REPORT TO

THE OWNERS OF 1174 TO 1182 PITTWATER ROAD, NARRABEEN

ON

ADDITIONAL GEOTECHNICAL ASSESSMENT

FOR

PROPOSED COASTAL PROTECTION WORKS

AT

1174 TO 1182 PITTWATER ROAD NARRABEEN, NSW

Date: 21 February 2020
Ref: 30751RDrpt2 Rev2

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

David Schwarzer
Senior Geotechnical Engineer

Report reviewed by:

Paul Roberts
Principal Associate| Engineering Geologist

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
30751Rrpt	Draft Report	29/5/19
30751Rrpt Rev1	Final Report	16/8/19
30751RDrpt Rev2	Final Revised Report	21/2/20

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:
a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.
If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

At the Company's discretion, JKG may send a paper copy of this report for confirmation. In the event of any discrepancy between paper and electronic versions, the paper version is to take precedence. The USER shall ascertain the accuracy and the suitability of this information for the purpose intended; reasonable effort is made at the time of assembling this information to ensure its integrity. The recipient is not authorised to modify the content of the information supplied without the prior written consent of JKG.

Table of Contents

ATTACHMENTS

1 INTRODUCTION

This report presents the results of our additional geotechnical assessment of the proposed coastal protection works at 1174 to 1182 Pittwater Road, Narrabeen, NSW. A site location plan is presented as Figure 1. The assessment was originally commissioned on behalf of the owners of 1174 to 1182 Pittwater Road (The Client) by Morgan Hill by signed ‘Acceptance of Proposal’ form dated 26 July 2017. The final scope of work and fees for this current report (based on our original commission) was confirmed by Peter Horton (Horton Coastal Engineering Pty Ltd [HCEPL]) in an email dated 27 November 2019. The final scope of works responds to the request from The Client for the design life of the coastal protection works to be increased from 60 years to 100 years.

In June 2016, an East Coast Low Storm caused erosion over the seaward portions of the subject properties. The affected property owners engaged Peter Horton (Horton Coastal Engineering Pty Ltd [HCEPL]) to provide advice on coastal protection works. The proposed coastal protection works will be submitted as a Development Application to Northern Beaches Council.

We have been provided with the following information:

- Coastal Engineering drawings (Project No. 6035, Drawing Numbers S01 to S04, S10, S11, S16 and S20 Rev C, dated 21 February 2020) prepared by HCEPL and James Taylor & Associates.
- Survey plan (Ref. 031/17, Drawing Numbers 1 and 2, dated 28 May 2017) prepared by Detailed Surveys.
- Coastal engineering reports (Ref. lrJ0031-1172-1182 Pittwater Rd Narrabeen, dated 13 September 2016; Ref. lrJ0031-1172-1182 Pittwater Rd Narrabeen-sections, dated 26 March 2017; and rpJ0094-1174-1182 Pittwater Rd Coastal Engineering Report-A, dated 21 February 2020) prepared by HCEPL.

Based on a review of the provided information, and information provided by Richard Yates (James Taylor and Associates), we understand that the proposed coastal protection work will include:

- An anchored contiguous piled wall (0.75m diameter) embedded to RL–7.5m AHD.
- Permanent ground anchors comprising steel screw piles (3m centre to centre lateral spacing) to support the contiguous piled wall, when sand levels have eroded on the seaward side of the wall. There is also the potential for alternative anchor set-outs (e.g. deadman scheme) and or types (e.g. bar or strand anchors) to be developed in consultation with the contractor (subject to confirmation by the structural and geotechnical engineers). The steel screw pile anchors (approximately 13.0m length) would be installed at RL3.5m AHD (the retaining wall will be thickened at the base where anchors are proposed to be embedded), at a downward angle of about 20° below the horizontal, extending approximately 7.5m beyond a line projected up from the toe of the piled wall at 60° (subject to detailed pile design by the piling contractor, with approval of the structural and geotechnical engineers). The steel screw pile anchors would be embedded in the cemented sand.
- Smaller (0.45m diameter) mass concrete infill plug piles on the landward side of the contiguous piled wall (or alternatively smaller diameter plug piles between the contiguous piles) at the pile intersections to assist in preventing erosion of the retained sands from the landward side of the piled wall at the interfaces of the piles. The infill piles would be embedded to RL–3.5m AHD. However, there is also the

potential for jet grout to be used to fill gaps between the contiguous piles. The top of the pile capping beam will be formed at RL2.9m AHD.

- A reinforced concrete retaining wall (minimum 0.35m thick) extending from the top of the capping beam to RL6.5m AHD (No. 1174 to No. 1180) and RL7m AHD (No. 1182). The retaining wall will be provided with a wave return at the crest projecting seaward approximately 0.5m.
- Three beach access stairs will be provided along the length of the coastal protection works and will extend down from RL6.5m AHD to RL0.61m AHD, or the cemented sand level, whichever is higher. No. 1174 will be provided with a set of stairs, and No. 1176 and No. 1178 and No. 1180 and No. 1182 will each be provided with shared beach access stairs. At No. 1182 and No. 1180 the sharing stairs will run down northwards from the southern boundary of No. 1182. At No. 1178 and No. 1176 the sharing stairs will run down northwards from the southern boundary of No. 1178. The upper portion of the stairs will be supported by additional piles located landward of the contiguous/plug piled wall and installed to the cemented sand level. The base of each of the lower sections of the stairs will be supported by a 0.45m diameter pile installed to RL–4.5m AHD (to be confirmed in detailed design), with the top of the pile formed at about RL0.61m AHD or the cemented sand level, whichever is higher.
- The construction of the proposed coastal protection works will require excavation of the beach sands and existing revetments to a maximum depth of about 2.5m. The temporary batter slopes on the seaward side will be formed at approximately 1 Vertical (V) in 2.5 Horizontal (H), and on the landward side will be formed at an angle of about 30^{o}.

In addition, the Collaroy – Narrabeen Beach Coastal Protection Works Design Specifications require that *"The seawall shall have a minimum factor of safety of 1.5 against global slope stability failure. The global slope stability shall be demonstrated using a recognised slope stability program. Slope stability analysis shall be conducted by a suitably qualified engineer. Factors to consider in the analyses should include, but not necessarily be limited to: beach scour in front of the seawall, elevated landward groundwater table level, and surcharge behind the seawall".*

We note that we have completed a previous geotechnical investigation report on behalf of Patterson Britton & Partners Pty Ltd for the Collaroy/Narrabeen Sea Wall Upgrade (Ref. 15048WDrpt) dated 11 July 2000. In addition, we have completed a geotechnical investigation of the existing foreshore protection measures between No. 1168 and No. 1182 Pittwater Road (Ref. 30005ZRrpt) dated 13 December 2016.

The purpose of the assessment herein was to:

1 Complete a numerical analysis with regard to the structural and geotechnical stability of the anchored contiguous/plug piled wall and upper retaining wall;

2 Complete a stability analysis with regard to the 'global' stability of the proposed coastal protection works; and

3 Based on the results of our analyses provide our comments and recommendations on the geotechnical aspects of the proposed coastal protection works.

2 ASSESSMENT PROCEDURE

The assessment included a review of available desk top information, which included our previous geotechnical reports: Ref. 15048WDrpt, dated 11 July 2000 and Ref. 30005ZRrpt, dated 13 December 2016.

An engineering geologist completed an inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs from the beach, road reserves and rear yards during the fieldwork completed between 30 November and 2 December 2016. Features described in Section 3 below have been measured by hand held inclinometer and tape measure techniques and hence are only approximate.

3 RESULTS OF ASSESSMENT

3.1 Site Description

The site description which follows is based on our observations recorded between 30 November and 2 December 2016. We have not been provided with any information to indicate that the details provided below have altered significantly since completion of our previous fieldwork.

The site is located on the crest area of the foreshore slope lining the seaward margin of the properties between 1174 and 1182 Pittwater Road. The properties extended landward to the eastern side of Pittwater Road.

The site is occupied by two storey brick and rendered houses with paved and grass surfaced surrounds. The houses were set-back between about 10m and 25m landward of the crest of the foreshore slope. The timber deck at No. 1182 was set-back about 6m landward of the crest of the foreshore slope. The grass surfaced or brick paved rear yards extended to the crest of the foreshore slope and an in-ground pool was located close to the seaward margin of the house at No. 1176 Pittwater Road. Small to large sized trees were situated within the rear yards.

The seaward margins of the properties were lined by a partially sand covered rock revetment typically comprising sandstone boulders of between about 1m3 and 2m3 size. We note that voids were observed landward of the revetment face at No. 1174 Pittwater Road. The exposed and partially buried rock revetment sloped down to the east at between approximately 25o and 45o. A detailed description of the rock revetment when exposed following the early June 2016 storms is presented in the HCEPL report dated 13 September 2016. The rock revetments extended north and south beyond the site boundaries.

A neighbouring four storey rendered unit building (No. 1172 Pittwater Road) and neighbouring one and two storey brick and weatherboard mixed use building (No. 1184 Pittwater Road) were set-back approximately 3.0m and at least 5m from the northern (No. 1172) and southern (No. 1184) site boundaries, respectively.

Surface levels were generally similar across the northern and southern subject site boundaries with No. 1172 and No. 1184 respectively. Based on a cursory inspection from within the site, the buildings appeared to be in good condition.

3.2 Subsurface Conditions

Based on our site observations and review of the nearby BH204, BH205, TP120 and TP4 from our previous report dated 11 July 2000 and DCP5 to DCP9 and TP5 to TP9 from our previous report dated 13 December 2016, the pertinent subsurface conditions were as follows:

- Sandy fill extending to depths between 2.0m and 3.5m. The fill was assessed to be poorly compacted. The sandy fill in TP5 to TP9 from our previous report dated 13 December 2016 covered the landward section of the existing rock revetment.
- Natural loose sands with occasional medium dense bands over the landward side of the foreshore slope, extending to 6.0m depth (about RL0.3m AHD [BH204] and RL 0.9m AHD [BH205]) and between about 4.9m and 5.2m depth in DCP5 to DCP9.
- Very dense and dense sands with cemented bands in BH204 and BH205 (about RL0.3m AHD and RL 0.9m AHD, respectively). Over the seaward portion of the foreshore slope, the top surface of similar very dense sands was encountered at about RL–0.2m AHD (TP120), which presented 'hard digging' conditions for a bucket attachment to a 20 tonne excavator. TP4 also encountered very dense sands at RL–0.4m AHD. TP120 and TP4 were terminated in the cemented/very dense sands. The refusal of DCP5 to DCP9 (from our previous report dated 13 December 2016) at between about RL0.7m AHD (DCP7) and RL1.4m AHD (DCP9) was interpreted to indicate cemented sands. DCP5 to DCP9 were also characterised by a 0.15m to 0.5m thick band of dense to very dense sands above the refusal depths. Based on the previous investigation results, the top surface of the cemented sands has been interpreted to gently slope down to the east.
- Sands of variable density (very loose, loose, medium dense and dense) below about RL–3.7m AHD (BH204) and RL-2.2m AHD (BH205). Both BH204 and BH205 were terminated in the sands at respective depths of 19.5m (RL–13.2m AHD) and 18.3m (RL-11.4m AHD).
- BH204 and BH205 respectively encountered groundwater seepage at about RL0.1m AHD and RL0.7m AHD. A standing tidal groundwater level was recorded in TP4 at about RL0.0m AHD. The remaining test pits were 'dry' during and on completion of excavation.

With regard to the revetment encountered in the test pits from our previous report dated 13 December 2016, the landward margin of the rock revetment exposed in the test pits was assessed to be at the following set-back distances:

- TP5: approximately 6.8m landward of the slope crest lining the eastern end of the yard area within No. 1174 Pittwater Road (see Figure 7 in Appendix B);
- TP6: approximately 6.9m landward of the slope crest lining the eastern end of the yard area within No. 1176 Pittwater Road (see Figure 8 in Appendix B);
- TP7: approximately 7.4m landward of the slope crest lining the eastern end of the timber decking within No. 1178 Pittwater Road (see Figure 9 in Appendix B);

- TP8: approximately 5.4m landward of the slope crest seaward of the eastern end of the yard area within No. 1180 Pittwater Road (see Figure 10 in Appendix B); and
- TP9: approximately 8.7m landward of the slope crest seaward of the eastern end of the yard area within No. 1182 Pittwater Road (see Figure 11 in Appendix B).

4 NUMERICAL AND STABILITY ANALYSIS

4.1 Geotechnical Model

The location of the cross section selected for the analyses is indicated on the attached Figure 2. This location was adopted as this represented the minimum landward off-set distance from the proposed coastal protection works of a significant existing structure (the dwelling at No. 1182 Pittwater Road) that would create a surcharge load on the seawall, and also has the highest rear yard finished surface level (RL7.0m AHD). We also adopted a conservative assumption on the extent of beach scour in relation to the cemented sand level at this location. As part of detailed design refinement, other sections could be analysed to assess variability in dwelling surcharge loads, crest levels and ground conditions along the north-south extent of the works.

The subsurface profile adopted for the analyses has been based on the subsurface conditions outlined in Section 3.2 above and an assessment by HCEPL of the likely eroded beach profile following a storm event and refined to include (where possible) sloping interfaces between the various soils (described below). The borehole logs, test pit cross-sections and location plans from our previous reports are presented in attached Appendix A and Appendix B. The subsurface profile adopted for our geotechnical models for the analyses is presented on the attached Figure 3, and is tabulated below for a cross-shore position on the immediate landward side of the seawall:

Subsurface Profile	WALLAP ANALYSIS	SLOPE/W ANALYSIS
	Top of Stratum RL (mAHD)	
Engineered Fill (placed as part of the works)	7.0	7.0
Existing sand Fill	N/A	7.0
Sand (Loose relative density)	3.0	4.5 and 2.0
Sand (Medium Dense relative density)	1.0	0.4
Cemented Sand	0.5	0.0
Sand (Dense relative density)	-3.9	-3.9
Sand (Very Loose relative density)	-4.7	-4.7
Sand (Medium Dense relative density)	-6.8	-6.8
Sand (Dense relative density)	-11.5	-11.5

In general, a downward seaward slope of about 1 Vertical (V) in 15 Horizontal (H) was adopted for the interfaces between the various layers of the SLOPE/W model (i.e. a similar slope to the top surface of the cemented sand). However, in WALLAP, the program only allows horizontal subsurface interfaces to be input into the model (the sloping scour surface on the seaward side of the wall is allowable). The horizontal layer interfaces were extended landward from the contiguous piled wall. Therefore, this included a continuous engineered fill layer, an averaged upper loose sand layer thickness and slightly thicker cemented sand layer when compared to the sloping interfaces adopted in the SLOPE/W analysis. The horizontal cemented sand level of 0.5m AHD adopted in WALLAP on the landward side of the wall is conservative as the cemented sand is expected to slope upwards at about 1:10 moving landward.

In our analysis we did not include the existing revetment, and conservatively assumed complete removal of all rock boulders; the existing revetment boulders that will remain in place may be regarded as representing a zone of higher shear strength material.

Our analysis also included the following assumptions:

- The Highest Astronomical Tide (HAT) is about RL1.0m AHD and the Lowest Astronomical Tide (LAT) is RL–1m AHD at present. HCEPL advised that it was more appropriate to design for the 100 year ARI still water level which is RL1.5m AHD, or RL2.0m AHD when adding 0.5m to account for sea level rise (SLR), and included as the landward groundwater level. HCEPL also assumed that wave setup effects were not significant in defining groundwater levels.
- The landward RL2m AHD groundwater level was further increased to RL3.5m AHD (i.e. about 1.3m above the weep holes installed immediately below the capping beam at the base of the concrete retaining wall; with the weep holes at about RL2.2m AHD). The elevated groundwater level accounts for:
    - The lack of drainage through the contiguous/plug piled wall and the resulting potential raising of groundwater levels should heavy rainfall occur at the same time as elevated ocean water levels, and
    - Some wave overtopping extending landward of the coastal protection works and infiltrating into the well-drained engineered fill profile.

  The build-up of groundwater to RL3.5m AHD does not take into account lateral groundwater outflows to the north and south of the site where groundwater would readily discharge through the neighbouring rock revetments, while assuming that lateral elevated ocean water levels can inflow landward of the wall at the site. It also ignores the drainage provided by the weep holes at the base of the capping beam. The elevated landward groundwater level of RL3.5m AHD is therefore considered to be a conservative assumption, and even conservative if the weep holes were blocked.
- The groundwater level was raised to RL–0.5m AHD on the seaward side to account for a 0.5m SLR above LAT.
- The beach design erosion scour level was input at RL–1.8m AHD and from seaward side of the wall sloped down seaward at 1V:30H.
- The erosion scour on the seaward side of the wall was modified to achieve theoretical failure of the wall (assumed to be when a Factor of Safety (FOS) just less than 1 was indicated) for both the WALLAP and SLOPE/W analyses.
- A surcharge load associated with standing water over the yard surface due to wave overtopping was applied. This was assumed to be a depth of 0.5m resulting in a 5kPa surcharge acting over the rear yard landward of the pile wall to the seaward margin of the building. This is not considered to be a realistic assumption as overtopping would be expected to immediately drain away. However, although conservative, the short term surcharge of the overtopped water was included.
- A building surcharge of 80kPa was applied for the existing building assuming high level footings.

4.2 Model Parameters

The analysis parameters adopted for the subsurface profile are tabulated below. The soil strength parameters were assessed from our previous geotechnical investigation, our past experience of similar material types and empirical correlations well established in geotechnical engineering.

TABLE OF MATERIAL PARAMETERS

Parameters	Natural Loose Sand and Existing Fill	Very Loose Sand	Medium Dense Sand	Dense Sand	Cemented Sand	Engineered Fill
Unit Weight γ (kN/m³)	16	16	18	19	20	18
Elastic Modulus E (MPa)	20	10	40	80	80	40
Poisson's Ratio ν	0.3	0.3	0.3	0.3	0.3	0.3
Cohesion c (kPa)	0	0	0	0	5	0
Internal Angle of Friction φ (°)	28	26	33	36	40	33

The concrete forming the contiguous wall and concrete retaining wall above were input as 'high strength' materials in the SLOPE/W analysis.

In the WALLAP analysis, an interface friction angle of 50% of the internal angle of friction was adopted for the soil to wall interface on the active and passive sides of the pile wall.

The parameters adopted for the contiguous/plug piled wall and concrete retaining wall in the WALLAP analysis are tabulated below:

TABLE OF WALL PARAMETERS

Wall Type	Pile Diameter/ Thickness (m)	Moment of Inertia (m⁴)/m	Elastic Modulus (MPa)
Contiguous/plug Piled Wall	0.75	2.07×10^{-2}	25,000
Concrete Retaining Wall	0.35	3.57×10^{-3}	25,000

For the WALLAP analysis, the 0.45m diameter infill plug piles were conservatively ignored when determining the piled wall parameters.

TABLE OF STEEL SCREW PILE ANCHOR PROPERTIES

Elevation RL (m)	Spacing (m)	Approximate Length (m)	Free Length (m)	Inclination Below Horizontal (°)
3.5	3	13	5.5	22

4.3 Analysis Procedure

4.3.1 Theoretical Slip Surfaces Using SLOPE/W

The 'global' stability analysis was completed using the computer program 'SLOPE/W' which applies circular slip surface analyses to the model. For the 'rapid drawdown' groundwater levels as described in Section 4.1 above, the analysis considered the following:

- The FOS for the design beach erosion scour level at RL–1.8m AHD, and
- The scour level required to result in theoretical wall failure (FOS just less than 1).

Slip circle analyses were run for the above scenarios in order to determine the Factor of Safety (FOS) for a theoretical global circular failure plane passing under the toe of the pile wall embedded at RL–7.5m AHD.

The analysis to assess the wall failure scour level was carried out by gradually lowering the scour level below RL-1.8m sloping at 1V:30H seaward until the FOS was just less than 1.

We adopted an 'acceptable' Factor of Safety (FOS) of greater than 1.5 for 'global' stability as required by the Collaroy – Narrabeen Beach Coastal Protection Works Design Specifications.

4.3.2 Anchored Contiguous Piled Seawall Using WALLAP

The computer program WALLAP was adopted for the analyses of the proposed pile wall (0.75m diameter) and to assess the expected construction sequence in consultation with James Taylor & Associates and HCEPL. The pile toe embedment at RL–7.5m AHD was adopted together with a row of permanent anchors installed at RL3.5m AHD and at a lateral spacing of 3m.

The stability of the proposed anchored pile wall retention system and concrete retaining wall, and the prediction of wall deflections was analysed by balancing disturbing forces and moments created by the 'active' earth pressures on the landward side of the pile wall with restoring forces and moments from the sand profile on the seaward side of the pile wall below the design beach scour level of RL -1.8m AHD.

In a similar manner as for the 'global' stability analysis described above, the scour level was gradually lowered below the design beach scour level of RL-1.8m AHD until a FOS just less than 1 was achieved (theoretical wall failure).

4.4 Analysis Results

4.4.1 Theoretical Slip Surfaces Using SLOPE/W

The results of the stability analyses are presented on the attached Figures 4 and 5 and have indicated the following:

- The minimum FOS for a theoretical global circular failure plane passing under the toe of the contiguous piled seawall for the design erosion scour level of RL-1.8m AHD (in the cemented sand layer) was in excess of 1.5 (FOS = 1.668); see Figure 4, and
- A scour level at RL-4.9m AHD (in the very loose sand layer) was required to result in a theoretical wall failure; FOS = 0.985; see Figure 5.

4.4.2 Anchored Contiguous Piled Seawall Using WALLAP

The results of the WALLAP analyses are summarised below and print outs from WALLAP are presented in the attached Appendix C.

Piled Wall RL 3.8m to RL -7.5m

Scour RL (mAHD)	FOS	Maximum Displacement at crest (mm)	Maximum Displacement (mm)	Maximum Displacement Acting at RL (mAHD)	Maximum Bending Moment (kN.m/m)	Maximum Bending Moment Acting at RL (mAHD)	Maximum Shear Force (kN/m)	Maximum Shear Force Acting at RL (mAHD)	Anchor Force (kN/m)
-1.8	1.646	2	20	-7.5	152.3	3.5	137.9	3.5	221.32
-2.8	0.997	1	50	-7.5	324.6	3.5	211.2	3.5	372.11

Note

- Scour at RL -1.8m AHD, anchor load is 663.96kN.
- Scour at RL -2.8m AHD, anchor load is 1,116.33kN.

Concrete Retaining Wall RL 7.0m to RL 3.8m

Scour RL (mAHD)	FOS	Maximum Displacement at crest (mm)	Maximum Displacement (mm)	Maximum Displacement Acting at RL (mAHD)	Maximum Bending Moment (kN.m/m)	Maximum Bending Moment Acting at RL (mAHD)	Maximum Shear Force (kN/m)	Maximum Shear Force Acting at RL (mAHD)
-1.8	1.646	2	2	7.0	128.3	3.8	76.9	3.8
-2.8	0.997	8	8	7.0	277.2	3.8	155.0	3.8

We forewarn that the anchor load of over 900kN is very large and may not be achievable, depending on advice from the anchor contractor. It may be that a CFA pile deadman system, as depicted on Drawing S16, is considered by the contractor as an alternative to reduce anchor loads. Such an alternative system would need to be reviewed, as part of detailed design and at tender stage.

4.4.3 Additional Comments

The advice from HCEPL is that the design scour level of RL–1.8m AHD would require very adverse circumstances for it to occur over the design life. We note that the erosion scour level to result in a FOS just less than 1 of RL –2.8m AHD (WALLAP analysis) or RL -4.9m AHD (SLOPE/W analysis) may be regarded as an inconceivable scour level. However, these analyses to result in a FOS just less than 1 provide an indication of the sensitivity of the design to a more onerous level of scour erosion.

The WALLAP FOS calculation does not represent a 'global' failure, as it represents the balance of disturbing forces and moments with restoring forces and moments acting on the piled wall.

4.4 Conclusion

Based on the results of the analysis, we consider that the Collaroy – Narrabeen Beach Coastal Protection Works Design Specifications requirement for a minimum FOS of 1.5 against global slope stability failure has been met.

5 GEOTECHNICAL ADVICE

5.1 Proposed Construction Sequence and Methodology

Based on the proposed form of the coastal protection works, we recommend the following generalised construction sequence and methodology:

1 Preparation of a Construction Methodology Plan (CMP).
2 Review and approval of the CMP by the project coastal, structural and geotechnical engineering consultants.
3 If required, complete dilapidation surveys of the seaward ends of the subject properties and structures.
4 Establishment of appropriate construction zone fencing/traffic control, etc to Council requirements.
5 Geotechnical consultant to complete a piling rig working platform design based on information supplied by the piling contractor.
6 Excavate along the seaward portion of the subject properties to remove any obstructions (boulders etc) and form the temporary batter slope extending landward into the rear yards of the subject properties. Subject to assessment by the geotechnical engineer, if the temporary batters are formed through the existing revetment (with the remaining landward portion of the revetment left in place), steeper batters may be feasible. However, this would also need to be assessed in relation to the feasibility of drilling anchors through the remainder of the existing revetment boulders left in place. Further advice from the anchoring contractor will need to be sought in this regard. **We note that the landward extent of the temporary batter slope formed entirely through sandy soils will extend close to the existing pool in No. 1176; if there are concerns that the existing pool may be undermined, then test pits will need to be excavated to confirm the footing details before bulk excavations commence. The test pits will need to be inspected by the structural and geotechnical engineers and the need for underpinning and/or temporary support measures can then be detailed.** Excavated sand seaward of the properties is to be placed on the beach seaward of the works (sand from within the properties may be stockpiled within properties), with rock and other non-sandy materials separated out.

7 Form a sand bund seaward of the works site.
8 Reinstate sand up to piling working platform level (approximately the underside of the capping beam) in accordance with geotechnical advice.
9 Install contiguous/plug piled wall and landward ‘plug’ piles down to the design toe levels.
10 Install piles at recessed stair locations seaward of contiguous/plug piled wall.
11 Install piles at the base of the lower stairs.
12 Form and pour concrete capping beam and retaining wall thickening at the proposed anchor locations. The thickenings to include an opening of sufficient diameter (and appropriate inclination) to accommodate the steel screw pile anchors.
13 Install steel screw pile anchors and complete tension load testing in accordance with the requirements of AS2159-2009, in particular Table A2 Stages S1 and S2 procedure. **THIS IS A HOLD POINT UNTIL SATISFACTORY PERFORMANCE OF THE STEEL SCREW PILES HAS BEEN DEMONSTRATED BY THE CONTRACTOR AND CONFIRMED BY THE GEOTECHNICAL AND STRUCTURAL ENGINEERS.**
14 Form and pour upper stairs over capping beam.
15 Form and pour lower stairs down to the lower pile.
16 Excavate sand down to cemented sand level or–1m AHD (whatever is higher) seaward of contiguous/plug piled wall.
17 Inspect piling for adequacy (e.g. no gaps in the piling that would allow migration of soil though the wall) and make good as required and install weep hole drains. **THIS IS A HOLD POINT UNTIL THE STRUCTURAL ENGINEER HAS CONFIRMED THE ADEQUACY OF THE PILING.**
18 Form and pour the reinforced concrete seawall from the top of the capping beam.
19 Reinstate remaining seaward portion of rear yard areas within the subject properties with engineered fill as required, including establishing landscaped areas landward of the concrete seawall.
20 Replace sand seaward of the pile wall to form a natural beach profile. Material would be screened to remove any non-sandy inclusions.
21 Post construction dilapidation survey.

5.2 Site Preparation

5.2.1 General

We recommend that the contractor prepares a Construction Methodology Plan (CMP) prior to works commencing which should be completed with due regard to the geotechnical advice provided in this report, the coastal/structural engineering drawings prepared by HCEPL and James Taylor & Associates, and any relevant Council DA Consent Conditions. The CMP must include, but not be limited to, proposed excavation techniques, the proposed excavation equipment, sequencing of the excavation, required inspections by the geotechnical, structural and coastal engineers, hold points etc, if required. The geotechnical, structural and coastal engineers should review and approve the CMP.

Prior to works commencing, consideration should be given to preparing detailed dilapidation reports on the seaward sides of the subject properties and structures and the seaward portion of No. 1172 to the south and No. 1184 to the north. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the clients in pursuing any claims against the contractor for damage.

5.2.2 Excavation Conditions

Excavation recommendations provided below should be completed by reference to the current Safe Work Australia Code of Practice *'Excavation Work'*.

Bulk excavations locally required to achieve design subgrade levels will extend to a maximum depth of about 5.5m below the existing rear yard and current beach/existing boulder revetment surface levels. The seaward margin of the excavations are not expected to extend any lower than about RL0m to –1m AHD (i.e. the top surface of the cemented sand layer). The excavations will extend through the sandy soil profile and encounter gravel, cobble and boulder sized inclusions. The excavations are expected to be readily completed using tracked excavators but with over-excavation to remove obstructions. Any topsoil or root affected soils should be stripped and separately stockpiled for re-use in landscape areas as such soils are not suitable for re-use as engineered fill.

Care will need to be exercised in order to maintain the stability of the adjacent sections of neighbouring rock revetments to the south and north, which support the seaward sections of the rear yards within No. 1172 and No. 1184 respectively. This work will need to be completed using suitably experienced (and insured) contractors and supervised by a suitably qualified engineer.

5.2.3 Potential Ground Surface Movement Risks

Due to the loose natural sands (including beach sands) and possibly poorly compacted fill, which we expect will extend across the general area, we advise that sudden stop/start movements of tracked excavators and dropping of items causing ground impacts should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures within and neighbouring the site.

5.2.4 Groundwater Seepage and Tidal Levels

Groundwater inflow is expected within the excavations within the sandy soil profile, due to tidal fluctuations. Consideration of appropriate sequencing of the works in relation to tidal levels will be required.

In general, we expect any groundwater inflows to be of small volume and managed by infiltration into the sandy subgrade. Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design.

The Highest Astronomical Tide (HAT) is about RL1.0m AHD and excavations over the area surrounding the proposed wall and piling will extend below tidal water levels and some instability can be expected; further advice is presented in Section 5.3.1, below.

5.3 Temporary Batter Slopes and Retention

5.3.1 General

Temporary excavation batters no steeper than 1 Vertical (V) in 1.5 Horizontal (H) are considered feasible for the sandy soils above the groundwater levels. These temporary batter slopes are only expected to be accommodated over the landward and seaward sides of the proposed works, although care will need to be exercised close to the seaward margin of any existing rear yard landscape structures that are to remain, and the pool in No. 1176. We recommend that the 'as built' details of the existing pool that the owners and/or Council may have be sourced and reviewed in relation to existing footing details and founding level. If such information is not available, then further input from the geotechnical and structural engineers will be required in order to appropriately sequence the works and maintain the stability of the pool. The further input from the geotechnical and structural engineers (as described in Item 6 of the construction methodology outlined in Section 5.1 above) would be expected to include:

- Inspection of test pits excavated in an attempt to expose pool footings in order to determine their nature, embedment depth and foundation soils.
- Detailing temporary support measures, underpinning, shoring etc as necessary.

The above temporary batter slopes will not be achievable over the northern and southern end of the proposed works, where there are existing rock revetments. However, to form the piled wall returns on the boundaries with No. 1172 and No.1184, some revetment boulders will need to be removed to allow the installation of the piles.

Excavations must therefore be carefully completed in order to expose the basal profile of the adjacent rock revetments. The coastal engineer and the geotechnical engineer will need to inspect the exposed profile in order to assess the extent of boulder excavation required. Such details will need to be confirmed by initially excavating test pits which should be inspected by the coastal engineer and the geotechnical engineer. Subject to assessment by the geotechnical engineer, it may also be feasible to form steeper temporary batters through the existing revetment. It is likely to be unavoidable that temporary excavation batters extend into No. 1172 and No. 1184 and permission would need to be sought for works on these properties. Due to the presence of the revetment, conventional temporary shoring of excavations (for instance using sheet piles), is not feasible and steep batters through the revetment are considered to be the only viable alternative, with any encroachment into neighbouring property kept to a practical minimum. On completion of the piled wall returns, the adjacent sections of revetment will need to be restored to their pre-works condition, with any impacted neighbouring rear yard surfaces reinstated to their pre-works condition.

To facilitate backfilling in order to reinstate the rear yard areas, we recommend that the landward batter slope be formed with a stepped profile (within an overall slope of 1V in 1.5H) to facilitate the use of compaction equipment; see Section 5.5, below.

We note that the bulk excavations over the seaward side of the works will extend below the tidal groundwater level and will affect the stability of the excavation sides. Allowance should be made for use of sand bags to support temporary batters close to, and below, the groundwater levels.

Due to the collapsible nature of the sandy soil profile and the tidal groundwater levels, auger grout injected (CFA) piles would need to be used to form the contiguous/plug piled wall. To prevent soil loss through any gaps in the contiguous piles, a second row of smaller diameter CFA plug piles will also be installed landward of the piled wall (or potentially between the contiguous piles). There is also the potential for jet grout to be used to fill the gaps between the contiguous piles.

For the proposed contiguous piles founded at RL–7m AHD, the cemented sands will need to be penetrated. We note that the existing boulders will pose problematic pile drilling conditions and allowance for their removal has been described in Item 6 of the construction methodology outlined in Section 5.1, above.

5.3.2 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the static design of the contiguous/plug piled wall and concrete retaining wall:

- For design of the permanent cantilever contiguous/plug piled wall retention system and the section of concrete retaining wall extending from RL3m to RL7.0m AHD, we recommend using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.35 and an 'at rest' earth pressure coefficient, K_o, for the soil profile, assuming a horizontal backfill surface. The earth pressure coefficients applicable to the design vary depending on the construction stage.
- Any surcharge (including construction traffic, compaction stresses, landscaping, inclined retained surfaces, footings etc) affecting the retention system should be allowed for in the design using the above earth pressure coefficient.
- The piled wall must be designed for hydrostatic pressures based on the adopted design groundwater level at RL3.5m AHD, as discussed in Section 4.1, above.
- Drainage landward of the upper wall should comprise single sized granular material (or 'no fines' gravel) as defined in Section 5.5.2 below, and connected to PVC pipe weep hole drains to be formed at the top of the contiguous piled wall (about RL2.2m AHD).
- Toe restraint may be provided by the passive pressure of the soil below the design beach erosion scour level. A passive earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections.
- Bulk unit weights as outlined in Section 4.2, above should be adopted for the retained profile above the water level, and reduced by $9.8kN/m^3$ below the groundwater level.

5.3.3 Steel Screw Pile Anchor Design

We recommend for ease of construction that the ground anchors required to support the proposed contiguous piled wall comprise steel screw piles. The pull-out resistance of the screw pile is governed by the cone of sand projected up from the perimeter of the helix at an angle of 30^o. However, no positive contribution to anchor support can be assumed within the 'active' zone landward of the contiguous piled wall.

We note that installation of the anchors will be problematic due to the existing revetment. Therefore, the excavation works described in Item 6 of the construction methodology outlined in Section 5.1 above, will need to be carefully completed to ensure that all such obstructions are removed, unless the anchoring contractor can devise a suitable methodology to the satisfaction of the geotechnical and structural engineers to penetrate through the boulders and other obstructions.

Assuming the helix of the screw pile is anchored into the cemented sand and has a minimum embedment length of 6.0m (measured along a line orthogonal to the concrete wall down from the 'active' zone line projected up from the base of the piled wall at 60°) then, as a guide, the working load per screw pile (adopting a 0.35m diameter helix) would be approximately 800kN. This assumes:

- A FOS of 2.
- The pull-out cone providing restraint is submerged (i.e. below the design groundwater level, assumed to be RL3.5m AHD in our analyses described in Section 4, above, and likely to always be submerged with its extent below 0m AHD).
- No friction on the screw pile shaft.
- One screw pile helix.
- No interaction between the pull-out cones from adjacent anchors; where this occurs the load capacity per anchor will be reduced.

The capacity of steel would also need to be checked for the loading conditions and designed with due regard for corrosion in this marine environment.

We note that specialist steel screw pile suppliers would have their own 'in-house' data and may provide more specific advice in relation to the anchor design. We recommend that a pull out test be performed on a test pile to verify the design. In addition, we recommend that all steel screw pile anchors are load tested in accordance with the requirements of AS2159–2009, in particular the test procedure outlined in Table A2 Stages S1 and S2. The pull-out test and the load testing of the permanent anchors should be witnessed by a geotechnical or structural engineer independent of the anchor contractor. The drawings require the geotechnical and structural engineers to certify the anchors.

5.4 Piling Rig Working Platform

The piling rig may need to be provided with a suitable working platform determined by a geotechnical engineer. The design of the working platform will need to be based on the specific loadings and track dimensions supplied by the piling contractor for the proposed piling rig. Further, the assessment of the working platform thickness will need to be based on the methodology outlined in BR 470 *'Working Platforms for Tracked Plant'* (2004, prepared by BRE). In addition, should any works be completed close to sloping surfaces then computer based stability analyses may also be required.

The working platform will need to be constructed using DGB20 (or a similar durable granular material approved by the geotechnical engineer) compacted to at least 95% Modified Maximum Dry Density (MMDD) using a large roller. The subgrade will need to be prepared as outlined in Section 5.5.1 below.

Density tests should be regularly carried out on the working platform materials to confirm the above density has been achieved. The frequency of density testing should be at least one test per layer per 2500m^2 or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798–2007. However, our preference would be for Level 1 control of fill placement and compaction, in accordance with AS3798-2007.

On completion of the piling works, the granular fill should be removed from the beach (with the assistance of screening, as required) and may be used as backfill to reinstate the rear yards.

5.5 Earthworks

The following earthworks recommendations should be complemented by reference to AS3798–2007 *"Guidelines on Earthworks for Commercial and Residential Developments".*

With regard to the proposed works, the following earthworks are envisaged:

- Formation of a piling rig working platform.
- Reinstating the rear yards of the subject properties landward of the new contiguous/plug piled wall and concrete retaining wall.

5.5.1 Subgrade Preparation

Prior to placement of fill to reinstate the rear yards and/or construction of the piling rig platform (if required), preparation of the soil subgrade should consist of the following:

- Following completion of bulk excavations the sandy subgrade over the areas of the piling rig working platform should be proof rolled with at least eight passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. Over landscape areas proof rolling may be completed using a vibrating plate compactor (attached to an excavator or hand held) or, if space permits, with at least eight passes of a static (non-vibratory) smooth drum roller of at least 2 tonnes deadweight. The sandy subgrade should be thoroughly moistened prior to proof rolling.
- To assist with proof rolling, we recommend that a thin layer of road base (75mm thick) be placed over the sand subgrade to improve near surface compaction and prevent shearing during rolling.
- The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas which should be removed and replaced with engineered fill (if required), as outlined in Section 5.5.2, below.
- Care should also be taken when using vibrating equipment not to cause damage to any adjacent structures. The vibrations should be qualitatively monitored by site personnel. If there is any cause for concern then proof-rolling should cease and further advice sought. Alternatively, where appropriate, the static (non-vibration) mode may be used.

5.5.2 Engineered Fill

Fill required to reinstate rear yard areas and unstable areas of subgrade and to backfill any landscape walls should comprise engineered fill.

Engineered fill (including backfill to the proposed concrete retaining wall) should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the excavated sands may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller(s) to achieve a minimum density index (I_D) of 65% for the sandy soils. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

Single sized granular material (or 'no fines' gravel), such as 'blue metal', may be used as backfill to the upper concrete retaining wall and this would also act as the drainage landward of the wall and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non-woven geotextile fabric (eg. Bidim A34) to prevent migration of sand and finer materials into the drainage layer voids between particles.

Sand sized material for backfill shortfalls shall not be sourced from the beach seaward of the subject properties. Backfill material imported to the site shall be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), or recycled materials obtained from an EPA licensed facility. Any backfill material must be consistent with Section 9(d) of Council's Coastal Erosion Policy 2016. For ENM and recycled materials, supporting documentation should be reviewed by a suitably qualified environmental consultant who is a member of the Australian Contaminated Land Consultants Association Inc. All backfill shall be to the satisfaction of the geotechnical engineer.

Boulders excavated as part of the works may be crushed to use as backfill with agreement of the geotechnical engineer, or taken offsite to be used by others.

5.6 Staircase Footings

The proposed staircase beach access will be supported by individual piles on the landward side of the contiguous/plug piled wall and, at the lower end, by pile footings founded below the design scour level of RL -1.8m AHD. The piles will be 0.45m diameter (to be confirmed in detailed design).

The proposed pile footings supporting the lower end of the staircase structures should be founded at a minimum depth of 4 pile diameters below the design scour level of RL–1.8m AHD (i.e. RL–3.6m AHD). We expect that dense sands will be encountered at this depth with very loose sands about 1.0m below this pile toe depth. Piles founded in the dense sands may be designed using a maximum bearing pressure of 400kPa.

The CFA piles that will be used to form the contiguous/plug piled wall are suitable. Screw piles are not considered suitable due to their limited capacity to resist lateral loads in the event of the design scour event occurring.

For the isolated pile footings on the landward side of the contiguous/plug piled wall, we recommend that they be founded in the cemented sand (estimated to be at about RL0.5m to 1m AHD). These staircase piles will be protected from erosion scour by the proposed coastal protection works. We note that the existing boulders will pose problematic pile drilling conditions and that allowance for their removal has been described in Item 6 of the construction methodology outlined in Section 5.1 above.

The CFA piles that will be used to form the contiguous/plug piled wall are suitable for the pile footings on the landward side of the contiguous/plug piled wall. Alternatively, steel screw piles may be used. The piles may be designed for a maximum allowable end bearing pressure of 600kPa. The advantage of screw piles is that they may be removed and reinstalled if large boulder obstructions are encountered.

5.7 Wave Inundation Erosion Protection

Any potential inundation of the rear yard areas due to wave overtopping is expected to naturally infiltrate through the natural sands, drainage layer and free-draining fill. There is the potential for some localised erosion of the landscaped surfaces. However, this may be reduced by establishing a vegetative cover suitable for this marine environment. Any areas of localised erosion may be reinstated, if required, and would not be expected to impact on wall stability.

5.8 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Review of contractors CMP.
- Dilapidation report on the seaward portion of the building and structures within the subject properties.
- Inspection of excavations exposing the neighbouring revetments to the north and south.
- Review of 'as built' records of existing building footings and inspection of test pits exposing existing building footings, together with detailing any temporary support measures and/or underpinning, if required.
- Monitoring of groundwater seepage into bulk excavations.
- Proof rolling of exposed sub-grade.
- Qualitative vibration monitoring during use of vibratory compaction equipment.
- Density testing of engineered fill.
- Piling rig working platform design.
- Witnessing load testing of test anchor and permanent anchors.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

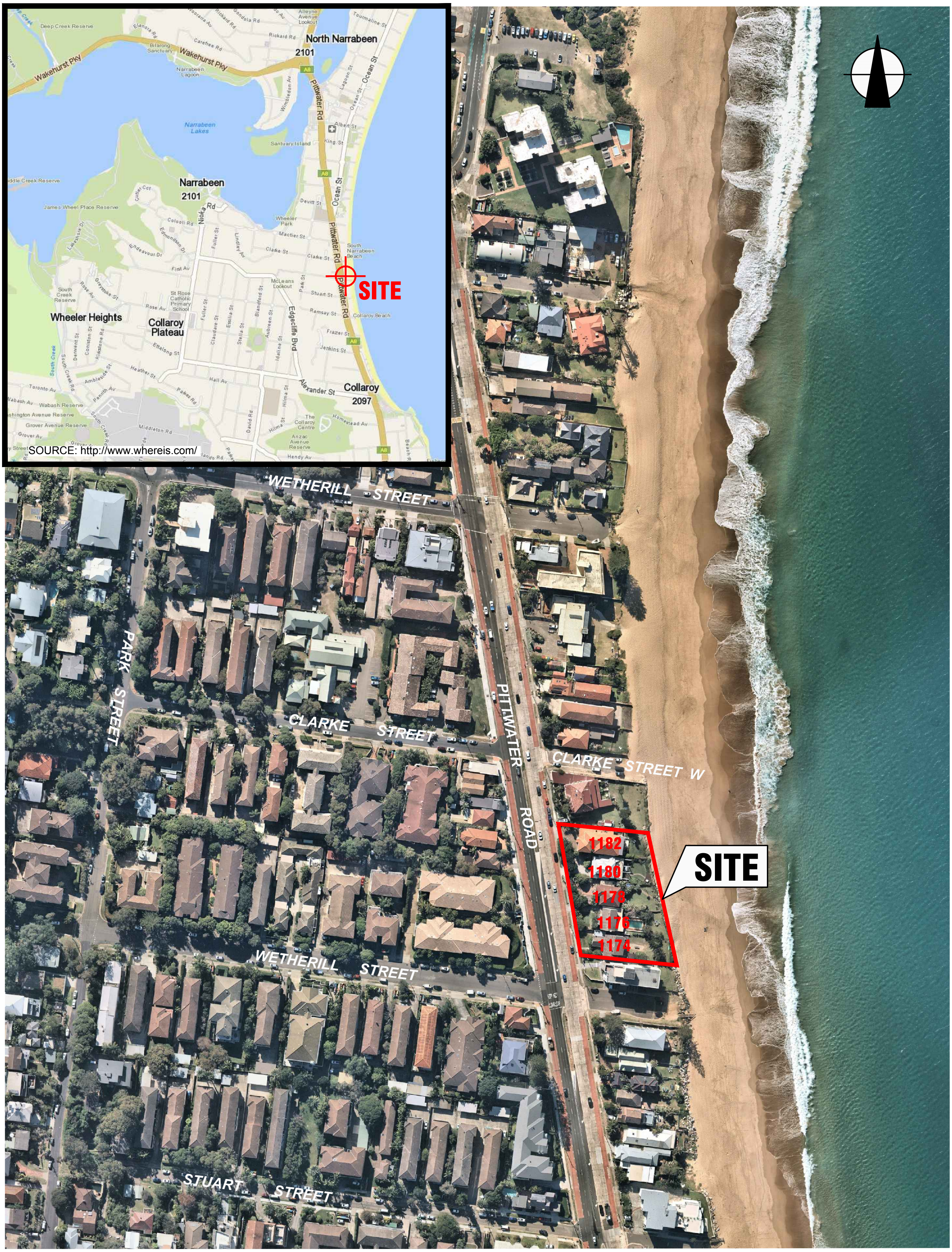
This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.



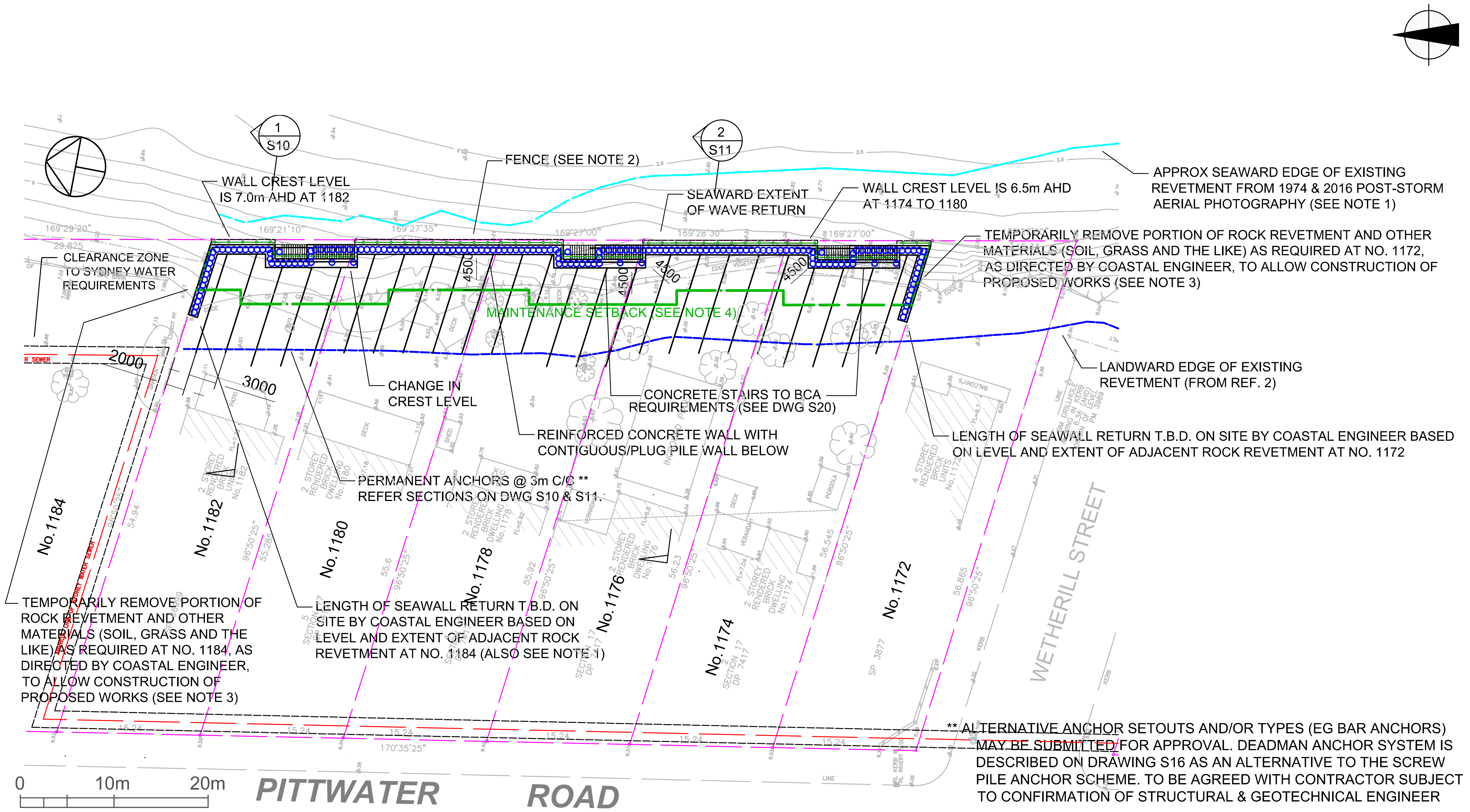
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 12 MAY 2019.

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

Title:	**SITE LOCATION PLAN**
Location:	1174 TO 1182 PITTWATER ROAD NARRABEEN, NSW
Report No: 30751R	Figure No: 1

JKGeotechnics

PLOT DATE: 29/05/2019 1:33:55 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30751R NARRABEEN\CAD\30751R.DWG

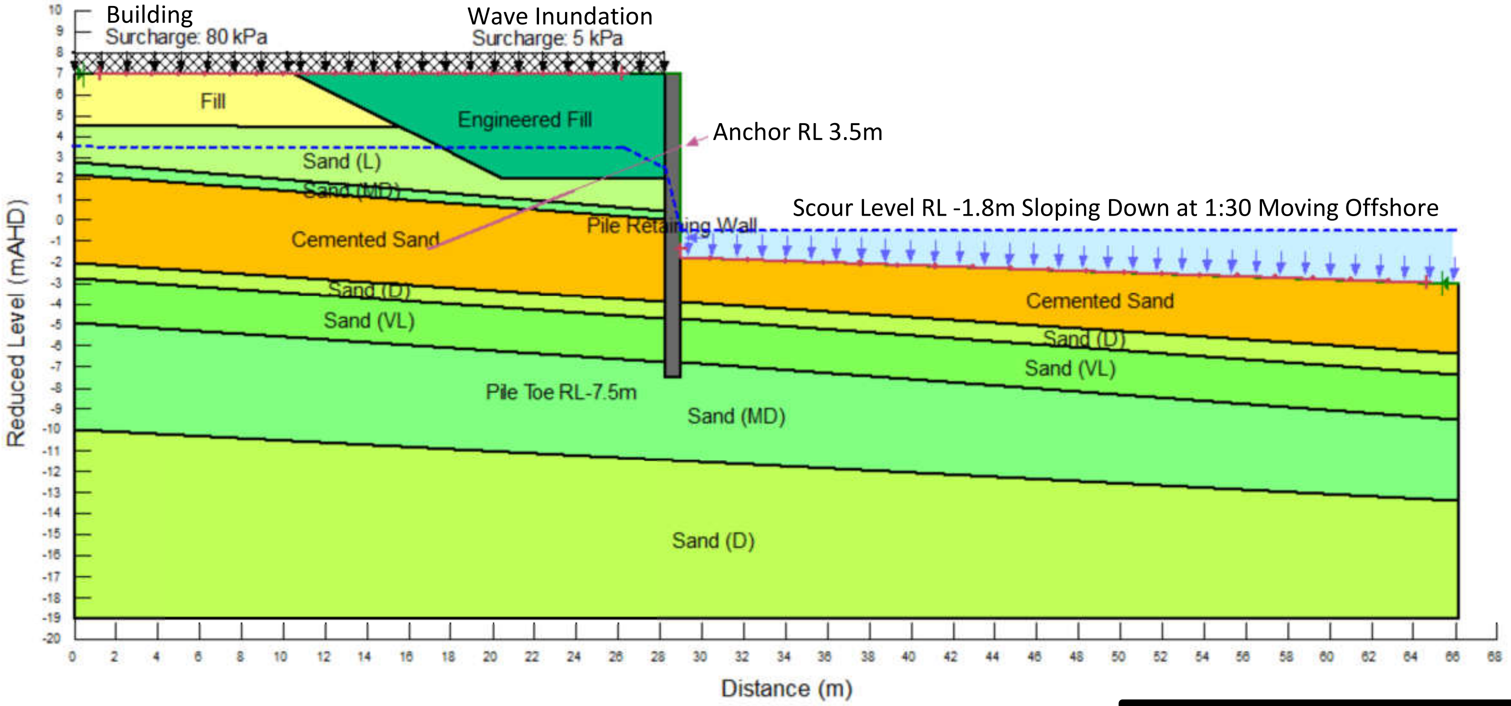


NOTE:

ANALYSIS SECTION BASED ON SECTION 1 (DRAWING S10).

SCALE 1:400 @A3 (0 4 8 12 16 20 METRES)	Title: **PLAN OF PROPOSED COASTAL PROTECTION WORKS**	
	Location: 1174 TO 1182 PITTWATER ROAD NARRABEEN, NSW	
	Report No: 30751R	Figure No: 2
This plan should be read in conjunction with the JK Geotechnics report.	JKGeotechnics	

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Engineered Fill	18	0	33	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (D)	19	0	36	0	1
	Sand (L)	16	0	30	0	1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1

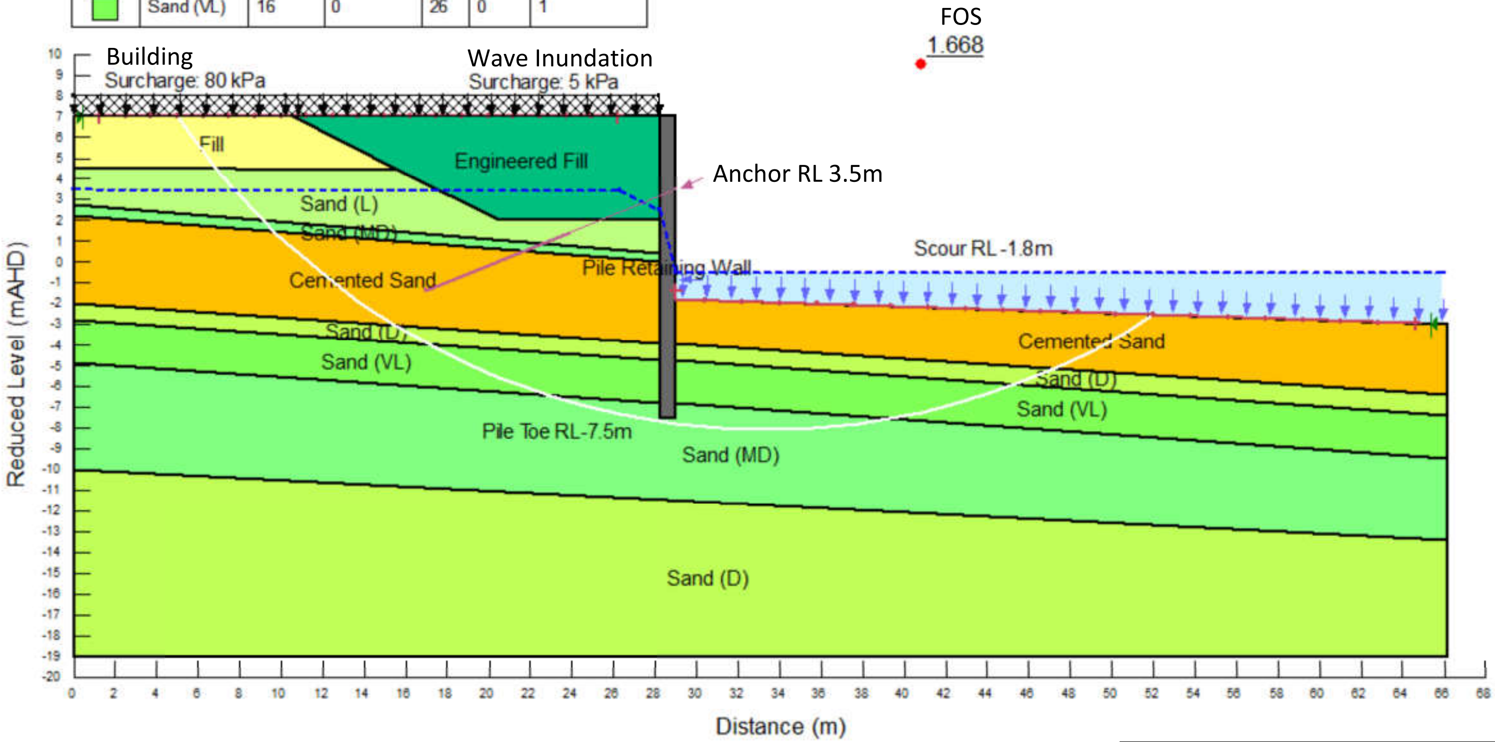


Geotechnical Model

JKGeotechnics

Report No. 30751RD Figure No. 3

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Engineered Fill	18	0	33	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (D)	19	0	36	0	1
	Sand (L)	16	0	30	0	1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1

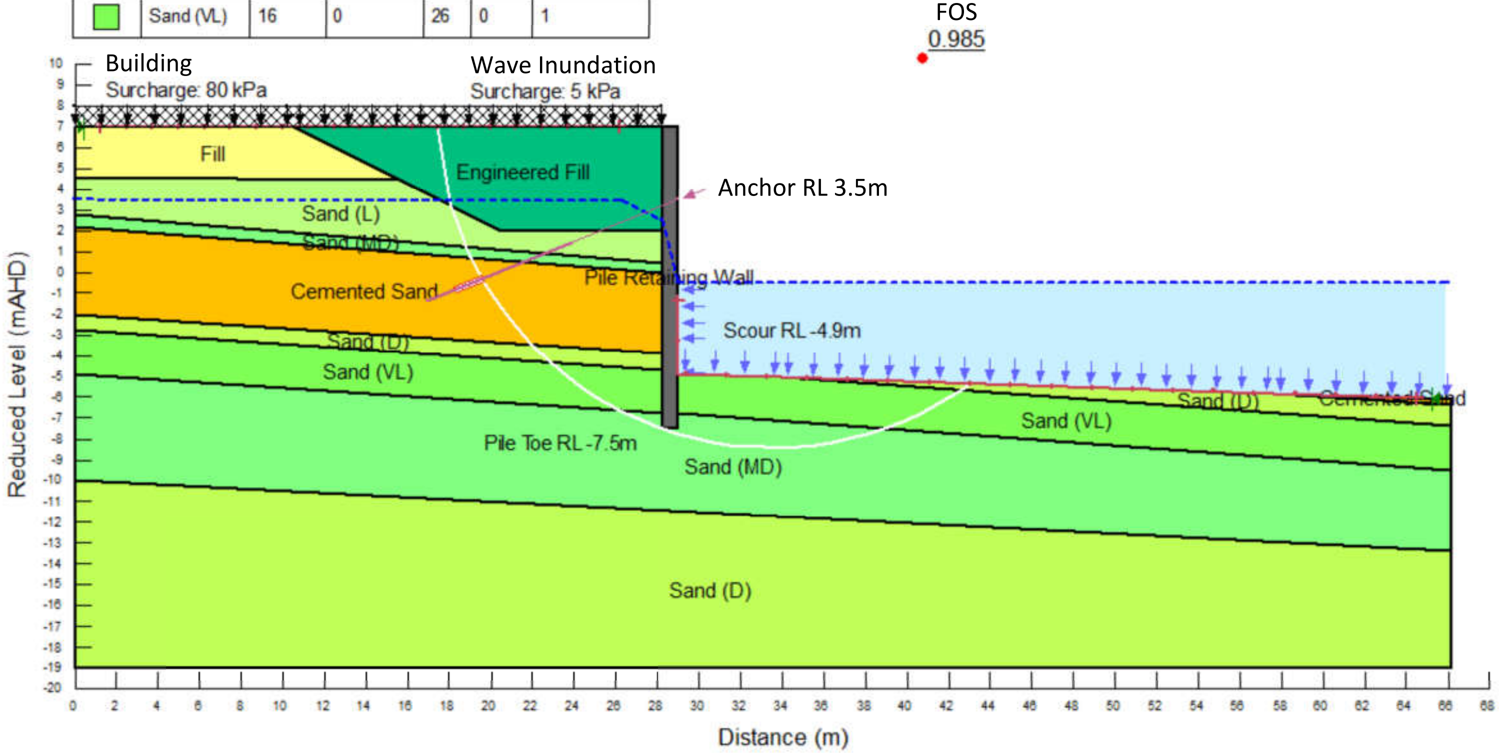


Theoretical Slip Circle
(Scour Level RL -1.8m Sloping Down at 1:30 Moving Offshore)
Global Failure

JKGeotechnics

Report No. 30751RD Figure No. 4

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Cemented Sand	20	5	40	0	1
	Engineered Fill	18	0	33	0	1
	Fill	16	0	30	0	1
	Pile Retaining Wall	24				1
	Sand (D)	19	0	36	0	1
	Sand (L)	16	0	30	0	1
	Sand (MD)	18	0	33	0	1
	Sand (VL)	16	0	26	0	1



Theoretical Slip Circle
(Scour Level RL -4.9m Sloping Down at 1:30 Moving Offshore)
Global Failure

JKGeotechnics

Report No. 30751RD Figure No. 5

APPENDIX A

PREVIOUS INVESTIGATION RESULTS FROM JK REPORT

DATED JULY 2000

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **204** 1/3
WETHERILL STREET

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 15-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.3m
Datum: AHD
Logged/Checked by: S.E./

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Sand, fine to medium grained, brown, with fine to medium grained sandstone gravel.	M			BOREHOLE RELOCATED SEVERAL TIMES DUE TO CONCRETE >200mm.t AT 1.0m
		N > 15 7,5,10/ 50mm								
			1			as above, but pale brown, with a trace of fibro cement sheeting fragments.				APPEARS POORLY COMPACTED
		N = 2 1,1,1								
			2		SP	SAND: fine to medium gained, pale brown and yellow.	M	L	–	–
		N = 6 3,3,3	3							
			4					MD		
		N = 11 3,6,5	5							
▶		N > 20 20/ 120mm	6			SAND: fine to medium grained, yellow and pale brown, with shell fragments, frequent cemented bands and silt fines.		VD		START WASHBORING
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **204** 2/3
WETHERILL STREET

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 15-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.3m
Datum: AHD
Logged/Checked by: S.E./

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			7		SP	SAND: fine to medium grained, pale brown and yellow, with fine shell fragments, frequent cemented bands and silt fines.	W	VD		
	DS	N > 25 25								
			8							
	DS	N > 15 15/50mm	9							
			10			SAND: fine to medium grained, pale brown and yellow with fine shell fragments.		D		
	DS	N = 31 6,12,19								
			11			SAND: fine to medium grained, pale brown, with silt fines and a trace of cemented sand nodules.		VL		
	DS	N < 2 1/ 300mm,1	12							
			13			SAND: fine to medium grained, brown, with clay fines and clayey sand and sandy bands.		L-M		
	DS	N = 10 3,4,6								

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **204** 3/3
WETHERILL STREET

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 15-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.3m
Datum: AHD
Logged/Checked by: S.E./

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			14		SC	CLAYEY SAND: fine to medium grained, pale brown with pale grey sandy bands.	W	MD-D		
		N = 31 4,10,21	15							
			16		SP	SAND: fine to medium grained, brown, with a trace of clay fines.		D-VD		
		N > 25 25								
			17							
		N > 25 25	18			as above, but red brown.				
			19							
		N > 20 20/80mm				END OF BOREHOLE AT 19.5m				SLOTTED PVC STANDPIPE INSTALLED
			20							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **205** 1/3

CLARKE STREET

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 9-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.9m
Datum: AHD
Logged/Checked by: SE./RD

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Sand, fine to medium grained, pale brown, with a trace of fine to medium grained igneous gravel.	M	L		APPEARS POORLY COMPACTED
	DS	N = 8 6,5,3								
			1							
	DS	N = 6 3,3,3								
			2							
			3							
	DS	N = 6 2,2,4								
					SP	SAND: fine to medium grained, pale brown, with a trace of fine shell fragments.	M	L	–	–
			4							
	DS	N = 9 5,5,4								
			5							
			6			SAND: fine to medium grained, pale brown, with shell fragments, frequent cemented bands and silt fines.	W	VD-D		
▶	DS	N > 20 20/ 150mm								START WASHBORING
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 205 2/3
CLARKE STREET

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 9-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.9m
Datum: AHD
Logged/Checked by: SE / [illegible]

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			7		SP	SAND: fine to medium grained, pale brown, with shell fragments, frequent cemented bands and silt fines.	W	VD-D		
	DS	N > 20 20/ 150mm								
			8							
	DS	N > 25 25/ 150mm	9			SAND: fine to medium grained, pale brown and yellow, with fine shell fragments.		VL		50% WATER LOSS
			10							
		SPT SUNK 1.5m UNDER WEIGHT OF RODS	11							
	DS	N = 22 5,9,13	12					MD		
			13			SAND: fine to medium grained, brown, with a trace of silt fines.				
	DS	N = 11 3,5,6								

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **205** 3/3

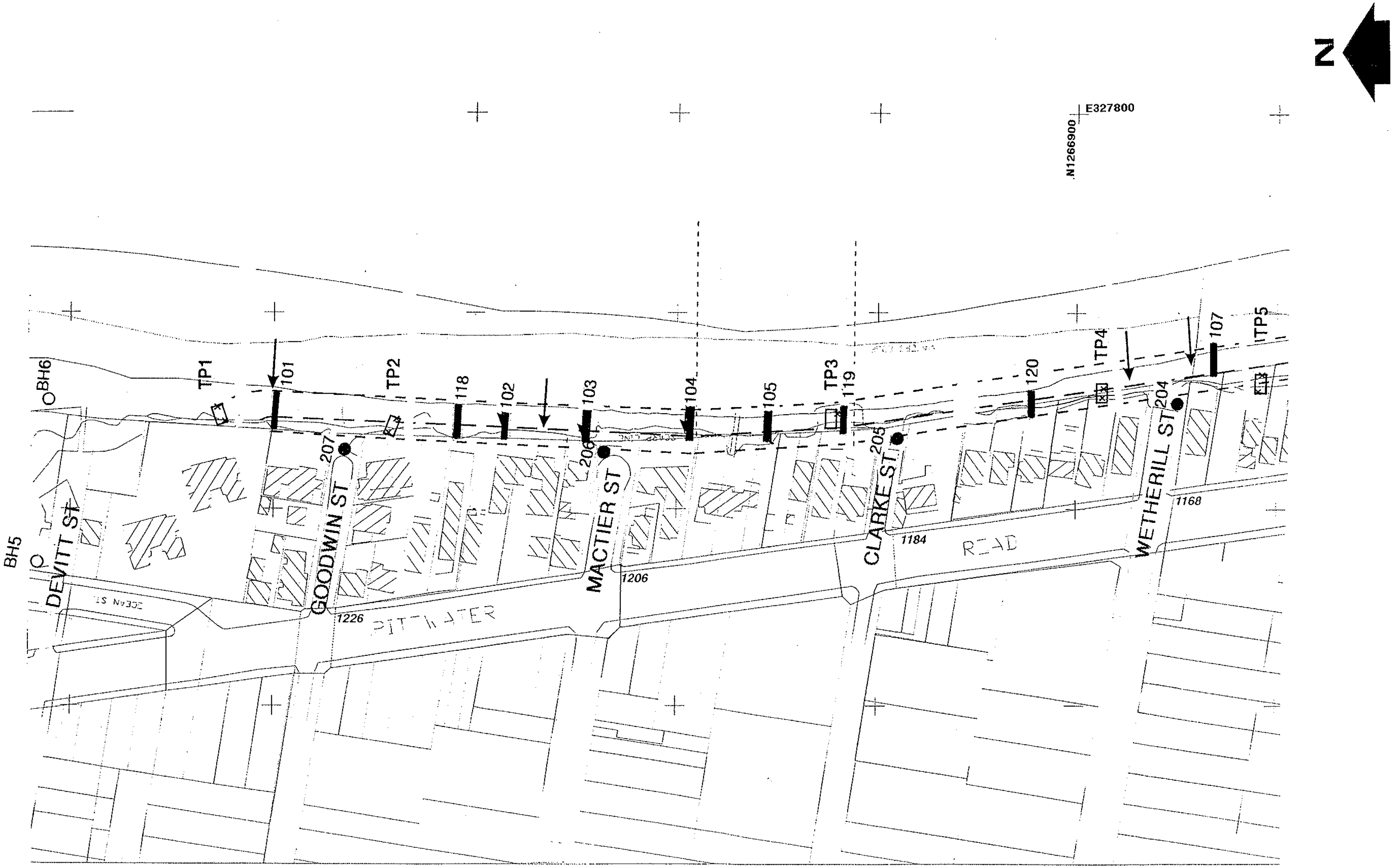
CLARKE STREET

BOREHOLE LOG

Client: PATTERSON BRITTON & PARTNERS PTY LTD
Project: PROPOSED SEAWALL UPGRADING
Location: COLLAROY / NARRABEEN BEACH SEAWALL, NSW.

Job No. 15048WD
Date: 9-5-00
Method: SPIRAL AUGER INTERTECH 550
R.L. Surface: 6.9m
Datum: AHD
Logged/Checked by: SE./[illegible]

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			14		SP	SAND: fine to medium grained, brown, with a trace of silt fines.	W	MD		
	DS	N = 11 8,6,5	15							
		N_c = 20R	16					MD-D		
			17							
	DS	N > 25 20,25/ 150mm	18							
						END OF BOREHOLE AT 18.3m				SLOTTED PVC STANDPIPE INSTALLED
			19							
			20							



LEGEND

- Borehole
- Test Pit
- Previous Borehole – Refer DPWS Report MHL 974 dated July, 1999
- Previous Test Pit – Refer DPWS Report MHL 974 dated July, 1999

SCALE 0 100m

TEST LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No. 15048WD Figure No. 122

Surface Profile for Test Pit 4

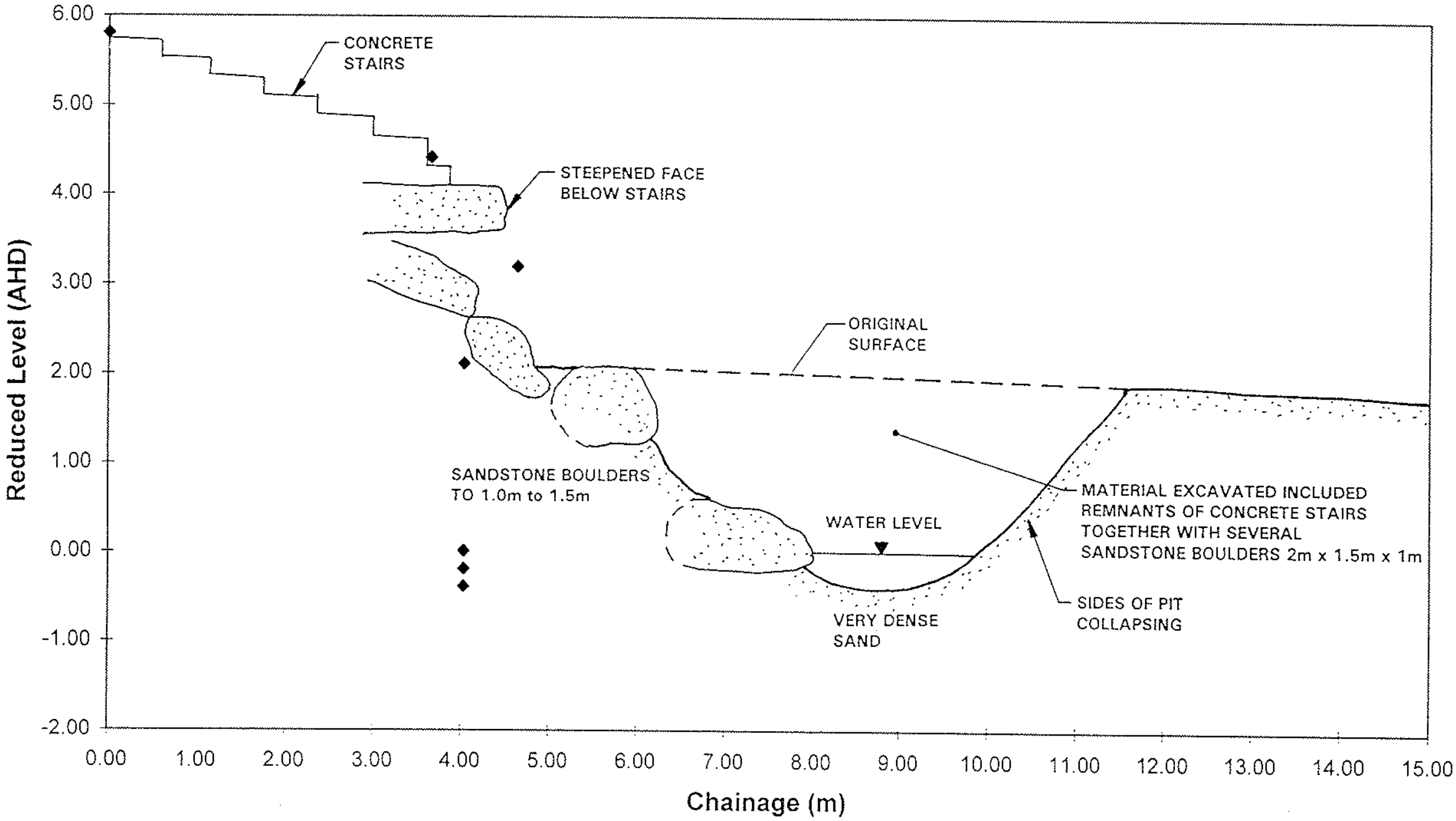
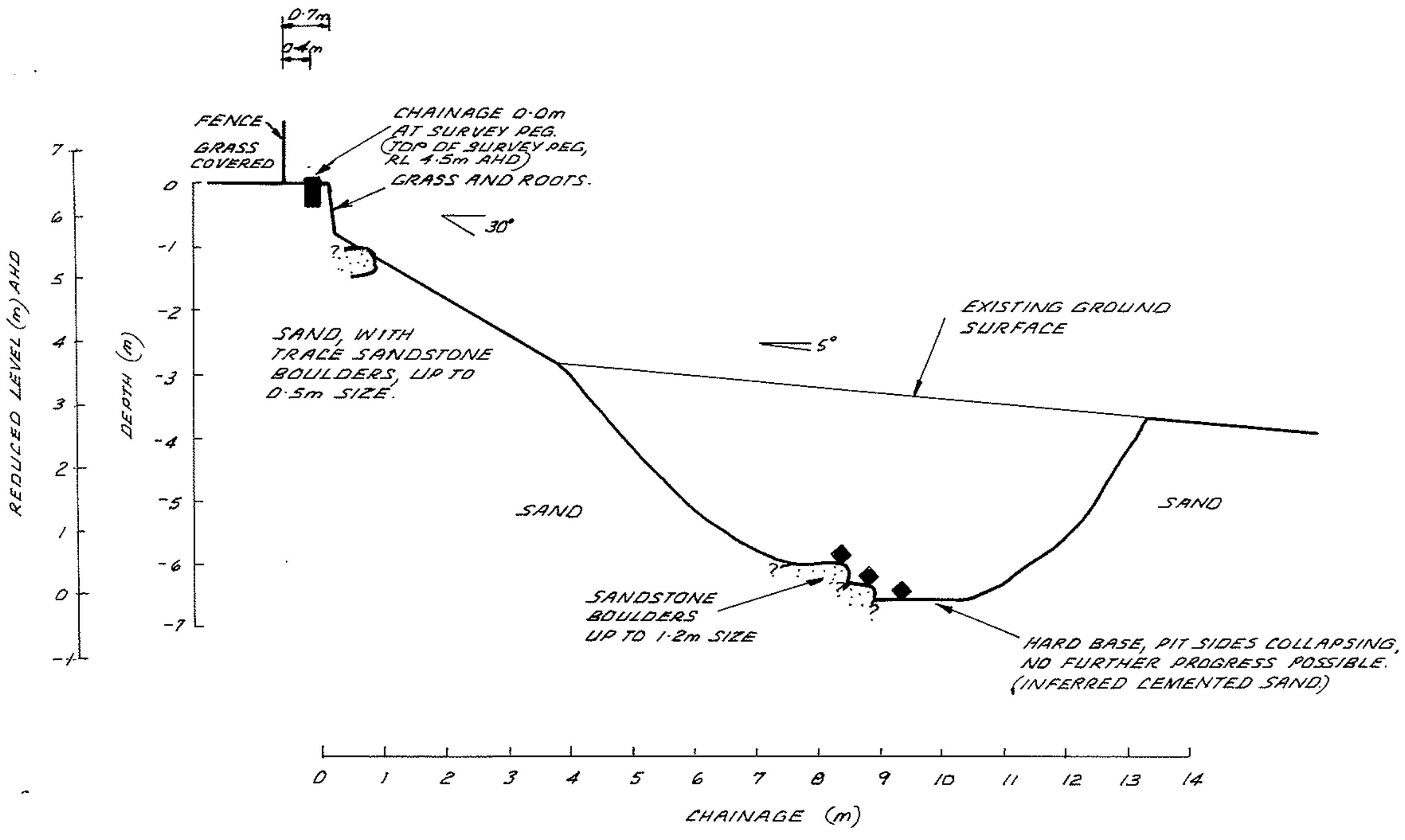


FIGURE A4



NOTE

◆ INDICATES LEVEL TAKEN AT THIS POINT.

TEST PIT 120

SCALE 0 5m

Jeffery & Katauskas Pty Ltd

Report No. 15048WD Figure No. 120

APPENDIX B

PREVIOUS INVESTIGATION RESULTS FROM JK REPORT DATED DECEMBER 2016

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	HORTON COASTAL ENGINEERING / HASKONING AUSTRALIA PTY LTD
Project:	EXISTING FORESHORE PROTECTION MEASURES
Location:	1168 - 1182 PITTWATER ROAD, COLLAROY, NSW

Job No.	30005ZR	Hammer Weight & Drop: 9kg/510mm
Date:	2-12-16	Rod Diameter: 16mm
Tested By:	L.M.	Point Diameter: 20mm

Number of Blows per 100mm Penetration							
Test Location	RL≈6.3m	RL≈6.4m	RL≈6.5m	Test Location			
Depth (mm)	**1**	**2**	**3**	Depth (mm)			**3**
0 - 100	EXCAVATED	EXCAVATED	EXCAVATED	3000-3100			2
100 - 200				3100-3200			1
200 - 300				3200-3300			2
300 - 400				3300-3400			2
400 - 500				3400-3500			2
500 - 600				3500-3600			3
600 - 700				3600-3700			3
700 - 800	3			3700-3800			4
800 - 900	4			3800-3900			4
900 - 1000	13			3900-4000			4
1000 - 1100	6			4000-4100			5
1100 - 1200	12			4100-4200			5
1200 - 1300	17			4200-4300			4
1300 - 1400	REFUSAL			4300-4400			5
1400 - 1500				4400-4500			5
1500 - 1600		1		4500-4600			6
1600 - 1700		2		4600-4700			6
1700 - 1800		2		4700-4800			8
1800 - 1900		3		4800-4900			8
1900 - 2000		4		4900-5000			9
2000 - 2100		13		5000-5100			9
2100 - 2200		20		5100-5200			12
2200 - 2300		6		5200-5300			17
2300 - 2400		5	1	5300-5400			26
2400 - 2500		17	1	5400-5500			28
2500 - 2600		REFUSAL	1	5500-5600			REFUSAL
2600 - 2700			2	5600-5700			
2700 - 2800			2	5700-5800			
2800 - 2900			2	5800-5900			
2900 - 3000			2	5900-6000			

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	HORTON COASTAL ENGINEERING / HASKONING AUSTRALIA PTY LTD		
Project:	EXISTING FORESHORE PROTECTION MEASURES		
Location:	1168 - 1182 PITTWATER ROAD, COLLAROY, NSW		
Job No.	30005ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	2-12-16	Rod Diameter: 16mm	
Tested By:	L.M.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration							
Test Location	RL≈6.5m	RL≈6.2m	RL≈6.4m	Test Location			
Depth (mm)	**4**	**5A**	**5B**	Depth (mm)	**4**	**5A**	**5B**
0 - 100	EXCAVATED	EXCAVATED	PUSHED	3000-3100	6	1	6
100 - 200			1	3100-3200	5	2	7
200 - 300			1	3200-3300	5	1	5
300 - 400			1	3300-3400	5	1	4
400 - 500			1	3400-3500	4	2	5
500 - 600			1	3500-3600	4	1	6
600 - 700			2	3600-3700	4	1	7
700 - 800			2	3700-3800	4	2	8
800 - 900			2	3800-3900	4	1	8
900 - 1000			2	3900-4000	4	1	9
1000 - 1100			1	4000-4100	4	2	9
1100 - 1200			2	4100-4200	5	2	8
1200 - 1300			2	4200-4300	5	2	7
1300 - 1400			2	4300-4400	5	1	6
1400 - 1500			2	4400-4500	5	2	6
1500 - 1600		PUSHED	3	4500-4600	5	2	6
1600 - 1700			4	4600-4700	6	2	6
1700 - 1800		2	4	4700-4800	6	10	8
1800 - 1900		3	4	4800-4900	5	10	8
1900 - 2000	2	32	4	4900-5000	6	8	10
2000 - 2100	3	1	4	5000-5100	16	12	10
2100 - 2200	4	1	4	5100-5200	24	13	9
2200 - 2300	6	2	4	5200-5300	6/50mm	19	17
2300 - 2400	6	2	5	5300-5400	REFUSAL	25/50mm	26
2400 - 2500	6	3	5	5400-5500		REFUSAL	31
2500 - 2600	8	4	5	5500-5600			REFUSAL
2600 - 2700	10	5	5	5600-5700			
2700 - 2800	8	5	6	5700-5800			
2800 - 2900	6	3	6	5800-5900			
2900 - 3000	4	1	6	5900-6000			

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	HORTON COASTAL ENGINEERING / HASKONING AUSTRALIA PTY LTD		
Project:	EXISTING FORESHORE PROTECTION MEASURES		
Location:	1168 - 1182 PITTWATER ROAD, COLLAROY, NSW		
Job No.	30005ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	2-12-16	Rod Diameter: 16mm	
Tested By:	L.M.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration

Test Location	RL≈6.2m	RL≈6.3m	RL≈6.8m	Test Location			
Depth (mm)	**6**	**7**	**8**	Depth (mm)	**6**	**7**	**8**
0 - 100	EXCAVATED	EXCAVATED	EXCAVATED	3000-3100	3	3	7
100 - 200				3100-3200	3	3	8
200 - 300				3200-3300	3	4	8
300 - 400				3300-3400	2	5	9
400 - 500				3400-3500	1	5	6
500 - 600				3500-3600	2	10	4
600 - 700				3600-3700	2	10	4
700 - 800				3700-3800	2	10	3
800 - 900				3800-3900	2	12	4
900 - 1000				3900-4000	2	13	4
1000 - 1100				4000-4100	2	7	4
1100 - 1200				4100-4200	2	4	4
1200 - 1300	1			4200-4300	2	4	4
1300 - 1400	1			4300-4400	2	5	4
1400 - 1500	2			4400-4500	2	6	5
1500 - 1600	2			4500-4600	4	9	6
1600 - 1700	3		1	4600-4700	3	9	6
1700 - 1800	4			4700-4800	7	10	6
1800 - 1900	3			4800-4900	6	11	6
1900 - 2000	4			4900-5000	7	12	9
2000 - 2100	3	1	3	5000-5100	12	19	17
2100 - 2200	3	1	3	5100-5200	13	24	17
2200 - 2300	4	1	4	5200-5300	14	26	18
2300 - 2400	2	1	3	5300-5400	15	27	21
2400 - 2500	1	1	4	5400-5500	28	30	29
2500 - 2600	1	2	4	5500-5600	REFUSAL	REFUSAL	REFUSAL
2600 - 2700	2	2	5	5600-5700			
2700 - 2800	3	2	6	5700-5800			
2800 - 2900	3	2	7	5800-5900			
2900 - 3000	3	2	8	5900-6000			

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

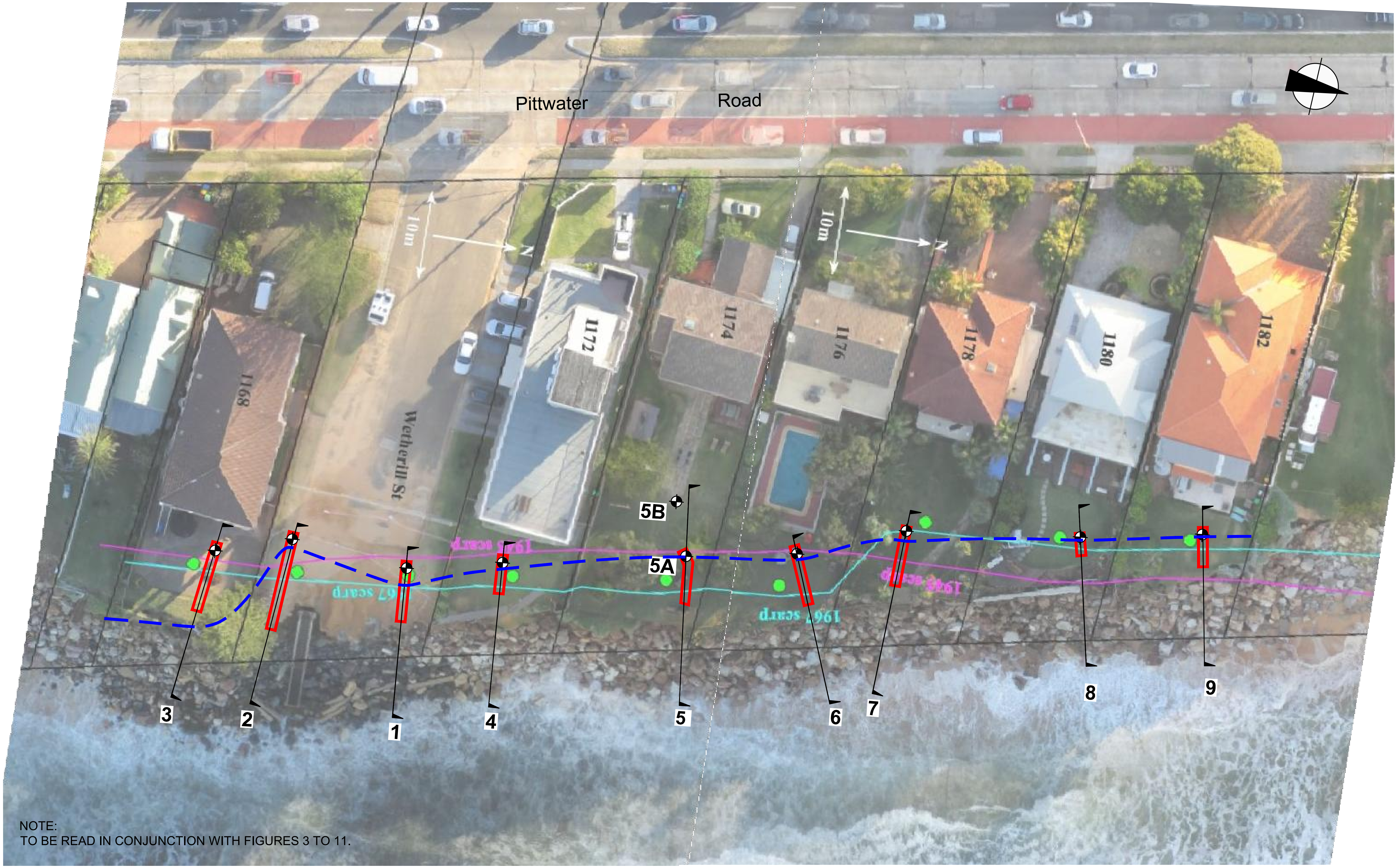
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	HORTON COASTAL ENGINEERING / HASKONING AUSTRALIA PTY LTD		
Project:	EXISTING FORESHORE PROTECTION MEASURES		
Location:	1168 - 1182 PITTWATER ROAD, COLLAROY, NSW		
Job No.	30005ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	2-12-16	Rod Diameter: 16mm	
Tested By:	L.M.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration

Test Location	RL≈6.8m			Test Location			
Depth (mm)	**9**			Depth (mm)	**9**		
0 - 100	EXCAVATED			3000-3100	6		
100 - 200				3100-3200	8		
200 - 300				3200-3300	7		
300 - 400				3300-3400	8		
400 - 500				3400-3500	8		
500 - 600				3500-3600	7		
600 - 700				3600-3700	7		
700 - 800				3700-3800	8		
800 - 900				3800-3900	8		
900 - 1000				3900-4000	8		
1000 - 1100				4000-4100	5		
1100 - 1200				4100-4200	4		
1200 - 1300				4200-4300	4		
1300 - 1400				4300-4400	4		
1400 - 1500				4400-4500	6		
1500 - 1600				4500-4600	9		
1600 - 1700				4600-4700	10		
1700 - 1800				4700-4800	10		
1800 - 1900				4800-4900	12		
1900 - 2000				4900-5000	15		
2000 - 2100	2			5000-5100	19		
2100 - 2200	3			5100-5200	22		
2200 - 2300	3			5200-5300	28		
2300 - 2400	4			5300-5400	32		
2400 - 2500	5			5400-5500	REFUSAL		
2500 - 2600	5			5500-5600			
2600 - 2700	6			5600-5700			
2700 - 2800	6			5700-5800			
2800 - 2900	6			5800-5900			
2900 - 3000	7			5900-6000			

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD.



LEGEND

DCP TEST

TEST PIT

SECTION LINE

INFERRED LANDWARD MARGIN OF REVETMENT

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title: **TEST LOCATION PLAN**

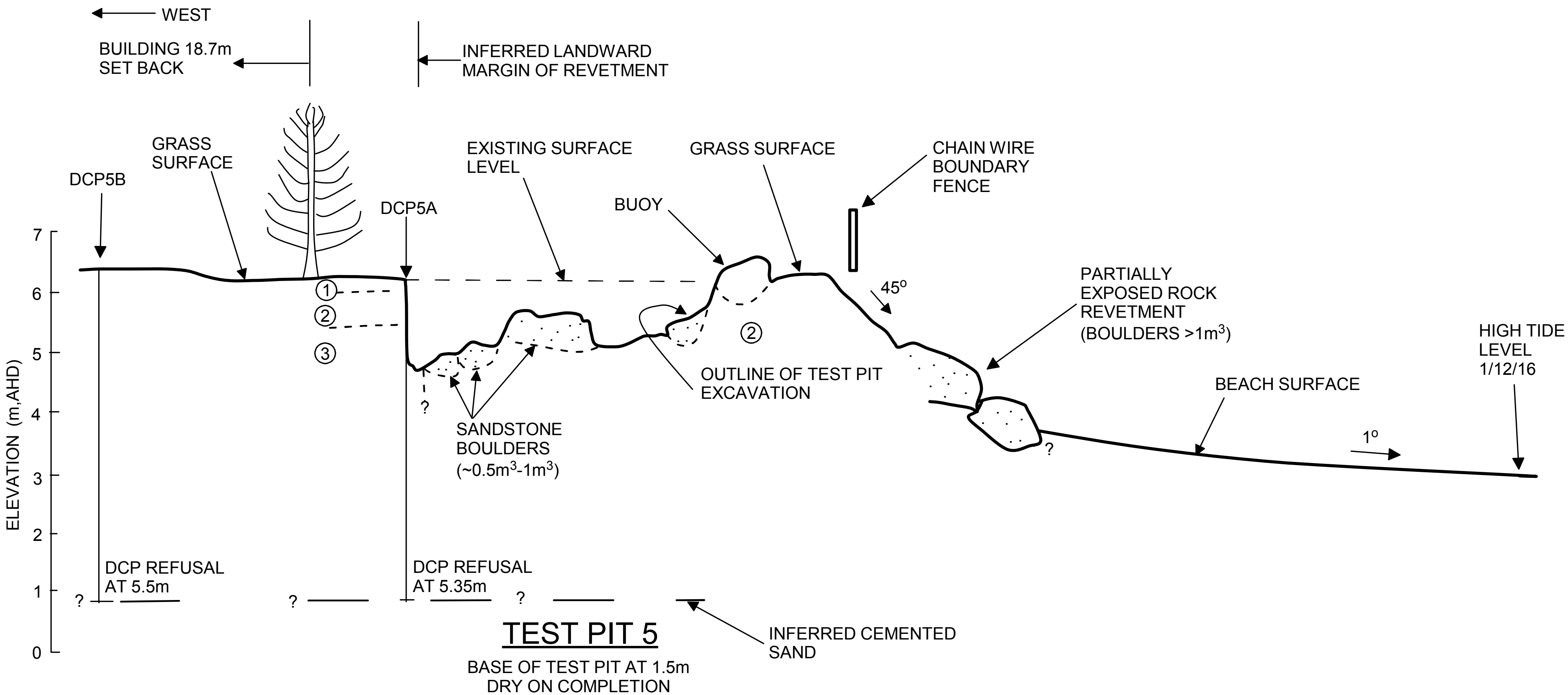
Location: 1168-1182 PITTWATER ROAD COLLAROY, NSW

Report No: 30005ZR

Figure No: 2

JK Geotechnics

PLOT DATE: 19/12/2016 1:49:12 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30005ZR COLLAROY\CAD\30005ZR.DWG



1 0.0-0.15m FILL: Silty sand topsoil, fine to medium grained, brown, with root fibres.

2 0.15-1.0m FILL: Silty sand, fine to medium grained, light brown, with gravel and cobbles sized bricks, concrete, tile and metal inclusions, trace of ash. Appears poorly compacted.

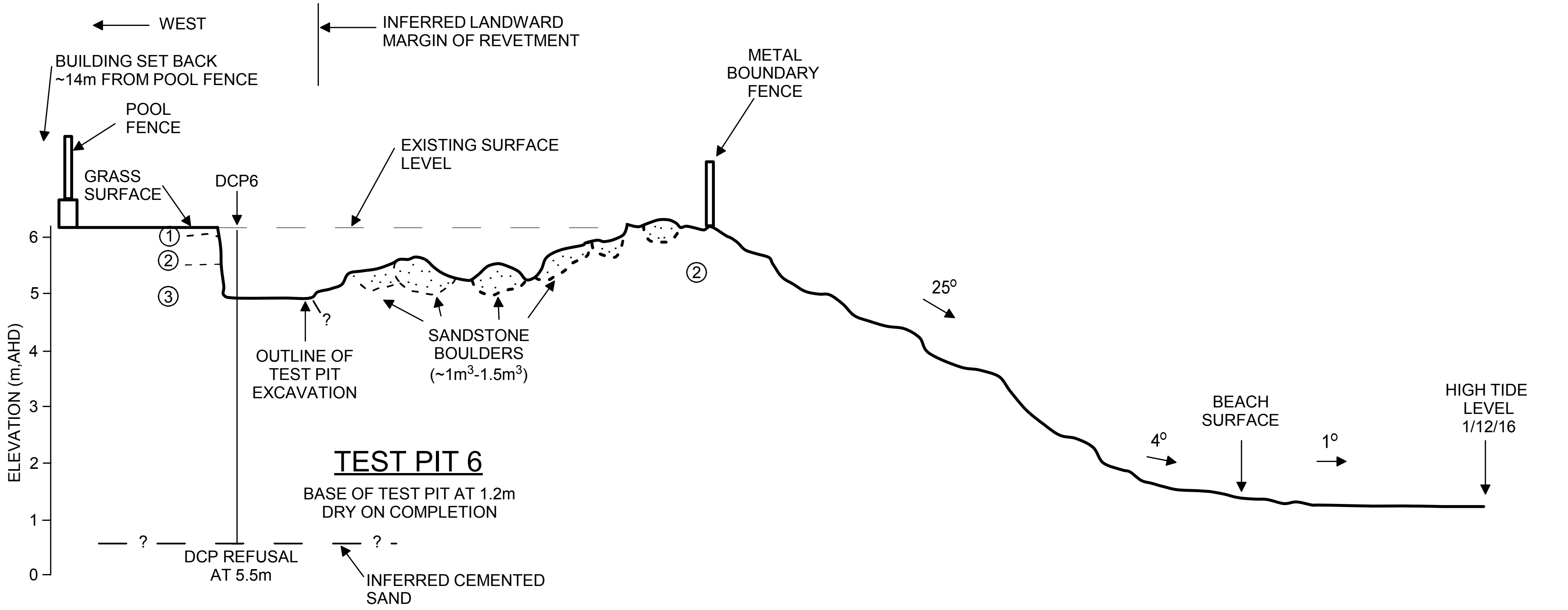
3 1.0-1.9m SAND (SP): fine to medium grained, yellow. Dry

1174 PITTWATER ROAD SECTION LINE FIVE (LOOKING NORTH)

0 5
Scale 1:100 @A4 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30005ZR Figure No. 7



① 0.0-0.20m FILL: Silty sand topsoil, fine to medium grained, brown, with root fibres.

② 0.2-0.7m FILL: Silty sand, fine to medium grained, brown, with fine to coarse grained sandstone gravel, cobbles and boulders, and gravel and cobbles sized brick and concrete inclusions, and metal, trace of ash. Inferred to be poorly compacted.

③ 0.7-1.2m SAND (SP): fine to medium grained, yellow. Dry

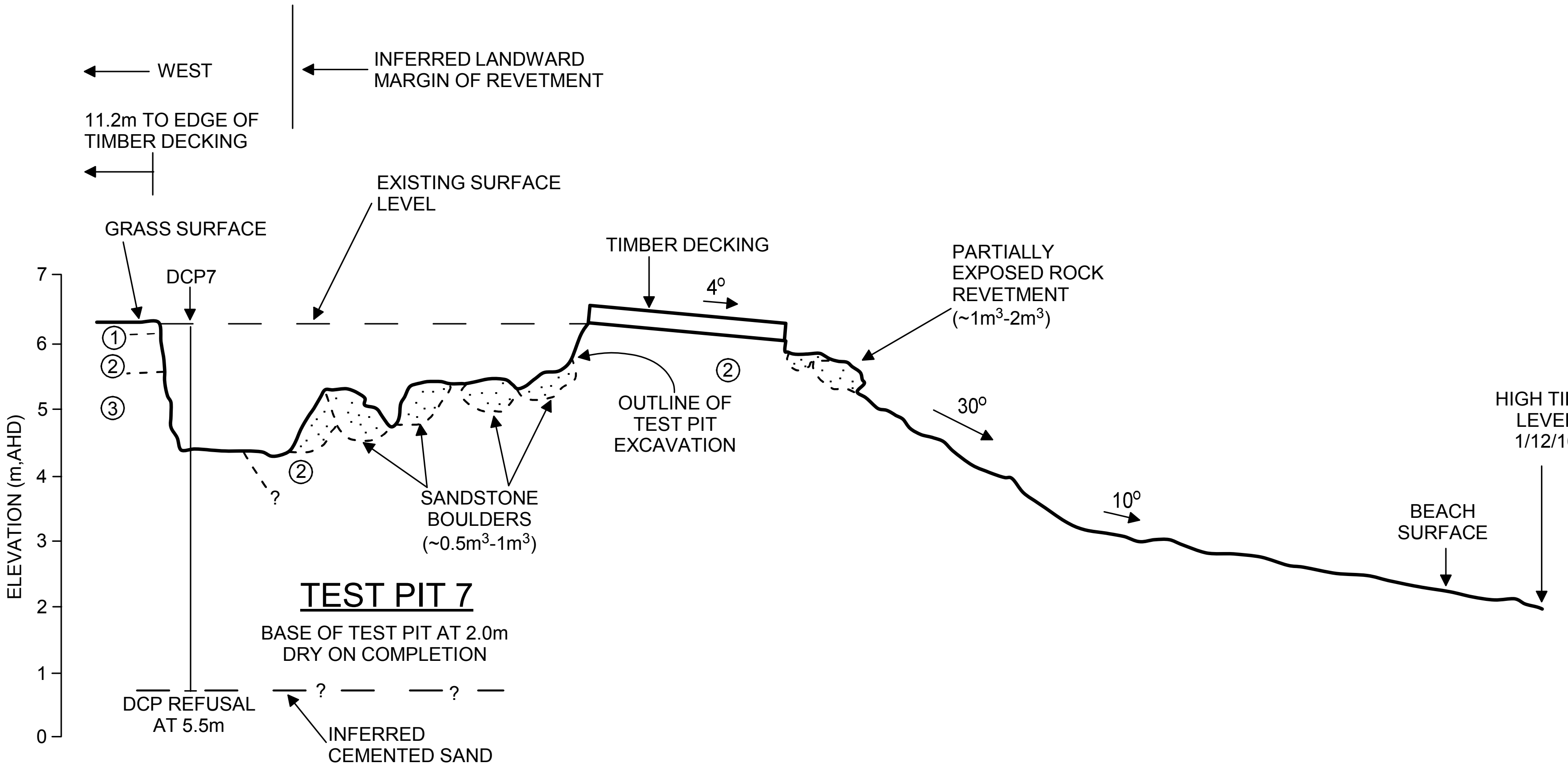
Scale 0 – 5 Metres 1:100 @A4

1176 PITTWATER ROAD SECTION LINE SIX (LOOKING NORTH)

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30005ZR

Figure No. 8



1 0.0 - 0.20m FILLL: Silty sand topsoil, fine to medium grained, brown, with root fibres.

2 0.2 - 0.8m FILL: Silty sand, fine to medium grained, brown and dark orange brown, with fine to coarse grained sandstone gravel cobbles and boulders, gravel and cobble sized bricks and concrete inclusions, and metal, trace of ash. Inferred to be poorly compacted.

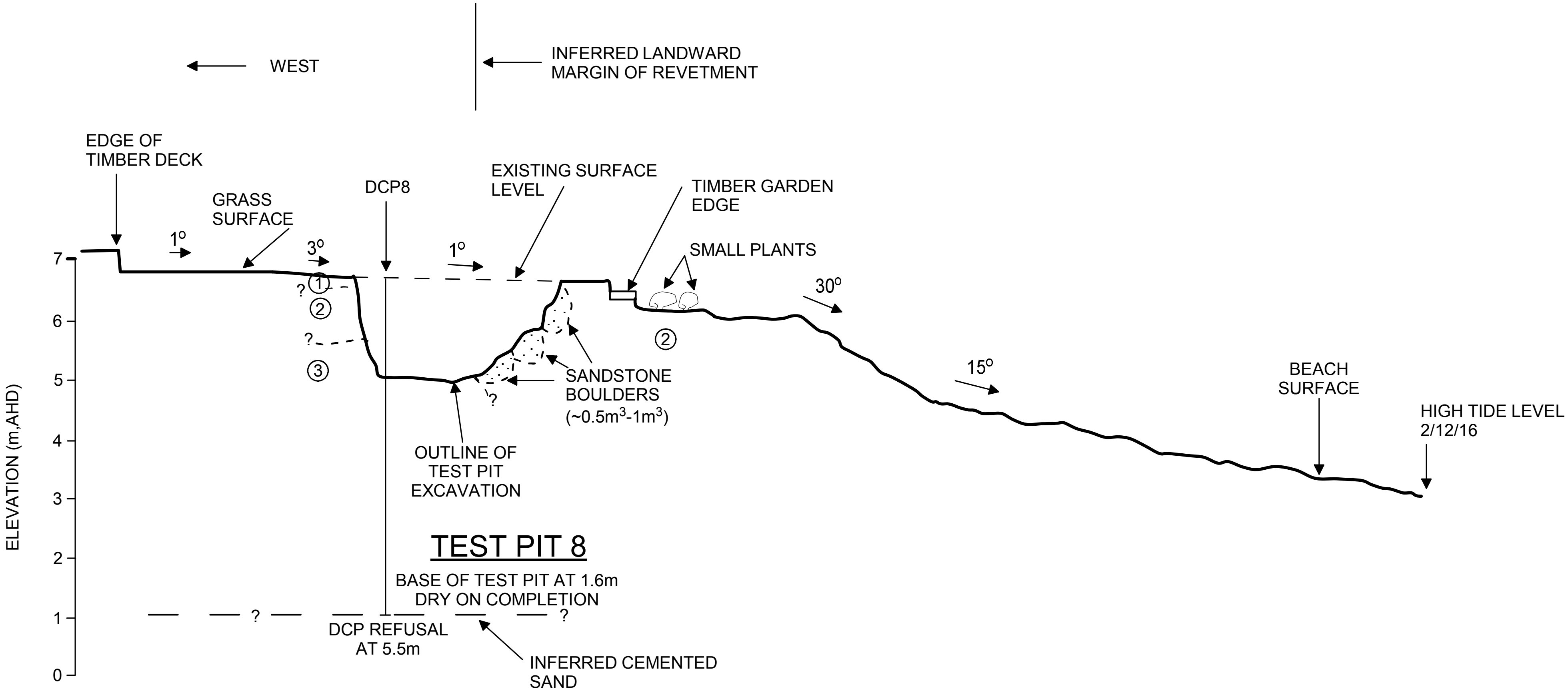
3 0.8-2.0m SAND (SP): fine to medium grained, yellow. Dry

1178 PITTWATER ROAD SECTION LINE SEVEN (LOOKING NORTH)

0 5
Scale 1:100 @A4 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30005ZR Figure No. 9



① 0.0 - 0.20m FILL: Silty sand topsoil, fine to medium grained, dark brown, with root fibres.

② 0.2 -1.0m FILL: Silty sand, fine to medium grained, brown, with gravel and cobble sized sandstone brick and concrete. Inferred to be poorly compacted.

③ 1.0m-1.6m SAND (SP): fine to medium grained, yellow. Dry

0 5

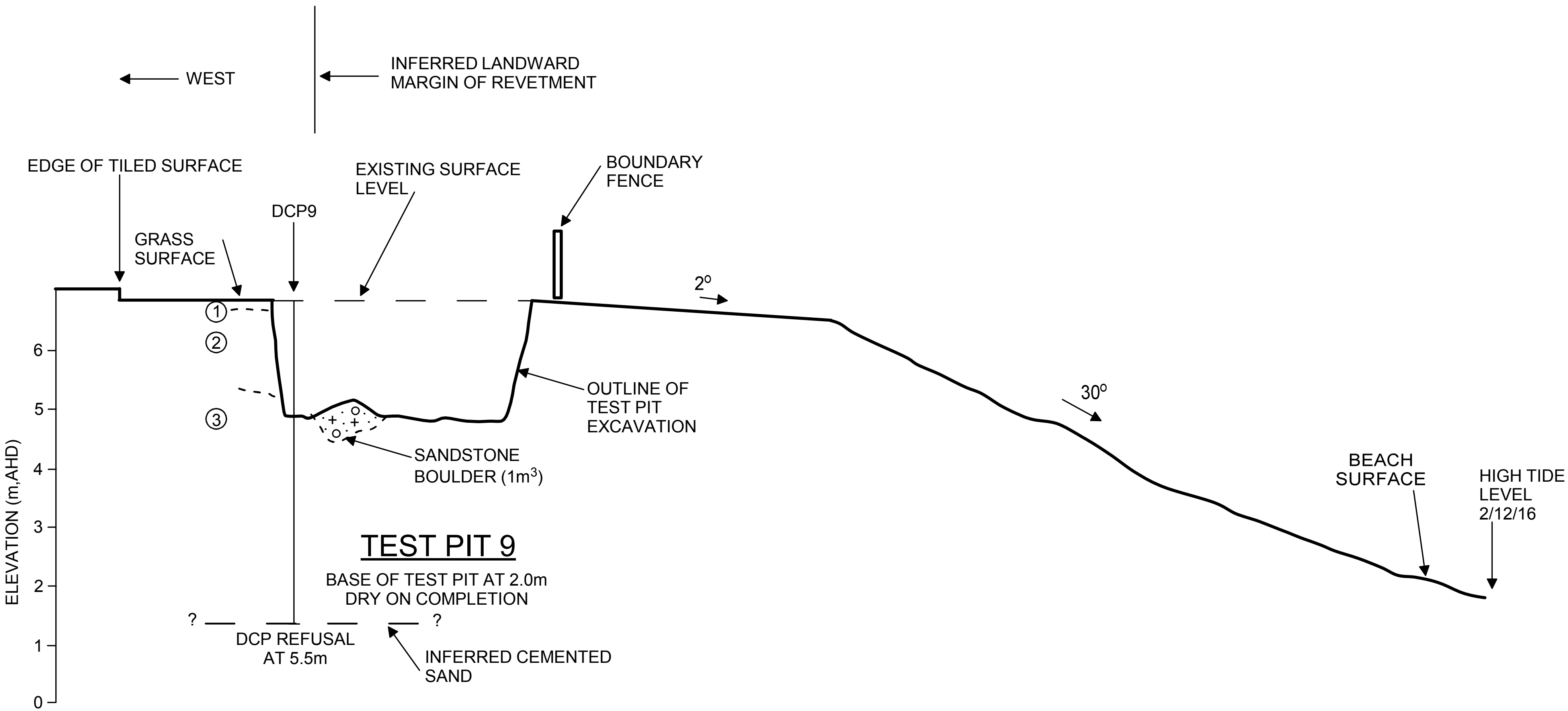
Scale 1:100 @A4 Metres

1180 PITTWATER ROAD
SECTION LINE EIGHT
(LOOKING NORTH)

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30005ZR

Figure No. 10



1 0.0-0.20m FILL: Silty sand topsoil, fine to medium grained, brown, with root fibres.

2 0.2-1.6m FILL: Silty sand, fine to medium grained, brown, with sandstone gravel, cobbles and boulders, trace of brick, tile and ash inclusions. Inferred to be poorly compacted.

3 1.6 -2.0m SAND (SP): fine to medium grained, yellow. Dry

1182 PITTWATER ROAD SECTION LINE NINE (LOOKING NORTH)

0 5
Scale 1:100 @A4 Metres

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30005ZR Figure No. 11

APPENDIX C

WALLAP ANALYSES OUTPUT SUMMARY

J.K. GEOTECHNICS
Program: WALLAP Version 6.06 Revision A51.B69.R54
Licensed from GEOSOLVE
Data filename/Run ID: 30751RDSection1(1182)FOS=1
Contiguous Pile Sea Wall
Section 1 (1182) Pittwater Road, Narrabeen

| Sheet No.
| Job No. 30751RD
| Made by : DS
|
| Date:17-02-2020
| Checked :

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types Left side		Soil types Right side	
1	2.00	8	Sand (loose)	8	Sand (loose)
2	1.00	3	Sand (Medium Dense)	3	Sand (Medium Dense)
3	0.50	4	Cemented Sand	4	Cemented Sand
4	-3.90	11	Sand (Dense)	11	Sand (Dense)
5	-4.70	12	Sand (Very Loose)	12	Sand (Very Loose)
6	-6.80	3	Sand (Medium Dense)	3	Sand (Medium Dense)

SOIL PROPERTIES

-- Soil type -- No.	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	Sand fill	16.00	20000	0.500	OC (0.300)	0.318 (0.000)	3.868 (0.000)	
2	Not defined							
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 (loose)	16.00	20000	0.500	OC (0.300)	0.292 (0.000)	4.369 (0.000)	
9	Compacted Sand Fill	18.00	40000	0.500	OC (0.300)	0.256 (0.000)	5.303 (0.000)	
10	Not defined							
11	Sand (Dense)	19.00	80000	0.350	OC (0.300)	0.224 (0.000)	6.535 (0.000)	
12	Sand (Very Loose)	16.00	10000	0.500	OC (0.300)	0.346 (0.000)	3.442 (0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No.	Description	--- parameters for Ka --- Soil friction angle	Wall adhesion coeff.	Back-fill angle	--- parameters for Kp --- Soil friction angle	Wall adhesion coeff.	Back-fill angle
1	Sand fill	28.00	0.500	0.00	28.00	0.500	0.00
2	Not defined						
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 (loose)	30.00	0.500	0.00	30.00	0.500	0.00
9	Compacted Sand Fill	33.00	0.500	0.00	33.00	0.500	0.00
10	Not defined						
11	Sand (Dense)	36.00	0.500	0.00	36.00	0.500	0.00
12	Sand (Very Loose)	26.00	0.500	0.00	26.00	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 side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Right side Point no.	Elev. m	Piezo elev. m	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 = -7.50
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.5000E+07 kN/m2
Moment of inertia of wall I = 0.020760 m4/m run
E.I = 519000 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+09	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	Far edge	Equiv. soil type	Partial factor/ Category
1	6.50	18.00(L)	1.00	0.50	80.00	=	N/A	N/A
2	7.00	0.00(L)	1.00	18.00	5.00	=	N/A	N/A

Note: L = Left side, R = Right side

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Fill to elevation 3.00 on LEFT side with soil type 9
2	Change EI of wall to 89250 kN.m2/m run From elevation 6.50 to 3.80 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.1 at elevation 6.50
6	Fill to elevation 4.20 on RIGHT side with soil type 1
7	Excavate to elevation 4.20 on RIGHT side Toe of berm at elevation 2.50 Width of top of berm = 0.10 Width of toe of berm = 10.00
8	Apply surcharge no.2 at elevation 7.00
9	Apply water pressure profile no.1
10	Excavate to elevation -2.80 on RIGHT side Toe of berm at elevation -3.80 Width of top of berm = 0.01 Width of toe of berm = 30.00

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	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Fill to elev. 3.00 on LEFT side	No	No	No
2	Change EI of wall to 89250kN.m2/m run	No	No	No
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.1 at elev. 6.50	Yes	Yes	Yes
6	Fill to elev. 4.20 on RIGHT side	No	No	No
7	Excav. to elev. 4.20 on RIGHT side	No	No	No
8	Apply surcharge no.2 at elev. 7.00	Yes	Yes	Yes
9	Apply water pressure profile no.1	Yes	Yes	Yes
10	Excav. to elev. -2.80 on RIGHT side	Yes	Yes	Yes
*	Summary output	Yes	-	Yes

J.K. GEOTECHNICS | Sheet No.
Program: WALLAP Version 6.06 Revision A51.B69.R54 | Job No. 30751RD
Licensed from GEOSOLVE | Made by : DS
Data filename/Run ID: 30751RDSection1(1182)FOS=1 |
Contiguous Pile Sea Wall | Date:17-02-2020
Section 1 (1182) Pittwater Road, Narrabeen | Checked :

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. = -7.50 Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.250 Toe elev.	Wall Penetr -ation	Direction of failure
1	3.00	2.00	---	Conditions not suitable for FoS calc.				
2	3.00	2.00	---	Conditions not suitable for FoS calc.				
3	3.00	2.00		No analysis at this stage				
4	7.00	2.00	3.50	8.063	n/a	1.32	0.68	L to R
5	7.00	2.00	3.50	8.062	n/a	1.32	0.68	L to R
6	7.00	4.20	3.50	Conditions not suitable for FoS calc.				
7	7.00	4.20	3.50	9.925	n/a	***	***	L to R
8	7.00	4.20	3.50	9.925	n/a	***	***	L to R
9	7.00	4.20	3.50	8.086	n/a	***	***	L to R
10	7.00	-2.80	3.50	0.997	n/a	***	***	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	Y	Displacement		Bending moment		Shear force	
no.	coord	maximum	minimum	maximum	minimum	maximum	minimum
		m	m	kN.m/m	kN.m/m	kN/m	kN/m
1	7.00	0.001	-0.008	0.0	-0.0	0.0	0.0
2	6.50	0.001	-0.007	5.1	-0.0	24.0	0.0
3	6.05	0.001	-0.006	20.9	0.0	54.0	0.0
4	5.60	0.001	-0.005	51.7	0.0	81.1	0.0
5	4.90	0.001	-0.003	123.0	0.0	117.2	0.0
6	4.20	0.001	-0.001	216.8	-0.0	144.5	0.0
7	3.80	0.001	-0.000	277.2	-0.0	155.0	0.0
8	3.50	0.003	-0.000	324.7	-0.0	160.9	-211.2
9	3.00	0.005	-0.000	221.3	-0.0	0.0	-201.1
10	2.50	0.008	-0.000	123.8	0.0	0.6	-188.4
11	2.00	0.011	-0.000	33.5	-6.4	2.4	-172.7
12	1.50	0.014	-0.000	10.2	-47.7	5.1	-152.7
13	1.00	0.017	-0.000	14.0	-118.4	19.5	-129.7
14	0.50	0.020	-0.000	21.8	-177.2	30.5	-105.2
15	0.00	0.023	-0.000	31.2	-224.3	30.5	-83.2
16	-0.50	0.025	-0.000	41.1	-259.6	29.7	-58.2
17	-1.00	0.028	0.000	55.7	-281.9	27.8	-31.6
18	-1.45	0.030	0.000	67.7	-290.5	24.6	-7.1
19	-1.90	0.032	0.000	77.8	-288.0	19.1	0.0
20	-2.35	0.034	0.000	84.9	-274.2	42.8	0.0
21	-2.80	0.036	0.000	88.0	-249.0	68.4	-2.9
22	-3.30	0.038	0.000	85.0	-211.1	80.2	-14.6
23	-3.80	0.039	0.000	72.3	-176.5	58.5	-33.4
24	-3.90	0.040	0.000	68.7	-171.1	50.4	-37.5
25	-4.70	0.042	0.000	32.6	-135.9	29.9	-56.0
26	-5.15	0.044	0.000	11.1	-118.5	40.3	-39.5
27	-5.60	0.045	0.000	0.0	-98.2	49.5	-23.5
28	-6.20	0.047	0.000	0.0	-65.0	60.3	-2.9
29	-6.80	0.048	0.000	0.0	-25.8	69.5	0.0
30	-7.50	0.050	0.000	0.0	-0.0	0.0	-0.0

Summary of results (continued)

Maximum and minimum bending moment and shear force at each stage

Stage	Bending moment				Shear force			
no.	maximum kN.m/m	elev.	minimum kN.m/m	elev.	maximum kN/m	elev.	minimum kN/m	elev.
1	32.8	-2.80	-5.2	-6.20	10.0	0.50	-21.4	-4.70
2	32.8	-2.80	-5.2	-6.20	10.0	0.50	-21.4	-4.70
3	No calculation at this stage							
4	87.9	-2.80	-10.8	-6.20	39.9	3.50	-56.0	-4.70
5	88.0	-2.80	-10.8	-6.20	39.9	3.50	-56.0	-4.70
6	65.7	-2.80	-8.1	-6.20	36.2	3.50	-38.2	-4.70
7	65.7	-2.80	-8.1	-6.20	36.2	3.50	-38.2	-4.70
8	65.8	-2.80	-8.1	-6.20	37.7	3.50	-38.3	-4.70
9	78.2	-2.80	-9.6	-6.20	40.1	3.50	-48.9	3.50
10	324.7	3.50	-290.5	-1.45	160.9	3.50	-211.2	3.50

Maximum and minimum displacement at each stage

Stage	Displacement				Stage description
no.	maximum m	elev.	minimum m	elev.	
1	0.003	-7.50	-0.000	7.00	Fill to elev. 3.00 on LEFT side
2	0.000	-7.50	-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	-7.50	0.000	7.00	Fill to elev. 7.00 on LEFT side
5	0.006	-7.50	0.000	7.00	Apply surcharge no.1 at elev. 6.50
6	0.003	-7.50	0.000	7.00	Fill to elev. 4.20 on RIGHT side
7	0.003	-7.50	0.000	7.00	Excav. to elev. 4.20 on RIGHT side
8	0.003	-7.50	0.000	7.00	Apply surcharge no.2 at elev. 7.00
9	0.004	-7.50	0.000	7.00	Apply water pressure profile no.1
10	0.050	-7.50	-0.008	7.00	Excav. to elev. -2.80 on RIGHT side

Strut forces at each stage (horizontal components)

Stage	Strut no. 1 at elev. 3.50	
no.	kN/m run	kN/strut
4	93.84	281.52
5	93.84	281.52
6	72.08	216.23
7	72.08	216.23
8	74.78	224.35
9	89.05	267.16
10	372.12	1116.36

J.K. GEOTECHNICS | Sheet No.
Program: WALLAP Version 6.06 Revision A51.B69.R54 | Job No. 30751RD
Licensed from GEOSOLVE | Made by : DS
Data filename/Run ID: 30751RDSection1(1182)Final |
Contiguous Pile Sea Wall | Date:17-02-2020
Section 1 (1182) Pittwater Road, Narrabeen | Checked :

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types Left side		Soil types Right side	
1	2.00	8	Sand (loose)	8	Sand (loose)
2	1.00	3	Sand (Medium Dense)	3	Sand (Medium Dense)
3	0.50	4	Cemented Sand	4	Cemented Sand
4	-3.90	11	Sand (Dense)	11	Sand (Dense)
5	-4.70	12	Sand (Very Loose)	12	Sand (Very Loose)
6	-6.80	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	Sand fill	16.00	20000	0.500	OC (0.300)	0.318 (0.000)	3.868 (0.000)	
2	Not defined							
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 (loose)	16.00	20000	0.500	OC (0.300)	0.292 (0.000)	4.369 (0.000)	
9	Compacted Sand Fill	18.00	40000	0.500	OC (0.300)	0.256 (0.000)	5.303 (0.000)	
10	Not defined							
11	Sand (Dense)	19.00	80000	0.350	OC (0.300)	0.224 (0.000)	6.535 (0.000)	
12	Sand (Very Loose)	16.00	10000	0.500	OC (0.300)	0.346 (0.000)	3.442 (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	Sand fill	28.00	0.500	0.00	28.00	0.500	0.00
2	Not defined						
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 (loose)	30.00	0.500	0.00	30.00	0.500	0.00
9	Compacted Sand Fill	33.00	0.500	0.00	33.00	0.500	0.00
10	Not defined						
11	Sand (Dense)	36.00	0.500	0.00	36.00	0.500	0.00
12	Sand (Very Loose)	26.00	0.500	0.00	26.00	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 side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Right side Point no.	Elev. m	Piezo elev. m	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 = -7.50
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.5000E+07 kN/m2
Moment of inertia of wall I = 0.020760 m4/m run
E.I = 519000 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+09	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	6.50	18.00(L)	1.00	0.50	80.00	=	N/A	N/A
2	7.00	0.00(L)	1.00	18.00	5.00	=	N/A	N/A

Note: L = Left side, R = Right side

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Fill to elevation 3.00 on LEFT side with soil type 9
2	Change EI of wall to 89250 kN.m2/m run From elevation 6.50 to 3.80 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.1 at elevation 6.50
6	Fill to elevation 4.20 on RIGHT side with soil type 1
7	Excavate to elevation 4.20 on RIGHT side Toe of berm at elevation 2.50 Width of top of berm = 0.10 Width of toe of berm = 10.00
8	Apply surcharge no.2 at elevation 7.00
9	Apply water pressure profile no.1
10	Excavate to elevation -1.80 on RIGHT side Toe of berm at elevation -2.80 Width of top of berm = 0.01 Width of toe of berm = 30.00

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. 3.00 on LEFT side	No	No	No
2	Change EI of wall to 89250kN.m2/m run	No	No	No
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.1 at elev. 6.50	Yes	Yes	Yes
6	Fill to elev. 4.20 on RIGHT side	No	No	No
7	Excav. to elev. 4.20 on RIGHT side	No	No	No
8	Apply surcharge no.2 at elev. 7.00	Yes	Yes	Yes
9	Apply water pressure profile no.1	Yes	Yes	Yes
10	Excav. to elev. -1.80 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)

				FoS for toe elev. = -7.50		Toe elev. for FoS = 1.250		
Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation	Direction of failure
1	3.00	2.00	---	Conditions not suitable for FoS calc.				
2	3.00	2.00	---	Conditions not suitable for FoS calc.				
3	3.00	2.00		No analysis at this stage				
4	7.00	2.00	3.50	8.063	n/a	1.32	0.68	L to R
5	7.00	2.00	3.50	8.062	n/a	1.32	0.68	L to R
6	7.00	4.20	3.50	Conditions not suitable for FoS calc.				
7	7.00	4.20	3.50	9.915	n/a	***	***	L to R
8	7.00	4.20	3.50	9.915	n/a	***	***	L to R
9	7.00	4.20	3.50	8.050	n/a	***	***	L to R
10	7.00	-1.80	3.50	1.646	n/a	-3.86	2.06	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	0.001	-0.002	0.0	-0.0	0.0	0.0
2	6.50	0.001	-0.001	2.4	0.0	13.2	0.0
3	6.05	0.001	-0.001	11.2	0.0	24.7	0.0
4	5.60	0.001	-0.001	24.9	0.0	35.7	0.0
5	4.90	0.001	-0.001	56.3	0.0	52.3	0.0
6	4.20	0.001	-0.000	99.1	-0.0	68.2	0.0
7	3.80	0.001	-0.000	128.3	-0.0	76.9	0.0
8	3.50	0.002	-0.000	152.3	-0.0	83.4	-137.9
9	3.00	0.003	-0.000	86.3	-0.0	0.0	-126.3
10	2.50	0.004	-0.000	26.2	0.0	0.6	-113.0
11	2.00	0.005	-0.000	10.9	-26.4	2.4	-97.3
12	1.50	0.006	-0.000	10.2	-70.2	5.1	-77.2
13	1.00	0.007	-0.000	14.0	-103.1	19.4	-54.2
14	0.50	0.008	-0.000	21.8	-124.2	30.4	-29.8
15	0.00	0.009	0.000	31.2	-133.5	30.4	-7.7
16	-0.50	0.010	0.000	40.9	-131.1	29.6	0.0
17	-1.00	0.011	0.000	55.5	-115.7	43.9	0.0
18	-1.80	0.012	0.000	76.1	-62.3	87.6	0.0
19	-2.30	0.013	0.000	84.6	-15.1	98.5	0.0
20	-2.80	0.013	0.000	88.3	0.0	75.9	-2.9
21	-3.35	0.014	0.000	84.6	0.0	34.2	-16.4
22	-3.90	0.015	0.000	68.9	0.0	0.0	-37.6
23	-4.70	0.016	0.000	33.9	0.0	0.0	-72.0
24	-5.15	0.016	0.000	11.2	0.0	0.0	-49.6
25	-5.60	0.017	0.000	0.0	-10.8	0.0	-27.8
26	-6.20	0.018	0.000	0.0	-18.9	0.3	-2.9
27	-6.80	0.019	0.000	0.0	-10.4	27.5	0.0
28	-7.50	0.020	0.000	0.0	-0.0	0.0	-0.0

Summary of results (continued)

Maximum and minimum bending moment and shear force at each stage

Stage	Bending moment				Shear force			
no.	maximum kN.m/m	elev.	minimum kN.m/m	elev.	maximum kN/m	elev.	minimum kN/m	elev.
1	32.9	-2.80	-5.2	-6.20	10.0	0.50	-21.5	-4.70
2	32.9	-2.80	-5.2	-6.20	10.0	0.50	-21.5	-4.70
3	No calculation at this stage							
4	88.2	-2.80	-10.7	-6.20	39.9	3.50	-56.0	-4.70
5	88.3	-2.80	-10.7	-6.20	39.9	3.50	-56.0	-4.70
6	66.0	-2.80	-8.1	-6.20	36.2	3.50	-38.2	-4.70
7	66.0	-2.80	-8.1	-6.20	36.2	3.50	-38.2	-4.70
8	66.1	-2.80	-8.1	-6.20	37.7	3.50	-38.3	-4.70
9	78.5	-2.80	-9.5	-6.20	40.1	3.50	-48.9	3.50
10	152.3	3.50	-133.5	0.00	98.5	-2.30	-137.9	3.50

Maximum and minimum displacement at each stage

Stage	Displacement				Stage description
no.	maximum m	elev.	minimum m	elev.	
1	0.003	-7.50	-0.000	7.00	Fill to elev. 3.00 on LEFT side
2	0.000	-7.50	-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	-7.50	0.000	7.00	Fill to elev. 7.00 on LEFT side
5	0.006	-7.50	0.000	7.00	Apply surcharge no.1 at elev. 6.50
6	0.003	-7.50	0.000	7.00	Fill to elev. 4.20 on RIGHT side
7	0.003	-7.50	0.000	7.00	Excav. to elev. 4.20 on RIGHT side
8	0.003	-7.50	0.000	7.00	Apply surcharge no.2 at elev. 7.00
9	0.004	-7.50	0.000	7.00	Apply water pressure profile no.1
10	0.020	-7.50	-0.002	7.00	Excav. to elev. -1.80 on RIGHT side

Strut forces at each stage (horizontal components)

Stage	--- Strut no. 1 --- at elev. 3.50	
no.	kN/m run	kN/strut
4	93.87	281.62
5	93.87	281.61
6	72.09	216.27
7	72.09	216.27
8	74.80	224.39
9	89.08	267.24
10	221.32	663.97

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

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

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

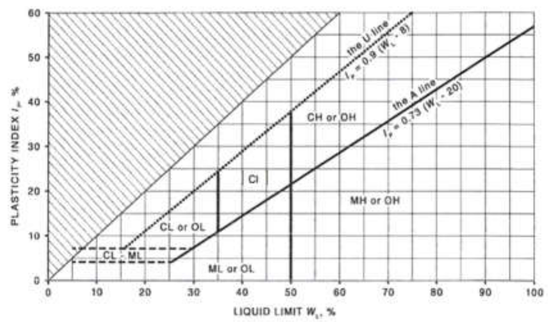
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.

Density Index/ Relative Density (Cohesionless Soils)	Symbol		Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres