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Coastal Engineering Advice Relating to Bumbora Point Cemetery Planning Proposal

1. INTRODUCTION AND BACKGROUND

The Southern Metropolitan Cemeteries Trust (SMCT) is seeking to amend the *Randwick Local Environmental Plan 2012* to permit the use of Bumbora Point[1] in the suburb of Port Botany for a cemetery. Urbis has been engaged to assist SMCT with preparing the planning proposal for this amendment.

As part of the planning proposal, it was necessary to obtain coastal engineering advice to guide whether coastal hazards such as erosion/recession and oceanic inundation (including consideration of long term sea level rise) would affect the landscape design and layout of the cemetery. Council also specifically requested consideration of any need for long term sand replenishment and sand dune stabilisation.

Horton Coastal Engineering Pty Ltd was engaged to provide this advice, as set out herein. The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional Coastal Engineer with 25 years of coastal engineering experience. He has postgraduate qualifications in coastal engineering, and is a Member of Engineers Australia (MIEAust) and Chartered Professional Engineer (CPEng) registered on the National Engineering Register (NER). He is also a member of the National Committee on Coastal and Ocean Engineering (NCCOE) and NSW Coastal, Ocean and Port Engineering Panel (COPEP) of Engineers Australia. Peter has completed numerous coastal engineering studies in the Botany Bay area, and inspected the subject site and surrounds on 28 March 2018.

Note that all levels given herein are to Australian Height Datum (AHD). Zero metres AHD is approximately equal to mean sea level at present.

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with the following:

[1] "Bumbora Point" is the name officially recognised by the Geographical Names Board of NSW, so has been used herein. "Bumborah Point" is also used in practice, with the nearby "Bumborah Point Road" supporting that usage.

- a Landscape Masterplan of the proposed cemetery prepared by Mathew Higginson Landscape Architecture, Project No. 16721, Dwg Nos. CP101 to 103, Issue A, dated 18 June 2018;
- a survey entitled "Plan showing detail and levels, Lot 4858 DP 752015, Prince of Wales Drive, Port Botany" prepared by Rygate, Reference No. 77318, Plan No. 77318.dgn, dated 27 February 2016; and
- geotechnical reports prepared by Douglas Partners (2018a, b).

3. EXISTING SITE DESCRIPTION

An aerial view of the subject site and surrounding area is provided in Figure 1, with a zoomed aerial view in Figure 2 (aerial photographs taken 1 June 2016). The subject site is located at the western end of Yarra Bay, adjacent to the western end of Yarra Bay Beach, the rocky Bumbora Point, and Bumbora Rock Beach[2].

Short (2007) described Yarra Bay Beach as a low energy reflective beach. Prior to construction of Banks Wall and reclamation to the west for construction of Port Botany in the 1970's, Bumbora Point was a headland, with the Botany Bay waterway to the west.

There is an inlet at Bumbora Point (at D in Figure 2), with a concrete landing and wall to the west, which is understood to be part of a salt water intake (continuing to C in Figure 2 and further north) of the former Bunnerong Power Station. This was a coal-powered electric power station located about 500m north of Bumbora Point, which operated from 1929 to 1973 (and was the largest power station in the southern hemisphere from 1947 to 1966) and was mostly demolished by 1987.

Banks Wall comprises mainly concrete armour units (at A in Figure 2 and west), transitioning to rock boulders at its eastern end (at B in Figure 2), with a concrete wall above. The area landward of the wall near A has a level of about 6.8m AHD, with the top of the wall at about 7.6m AHD. A view of the eastern end of Banks Wall, Bumbora Rock Beach and surrounds is provided in Figure 3.

The area landward of D in Figure 2 is lined by rock boulders with a crest elevation of about 6.9m AHD. Circulating water tunnels, manholes and inlets associated with the former power station are located landward of this rock bank. Access to the western end of Yarra Bay Beach is via tracks at E and F in Figure 2, with F visible in Figure 5. Natural rock outcrops are visible at the back of the beach as far east as about G in Figure 2, see Figure 4. The elevation of the seaward edge of the rock outcrop at the beach, seaward of E and F, is about 1.5m AHD on average, and varying between about 1.1m and 1.7m AHD. G and H in Figure 2 are understood to comprise operational structures associated with a high pressure Caltex pipeline, as visible in Figure 6 (the top of concrete at G is at a level of about 3m AHD). The eastern seaward portion of the site, east of G, is visible in Figure 7. The access track to the beach at the eastern end of the subject site (I in Figure 2) is visible in Figure 8, as well as a view of the seaward portion of the subject site. The top level of this access path where it joins a footpath is 9.2m AHD.

At the time of the survey, sand levels at the sand/vegetation interface on the seaward side of the subject site were at about 2.5m AHD. There was good vegetation coverage over the length of this interface at the time of the site inspection. Levels rise at about 1:3 (vertical:horizontal) from the beach moving landward at the non-rocky (sandy) eastern end of the site.

[2] "Bumbora Rock Beach" is not an official name, and has been denoted for convenience herein.

Figure 1: Broad aerial view of subject site (red outline) at western end of Yarra Bay

Figure 2: Zoomed aerial view of subject site

Figure 3: View of eastern end of Banks Wall and Bumbora Rock Beach on 28 March 2018, looking east

Figure 4: View of western end of Yarra Bay Beach on 28 March 2018, looking E to ESE, with rock outcrops visible

Figure 5: Access track at F in Figure 2 on 28 March 2018, looking NNE

Figure 6: Area west of F in Figure 2 on 28 March 2018, looking NNE, with G and H structures in Figure 2 noted

Figure 7: View of eastern seaward portion of subject site on 28 March 2018, looking ENE

Figure 8: Stitched view of seaward portion of subject site from upper end of I access path from Figure 2 on 28 March 2018, looking SW (left) to WNW (right)

4. SUBSURFACE CONDITIONS

Douglas Partners (2018a, b) has completed geotechnical investigations at the subject site. From Douglas Partners (2018b), the eastern portion of the subject site was found to be underlain by fill and sand, with Hawkesbury sandstone at depth and dipping to the south and east. No geotechnical test locations were located near Yarra Bay Beach, but on the basis that a cone penetration test undertaken near the upper end of the access track at I in Figure 2 found bedrock at 0.9m AHD, it has been assumed that there would be no bedrock in the active coastal erosion zone above -1m AHD over the eastern portion of the site. This could be confirmed by an additional geotechnical investigation if required.

Based on the geotechnical model of Douglas Partners (2018b) and site observations, it has been assumed that erosion/recession coastal hazards would be constrained by bedrock and/or protection works over the western and central portions of the site, while the eastern portion of the site would be sandy and freely erodible (see Figure 9). As noted above, this could be confirmed by an additional geotechnical investigation if required.

Figure 9: Approximate boundary between bedrock constrained and freely erodible areas of subject site, and beach profile location used in calculations

5. COASTAL HAZARDS

5.1 Preamble

Given the limited data to enable rigorous delineation of coastal hazards at Yarra Bay Beach, and nature of this study (for a Concept Masterplan), an approximate but generally conservative approach was adopted for hazard definition herein.

Erosion/recession hazard lines, and elevated water levels, are determined for a 1 in 100 Annual Exceedance Probability (AEP) storm occurring at present, and in 50 years (at 2068). Hazard lines are only determined in the "freely erodible" region of Figure 9, assuming that this has an entirely sandy subsurface. An additional geotechnical investigation would be able to verify or correct this assumption. It is conservative to assume an entirely sandy subsurface.

5.2 Storm Demand Volume

During storms, large waves, elevated water levels and strong winds can cause severe erosion to sandy beaches. Storm demand represents the volume of sand removed from a beach (defined herein as the volume lost above 0m AHD) in a severe storm or series of closely spaced storms.

Based on measurements at NSW beaches, Gordon (1987) derived relationships between storm demand and ARI, at both "high demand" (at rip heads) and "low demand" (away from rip heads) areas. He estimated that the storm demand above 0m AHD was about 220m^3/m for the 100 year Average Recurrence Interval (ARI) event (equivalent to 1 in 100 AEP), for exposed NSW beaches at rip heads, and depicted a relationship between storm demand and the logarithm of ARI that was linear.

However, Yarra Bay Beach is not exposed to the full offshore wave climate, being within Botany Bay. Based on a SWAN numerical wave model[3] of the region, the following simulations were undertaken:

- water level of 0.6m AHD (around Mean High Water Springs);
- offshore wave directions of SSE, SE and ESE;
- offshore significant wave height (H_s) of 10m; and
- offshore peak spectral wave period (T_p) of 14.6s.

This showed that the waves at the subject site were predominantly reflected waves off Banks Wall, with contour plots of significant wave height and vectors of peak wave direction depicted in Figure 10, Figure 11 and Figure 12 for the SSE, SE and ESE offshore wave directions respectively.

[3] SWAN is a third-generation wave model, developed at Delft University of Technology, that computes random, short-crested wind-generated waves in coastal regions and inland waters.

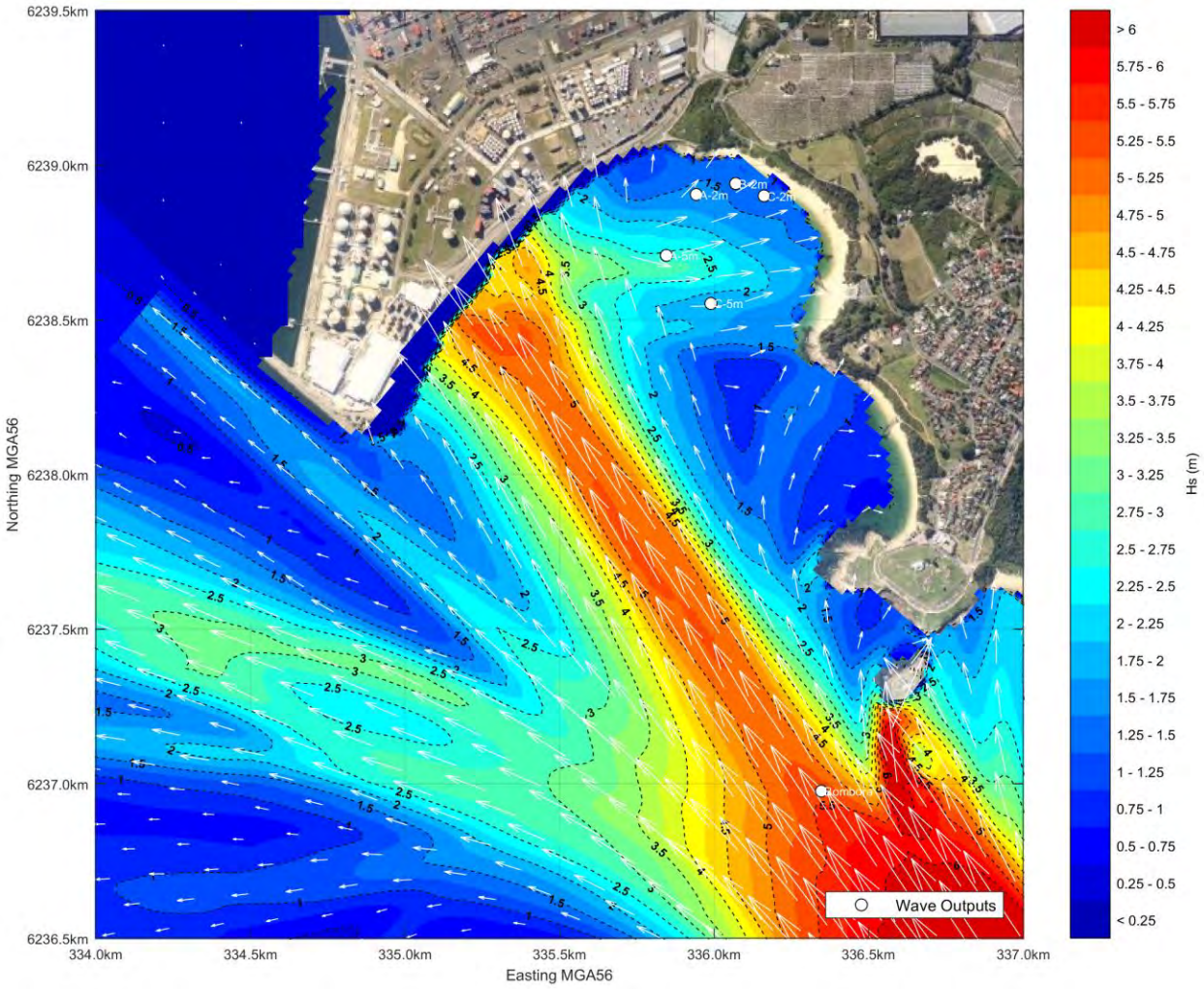

Figure 10: Contour plots of significant wave height and vectors of peak wave direction for SSE offshore wave direction

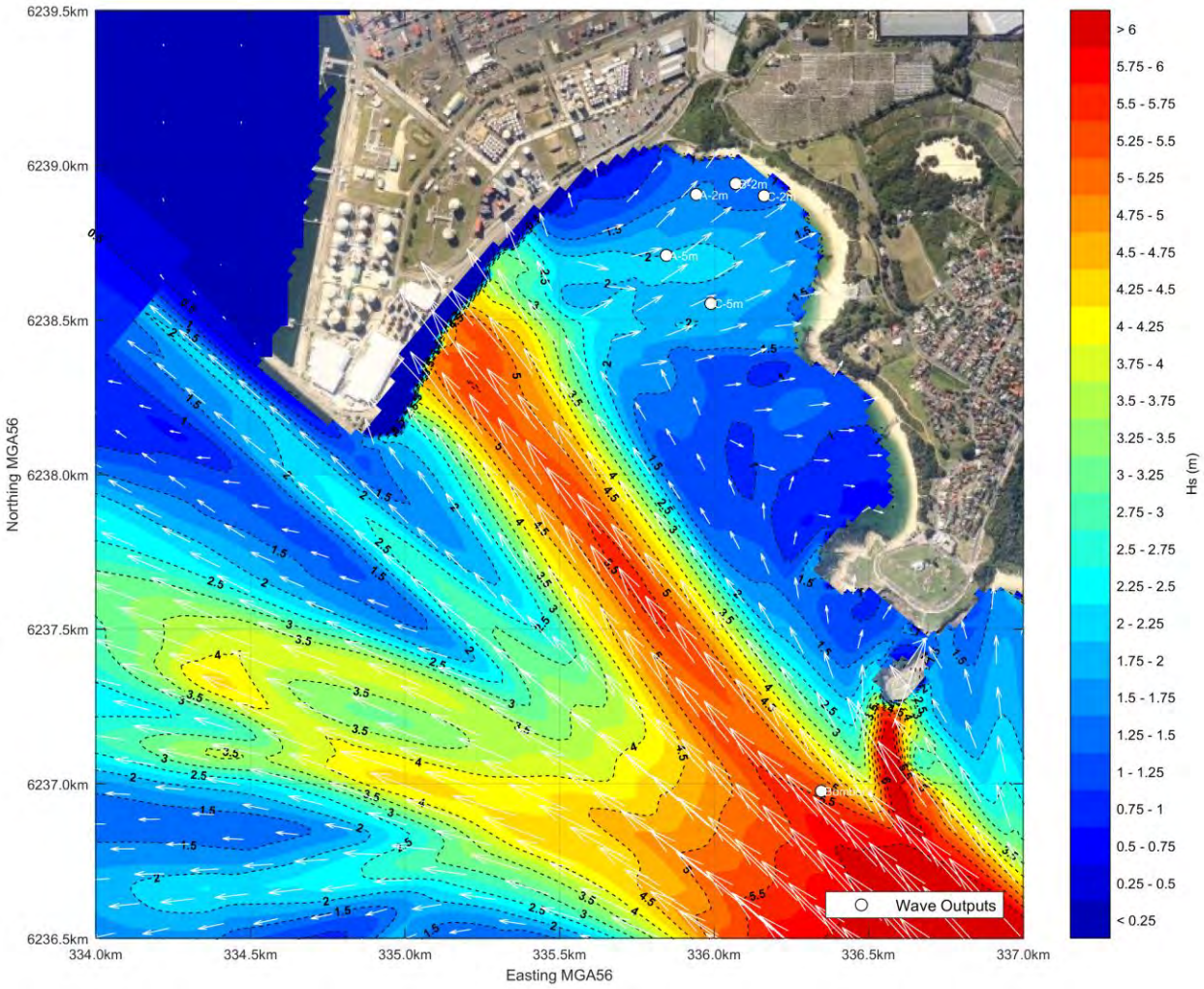

Figure 11: Contour plots of significant wave height and vectors of peak wave direction for SE offshore wave direction

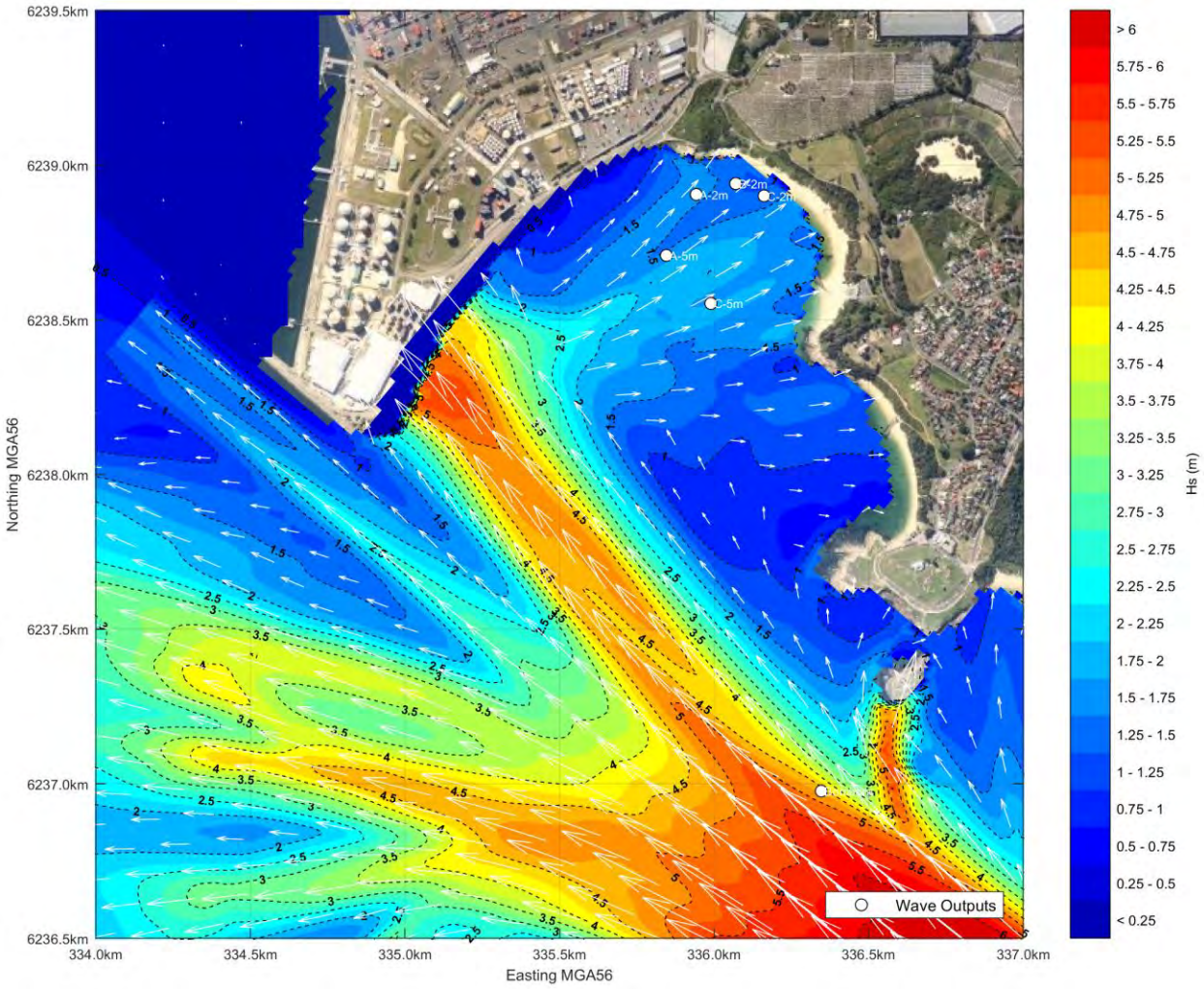

Figure 12: Contour plots of significant wave height and vectors of peak wave direction for ESE offshore wave direction

Taking the results at a depth of 5m, it was found for the critical wave direction (SSE), Location A had an H_s of 2.4m, B (midway between "A-5m" and "C-5m" in Figure 10 to Figure 12) had 2.6m, and C had 1.9m. Taking the highest value of 2.6, this means the inshore wave height is 26% of the offshore wave height.

Taking storm demand as proportional to wave height[4] and the square root of wavelength[5], this means that storm demand can be calculated as (2.6/10) × 0.56, that is 15%, of the storm demand experienced on a fully exposed beach. This means that for the 1 in 100 AEP storm and using the Gordon et al (1987) high demand relationship, the 1 in 100 AEP storm demand at Yarra Bay Beach can be estimated as 33m^3/m.

Using LiDAR information collected in 2013, which closely matched the 2016 survey, a beach profile was generated at the subject site at the location shown in Figure 9, as depicted in Figure 13.

[4] Wave energy is proportional to wave height squared, but it has been found that erosion is proportional to the square root of wave energy (Splinter et al, 2017), and hence proportional to wave height.
[5] With wave power per unit wave crest width being proportional to the square root of wavelength. For a T_p value of 14.6s, the offshore wavelength is 320m, and the wavelength in 5m water depth is 100m, with a square root ratio of 0.56.

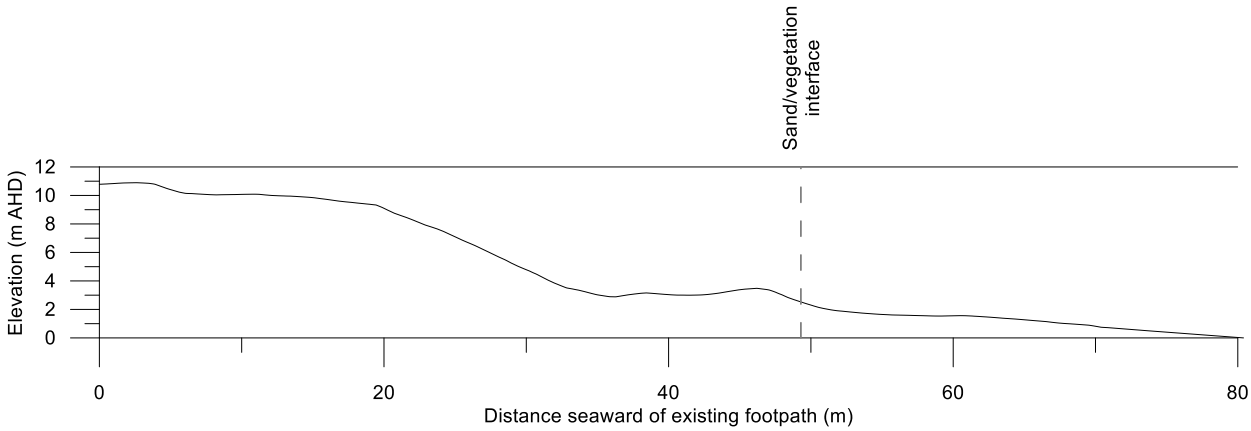

Figure 13: Beach profile at subject site

Applying the 33m^3/m storm demand to this profile, the position of the immediate 1 in 100 AEP storm demand (also extrapolating alongshore) is as shown in Figure 15. In this, the schema of Nielsen et al (1992) was applied, and the hazard line was drawn at the landward edge of the Zone of Slope Adjustment.

5.3 Long Term Recession due to Net Sediment Loss

There is no known historical beach profile data for Yarra Bay Beach, that would enable an assessment of trends in beach change at this location. Construction of Banks Wall and associated dredging of the Botany Bay entrance channel (which was designed to refract swell waves towards the northern and southern shorelines of Botany Bay, including Yarra Bay) would be expected to have increased wave energy reaching Yarra Bay Beach compared to the pre-1970's.

It was beyond the scope of the investigation reported herein to complete a rigorous analysis of historical aerial photography to assist in determining trends of beach change. However, comparison of 1943 and 2016 aerial photography would indicate that Yarra Bay Beach has receded slightly at the NW end (near the subject site), and prograded substantially at the SE end. With greater wave energy from the west as a result of reflected energy off Banks Wall, this trend in shoreline movement (NW to SE) is not unexpected.

With approximately 10m of recession over 73 years at the subject site from comparison of 1943 and 2016 photography (or 0.14m/year recession) and assuming this continues in the future (that is, assuming that Yarra Bay Beach has not already adjusted to the altered wave climate since construction of Banks Wall), then in 50 years (at 2068) this represents 7m of recession.

5.4 Sea Level Rise

In Intergovernmental Panel on Climate Change [IPCC] (2013), global mean sea level rise projections were presented for 4 representative concentration pathways (RCP) scenarios. The projections were based on results from 21 Atmosphere-Ocean Global Circulation Models for each scenario, with median, 95% and 5% exceedances reported (based on the range of model results).

To be conservative herein, the highest of the 4 RCP scenarios was adopted, namely the RCP8.5 scenario. The sea level rise values reported in IPCC (2013) at 2013 (base profile date) and

2068 (all relative to 1986-2005) are listed in Table 1, as well as the calculated sea level rise from 2013-2068.

Table 1: Global mean sea level rise values from IPCC (2013) for most conservative RCP8.5 scenario

Year	Sea level rise (m)		
	95% exceedance	Median	5% exceedance
2013	0.04	0.05	0.07
2068	0.30	0.40	0.52
2013 to 2068	0.26	0.35	0.45

It is also relevant to consider regional sea level rise variation, that is how the study area sea level rise may vary from the global mean. From Figure 13.21(a) of IPCC (2013), although the resolution is coarse, it can be estimated that sea level rise in NSW is projected to be 10-20% larger than the global mean at 2081-2100 (compared to 1986-2005). Assuming these increases also apply at 2068 and for all statistics, and applying a 20% increase, the sea level rise scenarios in Table 2 were determined.

Table 2: Sea level rise values considering regional sea level rise variation

Year	Sea level rise (m)		
	95% exceedance	Median	5% exceedance
2013 to 2068	0.31	0.42	0.54

The median sea level rise value of 0.42m was adopted herein for analysis, with consideration of the 95% and 5% exceedance values as part of sensitivity testing.

5.5 Long Term Recession Due to Sea Level Rise

Bruun (1962) proposed a methodology to estimate shoreline recession due to sea level rise, the so-called Bruun Rule. It can be described by the equation (Morang and Parson, 2002):

$$R = \frac{S \times B}{h + d_c} \qquad (1)$$

where R is the recession (m), S is the long-term sea level rise (m), h is the dune height above the initial mean sea level (m), d_c is the depth of closure of the profile relative to the initial mean sea level (m), and B is the cross-shore width of the active beach profile, that is the cross-shore distance from the initial dune height to the depth of closure (m). Equation 1 is a mathematical expression that the recession due to sea level rise is equal to the sea level rise multiplied by the average inverse slope of the active beach profile, with the variables as illustrated in Figure 14.

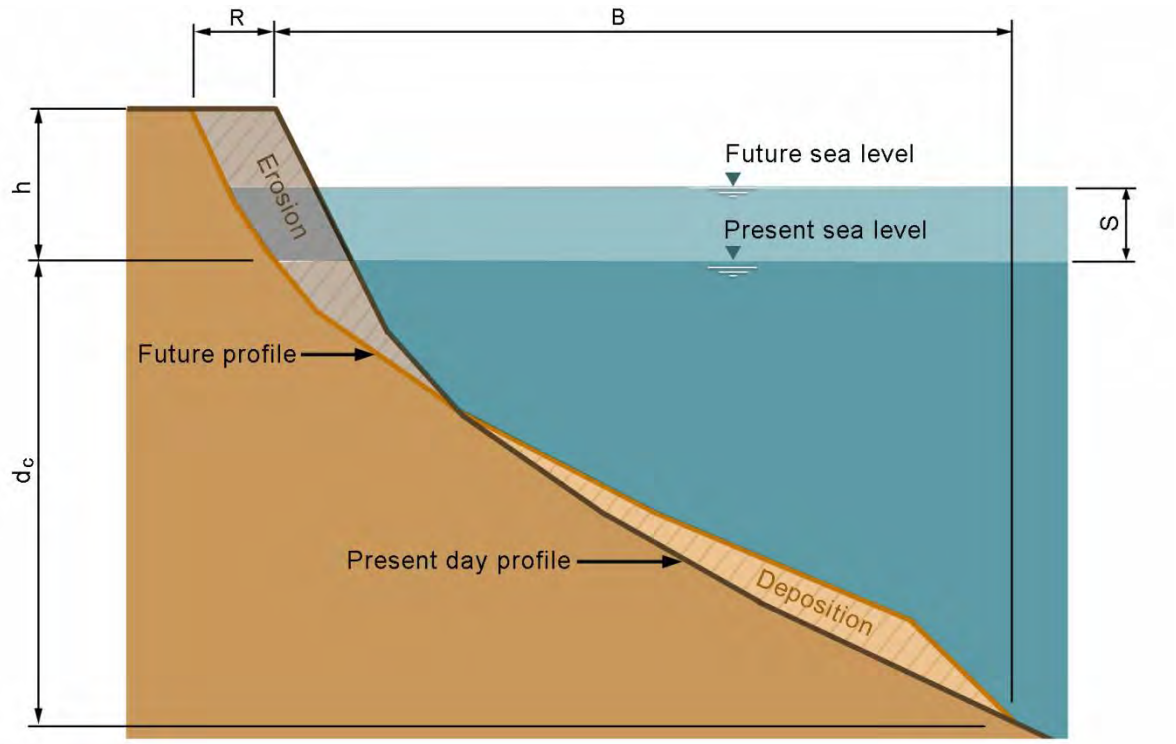

Figure 14: Illustration of variables in the Bruun Rule

There are a number of methods available to estimate the depth of closure, including techniques based on wave and sediment characteristics, sedimentological data, and field measurements. Observation of bathymetric data in Yarra Bay from Australian Hydrographic Chart AUS199 indicated that there was excess of sand (compared to an equilibrium profile) between about -3m AHD and -6m AHD, with a flat inverse slope of about 140 at this depth. On this basis, a depth of closure at -3m AHD was considered to be reasonable. The appropriate slope for the Bruun Rule is thus the steeper beach face and offshore slope down to -3m AHD, with an inverse slope of about 12.

For the median sea level rise of 0.42m at 2068, and for an inverse slope of 12, that would cause 5m of long term recession due to sea level rise at 2068.

5.6 Delineation of Immediate and 2068 Hazard Lines

Combining 7m of long term recession due to net sediment loss (Section 5.3), and 5m of long term recession due to sea level rise (Section 5.5), gives a total recession of 12m at 2068. The Immediate Hazard Line (determined as described in Section 5.2), and 2068 Hazard Line located 12m landward, are depicted in Figure 15. The seaward edge of the proposed cemetery development (excluding landscaping) is also depicted in Figure 15 (that is, the seaward extent of grave sites). It is evident that the proposed cemetery development (grave sites) is at least 5m landward of the 2068 Hazard Line, and generally about 10m landward. On this basis, and given the conservative hazard definition, the general layout of the cemetery is acceptable from a coastal engineering perspective (as long as an approximate 70 year planning period can be accepted[6]).

[6] It would be about 70 years before the cemetery development would be impacted by erosion/recession, using the parameters adopted herein. This would change to about 60 years or 80 years if the 5% or 95% exceedance sea level rise projections respectively were realised, and all other parameters remained the same.

Figure 15: Approximate Immediate and 50 year (at 2068) Hazard Lines at western end of Yarra Bay Beach (at landward edge of Zone of Slope Adjustment), with seaward edge of proposed cemetery grave sites development shown

There would be ample time to reassess coastal hazards in the future as projected sea level rise is realised, and consider management options if the cemetery grave sites became threatened by coastal erosion/recession. An additional geotechnical investigation may indicate that the adopted hazard lines are overly conservative if bedrock is found in the active coastal erosion zone above -1m AHD.

5.7 Need for Long Term Sand Replenishment and Sand Dune Stabilisation

Based on the hazard lines delineated in Section 5.7, there should be no need for any beach nourishment (importation of sand) at the subject site over the next 60 years to prevent the cemetery grave sites being impacted. Management of erosion/recession over the longer term would require consideration of other options besides beach nourishment, such as coastal protection works, as nourishment is not typically an efficient solitary option to prevent severe storm erosion impacts. There would be ample time to assess these management options in the future. In practice, adoption of coastal protection works to limit the landward extent erosion/recession, combined with beach nourishment to enhance beach amenity, can be an effective means of managing impacts associated with erosion/recession. In the future, this would have to be assessed along with consideration of social, economic, environmental, and engineering constraints and opportunities.

The trend in sand movement from NW to SE would also promote consideration of mechanical sand relocation from SE to NW as a possible future management option.

5.8 Elevated Water Levels

DECCW (2010) has noted that the present day 1 in 100 AEP ocean still water level at Fort Denison is 1.44m AHD. This would also be generally applicable at the subject site. Applying the median projected sea level rise of 0.42m over 50 years from Section 5.4, this would increase to 1.86m at 2068.

Wave action can cause temporary increases in water level above the still water level. A wave runup level of 5m AHD would be unlikely to be exceeded over the next 50 years at the subject site. With the cemetery development (grave sites) generally above 9m AHD, wave runup is unlikely to impact on the proposed cemetery grave sites over this design life and well beyond.

6. REVIEW OF MASTER PLAN FROM COASTAL ENGINEERING PERSPECTIVE

The Concept Master Plan developed by Mathew Higginson Landscape Architecture is depicted in Figure 16.

Figure 16: Concept Master Plan prepared by Mathew Higginson Landscape Architecture

Key features relevant to coastal hazards in Figure 16 are listed below:

5. Shared zone promenade landward of seawall
6. Foreshore boardwalk
7. Landmark destination with interpretive media
8. Foreshore vegetation zone (natural vegetation retained and enhanced)

A 20m foreshore buffer zone has been established (approximately landward of a level of 1.5m AHD at the eastern end of the site, and landward of the crest of coastal protection works at the western end of the site, with a transition between). A native vegetation zone is also to be located landward of this buffer zone, to the east of the landmark destination. No graves are to be located within this buffer zone and vegetation zone (see Figure 15 for seaward edge of cemetery development), with the only proposed development in this area being the landmark destination and foreshore boardwalk. On this basis, the general layout of the cemetery is acceptable from a coastal engineering perspective.

Comments on the key Master Plan features from a coastal engineering perspective are provided below.

For Item 5 (shared zone promenade landward of seawall), this would be enhancing a current use at the site, and would be protected by the revetment and concrete wall coastal protection works located seaward. These works would need to be maintained (by others), as would be expected to occur. Significant wave overtopping of these works would not be expected for severe storms, say up to the 1 in 100 Annual Exceedance Probability (AEP) event over the next 50 years. Overtopping in more severe storms would have the potential to damage the pathway landward, so it is recommended that a high strength surface (such as reinforced concrete) is considered in preference to loose pavers and the like.

For Item 6 (foreshore boardwalk), at the position in Figure 16 the boardwalk would be exposed to significant wave action at times. To prevent the boardwalk being damaged by wave action and erosion/recession, it would need to be piled down to bedrock, or sufficiently below -1m AHD in sandy areas, with allowance for wave forces on structural elements and sand slumping forces on piles. The depth of piling would increase moving east.

The boardwalk as described above would be a relatively expensive construction. If the boardwalk was further landward then the risk of undermining would be lower and it could have shallower foundations, although being further landward may have the potential to disturb dune vegetation. At present, alongshore foreshore access in this area is by foot over the exposed rock and sand, and to avoid the need to maintain a structure exposed to wave action, there could be consideration of removing the boardwalk altogether off the beach. This would also have the advantage of avoiding a structure in an area where people tend to lie on the beach.

The boardwalk is depicted with a deck level of about 2.5m AHD in the Masterplan. Being elevated above the beach, wave action would typically propagate under the boardwalk, but there is still the potential for wave overtopping on to the deck in severe storms.

If the boardwalk remains in its current position, then a relatively open deck construction (such as Fibre Reinforced Plastic mesh grating) would be beneficial in terms of reduced wave forces on the decking (a common failure mechanism of timber boardwalks with planks is popping out of the decking) and being non-slip. This would need to be balanced against less comfort under

bare feet compared to timber decking. The design life of various materials in the marine environment should also be considered.

For Item 7 (landmark destination with interpretive media), this is depicted (with an example image) as an elevated piled timber viewing platform in the Master Plan. The foreshore already juts out seaward at this location, so cost would be reduced if it is not extended beyond the existing foreshore, or it is extended by cantilevering. Any piling would need to be designed for wave impact forces. The current wall level at this location is about 7.3m AHD, and this would be a level at which significant wave uplift would not be expected, say up to the 1 in 100 AEP event over the next 50 years.

For Item 8 (foreshore vegetation zone), it is recommended that the principles in the *Coastal Dune Management Manual* (Department of Land and Water Conservation, 2001) are considered. Dune management involves the maintenance of dunes and their vegetative cover. Well maintained dunes hold a reserve of sand on the beach to cater for storm erosion and provide a barrier to oceanic inundation. The establishment and maintenance of dune vegetation also minimises loss of windblown sand from the beach compartment.

7. CONCLUSIONS

The proposed cemetery grave sites at Bumbora Point are unlikely to be impacted by erosion/recession for at least 60 years, with conservative hazard line delineation. If this planning period can be accepted, the general layout of the cemetery is acceptable from a coastal engineering perspective. An additional geotechnical investigation may indicate that the adopted hazard lines are overly conservative if bedrock is found in the active coastal erosion zone above -1m AHD.

There should be no need for any beach nourishment at the subject site over the next 60 years to prevent the cemetery grave sites being impacted. Management of erosion/recession over the longer term would require consideration of other options besides beach nourishment, such as coastal protection works. There would be ample time to assess these management options in the future. The trend in sand movement from NW to SE would also promote consideration of mechanical sand relocation from SE to NW as a possible future management option.

The proposed foreshore boardwalk in the Master Plan would be exposed to significant wave action at times. To prevent the boardwalk being damaged by wave action and erosion/recession, it would need to be piled down to bedrock, or sufficiently below -1m AHD in sandy areas, with allowance for wave and sand slumping forces.

The landmark destination in the Master Plan could be extended by cantilevering, if required, to avoid piling. Any piling would need to be designed for wave impact forces.

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9. SALUTATION

If you have any further queries, please do not hesitate to contact Peter Horton via email at peter@hortoncoastal.com.au or via mobile on +61 407 012 538.

Yours faithfully
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