Assessment of geomorphic changes, hydraulics and potential foreshore stabilisation options at Windang, Lake Illawarra

WRL TR 2023/08, December 2023

By T A Tucker, I R Coghlan and J T Carley

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Executive summary

The Windang foreshore study site is located on the northern bank of the Lake Illawarra entrance channel downstream of the Windang Bridge. It has significant value for the local community and is a popular recreation site. Furthermore, there are a number of significant indigenous heritage locations in its vicinity.

In 2007, the entrance channel of Lake Illawarra was permanently connected to the ocean following the construction of two training walls. Since this time, there has been significant erosion of the foreshore threatening assets and the amenity it provides to the local community. The permanent opening of the entrance resulted in fast flowing tidal water passing through the channel due to the difference in water levels between Lake Illawarra and the ocean. This fast flowing water which occurs on both incoming and outgoing tides has resulted in significant sediment transport within the entrance channel with erosion/deepening at some locations and accretion elsewhere.

The Windang foreshore has been acutely impacted by the erosive processes linked with the construction of the training walls. In October 2022, following years of partial collapse, the last sections of a jetty structure (previously known as the Pine Tree Park boardwalk) located at the Windang foreshore finally collapsed and were subsequently removed. The collapse of this jetty structure highlighted how continued erosion/deepening within the entrance channel caused by the training of the ocean entrance is continuing to impact public amenity.

The Lake Illawarra Coastal Management Program (CMP) was prepared in 2020 to guide the management of Lake Illawarra. In response to the negative changes which have occurred since the entrance was trained, addressing erosion within the entrance channel of Lake Illawarra was identified as action EC1 of the CMP. One of the tasks outlined within action EC1 is to prepare a “Lake entrance management options study”. Concurrently with this Windang foreshore investigation, the UNSW Sydney Water Research Laboratory (WRL) is also working with a Project Control Group led by Wollongong City Council (WCC), which includes several other stakeholders (including Crown lands) to address this task. The work with WCC aims to identify large-scale options to address changes to the entrance channel; primarily to reduce foreshore and bed erosion and reduce an increasing tidal regime within the lake.

While the “Lake entrance management options study” is ongoing, a need to assess the ongoing management of the Windang foreshore in the short-term has been identified. Subsequently, the investigation outlined in this report has been completed by WRL for Crown Lands to provide a local scale understanding of the Windang foreshore and inform its future management within a 20 year time horizon. The analysis and options for the Windang foreshore in this report are based on a scenario whereby actions from the “Lake entrance management options study” have not been implemented over this planning period.

A review of geomorphic change at the Windang foreshore showed the erosion along the Windang foreshore is occurring at the fastest rate of the entire entrance channel (erosion of up to 20 m^3/m/year between 2008 and 2022. This is equivalent to a vertical lowering of approximately 5 m over this time or 0.35 m/year). By extrapolating this erosion rate indicative cross-sections have been developed to illustrate the projected extent of erosion if it is allowed to continue for a further 20 years (i.e. to the year 2043) without intervention. Review of the projected cross-sections shows that, in the future, existing assets located along the Windang foreshore are likely to be at risk from erosion. This includes the existing roadway which provides access to Windang Surf Life Saving Club as well as park facilities such as tables, shelters and the playground.

Previous investigations have found that the Lake Illawarra entrance channel is continually changing and erosion could continue for another 120 years. WRL used a numerical model to assess the local scale hydraulics at the Windang foreshore for present day and plausible future (i.e. in 20 years' time) conditions including sea level rise. Hydraulic conditions were driven by tidal flows and subsequently, maximum velocity and bed shear calculations are based on day-to-day conditions. Review of the numerical model results found that during larger spring tides, the maximum velocities will be higher in a plausible future state (2043), compared to present day conditions. Significant levels of bed shear were observed in present day and plausible future model results, indicating that erosion along the Windang foreshore is likely to continue into the future. Unless the Windang foreshore is directly protected, or a large-scale mitigation measure is implemented elsewhere in the Lake Illawarra entrance channel, it is likely to continue to erode for the foreseeable future.

WRL prepared eight potential foreshore stabilisation design options (to a concept design level including indicative costings) to prevent further bank erosion and maintain the present position of the bank along the Windang foreshore. A 20 year design life (2023 to 2043) was adopted for the concept designs to protect the bank, assuming that no other mitigation measures have been implemented in the Lake Illawarra entrance channel over this planning period. The options developed for the study site were either vertical or sloping revetments composed of steel sheet piles, concrete secant piles, rock armour and sand-filled geotextile containers. WRL also assessed the advantages and disadvantages of each of the eight potential foreshore stabilisation design options.

While assessing the feasibility of a new jetty to replace the one that collapsed in October 2022 was outside the scope of this investigation, analysis from this study can inform the future management of the Windang foreshore and guide the future decision as to whether the jetty will be replaced.

This investigation focused on erosion and local scale hydraulics occurring at Windang presently (2023) and those likely to occur in the future (20 years from now) if a large-scale erosion mitigation measure (designed to reduce tidal conveyance) is not implemented elsewhere in the Lake Illawarra entrance channel. Based on this analysis, the following conclusions can be drawn:

- High velocities (>1 m/s) will continue to occur along the Windang foreshore for the foreseeable future with or without a bank stabilisation structure.
- While a bank stabilisation structure will prevent further landward erosion, significant deepening of the entrance channel immediately adjacent to the Windang foreshore will continue to occur with or without a bank stabilisation structure.

On this basis, WRL recommends that the following points be considered during the planning for a new jetty in the area:

- The costs associated with a new jetty structure will be significantly influenced by the future erosion/scour expected to occur around the jetty piles over its design life (e.g. design scour level).
- High water velocities at a new jetty structure at the same location as the previous one may pose a risk to safety for associated in-water recreation, particularly for some unpowered activities (e.g. swimming, canoeing/kayaking, stand-up paddle boarding, etc).
- An alternative location within the Lake Illawarra entrance with a lower long-term erosion rate and lower water velocities may be more suitable for a jetty structure.

Contents

List of tables

List of figures

1 Introduction

1.1 Study site

The Windang foreshore study site is located on the northern bank of the Lake Illawarra entrance channel downstream of the Windang Bridge (Figure 1.1). The Windang foreshore has significant value for the local community and is a popular recreation site. Furthermore, there are a number of significant indigenous heritage locations in its vicinity. Since 2007, there has been significant erosion of the foreshore threatening assets and the amenity it provides to the local community.

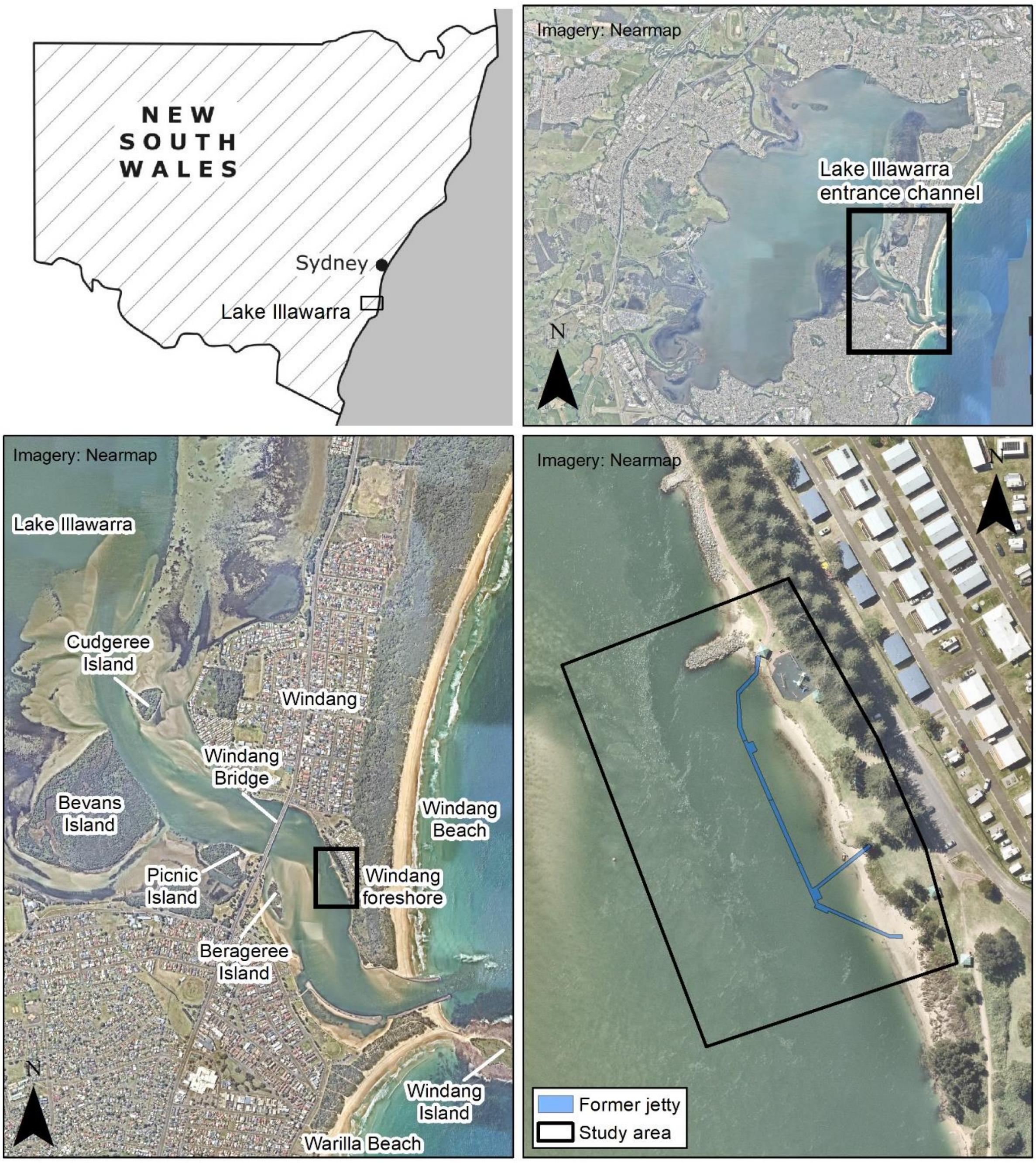

Figure 1.1 Windang foreshore study site location including the Lake Illawarra entrance (left) and the project extent (right)

In 2007, the entrance channel of Lake Illawarra was permanently connected to the ocean following the construction of two training walls. The permanent opening of the entrance resulted in fast flowing tidal water passing through the channel due to the difference in water levels between Lake Illawarra and the ocean. This fast flowing water which occurs on both incoming and outgoing tides has resulted in significant sediment transport within the entrance channel with erosion/deepening at some locations and accretion elsewhere.

The Windang foreshore has been significantly impacted by erosion. In October 2022, following years of partial collapse, the last sections of a jetty structure (previously known as the Pine Tree Park boardwalk) located at the Windang foreshore finally collapsed and were subsequently removed (Figure 1.2). The collapse of this jetty structure highlighted how continued erosion/deepening within the entrance channel caused by the training of the ocean entrance is continuing to impact public amenity.

Figure 1.2 Windang foreshore with the jetty in December 2019 (Source: AllTrails, 2023) and without the jetty in March 2023 (Source: Ian Coghlan)

1.2 Coastal Management Program (CMP)

The Lake Illawarra Coastal Management Program (CMP) was prepared in 2020 to guide the management of Lake Illawarra. In response to the negative changes which have occurred since the entrance was trained, addressing erosion within the entrance channel of Lake Illawarra was identified as action EC1 of the CMP (Rollason and Donaldson, 2020). Action EC1 is described in the CMP as: "*Investigate and Finalise Options to Manage Erosion and Accretion Changes in the Entrance Channel*". One of the tasks outlined within action EC1 is to prepare a "Lake entrance management options study". Concurrently with this Windang foreshore investigation, the UNSW Sydney Water Research Laboratory (WRL) is also working with a Project Control Group (PCG) led by Wollongong City Council (WCC), which includes several other stakeholders (including Crown lands) to address this task within action EC1. The work with WCC aims to identify large-scale options to address changes to the entrance channel; primarily to (1) reduce foreshore and bed erosion and (2) reduce an increasing tidal regime within the lake.

While the "Lake entrance management options study" is ongoing (refer to the following progress reports: Tucker et al., 2023 and Coghlan et al., 2023), there is still a need to address local issues resulting from changes in the entrance channel. Subsequently, the investigation outlined in this report has been completed to address site specific erosion along the Windang foreshore.

It is noted that the applicability of actions EC3 and EC5 from the CMP should also be considered if one of the foreshore stabilisation design options is selected to progress to detailed design. Action EC3 is described in the CMP as: "*Undertake emergency works or small scale no regrets actions as required to mitigate known risks to property and public safety*" and the Windang foreshore is referred to as an example action: "*shoreline protection and works to make safe sections of failing existing protection works east of Windang Bridge, northern side (i.e. at Tourist Park)*". Action EC3 describes works which may be progressed prior to the completion of action EC1, "*but their impact on the wider entrance channel needs to be considered and addressed*". Action EC5 is described in the CMP as: "*Monitor and maintain existing entrance channel infrastructure, with any works to be informed by EC1, EC3 and EC4".* Conversely to the emergency and small scale works referred to by action EC3, it is noted within action EC5 that the "*outcomes of EC1 shall guide any major upgrades to infrastructure, which may be required to ameliorate erosion/accretion impacts etc.*" (Rollason and Donaldson, 2020).

1.3 Context of this investigation

While the "Lake entrance management options study" is ongoing, a need to assess the ongoing management of the Windang foreshore in the short-term has been identified. The following investigation has been completed to provide a local scale understanding of the Windang foreshore and inform its future management within a 20 year time horizon. The analysis and options for the Windang foreshore in this report are based on a scenario whereby actions from the "Lake entrance management options study" have not been implemented over this planning period. Consequently, this report comprises the following sections:

Section 1: Introduction (this section)
Section 2: Site inspection – A summary of the site inspection conducted on 3 March 2023
Section 3: Review of geomorphic change at the Windang foreshore – A review of available data to assess the current and likely future state of the Windang foreshore
Section 4: Local scale hydraulic assessment – Results of a numerical model assessment of the velocities and bed shear for the Windang foreshore for the present day and a future scenario
Section 5: Foreshore stabilisation design options – Eight options available to stabilise the Windang foreshore and protect it against erosion into the future (including indicative costings)
Section 6: Afterword
Section 7: References

WRL understands that the subject Crown land is Crown waterway and part of WCC managed Crown Reserve 53977, for public recreation, which interfaces with the Crown waterway.

The potential for scour and its management around coastal structures under wave action and currents may involve input from coastal engineers, geotechnical engineers and structural engineers. WRL staff have expertise in coastal engineering, and frequently work with geotechnical and structural engineers if required. The scope of this project was limited to coastal engineering.

2 Site inspection

On 3 March 2023, a site inspection was conducted during a falling tide by WRL engineers Ian Coghlan and Toby Tucker to assess the current condition of the Windang foreshore (Figure 2.1, Figure 2.2, Figure 2.3 and Figure 2.4). During the site inspection a number of photographs of the site were taken to document the state of the foreshore. This included a number of high-resolution photos taken using a drone (Figure 2.5, Figure 2.6 and Figure 2.7).

While on-site, WRL engineers met with Simon Williams and Nikhil Ajgaonkar from Crown Lands to discuss the current foreshore condition and the ongoing project.

Figure 2.1 Erosion along the Windang foreshore immediately downstream of the groyne

Figure 2.2 Erosion along the Windang foreshore as viewed from the end of the groyne

Figure 2.3 Erosion along the Windang foreshore immediately downstream of the groyne

Figure 2.4 Erosion along the Windang foreshore looking towards the ocean entrance

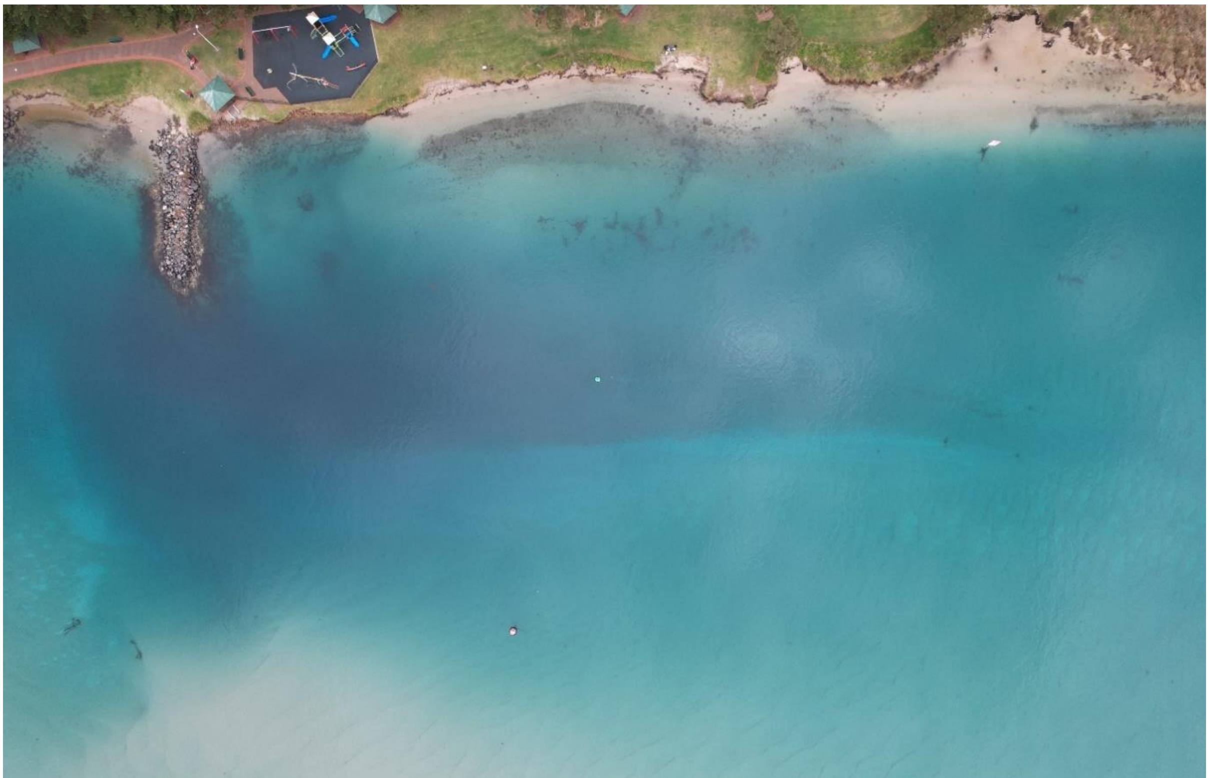

Figure 2.5 The Windang foreshore from directly above

Figure 2.6 The Windang foreshore looking towards the ocean

Figure 2.7 The Windang foreshore looking towards Lake Illawarra

3 Review of geomorphic change at the Windang foreshore

3.1 Preamble

The geomorphic change at the Windang foreshore has been assessed using two methods:

1. Aerial imagery (Section 3.2)
2. Bathymetric data (Section 3.3)

Aerial imagery has been used to provide a descriptive review of geomorphic change. Imagery was found to be useful in identifying key erosion events that occurred at Windang. Bathymetric data has allowed for an analytical approach for assessing geomorphic change. This analysis has reviewed existing data, identified historical erosion rates along the Windang foreshore, and used this information to inform the scale of potential future erosion.

3.2 Analysis of aerial imagery

3.2.1 1938 to 2012 (Young, 2013)

Review of aerial imagery has previously been completed by Young (2013) using historical photographs from 1938 to 2012 provided by the Lake Illawarra Authority. Their analysis focused on entrance openings with the following observations:

- Prior to 2000 the entrance was highly variable with multiple locations (including both sides of Windang Island) where opening would occur dependent upon wave-induced shoaling and rainfall.
- From 2002 to 2007 training of the southern side of the entrance meant the location of opening only occurred at one location to the north of Windang Island.
- Since training in 2007 the entrance remained permanently opened.

The aerial imagery provided by Young (2013) has been analysed for this study to identify key changes to the Windang foreshore prior to 2012:

- Prior to 2007 the entrance foreshore at Windang remained stable with seasonal fluctuations to the extent of seagrass/mudflats.
- Following the entrance opening in 2007 the establishment of a well-defined channel within the entrance resulted in the gradual erosion of the Windang foreshore.

3.2.2 2010 to 2023 (Nearmap aerial imagery)

Aerial imagery of the Windang foreshore study area has been captured regularly by Nearmap since 2010. By comparing selected images, key changes to the foreshore including the progressive failure of the former jetty structure can be observed (see Figure 3.1):

- January 2010 to April 2012: Significant erosion occurs resulting in the loss of shallow seagrass/mudflats.
- April 2012 to July 2012: The groyne is constructed upstream of the jetty.
- July 2012 to June 2014: The first sections of the jetty are removed.
- June 2014 to October 2016: Further sections of the jetty are removed.
- October 2016 to February 2022: A hole off the end of the groyne continues to deepen.
- February 2022: to April 2022: Another section of jetty is removed.
- April 2022 to March 2023: The remaining sections of the jetty are removed after final collapse in December 2022.

Note, comparison of estuarine vegetation mapping completed by DPI (2023) between 2005 and 2019 shows that there has been a significant decline in coastal wetland habitat (specifically seagrass), such that it no longer exists at the study site.

Figure 3.1 Changes along the Windang foreshore observed in Nearmap imagery from 2010 to 2023

3.3 Bathymetric analysis

3.3.1 Review of bathymetric survey data

The bathymetry in the Lake Illawarra entrance channel has been surveyed on eight occasions since 2007. Each of these surveys included measurements of the bathymetry at the Windang foreshore study site. The survey detail at the study site varied between surveys as shown in Figure 3.3. To assist in comparing the change in the bathymetry at the study site, three cross-sections were chosen where there was a regular overlap between surveys. The locations of the cross-sections are shown in Figure 3.2, with a comparison of the elevation levels shown from north to south order in Figure 3.4, Figure 3.5 and Figure 3.6.

Comparison of bathymetric data clearly shows the erosion of a hole in front of the groyne located within the study area. In Figure 3.3, the hole clearly forms between 2007 and 2018. Between 2008 and 2022 the hole caused by erosion has developed to be ~12 m deeper. Figure 3.4 shows that erosion of sand at the toe of the groyne is still continuing despite having some variability. Indeed, this is the case at each of the cross-sections identified (see Figure 3.5 and Figure 3.6).

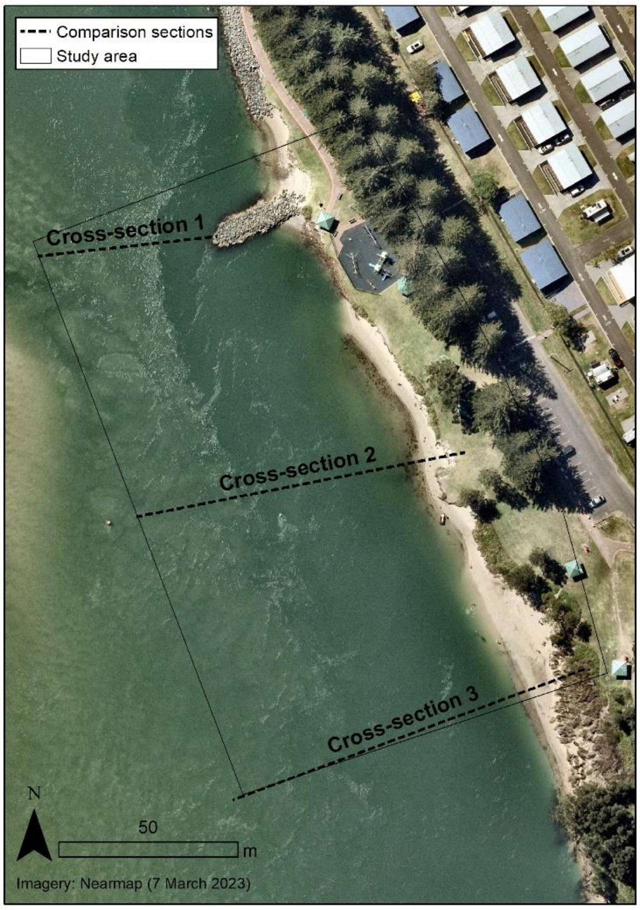

Figure 3.2 Location of comparison cross-sections

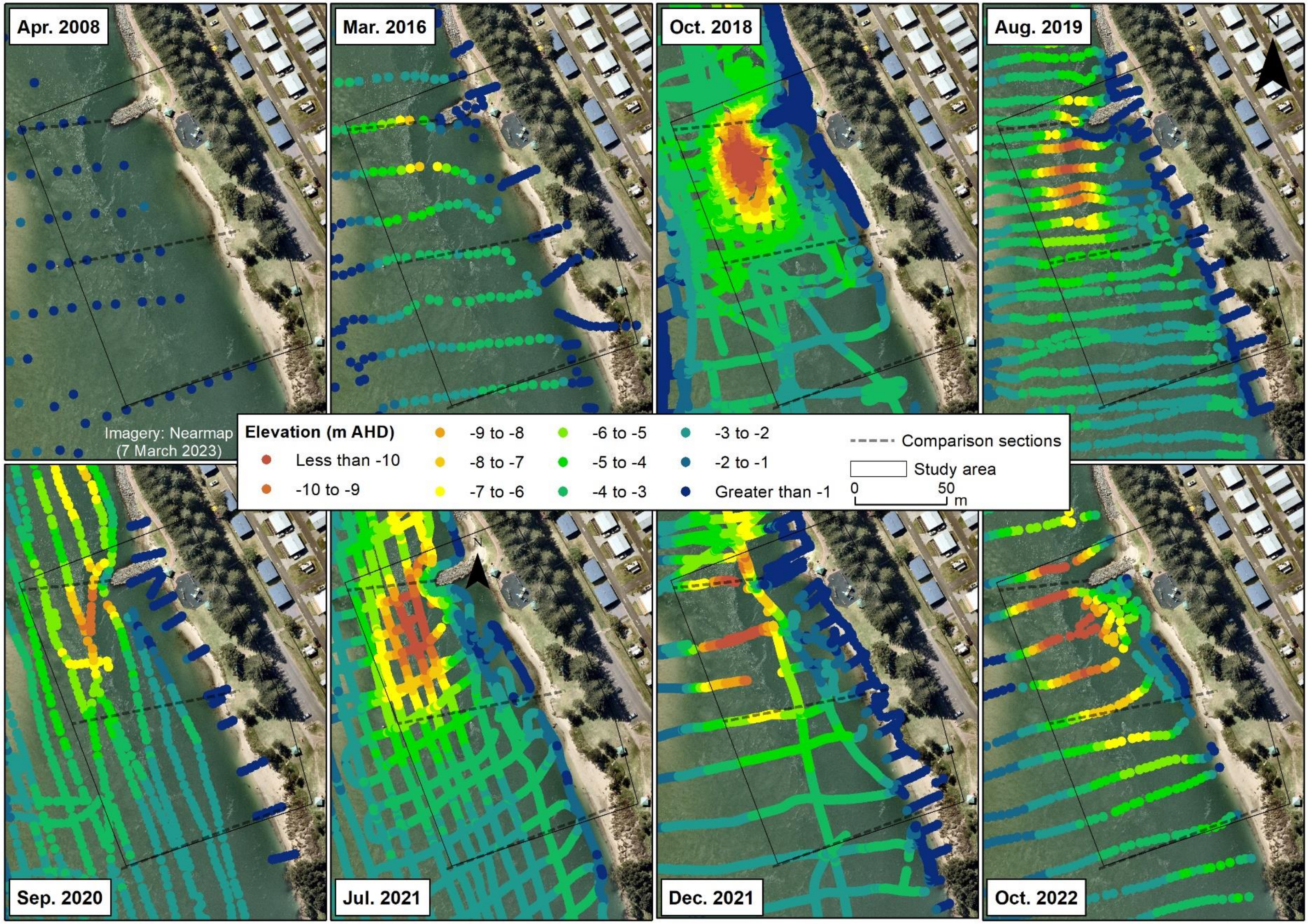

Figure 3.3 Change in bathymetry between eight surveys completed from 2008 to 2022 (note: all background images from Nearmap, 7 March 2023)

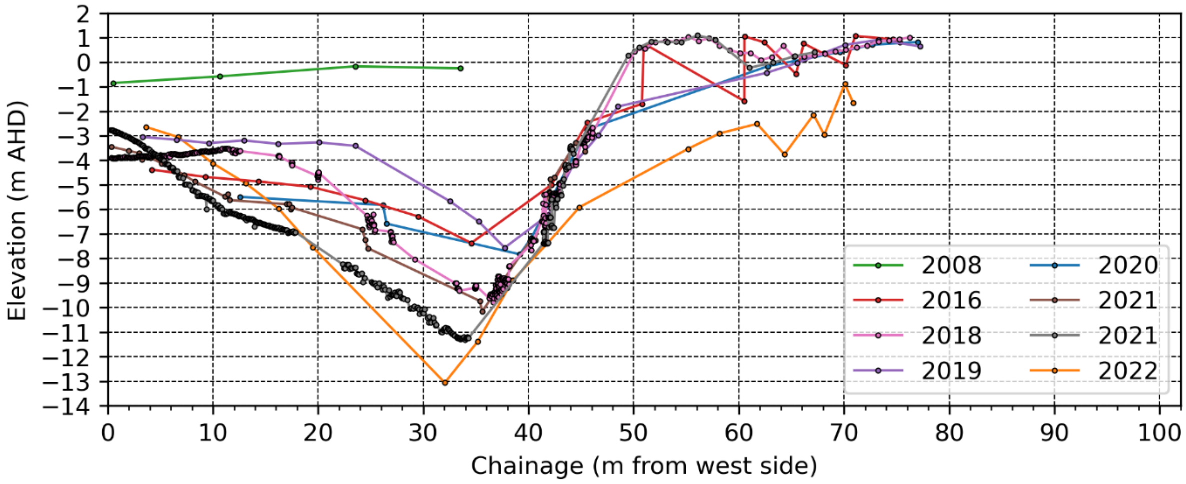

Figure 3.4 Change in bathymetry at cross-section 1 (Figure 3.2) to the north of the study area

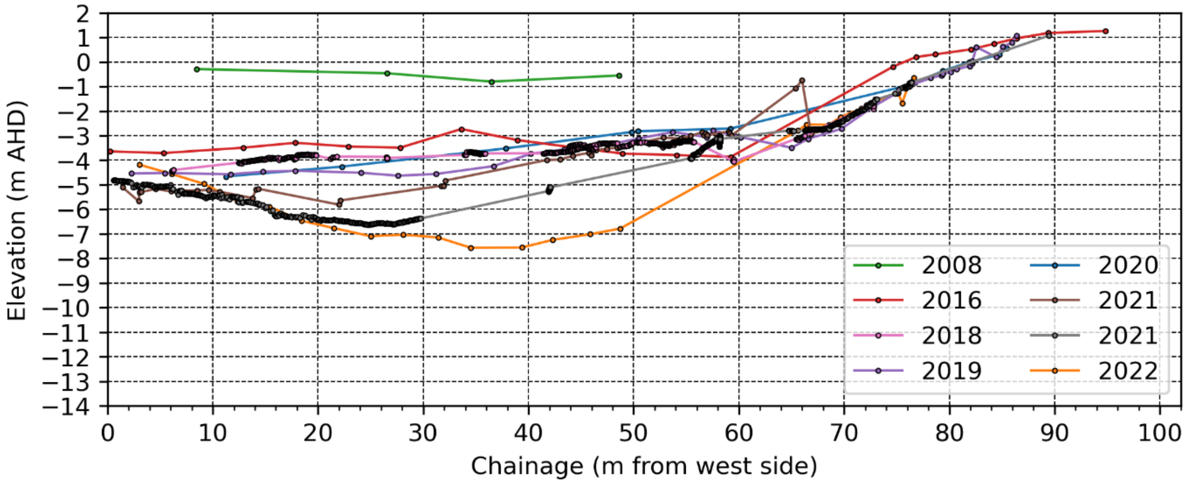

Figure 3.5 Change in bathymetry at cross-section 2 (Figure 3.2) in the middle of the study area

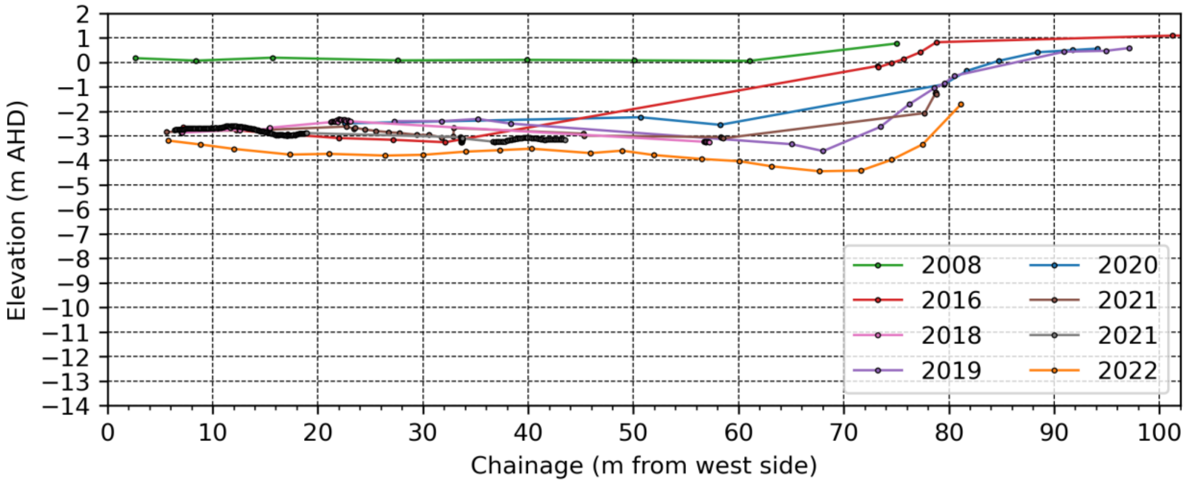

Figure 3.6 Change in bathymetry at cross-section 3 (Figure 3.2) to the south of the study area

3.3.2 Long-term rate of erosion

Review of erosion rates within the Lake Illawarra entrance channel completed by Tucker et al. (2023) noted that the erosion along the Windang foreshore is occurring at the fastest rate of the entire entrance channel (Figure 3.7). Utilising the bathymetric data shown in Section 3.3, the following section assesses the rate of erosion locally at the Windang foreshore.

Figure 3.7 Average annual sediment change rate within each sediment compartment of the Lake Illawarra entrance channel (Tucker et al., 2023)

Analysis of cross-section data across the study area indicated that the rate of erosion across the Windang foreshore varies from year to year. While this is the case, there is still a clear long-term erosion trend as is shown in the cross-section data across the study area (see Figure 3.4, Figure 3.5 and Figure 3.6). Using a subset of these same cross-sections to ensure bathymetry data was available for all years, long-term rates of erosion have been calculated across the study site using a line of best fit (cross-section locations are shown in Figure 3.2):

- North of the study area: 20 m^3/m/year of erosion (Figure 3.8; considering chainages: 13-70 m)
- Middle of the study area: 18 m^3/m/year of erosion (Figure 3.9; considering chainages: 12-65 m)
- South of the study area: 13 m^3/m/year of erosion (Figure 3.10; considering chainages: 24-75 m

Results of the assessment indicated that the rate of erosion decreased from north to south across the study area with the highest rate of erosion located at the groyne.

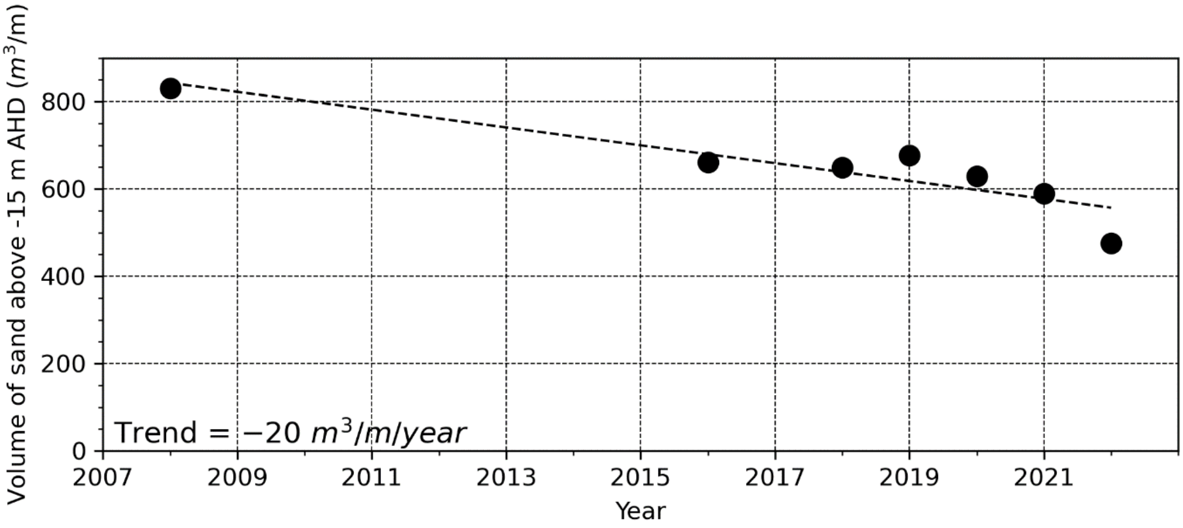

Figure 3.8 Rate of erosion at cross-section 1 (Figure 3.2) since 2007 (chainages: 13-70 m)

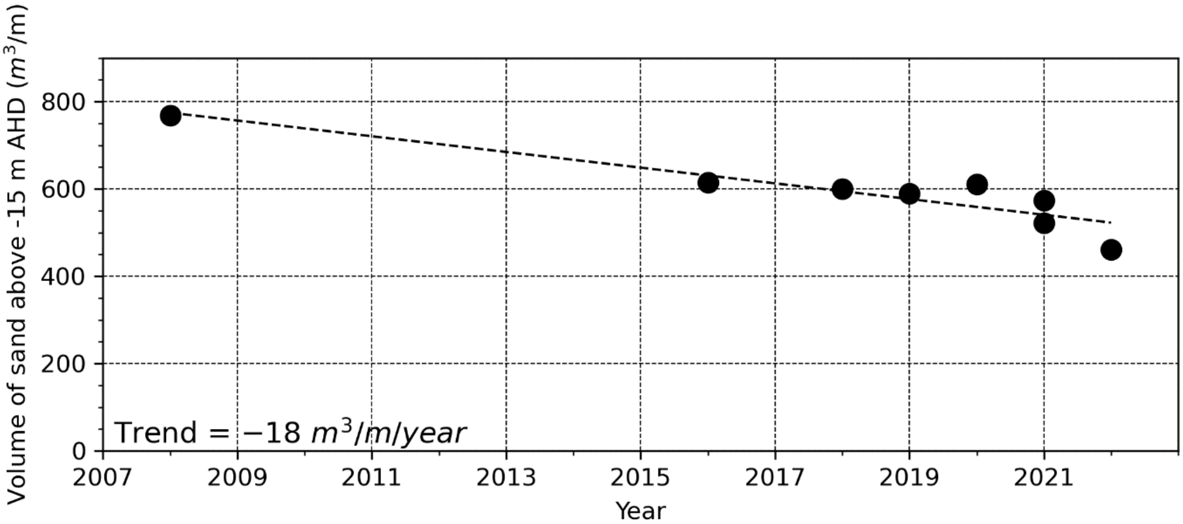

Figure 3.9 Rate of erosion at cross-section 2 (Figure 3.2) since 2007 (chainages: 12-65 m)

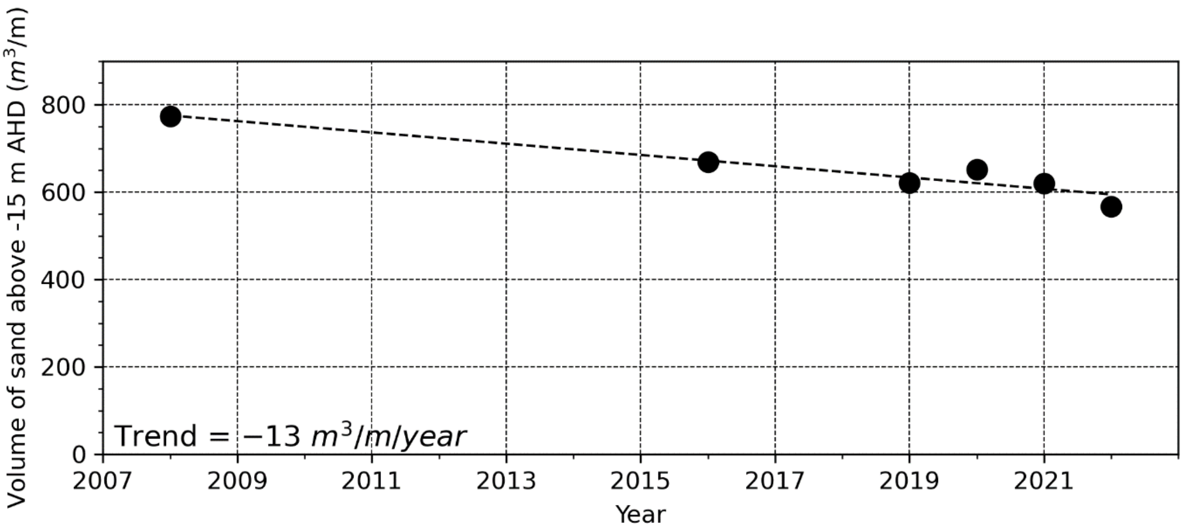

Figure 3.10 Rate of erosion at cross-section 3 (Figure 3.2) since 2007(chainages: 24-75 m)

3.3.3 Indicative future cross-sections

Based on the highest long-term erosion rate of 20 m^3/m/year for a cross-shore distance of 57 m (considering chainages 13-70 m at cross-section 1; equivalent to vertical change of 0.35 m across the profile), indicative cross-sections have been developed to illustrate the projected extent of erosion if it is allowed to continue for a further 20 years (i.e. to the year 2043) without intervention. Locations for the eight indicative cross-sections (denoted cross-sections F1 to F8) are shown in Figure 3.11 and were chosen based upon existing data availability. Indicative cross-sections for future erosion states, compared to 2008 and 2023 states, are shown from Figure 3.12 to Figure 3.19 in order from north to south.

To determine the difference in volume per lineal metre (i.e. cross-sectional area) due to erosion of sand over a 20-year period at cross-sections F1 to F8, the maximum chainage of each 2023 cross-section was first multiplied by 7 m (i.e. the highest vertical long-term erosion rate 0.35 m × 20 years) effectively deepening the 2023 profile by 7 m. For example, for cross-section F4 (Figure 3.15), the maximum chainage on the 2023 cross-section is 84 m, resulting in a future eroded volume of 588 m^3/m. Since erosion landward of the 2023 bank position would contribute to this future eroded volume, the deepened 2023 profile was then raised slightly. For cross-section F4, the 2023 profile was deepened by only 5.7 m (479 m^3/m) to account for the estimated 109 m^3/m of erosion landward of the 2023 bank position. The sum of these two volumes resulted in the equivalent future eroded volume of 588 m^3/m.

Review of the projected cross-sections shows that, in the future, existing assets located along the Windang foreshore are likely to be at risk from erosion. This includes the existing roadway which provides access to Windang Surf Life Saving Club as well as park facilities such as tables, shelters and the playground. Note, the actual slope angle of the foreshore will depend upon factors such as the forces leading to erosion and sediment type/size. Indicative erosion profiles presented here are approximations only and the actual slope angles may vary. Slope angles measured from existing data at the Windang foreshore ranged from 1 Vertical (V) : 2 Horizontal (H) (typical for sandy clays) to flatter than 1V:5H (typical for fine sands) (Bray, 1979). Sediment samples collected in the channel adjacent to the foreshore and presented by Tucker et al. (2023) found that the D_{50} sediment size was between 0.5 and 0.25 mm. Sediment of this size is classified as medium to coarse sand on the Wentworth scale and would be expected to have a stable slope of approximately 1V:4H.

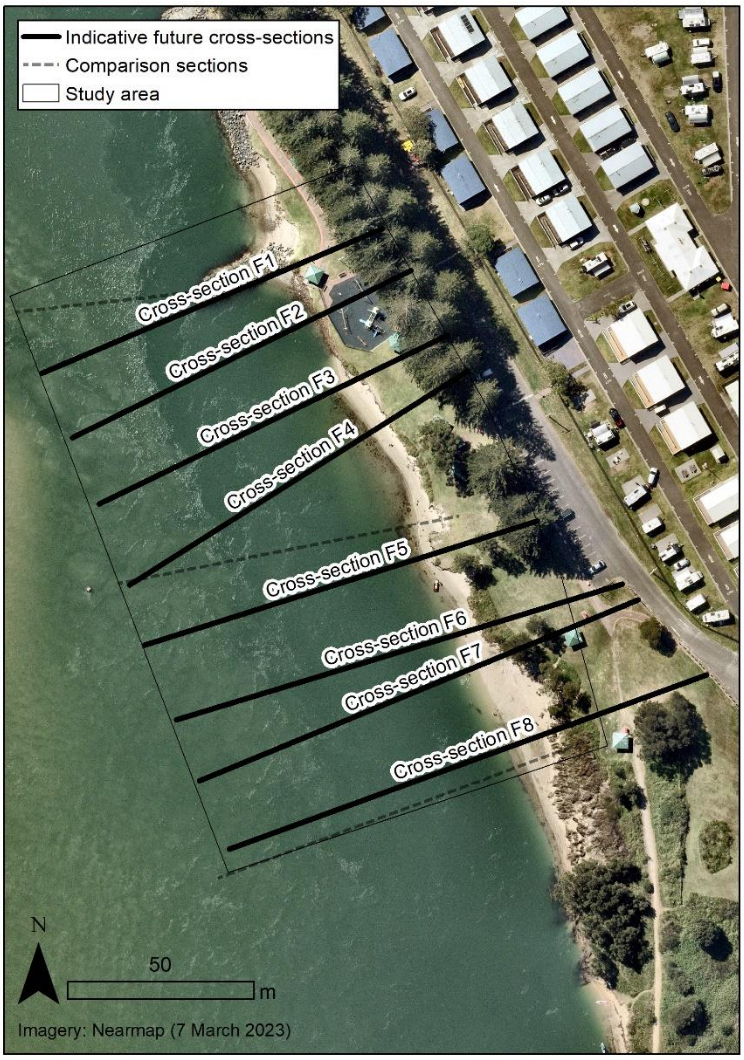

Figure 3.11 Locations for projected future cross-section estimates

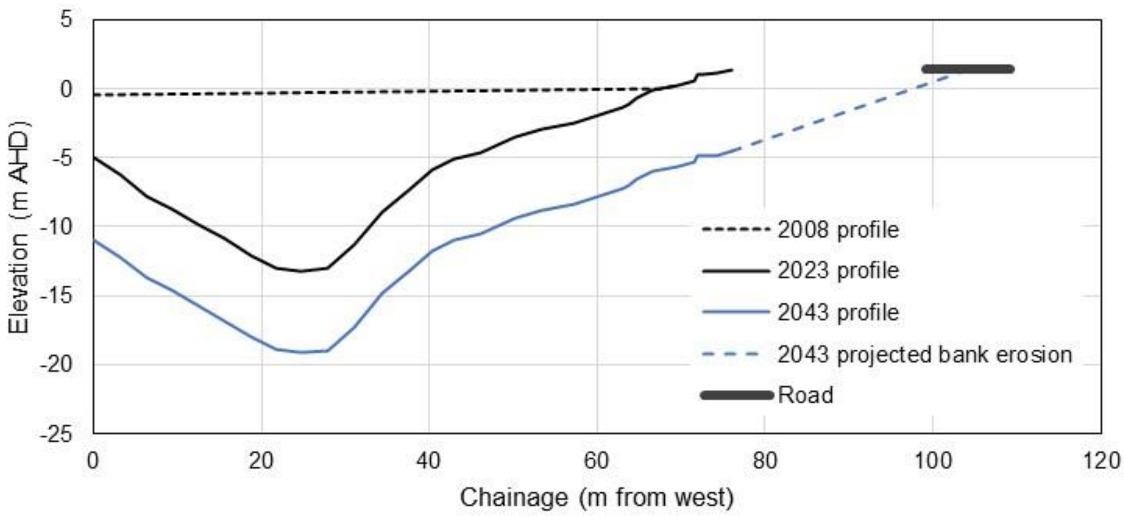

Figure 3.12 Indicative future erosion prediction at cross-section F1

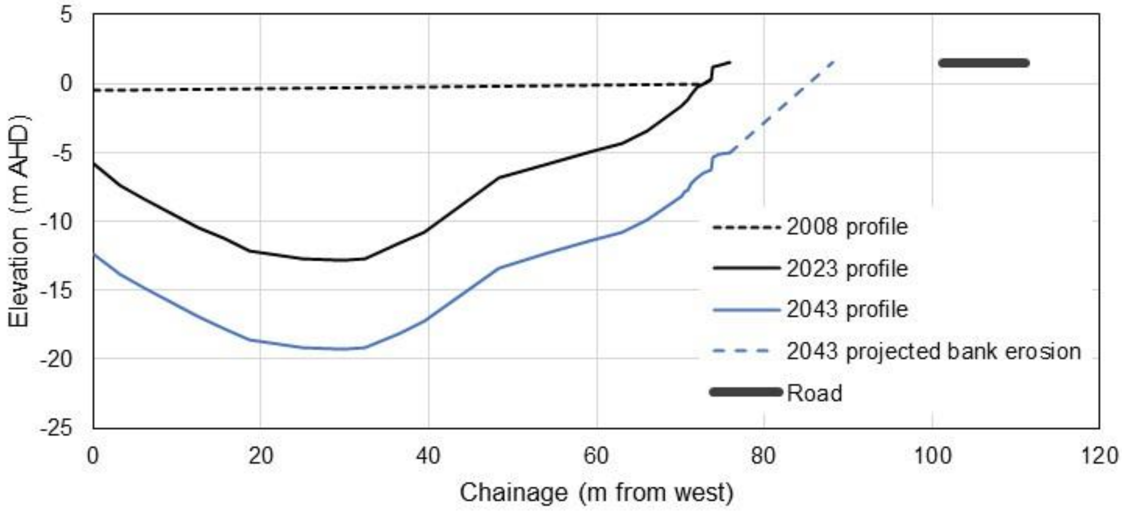

Figure 3.13 Indicative future erosion prediction at cross-section F2

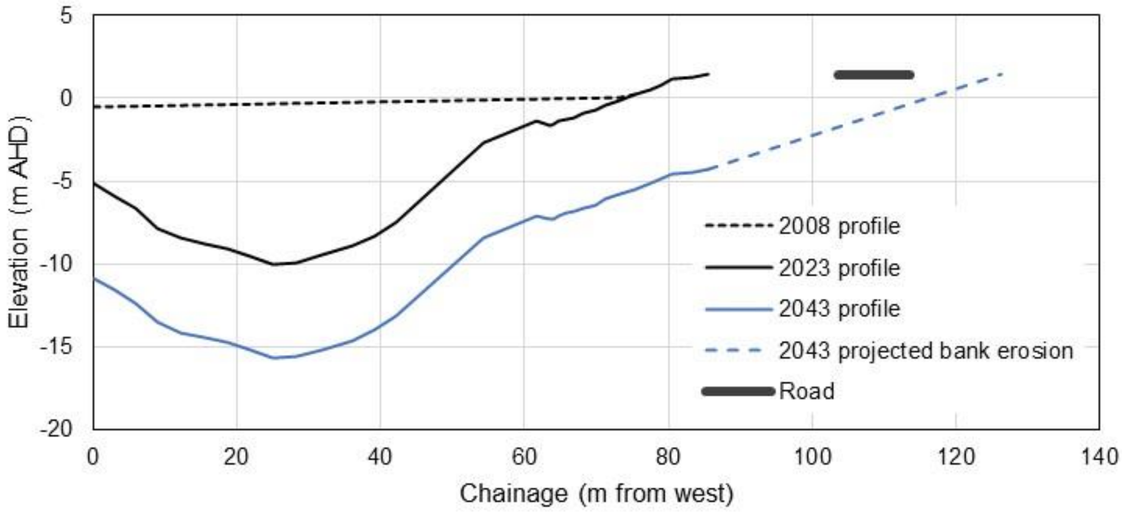

Figure 3.14 Indicative future erosion prediction at cross-section F3

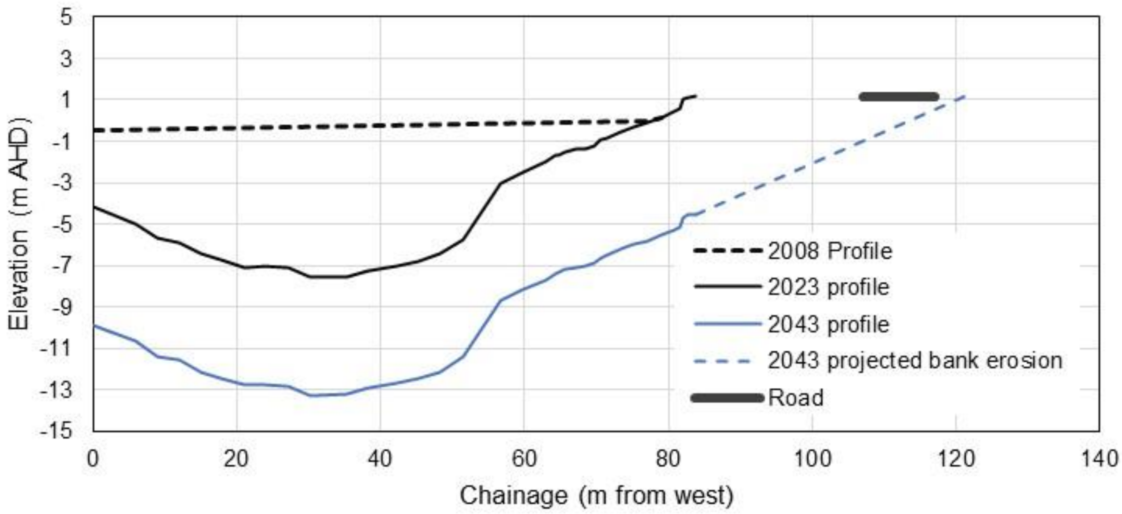

Figure 3.15 Indicative future erosion prediction at cross-section F4

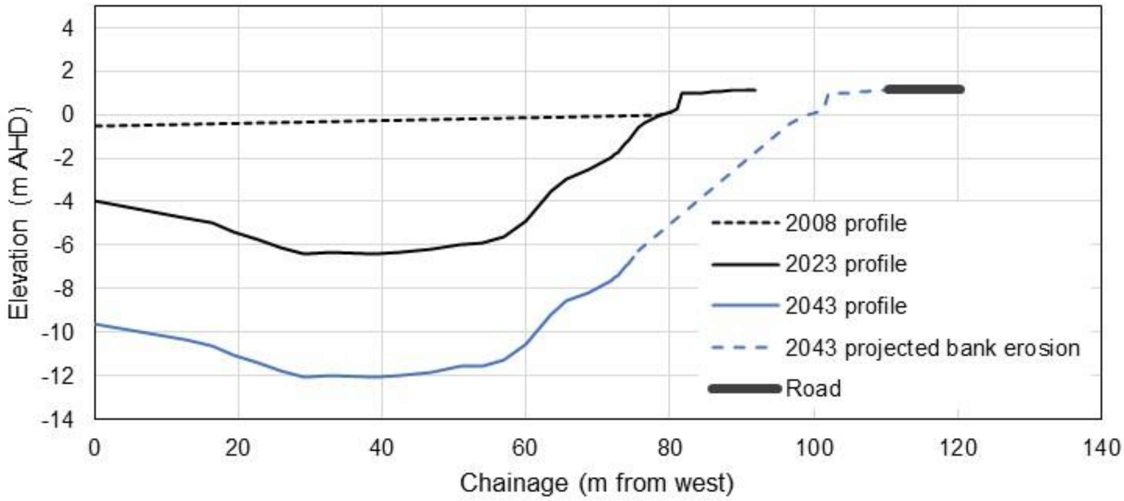

Figure 3.16 Indicative future erosion prediction at cross-section F5

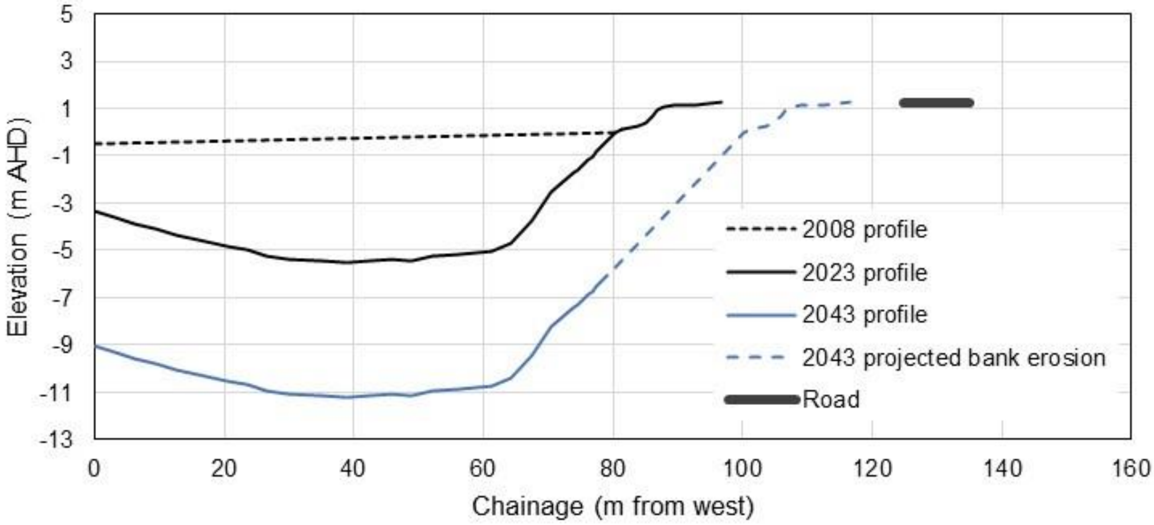

Figure 3.17 Indicative future erosion prediction at cross-section F6

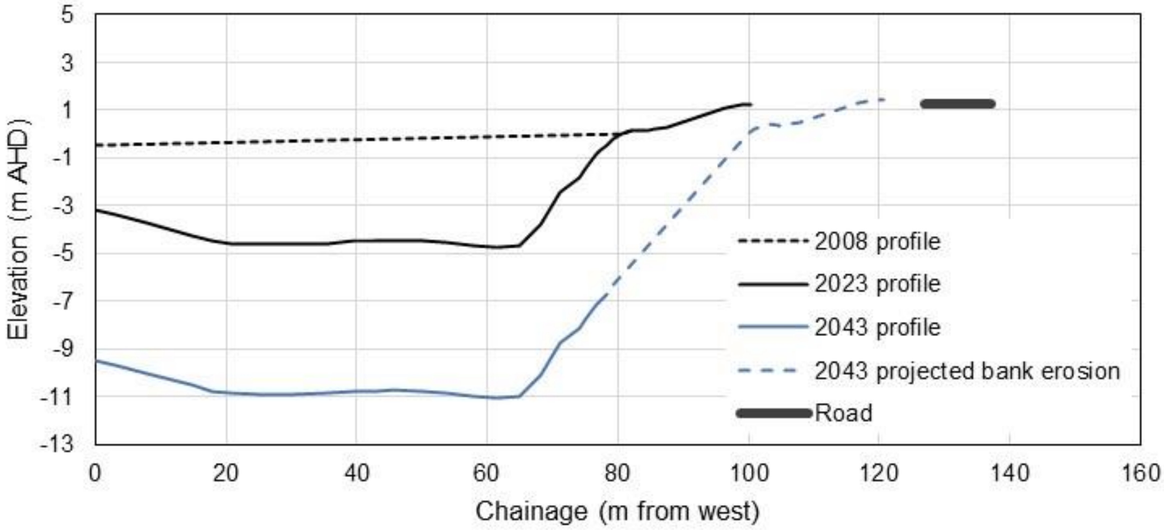

Figure 3.18 Indicative future erosion prediction at cross-section F7

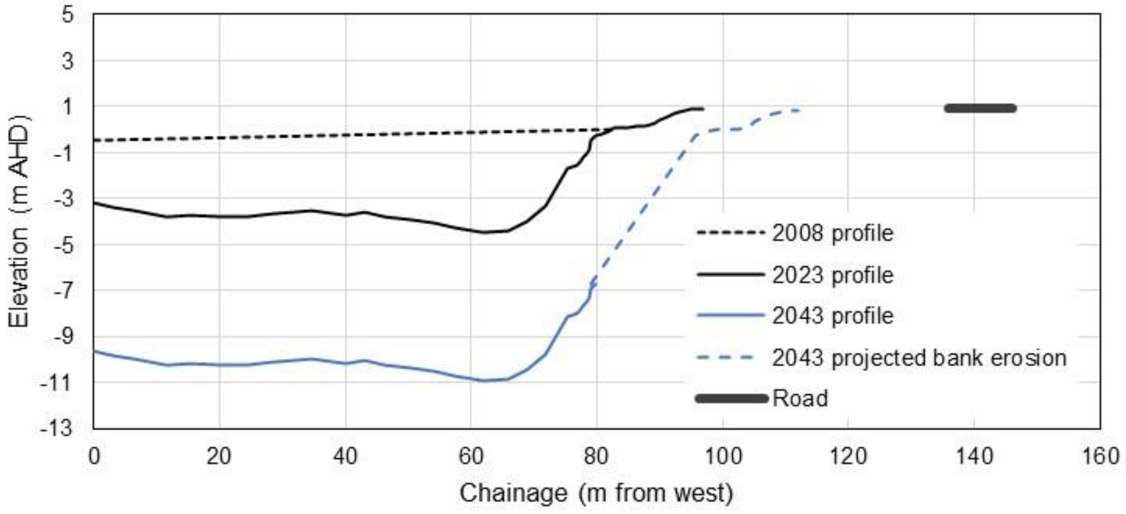

Figure 3.19 Indicative future erosion prediction at cross-section F8

4 Local scale hydraulics

4.1 Preamble

A numerical model has been used to assess the local scale hydraulics at the Windang foreshore for present day and plausible future (i.e. in 20 years' time) conditions including sea level rise. For this investigation a depth averaged two-dimensional model was used with a horizontal resolution of approximately 5 m^2 within the study area. Hydraulic conditions were driven by tidal flows and subsequently, maximum velocity and bed shear calculations are based on day-to-day conditions. Variations in velocity and bed shear would be expected during flood conditions, however, for the purpose of this investigation a tidally driven flow regime is considered appropriate. Further details regarding the model development and analysis can be found in Appendix A. The following sections provide the modelling results for the present day (Section 4.2) and future (i.e. 2024) (Section 4.4) local scale hydraulics, including depth-averaged velocity and bed shear, along with an assessment of the findings (Section 4.5).

4.2 Overview of changes to the Lake Illawarra entrance channel hydraulics

Previous investigations have found that the Lake Illawarra entrance channel is continually changing and erosion could continue for another 120 years (Glatz, 2023). Analysis of numerical model results shows how erosion within the entrance channel may influence hydraulic conditions. Water level data outputs from the numerical model show that as continued erosion within the entrance channel occurs, the tidal range will increase (note, water levels will also be influenced by sea level rise which was modelled as +0.1 m to 2043) (Figure 4.1). Similarly, as erosion and sea level rise cause a larger cross-sectional area for the entrance channel, the total flow through the Lake Illawarra entrance channel will also increase (Figure 4.2). Note, this modelling only considered a 20 year time horizon (i.e. until 2043) – for longer time horizons higher tide elevations and larger flow rates would be expected.

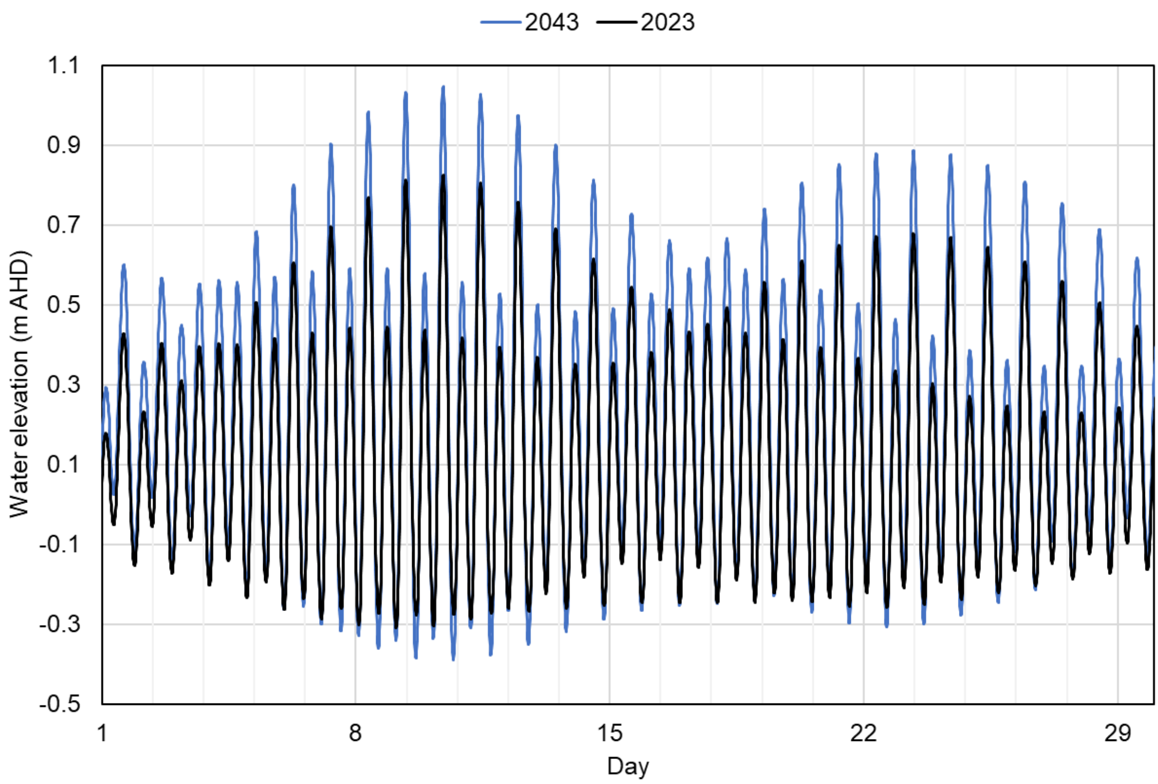

Figure 4.1 Modelled present and future water levels adjacent to the Windang foreshore

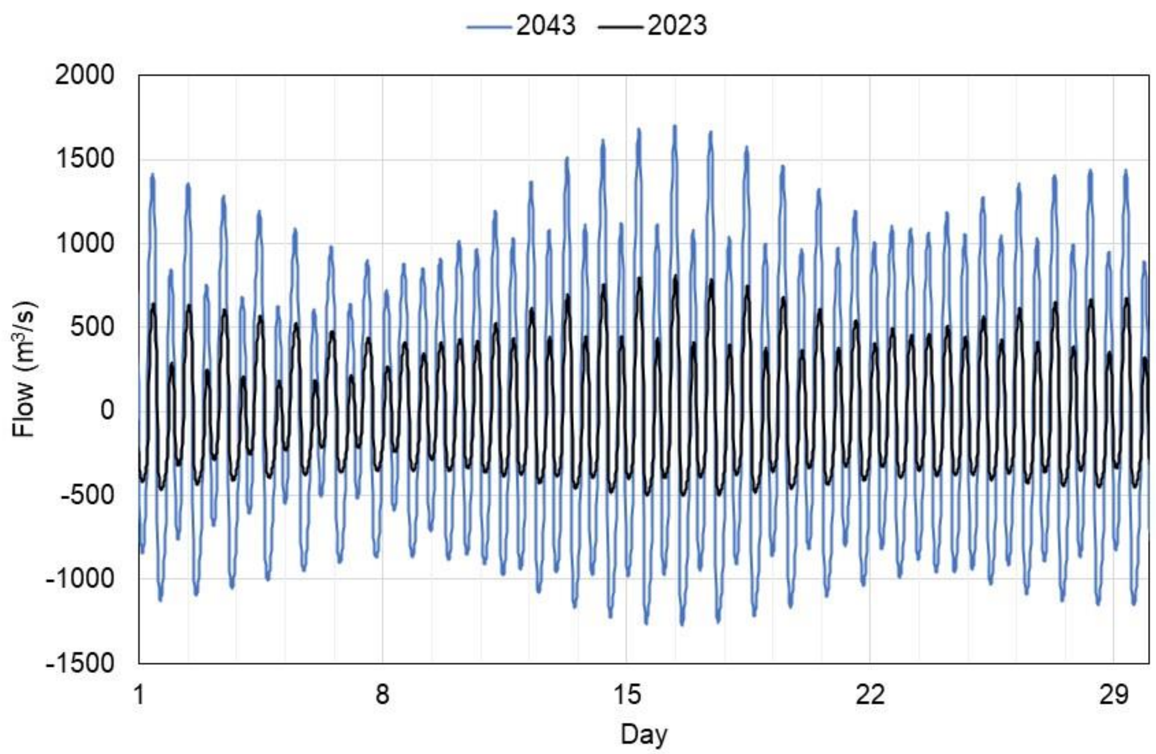

Figure 4.2 Modelled present and future flows through the Lake Illawarra entrance channel adjacent to the Windang foreshore

4.3 Present day local scale hydraulics

4.3.1 Local scale velocities (present day)

Maximum present day depth-averaged velocities within the Windang foreshore study area during larger spring tide conditions are shown in Figure 4.3 and Figure 4.4 for flood (incoming) and ebb (outgoing) tides, respectively.

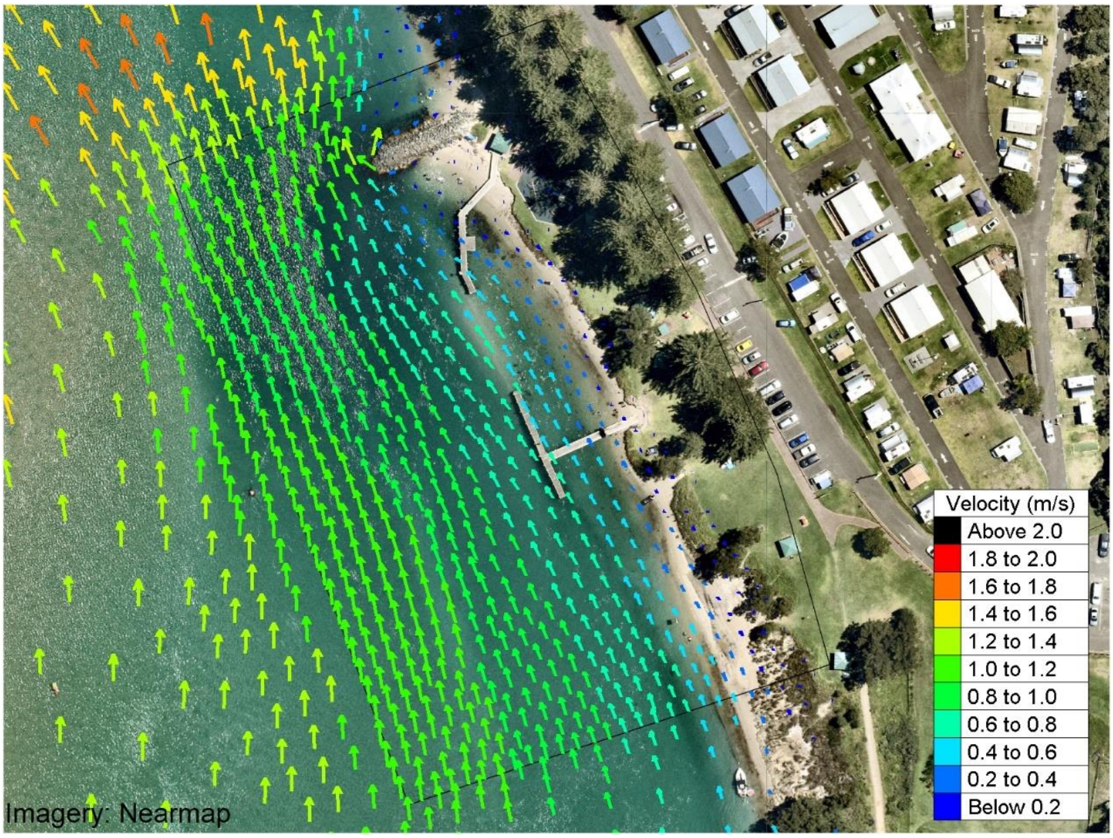

Figure 4.3 Maximum local velocities during a flood (incoming) tide (present day) (Imagery: Nearmap, December 2021)

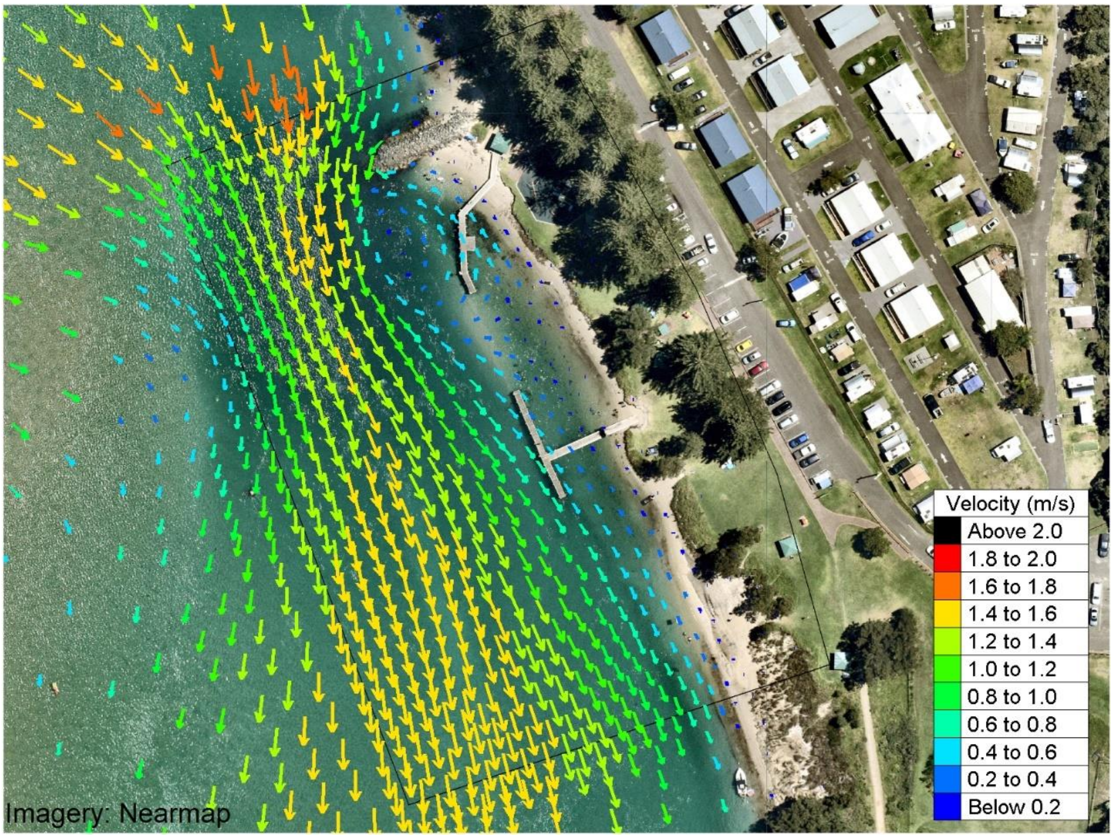

Figure 4.4 Maximum local velocities during an ebb (outgoing) tide (present day) (Imagery: Nearmap, December 2021)

4.3.2 Local scale bed shear (present day)

Maximum present day bed shear values within the Windang foreshore study area during larger spring tide conditions are shown in Figure 4.5 and Figure 4.6 for flood (incoming) and ebb (outgoing) tides, respectively. For details on the calculation of bed shear see Appendix A. Note, based upon Berenbrock and Tranmer (2008) and measured sediment sizes, the critical bed shear for medium sand at this location is ~0.2 N/m^2. Where the bed shear is above this value this indicates a higher potential rate of sediment transport.

It is worth noting that while bed shear on the banks may be stable (i.e. less than 0.2 N/m^2), erosion within the channel is high and likely to scour creating a deep channel. As erosion occurs within the channel, the bank slope will steepen reducing its stability (a stable slope will be less than 1V:4H – see Section 3.3.3). Subsequently, the Windang foreshore is still susceptible to bank erosion despite lower bed shear.

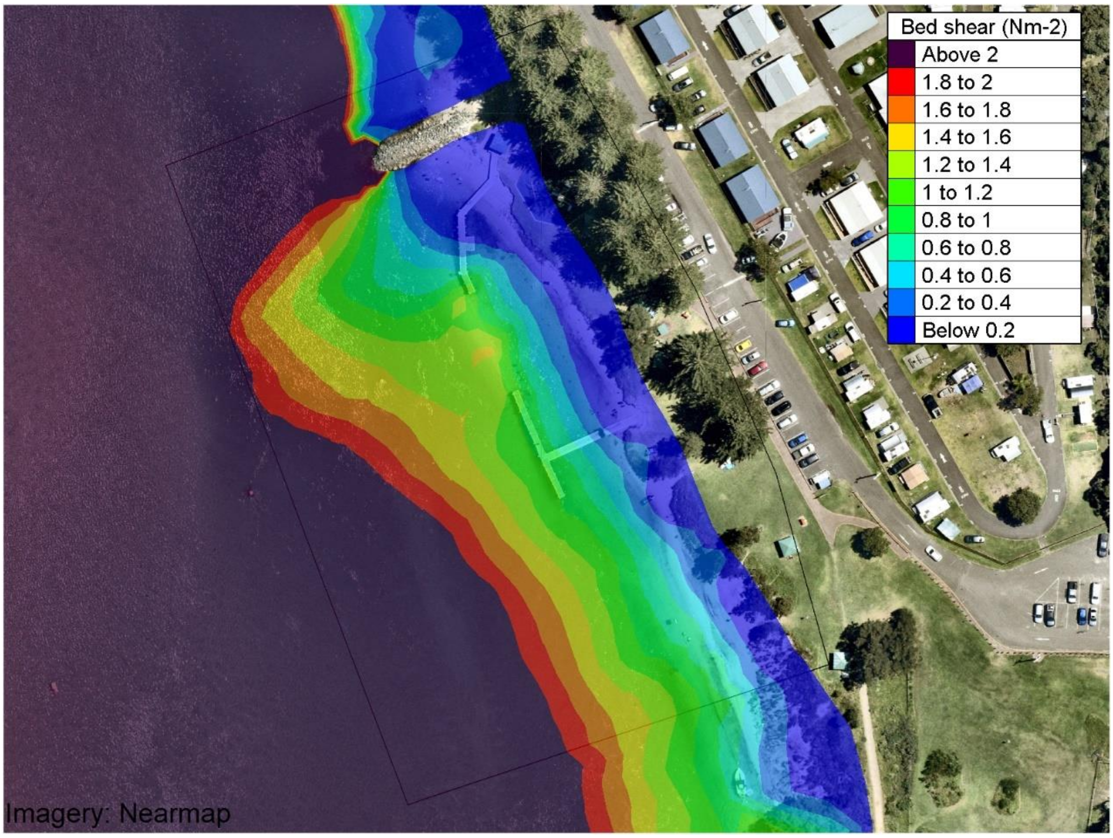

Figure 4.5 Maximum local bed shear during a flood (incoming) tide (present day) (Imagery: Nearmap, December 2021)

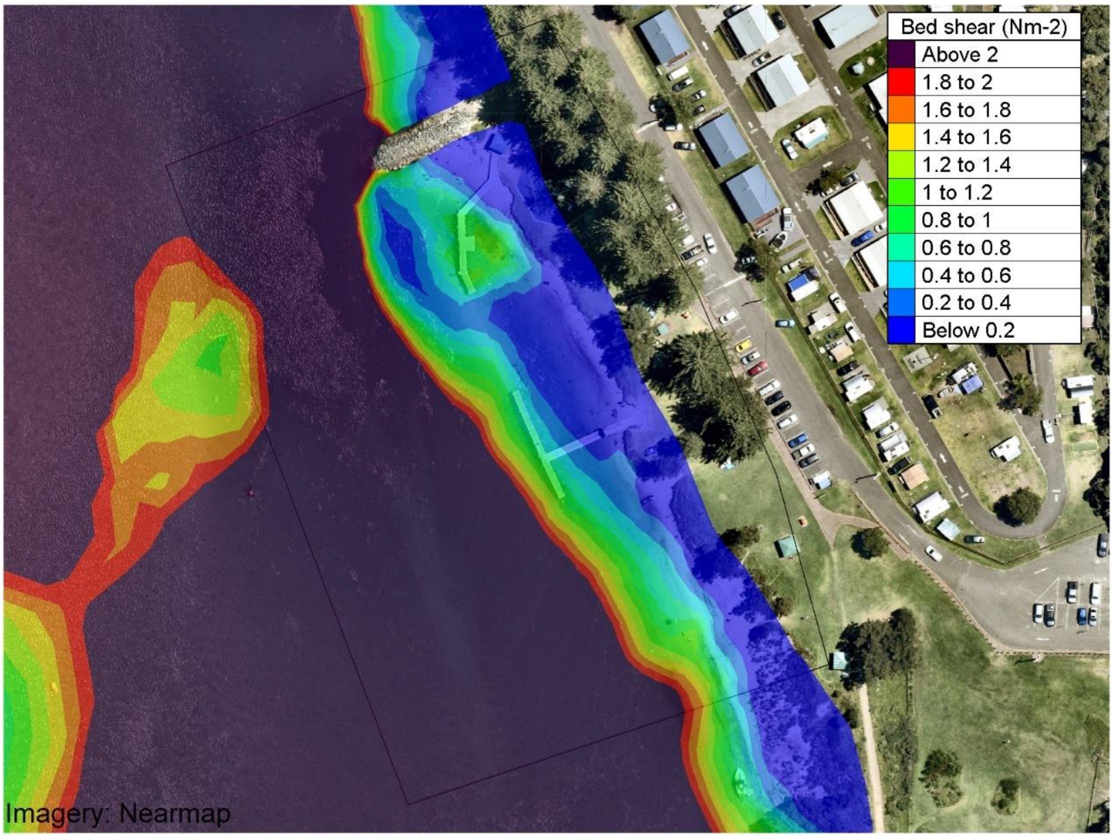

Figure 4.6 Maximum local bed shear during an ebb (outgoing) tide (present day) (Imagery: Nearmap, December 2021)

4.4 Future 2043 local scale hydraulics

4.4.1 Local scale velocities (2043)

Maximum future depth-averaged velocities within the Windang foreshore study area during larger spring tide conditions are shown in Figure 4.7 and Figure 4.8 for flood (incoming) and ebb (outgoing) tides, respectively.

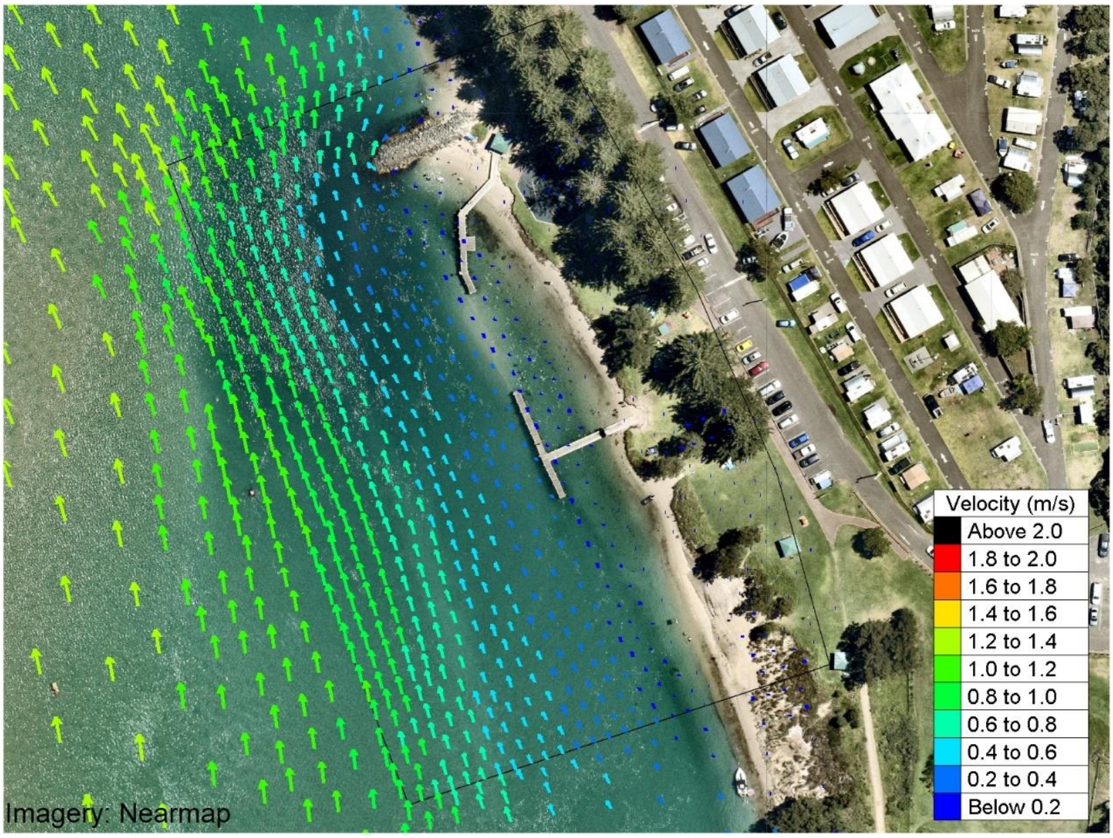

Figure 4.7 Maximum local velocities during a flood (incoming) tide (2043) (Imagery: Nearmap, December 2021)

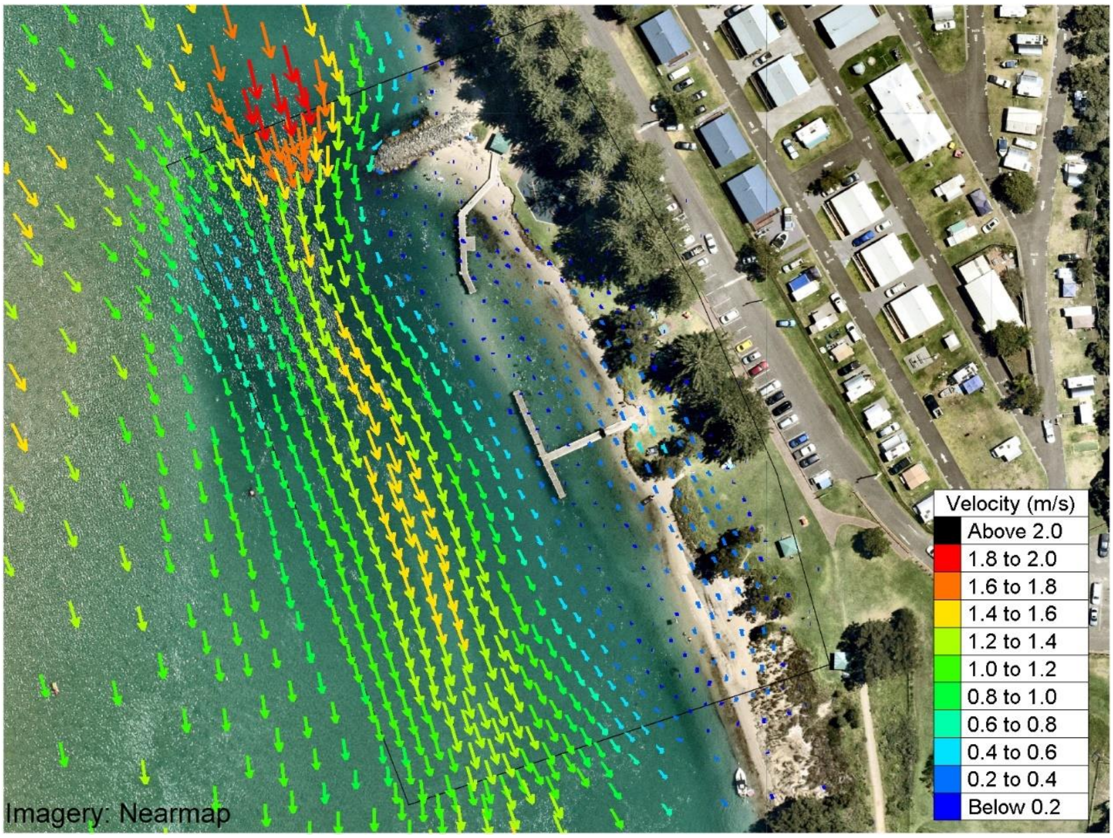

Figure 4.8 Maximum local velocities during an ebb (outgoing) tide (2043) (Imagery: Nearmap, December 2021)

4.4.2 Local scale bed shear (2043)

Maximum future bed shear values within the Windang foreshore study area during larger spring tide conditions are shown in Figure 4.9 and Figure 4.10 for flood (incoming) and ebb (outgoing) tides, respectively. For details on the calculation of bed shear see Appendix A. Note, based upon Berenbrock and Tranmer (2008) and measured sediment sizes, the critical bed shear for medium sand at this location is ~0.2 N/m^2. Where the bed shear is above this value this indicates a higher potential rate of sediment transport.

It is worth noting that while bed shear on the banks may be stable (i.e. less than 0.2 N/m^2), erosion within the channel is high and likely to scour creating a deep channel. As erosion occurs within the channel, the bank slope will steepen reducing its stability (a stable slope will be less than 1V:4H – see Section 3.3.3). Subsequently, the Windang foreshore is still susceptible to bank erosion despite lower bed shear.

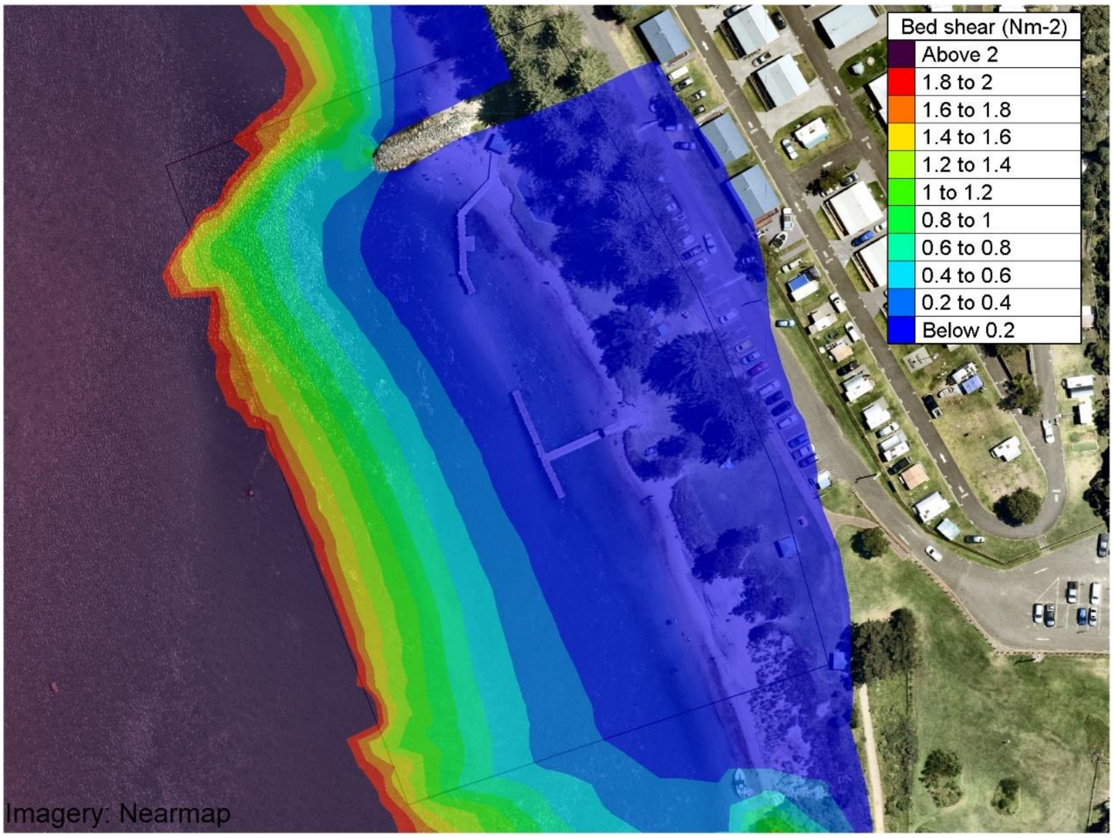

Figure 4.9 Maximum local bed shear during a flood (incoming) tide (2043) (Imagery: Nearmap, December 2021)

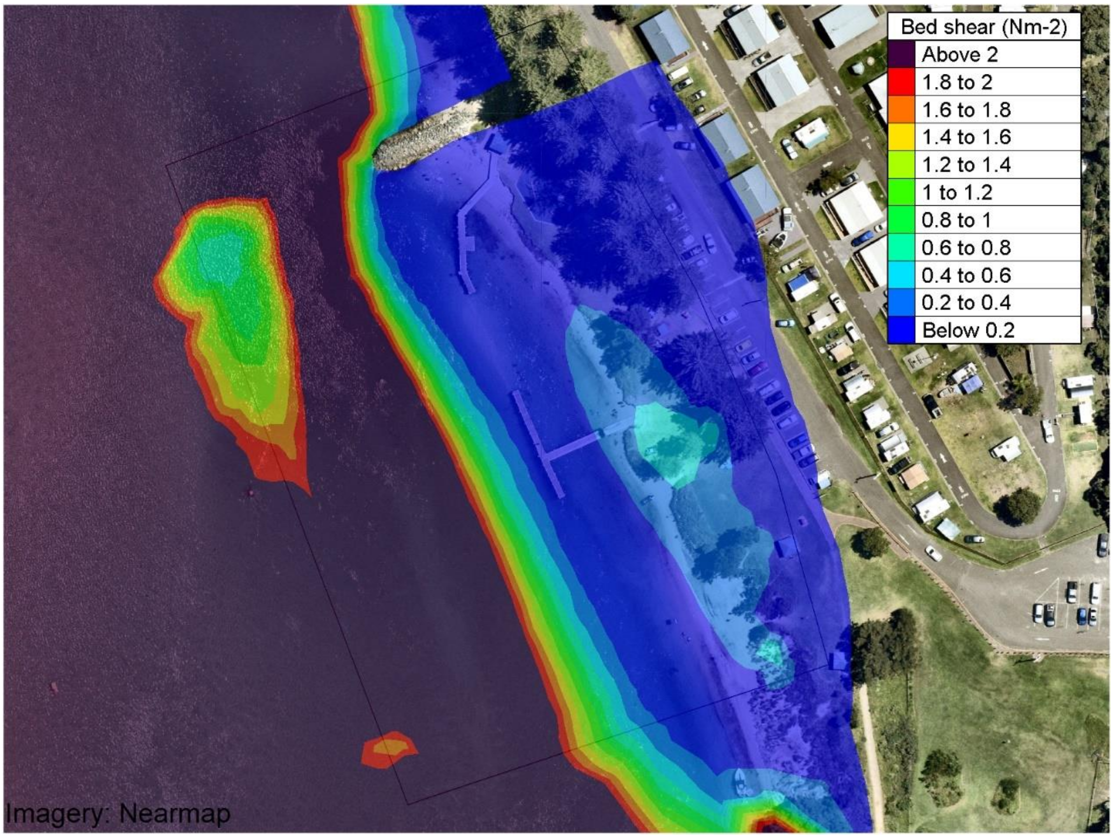

Figure 4.10 Maximum local bed shear during an ebb (outgoing) tide (2043) (Imagery: Nearmap, December 2021)

4.5 Assessment of local scale hydraulics

Review of numerical model results found that during larger spring tides the maximum velocities recorded during present day conditions occurred during outgoing (ebb) tides with a maximum depth averaged velocity of 1.6 m/s. This is compared to the maximum depth averaged velocities recorded during an incoming (flood) tide of 1.4 m/s. During incoming (flood) tides the velocities across the study area were uniform in comparison to the outgoing (ebb) tide where a channel of high-velocity flow formed off the end of the groyne and caused eddies to form on the leeward side of the groyne.

In a plausible future state (2043) if no mitigation measures are implemented within the Lake Illawarra entrance channel, the model predicted that the maximum depth averaged velocity during outgoing (ebb) tides would increase to approximately 1.8 m/s. During incoming (flood) tides, the maximum depth averaged velocities within the study area would be expected to decrease to 1.2 m/s. Velocity patterns observed during present day conditions were found to continue in the future with relatively uniform velocities during incoming (flood) tides, while concentrated velocities and eddies occurred during outgoing (ebb) tides.

The reason for increased velocities at the study area during ebb tides has to do with the geometry of the broader Lake Illawarra entrance channel. During an ebb tide, the peak flows occur when the tide elevation is low and there are emergent sections of sand within the Lake Illawarra entrance channel. Comparatively, during a flood tide the peak flows occur when the tide elevation is high. This means that the channel cross-sectional area for peak ebb flows is much smaller than the channel cross-sectional area for peak flood flows. Since velocity is a function of flow and area (flow = area x velocity), this results in the velocities during an ebb tide being larger than those in a flood tide.

Significant levels of bed shear were observed in present day and plausible future model results indicating that erosion along the Windang foreshore is likely to continue into the future (bed shear was recorded above 0.2 N/m^2 throughout the study area). As with velocity observations, the bed shear pattern is likely to change between outgoing (ebb) and incoming (flood) tides. Note, this model did not consider any sediment transport which would likely occur over a tidal cycle as high levels of bed shear occur. Unless the Windang foreshore is directly protected, or a large-scale mitigation measure is implemented elsewhere in the Lake Illawarra entrance channel, it is likely to continue to erode for the foreseeable future.

5 Foreshore stabilisation design options

5.1 Preamble

WRL has prepared the following eight potential options (to a concept design level) to prevent further bank erosion and maintain the present position of the bank along the Windang foreshore:

- Option 1: Steel sheet pile revetment (Figure 5.3)
- Option 2: Concrete secant pile revetment (Figure 5.4)
- Option 3: Excavated rock revetment (Figure 5.6)
- Option 4: Minimal excavation rock revetment (Figure 5.7)
- Option 5: Minimal excavation rock revetment with narrow toe width (Figure 5.8)
- Option 6: Excavated geotextile container revetment (Figure 5.10)
- Option 7: Minimal excavation geotextile container revetment (Figure 5.11)
- Option 8: Minimal excavation geotextile container revetment with narrow toe width (Figure 5.12)

Each of these figures have been prepared at the same scale to facilitate comparison of the difference in size of each of the options. The representative concept design cross-sections have been prepared in sufficient detail to allow relative comparison between each of the eight options.

While WRL has prepared concept designs for vertical revetments, rock revetments and geotextile container revetements, alternative materials for foreshore stabilisation works could still be considered during detailed design. While they are likely to have higher capital costs than the rock and geotextile container options, armour suited for environments with larger wave climate could be used. These include (but are not limited to):

- Articulated concrete block mattresses
- Concrete armour units (e.g. Hanbars)
- Rock bags (which are proposed for a similar environment in Swansea Channel; Salients, 2022)
- Concrete caissons

WRL also considered the possibility of using concrete filled scour mattresses to stabilise the Windang foreshore but dismissed them as being unsuitable due to their inability to adapt to highly erosive conditions at the toe.

A 20 year design life (2023 to 2043) has been adopted for the concept designs to protect the bank for an alongshore extent of 150 m downstream of the groyne, assuming that no other mitigation measures have been implemented in the Lake Illawarra entrance channel over this planning period.

All design development has been undertaken for a single profile: cross-section F4 (Figure 3.11 and Figure 3.15). If one of the options is selected to progress to detailed design; the adopted geometry would vary across the other seven cross-sections.

General design conditions, indicative capital cost assumptions and details specific to each of the options are outlined in Appendix B. Example photographs of each structure type are shown in the following sub-sections along with the associated concept design cross-sections figures.

5.2 Vertical revetment options

A vertical revetment comprising either steel sheet piles (Option 1, Figure 5.3) or concrete secant piles (Option 2, Figure 5.4) was developed to stabilise the Windang foreshore. Example photographs of both types of structures are shown in Figure 5.1 and Figure 5.2, respectively.

Figure 5.1 Example vertical steel sheet pile revetment

Figure 5.2 Example vertical concrete secant pile revetment (Kingscliff, NSW)

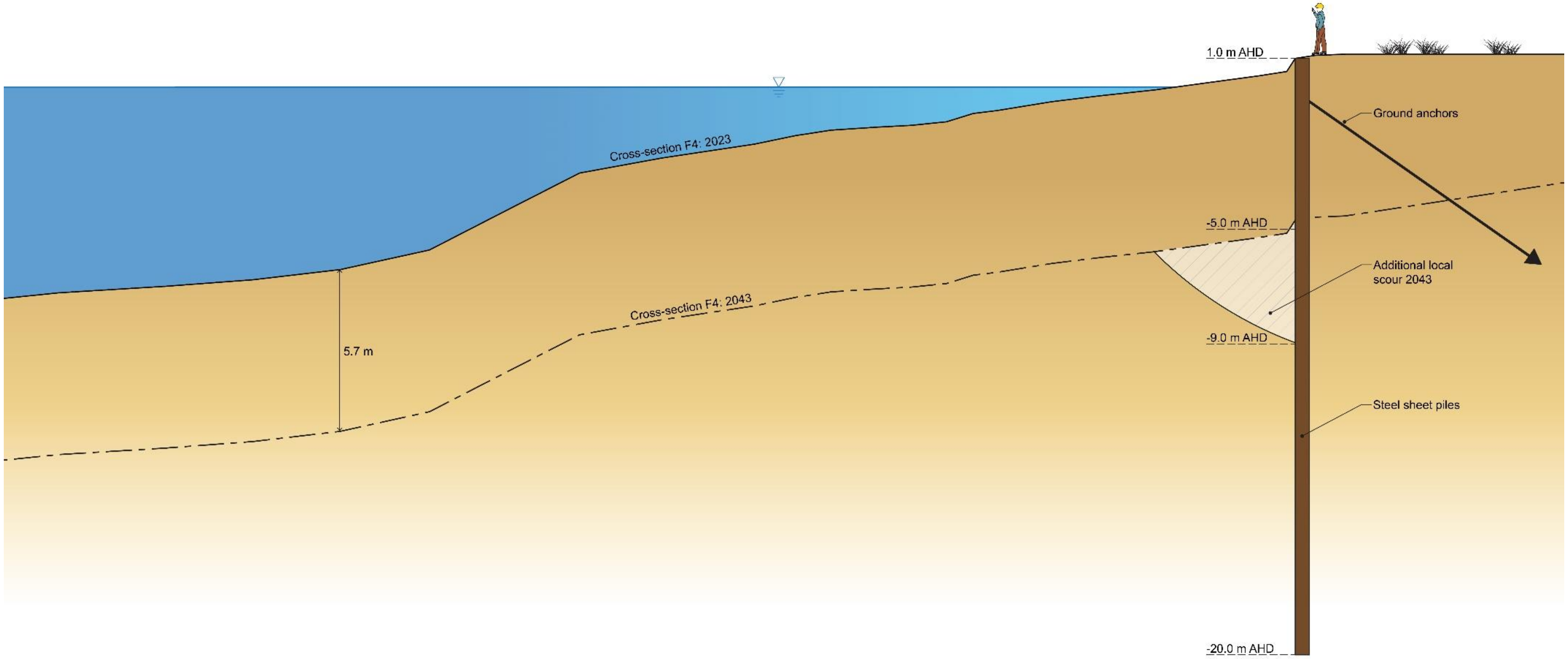

Figure 5.3 Option 1 – Steel sheet pile revetment

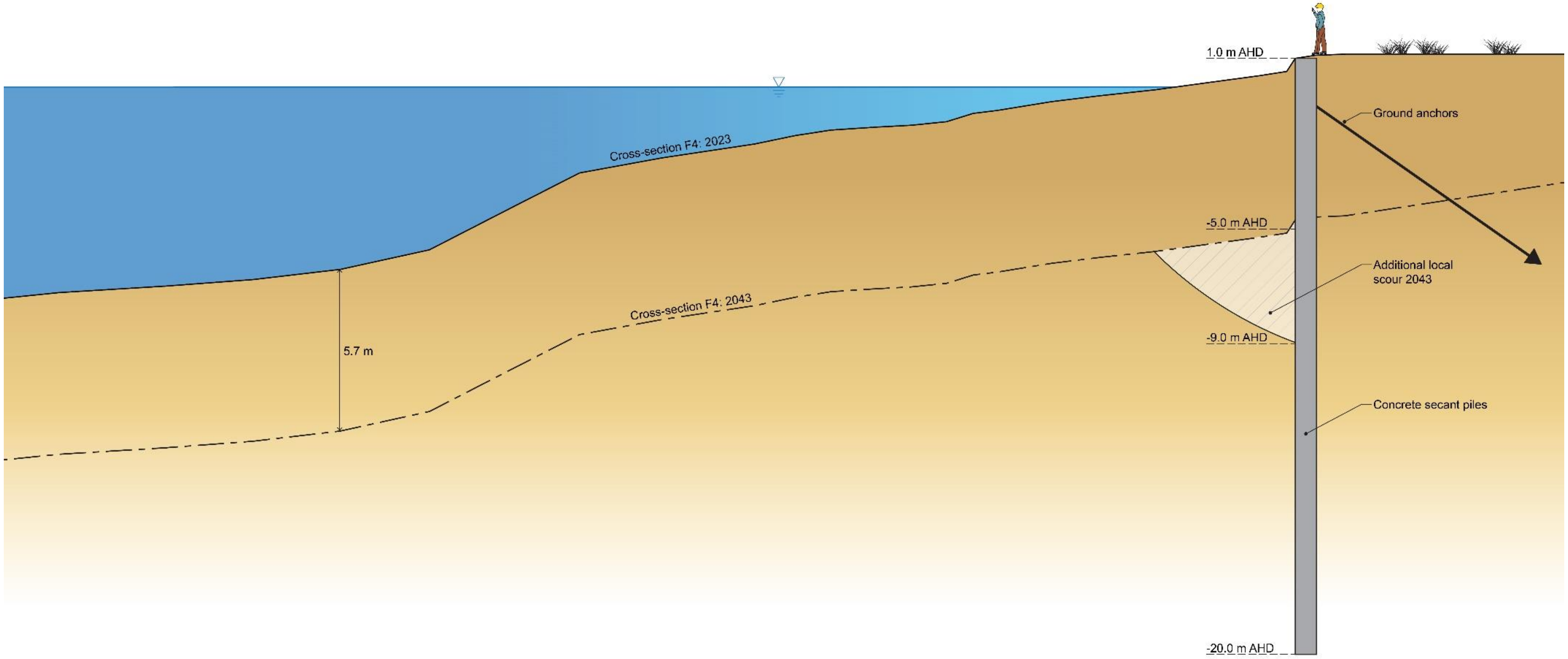

Figure 5.4 Option 2 – Concrete secant pile revetment

5.3 Rock revetment options

Three different rock revetment concept designs, Option 3 (Figure 5.6), Option 4 (Figure 5.7) and Option 5 (Figure 5.8), were developed to stabilise the Windang foreshore. An example photograph of this type of structure is shown in Figure 5.5, albeit with a relatively small toe.

Figure 5.5 Example rock revetment (Ermington, NSW)

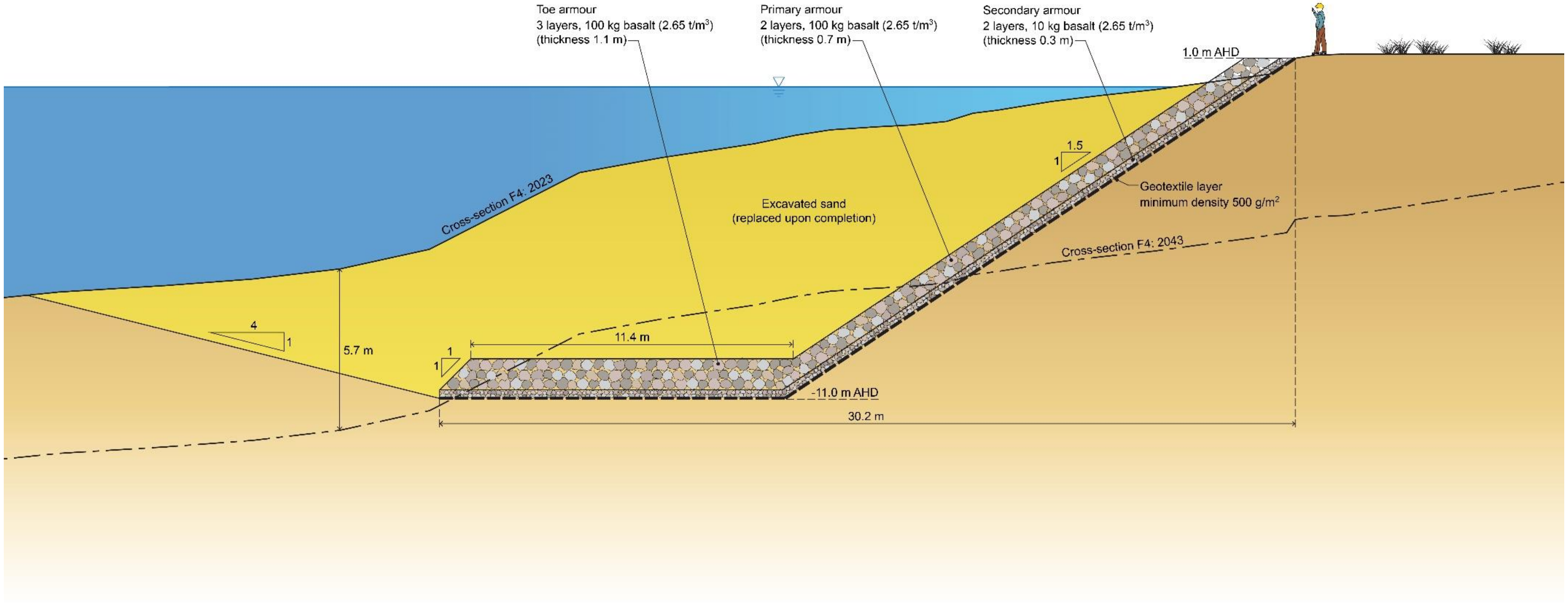

Figure 5.6 Option 3 – Excavated rock revetment

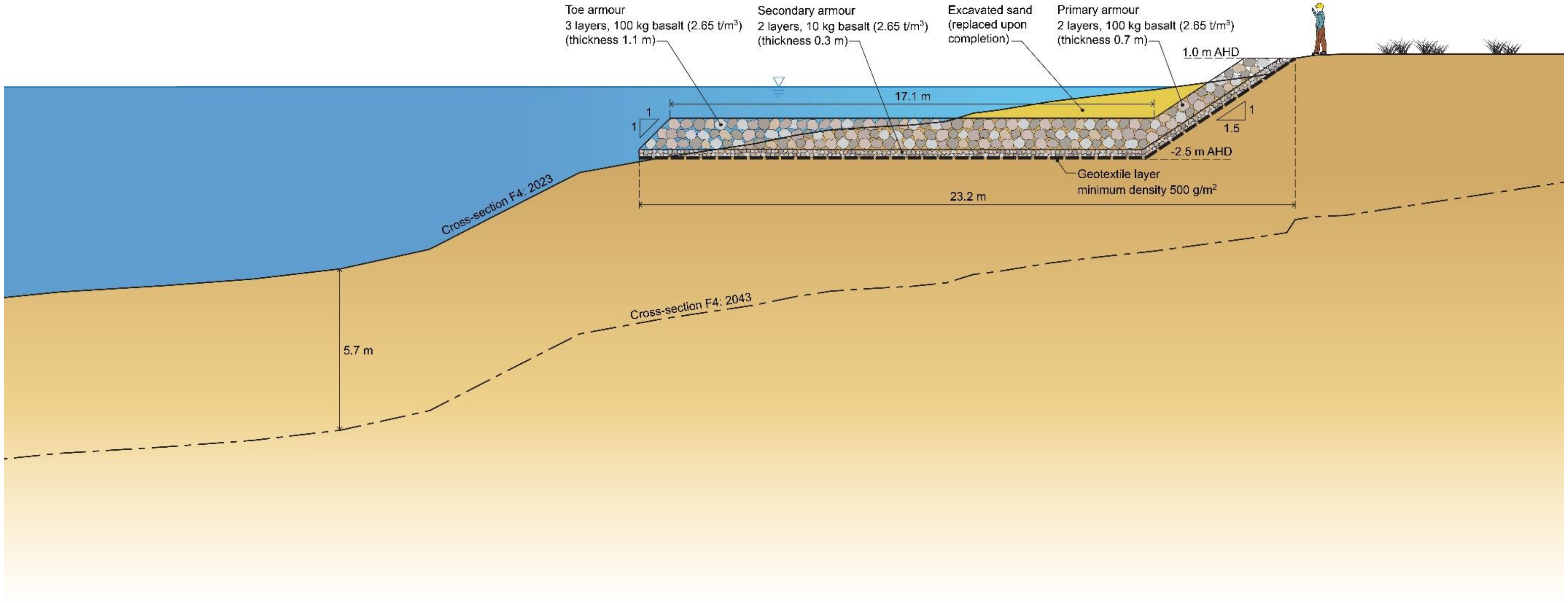

Figure 5.7 Option 4 – Minimal excavation rock revetment

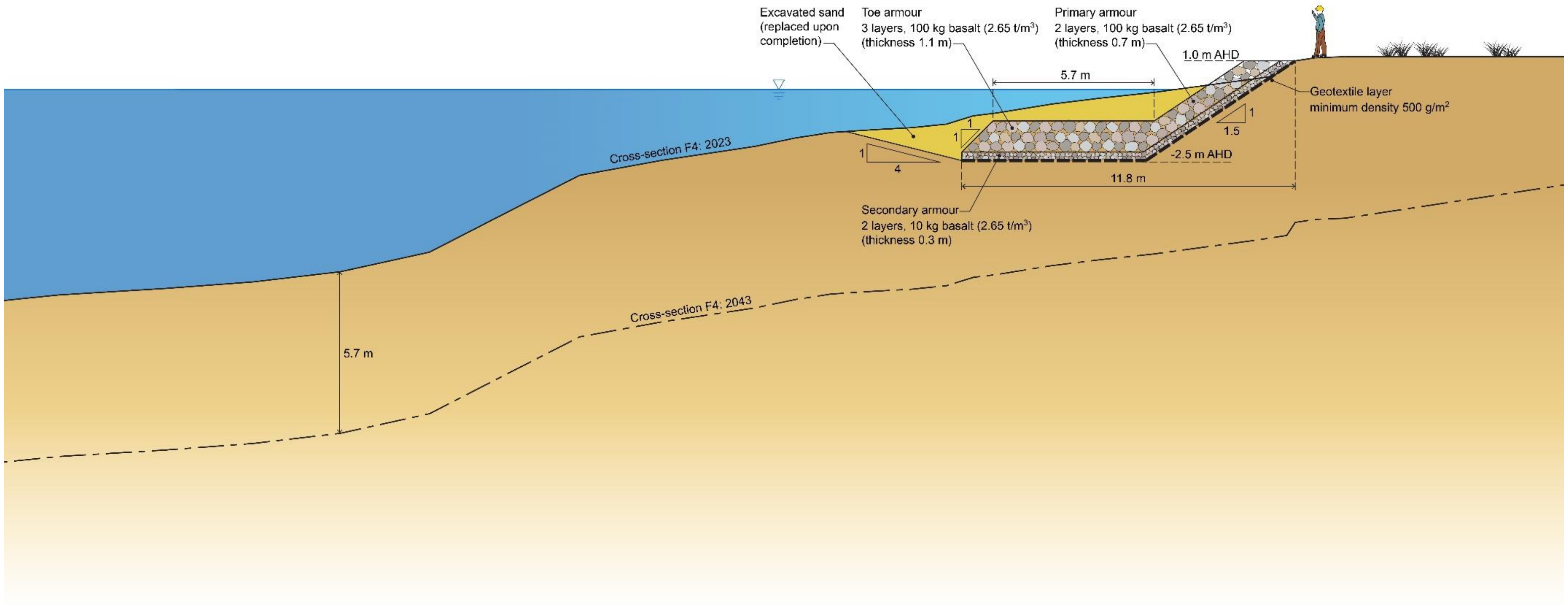

Figure 5.8 Option 5 – Minimal excavation rock revetment with narrow toe width
(Note that this option will require constant monitoring and progressive upgrading to the toe width of Option 4 over the 20 year planning period)

5.4 Geotextile container revetment options

Three different geotextile container revetment concept designs, Option 6 (Figure 5.10), Option 7 (Figure 5.11) and Option 8 (Figure 5.12), were developed to stabilise the Windang foreshore which mimic the corresponding three rock revetment concept designs (Options 3, 4 and 5). An example photograph of this type of structure is shown in Figure 5.9, again with a relatively small toe.

Figure 5.9 Example geotextile container revetment (0.75 m^3 containers at Bonnie Vale, NSW)

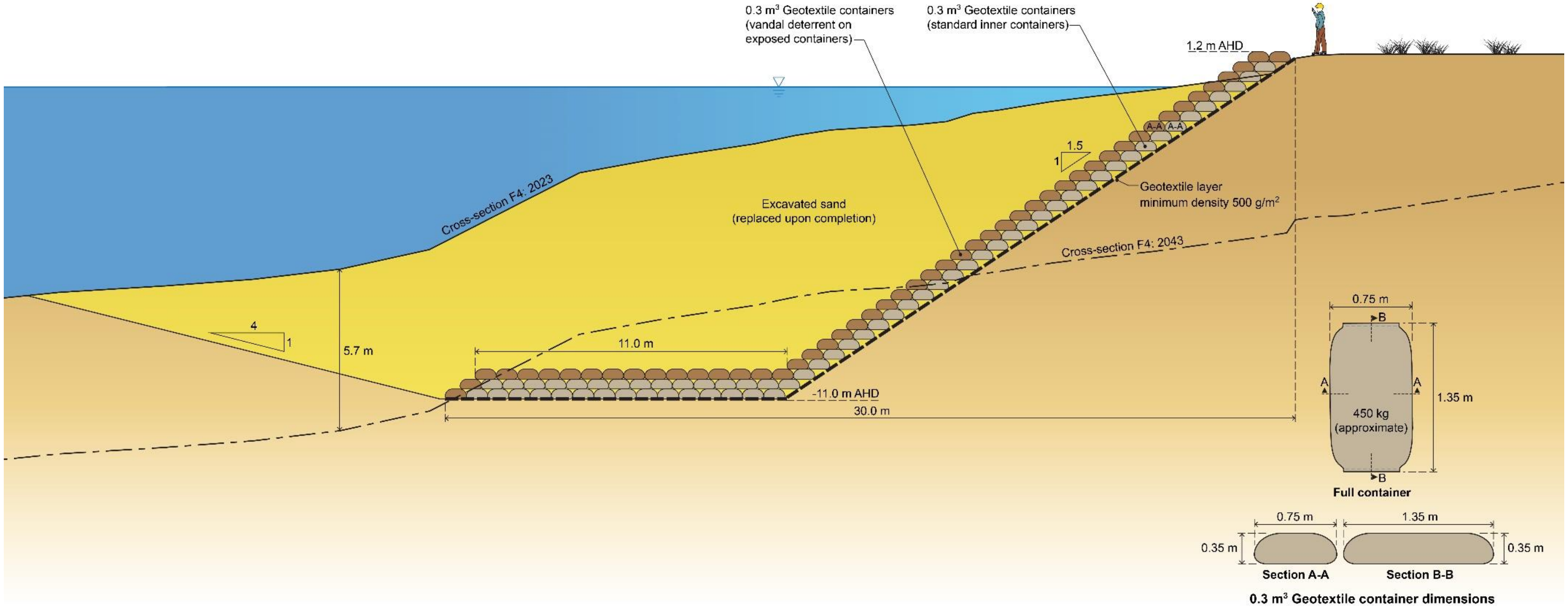

Figure 5.10 Option 6 – Excavated geotextile container revetment

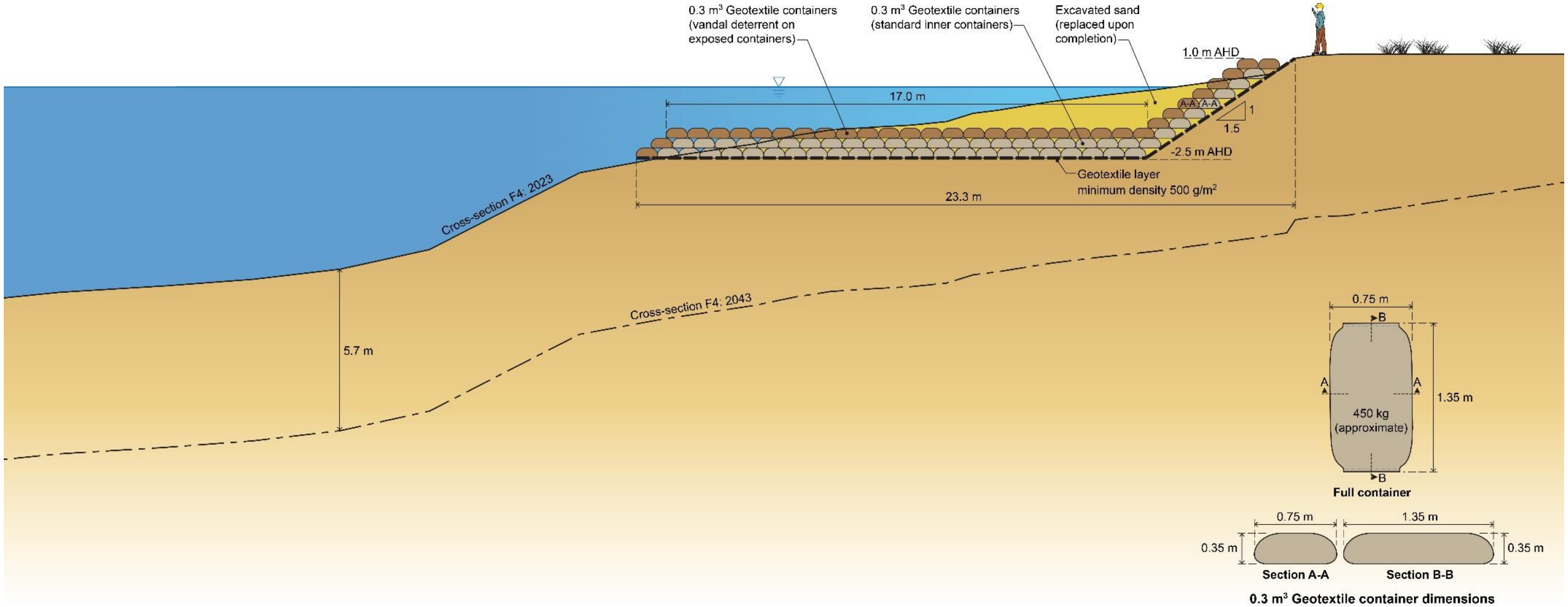

Figure 5.11 Option 7 – Minimal excavation geotextile container revetment

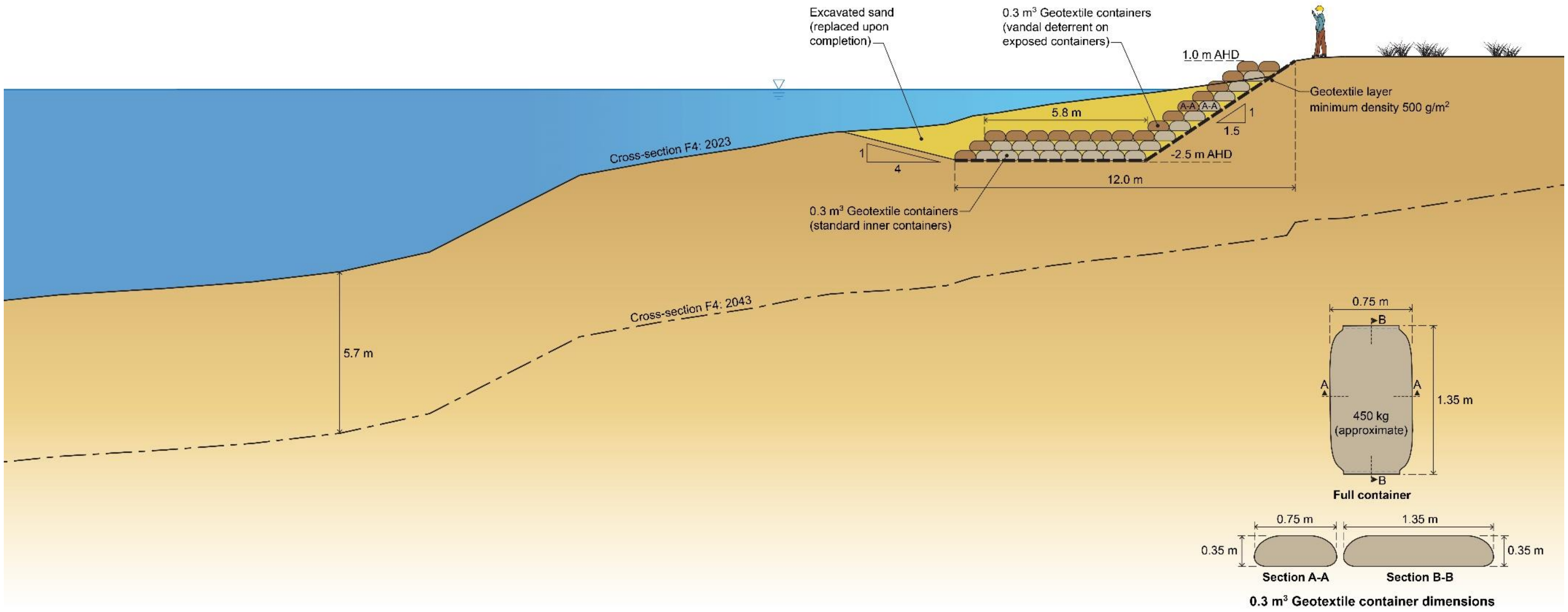

Figure 5.12 Option 8 – Minimal excavation geotextile container revetment with narrow toe width
(Note that this option will require constant monitoring and progressive upgrading to the toe width of Option 7 over the 20 year planning period)

5.5 Comparison of options

5.5.1 Indicative capital cost summary

The estimated capital costs to construct each of the eight foreshore stabilisation options are summarised in Table 5.1.

Table 5.1 Summary of indicative capital cost estimates for foreshore stabilisation options

Option	Indicative capital cost estimate	
	Total	**Lineal**
1: steel sheet pile revetment	$5.2M	$34,500/m
2: concrete secant pile revetment	$6.8M	$45,500/m
3: excavated rock revetment	$2.0M	$13,300/m
4: minimal excavation rock revetment	$1.2M	$8,100/m
5: minimal excavation rock revetment with narrow toe width	$0.7M	$4,500/m
6: excavated geotextile container revetment	$2.4M	$16,200/m
7: minimal excavation geotextile container revetment	$1.5M	$9,900/m
8: minimal excavation geotextile container revetment with narrow toe width	$0.8M	$5,500/m

5.5.2 Vertical revetment options

The main advantages of the vertical revetment options are:

- Small footprint
- Low maintenance and high durability
- At the eastern end, easier to terminate structure landward of the present bank position

The main disadvantages of the vertical revetment options are:

- High capital cost
- Need to be driven deep into the ground which requires specialised machinery
- The structure is rigid which can lead to catastrophic failure
- Difficult to interface with the upstream groyne
- Cannot be topped up, repaired or easily extended in the same manner as rock and geotextile container options to increase its design life
- Low absorption of wave energy leading to wave reflection and potential wave overtopping
- Limited provision of ecological habitat

While a steel sheet pile revetment (Option 1) has a slightly lower capital cost than a concrete secant pile revetment (Option 2), its disadvantage is that the steel will corrode over time.

5.5.3 Rock revetment options

The main advantages of the rock revetment options are:

- Lowest capital cost compared to piles and geotextile containers
- Provision of ecological habitat
- The structure is relatively flexible so damage/failure tends to be progressive rather than catastrophic
- Can be topped up, repaired or easily extended to increase its design life
- Easy to interface with the upstream groyne
- Potentially more natural looking than other options
- Have been used for centuries, so better understood than newer options
- Absorbs a large proportion of wave energy
- May be more economically viable if low cost rock can be sourced from nearby construction projects.

The main disadvantages of the rock revetment options are:

- Large footprint
- Fracturing rock may wash onto neighbouring beach and foreshore
- The uneven rock crest may require signage due to its potential for public trip/fall
- Rodents and reptiles (snakes) can live in the voids
- At the eastern end, more difficult to terminate structure landward of the present bank position

Within the three rock revetment options:

- The excavated rock revetment (Option 3):
    - Has the highest capital cost
    - Has the largest footprint
    - Requires significant excavation
    - Is expected to have the least monitoring and maintenance requirements
- The minimal excavation rock revetment (Option 4):
    - Has a moderate capital cost
    - Has a relatively large footprint
    - Requires less excavation
    - Is expected to have moderate monitoring and maintenance requirements
    - Is expected to require precautions to prevent boats running aground on the revetment
- The minimal excavation rock revetment with narrow toe width (Option 5):
    - Has the lowest capital cost
    - Has a relatively small footprint
    - Requires less excavation
    - Offers flexibility if a large-scale erosion mitigation measure is implemented prior to 2043
    - Is not anticipated to provide foreshore stabilisation for the next 20 years without upgrade
    - Is expected to have high monitoring and maintenance requirements
    - Is expected to require future addition of rock armour to widen the toe as per Option 4

5.5.4 Geotextile container options

The main advantages of the geotextile container revetment options are:

- Similar (but slightly higher) capital cost compared to rock
- The structure is relatively flexible so damage/failure tends to be progressive rather than catastrophic
- Can be topped up, repaired or easily extended to increase its design life
- Moderately easy to interface with the upstream groyne
- Minimal voids for reptiles and rodents
- Provides recreation space and water access
- Soft and able to be removed easily

The main disadvantages of the geotextile container revetment options are:

- Large footprint
- Degradation of geotextile due to UV
- Potential for damage due to debris and vandalism
- Expected performance is not as well known compared to rock
- At the eastern end, more difficult to terminate structure landward of the present bank position

The differences between the three geotextile container revetment concept designs (Options 6, 7 and 8) are similar to those discussed for the three rock revetment options (Section 5.5.3).

6 Afterword

As a result of the permanent opening of the Lake Illawarra entrance channel in 2007, erosion of the bed caused the progressive collapse of the jetty structure adjacent to the Windang foreshore. This jetty structure provided public amenity which is now unavailable. While assessing the feasibility of a replacement jetty was outside the scope of this investigation, analysis from this study can inform the future management of the Windang foreshore and guide the future decision as to whether the jetty will be replaced.

This investigation focused on erosion and local scale hydraulics occurring at Windang presently (2023) and those likely to occur in the future (20 years from now) if a large-scale erosion mitigation measure (designed to reduce tidal conveyance) is not implemented elsewhere in the Lake Illawarra entrance channel. Based on this analysis, the following conclusions can be drawn:

- High velocities (>1 m/s) will continue to occur along the Windang foreshore for the foreseeable future with or without a bank stabilisation structure.
- While a bank stabilisation structure will prevent further landward erosion, significant deepening of the entrance channel immediately adjacent to the Windang foreshore will continue to occur with or without a bank stabilisation structure.

On this basis, WRL recommends that the following points be considered during the planning for a new jetty in the area:

- The costs associated with a new jetty structure will be significantly influenced by the future erosion/scour expected to occur around the jetty piles over its design life (e.g. design scour level).
- High water velocities at a new jetty structure at the same location as the previous one may pose a risk to safety for associated in-water recreation, particularly for some unpowered activities (e.g. swimming, canoeing/kayaking, stand-up paddle boarding, etc.).
- An alternative location within the Lake Illawarra entrance with a lower long-term erosion rate and lower water velocities may be more suitable for a jetty structure.

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Appendix A Numerical modelling

A1 Background

Local scale hydraulics including velocity and bed shear have been assessed using a numerical model of Lake Illawarra which simulates the hydrodynamic conditions within the entrance channel including at the Windang foreshore. Numerical modelling was completed using the RMA modelling package. An RMA-2 (King, 2019), two-dimensional, depth-averaged, finite element hydrodynamic model was originally developed for Lake Illawarra by WRL in 2019 to assess the impact of rock protection works beneath the northern end of Windang Bridge. The Lake Illawarra model domain (Figure A-1) covers the entire lake to the open sea boundary. The RMA-2 model computes finite element solutions for water surface elevations and horizontal velocity components. This model was verified against tidal flow and water level data captured in September and October 2018. WRL (2019) provides further details about the model setup and development. The configuration of the numerical model has been updated (as described in the following sections) to provide increased detail and inform the local scale hydraulics along the Windang foreshore.

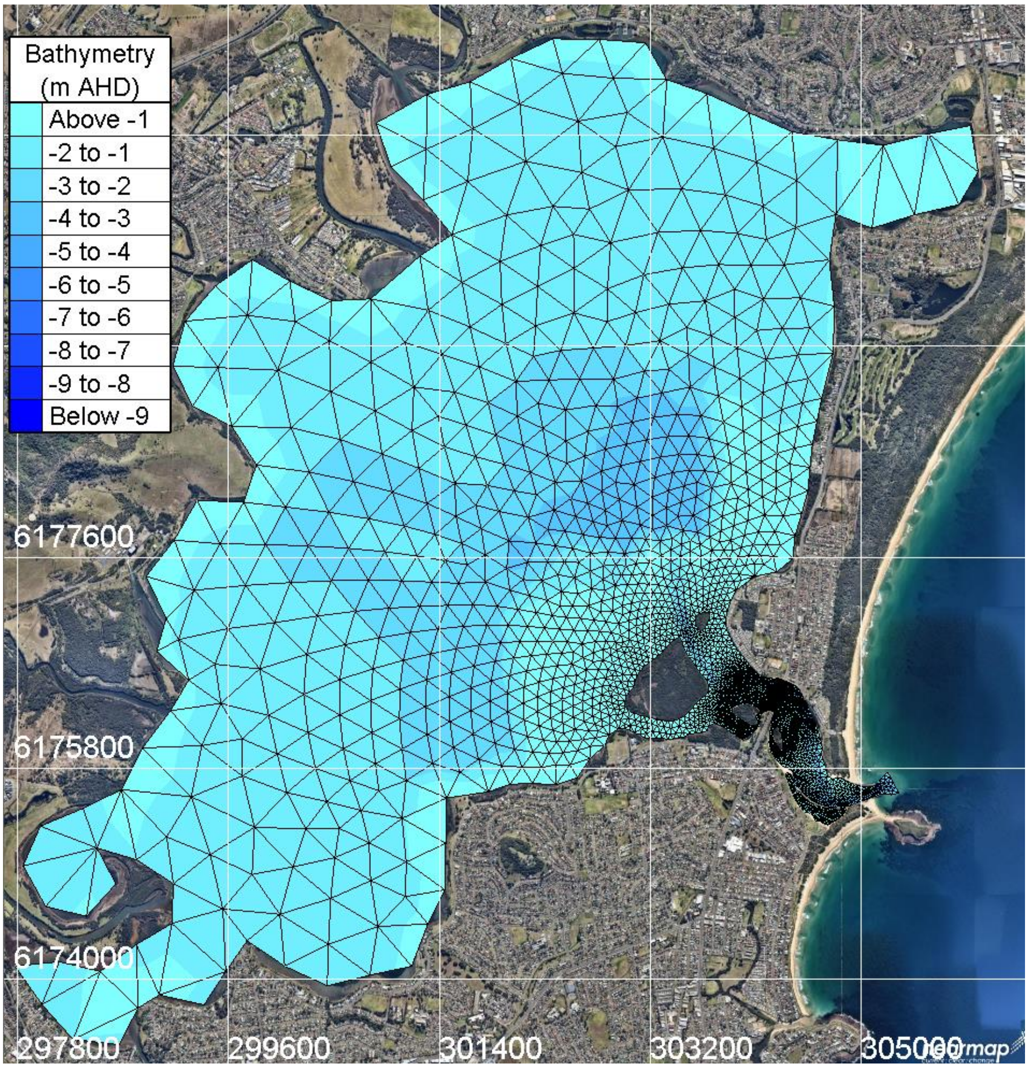

Figure A-1 RMA-2 numerical hydrodynamic model domain (Source: WRL, 2019)

A2 Boundary conditions

A2.1 Tidal boundary conditions

A synthetic downstream tidal boundary was generated for the model from the tidal constituents. The tidal constituents were calculated from actual tide measurements collected at Port Kembla (Bureau of Metrology [BoM] station number 068253) from 1 January 2019 to 21 August 2019 (BoM, 2019).

To account for expected sea level rise that would occur between 2023 and 2043, 0.1 m was added to the tidal boundary conditions for future model scenarios. This was based upon sea level rise projections included in the Sixth Assessment Report (AR6; Garner et al., 2021) by the United Nations Intergovernmental Panel on Climate Change (IPCC) for Port Kembla (accessed via NASA, 2023) considering the medium confidence Shared Socioeconomic Pathway 5 – 8.5 (SSP5-8.5).

A2.2 Catchment inflows

There were no catchment inflows or rainfall included in the model simulations.

A3 Model bathymetry and mesh

A3.1 Present day bathymetry and model mesh

The numerical model utilised bathymetry collected by the UNSW Sydney Water Research Laboratory in December 2021 (see Tucker et al., 2023). Note, this survey did not include the rock surcharge located under Windang Bridge. Subsequently, for this area a bathymetric survey commissioned by Transport for NSW (TfNSW) was utilised (completed by North Coast surveys in July 2021). A more recent survey of the entrance channel was commissioned by TfNSW in 2022 (completed by North Coast surveys in October 2022), however, this survey did not include key areas within the entrance channel such as a shoal which has developed opposite the Windang foreshore on the southern side of the Lake Illawarra entrance channel. Subsequently, the 2021 surveys were used for the model bathymetry.

To provide additional detail at the Windang foreshore study location, the model mesh at this location was refined so that additional detail regarding velocity and depth information could be ascertained. Figure A-2 shows the changes to the model mesh between the WRL (2019) model and the one used for this assessment.

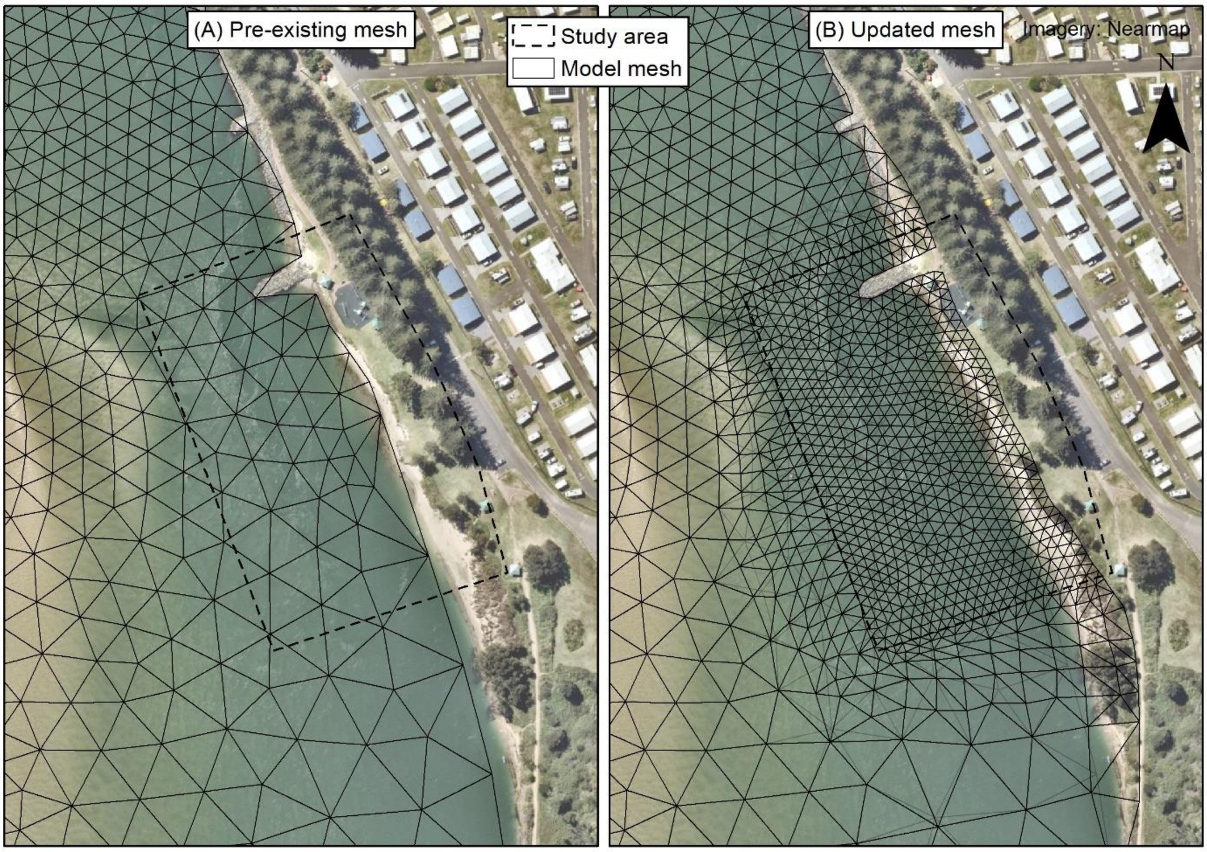

Figure A-2 Pre-existing WRL (2019) (A) and updated (B) model mesh

Note, while the pre-existing WRL (2019) numerical model was calibrated, the modified model used for this study has not been verified against tidal flow and water level data. Despite this, the accuracy of the model is considered sufficient for the purposes of defining the local scale hydraulic conditions along the Windang foreshore.

A3.2 Future bathymetry and model mesh

Bathymetric analysis of geomorphic change from 2008 to 2023 across the Windang foreshore indicated that sediment within the study area has had a maximum erosion rate of 20 m^3/m/year (see Section 3.3.2). Based upon this, indicative cross-sections were developed for the study area for a plausible future state where this erosion rate continues for a further 20 years (see Section 3.3.3). These cross-sections have been interpolated across the study area to provide a theoretical bathymetry for the future model scenario.

Previous investigations have found that while erosion along the Windang foreshore is higher than other areas in the entrance channel, erosion across the entire entrance channel still occurs (Tucker et al., 2023). To account for this, in addition to lowering the bathymetry in the study area based upon the rates identified in Section 3.3, the bathymetry for other areas within the entrance channel have also been lowered. Glatz (2023) completed an Escoffier analysis to identify the stability of the Lake Illawarra entrance. This analysis was completed for an indicative cross-section across the entrance channel which was determined to have an area of 600 m^2. Based on the analysis, the entrance was hypothesised to stabilise in 120 years when the cross-sectional area expands to 4,500 m^2. Assuming this occurs linearly, the rate at which the area increases each year can be calculated as 32.5 m^2/year. Assuming that the channel width does not change, this rate can be converted to a vertical erosion rate of 0.12 m/year, which would result in 2.3 m of erosion over a 20 year period. Subsequently, assuming that the banks along the Lake Illawarra entrance channel remain fixed (with the exception of the Windang foreshore which is assessed as previously discussed), an arbitrary 2.3 m lowering in the entrance channel elevation was applied. Note, it was assumed that the rock surcharge underneath Windang bridge remains unchanged over this 20 year period and that the groyne remains in place. A summary of the bathymetric changes applied to the Lake Illawarra entrance channel for simulations of plausible future hydraulic conditions is provided in Figure A-3.

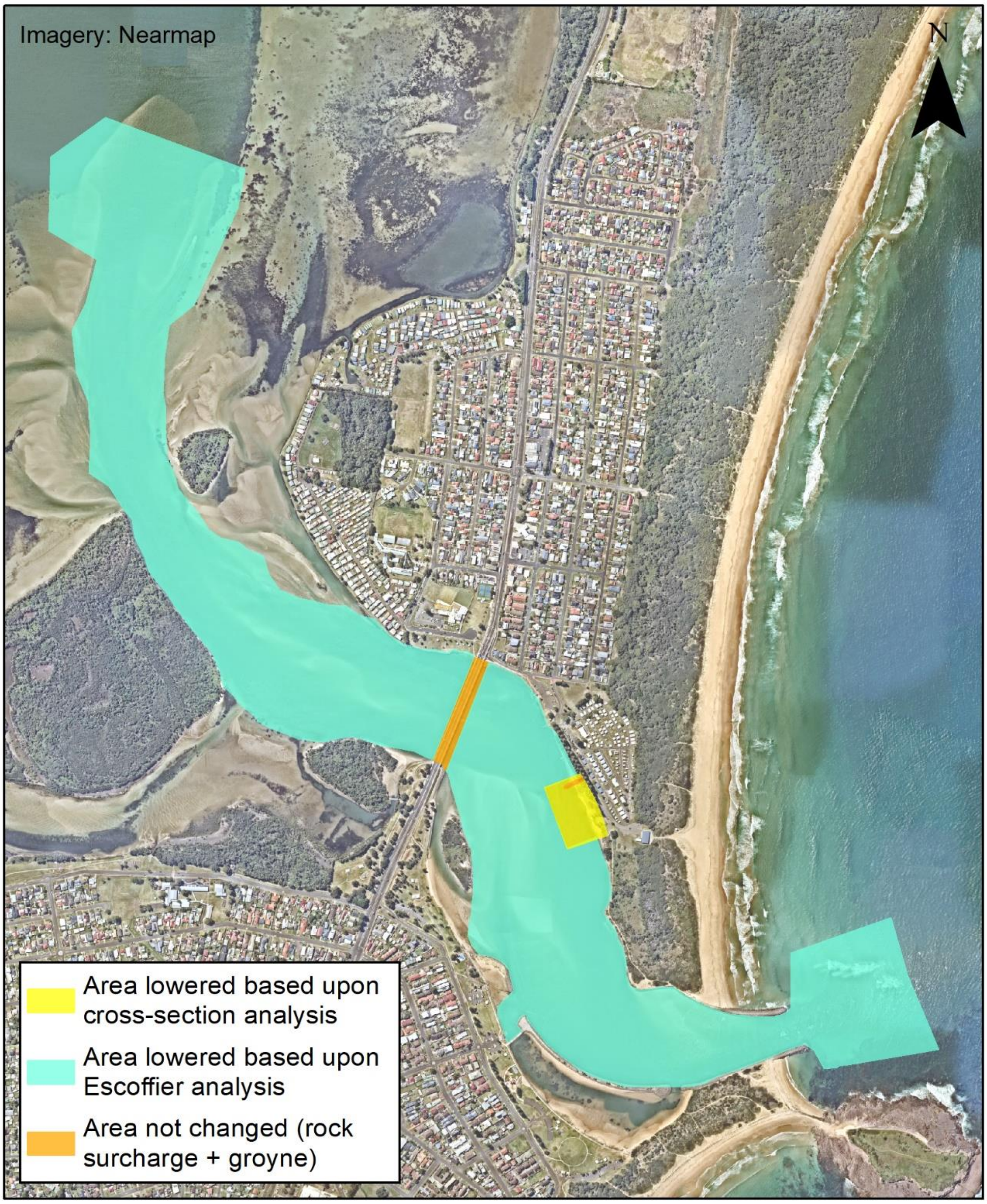

Figure A-3 Locations where different areas of the bathymetry were lowered based upon different analyses

Within the study site, potential erosion estimates indicated that the foreshore would retreat to the north-east. To allow for this within the model domain, the mesh was updated through the addition of new elements. Note, this was only applied on the section of foreshore adjacent to the study site, as shown in Figure A-4.

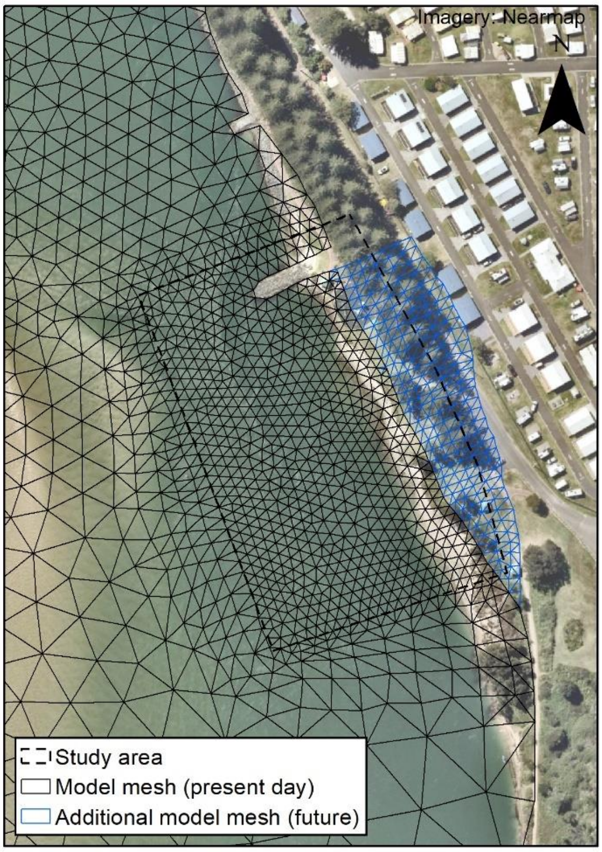

Figure A-4: Additional mesh nodes to account for erosion of the Windang foreshore in the future (2043)

A4 Model simulation period

A 37 day model simulation period was chosen to assess the local scale hydraulics. This period was selected to allow 1 week for the model spin up followed by a month of simulation which included larger spring tide conditions (Figure A-5). The model was run on a 15 minute timestep.

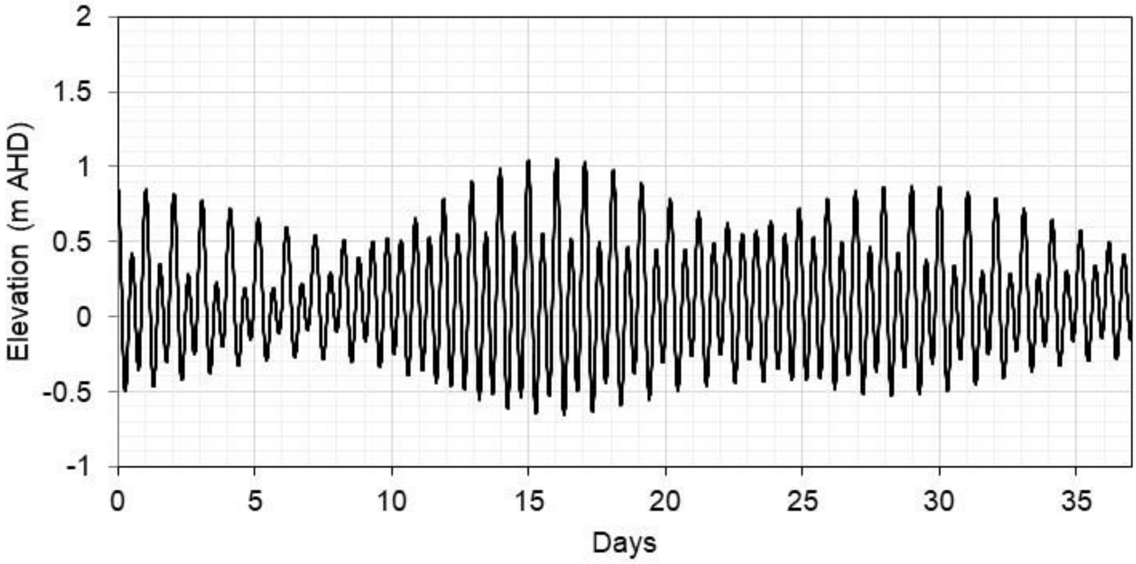

Figure A-5 Water level tidal boundary for modelling of present day (2023) conditions

A5 Model output

Velocity and depth data generated by the numerical model were used to assess the local scale hydraulics along the Windang foreshore. Bed shear (τ_b) throughout the model domain was calculated using the approach outlined by Chow (1959) shown in Equation A.1.

$$\tau_b = \frac{gn^2\rho u^2}{R_h^{\frac{1}{3}}} \qquad \text{Equation A.1}$$

where:

g = Acceleration due to gravity (9.81 m/s^2)
n = Manning's 'n' friction (0.02 as per the previous model verification; see WRL, 2019)
ρ = Density of fluid (~1,024 kg/m^3 for seawater)
u = depth averaged velocity (m/s) (model output)
R_h = Hydraulic Radius (approximated as depth (m)) (model output)

Sediment samples presented by Tucker et al. (2023) identify that the majority of sediment in the entrance channel adjacent to the Windang foreshore was medium sand (0.25 to 0.50 mm diameter). Berenbrock and Tranmer (2008) identified the critical bed shear for medium sand is between 0.194 and 0.27 N/m^2. Subsequently, a bed shear greater than this value is likely to result in sediment mobilisation and foreshore and/or bed erosion (e.g. higher bed shear can be inferred to equal a higher rate of erosion).

Appendix B Foreshore stabilisation design information and environmental conditions

B1 Assumptions for indicative capital cost estimates

The capital cost estimates for the eight foreshore stabilisation options should be considered indicative only. These estimates were intended for comparison between the options and were not prepared to the level of detail of a tender submission.

Material quantities for the options were derived by WRL from the geometry included in the representative concept design cross-sections over a 150 m length. Unit and bulk rates for materials were determined from information available to WRL from similar projects in Australia. Note that these costs are quoted exclusive of Goods and Services Tax (GST) in 2023 AUD$.

A contingency sum of 15% has been added to the sub-total costs. This is based on standard industry practice for concept design estimates of this nature.

A profit of 10% for the primary contractor has been allowed for in addition to all other costs.

B2 General design conditions

B2.1 Design scour level and erosive processes

The primary consideration for designing an option to maintain the present position of the bank in this highly erosive regime is the design scour level (i.e. the lowest sand level that the options will withstand before failure). This may be equal to the toe level for rock options but may be higher for vertical structures which require a minimum embedment. If a wide toe is adopted for rock options (i.e. a falling apron), the design scour level may be lower than the structure toe level because there is sufficient rock armour to drop down to a lower level and still support the sloped section of the structure. If the sand level at these options falls below their design scour level, they are likely to fail by undermining (see illustrative sketches in Figure B-1 reproduced from the USACE (2006) *Coastal Engineering Manual* of this failure mode for example vertical steel sheet pile and rock revetments).

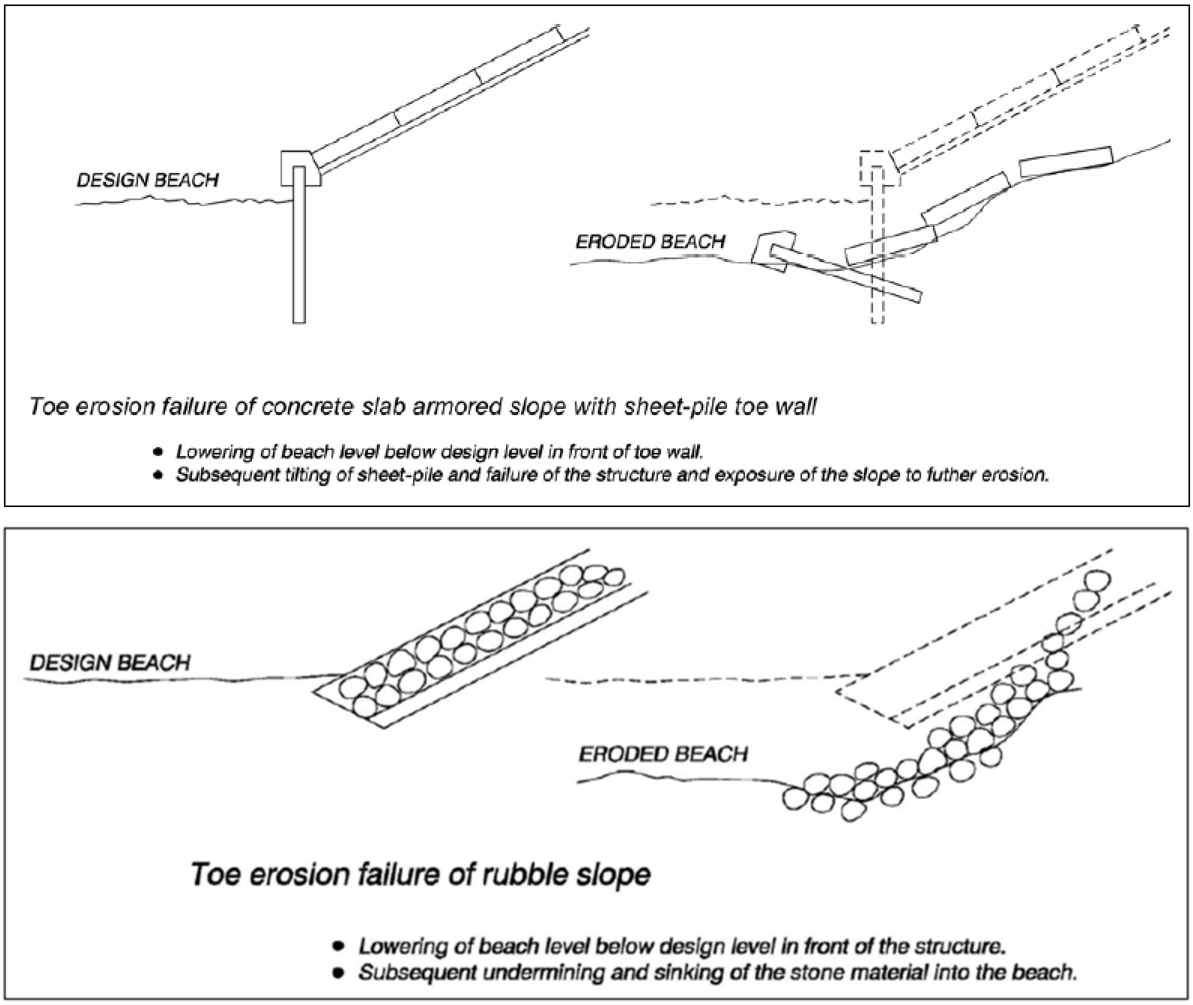

Figure B-1 Failure of sheet pile toe wall (top) and rock revetment (below)
(Reproduced from CEM Figures VI-2-45 and VI-2-44, respectively [USACE, 2006])

As discussed in Section 4, the ongoing erosion of the bed and foreshore in the Lake Illawarra entrance channel is due to high velocities associated with incoming and outgoing tides which occur each month. Erosion in this area of the entrance channel is the highest, in part, because it is located on an outer bend close to the thalweg (i.e. the line of lowest elevation along the entrance channel) where most tidal flow is concentrated.

It is also important for the foreshore stabilisation options to consider different types of scour. The change that the entrance channel is undergoing because of the permanent opening may be considered as general scour. For cross-section F4 (Figure 3.15), this general scour was estimated to result in lowering of the bed by approximately 5.7 m by 2043 (~0.3 m/year) and erosion of the current bank position. However, if one of the revetment options is constructed, additional local scour would be expected to occur since the presence of the structure would impact tidal flows.

There are nearby examples of where this type of local scour has occurred. Local scour of sand under Windang Bridge required rock to be placed beneath a section of the northern end of the bridge in 2019-2020 (Glatz, 2019). Also, near the head of the groyne constructed within the study area in 2012 (Figure 3.4), local scour has created a deep hole. While a bathymetry survey was not conducted at the time of the groyne construction, the bathymetry in this area has been lowered by between 6.2 m (2016 to 2022 surveys) and 12.4 m (2008 to 2022 surveys) by a combination of general and local scour processes. This local scour needs to be accounted for in the design of protection options for the Windang foreshore either with a design scour level which includes both general and local scour for vertical revetment options or with a falling apron of suitable width for rock revetment options.

Note that potential compaction or liquefaction of the sand under the foreshore stabilisation options, which could contribute to their settlement or collapse, has not been considered by WRL. Assessment of these, and other geotechnical failure modes, would require marine geotechnical expertise beyond WRL's capabilities.

B2.2 Design wave climate

WRL considers that wave climate, from a range of sources, is not a significant influence on the design of the foreshore stabilisation. Based on the 4 knot speed limit and existing vessel types, waves generated by boats adjacent to the Windang foreshore have small wave heights and short wave periods. Similarly, due to the short fetches within this part of the entrance channel (less than 1.4 km), wind generated waves have small heights and short periods. Long period ocean swell waves do penetrate into the Windang foreshore but they also have small wave heights. Tsunamis are extremely rare and have not been considered in this study.

B2.3 Design velocities

While the tidal velocities on larger spring tides are high (as discussed in Section 4), the design velocities for armour stability are anticipated to occur during freshwater floods. First-pass numerical flood modelling undertaken by Manly Hydraulics Laboratory (Glatz, 2020) based on the condition of the entrance channel in September 2020 for the design 1% annual exceedance probability flood event for Lake Illawarra indicated peak velocities of 2.2 to 2.4 m/s just downstream of the groyne.

The rock mass required to remain stable under these velocities is not very large. Austroads (2019) *Guide to Bridge Technology Part 8: Hydraulic Design of Waterway Structures* indicates that a median (M_{50}) rock mass of 40 kg (classed as "facing" rock with density 2.65 t/m^3) is recommended for velocities of 2.0 to 2.6 m/s. This confirms that any armour used to stabilise the Windang foreshore does not have to be heavy (compared to a revetment on the open coast), but it does have to have a large spatial extent to accommodate the aggressive scour processes present.

B2.4 Design crest level

A design crest level of approximately 1.0 m AHD has been adopted for each of the foreshore stabilisation options as this is the typical ground level at the edge of the bank. The crest level for Option 6 is slightly higher than this at 1.2 m AHD due to the needed increment in geotextile container height.

B2.5 Geotechnical conditions

Review of the coastal quaternary geological map (Figure B-2) shows only the presence of "marine sand, silt, clay, shell, gravel" (denoted Qhef/Qheb on the map) in the study area. As such, WRL has assumed that there is an absence of bedrock in the development of the foreshore stabilisation options. If one of the deeper concept designs (Options 1, 2, 3 and 6) is selected to progress to detailed design, a geotechnical investigation will be required to confirm this assumption.

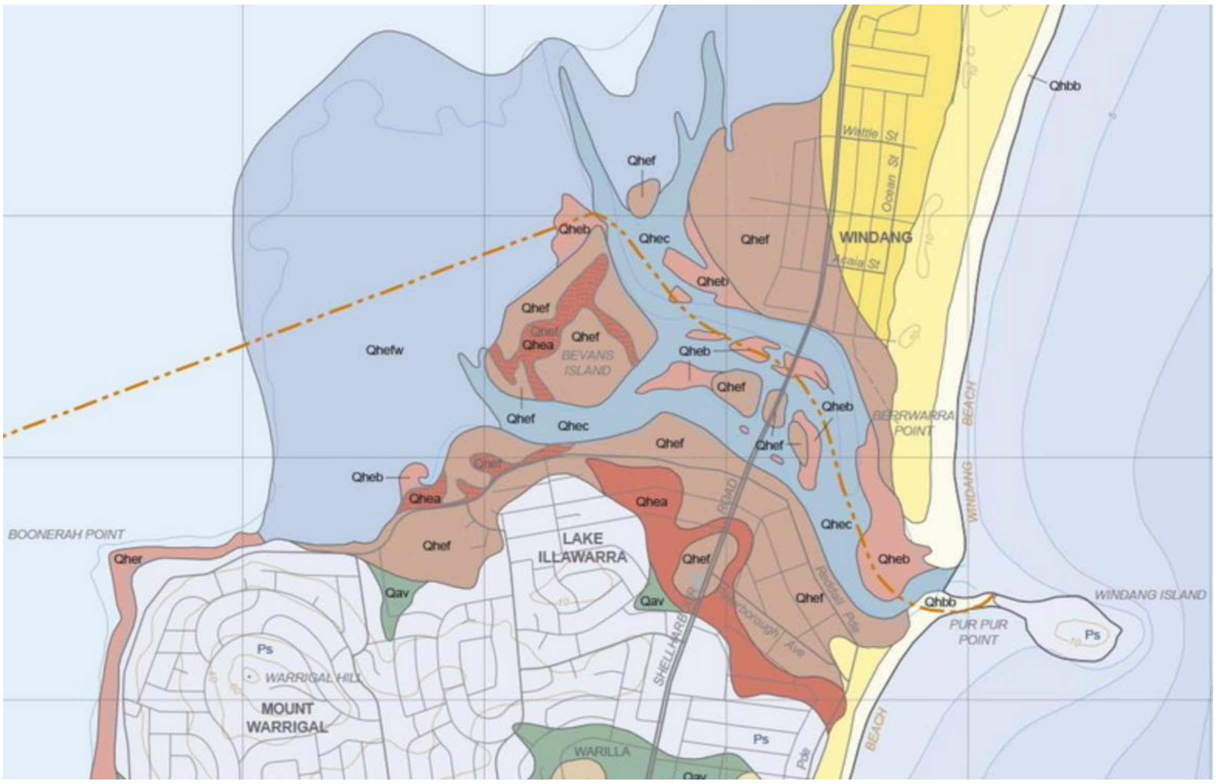

Figure B-2 Quaternary geology of the Lake Illawarra entrance channel. Note the absence of bedrock in the study area and the presence of "marine sand, silt, clay, shell, gravel" Qhef/Qheb (Source: Troedson & Hashimoto, 2013)

B3 Vertical revetment options

A vertical revetment comprising either steel sheet piles (Option 1) or concrete secant piles (Option 2) was developed to stabilise the Windang foreshore.

While the cross-shore structure footprint is relatively small for these options, the structure toe needs to be very deep to prevent failure by overturning from the retained earth. At cross-section F4, the general scour level in 20 years' time is estimated to be approximately -5.0 m AHD at the vertical revetment alignment. However, it is also necessary to make an allowance for additional local scour for these options since they are rigid and are likely to fail catastrophically rather than gradually as with rock and geotextile container revetments.

Local scour (y_s) at the revetment wall at the end of the design life (2043) was estimated using the approach outlined in HEC-23 (2001) *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance* for flow parallel to a vertical wall reproduced in Equation B.1.

$$y_s = y_1(0.73 + 0.14\pi F_r^2) \qquad \text{Equation B.1}$$

where:

y_s = Equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole)

y_1 = Average upstream flow depth (5 m)

F_r = Upstream Froude Number = $\frac{u}{\sqrt{y_{1\times g}}}$

u = Upstream velocity (2 m/s)

g = Acceleration due to gravity (9.81 m/s^2)

Assuming a flow depth of 5 m and spring tidal velocity of 2 m/s, results in an estimated local scour depth of 3.8 m. Accordingly, WRL has adopted a nominal local scour depth of 4 m, resulting in a design scour level (incorporating both general and local scour) against the vertical revetment of -9.0 m AHD. Based on discussions with an experienced piling contractor (see contact details in Appendix C), it is envisaged that the vertical wall would need to be driven down to an elevation of -20.0 m AHD. In addition to this minimum embedment of 11 m, ground anchors would also be required to withstand the overturning moment. The pile penetration level and need for ground anchors was assumed to be the same for both the steel sheet pile (Figure 5.3) and concrete secant pile (Figure 5.4) options. The estimated capital costs to construct these options are:

- Option 1: Steel sheet pile revetment - $5.2M ($34,500/m)
- Option 2: Concrete secant pile revetment - $6.8M ($45,500/m)

B4 Rock revetment options

Three different rock revetment concept designs (Options 3, 4 and 5) were developed to stabilise the Windang foreshore.

As discussed in Section B2.3, while a median rock mass of 40 kg may be of sufficient mass to remain stable under design flood velocities, WRL has adopted the next rock class up to discourage theft. A median (M_{50}) rock mass of 100 kg (classed as “light” rock with density 2.65 t/m^3; likely basalt) is recommended for velocities of 2.6 to 2.9 m/s (Austroads, 2019). The grading for this standard class of primary armour is shown in Table B-1.

Table B-1 Adopted primary armour rock grading (density: 2.65 t/m^3)

Minimum % of rock larger than	Rock mass (kg)	Rock spherical diameter (m)
0	250	0.55
50	100	0.40
90	10	0.20

The Rock Manual (CIRIA; CUR; CETMEF, 2007) provides guidance on toe design for rock revetments on a sand/gravel foreshore with low, moderate and severe scour potential. WRL considers that the

Windang foreshore has severe scour potential and based the concept designs for Options 3 and 4 on two different toe details from *The Rock Manual* reproduced in Figure B-3 and Figure B-4, respectively. The minimum slope of 1V:1.5H was adopted for all three rock concept-designs to minimise the cross-shore structure footprint. Secondary armour with M_{50} of 10 kg (2.65 t/m^3) is also included which complies with the USACE (1984) recommendation that median mass of primary armour be 10 to 15 times that of the secondary armour. A heavy grade geotextile layer has been included under the secondary armour for all rock options to prevent loss of fines, however, a “dutch toe” (where the geotextile layer is wrapped back under the primary armour at the end of the toe) has been omitted from the options because of the difficulties faced when constructing underwater (as opposed to when constructing at or above the mid-tide level). Two layers of both primary (total thickness: 0.7 m) and secondary armour (total thickness: 0.3 m) have been included on the 1V:1.5H slope. The same gradings of primary and secondary armour have been included in the toe except that there are three layers of primary armour (total thickness: 1.1 m). For Options 3 and 5 which require excavation of sand extending past the end of the toe; a slope of 1V:4H has been adopted as this would be expected to be stable underwater (Bray, 1979).

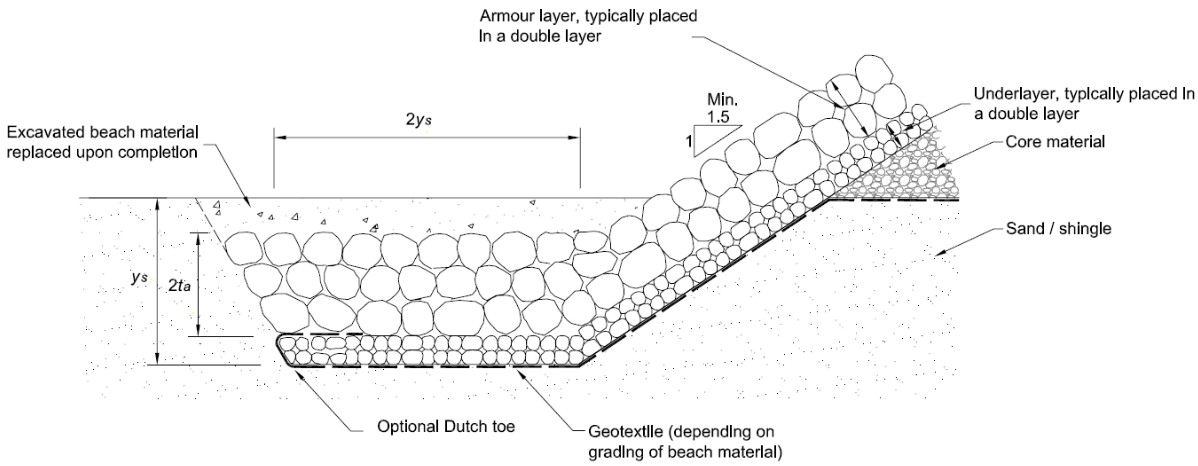

Figure B-3 Toe detail for severe scour potential (excavated trench): used to inform Option 3 (Reproduced from The Rock Manual Figure 6.63 [CIRIA; CUR; CETMEF, 2007])

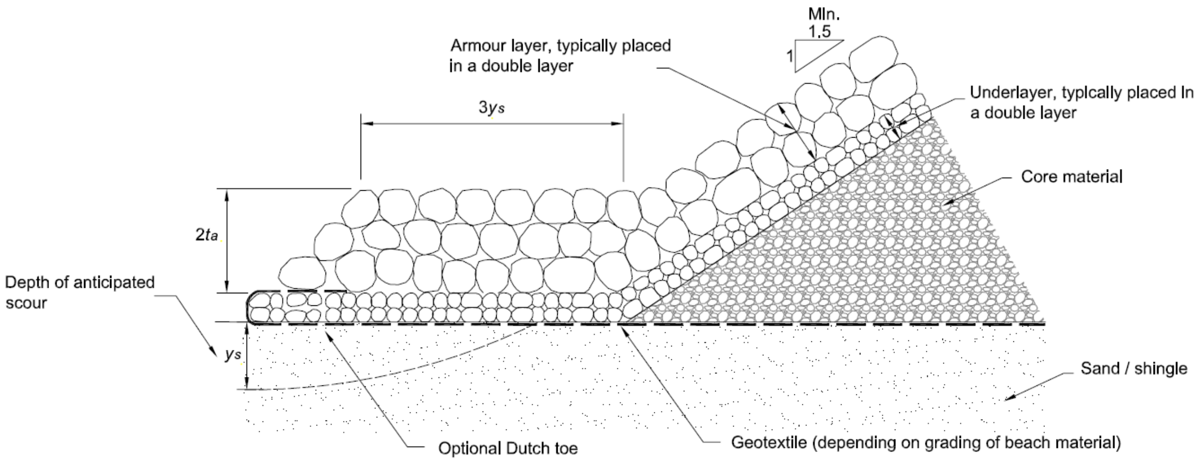

Figure B-4 Toe detail for severe scour potential (no excavation): used to inform Option 4 (Reproduced from The Rock Manual Figure 6.64 [CIRIA; CUR; CETMEF, 2007])

Both toe details for severe scour from *The Rock Manual* will account for both general and local scour (it is not necessary to estimate additional local scour for these concept designs) with rock armour and the geotextile underlayer progressively dropping down to a lower level as scour continues while continuing to support the sloped section of the structure above (as illustrated in Figure B-5).

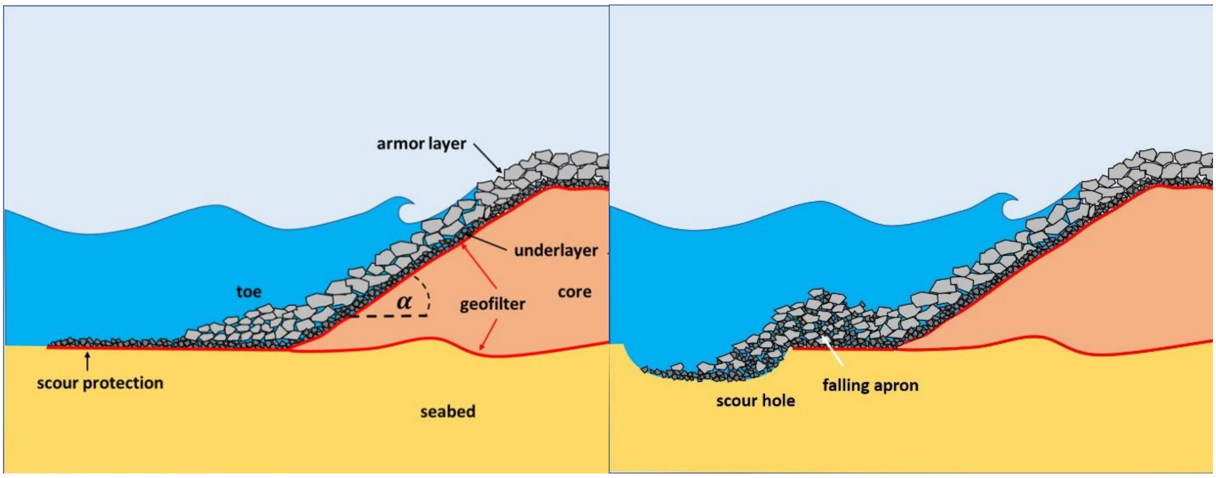

Figure B-5 Illustration of falling apron behaviour before (left) and after (right) scouring event (Source: coastalwiki.com)

The first toe detail (Figure B-3) is an excavated trench (with the "as-constructed" toe intersecting the anticipated 2043 bed profile), has a toe width equal to twice the expected general scour depth (5.7 m) to accommodate additional local scour. This detail has been adopted in Option 3, resulting in a toe elevation of -11.0 m AHD and toe width of 11.4 m. Note that the angle of the end of the toe has been angled back towards the structure to aid in underwater construction (in contrast to Figure B-3 where the toe is angled away from the structure). The second toe detail (Figure B-4) has an alternative approach, opting for no excavation but adopting a wider toe width equal to three times the expected general scour depth. This detail has been adopted in Option 4; a small amount of excavation is required (so it is identified as a minimal rather than no excavation option) resulting in a toe elevation of -2.5 m AHD and toe width of 17.1 m.

Finally, a third rock revetment concept design was also prepared as Option 5 which is not directly based on toe detail guidance from *The Rock Manual*. It has the same toe elevation as Option 4 (-2.5 m AHD), but has a narrower toe width equal to the expected general scour depth (5.7 m). This concept design is not anticipated to provide foreshore stabilisation for the next 20 years, however, it offers some flexibility, particularly if a large-scale erosion mitigation measure is implemented elsewhere in the Lake Illawarra entrance channel prior to 2043. As such, this option will require constant monitoring and progressive upgrading (e.g. future addition of rock armour to widen the toe) to the equivalent toe width of Option 4 over the 20 year planning period if other erosion management is not implemented. While this option offers some flexibility, there is also a risk of undermining and failure if either monitoring or upgrading does not occur consistently after construction throughout the 20 year design life.

The estimated capital costs to construct these options are:

- Option 3: Excavated rock revetment - $2.0M ($13,300/m)
- Option 4: Minimal excavation rock revetment - $1.2M ($8,100/m)
- Option 5: Minimal excavation rock revetment with narrow toe width - $0.7M ($4,500/m)

B5 Geotextile container revetment options

Three different geotextile container revetment concept designs (Options 6, 7 and 8) were developed to stabilise the Windang foreshore which mimic the corresponding three rock revetment concept designs (Options 3, 4 and 5).

While 100 kg geotextile containers (equivalent to the M_{50} mass adopted for the rock revetment options) are readily available, WRL has adopted the next size up for the concept designs (0.3 m^3 containers; with approximately 450 kg mass) since they have a lower specific gravity [density] and their stability thresholds under flood velocities are presently unknown (and require physical modelling if selected for detailed design of the Windang foreshore). Two layers of 0.3 m^3 containers have been included on the 1V:1.5H slope and three layers in the toe placed in a "stretcher-bond" fashion, with the long axis of the containers parallel to the bank. The outer layer of containers are proposed to be fabricated from vandal deterrent material with the inner layer(s) to be the standard type.

As with Option 5 in rock, Option 8 in geotextile containers will require constant monitoring and progressive upgrading (e.g. future addition of geotextile containers to widen the toe) to the equivalent toe width of Option 7 over the 20 year planning period).

The estimated capital costs to construct these options are:

- Option 6: Excavated geotextile container revetment - $2.4M ($16,200/m)
- Option 7: Minimal excavation geotextile container revetment - $1.5M ($9,900/m)
- Option 8: Minimal excavation geotextile container revetment with narrow toe width - $0.8M ($5,500/m)

The above cost estimates assume that sand to fill the geotextile containers is supplied at a cost of $20 per m^3. If instead the sand is "won" at no cost from nearby the site, there would be a saving of between approximately $30,000 (Option 8) to $80,000 (Option 6).

B6 Suggested considerations during subsequent detailed design

B6.1 General considerations for all options

The applicability of Actions EC3 and EC5 from the CMP (Rollason and Donaldson, 2020) should be considered prior to and during detailed design.

The design scour level (considering both general and local scour) is the primary influence on the geometry of each option. It is recommended that this be reviewed during detailed design in conjunction with a new bathymetry survey of the study area.

During detailed design of the selected option, it is suggested to consider how the revetment terminates at both ends of its 150 m length. At the upstream end, the revetment will interface with the groyne which may require a transition section. At the downstream end, the revetment will need to transition landward of the present bank position to prevent outflanking of the structure from ongoing erosion/scour of the adjacent unprotected foreshore.

B6.2 Specific considerations for some options

For the minimal excavation concept designs (Options 4, 5, 7 and 8), consideration should be given to any necessary precautions required to prevent boats running aground on the revetment.

As mentioned in Section B2.5, for the deeper concept designs (Options 1, 2, 3 and 6) a geotechnical investigation will be required to confirm the absence of any bedrock.

For the geotextile container concept designs (Options 6, 7 and 8), physical modelling is recommended to confirm that the adopted 0.3 m^3 units can withstand the design flood velocities.

B6.3 Monitoring and maintenance program

The implementation of an adequate monitoring and maintenance program is critical in order to ensure that a foreshore stabilisation structure continues to operate during its designed life. USACE (2006) defines the goal of a revetment monitoring and maintenance program as "*to recognize potential problems and to take appropriate actions to assure the project continues to function at an acceptable level*".

Foreshore stabilisation structure monitoring and maintenance consists of the following essential elements:

- Structure inspection and monitoring of both environmental conditions and structure response.
- Evaluation of inspection and monitoring data to assess the structure's physical condition and its performance relative to the design specifications.
- Determining an appropriate response based on evaluation results. Possible responses are no action, rehabilitation, or repair, of all or portions of the structure.

The main monitoring issues are to assess what parameters of the foreshore stabilisation structure to monitor, how to evaluate the monitoring data and consequently if preventive or corrective action needs to be undertaken.

At a minimum, the key structural parameters to monitor include:

- Toe Level, Crest Levels and Sand Levels on both sides of the structure
- For the rock and geotextile containers designs (Options 3-8) evolution of toe armour, structure slope, integrity of armour
- For vertical revetment designs (Options 1 and 2), structural integrity of the piles
- Potential erosion of the grassy foreshore landward of the structure following a flood event

It is recommended that additional canvassing of monitoring options be developed and a monitoring and maintenance program commensurate with the scale of the works be established during detailed design.

Appendix C List of individuals and organisations contacted during foreshore stabilisation option development

Organisation	Location	Contact Person	Telephone	E-mail
Geofabrics Australasia Pty Ltd	Moorebank	Mr Gavin Gray	0417-236-449	g.gray@geofabrics.com.au
Keller Pty Ltd	Macquarie Park	Mr James Tang	0433-935-228	James.Tang@keller.com.au