Horton Coastal Engineering
Coastal & Water Consulting

Coastal Engineering Report for Construction of Upgraded Coastal Protection Works at 1174-1182 Pittwater Road Narrabeen

Prepared by Horton Coastal Engineering Pty Ltd

for the Owners of 1174-1182 Pittwater Road Narrabeen

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1. INTRODUCTION

The report herein has been prepared as part of a Development Application to Northern Beaches Council for construction of upgraded coastal protection works at 1174-1182 Pittwater Road Narrabeen.

In the *Collaroy – Narrabeen Beach Coastal Protection Works Design Specifications* (hereafter denoted as "the *Specifications*") prepared by Northern Beaches Council in 2016, it is stated that:

> "A Basis of Design (BoD) statement shall be prepared as part of the seawall design process and submitted with the Development Application. The BoD shall clearly state all of the design factors, assumptions and qualifications adopted in the design, including specific reference to the above design criteria".

The report herein has been formulated to meet this requirement.

The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional Coastal Engineer with 27 years of coastal engineering experience. He has postgraduate qualifications in coastal engineering, and is a Member of Engineers Australia (MIEAust) and a 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.

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. GENERAL DESCRIPTION OF PROPOSED DESIGN AND MATERIALS

As depicted on the Drawings, the proposed coastal protection works design comprises a reinforced concrete wall supported on continuous flight auger (reinforced concrete/grout) contiguous piles with infill concrete/grout plug piles[1]. Anchors attached to the wall (and permanently buried landward of it) have been designed to provide support for the wall and piling at times of beach erosion when sand levels lower on the seaward side of the wall, with two anchoring options shown on the Drawings (steel screw pile anchors at 3m centres[2], or deadman continuous flight auger concrete piles at 4m or 5m centres with a connecting concrete beam).

The design was prepared as an integrated coastal, geotechnical and structural engineering investigation, with iterative input from these three disciplines to produce a robust solution. For example, the design took into account coastal engineering issues (beach scour, elevated water levels, waves), geotechnical engineering issues (subsurface conditions, global stability, analysis to determine pile embedment and anchor capacity) and structural engineering issues (bending moments, shear forces, deflections, strength, serviceability and durability) leading to concrete member and anchor concept design.

The contiguous/plug piles have been designed as a complete and permanent barrier to soil migration through the wall. A wave return (concrete face that slopes seaward and directs waves seaward) has been provided at the top of the concrete wall to reduce wave overtopping of the wall. This wave return extends 0.5m seaward of the main face of the wall.

The existing rock revetment coastal protection works at the subject properties will be removed as part of the proposed works. These rock works currently extend up to about 8m seaward of the subject properties onto the public beach (Crown Land) at 1174, 1176 and 1178, and up to about 3m onto the public beach at 1180 and 1182 Pittwater Road.

The proposed works are to be located entirely on private property, with the main face of the concrete wall located immediately landward of the seaward property boundaries of the 5 subject properties (as per the Drawings, stairs are also recessed into the wall at various locations). For 100% of the time, the public beach seaward of the subject properties will be theoretically[3] accessible by the public (that is, the proposed works do not restrict public access at any time as they are located entirely on private property). The proposed vertical wall alignment is coincident with a current rock boulder surface level of about 3m to 4m AHD at 1174 and 1176, 2m AHD at 1178, and 1m AHD at 1180 and 1182.

A minimum 4.5m setback landward of the wall has been adopted as a maintenance setback, and also to allow for dissipation of any wave overtopping of the wall. It is proposed that no future structures, except readily relocatable or removable structures that do not interrupt views, would be constructed seaward of this setback (as specified on the Drawings). The purpose of this setback is to enable clear passage of construction plant as required for future works maintenance. A maintenance setback of 5m to 6m was recommended in the

[1] As an alternative to plug piles, jet grout may be used to infill the gaps between the contiguous piles.
[2] Alternative anchoring setouts and types (eg grouted and stressed anchors) may also be used.
[3] Of course, after severe coastal storms when there is beach erosion and a lowering of beach elevations, there may be no dry public width available, as occurs at present after severe storms. The proposed works do not affect this process, with any additional scour hole associated with the vertical wall expected to be relatively quickly infilled by sand as part of natural sediment transport processes (sand that erodes off the subaerial portion Collaroy-Narrabeen Beach in coastal storms remains in the beach system as it moves to offshore bars, returning to the subaerial beach as storm conditions subside).

Specifications, but this was developed in the context of a rock revetment rather than a vertical concrete wall. The available space for maintenance could be quickly increased, if required, with temporary removal of the fence above the seawall. Removal of the fence would increase the setback distance to 5.35m from the seaward property boundary (6.85m at stairs), and with rotation of an excavator beyond the boundary or over the stairs being possible, this would increase the available space for maintenance further. That stated, there may be no need for maintenance to be undertaken from landward of the wall, with any wall maintenance (if required) on the seaward face of the wall undertaken some duration after storms when the beach had partially recovered and was accessible.

Concrete beach access stairs have been provided at all properties, recessed into the wall and shore-parallel. There are individual stairs at 1174, and shared stairs with the upper landing at the common property boundaries at 1176/1178 and 1180/1182. This would essentially provide permanent beach access at the properties, with the stairs integrated into the wall. The stairs do not project seaward of the seaward face of the wall, and are thus within the subject private properties.

3. PROPOSED DESIGN LIFE OF PROTECTION WORKS

A design life of 100 years has been adopted for the proposed protection works (that is, at the year 2120). As outlined in Horton et al (2014) and Horton and Britton (2015), a 60 year design life is considered to be appropriate in relation to beachfront development (that relies on the protection works for protection against erosion/recession over the design life) as:

- it is consistent with Australian Standards applying to the residential development landward of the protection works:
  - in *AS 3600-2018 (Concrete structures)*, a 50 years ± 20% design life[4] (that is, 40 years to 60 years) is used in devising durability requirements for concrete structures;
  - in *AS 2870-2011 (Residential slabs and footings)*, for design purposes the life of a structure is taken to be 50 years for residential slabs and footings construction; and
  - in *AS 4678-2002 (Earth-retaining structures),* the design life for earth-retaining structures (structures required to retain soil, rock and other materials) is noted as 60 years for river and marine structures and residential dwellings.
- the cost of new residential development is amortised for tax purposes over 40 years based on Subdivision 43-25 of the *Income Tax Assessment Act 1997*;
- a design life of at least 50 years would be considered to be reasonable for permanent structures used by people (AGS, 2007a, b); and
- this design life of 60 years was adopted in the gazetted *Collaroy-Narrabeen Beach and Fishermans Beach Coastal Zone Management Plan* (CZMP).

A minimum 60-year design life was adopted in the *Specifications*. As noted therein, this design life recognises, among other things, that redevelopment of beachfront properties typically occurs within such a period.

The proposed design life of 100 years is thus conservative, and exceeds the minimum requirement in the *Specifications*. The subject property owners requested a 100 year design life (rather than 60 years) as they considered that this would provide greater long term economic benefits.

[4] Period for which a structure or a structural member is intended to remain fit for use for its designed purpose with maintenance.

4. APPLICATION OF 100 YEAR DESIGN LIFE TO CONCRETE AND ANCHOR DESIGN

A 100 year design life is achievable for the concrete wall and concrete piling (including the deadman anchoring if adopted). As noted in Section 3, *AS 3600* applies to structures with a design life of 40 to 60 years, while *AS 5100* (although for bridge design) can be used to provide guidance on extending the design life of concrete structures to 100 years. For ≥ 50MPa concrete, as would be applied, the required cover for an Exposure Classification of C2 (in the tidal or splash zone) from *AS 3600* and *AS 5100*, is 65mm and 80mm respectively (the latter applying to ≥ 55MPa concrete). The proposed wall would only occasionally be in the tidal and splash zone, and would generally be in the spray zone (Exposure Classification of C1) from *AS 3600* and *AS 5100*, for which the required cover is 50mm and 70mm respectively. Nonetheless, a cover of 80mm would be adopted, thus meeting the requirement for a 100 year concrete life in the tidal or splash zone. A characteristic strength of 55 MPa would be adopted, consistent with the requirements in AS 5100.5.

Other features that would be adopted to ensure a minimum 100 year life for the concrete would include specification of workmanship standards to exceed the base level performance assumed by the deemed to satisfy the provisions of the relevant Australian Standards. Such items include concrete cover and tolerance, standard of formwork and vibration, use of non-ferrous bar chairs, and regular quality inspections.

A 100 year design life is achievable for the steel screw pile anchoring (and alternative anchor types), and a minimum 100 year life has been specified on the Drawings. Features that would be adopted to ensure a minimum 100 year life for the anchors (if the steel screw pile anchoring option was adopted) would include assessment of the corrosive environment that the anchors would be located in.

Design life for anchors such as screw piles is provided through an assessment of the corrosion rates for items in ground. The elements making up the anchors (pipes, helixes and welds) are then increased in thickness and detailed in such a way as to allow for the corrosion to happen at the predicted rate while ensuring adequate material remains to act as a serviceable anchor after the nominated design life period. Additional means of protection such as coatings (galvanic) or grout filling pipes are also available for extending the life of ground anchors.

5. SUBSURFACE CONDITIONS

JK Geotechnics (2016) found inferred cemented sand at elevations of 0.7m to 1.4m AHD (mean and median of 1.0m AHD) at 8 Dynamic Cone Penetrometer (DCP) test locations at each of the properties 1168 Pittwater Road Collaroy (located immediately south of Wetherill Street), and 1172 (located immediately south of the subject properties), 1174 (two locations), 1176, 1178, 1180 and 1182 Pittwater Road Narrabeen. These 8 DCP test locations were about 10m to 13m landward of the seaward property boundaries, as shown in yellow in Figure 1.

A geotechnical investigation was completed by Jeffery & Katauskas (2000), which included boreholes at Wetherill Street (BH204) and Clarke Street (BH205), located about 12m and 5m respectively landward of the seaward property boundaries, as shown in blue in Figure 1. This indicated levels of the upper surface of the cemented sand layer of about 0.3m AHD at Wetherill Street and 0.9m AHD at Clarke Street. Note that the cemented bands continued below the upper surface of the cemented sand for a depth of about 4.0m at Wetherill Street, and 3.1m at Clarke Street, showing the significant vertical extent of the cemented sand layer.

Jeffery & Katauskas (2000) completed a test pit (TP120) seaward of 1178 Pittwater Road, at a cross-shore position about 5m seaward of the seaward property boundary. This indicated a cemented sand level of about -0.2m AHD. They also completed a test pit (TP107) seaward of 1168 Pittwater Road, at a cross-shore position about 8m seaward of the seaward property boundary. This indicated a cemented sand level of about -1.1m AHD. These two test pit locations are depicted in red in Figure 1.

Coffey Partners International (1998) completed a test pit seaward of 1174 Pittwater Road (TP4), at a cross-shore position about 6m seaward of the seaward property boundary. This indicated a cemented sand level of about -0.4m AHD. They also completed a test pit (TP5) seaward of 1164 Pittwater Road, at a cross-shore position at the seaward property boundary. This indicated a cemented sand level of about -0.1m AHD. These two test pit locations are depicted in green in Figure 1.

Comparing the test results depicted in Figure 1, it is evident that cemented sand levels dip moving seaward with an approximate slope down of 1:10 to 1:15 vertical:horizontal (V:H), or 5.7° to 3.8°, moving seaward. This means that cemented sand levels along the proposed wall alignment (that is, along the seaward property boundaries) are likely to be around 0m AHD (±0.3m), with a trend for cemented sand levels to also be higher moving north.

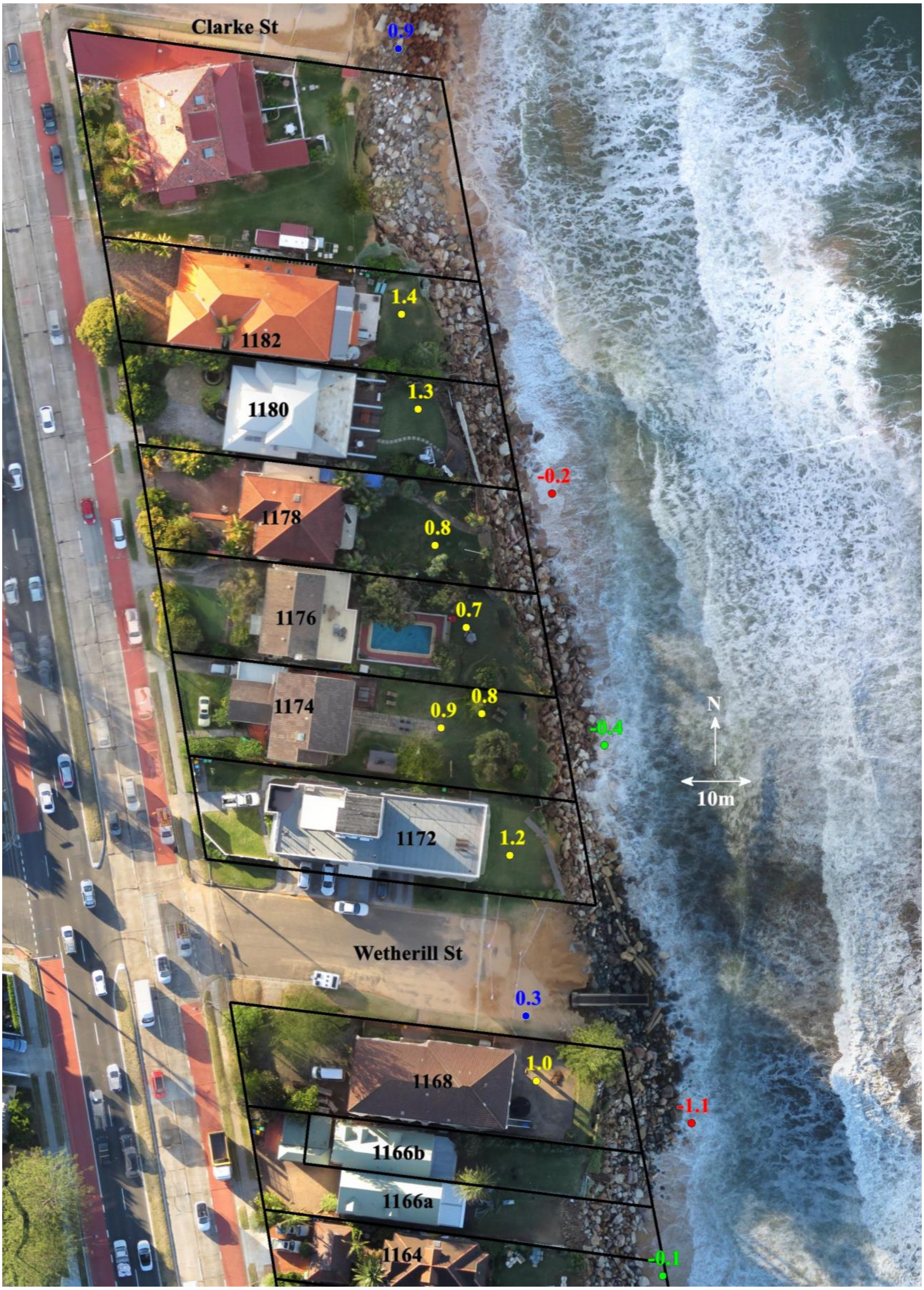

Figure 1: DCP test locations of JK Geotechnics (2016) in yellow, boreholes and test pits of Jeffery & Katauskas (2000) in blue and red respectively, and test pits of Coffey Partners International (1998) in green, with cemented sand levels (m AHD) shown (aerial photograph taken 8 June 2016)

6. BEACH SCOUR

A storm scour level of -1m AHD is typically adopted at NSW beaches. This is based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms (Carley et al, 2015).

For a rock structure, the toe level is based on the anticipated scour level. In the *Specifications*, a minimum toe level for protection works of -1m AHD was adopted, "although a higher toe level may be considered if there is evidence of an inerodible layer at a level above -1m AHD".

Carley et al (2015) noted that it is common practice for vertical seawalls on the open coast of eastern Australia to be designed for a beach scour level of -2m AHD (assuming an erodible sandy subsurface), although that does not consider the required design life of the seawall, which is factor affecting the potential scour magnitude. The longer the design life, the lower beach levels would reduce to due to long term recession, and the rarer the design storm event must be to achieve an acceptable risk level over that life (as a particular probability storm has a greater cumulative probability of occurring over a longer design life). The toe level of a vertical seawall must continue well below the scour level.

In Figure 2, the actual cemented sand level (assuming a 1:10 slope) at 1182 Pittwater Road (which is the critical design location with the highest crest level and closest proximity of a dwelling to the wall), and lowest cemented sand level over the 1174-1182 block, are depicted relative to historical beach profiles at the property. It is evident that cemented sand levels sit well (about 5m) below typical beach profiles, and about 1m below the scour level that occurred in the 2016 storm.

As long-term recession due to sea level rise is realised, scour levels may lower at a particular cross-shore position as the beach profile translates landward (given that beach levels essentially lower moving seaward). In the CZMP, a "best estimate" inverse slope of the active beach profile of 30 was adopted, which would cause 23.7m of long term recession due to sea level rise over the design life based on the Bruun Rule (using the adopted sea level rise value of 0.79m from Section 8.3), and about 1.0m of lowering assuming no restriction from cemented sand[5].

In Figure 3, the same information in Figure 2 is depicted, except that the historical beach profiles are translated landward by 23.7m and raised by 0.79m to account for projected long term recession due to sea level rise over the design life. It is evident that the receded profiles generally sit well (over 2m) above the cemented sand, including some of the lowest profiles that occurred in 1974, 1986, 1988, 1998 and 2018[6]. The lowest receded profile, in 2016, terminates well landward of the proposed seawall, and would be the only receded profile in Figure 3 that would be expected to have potentially exposed the cemented sand layer adjacent to the seawall. The other profiles in the historical record, when receded, generally sit about 3m to 5m above the cemented sand layer. Figure 3 thus indicates that long term recession due to sea level rise is unlikely to cause beach profiles to lower such that regular interaction with the cemented sand layer would occur. The cemented sand layer would only be expected to be exposed occasionally over the design life, in severe storm events.

[5] Based on 1.82m of lowering due to landward translation of the profile (with an average beach slope of 1:13 vertical:horizontal), and 0.79m of profile raising due to the sea level rise itself.
[6] The 1998 profile would eventually intersect with the cemented sand profile about 7m seaward of the proposed seawall if extrapolated at the same slope as the most seaward two points in the profile.

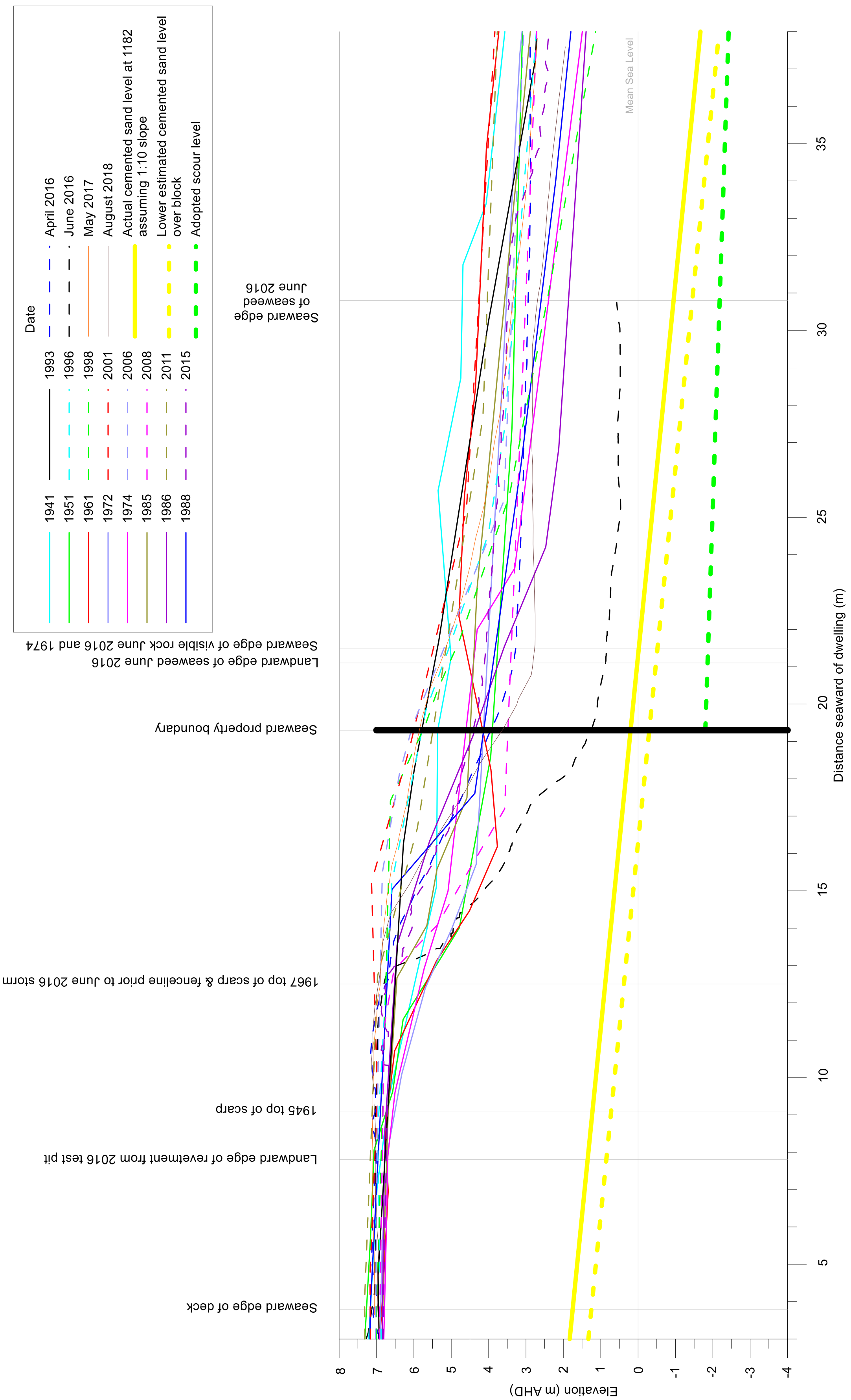

Figure 2: Adopted scour level in relation to cemented sand levels and historical beach profiles at 1182 Pittwater Road

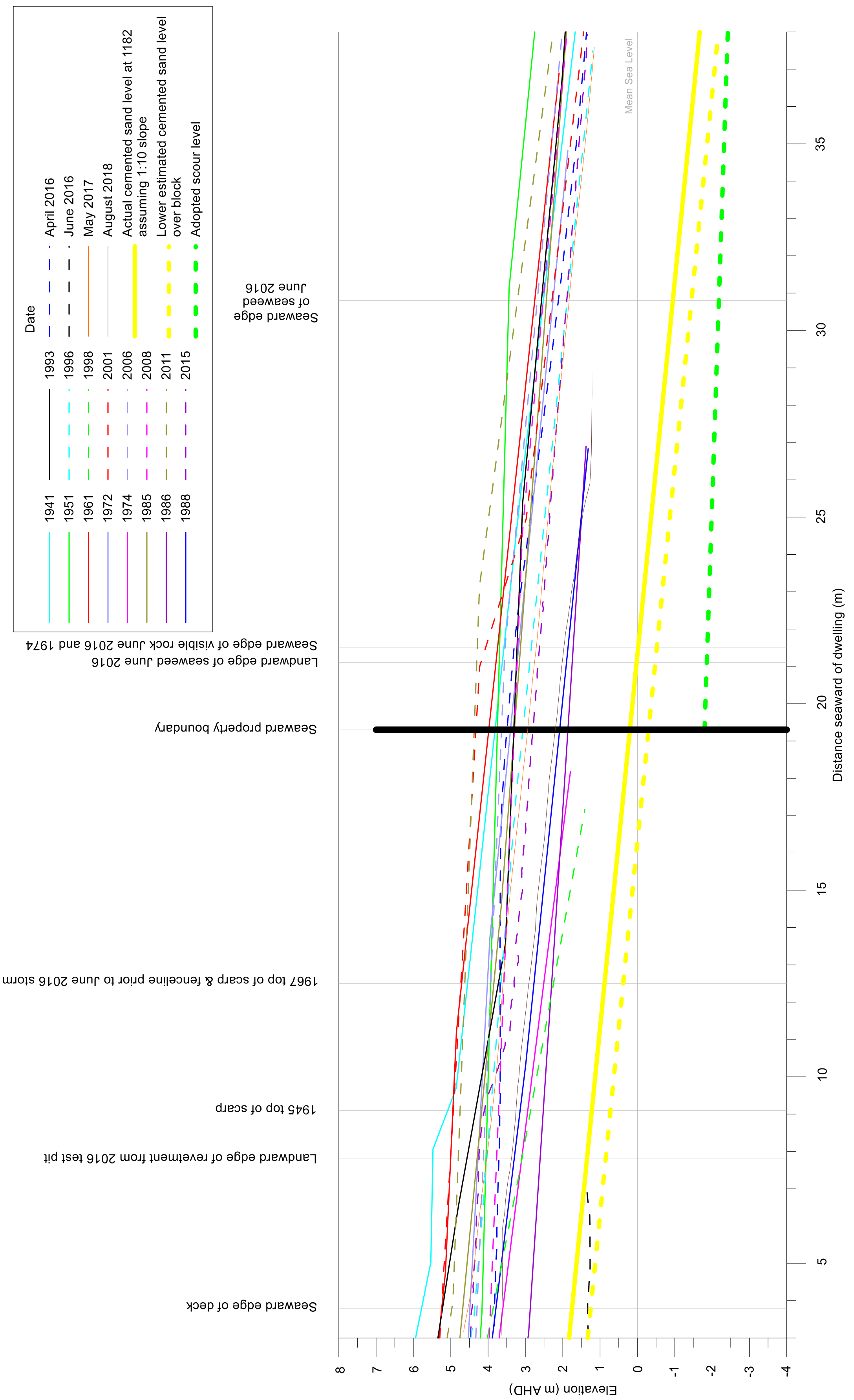

Figure 3: Adopted scour level in relation to cemented sand levels and historical beach profiles (receded over 100 years) at 1182 Pittwater Road

The adopted design scour level is depicted Figure 2 and Figure 3 as a thick green dashed line. The adopted scour level is -1.8m AHD at the seawall, sloping downwards at 1:30 (vertical:horizontal) moving seaward.

The level of -1.8m AHD was adopted cognisant that a rare (2,000 year Average Recurrence Interval [ARI]) scour event needed to be selected (see Section 7), and that such an event is beyond the observed wave events in the historical record and there are no reliable analytical means to calculate the scour for such an event. Using the terminology in the risk assessment methodology of the Australian Geomechanics Society (AGS, 2007a, b), a 2,000 year ARI (unlikely) event "might occur under very adverse circumstances over the design life". The adopted scour level would only be expected to occur under very adverse circumstances over the design life, thus consistent with this terminology.

The reasoning for the adopted -1.8m AHD scour level was as follows:

- there is a cemented sand level of 0m±0.3m AHD at the proposed seawall;
- taking the lowest level (-0.3m AHD) and applying 1.0m of lowering to allow for scour of the cemented sand in storms over the design life (including the potential for additional scour at the toe of the seawall), gives a level of -1.3m AHD; and
- applying a factor of safety of 1.5 on the lowering gives an additional lowering of 0.5m, and hence the adopted level of -1.8m AHD.

This is considered to be a highly unlikely level, as it is only 0.2m vertically above a typical design scour level of -2m AHD that is applied at a vertical seawall on fully erodible beaches, the cemented sand at this location is significantly less erodible than normal (non-cemented) beach sands, and long term recession over the design life is not expected to regularly expose the cemented sand.

When beaches scour in severe erosion events, they tend to flatten to be nearly horizontal. Pre-storm beach slopes in the order of 1:13 (vertical:horizontal) do not continue to apply when the back beach lowers towards the design scour level. The downward slope of 1:30 moving offshore for the scour level is considered to be a reasonable estimate for design.

Additional ground investigations to assess the extent and level of the cemented sand layer seaward of the proposed wall may lead to a revised (potentially less conservative) scour level being adopted for detailed design.

Analysis (JK Geotechnics, 2020) of the proposed design with the adopted scour level of -1.8m AHD sloping downwards at 1:30 moving offshore obtained a factor of safety exceeding 1.5 for both global stability and structural stability (with consideration of disturbing and balancing forces and moments). A Factor of Safety of 1.67 was obtained for global stability, and 1.65 for structural stability.

This exceeds the required 1.5 factor of safety in the *Specifications*, and shows that an adequate factor of safety is obtained if at least 1.5m of scour of the cemented sand layer occurred over the design life at the seawall. Such scour is highly unlikely to occur in practice. Also note that to give a factor of safety equal to 1.5, the scour level would be -2.0m AHD sloping downwards at 1:30 moving offshore.

To determine the scour level that could cause theoretical failure[7] of the adopted design, JK Geotechnics (2020) found that a factor of safety of just less than 1.0 was obtained for a scour level of -2.8m AHD and -4.9m AHD on the seaward side of the wall, considering structural stability (in the model WALLAP) and global stability (in the model SLOPE/W) respectively. With the structural stability (WALLAP) mode of failure governing with the scour level of -2.8m AHD, this indicates that there would need to be an additional 1m of scour than the design scour if the seawall was to fail from excessive scour, and at least 2.3m of cemented sand scour overall (and an average of 2.6m of cemented sand scour)[8]. Using the AGS (2007a, b) terminology, a 200,000 year ARI (barely credible) event is considered as “inconceivable or fanciful over the design life”. The adopted scour level is not really conceivable, thus consistent with this terminology, and this shows that there would be the expectation of some redundancy in the structure if the design scour level was exceeded.

Furthermore, as discussed in JK Geotechnics (2020), structural design has been undertaken assuming a conservative groundwater level difference of 4m between the landward and seaward sides of the wall at the time of maximum scour.

[7] Defined herein as the factor of safety being just less than 1.0. For example, when considering structural stability, where disturbing forces and moments are just more than balancing forces and moments acting on the piled wall.

[8] It is recognised that the factor of safety of just less than 1.0 for structural stability in WALLAP is only one potential mode of failure in one model. For example, as scour lowers, bending moments increase in the piles and loads on the anchors increase, which may be problematic. Therefore, failure of the seawall is more complex than just considering structural stability in WALLAP. That stated, the scour level of -2.8m AHD is considered to be indicative of the scour level for theoretical failure.

7. ADOPTED DESIGN PROBABILITY AND RISK USED IN THE DESIGN

In Australian Standard *AS 4997-2005*, *Guidelines for the design of maritime structures*, recommendations are given for the design wave height event to adopt for various design lives and types of structures. Normal maritime structures are considered to be Function Category 2, while "high property value or high risk to people" structures are considered to be Function Category 3.

In Table 1, the *AS 4997* recommended design wave height event Average Recurrence Interval (ARI) is presented for both Function Category 2 and Function Category 3, for two different design lives, namely 50 and 100 years respectively. A design life of 50 years is recommended in *AS 4997* for normal maritime structures, while a design life of 100 years or more is recommended for "special structures / residential developments". For each of these 4 scenarios, the probability of the event occurring over the design life is calculated as shown in Table 1.

Table 1: Design lives and design event ARI's for various Function Categories in *AS 4997*, with probability of event occurring over design life shown

Function Category	Design Life (years)	Design Event (ARI)	Probability of event occurring over design life (%)
2	50	500	9.5%
2	100	1,000	9.5%
3	50	1,000	4.9%
3	100	2,000	4.9%

It is evident that both Function Category 2 scenarios in Table 1 have a 9.5% probability of occurring over the design life, while both Function Category 3 scenarios have a 4.9% probability. Given the high property values of the subject properties, a Function Category 3 has been adopted herein (that is, with a 4.9% probability of the design event occurring over the design life) as the minimum requirement. This is also consistent with Gordon et al (2019), who recommended a 4 to 5% encounter probability for design of coastal protection works for normal residential structures.

It is considered that beach scour is the key design parameter for structural stability of the wall and the key determinant for the design life probability, and can be treated as an equivalent parameter to the design wave height in *AS 4997*. As discussed in Section 6, a highly unlikely scour level has been adopted for design. The ARI event to potentially cause the design scour (assuming that the cemented sand would erode) is considered to be rarer than 2,000 year ARI. The probability of such a scour level (at 2,000 year ARI) being realised over the adopted 100 year design life is 4.9%, which is satisfactory in relation to *AS 4997* and Gordon et al (2019).

8. WATER LEVELS AND WAVES

8.1 Design Event and Design Life

A 2,000 year ARI event (Section 7) was adopted over a 100 year life (Section 3) for structural design of the proposed works.

For consideration of wave overtopping, which is generated by depth-limited waves, it is considered to be reasonable to adopt 100 year ARI water level and wave parameters in conjunction with the highly unlikely (> 2000 year ARI) scour level of -1.8m AHD sloping down moving seaward at 1:30 (derived in Section 6). This is because the scour level governs water depths and hence the depth-limited wave heights impacting on the proposed seawall. This combination of 100 year ARI water level and wave parameters, and a > 2000 year ARI scour level, is likely to be in the order of a 2000 year ARI event or rarer. It could also be argued that a lower ARI event should be adopted for overtopping as the potential damages from wave overtopping are not as significant as from structural failure of the seawall, a minimum no-development setback of 4.5m landward of the wall will apply at the subject properties (to allow some dissipation of wave overtopping), and to be consistent with the design of rock revetment coastal protection works at Collaroy-Narrabeen Beach[9].

That stated, to illustrate sensitivity to 2,000 year ARI water level and wave parameters (in conjunction with the > 2000 year ARI scour level), calculations have been presented herein for this conservative scenario.

8.2 Present Design Ocean Still Water Level

Based on Department of Environment, Climate Change and Water [DECCW] (2010), the 100-year ARI ocean water level (in the absence of wave action) as of 2010 in Sydney is 1.44m AHD. This is similar to be the corresponding value reported by Manly Hydraulics Laboratory [MHL] (2018)[10]. Extrapolating the water levels (linear-log) provided in DECCW (2010) for various ARI's, the corresponding 2,000 year ARI value is 1.57m AHD.

Applying these values to the present (2020) using a rate of sea level rise of 3mm/year from 2010 to 2020, the 100 year ARI and 2,000 year ARI present day ocean water levels (in the absence of wave action) are 1.47m and 1.60m AHD respectively.

8.3 Sea Level Rise

In the *Specifications*, it is noted that sea level rise projections of 0.4m at 2050 and 0.9m at 2100 (both relative to 1990) may be adopted. For the proposed design life of 100 years (at 2120), it would be possible to extrapolate using the same rate of rise from 2050 to 2100 to obtain a sea level rise value at 2120 of 1.1m AHD relative to 1990[11].

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[9] In the Specifications, a minimum 50 year ARI design event is required for rock structures, and other designs of Horton Coastal Engineering for approved rock structures along Collaroy-Narrabeen Beach (eg at 1126-1144 Pittwater Road Collaroy) have adopted a 100 year ARI design event. It is recognised that rock structures (due to their flexible nature and reliance on depth-limited waves for stability that can only occur at the end of the design life) should have a lower ARI design event for stability design than vertical concrete structures. However, if a 100 year event is being applied for consideration of wave overtopping for rock structures, then applying 2,000 year ARI water levels and waves for vertical seawalls would be forcing the seawalls to have a higher design standard for overtopping.

[10] MHL (2018) determined a corresponding level of 1.42m AHD (along with lower and upper 95% confidence limits of 1.38m AHD and 1.53m AHD respectively).

[11] This is a sea level rise of 1.0m relative to 2010, discounting historical sea level rise at 3mm/year as recommended in DECCW (2010).

However, based on the *Specifications*, "variations to the above sea level rise projections may be considered. Where a variation is proposed, it shall be supported by a report prepared by a suitably qualified engineer". Given the non-linear rate of sea level rise and conservatism in the above benchmarks, it is considered to be most appropriate to apply a variation to the *Specifications* and to directly derive sea level rise values from Intergovernmental Panel on Climate Change [IPCC] (2013), which is widely accepted by competent scientific opinion. Furthermore, the methodology used to adopt the sea level rise values herein is the same as that used in the CZMP.

With a base year of 2010, as DECCW (2010) water levels were derived at 2010, the sea level rise values presented in Table 2 (at 2120) were determined for various emissions scenarios, assuming that the rate of rise from 2090 to 2100 continued to 2120.

Table 2: Global mean sea level rise (m) from 2010 to 2120 derived from IPCC (2013)

Emissions Scenario	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SRES A1B	0.51	0.72	0.97
RCP2.6	0.29	0.48	0.72
RCP4.5	0.41	0.61	0.84
RCP6.0	0.45	0.67	0.88
RCP8.5	0.66	0.94	1.27
Average	0.46	0.68	0.93

Taking the median exceedance probability and average of the 5 emissions scenarios, and adding 15% for local sea level rise variation based on IPCC (2013), a sea level rise value of 0.79m at 2120 (relative to 2010) was derived. Therefore, the 100 year ARI still water level (in the absence of wave action) at 2120 based on IPCC (2013) is 2.23m AHD. The corresponding 2,000 year ARI value is 2.36m AHD.

8.4 Design Ocean Still Water Level at End of Design Life

As noted in Section 8.3, the adopted 100 year ARI still water level (in the absence of wave action) at 2120 (at the end of the design life) is 2.23m AHD. The corresponding value for the 2,000 year ARI event is 2.36m AHD. Wave setup, caused by breaking waves adjacent to a shoreline, can also increase still water levels, as discussed further in Section 8.7.

8.5 Scour Level

As noted in Section 6, a design scour level of -1.8m AHD has been adopted at the proposed wall for structural design, sloping down moving seaward at 1:30. For design purposes, depth limited wave conditions must be determined at a plunging distance (or plunge length) seaward of the toe of the proposed works. Based on Coastal Engineering Research Center (1984) and Smith and Kraus (1991), the plunging distance is approximately equal to 10m. For a 1V:30H bed slope and with a -1.8m AHD scour level at the works, the bed level 10m seaward of the works is -2.1m AHD.

The 100 year ARI design ocean depth (excluding wave setup) at 2120 is thus 4.4m at the plunging distance. The corresponding 2,000 year ARI value is 4.5m.

8.6 Ocean Waves

Extreme value offshore wave conditions have recently (since the June 2016 storm) been re-evaluated for Sydney by Louis et al (2016), based on offshore Waverider buoy records. They determined 100-year ARI offshore significant wave heights (H_s) of 9.5m and 8.7m for 1 hour and 6 hour durations respectively.

Beach erosion and relatively large wave run-up is strongly linked to the occurrence of high wave conditions with elevated ocean water levels, so erosion and run-up are more likely to be significant when large waves coincide with a high tide. Consistent with MHL (2016), a 6-hour duration is considered to be appropriate for design, as storms with a duration of 6 hours are likely (50% probability) to coincide with high tide on the NSW coast (which is a prerequisite for elevated water levels to occur). A 1 hour duration only has an 8% probability of coinciding with high tide. Therefore, an offshore H_s (or H_o) of 8.7m was adopted herein. Extrapolating the H_s values (linear-log) provided in Shand et al (2011) for various ARI's, the corresponding 2,000 year ARI value is 10.8m.

In adopting 100-year ARI wave conditions herein, it was assumed that the design water level and wave can occur at the same time, which is conservative. Shand et al (2012) found that considering the joint probability of waves and tidal residuals for Sydney, the wave height for the joint 100 year ARI event reduced by about 10% as the tidal residual increased from 0.05m to 0.4m (with the latter necessary to achieve the design water level). That stated, adopting joint 100 year ARI water level and wave conditions is not entirely unreasonable, as elevated waves and water levels can be generated by the same weather systems. The same reasoning applies to the 2,000 year ARI water level and wave combination.

A design peak spectral wave period (T_p) of 13s was adopted, based on Shand et al (2011), who determined the associated wave period for the 100 year ARI H_s event at Sydney as 13.0s (± 0.7s considering 90% confidence intervals).

8.7 Wave Setup and Design Depth

Goda (2010a) has presented a relationship between wave setup at the shoreline, wave steepness, and beach slope. For a T_p of 13s, the deepwater wavelength (L_o) is 264m, and hence for an H_o of 8.7m the wave steepness is 0.03m. The beach slope offshore of the subject properties, between -40m and -10m AHD, is approximately 1:100 (MHL, 2016). For this slope and wave steepness, Goda (2010a) estimated that wave setup at the shoreline was 11% of H_o.

However, it is conservative to apply the full quantum of shoreline wave setup to define the depth limited breaking wave height at the proposed wall, as the design wave should be applied at a plunging distance offshore of the wall. With a depth (h) of 4.4m at the plunging distance at 2120, based on Goda (2010a) then h/H_o is 0.50, and wave setup at the plunging location is 6.9% of H_o. Therefore, setup at the plunging location is 0.60m, and the design depth at the plunging location at 2120 is 5.0m.

As it used in wave overtopping calculations (see Section 9), the present day depth at the plunging location was determined as the present (at 2020) design ocean water level (1.47m AHD, see Section 8.2), plus scour down to -2.1m AHD at the plunging distance as per Section 8.5 (thus a depth of 3.6m excluding wave setup, so h/H_o is 0.41), then wave setup at the plunging location is 7.5% of H_o from Goda (2010a), setup is 0.65m, and the design depth at the plunging location at present is 4.3m.

For the 2,000 year ARI event at present, the depth at the plunging location was determined as the present (at 2020) design ocean still water level (1.60m AHD, see Section 8.2),plus scour down to -2.1m AHD at the plunging distance as per Section 8.5 (thus a depth of 3.7m excluding wave setup, so h/H_o is 0.35), then wave setup at the plunging location is 7.9% of H_o from Goda (2010a), setup is 0.85m, and the design depth at the plunging location is 4.6m.

For the 2,000 year ARI event at 2120, the depth at the plunging location was determined as the 2010 design ocean still water level (1.57m AHD, see Section 8.2),plus sea level rise of 0.79m, plus scour down to -2.1m AHD at the plunging distance (thus a depth of 4.5m excluding wave setup, so h/H_o is 0.42), then wave setup at the plunging location is 7.4% of H_o from Goda (2010a), setup is 0.80m, and the design depth at the plunging location is 5.3m.

8.8 Design Wave Height at Structure

The method of Goda (2010b) for incipient breaking of significant waves was employed with the following parameters (for the 100 year ARI event with a > 2,000 year ARI scour level at 2120):

- water depth of 5.0m as defined in Section 8.7;
- L_o of 264m based on a wave period of 13s; and
- beach slope of 1:30, which is the bed slope down to the “inner Hallermeier” depth that was adopted in the CZMP.

This gave an H_s for incipient breaking of 3.0m (with a breaker index of 0.6), which was adopted as the design wave height at the structure.

Using the methodology in Battjes and Groenendijk (2000) for wave height distributions in the shoaling and breaking zone, $H_{10\%}$[12], $H_{2\%}$ and $H_{1\%}$ values of 3.7m, 3.9m and 4.1m were derived as these respective design wave heights at the structure for the 100 year ARI event (with a > 2,000 year ARI scour level) at 2120.

As it is used in wave overtopping calculations (see Section 9), a present-day 100 year ARI H_s for incipient breaking of 2.6m was calculated using the Goda (2010b) methodology. The corresponding 2,000 year ARI value was 2.8m, and 3.2m at 2120.

[12] Denoted as $H_{1/10}$ by Battjes and Groenendijk (2000).

9. WAVE OVERTOPPING AND WALL CREST LEVEL

As per the Drawings, a minimum wall crest level of 6.5m AHD has been adopted (at 1174 to 1180), increasing to 7.0m AHD at 1182. In the *Specifications*, a minimum crest level of 6.5m AHD is specified, so this requirement has been met.

The Neural Network tool[13] that is part of the second edition of the EurOtop manual (van der Meer et al, 2018) was utilised to calculate average wave overtopping rates in a 100 year ARI storm (with > 2,000 year ARI scour level) and 2,000 year ARI storm (again with > 2,000 year ARI scour level) at the proposed works for both present conditions and in 2120 (at the end of the design life).

Input parameters are summarised in Table 3 for the 100 year ARI event (with > 2,000 year ARI scour level) and 6.5m AHD crest level, as further explained below:

- parameters at the structure toe (water depth and wave height) were determined at the plunging distance, which is more conservative than determining them directly at the structure, but considered appropriate as this gives the wave height that the structure is impacted by;
- the spectral mean wave period was derived using the methodology of Hofland et al (2017) for long crested waves. In offshore (deepwater) conditions, the spectral mean wave period is approximately equal to the peak spectral wave period (of 13s for the 100 year ARI event, see Section 8.6) divided by 1.1 (that is, equal to 11.8s). However, the spectral mean wave period may change considerably if the waves are breaking on a very shallow foreshore ($h/H_o < 1$), as applies here, caused by the presence of low-frequency waves or infra-gravity waves (release of bound long waves in the breaking process on the mild foreshore);
- as there is to be no berm, the berm submergence and berm width were set to zero;
- the “angle of down slope (cotangent) and “angle of upper slope (cotangent)” of 0.017 was derived from the concrete wall being raked at 1° (that is, at an angle of 89° to the horizontal);
- the roughness factor was derived from Table 5.2 of van der Meer et al (2018) for closed concrete blocks; and
- for smooth structures, the size of the structure elements is equal to zero.

For the 7.0m AHD crest level, the crest freeboard was increased to 4.9m (present-day) and 4.2m (2120).

Table 3: Input parameters for Neural Network tool for 6.5m AHD crest level

Parameter	Value	
	Present-day	2120
Cotangent of foreshore slope	30	30
Water depth at the structure toe (m), see Section 8.7	4.3	5.0
Significant wave height at the toe of structure (m), see Section 8.8	2.6	3.0
Spectral mean wave period, $T_{m-1,0,t}$ (s)	35.1	31.2
Wave obliquity, ie angle of wave attack (°)	0.0	0.0
Toe submergence, ie water depth above toe of structure (m)	4.3	5.0
Width of toe (m)	0.0	0.0
Berm submergence (m)	0.0	0.0

13 Formentin et al (2017) and Zanuttigh et al (2016).

Parameter	Value	
	Present-day	2120
Berm width (m)	0.0	0.0
Angle of down slope (cotangent)	0.017	0.017
Angle of upper slope (cotangent)	0.017	0.017
Roughness factor (lower and upper)	1.0	1.0
Size of the structure elements (lower and upper) (m)	0	0
Crest freeboard in relation to still water level (m)	4.4	3.7
Wall freeboard in relation to still water level (m)	4.4	3.7
Crest width (m)	0	0

The wave return was not included in the Neural Network tool analysis, as this complexity cannot be adequately resolved in the model in conjunction with the other geometric factors. To account for the reduction in wave overtopping caused by the wave return, by deflecting uprushing water seaward, the wave return was applied to the results of the Neural Network tool as per the methodology in Figure 7.23 of van der Meer et al (2018). This gives a multiplier (k_{bn}) that factors down the overtopping discharge from the Neural Network. The input parameters for determination of the wave return overtopping multiplier were as listed in Table 4.

Table 4: Input parameters for determination of the wave return overtopping multiplier (for 6.5m AHD crest and 100 year ARI event with > 2,000 year ARI scour level)

Parameter	Value	
	Present-day	2120
Height of wave return wall (h_r, m)	1.2	1.2
Horizontal extension of wave return (B_r, m)	0.5	0.5
Crest freeboard (R_c, m)	4.4	3.7
Wave height at toe of structure (H_{mo}, m)	2.6	3.0

The resulting mean overtopping discharges, including the effect of the wave return, for the various simulations undertaken for the 100 year ARI event (with > 2,000 year ARI scour level) are summarised in Table 5. The corresponding discharges for the 2,000 year ARI event (with > 2,000 year ARI scour level) are summarised in Table 5. The adopted crest levels of 6.5m AHD (1174-1180) and 7.0m AHD (1182) were simulated, as well as 7.5m and 8.0m AHD, which would be representative of placing a (suitably designed) 1m high solid fence at the top of the wall for each of these wall levels respectively.

Table 5: Mean overtopping discharges for 100 year ARI event (with > 2,000 year ARI scour level) from Neural Network tool, with consideration of the wave return as per Figure 7.23 of van der Meer et al (2018)

Crest level (m AHD)	Mean overtopping discharge (L/s/m)	
	Present-day	2120
6.5	5.1	13.7
7.0	3.7	10.3
7.5	0.2	5.0
8.0	0.1	4.0

Table 6: Mean overtopping discharges for 2,000 year ARI event (with > 2,000 year ARI scour level) from Neural Network tool, with consideration of the wave return as per Figure 7.23 of van der Meer et al (2018)

Crest level (m AHD)	Mean overtopping discharge (L/s/m)	
	Present-day	2120
6.5	8.5	18.1
7.0	6.3	15.7
7.5	4.6	12.2
8.0	3.6	9.3

Historically, based on the previous (2007) version of EurOtop, a 50L/s/m overtopping discharge would have been considered a threshold for damage to a grassed or lightly protected promenade. That is, based on the 2007 version of EurOtop, the estimated mean overtopping discharges for the 100 year ARI and 2,000 year ARI events (with > 2,000 year ARI scour levels) would not have been considered to be damaging even to grass landward of the wall.

In the latest version of EurOtop (van der Meer et al, 2018), there is more of a focus on linking tolerable overtopping with the peak volume, and hence on the wave height that causes the overtopping, thus changing the limits for tolerable overtopping. For a grass covered crest and landward slope, maintained and closed grass cover and with H_{m0} (spectral significant wave height) of between 1m and 3 m (as applies here), a limit of 5L/s/m was adopted.

On this basis, no significant wave overtopping damage would be expected in the 100 year ARI storm (with > 2,000 year ARI scour level) occurring at present for crest levels of 7.0m AHD and higher (and only minor damage for the 6.5m AHD crest level), and in 2120 for crest levels of 7.5m AHD and higher. That stated, some landscaped backyard area damage can be tolerated, as any damage can be reinstated with new landscaping, the economic implications of any damage are relatively insignificant, and the overtopping would not impact on the structural integrity of the wall. On this basis, and given that raising of crest levels with solid fences can be undertaken over time (if not initially), the discharges in Table 5 are considered to be acceptable.

It is evident from Table 6 that the discharges for the 2,000 year ARI event (with > 2,000 year ARI scour level) are about 1.7 times larger for the present day and 1.3 to 1.5m larger at 2120 (in both cases for the 6.5m and 7.0m AHD crest), compared to the 100 year ARI event (with > 2,000 year ARI scour level). For the 7.5m and 8.0m AHD crest levels, the discharges for the 2,000 year ARI event (with > 2,000 year ARI scour level) are below the 5L/s/m threshold for damage of grass.

Future dwellings would be setback a minimum of 4.5m from the wall, which with coastal engineering input into the design of these structures (as would be required for a DA) may be considered an appropriate setback to reduce the risk of damage to these structures to an acceptably low level on a case by case basis.

After construction of the proposed works, overtopping would not be a significant issue at the subject properties in terms of inundation of dwellings, given the current setbacks and the fact that the floor levels of most dwellings are raised at least 0.3m above surround natural ground levels. As dwellings are redeveloped it is recommended that ground floor levels are increased to at least 0.5m above surrounding natural ground levels (typically to a level of about 6.8 to 7.4m AHD with natural ground at 6.3 to 6.9m AHD), to reduce the risk of inundation further.

The current minimum floor level at any of the subject properties is 6.8m AHD at 1176 and 1178 (with 7.0m at 1174, 7.2m at 1180 and 7.3m AHD at 1182).

With regard to safety of humans, a tolerable limit of 0.3L/s/m (for H_{m0} of 3m) and 1L/s/m (for H_{m0} of 2m) is noted in van der Meer et al (2018) for people at the wall crest with a clear view of the sea. The present day overtopping rates for the 100 year ARI event (with > 2,000 year ARI scour level) are below these limits for the 7.5m and 8.0m AHD crest levels, while the 2120 overtopping rates exceed these limits for all crest levels. A range of 1 to 10L/s/m was adopted in the 2007 version of EurOtop for pedestrians (trained staff, well shod and protected, expecting to get wet).

For their safety, it would be necessary for people to remain several metres landward of the wall crest in severe storms. However, the subject properties would be far more unsafe in severe storms if the protection works were not constructed.

10. GLOBAL STABILITY

In the *Specifications*, it is stated that “the seawall shall have a minimum factor of safety of 1.5 against global slope stability failure”. This has been demonstrated in a separate report (submitted with the DA documentation) prepared by JK Geotechnics (2020).

In the *Specifications*, it is stated that:

> “A geotechnical investigation shall be conducted at the property as part of the seawall design process to confirm, among other things, the extent of existing rock protection. The investigation shall be carried out by a suitably qualified engineer. The investigation shall include, as a minimum, excavation of three test pits along the seaward property boundary with the pits generally aligned perpendicular to the seaward property boundary”.

This requirement for three test pits at each property is not relevant at the subject properties for DA concept design purposes, as the existing rock revetments are not to be retained and will be excavated to remove potential piling obstructions. The extent of existing rock protection will affect the lateral extent and depth of required excavation, and can be considered as part of detailed design. That stated, JK Geotechnics (2016) undertook investigations at the subject properties which indicated the landward extent of the existing rock revetments.

Adequate subsurface investigations have been undertaken, as reported in JK Geotechnics (2020), for a sufficiently accurate geotechnical model of the subsurface to be developed for concept design (DA) purposes.

Furthermore, as discussed in Section 5 herein, adequate subsurface investigations have been undertaken to estimate the cemented sand level in the vicinity of the proposed works for concept design.

11. STRUCTURAL STABILITY

Structural/geotechnical engineering design of the proposed works has been undertaken with consideration of the results of the software packages WALLAP and PLAXIS.

WALLAP analyses the stability of cantilevered and propped retaining walls, with limit state equilibrium analysis for calculation of Factors of Safety, and bending moment and displacement analysis stage by stage as construction proceeds.

The two-dimensional finite element PLAXIS software has been used to assess the deadman anchor support system for a similar project at Collaroy-Narrabeen Beach (JK Geotechnics, 2018), with the principles applied for the subject DA. If the deadman system is adopted for construction, further site-specific analysis may be completed as part of detailed design.

The proposed design was found to have an adequate factor of safety for the ultimate design case. It was also found that simulated bending moments, shear forces and deflections in the wall for the ultimate design case would be accommodated by the proposed design. The geotechnical stability analyses were used in addition to the structural analysis of the wall components to derive the limit state strength requirements of each component and their connections. These loading scenarios form the limit state strength load cases used in the design process in accordance with *AS 3600-2018 Concrete structures* and *AS 2159-2009 (Piling - Design and installation)*.

A structural engineering design statement by James Taylor & Associates (2020) has been submitted with the DA documentation.

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