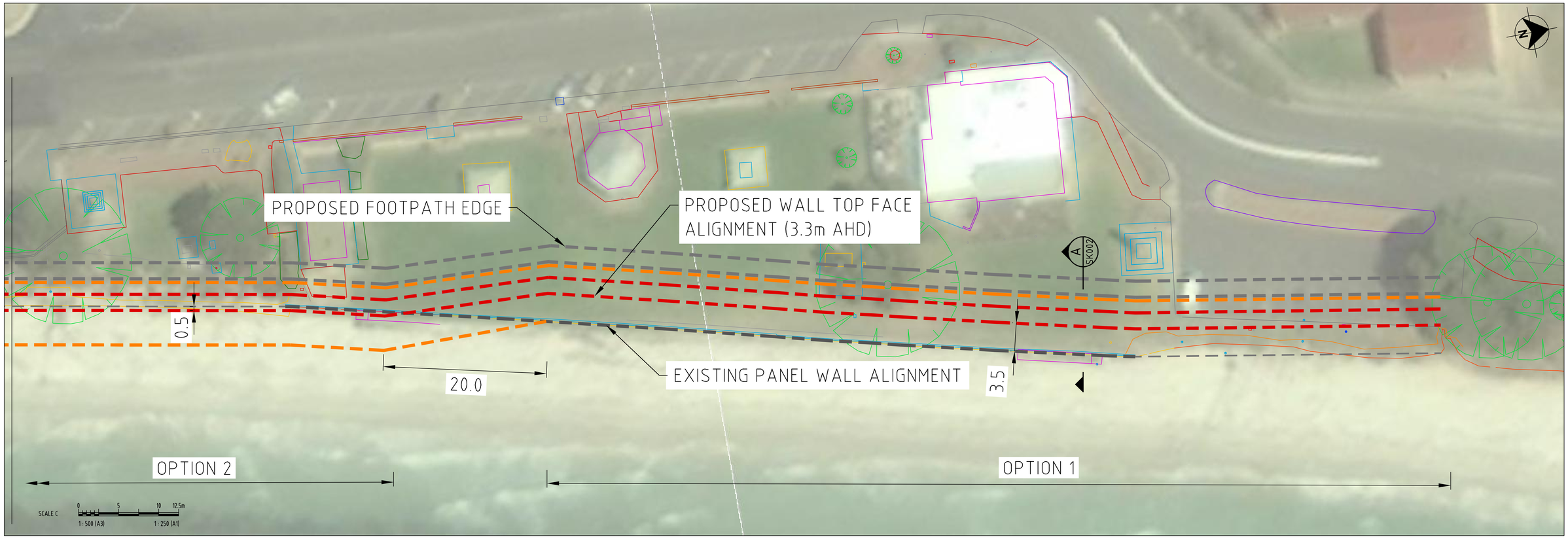

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A.B.N. 20 093 846 925

Project: DISTRICT COUNCIL OF TUMBY BAY

Title: TUMBY BAY FORESHORE PROTECTION CONSULTANCY SERVICES
PROPOSED SEAWALL CONCEPT PLAN
OPTION 3

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