Horton Coastal Engineering
Coastal & Water Consulting

Coastal Engineering Report for Construction of Upgraded Coastal Protection Works at 1150-1168 Pittwater Road Collaroy

Prepared by Horton Coastal Engineering Pty Ltd

for SWNA Pty Ltd

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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 1150-1168 Pittwater Road Collaroy.

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.

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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 (concrete) contiguous piles with infill concrete plug piles, with a rock toe (comprising 3 boulders in section) located seaward. 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, 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 solution, with iterative input from these three disciplines to produce a robust design. For example, the design took into account coastal engineering issues (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 concrete wall and piling was designed assuming that the rock toe was not present. The rock toe would provide some dissipation of wave energy and reduction of sand scour adjacent to the wall in practice, and infill any scour hole that formed, but has conservatively been ignored in the stability analysis for the wall, with scour allowed for down to -2m AHD.

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, which extends 0.5m seaward of the main face of the wall.

The rock toe is not a conventional rock revetment. That is, it does not comprise a primary layer, secondary layer and underlayer. Rather, it comprises 3 primary armour boulders in section arranged such that the lower 2 boulders are founded on a flat slope (enhancing stability), with the upper boulder sloping landwards to enhance its stability. Given the non-critical function of the rock toe for this design, prevention of soil migration at the contiguous/plug piles, and likely placement of the boulders on cemented sand, use of secondary layers and underlayers is not necessary and would be redundant (as well as increasing the works footprint).

The works are located entirely on private property, with the main face of the concrete wall located 5.7m landward of the seaward property boundaries at 1150-1164 (as per the Drawings, stairs are recessed into the wall at various locations, extending landward of this alignment) and 2m landward at 1168, with a transition at 1166a and 1166b. The seaward edge of the works (seaward edge of the rock toe) would be about 2.7m landward of the seaward property boundaries at 1150-1164, and on the seaward boundary at 1168, again with a transition at 1166a and 1166b. For most of the time (that is, except when the rock toe is exposed by beach erosion), the public will be able to access the sandy beach on what is actually private property for a distance of 5.7m at 1150-1164, and for a distance of 2m at 1168. Even with the rock toe exposed, there would be about 2.7m of sandy beach exposed within 1150-1164, that would essentially be accessible by the public.

The seaward extent of the works has been considerably reduced compared to the existing rock revetments at and seaward of the subject properties. In particular, at 1168, where existing works extend about 5m to 11m (average 8m) seaward of the property on to Crown Land, this rock will be entirely removed. The seaward extent of rock will also be reduced at 1150, 1166a and 1166b, and essentially be the same as existing at the other 7 subject properties.

A minimum 5.5m 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. No future structures, except readily relocatable or removable structures that do not interrupt views, are to be constructed seaward of this setback, to enable clear passage of construction plant as required for future works maintenance. This is consistent with the *Specifications*. That stated, there may be no need for maintenance to be undertaken from landward of the wall, as any rock toe maintenance (if required) could be undertaken some duration after storms when the beach has partially recovered and is accessible (this delay would be possible as rock toe integrity is not critical to wall stability).

Concrete beach access stairs have been provided at all properties except 1168, comprising upper and lower stairs. The upper stairs are recessed into the wall and shore-parallel, with a lower landing at the common property boundaries at 1158/1160 and 1166a/1166b, and individually at 1150 (with double upper stairs), 1154, 1156, 1162, and 1164. The lower stairs project perpendicular to the wall, supported on a pile at the lower end. This would essentially provide permanent beach access at the properties, with the stairs integrated into the wall. The lower stairs project no further seaward than the seaward extent of the rock toe, and thus are within the subject private properties.

3. PROPOSED DESIGN LIFE OF PROTECTION WORKS

A design life of 60 years has been adopted for the proposed protection works (that is, at the year 2078). As outlined in Horton et al (2014) and Horton and Britton (2015), this 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[1] (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;
  - in *AS 1170.0:2002* (*Structural Design Actions – General Principles)*, the design life for normal structures (Importance Level 2, as applies to typical residential development) is generally taken as 50 years; 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. In practice, rock incorporated within the proposed works would have a considerably greater life than 60 years based on the durability criteria specified in the design.

The proposed design life of 60 years is thus appropriate, and meets the minimum requirement in the *Specifications*.

[1] 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 60 YEAR DESIGN LIFE TO CONCRETE AND ANCHOR DESIGN

A 60 year design life (and beyond) is achievable for the concrete wall and concrete piling (including the deadman anchoring if adopted). As noted above, *AS 3600:2018* 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 65mm to 80mm would be adopted.

Other features that would be adopted to ensure a minimum 60 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 60 year design life (and beyond) is achievable for the steel screw pile anchoring, 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. ADOPTED DESIGN PROBABILITY AND RISK USED IN THE DESIGN

A 100-year Average Recurrence Interval (ARI) storm event[2] has been adopted for design. This exceeds the minimum 50-year ARI requirement in the *Specifications*.

It is important to understand that adoption of the 100 year ARI event for design actually leads to a much rarer storm being able to cause "failure" of the rock toe. This is because:

- the rock toe design is to the 0-5% damage level (generally referred to as the "no damage" condition), whereas failure is generally considered to be the 20% damage level;
- the required rock mass is governed by wave height, which is depth-limited;
- the rock toe is only subject to the design wave at the end of the design life, after projected sea level rise (the highest possible water level) has been realised;
- for most of the design life, the design wave height cannot occur as it is depth-limited;
- water levels only increase slightly as ARI's become exponentially rarer; and
- rock structures can accommodate some damage without failure.

Considering a single parameter (such as wave height, or water level), a 100-year ARI event has a 45% probability of occurring over a 60-year life. However, this event for a single parameter only has a 1% probability of occurring in Year 60 of the design life, which is the only year when the design wave height would be physically able to occur due to being depth-limited in earlier years[3].

In Australian Standard *AS 4997:2005*, a design life of 50 years is recommended for normal maritime structures (specifically excluding rock structures), in conjunction with a 500 year ARI design wave height (this event has a 10% probability of occurring over the design life). However, this does not apply to rock structures (both explicitly within the Standard, and as explained by the logic above). It is reiterated that due to depth-limited conditions, the design wave height cannot occur until the last year of the design life, such that the selection of a 100 year ARI design wave event is considered to be conservative for the rock component of the proposed protection works.

However, the proposed works have been structurally designed ignoring the presence of the rock toe, and allowing for scour down to -2m AHD adjacent to the wall (see Section 6.4 for further discussion on scour). This is conservative as:

- the rock toe would be expected to limit scour, or infill any scour hole that formed; and
- significant scour of the (hard) cemented sand layer would not be expected, with about 2m depth of scour of cemented sand required to reach the design scour level of -2m AHD.

[2] Also known as a 1 in 100 Annual Exceedance Probability (AEP) event.
[3] There is a 10% probability of the design storm occurring in any of the last 10 years of the design life, again only considering a single parameter. However, it should be noted that the sea level rise value adopted (see Section 6.2) is not a certain occurrence, and has an associated probability of exceedance that can be approximately estimated in the order of 50%. Similarly, the scour level adopted (Section 6.4) is not a certain occurrence, indeed it is conservative. That is, use of a 100 year ARI wave height and water level cannot be used to directly imply particular encounter probabilities for damage to the rock toe due to the multiple probabilistic factors at play.

Realisation of a scour level of -2m AHD adjacent to the wall is barely credible, and the probability of this scour level being realised over the design life is considered to be much less than 10%.

Furthermore, as discussed in the geotechnical report submitted with the subject DA (JK Geotechnics, 2018), structural design has been undertaken assuming a conservative groundwater level difference of 3.5m between the landward and seaward sides of the wall at the time of maximum scour.

With these conservative scour and groundwater assumptions, it is considered that the piling and anchor design is consistent with the intent of *AS 4997*, with regard to much less than a 10% probability of the ultimate design conditions being realised over the design life.

6. WATER LEVELS, SCOUR AND WAVES

6.1 Present Design Ocean 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 is 1.44m AHD. This is also consistent with Manly Hydraulics Laboratory [MHL] (2016a)[4].

6.2 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 60 years (at 2078), it would be possible to interpolate between the 2050 and 2100 projections to obtain a sea level rise value of 0.68m AHD relative to 1990[5].

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] (2013a, b), 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 1 (at 2078) were determined for various emissions scenarios.

Table 1: Global mean sea level rise (m) from 2010 to 2078 derived from IPCC (2013b)

Emissions Scenario	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SRES A1B	0.27	0.39	0.51
RCP2.6	0.20	0.30	0.42
RCP4.5	0.24	0.36	0.47
RCP6.0	0.24	0.35	0.46
RCP8.5	0.33	0.45	0.59
Average	0.26	0.37	0.49

Taking the median exceedance probability and average of the 5 emissions scenarios, and adding 15% for local sea level rise variation based on IPCC (2013b), a sea level rise value of 0.42m at 2078 (relative to 2010) was derived. Therefore, the 100 year ARI still water level at 2078 based on IPCC (2013b) is 1.86m AHD.

6.3 Design Ocean Water Level at End of Design Life

As noted in Section 6.2, the adopted 100 year ARI still water level at 2078 (at the end of the design life) is 1.86m AHD. Wave setup, caused by breaking waves adjacent to a shoreline, can also increase still water levels, as discussed further in Section 6.6.

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[4] Although MHL (2016a) also found that uncertainties in the extreme value analysis methods adopted to be of the order of +0.1m, which could be applied as an additional increment on the design water level in a more conservative analysis.
[5] This is a sea level rise of 0.62m relative to 2010, discounting historical sea level rise at 3mm/year as recommended in DECCW (2010).

6.4 Scour Level

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".

Immediately south of the subject properties, UNSW Water Research Laboratory (WRL) undertook a survey of cemented sand outcrops observed seaward of 1126-1144 Pittwater Road Collaroy after the June 2016 storm. They found cemented sand levels varied from about -0.3 to 0.6m AHD, with a mean and median level of about 0.1m AHD, and standard deviation ranging from -0.1m to 0.3m AHD. This was at an alignment about 10m landward of the subject property boundaries.

Again, immediately south of the subject properties, Douglas Partners (2016) undertook a geotechnical investigation which included drilling of boreholes at the Council reserve south of Stuart Street, and at Ramsay Street. These boreholes were at a cross-shore position at an alignment about 40m landward of the cemented sand outcrops observed by WRL. Correcting approximately estimated surface levels of 5m AHD used by Douglas Partners (2016), which based on survey would be closer to 6.1m AHD at Stuart Street and 5.6m AHD at Ramsay Street, this would put dense to very dense sand at -2.4m AHD to 2.4m AHD at Stuart Street, and -2.1m AHD to 1.7m AHD at Ramsay Street. Analysing the cone penetration test results of Douglas Partners (2016), the upper surface of the cemented sand layer can be interpreted to be at 1.1m AHD at both locations[6].

Comparing the Douglas Partners (2016) and WRL cemented sand levels, this would indicate that the upper surface of the cemented sand layer dips at about 1V:40H (1.4°) moving seaward.

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, and 1172, 1174 (two locations), 1176, 1178, 1180 and 1182 Pittwater Road Narrabeen[7]. These DCP test locations were about 10m to 13m landward of the seaward property boundaries.

Applying a 1V:40H dip to the JK Geotechnics (2016) test results, it would be expected that there would be a mean cemented sand level of about 0.8m AHD along the proposed wall alignment, assuming that it continues at the same level moving south.

A geotechnical investigation was completed by Jeffery & Katauskas (2000), which included boreholes at Stuart Street and Wetherill Street (located about 15m landward of the seaward property boundaries). This indicated levels of the upper surface of the cemented sand layer of about 0m AHD at Stuart Street, and 0.3m AHD at Wetherill Street, which with a 1V:40H dip would be about -0.2m to 0.1m AHD along the proposed wall alignment.

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[6] As interpreted by Horton Coastal Engineering and reviewed by Paul Roberts of JK Geotechnics.
[7] 1168 is the northernmost of the subject properties (where cemented sand was found at 1m AHD), while 1172-1182 are at the block immediately north of the subject properties and Wetherill Street.

Jeffery & Katauskas (2000) also completed three test pits, seaward of 1150, 1158 and 1168 Pittwater Road respectively, and at a cross-shore position about 5m seaward of the seaward property boundaries. This indicated a cemented sand level of about 0m AHD at 1150, -0.5m AHD at 1158, and -1.4m AHD at 1168 (which with a 1V:40H dip would be about 0.3m, -0.2m and -1.1m AHD respectively along the proposed wall alignment). The 1168 level seems inconsistent with the borehole results at Wetherill Street, and the former was probably less accurately defined, so has been considered anomalous herein.

Coffey Partners International (1998) completed two test pits, seaward of Stuart Street and 1166a respectively, at an alignment similar to or a few metres landward of the seaward property boundaries. They found cemented sand levels of around -0.5m and -0.2m AHD respectively, which with a 1V:40H dip would be about -0.4m to -0.1m AHD along the proposed wall alignment.

Therefore, at the alignment of the proposed wall, Jeffery & Katauskas (2000) and Coffey Partners International (1998) would indicate cemented sand levels of about 0.3m to -0.4m AHD.

There is thus some inconsistency between the Jeffery & Katauskas (2000) and Coffey Partners International (1998) cemented sand levels of 0.3m to -0.4m AHD, and JK Geotechnics (2016) extrapolated results of 0.8m AHD. The Drawings have been prepared assuming a cemented sand level of 0m AHD. In practice, the configuration of the rock toe will be adjusted depending on the cemented sand level found in construction.

As long-term recession is realised, scour levels may lower at a particular cross-shore position where the beach profile translates landward. In the CZMP, a "best estimate" inverse slope of the active beach profile of 30 was adopted, which would cause 13m 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 0.42m from Section 6.2), and about 0.4m of lowering assuming no restriction from cemented sand.

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:40H cemented sand slope and assuming a 0m AHD cemented sand level at the works, the cemented sand level 10m seaward of the works is about -0.3m AHD. Adding the 0.4m of lowering from long term recession due to sea level rise, the bed level at a plunging distance from the works is -0.7m AHD.

Although it is expected that cemented sand (where present) would restrict this lowering, the level and extent of cemented sand at a location 10m seaward of the works is unknown. On this basis, it has been conservatively assumed herein that the scour level at the plunging distance would be -1m AHD, consistent with general NSW beach scour levels. The design ocean depth (excluding wave setup) is thus 2.9m at the plunging distance.

Placing the rock toe on the cemented sand layer (expected to be at around 0m AHD), is considered to provide a suitable toe level and to meet the intent of the *Specifications* (given the evidence of an inerodible layer at a level above -1m AHD).

6.5 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 (2016b), 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.

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.

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).

6.6 Wave Setup and Design Depth

Goda (2000) 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, 2016b). For this slope and wave steepness, Goda (2000) 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 on the proposed rock toe, as the design wave should be applied at a plunging distance offshore of the rock toe[8]. With a depth (h) of 2.86m at the plunging distance, based on Goda (2000) then h/H_o is 0.3, and wave setup at the plunging location is 9% of H_o. Therefore, setup at the plunging location is 0.8m, and the design depth at the plunging location is 3.7m.

As it used in wave overtopping calculations (see Section 8), the present day depth at the plunging location was determined as the present design ocean water level (1.44m AHD, see Section 6.1), plus an allowance for scour down to -1m AHD as per Section 6.4, plus approximate wave setup of 0.8m as above. This gave the present-day 100 year ARI depth at the plunging location as 3.2m.

[8] Furthermore, the subject properties may not be fully exposed to the design offshore wave height, due to sheltering provided by Long Reef headland from wave directions from the S to SE.

6.7 Design Wave Height at Structure

The method of Goda (2010) for incipient breaking of significant waves was employed with the following parameters:

- water depth of 3.7m as defined in Section 6.6;
- 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 2.3m (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, $H_{10\%}$[9] and $H_{2\%}$ values of 2.8m and 3.0m were derived as these respective design wave heights at the structure.

As it used in wave overtopping calculations (see Section 8), a present-day 100 year ARI H_s for incipient breaking of 2.0m was calculated using the Goda (2010) methodology.

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

7. ROCK TOE HYDRAULIC STABILITY DESIGN

7.1 Preamble

The most comprehensive and up to date guidance on rock revetment design is provided in the *Rock Manual* (CIRIA et al, 2012). For design of revetments in shallow water conditions, four different methodologies for rock armour sizing are outlined therein, namely:

1. the Hudson formula;
2. a modified Hudson formula proposed by van der Meer (1988);
3. van der Meer formulae for shallow water conditions; and
4. van Gent et al (2004) stability formula.

Method 3 was preferred herein (see Section 7.2), with checks of the other methodologies as discussed in Section 7.3, with ultimately a slight rounding up of the Method 3 required primary armour mass adopted for design as discussed in Section 7.4.

With only 3 boulders in the rock toe, hydraulic design as per these methodologies is not strictly applicable (eg as there is only limited two-layer primary armour, and no secondary layers). However, calculations were undertaken assuming the steepest allowable hydraulically stable slope of 1:1.5 (vertical:horizontal) which is also the steepest allowable slope in the *Specifications*, as a means of applying some conservatism to the calculations (given that the base boulders will actually be placed at a flat slope and the upper boulder will slope landwards).

7.2 Method 3 (van der Meer Formulae for Shallow Water)

Method 3 was adopted herein with parameters as follows:

- H_s of 2.3m and $H_{2\%}$ of 3.0m (from Section 6.7);
- T_p of 13s (from Section 6.5), with $T_{m-1,0}$ assumed to be T_p ÷1.1;
- number of waves (N) in storm calculated assuming a 2.5 hours storm duration, given that waves are depth-limited and elevated water levels can only occur at high tide (for a T_p of 13s, which is a $T_{m-1,0}$ of 11.8s, N is about 760);
- (basalt) rock density of 2,650kg/m^3;
- (sea) water density of 1,025kg/m^3;
- structure slope of 1:1.5 (vertical:horizontal);
- notional permeability of structure of 0.4;
- coefficient c_{pl} of 7.25, and c_s of 1.05, as applies for a 5% damage limit; and
- S_d of 2 as applies for 0-5% damage (start of damage) for armourstone in a double layer.

Method 3 is the only one of the formulae considered herein to include wave period as an input parameter. Under sensitivity testing of a range of wave periods from 6s to 13s, it was found that the design T_p of 13s did not produce the largest required armour size, but rather a T_p of about 10s. This was adopted as the critical case.

For the critical case, the median primary armour mass was determined to be 3.7 tonnes.

7.3 Other Methodologies

7.3.1 Method 1 - Hudson

For the Hudson methodology, parameters were adopted as follows:

- $H_{10\%}$ of 2.8m (from Section 6.7);
- stability coefficient (K_D) of 2.0 (for breaking waves); and
- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 4.9 tonnes.

7.3.2 Method 2 - Modified Hudson (van der Meer, 1988)

For the modified Hudson methodology, parameters were adopted as follows:

- H_s of 2.3m (from Section 6.7);
- stability coefficient (K_D) of 4.0 (for permeable core); and
- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 1.2 tonnes. Realisation of a permeable core is uncertain for the proposed rock toe. For an impermeable core assumption with K_D equal to 1.0, the median primary armour mass was determined to be 7.6 tonnes. Assuming an intermediate K_D of 2.0 as per Section 7.3.1, the median primary armour mass was determined to be 3.8 tonnes.

7.3.3 Method 4 – van Gent et al (2004)

For the van Gent et al (2004) methodology, parameters were adopted as follows:

- H_s of 2.3m (from Section 6.7);
- N determined as per Section 7.2 (it is 1,000 waves for the critical case);
- S_d of 2 as applies for 0-5% damage (start of damage) for armourstone in a double layer;
- core particle diameter of 0.4mm (typical median diameter of sand in the study area); and
- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 4.3 tonnes.

7.4 Adopted Design (Masses and Gradings)

A median primary armour mass of 3.8 tonnes was adopted for design, as reflected on the Drawings, which is the same as the minimum mass for igneous rock in the *Specifications*. This is slightly above the preferred Method 3 mass of 3.7 tonnes (Section 7.2) and equal to the Method 2 mass of 3.8 tonnes assuming an intermediate K_D of 2.0 (Section 7.3.2). It is below the Method 1 mass of 4.9 tonnes (Section 7.3.1), and below the Method 4 mass of 4.3 tonnes, although it can be noted that Method 1 is known to be conservative.

A grading of 0.75 to 1.25 times the median mass was adopted for the primary armour, consistent with the *Specifications*.

Masses stated on the Drawings are based on hydraulic stability formulae as outlined above, with dimensions only approximate, and expressed on the Drawings as square opening sieve size dimensions (calculated as 1.15×D_{n50}, where D_{n50} is the median dimension of an equivalent cube giving the median rock mass). Note that in CIRIA et al (2012), square opening sieve size dimensions are calculated as 1.19×D_{n50}.

7.5 Rock Durability

Notes are included on Drawing S01 to specify required rock durability criteria, consistent with the *Specifications* for igneous rock.

8. WAVE OVERTOPPING AND WALL CREST LEVEL

As per the Drawings, a minimum wall crest level of 6.0m AHD has been adopted (at 1150 and 1154), increasing to 6.5m AHD at 1156-1168.

The Neural Network for Wave Overtopping Predictions (van Gent et al, 2007) associated with EurOtop (van der Meer et al, 2016), Version 2.04 (March 2016), was utilised to calculate average wave overtopping rates in a 100 year ARI storm at the proposed works for both present conditions and in 2078 (at the end of the design life).

Input parameters are summarised in Table 2 for the 6.5m AHD crest level. To approximately discretise the proposed works in the model, the rock toe or stair landing was treated as a berm of 1.5m width and crest elevation of 3.0m AHD, with its seaward face sloping at 30° (representing the slope down to the lower and most seaward boulder, as per the "angle of down slope (cotangent)", or slope of the lower stairs). Sensitivity testing was also undertaken, with little difference in results if the rock toe was ignored. The highest overtopping discharges were obtained with the parameters in Table 2. Sensitivity testing was also undertaken considering the steps at various levels in the wall.

The "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 mean wave period was derived as the peak spectral wave period of 13s divided by 1.1. The roughness coefficient was derived from Table 6.2 of van der Meer et al (2016) for a smooth impermeable surface. For the 6.0m AHD crest level, the crest freeboard was reduced to 3.8m (present-day) and 3.3m (2078).

Table 2: Input parameters for Neural Network for Wave Overtopping Predictions for 6.5m AHD crest level

Parameter	Value	
	Present-day	2078
Angle of wave attack (°)	0.0	0.0
Water depth in front of structure (m), see Section 6.6	3.2	3.7
Significant wave height at the toe of structure (m), see Section 6.7	2.0	2.3
Mean wave period (s)	11.8	11.8
Water depth at the toe of structure (m)	3.2	3.7
Width of toe (m)	0.0	0.0
Roughness coefficient	1.0	1.0
Angle of down slope (cotangent)	1.7	1.7
Angle of upper slope (cotangent)	0.017	0.017
Crest freeboard in relation to still water level (m)	4.3	3.8
Berm width (m)	1.5	1.5
Water depth at the berm of the structure (m)	0.2	0.7
Berm slope (tangent)	0.0	0.0
Armour freeboard in relation to SWL (m)	0.0	0.0
Armour width (m)	0.0	0.0

The wave return was not included in the Neural Network for Wave Overtopping Predictions 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 up-rushing water seaward, the wave return was applied to the results of the Neural Network for Wave Overtopping Predictions as per the methodology in Figure 7.23 of van der Meer et al (2016). 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 3.

Table 3: Input parameters for determination of the wave return overtopping multiplier (for 6.5m AHD crest)

Parameter	Value	
	Present-day	2078
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)	4.3	3.8
Wave height at toe of structure (H_{mo}, m)	2.0	2.3
Water depth at toe of structure (h, m)	3.2	3.7

The resulting mean overtopping discharges, including the effect of the wave return, for the various simulations undertaken are summarised in Table 4. The adopted crest levels of 6.0m and 6.5m AHD were simulated, as well as 7.0 and 7.5m AHD, which would be representative of placing a (suitably designed) 1m high glass fence at the top of the wall for each of these wall levels respectively.

Table 4: Mean overtopping discharges from Neural Network for Wave Overtopping Predictions with consideration of the wave return as per Figure 7.23 of van der Meer et al (2016)

Crest level (m AHD)	Mean overtopping discharge (L/s/m)	
	Present-day	2078
6.0	3.1	8.4
6.5	0.1	5.4
7.0	0.1	3.4
7.5	0.0	0.1

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 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, 2016), 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 present day 100 year ARI storm, for either the 6.0 or 6.5m AHD crest levels. By 2078, some promenade (backyard) damage would be expected for this storm, for both crest levels, although if a 1m glass fence was installed no significant wave overtopping damage would be expected for either crest level.

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.

Future dwellings would be setback a minimum of 7.8m from the wall (with low-level decks etc a minimum of 5.5m from the wall), which with coastal engineering input into the design of these structures (as would be required for a DA) is considered to be an acceptable setback to reduce the risk of damage to these structures.

To adapt to increasing overtopping volumes as sea level rise is realised beyond the design life, it would be possible to install 1m high glass fencing above the crest of the wall to provide sufficient reduction in wave overtopping for some time.

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. 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 6.5 to 7.0m AHD with natural ground at 6.0 to 6.5m AHD), to reduce the risk of inundation further. The current minimum floor level at any of the subject properties is 6.24m AHD at 1150.

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 (2016) for people at the wall crest with a clear view of the sea. 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.

In the *Specifications*, a minimum crest level of 6.5m AHD is specified, "although a lower crest level may be considered. Where a lower crest level is proposed it shall be supported by a report prepared by a suitably qualified engineer". As reported above, a minimum crest level of 6.0m AHD, as proposed (at only 2 of the 10 subject properties, with the other 8 properties having a wall crest level of 6.5m AHD), would give satisfactory overtopping discharges at present based on van der Meer et al (2016). These walls could be adapted to 1m higher with a glass fence in the future, and on this basis a variation to the minimum crest level is considered to be justified.

9. 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 prepared by JK Geotechnics (2018).

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 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 considered as part of detailed design. That stated, JK Geotechnics (2016) undertook investigations at 1168 which indicated that the landward extent of the existing rock revetment was about 4m into the property, although this could not be definitively determined due to the presence of concrete slabs in the upper subsurface.

Adequate subsurface investigations have been undertaken, as reported in JK Geotechnics (2018), for a sufficiently accurate geotechnical model of the subsurface to be developed for analysis purposes.

Furthermore, as discussed in Section 6.4 herein, adequate subsurface investigations have been undertaken to estimate the cemented sand level in the vicinity of the proposed works.

10. 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 was used to assess the deadman anchor support system.

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* and *AS 2159:2009 (Piling - Design and installation)*.

11. REFERENCES

Australian Geomechanics Society Landslide Taskforce, Landslide Practice Note Working Group [AGS] (2007a), "Practice Note Guidelines for Landslide Risk Management 2007", *Australian Geomechanics*, Volume 42, No. 1, March, pp. 63-114

Australian Geomechanics Society Landslide Taskforce, Landslide Practice Note Working Group [AGS] (2007b), "Commentary on Practice Note Guidelines for Landslide Risk Management 2007", *Australian Geomechanics*, Volume 42, No. 1, March, pp. 115-158

Battjes, Jurjen A and Heiko W Groenendijk (2000), "Wave height distributions on shallow foreshores", *Coastal Engineering*, Vol. 40, Issue 3, pp. 161-182

Burcharth, Hans F and Steven A Hughes (2011), "Fundamentals of Design", in Hughes, Steve (editor), *Coastal Engineering Manual*, Part VI, Design of Coastal Project Elements, Chapter VI-5, Engineer Manual 1110–2–1100, US Army Corps of Engineers, Washington, DC, Change 3 of 28 September 2011

Carley, James T; Coghlan, Ian R; Flocard, Francois; Cox, Ronald J and Thomas D Shand (2015), "Establishing the Design Scour Level for Seawalls", *Australasian Coasts & Ports Conference 2015*, 15-18 September, Auckland, New Zealand

CIRIA, CUR, and CETMEF (2012), *The Rock Manual, The Use of Rock in Hydraulic Engineering*, 2nd Edition, C683, CIRIA, London, originally published in 2007, reprinted (including errata) in 2012

Coastal Engineering Research Center (1984), *Shore Protection Manual*, Volume II, 4th Edition, US Army Corps of Engineers, Vicksburg, Mississippi

Coffey Partners International (1998), *Collaroy / Narrabeen Seawall Geotechnical Investigation*, for NSW Department of Public Works & Services, 13 October, S10962/1-AE

Department of Environment, Climate Change and Water [DECCW] (2010), *Coastal Risk Management Guide: Incorporating sea level rise benchmarks in coastal risk assessments*, DECCW 2010/760, August, ISBN 978 1 74232 922 2

Douglas Partners (2016), *Report on Geotechnical Investigation, Proposed New Foundations, 1130 Pittwater Rd Collaroy*, prepared for KPH Consulting, Project 85598.01, October

Goda, Y (2000), *Random Seas and Design of Maritime Structures*, 2nd Edition, Advanced Series on Ocean Engineering, Vol. 15, World Scientific, Singapore

Goda, Yoshimi (2010), "Reanalysis of Regular and Random Breaking Wave Statistics", *Coastal Engineering Journal*, Vol. 52, No. 1, pp. 71–106

Horton, Peter and Greg Britton (2015), "Defining Beachfront Setbacks Based on 'Acceptable Risk' – is it the New Approach", *Australasian Coasts & Ports Conference 2015*, Auckland, New Zealand, 15-18 September

Horton, Peter; Britton, Greg; Gordon, Angus; Walker, Bruce; Moratti, Mark and Daylan Cameron (2014), "Drawing a Line in the Sand – Defining Beachfront Setbacks Based On Acceptable Risk", *23rd NSW Coastal Conference*, Ulladulla, 11-14 November

Intergovernmental Panel on Climate Change [IPCC] (2013a), "Summary for Policymakers", in: *Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change*, [Stocker, TF; Qin, D; Plattner, G-K; Tignor, M; Allen, SK; Boschung, J; Nauels, A; Xia, Y; Bex, V and PM Midgley (editors)], Cambridge University Press, Cambridge, United Kingdom and New York, New York, USA

Intergovernmental Panel on Climate Change [IPCC] (2013b), *Climate Change 2013, The Physical Science Basis, Working Group I Contribution to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change*, Final Draft, 30 September

Jeffery & Katauskas (2000), *Report to Patterson Britton & Partners Pty Ltd on Geotechnical Investigation for Collaroy/Narrabeen Sea Wall Upgrade at Collaroy/Narrabeen Beach, NSW*, 11 July

JK Geotechnics (2016), *Report to Horton Coastal Engineering Pty Ltd & Haskoning Australia Pty Ltd on Geotechnical Investigation for Proposed Upgrade of Existing Foreshore Protection Measures at 1168 – 1182 Pittwater Road, Collaroy*, NSW, 13 December, Ref: 30005ZRrpt

JK Geotechnics (2018), *Report to SWNA Pty Ltd on Geotechnical Assessment for Proposed Coastal Protection Works at 1150 to 1168 Pittwater Road, Collaroy, NSW*, 20 July, Ref: 30444ZRrpt Rev4

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

Manly Hydraulics Laboratory (2016a), "NSW Ocean Water Levels", *Report MHL2236*, Draft Final

Manly Hydraulics Laboratory (2016b), "Collaroy-Narrabeen Beach Coastal Protection Assessment", *Report MHL2491*, December

Shand, TD; Mole, MA; Carley, JT; Peirson, WL and RJ Cox (2011), "Coastal Storm Data Analysis: Provision of Extreme Wave Data for Adaptation Planning", *WRL Research Report 242*, UNSW Water Research Laboratory, July

Shand, TD; Wasko, CD; Westra, S; Smith, GP; Carley, JT and WL Peirson (2012) "Joint Probability Assessment of NSW Extreme Waves and Water Levels", *WRL Technical Report 2011/29*, UNSW Water Research Laboratory, for Office of Environment and Heritage

Smith, ER and NC Kraus (1991), "Laboratory study of wave breaking over bars and artificial reefs", *Journal of Waterway, Port, Coastal and Ocean Engineering*, Volume 117, Issue 4, July, pp. 307–325

van der Meer, Jentsje W (1988), "Rock Slopes and Gravel Beaches under Water Attack", PhD Thesis, Delft University of Technology, and also Delft Hydraulics Publication No. 396

van der Meer, JW; Allsop, NWH; Bruce, T; De Rouck, J; Kortenhaus, A; Pullen, T; Schüttrumpf, H; Troch, P and B Zanuttigh (2016), *EurOtop, Manual on wave overtopping of sea defences and related structures, an overtopping manual largely based on European research, but for worldwide application*, pre-release version, October

van Gent, MRA; van den Boogaard, HFP; Pozueta, B and JR Medina (2007), "Neural network modelling of wave overtopping at coastal structures", *Coastal Engineering*, Vol. 54, pp. 586-593