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30 May 2019

Additional Coastal Engineering Advice on Bumbora Point Cemetery Planning Proposal

1. INTRODUCTION AND BACKGROUND

Southern Metropolitan Cemeteries Trust (SMCT) is seeking to amend *Randwick Local Environmental Plan 2012* to permit the use of Bumbora Point in the suburb of Port Botany for a cemetery. Urbis has been engaged to assist SMCT with preparing the planning proposal for the 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. Randwick Council also specifically requested consideration of any need for long term sand replenishment and sand dune stabilisation.

Horton Coastal Engineering was engaged to provide this advice, as set out in a report "Coastal Engineering Advice Relating to Bumbora Point Cemetery Planning Proposal" and dated 3 July 2018. Since completion of this 2018 report, the report has been reviewed by BMT (2018), and an additional geotechnical investigation has been undertaken (Douglas Partners, 2018) to refine the understanding of subsurface conditions in areas potentially affected by coastal erosion/recession. In the report herein, the BMT review is responded to (see Section 2), the Douglas Partners (2018) report is reviewed (see Section 3), and the coastal hazard definition is updated accordingly (see Section 4). The Concept Master Plan for the proposed cemetery has also been revised since the 2018 Horton Coastal Engineering report, with comments on this revised Master Plan provided in Section 5. 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. RESPONSE TO BMT (2018)

Comments of BMT (2018) are provided in italics below, with numbering added for reference, and the Horton Coastal Engineering response below each statement:

1. *Using 220m^3/m as representative of storm demand for an offshore wave height of 10 m will have underestimated potential storm demand (page 7).*

The 10m offshore wave height was not adopted for storm demand calculations as that was used in the SWAN modelling, but because it was considered to be the order of magnitude

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offshore wave height that would produce storm demands of 220m^3/m on the open coast (nominally a 100 year ARI event). This is consistent with the 100-year ARI offshore significant wave height of 9.5m for a 1 hour duration offshore of Sydney determined by Louis et al (2016). It is considered that the adopted storm demand of 33m^3/m is reasonable.

2. *The use of only 2 dates of aerial photography to determine long term trends may have over or under estimated actual recession, depending on beach state in the photographs (page 7). The historical long term beach position trend recommended in the Coastal Engineering Report should be corroborated by additional photographic (or other suitable) evidence of this location (page 9).*

Yarra Bay Beach is not a particularly dynamic beach environment subject to regular changes in beach state from accreted to eroded and vice versa. Furthermore, the position of the sand/vegetation interface was used to determine the recession rate, and this is not typically subject to short-term variations, but long-term trends.

The adopted rate of long term recession due to net sediment loss of 0.14m/year is considered to be conservative, as the adjustment in planform at Yarra Bay Beach over time is likely to have been mostly driven by construction of Banks Wall and associated dredging of the Botany Bay entrance channel in the 1970's, and Yarra Bay Beach is likely to have already adjusted to this altered wave climate. That is, the historical rate is unlikely to be sustained into the future. Furthermore, a conservative assessment of the historical photography variation was adopted to derive the rate, with the eastern portion of the proposed development site indicating no recession (no variation in sand/vegetation interface position) from 1943 to 2016.

It is considered to be beyond the scope of a planning proposal investigation to undertake formal aerial photography analysis of many dates. That stated, review of Google Earth photography from 2000 to 2019 would confirm that the adopted rate is conservative as the sand/vegetation interface has been consistently stable or moved seaward over this period (indicative of progradation, not recession).

3. *Simplistic wave run up calculations suitable to a high level assessment of this kind could have been calculated. Likewise, simple wave overtopping rates to consider impacts to proposed foreshore structures could also have been calculated (page 7).*

It was noted in the 2018 Horton Coastal Engineering report that a wave runup level of 5m AHD would be unlikely to be exceeded over the next 50 years at the subject site (so wave runup calculations were clearly undertaken), and with the cemetery development (grave sites) generally above 9m AHD, wave runup would be unlikely to impact on the proposed cemetery grave sites over this design life and well beyond.

BMT completed a runup calculation based on Nielsen and Hanslow (1991), but did not specify what wave period they adopted in the calculation, determining 100 year ARI 2% wave runup levels of 4.0m AHD at present and 4.4m AHD at 2068. It can be noted that the calculations were apparently erroneous as wave setup was included in the still water level, which is double counting (the runup value in the calculation method inherently includes wave setup). BMT also considered that wave uplift may be possible on the boardwalk. It is contended that wave uplift may be limited if there are not solid structures against which air may be trapped (eg a relatively open deck construction almost certainly would not experience wave uplift), although as already identified it is recognised that the boardwalk would be subject to significant (horizontal) wave forces.

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Horton Coastal Engineering stands by the conclusion that, using a number of wave runup calculation methodologies and assuming a variety of beach slopes, a wave runup level of 5m AHD would be unlikely to be exceeded over the next 50 years at the subject site, and that wave runup would be unlikely to impact on the proposed cemetery grave sites over this design life and well beyond. It is also considered that this can be applied to the boardwalk, that is, if the boardwalk was relocated to be above 5m AHD it would be at a sufficiently low risk of wave impact (for initial planning purposes).

The potential for wave overtopping on to the deck of the proposed boardwalk in severe storms was noted in the 2018 Horton Coastal Engineering report. Overtopping rates would be dependent on the deck level above the land (as runup could pass under the deck) and are not necessary for the scope of a planning proposal investigation when it has been identified that the boardwalk location itself may be problematic. It is recognised that this is an issue that would need to be considered if the boardwalk was to remain in its originally proposed position.

4. *The Coastal Engineering Report focuses largely on the coastal risk potential to the cemetery elements of the Planning Proposal, with limited commentary on recreational facilities on the foreshore such as the boardwalk, or other recreational uses (page 10).*

In the 2018 Horton Coastal Engineering report, key coastal risk issues related to four recreational facilities were outlined, namely the shared zone promenade landward of the seawall, foreshore boardwalk, landmark destination with interpretive media and foreshore vegetation zone (see pages 18 and 19 of that report).

5. *A 50 year planning period (to 2068) was applied to the development proposal for the purpose of assessing coastal risks. For the purpose of the subdivision of land, a planning period of 100 years typically applies as the subdivision is expected to apply in perpetuity (page 11).*

A planning period of 100 years has been considered as part of an updated coastal hazard definition herein. This also addresses a request from Randwick City Council (in an email from Ting Xu, Senior Strategic Planner, dated 23 August 2018), namely:

> "A peer review of the Coastal Engineering Study suggests that a 50-year planning period may not be adequate to assess use of the site as a cemetery as this land use will likely remain in perpetuity. The Concept Landscape Plan and the extent of the proposed foreshore buffer zone in particular is required to be amended to respond to the coastal processes and hazards over the 100-year planning period".

It is considered that the appropriate planning period to adopt for the grave sites would depend on whether they could be considered to be relocatable. If relocation is considered to be a concern, then 100 years may be more appropriate than 50 years.

3. REVIEW OF ADDITIONAL GEOTECHNICAL INFORMATION

In the 2018 Horton Coastal Engineering report, it was noted that an additional geotechnical investigation would assist in defining the potential for bedrock to constrain the realisation of erosion/recession coastal hazards, and this was also recognised by BMT (2018). Accordingly, Douglas Partners (2018) has completed an additional geotechnical investigation at the subject site. This investigation comprised Dynamic Cone Penetrometer (DCP) tests at the 12 locations (1, 2, 6, 7, 7A, 8, 9, 10, 12, 13 and 15) shown in Figure 1.

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Figure 1: Test locations, ground levels and inferred bedrock levels from Douglas Partners (2018)

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The inferred rock levels from the testing are shown in red in Figure 1. It is evident that bedrock levels of 0.1m to 0.3m AHD apply over the eastern portion of the site (DCP 9, 10[1], 13 and 15), increasing to 1.3m to 1.8m AHD over the central southern portion (DCP 7A, 8, 11 and 12). Bedrock levels would be expected to increase further over the NW portion of the site (DCP 1, 2 and 6[2]), but could not be determined as the bedrock depths were below the DCP testing limits at these locations.

Horton Coastal Engineering holds LiDAR data for the study area that was captured in April 2013. Generation of contours from this point data indicates that there is a kink in the contours around Line A in Figure 2 (where there is also a change in vegetation type), which in conjunction with analysis of the DCP inferred rock levels is considered to be related to elevated bedrock levels to the west of the kink and a more sandy dominated subsurface to the east of the kink. This could be further assessed with additional geotechnical investigations in this area as part of future project stages, if required.

4. REVISED COASTAL HAZARD DEFINITION

Taking account of the additional geotechnical information described in Section 3, and adding in a hazard line for a 100 year planning period (at 2119), a revised coastal hazard definition has been completed as outlined below.

A methodology that has previously been used by Horton Coastal Engineering (and peer reviewers have accepted) to approximately deal with inerodible materials in a beach profile is to reduce the beach erosion volume (storm demand) by the proportion of the profile above 0m AHD (the assumed scour level at this beach) that is inerodible up to the runup limit (where 5m AHD is the wave runup limit[3]). A 100 year ARI storm demand of 33m^3/m was adopted in the 2018 Horton Coastal Engineering report, assuming an entirely sandy subsurface. At DCP12 a rock level of 1.8m AHD was found. Reducing the erosion volume by 1.8 ÷ 5 or 0.36, the revised volume is 21m^3/m there. At DCP13 a rock level of 0.3m AHD was found, which does not significantly alter the immediate hazard line. However, at DCP8 with rock at 1.3m AHD, this would reduce erosion of the profile above DCP8, shifting the 2119 hazard line seawards. Rock levels of 0.3m AHD at DCP9, 10 and 15 do not allow a significant reduction in hazards at the eastern end.

The Immediate, 2069 (50 year) and 2119 (100 year) hazard lines, considering adjustments for subsurface bedrock, are shown in Figure 3. The hazard lines within the orange-coloured ellipse in Figure 3 have the highest uncertainty due to no geotechnical testing results there, but the general trend of the lines is considered reasonable. Additional geotechnical investigations in this area would reduce the uncertainty, if required as part of future project stages.

Douglas Partners (2018) noted that a minimum setback for burials of 20m from the toe of clifflines was appropriate, with a reduced setback of 10m for ash interments. This is to allow space for natural attenuation of chemical and nutrient loading in groundwater, that is, to reduce the risk of contaminated water (seepage) from the cliff face. Douglas Partners (Sally Peacock, personal communication) also noted that a 10m to 20m buffer should apply from the high water mark in sandy areas, adjusted for long term recession. However, coastal hazards, rather than burial setbacks, govern in sandy areas. Approximate locations of these 10m and 20m buffers are shown in Figure 3.

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[1] At DCP 10, blow counts were high enough for refusal, but without the characteristic 'bounce' on a hard stratum, and thus more suggestive of a clay or extremely weathered rock over bedrock (hence why this result has a "?").
[2] Although note that DCP 6 did have refusal, but this could be due to relatively dense or hard soils and not rock.
[3] Note that this is conservative for a 100 year ARI storm over a 100 year planning period.

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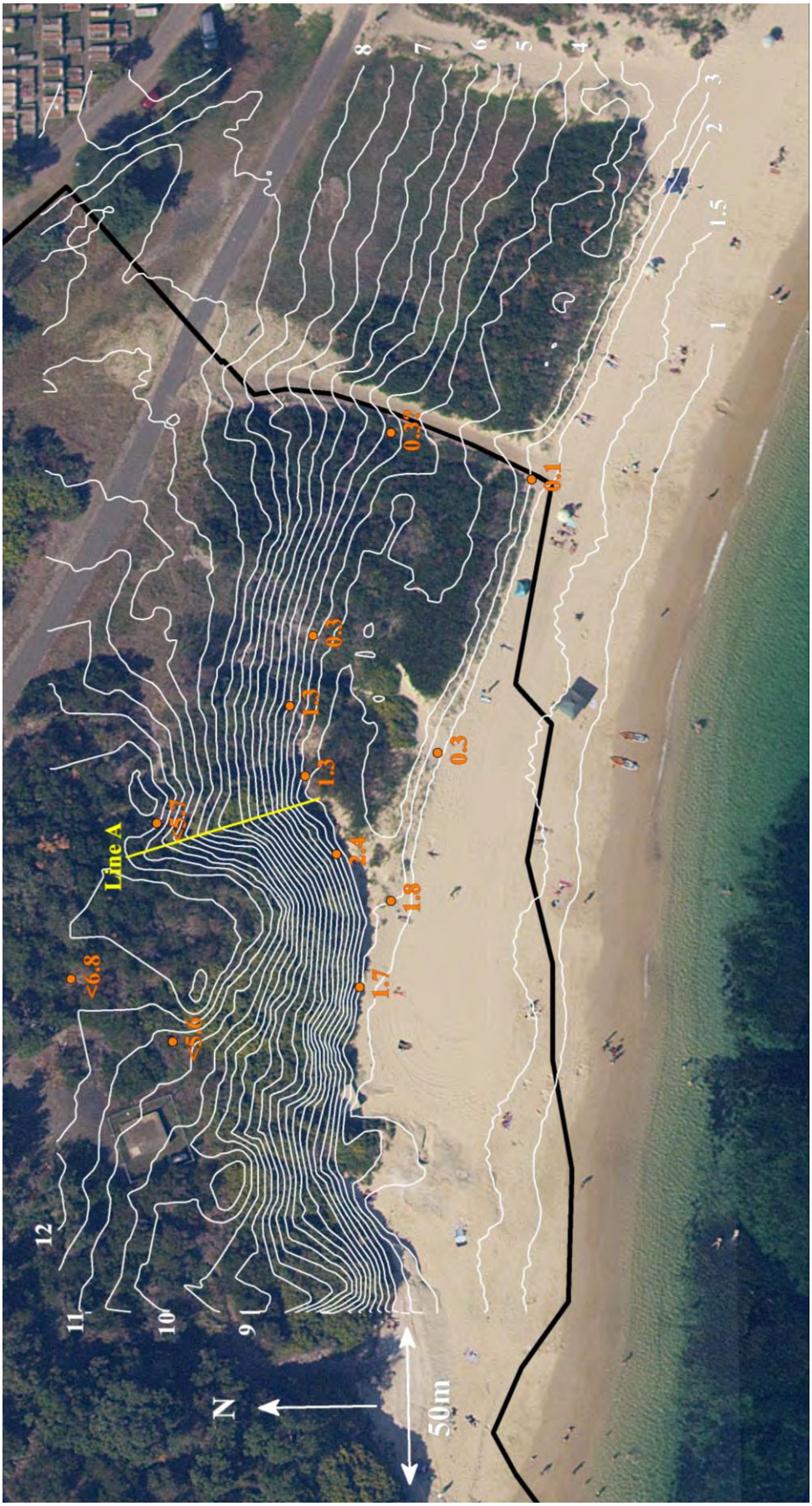

Figure 2: LiDAR contours (to AHD at 0.5m intervals) and DCP test results (inferred rock levels to AHD in orange), with kink in contours and change in vegetation evident at Line A

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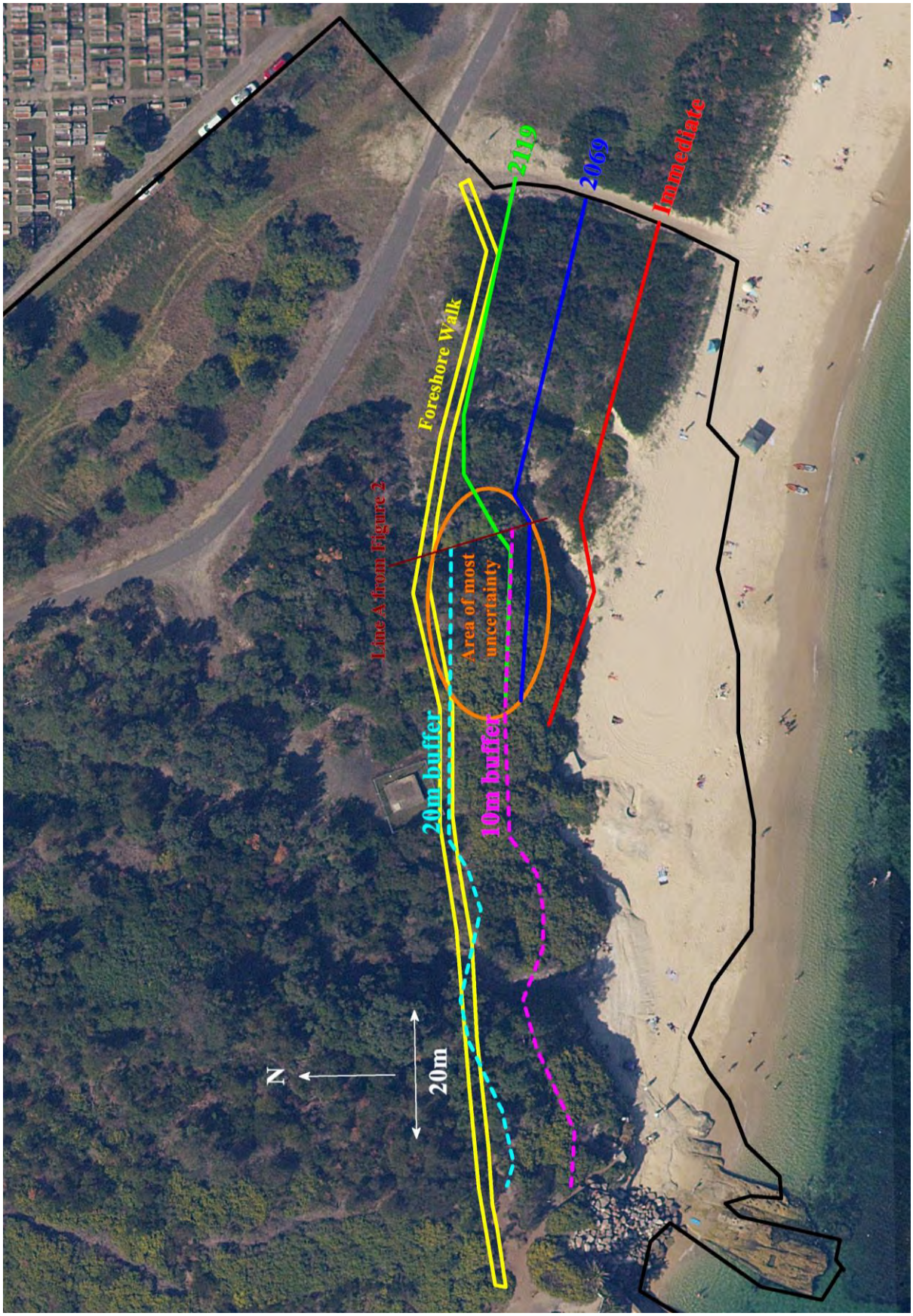

Figure 3: Approximate Immediate, 50 year (at 2069) and 100 year (at 2119) Hazard Lines (at landward edge of Zone of Slope Adjustment), and 10m and 20m groundwater buffer locations, at eastern end of proposed development site (with seaward edge of proposed Foreshore Walk, and area of most uncertainty in hazard definition, shown)

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It is evident in Figure 3 that west of Line A the groundwater buffers govern the required development setbacks, while east of Line A the coastal hazard lines govern. The proposed Foreshore Walk is depicted in Figure 3, which is the most seaward part of the cemetery development (that is, grave sites and ash interments are located landward of the Foreshore Walk). The Foreshore Walk (and hence cemetery development) is located landward of the 2119 Hazard Line, and the cemetery development is landward of the 20m groundwater buffer. The location of the Foreshore Walk and cemetery development is thus considered to be acceptable from a coastal engineering perspective.

5. REVIEW OF REVISED MASTER PLAN FROM COASTAL ENGINEERING PERSPECTIVE

A revised Concept Master Plan (Issue C, dated 26 May 2019) for the proposed cemetery, developed by Mathew Higginson Landscape Architecture, is depicted in Figure 4.

Figure 4: Revised Concept Master Plan prepared by Mathew Higginson Landscape Architecture

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Key features relevant to coastal hazards in Figure 4 are listed below:

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

Comments on these key Master Plan features from a coastal engineering perspective are provided below. Comments on Items 5, 7 and 8 are repeated verbatim from the 2018 Horton Coastal Engineering report, while the comments on Item 6 reflect the revised Master Plan.

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 Walk), it was noted above that its position is considered to be acceptable from a coastal engineering perspective. The Foreshore Walk is generally located above 9m AHD, so is at an acceptably low risk of damage and loss of serviceability from oceanic inundation and wave runup for an acceptably rare storm over a planning period of at least 100 years.

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.

6. CONCLUSIONS

Horton Coastal Engineering prepared a coastal engineering report relating to a Bumbora Point Cemetery Planning Proposal in 2018. Since completion of this report, BMT has undertaken a review, and an additional geotechnical investigation has been undertaken. The BMT review has been responded to, and the coastal hazard definition has been updated to take account of this additional geotechnical information and add a 100 year hazard line, as has been set out herein.

The Foreshore Walk (and hence cemetery development) is located landward of the 2119 Hazard Line, and the cemetery development is landward of the 20m groundwater buffer. The

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location of the Foreshore Walk and cemetery development is thus considered to be acceptable from a coastal engineering perspective.

7. REFERENCES

BMT (2018), *Peer Review of Coastal Engineering Report, Bumborah Point, Response to RFQ*, 23 August

Department of Land and Water Conservation [DLWC] (2001), *Coastal Dune Management: A Manual of Coastal Dune Management and Rehabilitation Techniques*, Coastal Unit, DLWC Newcastle, October 2001, ISBN 0-7347-5202-4

Douglas Partners (2018), *Investigation Summary Report, Client: Southern Metropolitan Cemeteries Trust, Project: Proposed Extension of Existing Cemetery, Address: Prince of Wales Drive, Matraville*, Project 86050.02, 5 December, R.001.Rev0

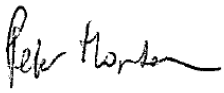
Louis, Simon; Couriel, Ed; Lewis, Gallen; Glatz, Matthieu; Kulmar, Mark; Golding, Jane and David Hanslow (2016), "NSW East Coast Low Event – 3 to 7 June 2016, Weather, Wave and Water Level Matters", *NSW Coastal Conference*, Coffs Harbour, November

Nielsen, Peter and David J Hanslow (1991), "Wave Runup Distributions on Natural Beaches", *Journal of Coastal Research*, Volume 7, No. 4, pp. 1139-1152

8. 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
HORTON COASTAL ENGINEERING PTY LTD



Peter Horton
Director and Principal Coastal Engineer

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Douglas Partners
Geotechnics / Environment / Groundwater

Investigation Summary Report

Client	Southern Metropolitan Cemeteries Trust	**Project No.**	86050.02
Project	Proposed Extension of Existing Cemetery	**Date**	5 December 2018
Address	Prince of Wales Drive, Matraville	**Doc No.**	R.001.Rev0

Proposed Development: An extension of the existing Eastern Surburbs Memorial Park is proposed towards the southwest, to Bumborah Point.

Scope of Investigation: Information is sought on the estimated rock levels in the area adjoining (upslope of) Yarra Bay Beach, to assist with modelling of likely coastal recession, for planning purposes. This is referred to as the "subject area" within this report.

Some additional comment is provided in relation to the proposed buffer zone for groundwater, as discussed in DP Report 86050.00.

Description of Site: The subject area adjoining the beach rises gently to the north over bare sand and some grasses (RL 2 to RL 3 m), then steeply up vegetated slopes to gently sloping ground (RL 8 to RL 12 m) that rises towards the west. The steep slopes are generally vegetated with low shrubs and grasses.

Exposed rock outcrops are present west of the subject area, while areas to the east are outside of the proposed extension. Sandy soils appear present within the subject area.

Regional Geology: Reference to the 1:100 000 Sydney Geological Series Sheet indicates that the subject area is on the boundary between Hawkesbury Sandstone and Quaternary Age soils, at the western and eastern sides of the subject area, respectively.

Background: Previous desktop assessment of the greater site area has been reported in Douglas Partners Pty Ltd's (DP's) Report 86050.00, and preliminary geotechnical investigation of the greater site area in DP Report 86050.01. The results of the previous investigation are relatively consistent with the regional mapping, but do not extend to the subject area. Reference should be made to those previous reports for further details.

Field Work Methods: A grid of potential dynamic cone penetrometer (DCP) locations was nominated in the subject area, and selected DCPs were undertaken on 15 October 2018 and 29 November 2018. DCP test methods are described in the attached notes. Twelve (12) dynamic cone penetrometer (DCP) tests were undertaken to depth of up to 6.15 m using an extendable DCP. DCPs were used instead of blunt-ended penetrometers (ie Perth Sand Penetrometers) in order to more clearly identify hard ground conditions.

On 10 October priority was given to test locations that may indicate rock levels around a 1 m AHD to -1 m AHD interval. Some test locations were rejected or moved due to proximity to services and/or access limitations due to vegetation.

Additional DCPs were undertaken on 29 November 2018 to provide additional information on rock levels, in an area of interest following the initial recession modelling.

The initial proposed test grid, together with the final test locations, is shown in the attached Drawing 1.

Test locations and ground surface levels for the final test locations were determined using differential GPS methods, with accuracy typically expected to be within 0.1 m.

Field Work Results: The results of the DCPs are given on the attached Dynamic Cone Penetrometer Results page.

Bouncing refusal was achieved at DCP 7, 7A, 8, 9, 11, 12, 13 and 15, while refusal (excessive blow counts) was encountered at DCP 10 and 11, and DCP 1 and 2 reached the limit of the testing equipment.

The test locations, and ground surface level to AHD from dGPS survey are summarised in Table 1, and shown in Drawing 1, attached.

Table 1: Test Locations

DCP	Easting (m)	Northing (m)	Surface Level (m AHD)	Limit of testing (m AHD)
1	336043	6239094	13.0	6.8
2	336063	6239083	11.8	5.7
6	336035	6239081	10.9	5.6
7	336059	6239060	3.1	2.4*
7A	336069	6239064	2.7	1.3*
8	336078	6239066	4.8	1.3*
9	336087	6239063	3.9	0.3*
10	336113	6239053	4.6	0.3
11	336042	6239057	2.5	1.7*
12	336053	6239053	2.5	1.8*
13	336072	6239047	2.8	0.3*
15	336107	6239035	2.7	0.1*

* Bouncing refusal

Comments:

Bedrock: It is considered highly likely that rock is at the level of bouncing refusal of the DCP tests, where encountered. Bouncing refusal may occur, however, due to very dense or iron-cemented layers within sand, or by other obstructions (eg unmarked services), although such conditions were not indicated by previous testing at the site, nor by the variability in level of DCP refusal, and are considered relatively unlikely.

The relatively sudden refusal at DCP 10 may possibly be associated with weathered bedrock or clay above bedrock, but this cannot be confirmed. At the remaining test locations (DCP1, 2 and 6) the test results are considered to be consistent with a soil profile.

The resulting levels of inferred bedrock are summarised in Drawing 1, and appear to suggest that a buried rock platform may be present at approximately 0.3 m AHD below the eastern side of the subject area, stepping up towards the west

It would be appropriate to review these results during the development of a geotechnical testing regime for future stages of development.

Groundwater: A minimum setback of burials from the sandstone clifflines of 20 m was suggested in DP's Report 86050.00, in order to allow space for natural attenuation of chemical and nutrient loading, and the subsurface measures that may be required to support this. This buffer is largely based on research on body interments.

While the available information on ash interments are limited, the chemical composition of ash from cremations is significantly different from that of bodies. Therefore, some impingement of ash interments into this buffer zone may be acceptable, although appropriate earthworks for management of groundwater may still be required. It is recommended that a 10 m minimum buffer be maintained for this purpose, and that interments in the 20 m to 10 m zone be placed at no more than 5 per standard grave space.

Limitations

Douglas Partners (DP) has prepared this summary report for this project at Prince of Wales Drive, Matraville in accordance with DP's proposal dated 20 September 2018 and acceptance received from Southern Metropolitan Cemeteries Trust c/- Kate Ryan of Urbis on 27 September 2018. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Southern Metropolitan Cemeteries Trust for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions

across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Reviewed by

Sally Peacock
Geotechnical Engineer/Associate

Michael J Thom
Principal

Attachments: Notes About This Report
Results of Dynamic Penetrometer Tests
Drawing 1

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

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Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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Geotechnics / Environment / Groundwater

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Results of Dynamic Penetrometer Tests

Client Southern Metropolitan Cemeteries Trust
Project Proposed Extension of Existing Cemetery
Location Prince of Wales Drive, Matraville

Project No. 86050.02
Date 15.10.2018
Page No. 1 of 3

Test Location	8	9	10	12	13	15				
RL of Test (AHD)	4.8	3.9	4.6	2.6	2.8	2.6				
Depth (m)	**Penetration Resistance** Blows/150 mm									
0 - 0.15	1	0	0	0	0	1				
0.15 - 0.30	0	1	1	2	1	1				
0.30 - 0.45	1	1	1	1	2	6				
0.45 - 0.60	2	0	2	4	4	9				
0.60 - 0.75	2	1	3	10/110	4	7				
0.75 - 0.90	3	2	2	B	3	7				
0.90 - 1.05	4	3	4		3	9				
1.05 - 1.20	2	4	4		2	9				
1.20 - 1.35	2	3	3		1	10				
1.35 - 1.50	2	5	3		2	9				
1.50 - 1.65	3	6	3		2	9				
1.65 - 1.80	2	3	3		5	9				
1.80 - 1.95	2	4	4		5	19				
1.95 - 2.10	2	7	5		5	21				
2.10 - 2.25	3	7	6		7	21				
2.25 - 2.40	2	5	4		15	19				
2.40 - 2.55	3	3	5		10/50	20/140				
2.55 - 2.70	4	5	3		B	B				
2.70 - 2.85	4	7	4							
2.85 - 3.00	4	12	3							
3.00 - 3.15	3	17	4							
3.15 - 3.30	4	18	8							
3.30 - 3.45	4	25	7							
3.45 - 3.60	10/50,B	25/130,B	7 cont							

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Sand Penetrometer ☐

Tested By RMM
Checked By SCP

Remarks R=Refusal, B=Bouncing, 24/110 indicates 25 blows for 110 mm penetration,
cont = continues page 3

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Results of Dynamic Penetrometer Tests

Client Southern Metropolitan Cemeteries Trust
Project Proposed Extension of Existing Cemetery
Location Prince of Wales Drive, Matraville

Project No. 86050.02
Date 29.11.2018
Page No. 2 of 3

Test Location	1	2	6	7	7a	11				
RL of Test (AHD)	13.0	11.8	10.9	3.1	2.7	2.5				
Depth (m)	**Penetration Resistance** Blows/150 mm									
0 - 0.15	0	1	1	1	1	1				
0.15 - 0.30	1	1	1	1	2	1				
0.30 - 0.45	2	1	1	0	1	2				
0.45 - 0.60	4	1	3	1	1	2				
0.60 - 0.75	6	1	2	0/100	2	2				
0.75 - 0.90	4	1	3	B	3	2/50				
0.90 - 1.05	2	2	4		3	B				
1.05 - 1.20	2	2	3		3					
1.20 - 1.35	4	2	3		3					
1.35 - 1.50	4	2	4		4/90					
1.50 - 1.65	4	3	3		B					
1.65 - 1.80	4	3	5							
1.80 - 1.95	4	3	6							
1.95 - 2.10	3	4	6							
2.10 - 2.25	4	2	5							
2.25 - 2.40	5	2	5							
2.40 - 2.55	5	3	6							
2.55 - 2.70	9	3	6							
2.70 - 2.85	15	2	7							
2.85 - 3.00	7	4	7							
3.00 - 3.15	8	3	8							
3.15 - 3.30	9	3	10							
3.30 - 3.45	10	4	10							
3.45 - 3.60	9 cont	4 cont	10 cont							

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Sand Penetrometer ☐

Tested By RMM
Checked By SCP

Remarks R=Refusal, B=Bouncing, 24/110 indicates 25 blows for 110 mm penetration
cont = continues page 3

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Results of Dynamic Penetrometer Tests

Client Southern Metropolitan Cemeteries Trust

Project Proposed Extension of Existing Cemetery

Location Prince of Wales Drive, Matraville

Project No. 86050.02

Date 15.10-29.11.2018

Page No. 3 of 3

Test Location	1	2	6		10					
RL of Test (AHD)	13.0	11.8	10.9		4.6					
Depth (m)	**Penetration Resistance** Blows/3750 mm									
3.6 - 3.75	8	4	13		10					
3.75 - 3.90	8	4	14		8					
3.90 - 4.05	7	5	16		16					
4.05 - 4.20	8	6	20		26					
4.20 - 4.35	10	6	21		25/120					
4.35 - 4.50	13	5	22		R					
4.50 - 4.65	14	6	20							
4.65 - 4.80	15	12	21							
4.80 - 4.95	15	16	22							
4.95 - 5.10	16	11	23							
5.10 - 5.25	15	11	25/140							
5.25 - 5.40	17	11	R							
5.40 - 5.55	17	10								
5.55 - 5.70	17	8								
5.70 - 5.85	20	8								
5.85 - 6.00	21	11								
6.00 - 6.15	22	10								
6.15 - 6.30	D	D								
6.30 - 6.45										
6.45 - 6.60										
6.60 - 6.75										
6.75 - 6.90										
6.90 - 7.05										
7.05 - 7.20										

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Sand Penetrometer ☐

Tested By RMM

Checked By SCP

Remarks R=Refusal, B=Bouncing, 24/110 indicates 25 blows for 110 mm penetration

Legend

- DCP Test Location
- Proposed Test Grid
- 3.0 Ground Surface Level (m AHD)
- 0.3 Inferred Rock Level (m AHD)

Note: Base image from Nearmap.com dated 18 August 2018

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT:	Southern Metropolitan Cemeteries Trust		
OFFICE:	Sydney	DRAWN BY:	SCP
SCALE:	As shown	DATE:	3.12.2018

TITLE: **Test Location and Summary Results**
Proposed Extension of Existing Cemetery
Prince of Wales Drive, Matraville

PROJECT No:	86050.02
DRAWING No:	1
REVISION:	0

Horton Coastal Engineering
Coastal & Water Consulting

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3 July 2018

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.

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- 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.

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

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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

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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

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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

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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.

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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.

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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.

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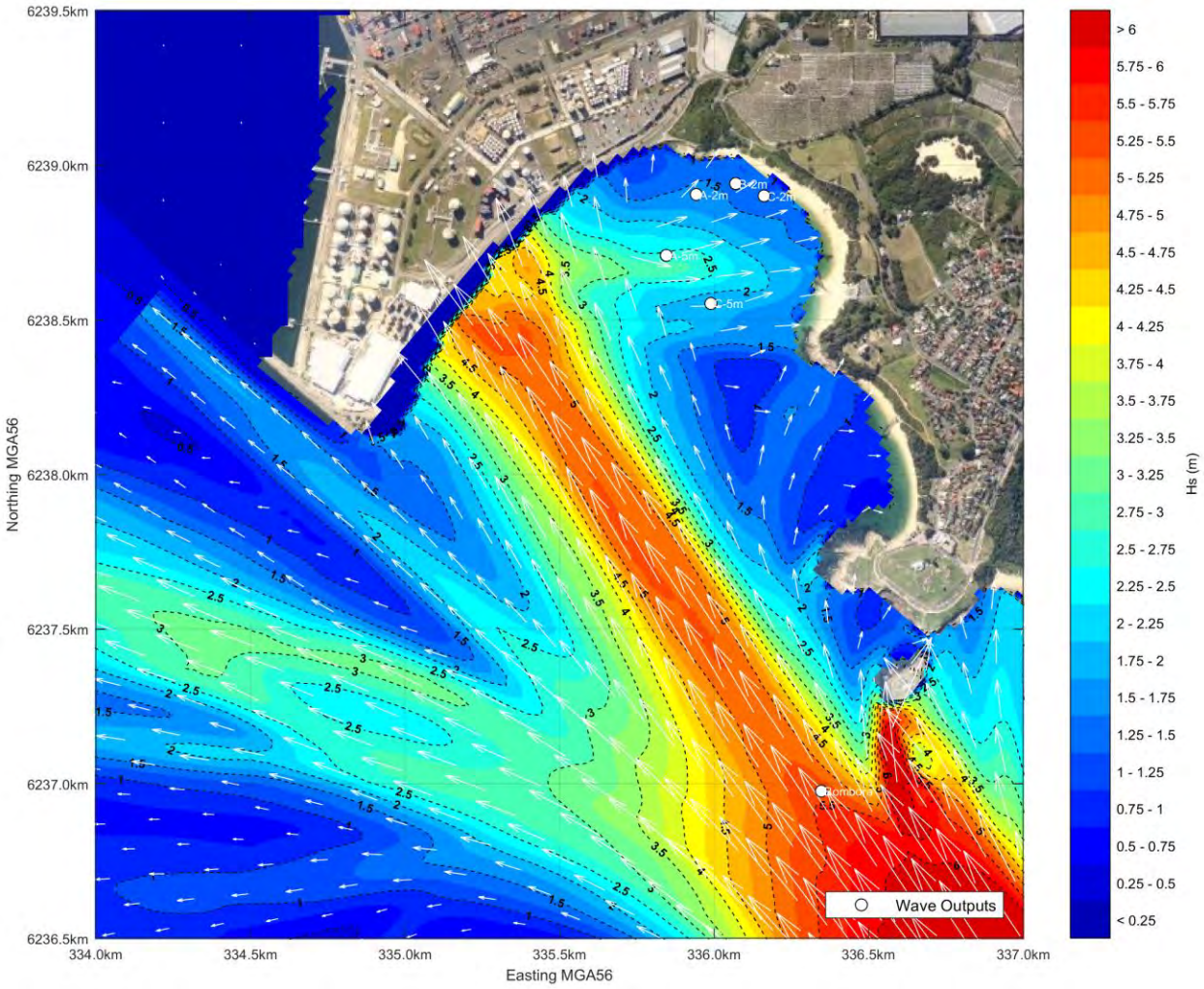

Figure 10: Contour plots of significant wave height and vectors of peak wave direction for SSE offshore wave direction

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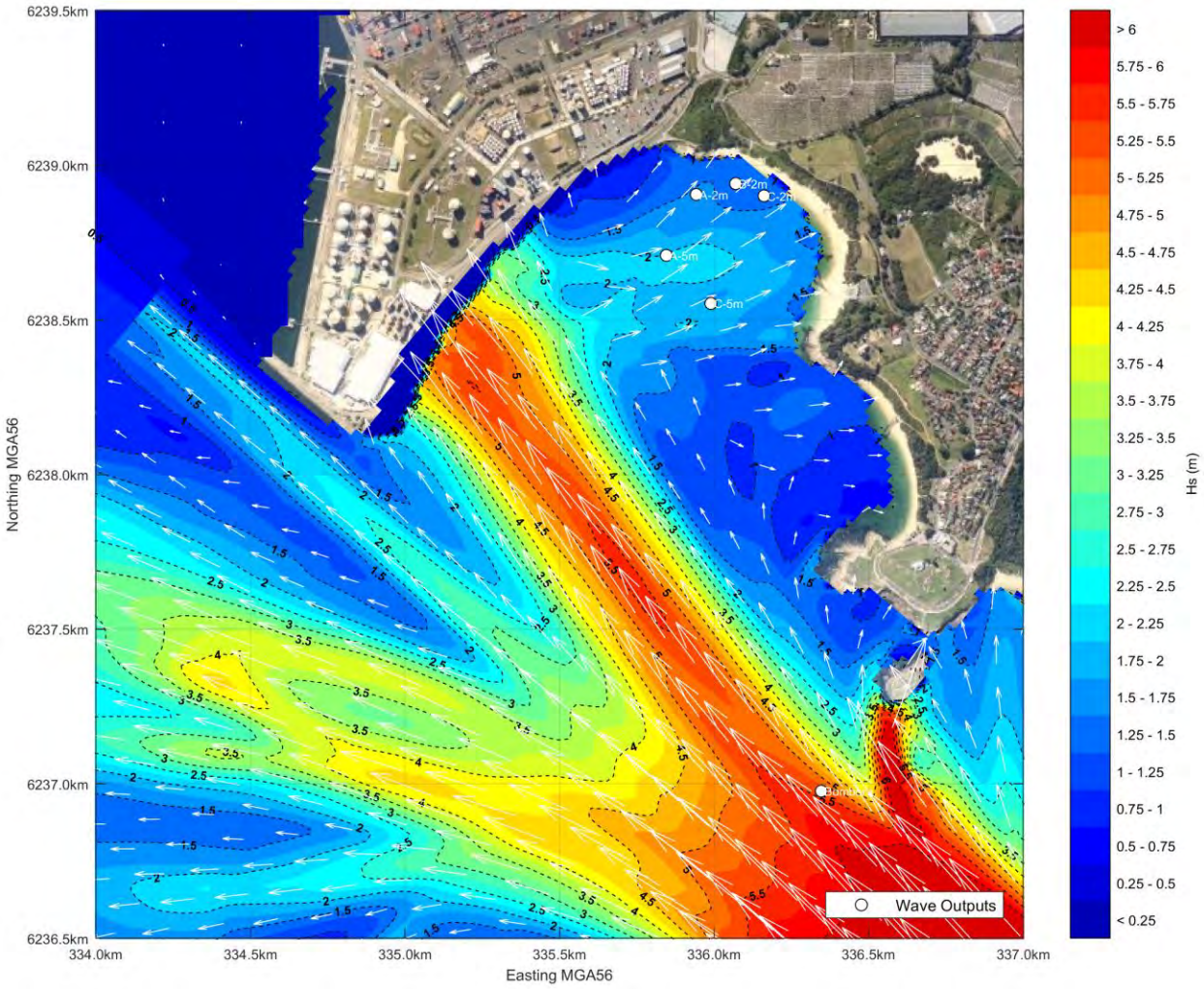

Figure 11: Contour plots of significant wave height and vectors of peak wave direction for SE offshore wave direction

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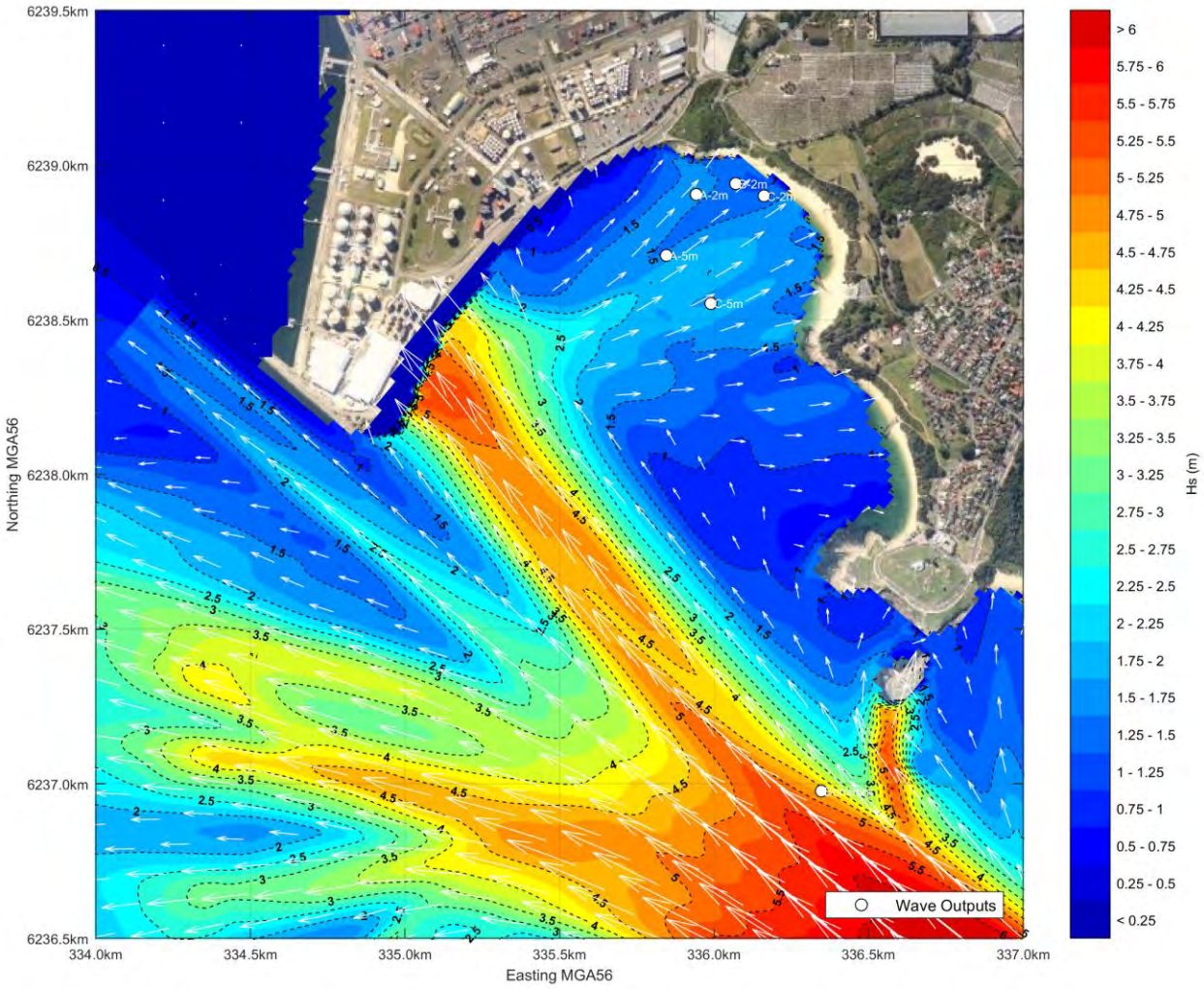

Figure 12: Contour plots of significant wave height and vectors of peak wave direction for ESE offshore wave direction

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.

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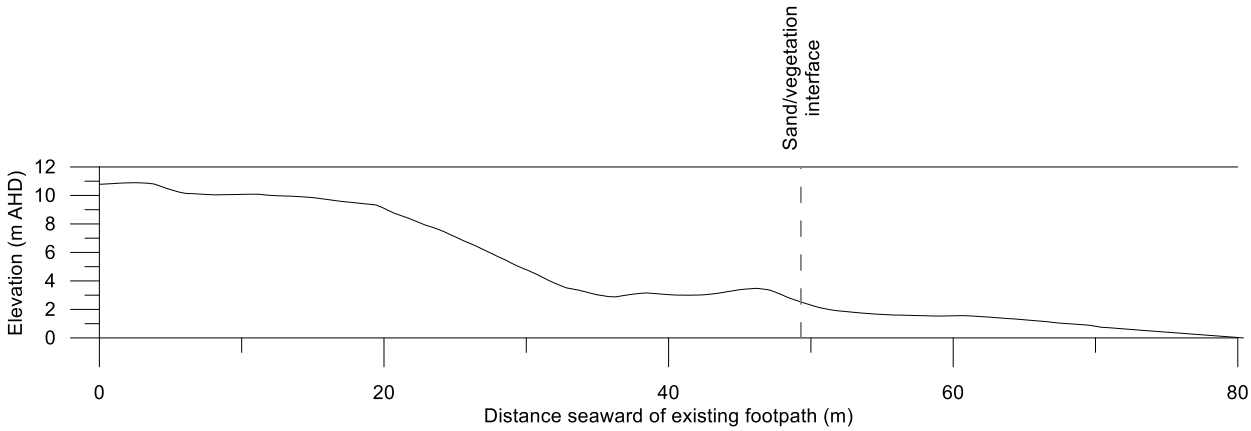

Figure 13: Beach profile at subject site

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

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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.

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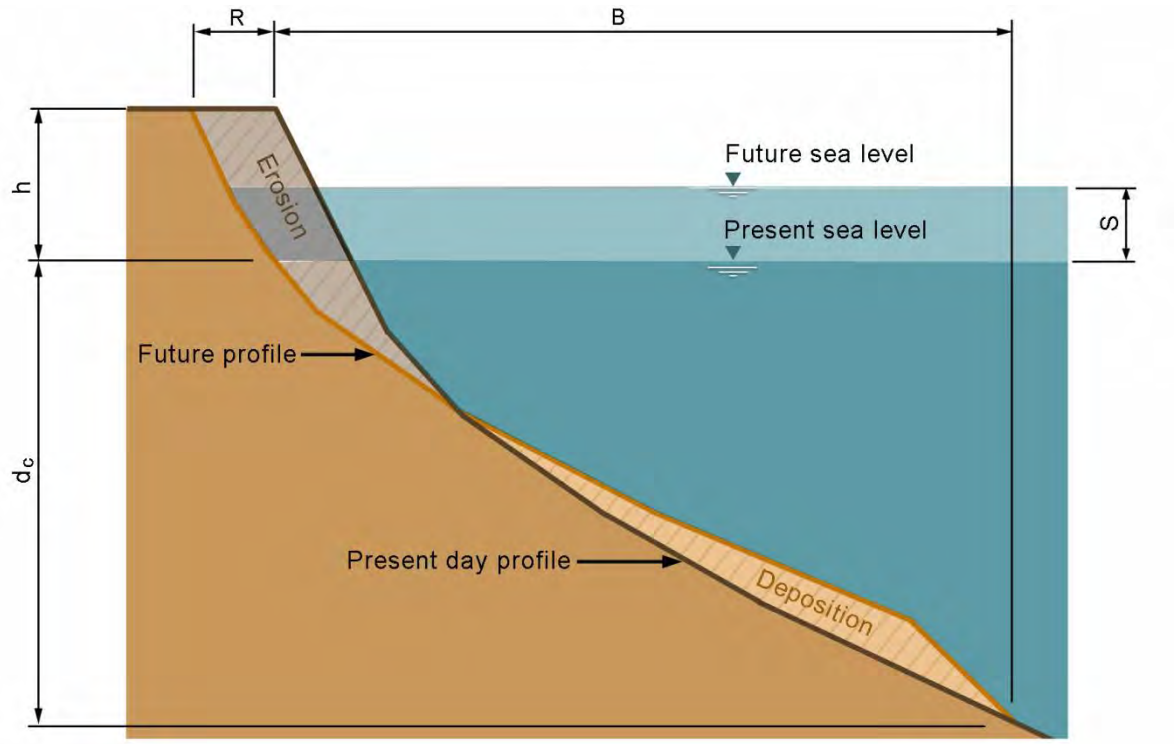

Figure 14: Illustration of variables in the Bruun Rule

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.

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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.

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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

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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

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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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