Horton Coastal Engineering
Coastal & Water Consulting

Coastal Engineering Report for Construction of Upgraded Coastal Protection Works at 1190-1196 and 1204 Pittwater Road Narrabeen

prepared by Horton Coastal Engineering Pty Ltd

for the owners of 1190-1196 and 1204 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 1190, 1192, 1194, 1196 and 1204 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 28 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) secant 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 (hollow bar anchors at 1.5m 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 (groundwater conditions, 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.

Initial geotechnical analysis to demonstrate both global stability and structural stability (with consideration of disturbing and balancing forces and moments) has been completed as discussed in Section 6 and shown in Appendix B and Appendix C. Detailed design is to be completed (potentially supplemented by a geotechnical field investigation to provide more accuracy to the subsurface model) prior to construction. This is not an issue for consent, as Council may apply a consent condition that an adequate factor of safety shall be demonstrated for the proposed works for both global stability and structural stability prior to obtaining the Construction Certificate. Based on similar designs at adjacent properties completed by Horton Coastal Engineering (in consultation with James Taylor & Associates and JK Geotechnics), there is no question as to the feasibility of obtaining an adequate factor of safety by adjusting the pile embedment and anchor loads as required.

The secant 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 (which has a crest level of 7.0m AHD) 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 about 5m seaward (at 1190-1196) and 7m to 10m seaward with an average of 9m (at 1204) of the properties onto the public beach (Crown Land).

The proposed works are to be located entirely on private property, with the main face of the concrete wall located 0.5m landward of the seaward property boundaries, with the seaward edge of the wave return adjacent to the boundary (as per the Drawings, stairs are also recessed into the wall). 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

[1] Contiguous piles with infill concrete/grout plug piles or jet grout may possibly be used instead of secant piles.
[2] Alternative anchoring setouts and types 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 beach 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

at any time as they are located entirely on private property). The proposed vertical wall alignment is coincident with a current average rock boulder surface level of about 3m AHD along the length of 1190-1196, and range of about 4m to 6m AHD (average of about 5m AHD) at 1204.

A minimum 4.5m setback landward of the landward edge of the vertical wall (and 1m landward of the stairs) 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, noting that road plates would have to be used to traverse over the stair indents[4].

A maintenance setback of 5m to 6m was recommended in the *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 fences above the seawall. Removal of the fences would increase the setback distance to 5.35m from the seaward edge of the wall (away from stairs), and with rotation of an excavator seaward of the wall 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 in the design, recessed into (landward of) the wall and shore-normal, with stairs at the 1190/1192 common boundary and 1194/1196 common boundary[5]. This would essentially provide permanent beach access at the properties down to the capping beam level of 2.8m AHD, 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. A recess would be provided within the stair walls to allow insertion of a removable wave barrier (stop log) to reduce the risk of wave overtopping propagating landward of the stairs, as shown on Drawing S20 at two locations.

The maintenance setback landward of the landward edge of the stairs has been reduced to 1m, given that:

- limited excavation would be required to maintain the landward end of the stairs (given that they are located near the ground surface);
- this is not a critical location for structural stability of the seawall; and
- this area can be maintained by accessing it from within the maintenance setback area further seaward.

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). As stated by MHL (2020): "beach width is most affected by the relative cross-shore position of a seawall within the active beach profile and....the seawall make-up [ie whether it is vertical concrete or sloping rock] does not by comparison significantly impact the time that the beach width is impacted following storms".

[4] It has been noted on Drawing S20 that machines up to 45 tonnes would be permitted to traverse over the stair indents.

[5] No stairs are proposed at 1204, with the expectation that Council will provide permanent beach access stairs at Mactier Street as part of their upgraded seawall works there.

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 2081). 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[6] (that is, 40 years to 60 years) may be 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 60 years is thus appropriate, and meets the minimum requirement in the *Specifications*.

[6] 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* 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 is achievable for the anchoring, and a minimum 100 year life has been specified on the Drawings. Actions that would be adopted to ensure a minimum 100 year life for the anchors would include assessment of the corrosive environment that the anchors would be located in.

Design life for anchors is provided through an assessment of the corrosion rates for items in ground. The elements making up the anchors 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

Note that reference to cemented sand levels in this Section means the top surface of cemented sand, unless stated otherwise. Cemented sand at and seaward of the subject properties is expected to extend several metres below the top levels quoted.

JK Geotechnics completed a field investigation at and seaward of 1204 in March 2019, which included 12 Dynamic Cone Penetrometer (DCP) tests at the locations shown in white in Figure 1, along a northern and southern cross-shore profile. The factual results from that investigation are provided in Appendix A.

Along the northern DCP profile at 1204, cemented sand levels varied from 0.8m AHD at 9.5m landward of the seaward property boundary, to -0.9m AHD at 28m seaward of the seaward property boundary, interpolated to be 0.4m AHD at the boundary (assuming a constant seaward dip of the cemented sand layer, this would be at a slope of 1:22 vertical:horizontal [V:H], ie 2.6°). Along the southern profile, cemented sand levels varied from 0.7m AHD at 5.4m landward of the seaward property boundary, to -0.7m AHD at 26m seaward of the seaward property boundary, interpolated to be 0.4m AHD at the boundary (assuming a constant seaward dip of the cemented sand layer, this would be at a slope of 1:24 V:H, ie 2.4°). The values shown in brackets in Figure 1 are not considered to be true cemented sand levels, and may represent rock revetment boulders or floaters within the 1204 property[7], while the 1.6m and 1.3m AHD values on the beach are considered to be spurious.

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, 1174 (two locations), 1176, 1178, 1180 and 1182 Pittwater Road Narrabeen (to the south of the subject properties). These 8 DCP test locations were about 10m to 13m landward of the seaward property boundaries, with the test locations at 1180 and 1182 shown in yellow in Figure 1 giving cemented sand levels of 1.3m and 1.4m AHD respectively.

A geotechnical investigation was completed by Jeffery & Katauskas (2000), which included boreholes at Clarke Street (BH205) and Mactier Street (BH206), located about 5m and 2.4m respectively landward of the alignment 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 0.9m AHD at Clarke Street and 1.0m AHD at Mactier Street. Note that the cemented bands continued below the upper surface of the cemented sand for a depth of about 3.1m at Clarke Street and 5.5m at Mactier Street, showing the significant vertical extent of the cemented sand layer.

Jeffery & Katauskas (2000) completed a test pit (TP104) seaward of 1204 Pittwater Road, reaching its maximum depth at about 11m seaward of the seaward property boundary at -1m AHD, where cemented sand was not encountered. This observation is considered to be spurious given the 2019 investigation results at and seaward of 1204. Jeffery & Katauskas (2000) also completed a test pit (TP105) at South Narrabeen SLSC, about 2m landward of the seaward property boundary. This only extended down to 2.2m AHD (not particularly useful) and no cemented sand was encountered. They also completed a test pit (TP119) seaward of

[7] Note that the 6.5m AHD elevation near the NE corner of 1204 applies to the two most seaward locations within the property at this location. If these elevated levels represent the rock revetment, which is considered unlikely, this is further landward of the assumed landward extent of the revetment based on 1967 and 2016 aerial photography that is depicted in Figure 1 and on Drawing S02.

1192 Pittwater Road, and at about 10m seaward of the seaward property boundary a cemented sand level of 0m AHD was found. These three test pit locations are depicted in red in Figure 1.

Coffey Partners International (1998) completed a test pit seaward of 1192 Pittwater Road (TP3), reaching its maximum depth at about 11m seaward of the seaward property boundary at only 2.4m AHD (not particularly useful), where cemented sand was not encountered. This test pit location is depicted in green in Figure 1.

To the south of the subject properties, JK Geotechnics completed a site investigation from 12-14 October 2020, which included the location depicted in orange in Figure 1, seaward of 1182. This involved drilling of a borehole to a depth of about 8m, ie down to about -6m AHD. Cemented sand was found to extend down from -0.7m AHD to -4.2m AHD (ie 3.5m thick).

Assuming the average seaward dip of 1:23 V:H that JK Geotechnics found at 1204 in March 2019, estimated cemented sand levels at the seaward property boundaries are as listed in Table 1.

Table 1: Estimated cemented sand levels at seaward boundaries of Mactier Street, 1204, 1192 and Clarke Street assuming layer dips at 1:23 V:H moving seaward

Location	Cemented sand level (m AHD) at seaward boundary
Mactier Street	0.9
1204 north	0.4
1204 south	0.4
1192	0.4
Clarke Street	0.7

For the investigation reported herein, it was assumed that the cemented sand level at the seaward property boundaries was 0.4m AHD, ie, the lowest level of the range of levels reported in Table 1 (which is conservative), with a slope downwards of 1:23 V:H moving seaward. It is considered that the approximate uncertainty in the cemented sand levels is ±0.5m, so the top surface of the cemented sand at the seaward boundary of the subject properties can be taken as 0.4m AHD (±0.5m).

Based on the boreholes at Clarke Street and Mactier Street, the thickness of the cemented layer seaward of the subject properties is likely to be greater than 3.1m, and increasing moving north to around 5.5m thick.

As part of detailed design, there will be further consideration of the geotechnical investigations described above, and potentially additional field testing. This may lead to some refinement of the assumed ground conditions in the global and structural stability models (see Section 10 and Section 11) and changes to the proposed pile embedments at the subject properties. The requirement to obtain a factor of safety of 1.5 for both global and structural stability would be maintained in any future analysis.

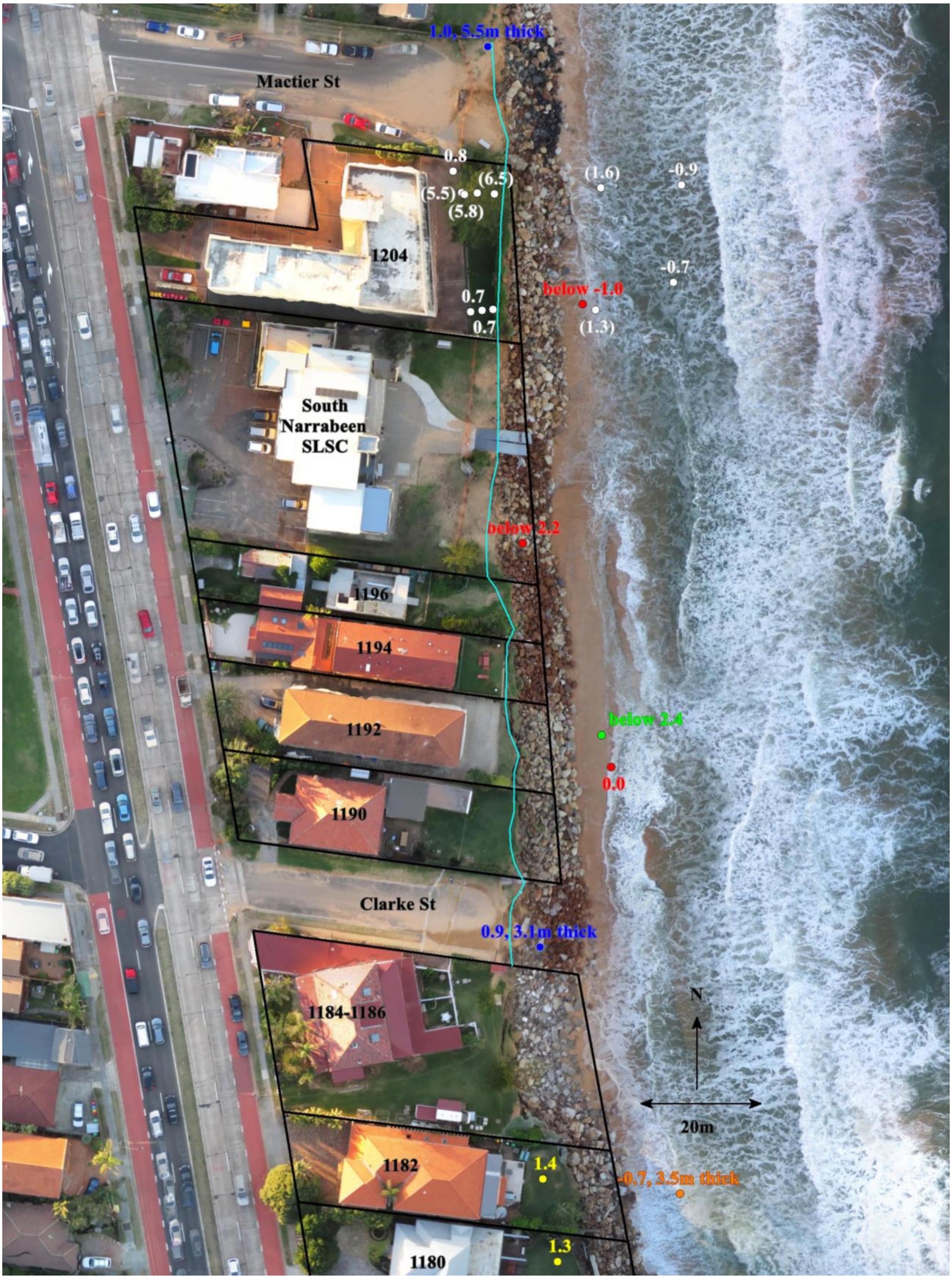

Figure 1: DCP test locations of JK Geotechnics (2016) and in 2019 in yellow and white respectively, boreholes and test pits of Jeffery & Katauskas (2000) in blue and red respectively, test pit of Coffey Partners International (1998) in green, and JK Geotechnics 2020 borehole locations in orange, with top surface of cemented sand levels (m AHD) and cemented layer thicknesses at 3 locations shown, plus landward edge of existing rock boulders with blue line (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 a 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 approximate cemented sand top surface level (assuming a 1V:23H seaward dip, and level of 0.4m AHD at the seaward property boundaries, as discussed in Section 5) is depicted relative to historical beach profiles at 1204, with a similar plot for 1194 provided in Figure 3. These profiles were derived from the NSW Government Beach Profile Database. It is evident that the top surface of the cemented sand sits well below typical beach profiles[8], and has only been exposed twice (in June 2016 and July 2020) in this historical record.

As long-term recession due to sea level rise is realised, scour levels may lower at a particular cross-shore position as the typical beach profile translates landward and hence gets lower (given that beach levels generally get lower moving seaward). In the CZMP, a "best estimate" inverse slope of the active beach profile of 30 was adopted, which would cause 13.5m 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 0.45m from Section 8.3), and about 0.59m of lowering assuming no restriction from cemented sand[9].

In Figure 4 and Figure 5, the same information in Figure 2 and Figure 3 respectively is depicted, except that the historical beach profiles are translated landward by 13.5m and raised by 0.45m to account for projected long term recession due to sea level rise over the design life. It is evident that most of the receded profiles generally sit well above the cemented sand[10], including some of the lowest profiles that occurred in 1974 and 1986. The lowest receded profiles, in June 2016 and July 2020, are the only receded profiles in Figure 4 and Figure 5 that would be expected to have potentially exposed the cemented sand layer adjacent to the seawall. Figure 4 and Figure 5 thus indicates that long term recession due to sea level rise is unlikely to cause typical 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.

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[8] Median of about 6.6m below at the seawall and 4.5m at 10m seaward of the seawall at 1204, and median of about 5.3m at the seawall and 3.8m at 10m seaward of the seawall at 1194.

[9] Based on 1.04m of lowering due to landward translation of the profile (with an average beach slope of 1:13 vertical:horizontal), and 0.45m of profile raising due to the sea level rise itself.

[10] Median of about 3.8m above at the seawall and 3.2m at 10m seaward of the seawall at 1204, and median of about 3.8m at the seawall and 3.1m at 10m seaward of the seawall at 1194.

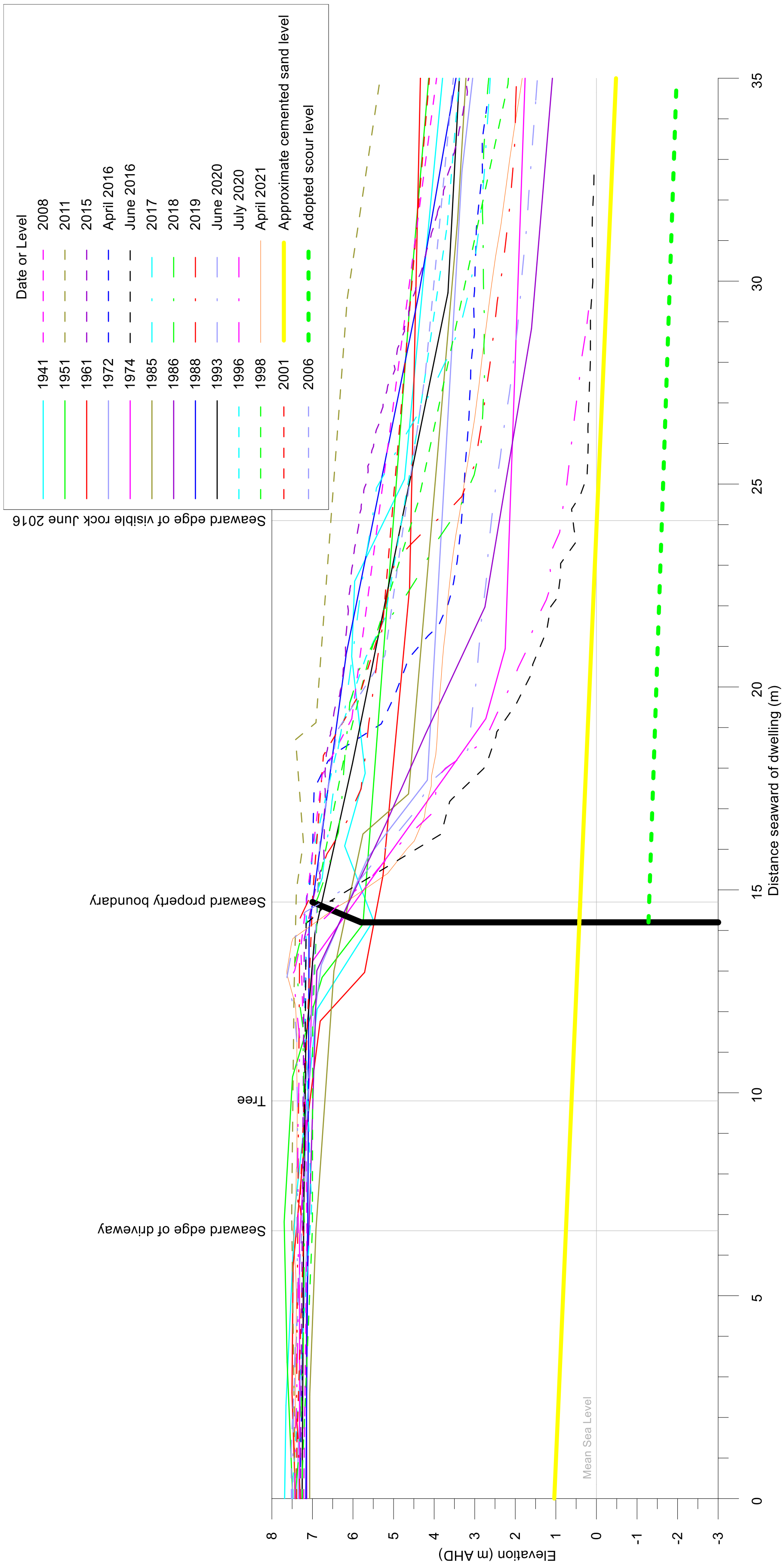

Figure 2: Adopted scour level compared to cemented sand levels and historical beach profiles at 1204

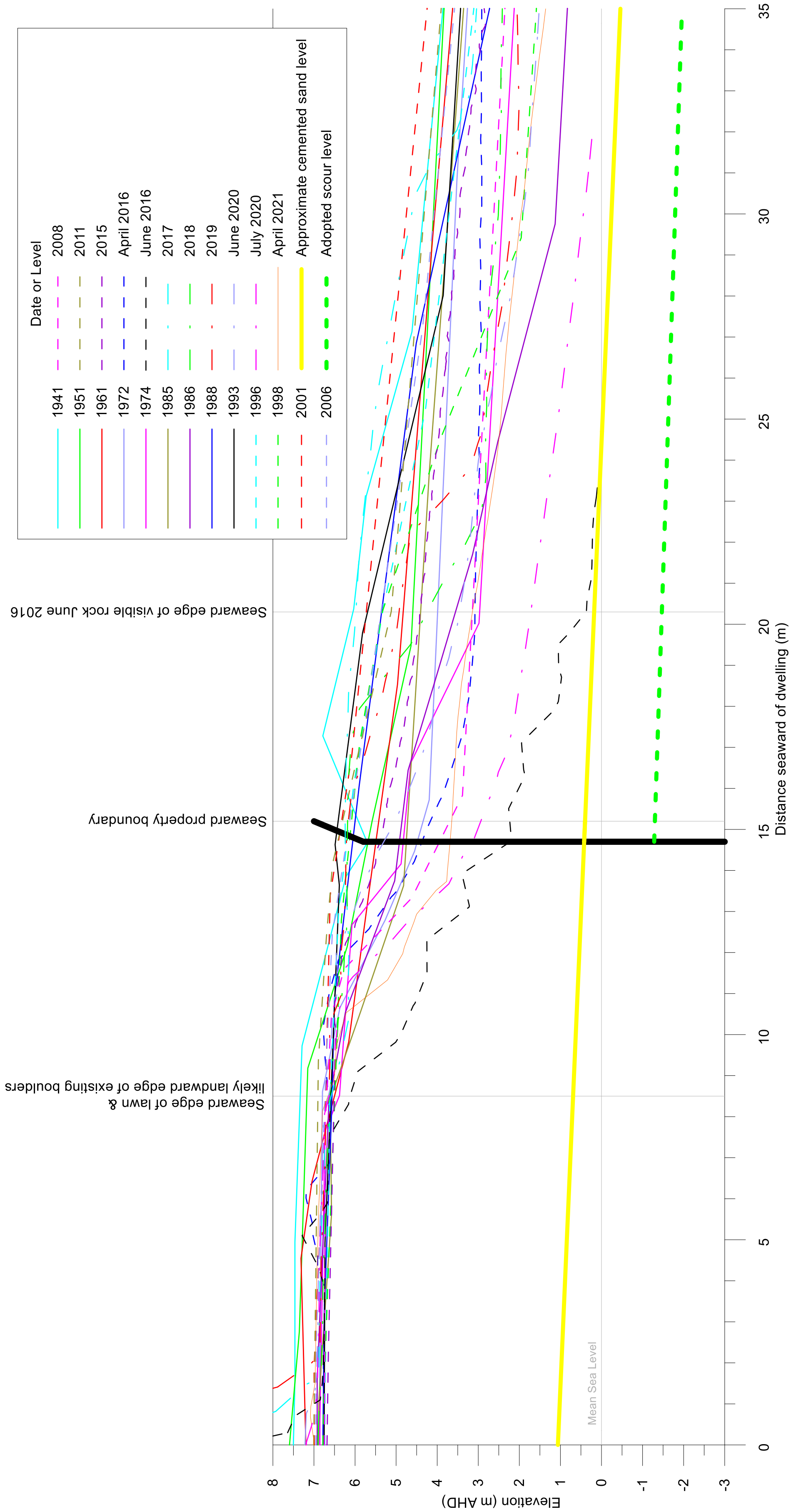

Figure 3: Adopted scour level compared to cemented sand levels and historical beach profiles at 1194

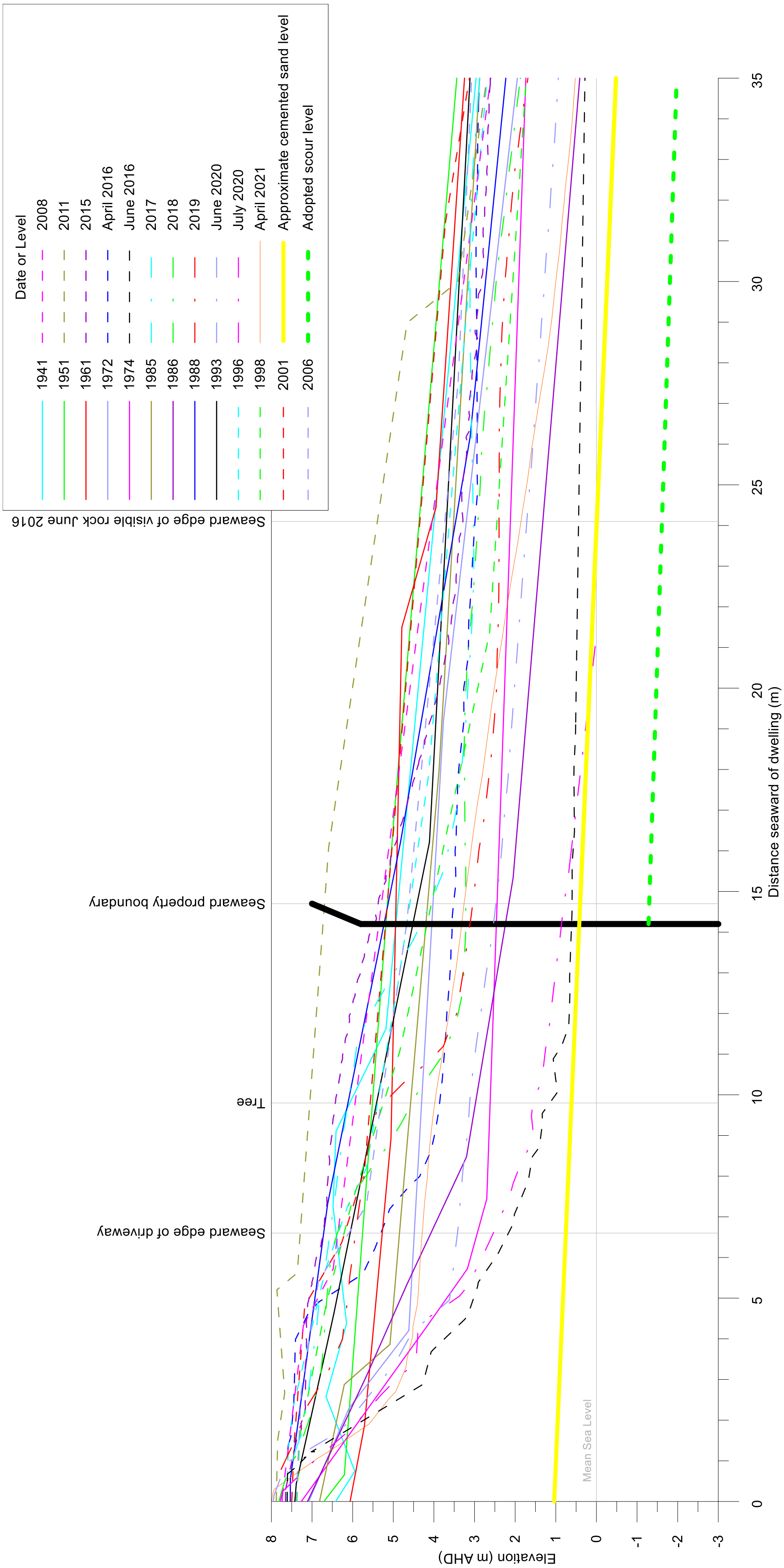

Figure 4: Adopted scour level compared to cemented sand levels and historical beach profiles (receded over 60 years) at 1204

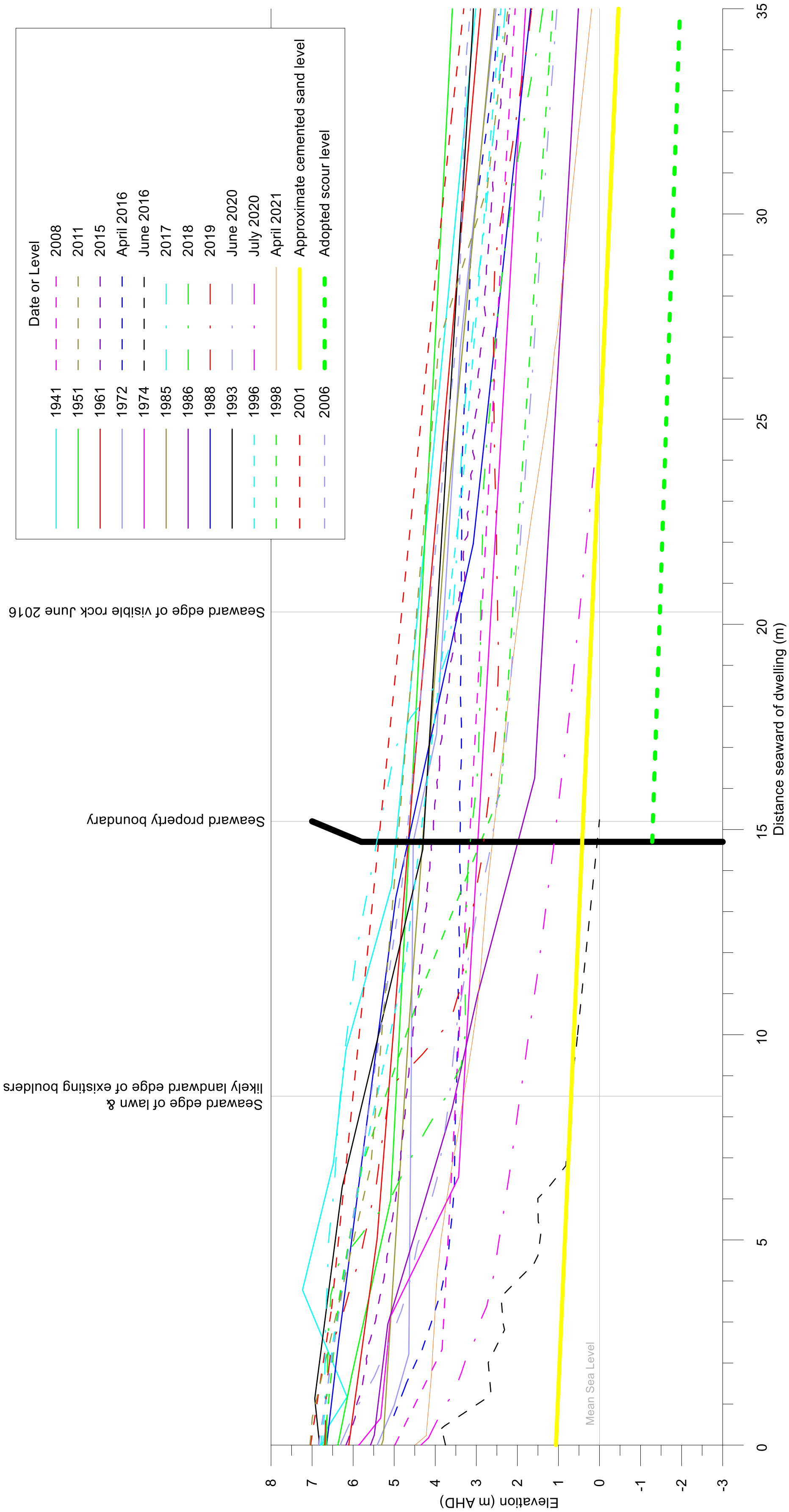

Figure 5: Adopted scour level compared to cemented sand levels and historical beach profiles (receded over 60 years) at 1194

The adopted design scour level is depicted Figure 2 to Figure 5 as a thick green dashed line. The adopted scour level is -1.3m AHD at the seaward property boundaries, sloping downwards at 1:30 (vertical:horizontal) moving seaward.

The level of -1.3 AHD was adopted cognisant that a low probability (1,200 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 1,200 year ARI ('possible') event "could occur under adverse conditions over the design life". However, the adopted scour level is actually considered to be 'unlikely' ("might occur under very adverse circumstances over the design life") or 'rare' ("the event is conceivable but only under exceptional circumstances over the design life").

The reasoning for the adopted -1.3m AHD scour level was as follows:

- in reality, limited scour of the cemented sand layer would be expected, and if any scour did occur at its top surface, this would only be expected to be as relatively small clumps when exposed by occasional wave action from storms;
- given that the cemented sand layer is about 3.1m to 5.5m thick (see Section 5), scour deep into this layer is not expected (that is, storm-induced scour of the cemented sand, or additional scour at the toe of the seawall due the presence of this structure, is expected to be limited);
- it is considered that allowing 0.3m of scour per major storm, allowing for two major storms over the design life (consistent with two profiles potentially exposing the cemented sand layer in Figure 4 and Figure 5 over 80 years), for the above process is conservative (thus a total of 0.6m of scour);
- as a statistical means of allowing for additional scour of the cemented sand due to long term beach profile recession, it is considered that applying the projected lowering of beach profiles of 0.59m (say 0.6m) due to long term recession due to sea level rise over the design life (see page 9 for its derivation) to the cemented sand would be conservative (given that the cemented sand would not be expected to adjust like the erodible sandy areas above it);
- there is an estimated cemented sand level of 0.4m±0.5m AHD at the proposed seawall (see Section 5); and
- taking the lowest level in that error band (-0.1m AHD), applying 0.6m of lowering to allow for scour of the cemented sand in storms over the design life, and applying 0.6m of lowering due to long term recession due to sea level rise, gives a level of -1.3m AHD.

This is considered to be a highly unlikely level, as the cemented sand at this location is significantly less erodible than normal (non-cemented) beach sand and is 3.1m to 5.5m thick, 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.

JK Geotechnics has completed analysis (see Appendix B and Appendix C) of a similar design to that proposed, with an adopted scour level of -1.6m AHD at the seawall (more conservative than adopted herein), applying an analysis cross section at 1204. A factor of safety exceeding 1.5 for both global stability and structural stability (with consideration of disturbing and balancing forces and moments) was obtained with a pile embedment to -7m AHD (which is the embedment shown on the DA drawings). This analysis included a conservative groundwater level difference of 4m between the landward and seaward sides of the wall at the time of maximum scour. A factor of safety of 2.0 was obtained in the model WALLAP (considering structural stability, see Appendix B), and 1.5 in the model SLOPE/W (considering global stability, see Appendix C).

This meets 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.

To determine the scour level that could cause theoretical failure[11] of the design analysed in Appendix B, JK Geotechnics found that a factor of safety of just less than 1.0 was obtained for a scour level lower than -2.6m AHD on the seaward side of the wall considering structural stability in the model WALLAP[12]. This indicates that there would need to be more than 1m of additional scour than the design scour if the seawall was to fail from excessive scour, and more than 2.5m of cemented sand scour overall[13]. 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 -2.6m AHD 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.

Adjustment to the adopted subsurface conditions in modelling may be undertaken as part of detailed design, particularly should further geotechnical investigation provide additional information about the cemented sand layer and depth, as discussed in Section 5.

[11] 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.
[12] The factor of safety was still 1.3 at this scour level, ie, the scour level for theoretical failure would be lower.
[13] 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 -2.6m AHD is considered to be elevated above 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 2, 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 2.

Table 2: 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 2 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 value 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. For the adopted 60 year design life, a 1,200 year ARI event has a 4.9% probability of occurring over the design life.

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 1,200 year ARI. The probability of such a scour level (at 1,200 year ARI) being realised over the adopted 60 year design life is 4.9%, which is satisfactory in relation to *AS 4997* and Gordon et al (2019).

It is considered that the adopted scour level can be treated as > 2,000 year ARI.

8. WATER LEVELS AND WAVES

8.1 Design Event and Design Life

A 2,000 year ARI event (Section 7) was adopted over a 60 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 (> 2,000 year ARI) scour level of -1.3m 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 > 2,000 year ARI scour level, is likely to be in the order of a 2,000 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[14].

That stated, to illustrate sensitivity to 2,000 year ARI water level and wave parameters (in conjunction with the > 2,000 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)[15]. 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 (2021) using a rate of sea level rise of 3mm/year from 2010 to 2021, as recommended in DECCW (2010), 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 60 years (at 2081), it

[14] 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 ARI 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 vertical seawalls to have a higher design standard for overtopping.

[15] 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).

would be possible to linearly interpolate to obtain a sea level rise value at 2081 of 0.71m AHD relative to 1990[16].

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 3 (at 2081) were determined for various emissions scenarios (Representative Concentration Pathways).

Table 3: Global mean sea level rise (m) from 2010 to 2081 derived from IPCC (2013)

Emissions Scenario	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SRES A1B	0.29	0.41	0.54
RCP2.6	0.21	0.32	0.44
RCP4.5	0.25	0.38	0.50
RCP6.0	0.26	0.37	0.49
RCP8.5	0.35	0.48	0.63
Average	0.27	0.39	0.52

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 0.45m at 2081 (relative to 2010) was derived. Therefore, the 100 year ARI still water level (in the absence of wave action) at 2081 based on IPCC (2013) is 1.89m AHD. The corresponding 2,000 year ARI value is 2.02m 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 2081 (at the end of the design life) is 1.89m AHD. The corresponding value for the 2,000 year ARI event is 2.02m 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 -1.3m 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 -1.3m AHD scour level at the works, the bed level 10m seaward of the works is -1.6m AHD.

[16] This is a sea level rise of 0.68m relative to 2010, discounting historical sea level rise at 3mm/year as recommended in DECCW (2010).

The 100 year ARI design ocean depth (excluding wave setup) at 2081 is thus 3.5m at the plunging distance. The corresponding 2,000 year ARI value is 3.6m. For present day calculations, the scour level is reduced to -0.7m AHD (that is, excluding the 0.6m long term recession component), or -1.0m AHD at the plunging distance.

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). From a log fit and extrapolating the Shand et al (2011) results, the corresponding 2,000 year ARI value is 14.3s.

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 3.5m at the plunging distance at 2081, based on Goda (2010a) then h/H_o is 0.40, and wave setup at the plunging location is 7.6% of H_o. Therefore, setup at the plunging location is 0.66m, and the design depth at the plunging location at 2081 is 4.2m.

As it used in wave overtopping calculations (see Section 9), the present day depth at the plunging location was determined as the present (at 2021) design ocean water level (1.47m AHD, see Section 8.2), plus scour down to -1.0m AHD at the plunging distance as per Section 8.5 (thus a depth of 2.5m excluding wave setup, so h/H_o is 0.29), then wave setup at the plunging location is 8.6% of H_o from Goda (2010a), setup is 0.75m, and the design depth at the plunging location at present is 3.3m.

For the 2,000 year ARI event at present, the depth at the plunging location was determined as the present (at 2021) design ocean still water level (1.60m AHD, see Section 8.2),plus scour down to -1.0m AHD at the plunging distance as per Section 8.5 (thus a depth of 2.6m excluding wave setup, so h/H_o is 0.24), then wave setup at the plunging location is 8.9% of H_o from Goda (2010a), setup is 0.96m, and the design depth at the plunging location is 3.6m.

For the 2,000 year ARI event at 2081, 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 0.45m, plus scour down to -1.6m AHD at the plunging distance (thus a depth of 3.7m excluding wave setup, so h/H_o is 0.34), then wave setup at the plunging location is 8.0% of H_o from Goda (2010a), setup is 0.86m, and the design depth at the plunging location is 4.5m.

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 2081):

- water depth of 4.2m 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 2.5m (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, $H_{10\%}$[17], $H_{2\%}$ and $H_{1\%}$ values of 3.1m, 3.3m and 3.5m 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 2081.

As it is used in wave overtopping calculations (see Section 9), a present-day 100 year ARI H_s for incipient breaking of 2.0m was calculated using the Goda (2010b) methodology. The corresponding 2,000 year ARI value was 2.2m, and 2.8m at 2081.

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

9. WAVE OVERTOPPING AND WALL CREST LEVEL

As per the Drawings, a wall crest level of 7.0m AHD has been adopted. In the *Specifications*, a minimum crest level of 6.5m AHD is specified, so this requirement has been met.

The Neural Network tool[18] 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 2081 (at the end of the design life).

Input parameters are summarised in Table 4 for the 100 year ARI event (with > 2,000 year ARI scour 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 was derived from the concrete wall being vertical (that is, at an angle of 90° 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.

Table 4: Input parameters for Neural Network tool for 100 year ARI event

Parameter	Value	
	Present-day	2081
Cotangent of foreshore slope	30	30
Water depth at the structure toe (m), see Section 8.7	3.3	4.2
Significant wave height at the toe of structure (m), see Section 8.8	2.0	2.5
Spectral mean wave period, $T_{m-1,0,t}$ (s)	26.3	21.4
Wave obliquity, ie angle of wave attack (°)	0.0	0.0
Toe submergence, ie water depth above toe of structure (m)	3.3	4.2
Width of toe (m)	0.0	0.0
Berm submergence (m)	0.0	0.0
Berm width (m)	0.0	0.0
Angle of down slope (cotangent)	0	0
Angle of upper slope (cotangent)	0	0
Roughness factor (lower and upper)	1.0	1.0

[18] Formentin et al (2017) and Zanuttigh et al (2016).

Parameter	Value	
	Present-day	2081
Size of the structure elements (lower and upper) (m)	0	0
Crest freeboard in relation to still water level (m) for 7.0m AHD crest	4.8	4.5
Wall freeboard in relation to still water level (m) for 8.0m AHD crest	5.8	5.5
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 5.

Table 5: Input parameters for determination of the wave return overtopping multiplier (for 100 year ARI event with > 2,000 year ARI scour level)

Parameter	Value	
	Present-day	2081
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) for 7.0m AHD crest	4.8	4.5
Wave height at toe of structure (H_{mo}, m)	2.0	2.5

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 6. The corresponding discharges for the 2,000 year ARI event (with > 2,000 year ARI scour level) are summarised in Table 7. The adopted crest level of 7.0m AHD was simulated, as well as 8.0m AHD, which would be representative of placing a (suitably designed) 1m high solid fence at the top of the wall.

Table 6: 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	2081
7.0	0.07	3.9
8.0	0.06	0.1

Table 7: 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	2081
7.0	2.5	7.2
8.0	0.09	3.9

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 and at 2081 for the 7.0m AHD and 8.0m AHD crest levels. For the 2,000 year ARI storm, some minor damage may be expected for the 7.0m AHD crest level at 2081, but not for the 8.0m AHD crest level. 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 6 and Table 7 are considered to be acceptable.

Future development 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.

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 overtopping rates for the 100 year ARI event (with > 2,000 year ARI scour level) are below these limits for the present day, and at 2081 for the 8.0m AHD crest level. For the 2,000 year ARI event (with > 2,000 year ARI scour level), the overtopping rates are below these limits for the present day and 8.0m AHD crest level. 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.

The stairs at the common boundaries of 1190/1192 and 1194/1196 would potentially allow increased wave overtopping into the properties compared to the wall crest locations. To reduce the risk of overtopping damage at the stairs, a removable wave barrier would be designed to slot into a recess in the stair walls. As part of detailed design, it is also recommended that there is consideration of installing solid gates at the top of the stairs to further reduce the risk, if required.

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 as discussed in Section 6 and shown in Appendix C for a similar design to that proposed, for a more conservative scour level of -1.6m AHD (compared to the adopted -1.3m AHD).

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 considered to be relevant at the subject properties, as the existing rock revetments are not to be retained seaward of where it will be excavated to remove potential piling obstructions, and sufficient subsurface investigations for DA purposes have been undertaken as described in Section 5.

As part of detailed design, there will be further consideration of the geotechnical investigations outlined in Section 5, and potentially additional field testing. This may lead to some refinement of the assumed ground conditions in the global stability model, and changes to the proposed pile embedments at the subject properties. The requirement to obtain a factor of safety of 1.5 for global stability would be maintained in any future analysis.

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.

A similar design to that proposed was found to have an adequate factor of safety for the ultimate design case (see Section 6 and Appendix B). 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)*.

As part of detailed design, there will be further consideration of the geotechnical investigations outlined in Section 5, and potentially additional field testing. This may lead to some refinement of the assumed ground conditions in the structural stability model, and changes to the proposed pile embedments at the subject properties. The requirement to obtain a factor of safety of 1.5 for structural stability would be maintained in any future analysis.

A structural engineering design statement by James Taylor & Associates (2021) has been submitted with the DA documentation.

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APPENDIX A: FACTUAL RESULTS FROM MARCH 2019 FIELD INVESTIGATION OF JK GEOTECHNICS

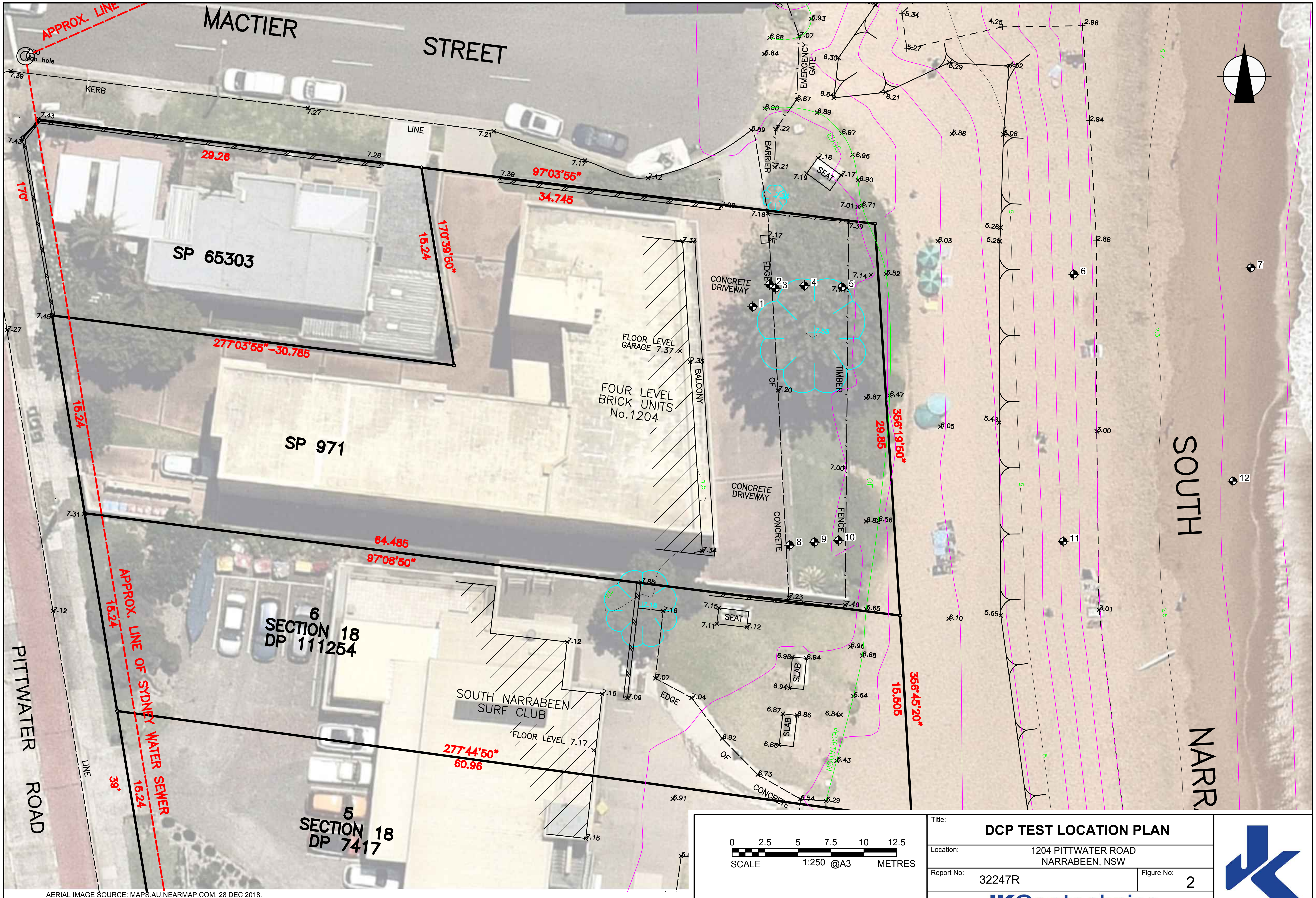
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 28 DEC 2018.

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	1204 PITTWATER ROAD NARRABEEN, NSW
Report No: 32247R	Figure No: 1

JKGeotechnics

PLOT DATE: 11/03/2019 5:59:34 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32247RE NARRABEEN\CAD\32247R.DWG



SCALE 0 2.5 5 7.5 10 12.5 METRES 1:250 @A3

This plan should be read in conjunction with the JK Geotechnics report.

Title: **DCP TEST LOCATION PLAN**

Location: 1204 PITTWATER ROAD NARRABEEN, NSW

Report No: 32247R

Figure No: 2

JKGeotechnics

PLOT DATE: 12/03/2019 2:24:17 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000\S32247RE NARRABEEN\CAD\32247R.DWG

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client: THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)
Project: PROPOSED COASTAL PROTECTION MEASURES
Location: 1204 PITTWATER ROAD, NARRABEEN, NSW

Job No. 32247R
Date: 4-3-19
Tested By: M.E.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	1	2	3	Test Location	1		
Surface RL	≈7.14	≈7.13	≈7.12	Surface RL	≈7.14		
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	26	2	1	3000-3100	6		
100 - 200	5	2	1	3100-3200	4		
200 - 300	4	1	1	3200-3300	4		
300 - 400	4	2	2	3300-3400	4		
400 - 500	5	2	2	3400-3500	4		
500 - 600	5	1	2	3500-3600	5		
600 - 700	4	2	2	3600-3700	5		
700 - 800	4	2	1	3700-3800	8		
800 - 900	4	1	1	3800-3900	7		
900 - 1000	3		2	3900-4000	7		
1000 - 1100	2		1	4000-4100	7		
1100 - 1200	2		2	4100-4200	6		
1200 - 1300	3	1	2/10mm	4200-4300	4		
1300 - 1400	4		REFUSAL	4300-4400	5		
1400 - 1500	3	16		4400-4500	4		
1500 - 1600	4	32/80mm		4500-4600	5		
1600 - 1700	5	REFUSAL		4600-4700	5		
1700 - 1800	5			4700-4800	7		
1800 - 1900	7			4800-4900	8		
1900 - 2000	6			4900-5000	14		
2000 - 2100	5			5000-5100	13		
2100 - 2200	5			5100-5200	17		
2200 - 2300	5			5200-5300	22		
2300 - 2400	4			5300-5400	14		
2400 - 2500	5			5400-5500	13		
2500 - 2600	5			5500-5600	13		
2600 - 2700	5			5600-5700	8		
2700 - 2800	7			5700-5800	10		
2800 - 2900	5			5800-5900	15		
2900 - 3000	6			5900-6000	22		

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)	
Project:	PROPOSED COASTAL PROTECTION MEASURES	
Location:	1204 PITTWATER ROAD, NARRABEEN, NSW	
Job No.	32247R	Hammer Weight & Drop: 9kg/510mm
Date:	4-3-19	Rod Diameter: 16mm
Tested By:	M.E.	Point Diameter: 20mm

Test Location	1			Test Location	1		
Surface RL	≈7.14			Surface RL	≈7.14		
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
6000-6100	22			9000-9100			
6100-6200	30			9100-9200			
6200-6300	34			9200-9300			
6300-6400	REFUSAL			9300-9400			
6400-6500				9400-9500			
6500-6600				9500-9600			
6600-6700				9600-9700			
6700-6800				9700-9800			
6800-6900				9800-9900			
6900-7000				9900-10000			
7000-7100				10000-10100			
7100-7200				10100-10200			
7200-7300				10200-10300			
7300-7400				10300-10400			
7400-7500				10400-10500			
7500-7600				10500-10600			
7600-7700				10600-10700			
7700-7800				10700-10800			
7800-7900				10800-10900			
7900-8000				10900-11000			
8000-8100				11000-11100			
8100-8200				11100-11200			
8200-8300				11200-11300			
8300-8400				11300-11400			
8400-8500				11400-11500			
8500-8600				11500-11600			
8600-8700				11600-11700			
8700-8800				11700-11800			
8500-8900				11800-11900			
8900-9000				11900-12000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)						
Project:	PROPOSED COASTAL PROTECTION MEASURES						
Location:	1204 PITTWATER ROAD, NARRABEEN, NSW						
Job No.	32247R			Hammer Weight & Drop: 9kg/510mm			
Date:	4-3-19			Rod Diameter: 16mm			
Tested By:	M.E.			Point Diameter: 20mm			

Test Location	**4**	**5**	**6**	Test Location			
Surface RL	≈7.10	≈7.08	≈3.91	Surface RL			
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	SUNK	3000-3100			
100 - 200	1	↓	1	3100-3200			
200 - 300	2	2	2	3200-3300			
300 - 400	2	2	4	3300-3400			
400 - 500	2	3	8	3400-3500			
500 - 600	3/60mm	2/20mm	8	3500-3600			
600 - 700	REFUSAL	REFUSAL	12	3600-3700			
700 - 800			15	3700-3800			
800 - 900			15	3800-3900			
900 - 1000			16	3900-4000			
1000 - 1100			18	4000-4100			
1100 - 1200			17	4100-4200			
1200 - 1300			16	4200-4300			
1300 - 1400			15	4300-4400			
1400 - 1500			14	4400-4500			
1500 - 1600			14	4500-4600			
1600 - 1700			15	4600-4700			
1700 - 1800			20	4700-4800			
1800 - 1900			26	4800-4900			
1900 - 2000			24	4900-5000			
2000 - 2100			29	5000-5100			
2100 - 2200			35	5100-5200			
2200 - 2300			REFUSAL	5200-5300			
2300 - 2400				5300-5400			
2400 - 2500				5400-5500			
2500 - 2600				5500-5600			
2600 - 2700				5600-5700			
2700 - 2800				5700-5800			
2800 - 2900				5800-5900			
2900 - 3000				5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)		
Project:	PROPOSED COASTAL PROTECTION MEASURES		
Location:	1204 PITTWATER ROAD, NARRABEEN, NSW		
Job No.	32247R	Hammer Weight & Drop: 9kg/510mm	
Date:	4-3-19	Rod Diameter: 16mm	
Tested By:	M.E.	Point Diameter: 20mm	

Test Location	7	8	9	Test Location	7	8	9
Surface RL	≈2.28	≈7.11	≈7.02	Surface RL	≈2.28	≈7.11	≈7.02
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	SUNK	1	1	3000-3100	25	9	6
100 - 200	↓	3	1	3100-3200	REFUSAL	7	6
200 - 300	1	2	1	3200-3300		6	6
300 - 400	1	2	15	3300-3400		6	6
400 - 500	1	2	3	3400-3500		8	6
500 - 600	2	1	4	3500-3600		10	7
600 - 700	2	1	8	3600-3700		9	7
700 - 800	2	1	13	3700-3800		11	7
800 - 900	3	1	6	3800-3900		12	8
900 - 1000	2	1	6	3900-4000		9	8
1000 - 1100	3	1	4	4000-4100		5	8
1100 - 1200	4	↓	2	4100-4200		5	7
1200 - 1300	4	1	2	4200-4300		4	9
1300 - 1400	4	1	2	4300-4400		4	7
1400 - 1500	2	1	3	4400-4500		5	8
1500 - 1600	3	3	2	4500-4600		5	5
1600 - 1700	6	1	3	4600-4700		5	5
1700 - 1800	6	2	4	4700-4800		6	5
1800 - 1900	11	2	4	4800-4900		6	10
1900 - 2000	13	2	4	4900-5000		6	8
2000 - 2100	12	2	3	5000-5100		5	4
2100 - 2200	14	4	4	5100-5200		7	5
2200 - 2300	15	4	4	5200-5300		12	5
2300 - 2400	13	5	4	5300-5400		12	5
2400 - 2500	9	5	4	5400-5500		11	10
2500 - 2600	10	5	4	5500-5600		11	14
2600 - 2700	12	7	5	5600-5700		13	12
2700 - 2800	18	6	5	5700-5800		15	12
2800 - 2900	27	6	5	5800-5900		17	17
2900 - 3000	27	8	6	5900-6000		17	23

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)
Project:	PROPOSED COASTAL PROTECTION MEASURES
Location:	1204 PITTWATER ROAD, NARRABEEN, NSW
Job No.	32247R — Hammer Weight & Drop: 9kg/510mm
Date:	4-3-19 — Rod Diameter: 16mm
Tested By:	M.E. — Point Diameter: 20mm

Test Location		8	9	Test Location			
Surface RL		≈7.11	≈7.02	Surface RL			
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
6000-6100		23	31	9000-9100			
6100-6200		31	35	9100-9200			
6200-6300		25	REFUSAL	9200-9300			
6300-6400		REFUSAL		9300-9400			
6400-6500				9400-9500			
6500-6600				9500-9600			
6600-6700				9600-9700			
6700-6800				9700-9800			
6800-6900				9800-9900			
6900-7000				9900-10000			
7000-7100				10000-10100			
7100-7200				10100-10200			
7200-7300				10200-10300			
7300-7400				10300-10400			
7400-7500				10400-10500			
7500-7600				10500-10600			
7600-7700				10600-10700			
7700-7800				10700-10800			
7800-7900				10800-10900			
7900-8000				10900-11000			
8000-8100				11000-11100			
8100-8200				11100-11200			
8200-8300				11200-11300			
8300-8400				11300-11400			
8400-8500				11400-11500			
8500-8600				11500-11600			
8600-8700				11600-11700			
8700-8800				11700-11800			
8500-8900				11800-11900			
8900-9000				11900-12000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE OWNERS OF 1204 PITTWATER ROAD, NARRABEEN (SP971)
Project:	PROPOSED COASTAL PROTECTION MEASURES
Location:	1204 PITTWATER ROAD, NARRABEEN, NSW

Job No.	32247R	Hammer Weight & Drop: 9kg/510mm
Date:	4-3-19	Rod Diameter: 16mm
Tested By:	M.E.	Point Diameter: 20mm

Test Location	**10**	**11**	**12**	Test Location			
Surface RL	≈6.87	≈4.12	≈2.25	Surface RL			
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	SUNK	SUNK	SUNK	3000-3100			
100 - 200	1	1	↓	3100-3200			
200 - 300	2	3	1	3200-3300			
300 - 400	1	5	2	3300-3400			
400 - 500	2	7	2	3400-3500			
500 - 600	4	11	2	3500-3600			
600 - 700	3	14	2	3600-3700			
700 - 800	6	15	2	3700-3800			
800 - 900	4	19	2	3800-3900			
900 - 1000	2	19	3	3900-4000			
1000 - 1100	2	21	6	4000-4100			
1100 - 1200	9/40mm	20	6	4100-4200			
1200 - 1300	REFUSAL	20	6	4200-4300			
1300 - 1400		19	7	4300-4400			
1400 - 1500		16	6	4400-4500			
1500 - 1600		15	8	4500-4600			
1600 - 1700		20	12	4600-4700			
1700 - 1800		15	12	4700-4800			
1800 - 1900		15	13	4800-4900			
1900 - 2000		14	10	4900-5000			
2000 - 2100		17	8	5000-5100			
2100 - 2200		19	5	5100-5200			
2200 - 2300		24	10	5200-5300			
2300 - 2400		28	11	5300-5400			
2400 - 2500		24	12	5400-5500			
2500 - 2600		31	22	5500-5600			
2600 - 2700		35	27	5600-5700			
2700 - 2800		REFUSAL	28	5700-5800			
2800 - 2900			REFUSAL	5800-5900			
2900 - 3000				5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

APPENDIX B: WALLAP OUTPUT SUMMARY OF JK GEOTECHNICS FOR PRELIMINARY ANALYSIS SECTION AT 1204

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types Left side	Soil types Right side
1	2.80	3 Sand (Medium Dense)	2 Beach Sand (L)
2	0.30	4 Cemented Sand	4 Cemented Sand
3	-5.00	8 Sand (VL)	8 Sand (VL)
4	-7.00	3 Sand (Medium Dense)	3 Sand (Medium Dense)

SOIL PROPERTIES

No.	-- Soil type -- Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1	Not defined							
2	Beach Sand (L)	16.00	20000	0.500	OC (0.300)	0.292 (0.000)	4.369 (0.000)	
3	Sand (Medium Dense)	18.00	40000	0.500	OC (0.300)	0.256 (0.000)	5.303 (0.000)	
4	Cemented Sand	20.00	80000	0.500	OC (0.300)	0.186 (0.970)	8.892 (9.405)	5.000d
5	Not defined							
6	Not defined							
7	Not defined							
8	Sand (VL)	16.00	10000	0.500	OC (0.300)	0.346 (0.000)	3.442 (0.000)	
9	Compacted Sand Fill	18.00	40000	0.500	OC (0.300)	0.256 (0.000)	5.303 (0.000)	

Additional soil parameters associated with Ka and Kp

No.	------- Soil type ------- Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1	Not defined						
2	Beach Sand (L)	30.00	0.500	0.00	30.00	0.500	0.00
3	Sand (Medium Dense)	33.00	0.500	0.00	33.00	0.500	0.00
4	Cemented Sand	40.00	0.500	0.00	40.00	0.500	0.00
5	Not defined						
6	Not defined						
7	Not defined						
8	Sand (VL)	26.00	0.500	0.00	26.00	0.500	0.00
9	Compacted Sand Fill	33.00	0.500	0.00	33.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Left side	Right side
Initial water table elevation	1.50	-1.00

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Left: Point no.	Left: Elev. m	Left: Piezo elev. m	Left: Water press. kN/m2	Right: Point no.	Right: Elev. m	Right: Piezo elev. m	Right: Water press. kN/m2
1	1	3.50	3.50	0.0	1	-0.50	-0.50	0.0

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -7.00
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.5000E+07 kN/m2
Moment of inertia of wall I = 0.020700 m4/m run
E.I = 517500 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	3.50	3.00	0.001400	2.000E+08	5.50	22.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	7.00	0.00(L)	1.00	14.40	5.00	=	N/A	N/A
2	6.70	14.40(L)	1.00	0.50	150.00	=	N/A	N/A

Note: L = Left side, R = Right side

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Fill to elevation 4.00 on LEFT side with soil type 9
2	Change EI of wall to 89250 kN.m2/m run From elevation 7.00 to 4.80 Yield moment not defined Reset wall displacements to zero at this stage
3	Install strut or anchor no.1 at elevation 3.50
4	Fill to elevation 7.00 on LEFT side with soil type 9
5	Apply surcharge no.2 at elevation 6.70
6	Fill to elevation 6.50 on RIGHT side with soil type 2
7	Excavate to elevation 6.50 on RIGHT side Toe of berm at elevation 4.00 Width of top of berm = 0.10 Width of toe of berm = 14.00
8	Apply surcharge no.1 at elevation 7.00
9	Apply water pressure profile no.1
10	Excavate to elevation -1.60 on RIGHT side
11	Excavate to elevation -2.60 on RIGHT side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.25

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on Left side of wall = 50.00 m
Width of excavation on Right side of wall = 50.00 m

Distance to rigid boundary on Left side = 50.00 m
Distance to rigid boundary on Right side = 50.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Output options: Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Fill to elev. 4.00 on LEFT side	No	No	No
2	Change EI of wall to 89250kN.m2/m run	Yes	Yes	Yes
3	Install strut no.1 at elev. 3.50	Yes	Yes	Yes
4	Fill to elev. 7.00 on LEFT side	Yes	Yes	Yes
5	Apply surcharge no.2 at elev. 6.70	Yes	Yes	Yes
6	Fill to elev. 6.50 on RIGHT side	No	No	No
7	Excav. to elev. 6.50 on RIGHT side	Yes	Yes	Yes
8	Apply surcharge no.1 at elev. 7.00	Yes	Yes	Yes
9	Apply water pressure profile no.1	Yes	Yes	Yes
10	Excav. to elev. -1.60 on RIGHT side	Yes	Yes	Yes
11	Excav. to elev. -2.60 on RIGHT side	Yes	Yes	Yes
*	Summary output	Yes	-	Yes

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method
Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -7.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Wall Penetr -ation	Direction of failure
1	4.00	2.80	Cant.	16.099	-5.71	1.81	0.99	L to R
2	4.00	2.80	Cant.	16.099	-5.71	1.81	0.99	L to R
3	4.00	2.80		No analysis at this stage				
4	7.00	2.80	3.50	10.370	n/a	***	***	L to R
5	7.00	2.80	3.50	10.368	n/a	***	***	L to R
6	7.00	6.50	3.50	Conditions not suitable for FoS calc.				
7	7.00	6.50	3.50	Conditions not suitable for FoS calc.				
8	7.00	6.50	3.50	Conditions not suitable for FoS calc.				
9	7.00	6.50	3.50	Conditions not suitable for FoS calc.				
10	7.00	-1.60	3.50	2.039	n/a	-3.49	1.89	L to R
11	7.00	-2.60	3.50	1.297	n/a	-4.75	2.15	L to R

Legend: *** Result not found

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 50.00 from wall
Right side 50.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	7.00	0.005	0.000	0.0	-0.0	0.0	0.0
2	6.70	0.005	0.000	1.1	-0.0	5.9	0.0
3	6.50	0.005	0.000	2.6	-0.0	8.2	0.0
4	6.05	0.006	0.000	7.1	-0.0	11.0	0.0
5	5.60	0.007	0.000	12.7	-0.0	13.8	0.0
6	4.80	0.009	0.000	25.9	-0.0	20.8	0.0
7	4.00	0.011	0.000	47.1	-0.0	30.6	0.0
8	3.50	0.012	0.000	64.3	0.0	38.3	-136.5
9	2.80	0.014	0.000	43.1	-26.6	3.3	-122.1
10	2.15	0.015	0.000	45.8	-99.3	5.8	-103.5
11	1.50	0.017	0.000	47.9	-159.2	13.5	-79.7
12	0.90	0.018	0.000	50.1	-199.3	23.4	-53.3
13	0.30	0.019	0.000	59.0	-222.3	35.3	-22.6
14	-0.50	0.021	0.000	84.2	-225.2	56.9	0.0
15	-1.00	0.021	0.000	97.0	-210.4	83.8	0.0
16	-1.60	0.022	0.000	109.9	-174.6	116.6	0.0
17	-2.10	0.022	0.000	118.0	-129.6	109.8	0.0
18	-2.60	0.022	0.000	122.9	-66.7	132.1	-1.7
19	-3.30	0.023	0.000	160.9	0.0	117.8	-8.4
20	-4.00	0.023	0.000	167.2	0.0	60.9	-27.8
21	-4.50	0.023	0.000	147.2	0.0	0.0	-61.9
22	-5.00	0.024	0.000	105.4	0.0	0.0	-107.3
23	-5.50	0.024	0.000	58.7	0.0	0.0	-79.5
24	-6.00	0.024	0.000	25.9	0.0	0.0	-52.4
25	-6.50	0.024	0.000	6.4	0.0	0.0	-25.9
26	-7.00	0.025	0.000	0.0	0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	55.6	-3.30	-0.0	4.00	14.4	0.30	-32.8	-5.00
2	55.6	-3.30	-0.0	4.00	14.4	0.30	-32.8	-5.00
3	No calculation at this stage							
4	122.8	-2.60	-0.0	7.00	35.3	0.30	-67.0	-5.00
5	122.9	-2.60	-0.0	7.00	35.3	0.30	-67.1	-5.00
6	63.0	-2.60	-0.0	7.00	25.3	3.50	-31.6	-5.00
7	63.0	-2.60	-0.0	7.00	25.3	3.50	-31.6	-5.00
8	63.6	-2.10	-0.0	7.00	25.7	3.50	-31.8	-5.00
9	81.0	-2.60	-0.0	7.00	23.3	3.50	-43.6	-5.00
10	167.2	-4.00	-118.7	0.90	116.6	-1.60	-107.3	-5.00
11	104.4	-4.50	-225.2	-0.50	132.1	-2.60	-136.5	3.50

Summary of results (continued)

Maximum and minimum displacement at each stage

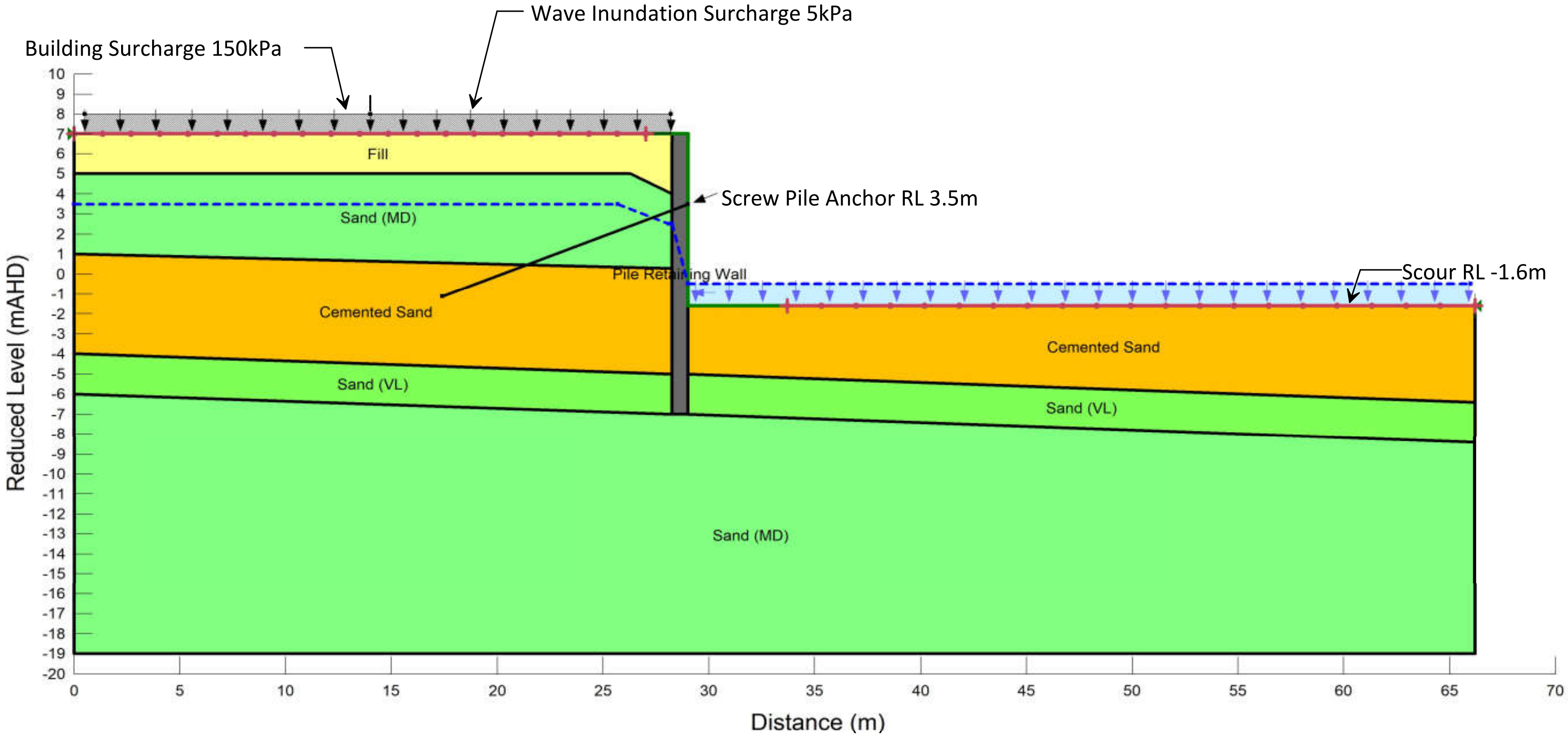
Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.003	-7.00	0.000	7.00	Fill to elev. 4.00 on LEFT side
2	0.000	-7.00	0.000	7.00	Change EI of wall to 89250kN.m2/m run
3	No calculation at this stage				Install strut no.1 at elev. 3.50
4	0.006	-7.00	0.000	7.00	Fill to elev. 7.00 on LEFT side
5	0.006	-7.00	0.000	7.00	Apply surcharge no.2 at elev. 6.70
6	0.003	7.00	0.000	7.00	Fill to elev. 6.50 on RIGHT side
7	0.003	7.00	0.000	7.00	Excav. to elev. 6.50 on RIGHT side
8	0.004	7.00	0.000	7.00	Apply surcharge no.1 at elev. 7.00
9	0.004	7.00	0.000	7.00	Apply water pressure profile no.1
10	0.017	-7.00	0.000	7.00	Excav. to elev. -1.60 on RIGHT side
11	0.025	-7.00	0.000	7.00	Excav. to elev. -2.60 on RIGHT side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 3.50 kN/m run	kN/strut
4	50.49	151.48
5	50.49	151.48
6	28.35	85.04
7	28.35	85.04
8	28.98	86.93
9	33.40	100.21
10	126.38	379.14
11	174.77	524.31

APPENDIX C: SLOPE/W OUTPUT SUMMARY OF JK GEOTECHNICS FOR PRELIMINARY ANALYSIS SECTION AT 1204

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1



Geotechnical Model

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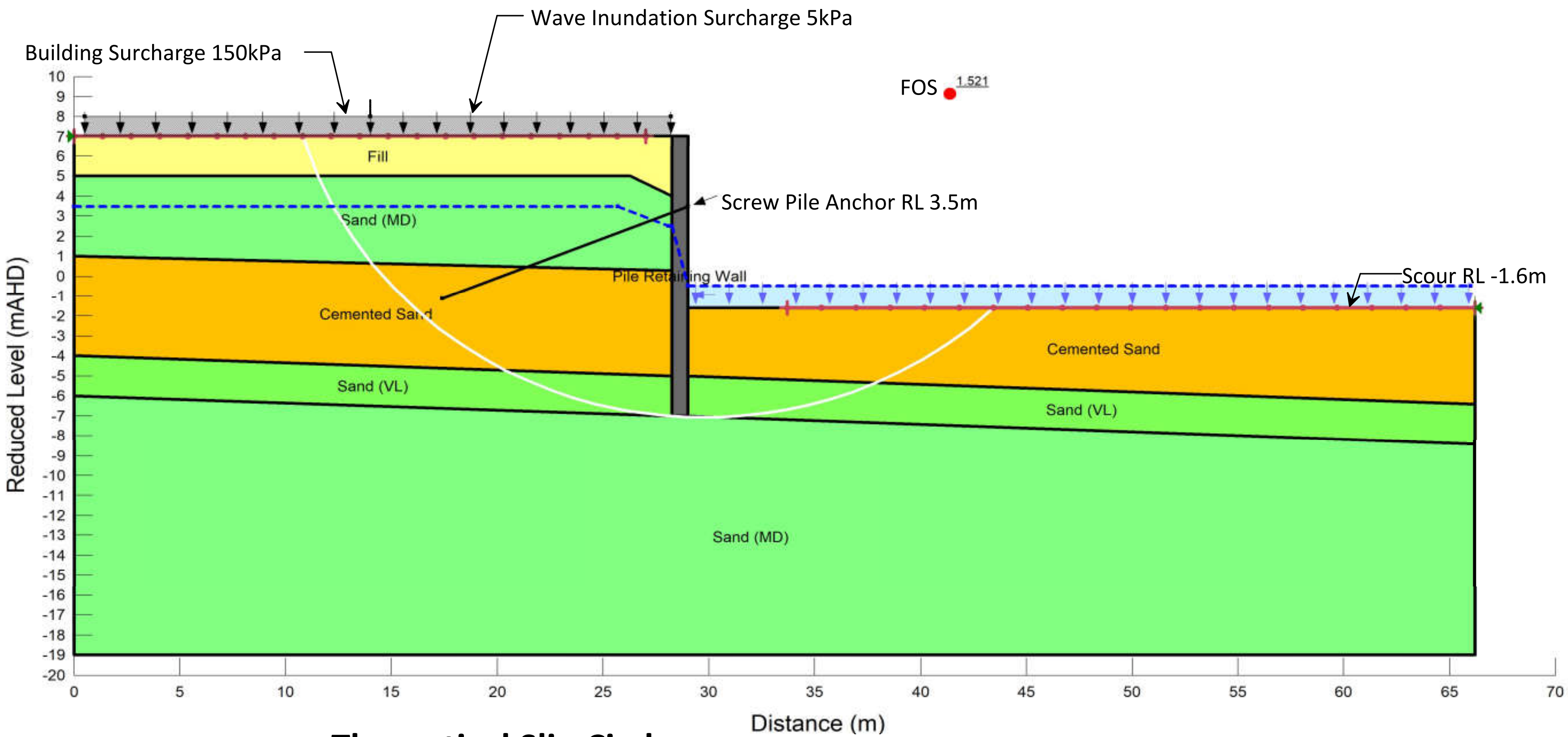
Figure No. 4

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1



Theoretical Slip Circle
(Scour Level RL-1.6m AHD)
Global Failure

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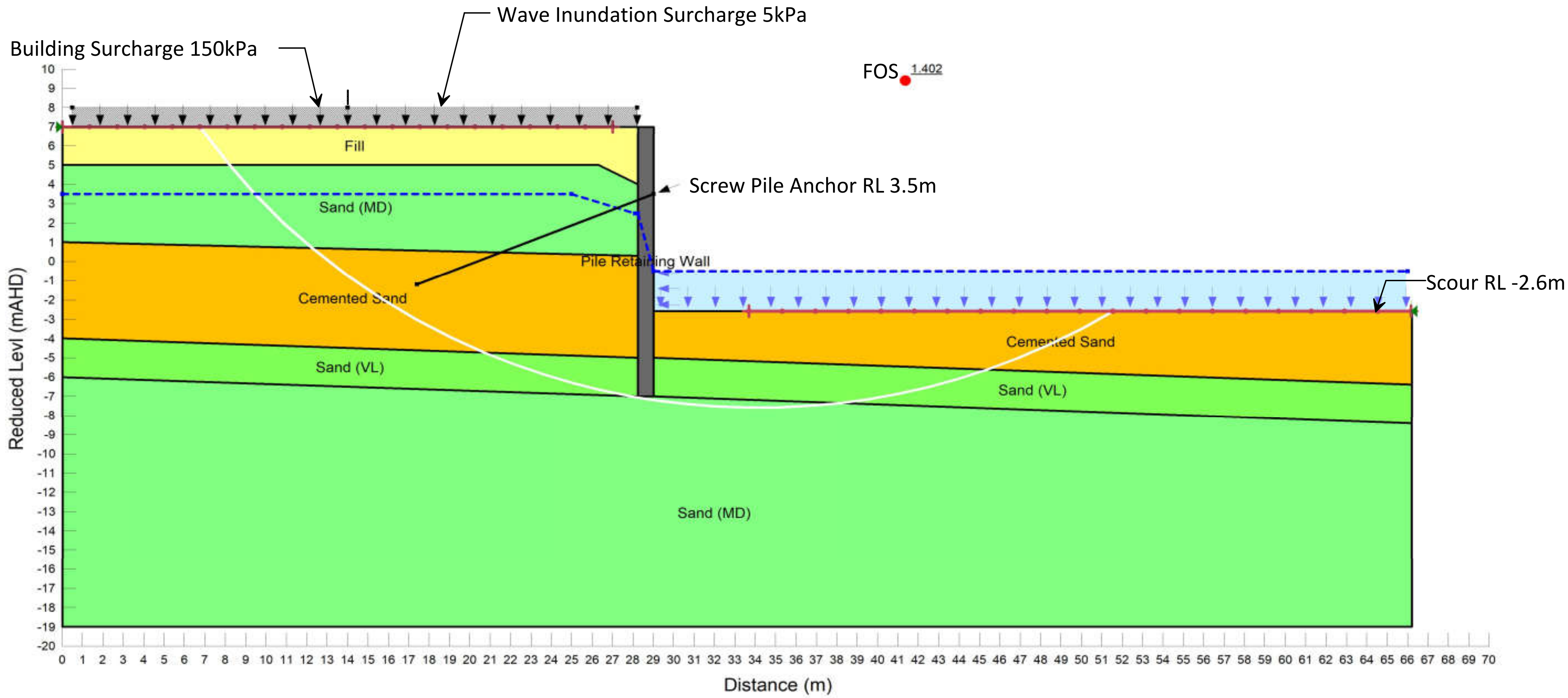
Report No. 32247R **Figure No. 5**

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1



Theoretical Slip Circle
(Scour Level RL-2.6m AHD)
Global Failure

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Report No. 32247R   Figure No. 6