Douglas Partners

Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Bondi Surf Bathers' Life Saving Club Upgrade
Queen Elizabeth Drive, Bondi Beach

Prepared for
Lockhart-Krause Architects Pty Ltd

Project 99567.00
November 2020

Integrated Practical Solutions

Douglas Partners
Geotechnics | Environment | Groundwater

Document History

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
Author	Peter Valenti	13 November 2020
Reviewer		13 November 2020

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Geotechnics / Environment / Groundwater

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Report on Geotechnical Investigation
Bondi Surf Bathers' Life Saving Club Upgrade
Queen Elizabeth Drive, Bondi Beach

1. Introduction

This revised report presents the results of a geotechnical investigation undertaken for the proposed Bondi Surf Bathers' Life Saving Club (BSBLSC) Upgrade, Bondi Beach. The work was commissioned by Lockhart-Krause Architects Pty Ltd.

It is understood that the proposed development includes a building extension over a single level basement at the rear of the existing two-storey BSBLSC. It is anticipated that bulk excavation will be required to depths of about 4.5 m, with localised deeper excavations for lift pits, services, shoring piles and foundations.

Geotechnical investigation was undertaken at accessible locations and included two cone penetration tests (CPTs), the excavation of two test pits to assess the existing foundation conditions of the building that will be restored and engineering analysis. Details of the field work and comments relevant to design and construction are given in this report.

DP also completed a geotechnical investigation for the adjacent Bondi Pavilion Upgrade. Relevant information from the previous investigation is provided in this report.

2. Site Description

The BSBLSC is located between Campbell Parade and Queen Elizabeth Drive, Bondi Beach, approximately midway along the beach. The site slopes gently to the south with ground surface levels ranging from about RL 10 m in the northern portion of the site adjacent to the carpark to RL 8 m around the front (south) end of the surf club.

The surf club includes one to two-storey buildings with a courtyard and ancillary structures at the rear (northern) end of the buildings. Paved paths and grassed parklands surround the BSBLSC. The Bondi Pavilion and a playground are located further to the west and east, respectively. The site is shown in Drawing 1 in Appendix B.

3. Regional Geology and Hydrogeology

The *Sydney 1:100 000 Geological Series Sheet* indicates that the site is underlain by medium to fine-grained marine sand with podsols. The sands are part of a transgressive dune system deposited by Aeolian (wind-blown) processes. The area to the south of the site is underlain by modern beach sands.

The groundwater table is anticipated to be about 5 – 6 m below the ground surface based on previous and current investigations. Groundwater is likely to follow the surface topography and flow to the south.

An extract from the geological map is shown in Figure 1.

Figure 1: Extract from geological map

The *Waverley Local Environmental Plan 2012* shows that the site is within Class 4 land in relation to acid sulfate soils. Class 4 land requires development consent in relation to acid sulfate soils for: works more than 2 m below the natural ground surface or works by which the water table is likely to be lowered more than 2 m below the natural ground surface.

4. Previous Investigation

Two CPTs (CPT5 and CPT6) were undertaken to approximate depths of 17.1 m with refusal occurring within very dense sand. The subsurface conditions generally included fill to about 0.6 – 1.3 m, underlain by sand of variable density although generally increasing in density with depth. Groundwater was encountered at a depth of 6.1 m (RL 4.1 m) in CPT5. The test hole collapsed at a depth of 5.2 m (RL 3.2 m) in CPT6 following the withdrawal of rods.

Two test pits (TP3 and TP4) were excavated to expose the existing footings that support the eastern wall of the Pavilion. The bases of the footings were founded at a depth of 0.55 m and 0.6 m (RL 8.1 m) on sandy filling, generally equivalent to a very loose to loose sand. These footings are located about 25 m away from the proposed BSBLSC basement.

The locations of the previous CPTs and test pits are shown on Drawing 1 in Appendix B. The results of the previous CPTs and test pits are provided in Appendix C.

The exposure classification was assessed as being Non-Aggressive towards buried steel and concrete structures founded above the water table in accordance with Australian Standard AS 2159 – 2009 *Piling – Design and installation*.

5. Field Work Methods

5.1 Cone Penetration Tests

Two cone penetration tests (CPT1 and CPT2) were undertaken to depths of 17.8 m and 15.7 m, respectively. A CPT involves pushing a 35 mm diameter instrumented cone and friction sleeve into the ground using hydraulic thrust from a ballasted truck-mounted testing rig. Measurements of cone resistance and sleeve friction are made at 20 mm depth intervals and are stored on a portable computer for subsequent interpretation.

The locations of the CPTs are shown in Drawing 1 in Appendix B.

The ground surface levels [relative to Australian height datum (AHD)] and co-ordinates at all test locations were measured using a differential global positional system, which is accurate to less than 0.1 m in the horizontal and vertical directions.

5.2 Test Pits

Two test pits (TP3 and TP4) were excavated to depths of 1.2 m and 1.0 m using hand tools. The purpose of these pits was to determine the depth of the existing building footings at the pit locations. A dynamic penetrometer test (DPT) was also undertaken at the base of each test pit to assess the strength/density of the foundation soils.

The locations of the test pits are shown in Drawing 1 in Appendix B.

6. Field Work Results

Notes relating to soil descriptions, sampling and testing procedures used are included in Appendix C, together with the results of the CPTs, and test pit logs with photographs of the footings prior to backfilling.

6.1 Cone Penetration Tests

The materials inferred from CPT1 and CPT2 can be described as follows:

- FILL: sandy fill to approximate depths of 0.5 m and 1.5 m; and
- SAND: silty sand and sand which was initially very loose and loose to 3.4 m in CPT1 and medium dense with loose bands to 2.2 m in CPT2, becoming medium dense to very dense to the base of the tests at 17.8 m and 15.7 m depth. There were also some loose layers within the dense materials, and a hard clay band about 14.0 m deep in CPT1 and 13.4 m deep in CPT2.

Groundwater was observed at depths of 5.7 m (RL 3.6 m) in CPT1. The test hole collapsed at 5.2 m (RL 4.1 m) in CPT2 following withdrawal of the CPT equipment.

6.2 Test Pits

The two pits (TP3 and TP4) encountered:

- FILL: sandy fill with varying proportions of brick, terracotta, gravel and concrete building materials to approximate depths of 0.5 m and 0.7 m; and
- SAND: medium dense sand below 0.5 m in TP1 and continuing to at least 2.7 m based on the DPT results. Initially very loose and loose sand to 1.2 m in TP2 becoming medium dense to about 2.9 m, at which depth the DPT was discontinued.

Groundwater was not observed within the depths of the test pits at the time of the field work.

The depths and levels of the base of the footing at each test pit location are summarised in Table 1. Further details are provided in Drawing 4 in Appendix B.

Table 1: Summary of Footing Depths and Levels

Location	Surface Level (RL m AHD)	Depth to Base of Footing (m)	Approximate Level of Base of Footing (RL m AHD)
TP3	8.3	0.91	7.4
TP4	8.3	0.96	7.3

7. Geotechnical Model

Two geotechnical cross-sections showing a summary of the interpreted subsoil layer boundaries, together with the proposed basement level are provided in Appendix B (Drawings 2 and 3).

The site appears to be underlain by sandy fill to depths of 0.5 m to 1.5 m, underlain by natural sandy soils. The natural sands were initially very loose and loose in some areas and medium dense in others, becoming medium dense to very dense with depth. A consistent very dense sand layer is present below approximate levels of RL 4 m to RL 5 m.

The regional groundwater table appears to be at or close to RL 3 to RL 4 m and is likely to fluctuate with climatic conditions.

8. Proposed Development

It is understood that the proposed development includes a building extension over a single level basement at the rear of the existing two-storey BSBLSC. It is anticipated that bulk excavation will be required to depths of about 4.5 m, with localised deeper excavations for lift pits, services, shoring piles and foundations. The proposed basement floor level is at RL 4.9 m. Parkland will extend around the new and existing buildings, except along the beach frontage.

9. Comments

9.1 Excavation Conditions

Following the removal of the existing buildings and slabs/pavements, excavation is expected to intersect sandy fill and sandy soils. Excavation in these materials should be readily achievable using conventional earthmoving equipment such as a hydraulic excavator with bucket attachment. Excavation in rock will not be required.

It should be noted that any off-site disposal of spoil will generally require assessment for re-use or classification in accordance with current *Waste Classification Guidelines* (NSW EPA, 2014).

Groundwater has been measured at levels of about RL 3 m to RL 4 m. It is understood that bulk excavation is required to about RL 4.6 m, assuming 0.3 m of over-excavation below the proposed finished floor level of RL 4.9 m to construct a slab-on-grade. The bulk excavation level is about 0.5 – 1.5 m above the groundwater table levels recorded to date, however given the potential for the water able to rise within the sands by possibly 1 – 2 m, excavation below the groundwater table may be required.

9.2 Groundwater and Dewatering

Given the proposed basement level at RL 4.9 m, it is recommended that long-term groundwater monitoring be undertaken to check the water levels during periods of prolonged rainfall, to allow comparison of groundwater levels to the proposed basement floor level and any deeper, localised excavations.

An understanding of the groundwater levels at this site will be important for the planning/approval process, design and construction of the proposed development. Depending on the results of the recommended long term groundwater monitoring, a drained or tanked basement design may be required.

If proposed excavations extend below groundwater then dewatering of the sand will be required. This may require a dewatering license or approval from WaterNSW to be obtained, and provision of a dewatering management plan.

Spear points connected to a pumping system are likely to be suitable for dewatering within sand. The groundwater level is typically lowered by 1 m below proposed excavation levels to provide a trafficable working surface and to allow for the installation of buried services. Dewatering locally around deeper lift pits within the basement area is considered appropriate.

An assessment of the groundwater contamination status for off-site disposal purposes and the potential impacts of dewatering on adjacent structures is recommended if dewatering is required. If a drained basement design is considered suitable for this project, and involves 'taking' groundwater from the aquifer over the project life then approvals from WaterNSW and other regulatory authorities are likely to be required.

9.3 Excavation Support

9.3.1 General

Vertical excavations in sandy fill and soil are not expected to be stable. Due to the proximity of the surf club building and footpaths, if the water table is below the proposed basement floor level of RL 4.9 m following periods of prolonged rainfall, it is recommended that a contiguous pile shoring wall comprising closely spaced (i.e. maximum gaps of 50 mm) continuous flight auger (CFA) piles be constructed for temporary and permanent support. Upon excavation drops, the pile gaps should be filled with dry-packed mortar or shotcrete to prevent sand loss between the piles.

If long term groundwater monitoring indicates the potential for groundwater to rise above proposed excavation levels then the use of a secant pile shoring wall with interlocking CFA piles to prevent sand loss between piles is recommended.

The use of driven steel sheet piles is considered to be inappropriate for this site given the proximity of vibration-sensitive structures such as the adjacent surf club building and the nearby Bondi Pavilion, both supported by shallow footings in very loose/loose sand. The use of sheet piles for localised excavations such as lift pits located within the perimeter of the shoring walls may be considered, using low-vibration sheet piling techniques.

Temporary batters within the perimeter of the shoring walls should be cut no steeper than 1.5(H):1(V), provided the batters are well above groundwater. Such batter slopes are applicable for excavations up to 3 m high and where not subjected to surcharge loads (i.e. from traffic or structures). Much flatter batter slopes would be required for excavations below groundwater.

Cantilevered or single-propped retaining walls that support very loose fill/sands could be designed on the basis of a coefficient of active earth pressure (K_a) of 0.4 where some wall movement is acceptable and a coefficient of at-rest earth pressure (K_o) of 0.6 where wall movement is to be limited such as adjacent to the existing surf club. An ultimate coefficient of passive earth pressure (K_p) of 3.0 could be used for medium dense sand. A bulk unit weight of 18 kN/m^3 for fill and sand above groundwater and a buoyant unit weight of 8 kN/m^3 for soil below groundwater is recommended.

A triangular lateral earth pressure distribution could be assumed for cantilevered retaining walls (not to be used adjacent to existing structures). For anchored/propped walls, it is recommended that shoring design is based on a trapezoidal pressure distribution where the maximum pressure acts over the central 60% of the wall. Lateral pressures due to surcharge loads from sloping ground surfaces, structures, construction machinery and traffic should be included where relevant.

Hydrostatic pressure should also be included in the design where adequate drainage is not provided behind the full height of the walls, and also on the passive side of the walls subject to results of further groundwater monitoring.

9.3.2 Ground Anchors

Temporary internal props or ground anchors may be required to restrict wall movements during the construction phase, with permanent support of retaining walls anticipated to be provided by the final structure.

As a preliminary guide, ground anchors are typically inclined at about 10° below the horizontal, have a free length equal to or greater than the height of the anchor above the base of the excavation and have a minimum free length of 3 m. A minimum bond length of 3 m should also be used.

For anchors in sands, the bond length design is dependent upon the overburden soil pressure, which depends upon the depth of the anchor below ground and the unit weight of the soil. The design of temporary ground anchors bonded into natural sands below at least 2 m depth may be carried out using an allowable bond stress of 20 kPa at the grout-sand interface, subject to review of groundwater levels from long term groundwater monitoring.

Secondary-grouted anchors could be used in the natural sand to increase the anchor capacity. This technique involves installing a conventionally-grouted anchor and then, once cured, injecting grout into the anchor at a higher pressure to crack the primary grout and densify the surrounding materials. This technique is fairly specialised and only experienced contractors should be engaged for the design and installation of secondary-grouted anchors.

9.4 Foundations

9.4.1 Raft Slab

A raft slab founded on loose to medium dense sand or engineered fill prepared in accordance with Section 9.7 of this report could be adopted.

Natural silty sand is anticipated to be exposed at the proposed basement level. It is recommended that a design subgrade California bearing ratio (CBR) of 6% be adopted for design of a raft slab following preparation as described in Section 9.7. Any material exposed during the works that exhibits clay-like properties should be reassessed on site, particularly as clayey soils are likely to have a lower CBR in the order of 3%.

9.4.2 Spread Footings

Spread footings (e.g. pad and strip footings) may be suitable for supporting new structures provided that they are founded in medium dense sand. The bearing capacity of a sand is dependent on the width and depth of the footing as well as the density of the foundation material. An allowable bearing pressure of 200 kPa could be assumed for footings founded in the medium dense sand provided that the footings are at least 0.6 m by 0.6 m (for pad footings), 0.6 m wide (for strip footings), founded at a depth of at least 0.6 m, and at least 1 m above the design groundwater level. A reduced bearing pressure would apply for spread footings founded within a distance equal to 1.5 times the minimum footing width to the design groundwater level.

The associated settlements are dependent upon the applied column load and should be assessed on a case-by-case basis using an elastic (Young's) modulus of 20 MPa for medium dense sand. The majority of the settlement would be expected to occur upon application of the initial load.

All footings should be inspected by a geotechnical professional to confirm the adequacy of the foundation material.

9.4.3 Piles

The basement shoring pile wall may also be used to support vertical loads from the building. The bearing pressures outlined in Table 2 could be used for the design of CFA piles.

Table 2: Design Parameters for CFA Piles

Material Description	Approximate Depth Below Ground Surface (m)	Allowable End-Bearing Pressure (kPa) [2]	Allowable Shaft Adhesion (kPa)[1]	Ultimate End-Bearing Pressure (kPa) [2]	Ultimate Shaft Adhesion (kPa)[1]	Young's Modulus (MPa)
Medium Dense Sand	3.5	400	20	1000	30	20
Dense to Very Dense Sand	10.5	1000	50	3000	120	75

Notes: [1] Reduce by 50% for uplift loads and ensure cone-pull-out criteria are met.
[2] Piles must be embedded a depth of at least 5 pile diameters below bulk excavation level

It should be noted that the serviceability limit-state is likely to govern the design of the CFA piles and the ultimate bearing pressures provided in Table 2 will probably need to be lowered in order to limit settlements to an acceptable level. An appropriate geotechnical strength reduction factor should be applied when using the limit-state approach as outlined in AS 2159 – 2009 *Piling – Design and installation*.

Settlement of a pile is dependent on the loads applied to the pile and the foundation conditions in the socket zone and below the pile toe. The total settlement of a CFA pile can be estimated by using the elastic (Young's) moduli provided in Table 2.

Soil decompression can occur during CFA piling when a strong stratum is encountered. In this case, the augers continue to rotate but the rate or auger progression decreases and soil from around the auger is displaced upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and should be avoided by monitoring auger speed and progression closely.

9.5 Acid Sulfate Soils

The soils above the groundwater table are not expected to be acid sulfate soils due to their wind-blown deposition. Development works in the upper 6 m to 7 m of fill/soil on the site should therefore not have to be undertaken with any special precautions in relation to acid sulfate soils.

The status of ASS below the groundwater table should be further assessed if shoring piles and excavations are proposed to extend below the groundwater table.

9.6 Seismicity

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class would be Class D_e due to the very loose sands encountered on the site.

9.7 Subgrade Preparation for Slabs-On-Grade

Natural sand is expected at the proposed excavation level. To support slabs-on-grade, it is recommended that subgrade preparation includes:

- Compaction of the exposed subgrade material using a smooth-drum roller;
- A proof roll test using the roller in non-vibration mode, observed by a geotechnical engineer to detect any soft spots that may be present. Rectification of any soft spots should be carried out as directed by the geotechnical engineer;
- Placement of engineered fill (if required) should be compacted to a dry density ratio of at least 98% relative to Standard compaction (or a Density Index of at least 75%); and

- Density testing of engineered fill should be undertaken in accordance with the recommendations provided in Australian Standard AS 3798 – 2007 Guidelines on earthworks for commercial and residential developments, to confirm that the material satisfies the compaction requirements.

9.8 Further Investigation

It is recommended that long-term groundwater monitoring be undertaken using data loggers in at least two groundwater wells to compare the groundwater table level relative to the proposed basement floor level and any other deeper, localised excavation levels. The groundwater level across the site will be important for the planning/approval process, design and construction of this project.

Further investigation of the status of ASS below the groundwater table should also be undertaken with a borehole investigation as shoring piles may extend below the groundwater table.

10. Limitations

Douglas Partners (DP) has prepared this report for this project at Queen Elizabeth Drive, Bondi Beach (Bondi Surf Bathers' Life Saving Club) in accordance with DP's proposal SYD191214.P.001.Rev1 dated 22 November 2019 and acceptance received from Lockhart-Krause Architects Pty Ltd dated 21 January 2020. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Lockhart-Krause Architects Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

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

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

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

Copyright

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

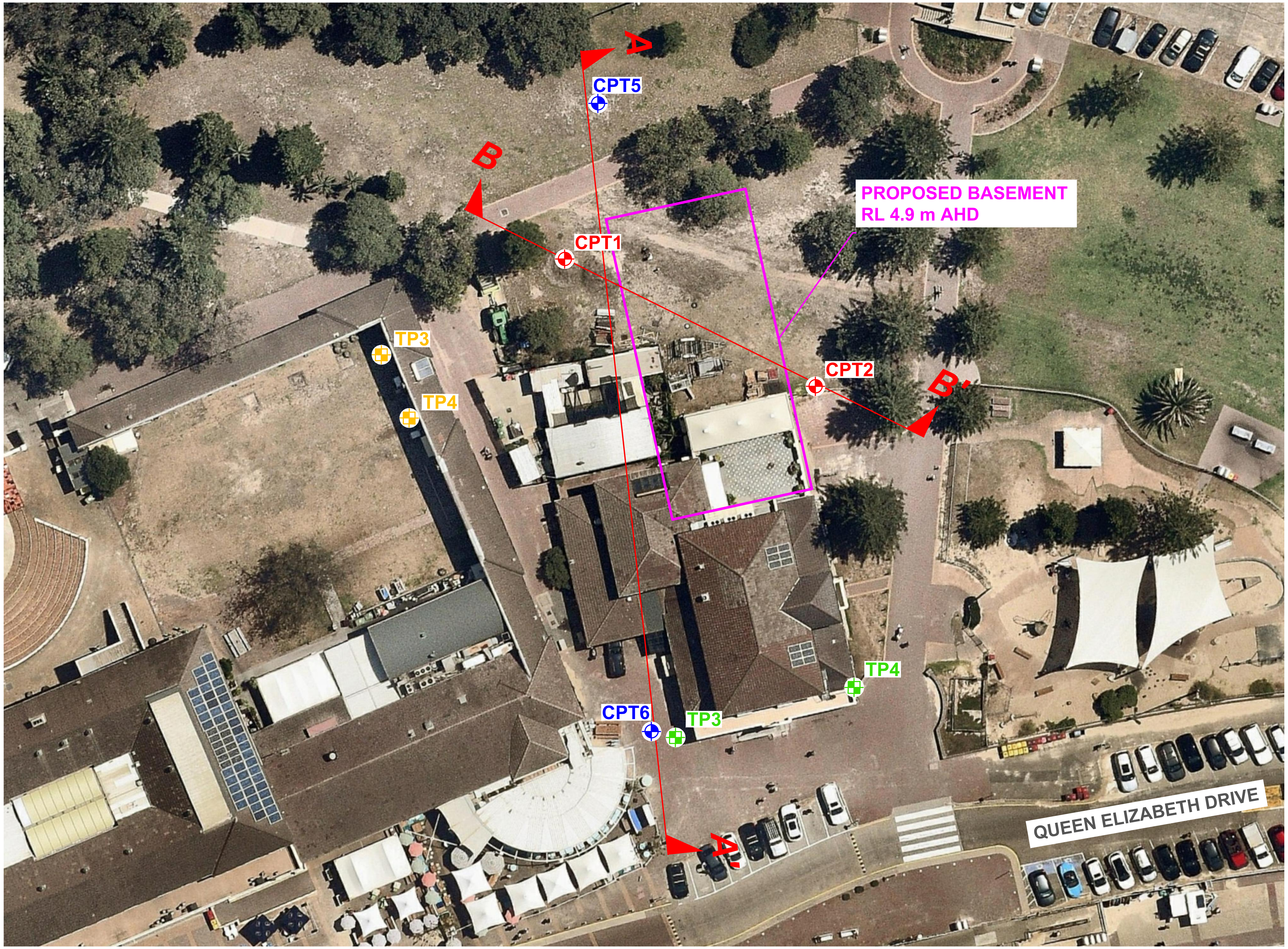
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

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

Appendix B

Drawings



Locality Plan

NOTE:
1: Base image from Nearmap.com (Dated 21.1.2020)
2: Test locations surveyed using differential GPS

0 5 10 15 20 30 40
1:500 @ A3

LEGEND

- Cone Penetration Test
- Test Pit/Footing Exposure
- Previous Cone Penetration Test (2015)
- Previous Test Pit/Footing Exposure (2015)
- A A' Geotechnical Cross-Section A-A'

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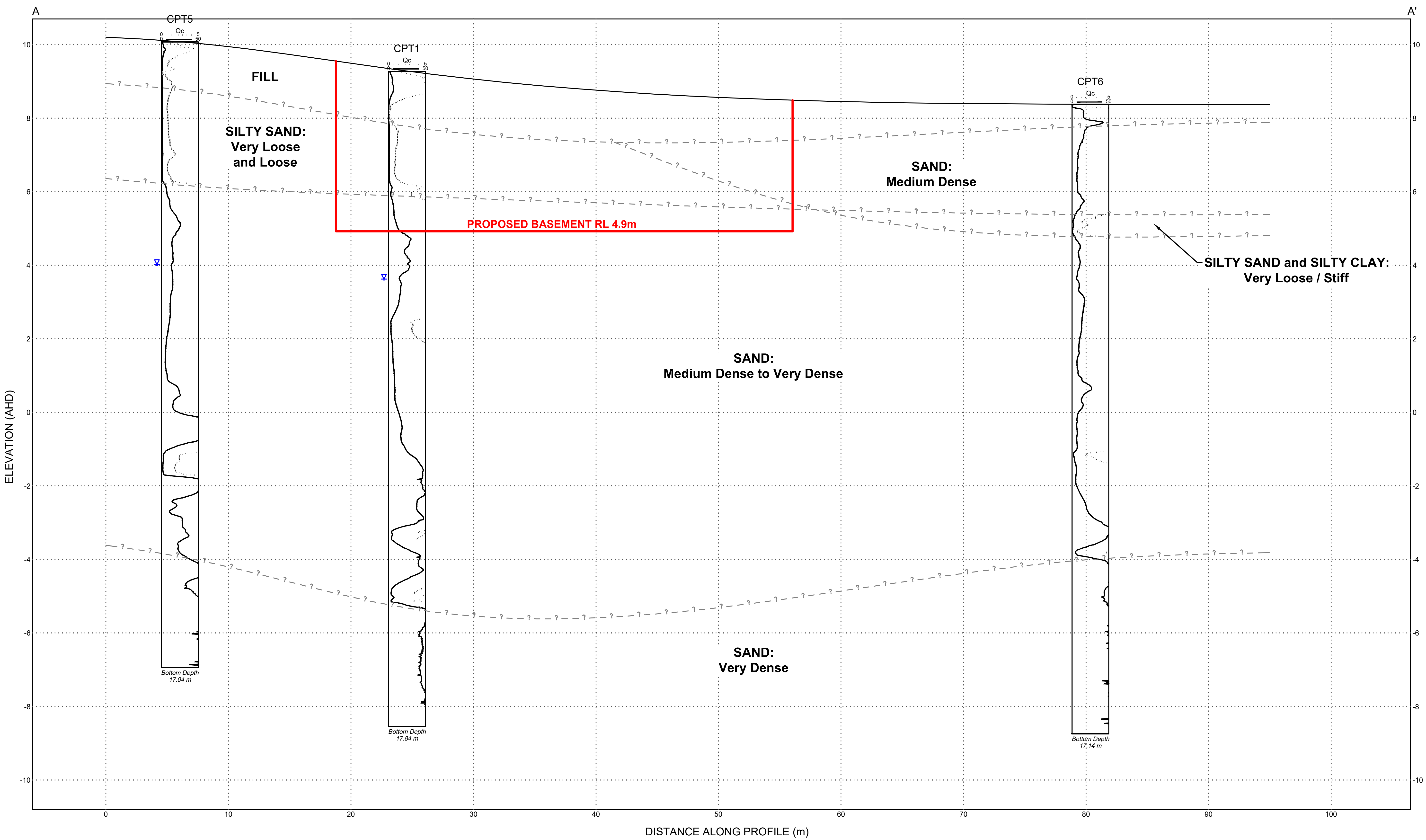
CLIENT: Lockhart-Krause Architects Pty Ltd		TITLE: **Test Location Plan**	PROJECT No: 99567.00
OFFICE: Sydney	DRAWN BY: JDB	**Bondi Surf Bathers' Life Saving Club Upgrade**	DRAWING No: 1
SCALE: 1:500 @ A3	DATE: 20.10.2020	**Queen Elizabeth Drive, Bondi Beach**	REVISION: 1



NOTES:
1. Subsurface conditions are accurate at the borehole locations only.
   Variations in subsurface conditions may occur between borehole locations.
   Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs included only and should be read in conjunction with detailed logs.

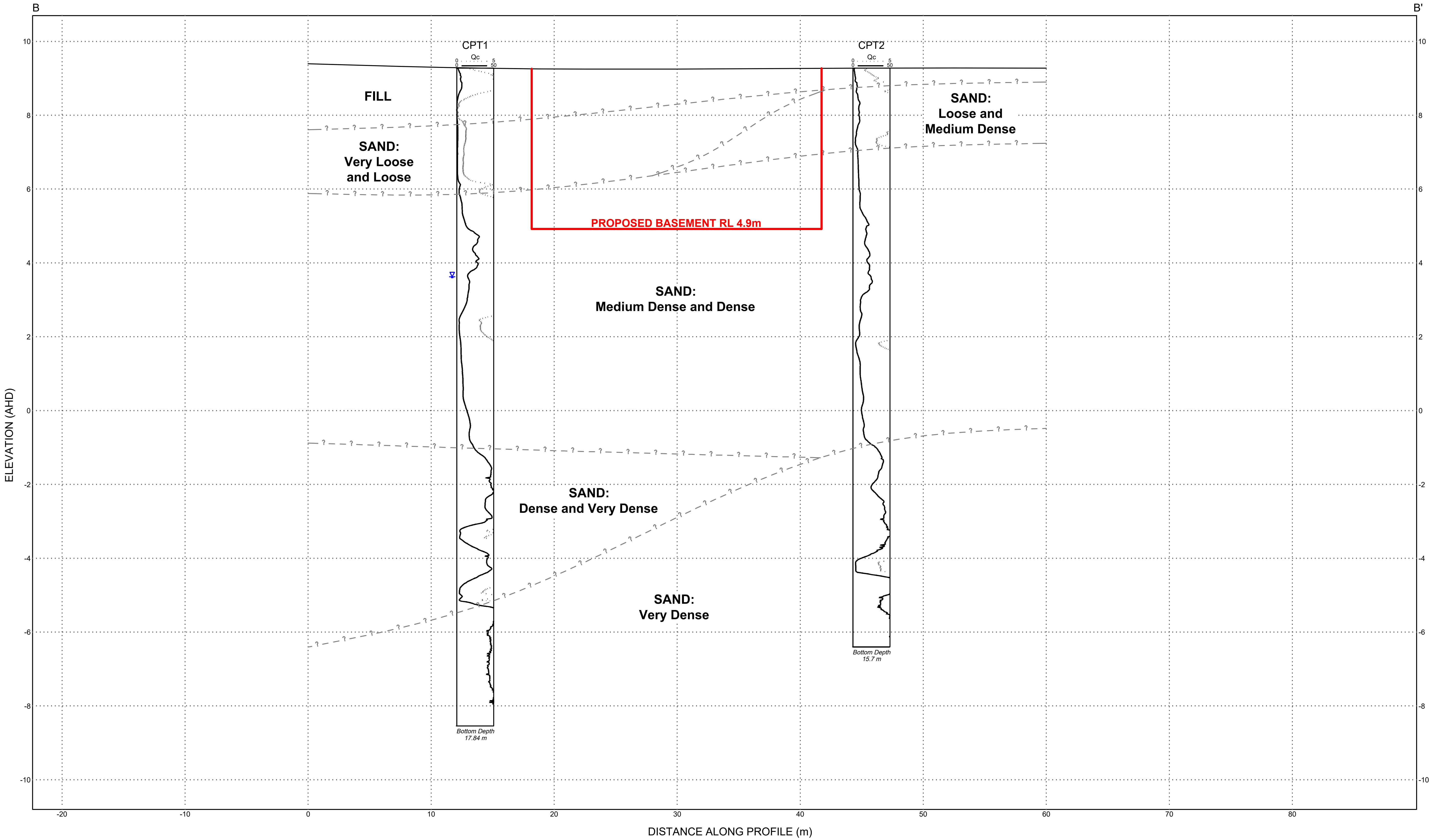
TESTS / OTHER
- Water level
— ? — — - Interpreted geotechnical boundary

Horizontal Scale (metres) 0 – 6

Vertical Exaggeration = 3.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Lockhart-Krause Architects Pty Ltd		TITLE: **Interpreted Geotechnical Cross-Section A-A'**	PROJECT No: 99567.00
OFFICE: Sydney	DRAWN BY: IT	**Bondi Surf Bathers' Life Saving Club Upgrade**	DRAWING No: 2
SCALE: 1:300 (H) 1:100 (V) @ A3	DATE: 20.10.2020	**Queen Elizabeth Drive, Bondi Beach**	REVISION: 1



NOTES:
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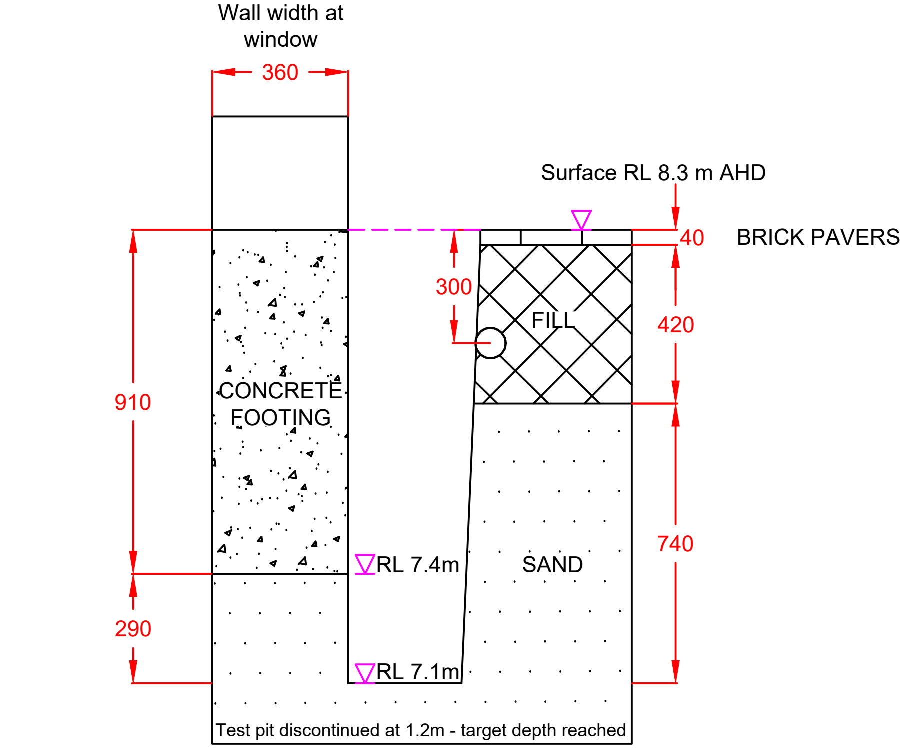
TESTS / OTHER
- Water level
— ? — — - Interpreted geotechnical boundary

0 6
Horizontal Scale (metres)

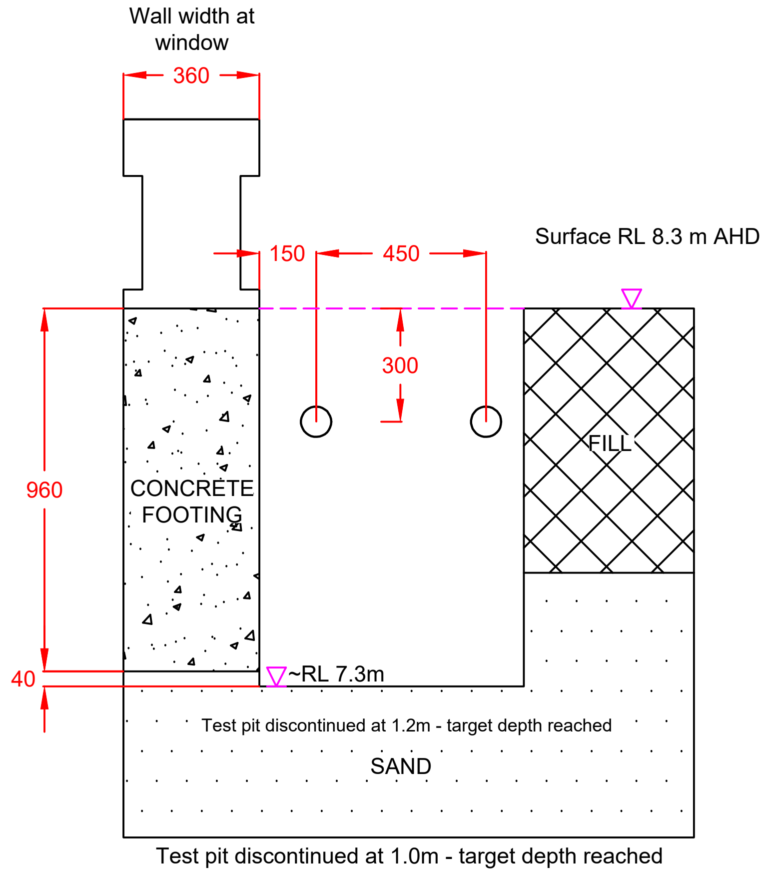
Vertical Exaggeration = 3.0

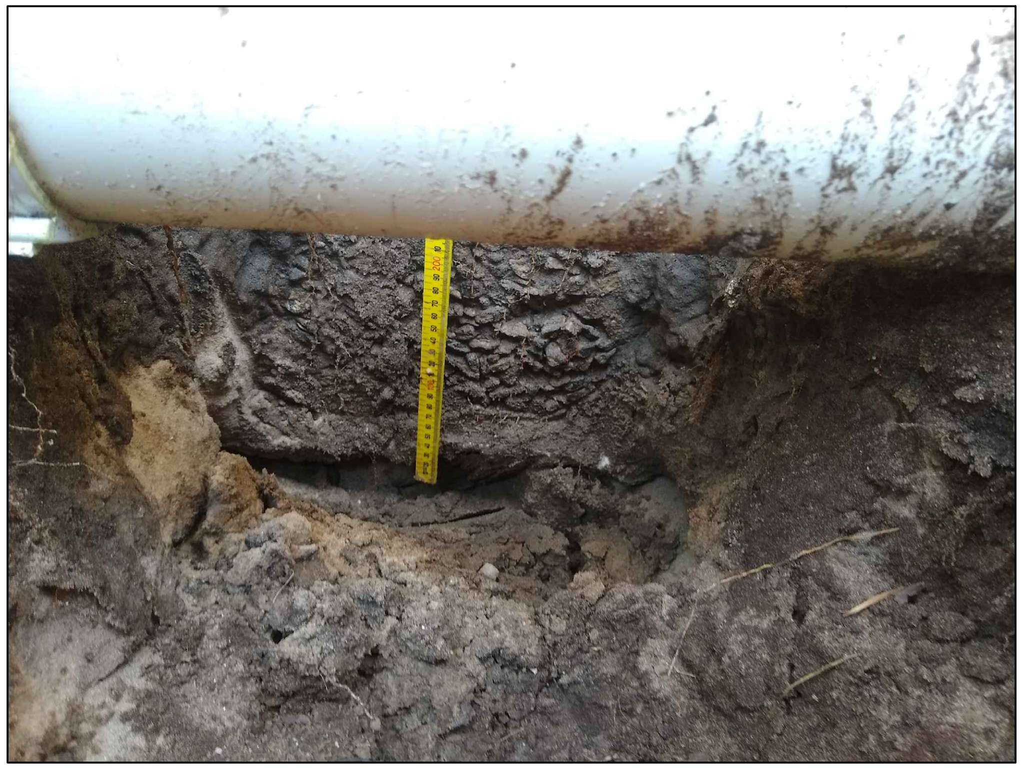
Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Lockhart-Krause Architects Pty Ltd	TITLE: **Interpreted Geotechnical Cross-Section B-B'**	PROJECT No: 99567.00
	OFFICE: Sydney — DRAWN BY: IT	**Bondi Surf Bathers' Life Saving Club Upgrade**	DRAWING No: 3
	SCALE: 1:300 (H) 1:100 (V) @ A3 — DATE: 20.10.2020	**Queen Elizabeth Drive, Bondi Beach**	REVISION: 1

TP3
(Refer to test pit log for more details on soil profile)



TP4
(Refer to test pit log for more details on soil profile)





Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Lockhart-Krause Architects Pty Ltd	TITLE: **Test Pit Drawings**	PROJECT No: 99567.00
	OFFICE: Sydney / DRAWN BY: IT	**Bondi Surf Bathers' Life Saving Club Upgrade**	DRAWING No: 4
	SCALE: 1:20 @ A3 / DATE: 14.2.2020	**Queen Elizabeth Drive, Bondi Beach**	REVISION: 0

Appendix C

Results of Field Work

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

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

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.
  Soil tends to stick together.
  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.
  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

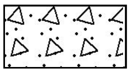
Symbols & Abbreviations

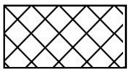
Graphic Symbols for Soil and Rock

General

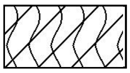
 Asphalt

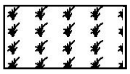
 Road base

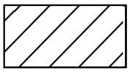
 Concrete

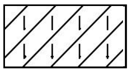
 Filling

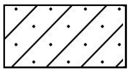
Soils

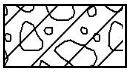
 Topsoil

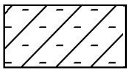
 Peat

 Clay

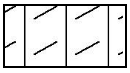
 Silty clay

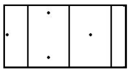
 Sandy clay

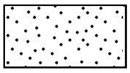
 Gravelly clay

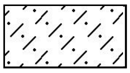
 Shaly clay

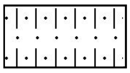
 Silt

 Clayey silt

 Sandy silt

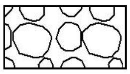
 Sand

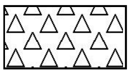
 Clayey sand

 Silty sand

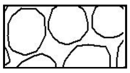
 Gravel

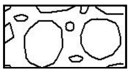
 Sandy gravel

 Cobbles, boulders

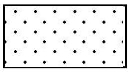
 Talus

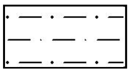
Sedimentary Rocks

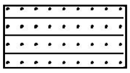
 Boulder conglomerate

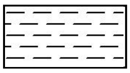
 Conglomerate

 Conglomeratic sandstone

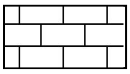
 Sandstone

 Siltstone

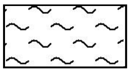
 Laminite

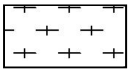
 Mudstone, claystone, shale

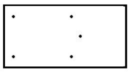
 Coal

 Limestone

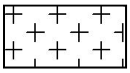
Metamorphic Rocks

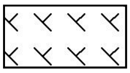
 Slate, phyllite, schist

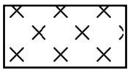
 Gneiss

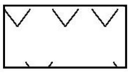
 Quartzite

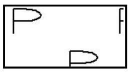
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

 Dacite, epidote

 Tuff, breccia

 Porphyry

Cone Penetration Tests

Douglas Partners

Introduction

The Cone Penetration Test (CPT) is a sophisticated soil profiling test carried out in-situ. A special cone shaped probe is used which is connected to a digital data acquisition system. The cone and adjoining sleeve section contain a series of strain gauges and other transducers which continuously monitor and record various soil parameters as the cone penetrates the soils.

The soil parameters measured depend on the type of cone being used, however they always include the following basic measurements

- Cone tip resistance q_c
- Sleeve friction f_s
- Inclination (from vertical) i
- Depth below ground z

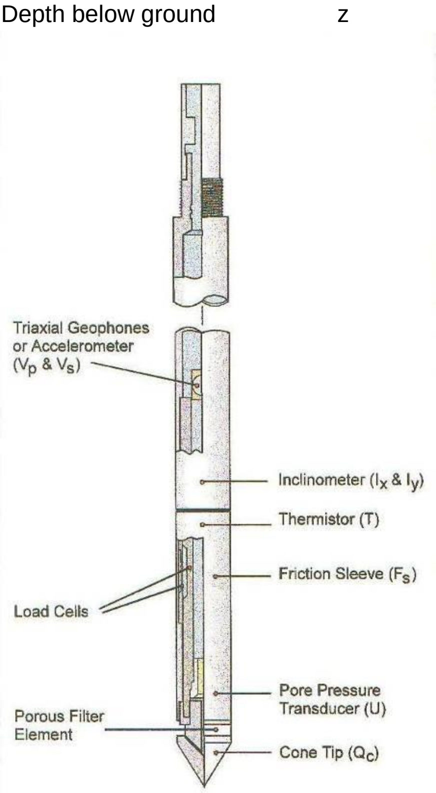


Figure 1: Cone Diagram

The inclinometer in the cone enables the verticality of the test to be confirmed and, if required, the vertical depth can be corrected.

The cone is thrust into the ground at a steady rate of about 20 mm/sec, usually using the hydraulic rams of a purpose built CPT rig, or a drilling rig. The testing is carried out in accordance with the Australian Standard AS1289 Test 6.5.1.

Figure 2: Purpose built CPT rig

The CPT can penetrate most soil types and is particularly suited to alluvial soils, being able to detect fine layering and strength variations. With sufficient thrust the cone can often penetrate a short distance into weathered rock. The cone will usually reach refusal in coarse filling, medium to coarse gravel and on very low strength or better rock. Tests have been successfully completed to more than 60 m.

Types of CPTs

Douglas Partners (and its subsidiary GroundTest) owns and operates the following types of CPT cones:

Type	Measures
Standard	Basic parameters (q_c, f_s, i & z)
Piezocone	Dynamic pore pressure (u) plus basic parameters. Dissipation tests estimate consolidation parameters
Conductivity	Bulk soil electrical conductivity (σ) plus basic parameters
Seismic	Shear wave velocity (V_s), compression wave velocity (V_p), plus basic parameters

Strata Interpretation

The CPT parameters can be used to infer the Soil Behaviour Type (SBT), based on normalised values of cone resistance (Qt) and friction ratio (Fr). These are used in conjunction with soil classification charts, such as the one below (after Robertson 1990)

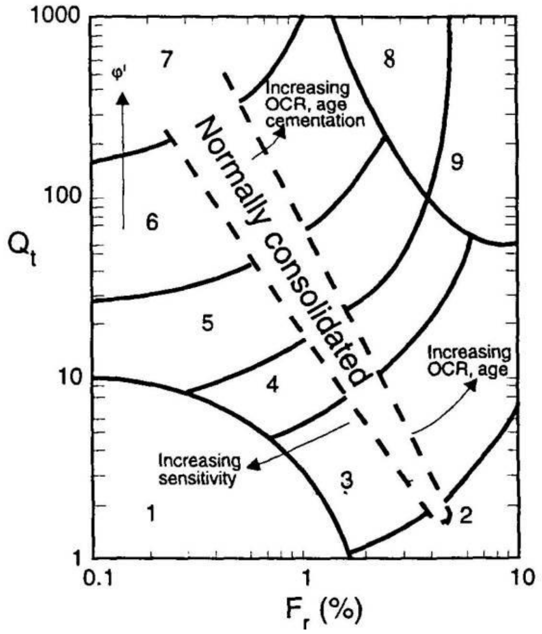


Figure 3: Soil Classification Chart

DP's in-house CPT software provides computer aided interpretation of soil strata, generating soil descriptions and strengths for each layer. The software can also produce plots of estimated soil parameters, including modulus, friction angle, relative density, shear strength and over consolidation ratio.

DP's CPT software helps our engineers quickly evaluate the critical soil layers and then focus on developing practical solutions for the client's project.

Engineering Applications

There are many uses for CPT data. The main applications are briefly introduced below:

Settlement

CPT provides a continuous profile of soil type and strength, providing an excellent basis for settlement analysis. Soil compressibility can be estimated from cone derived moduli, or known consolidation parameters for the critical layers (eg. from laboratory testing). Further, if pore pressure dissipation tests are undertaken using a piezocone, in-situ consolidation coefficients can be estimated to aid analysis.

Pile Capacity

The cone is, in effect, a small scale pile and, therefore, ideal for direct estimation of pile capacity. DP's in-house program ConePile can analyse most pile types and produces pile capacity versus depth plots. The analysis methods are based on proven static theory and empirical studies, taking account of scale effects, pile materials and method of installation. The results are expressed in limit state format, consistent with the Piling Code AS2159.

Dynamic or Earthquake Analysis

CPT and, in particular, Seismic CPT are suitable for dynamic foundation studies and earthquake response analyses, by profiling the low strain shear modulus G_0. Techniques have also been developed relating CPT results to the risk of soil liquefaction.

Other Applications

Other applications of CPT include ground improvement monitoring (testing before and after works), salinity and contaminant plume mapping (conductivity cone), preloading studies and verification of strength gain.

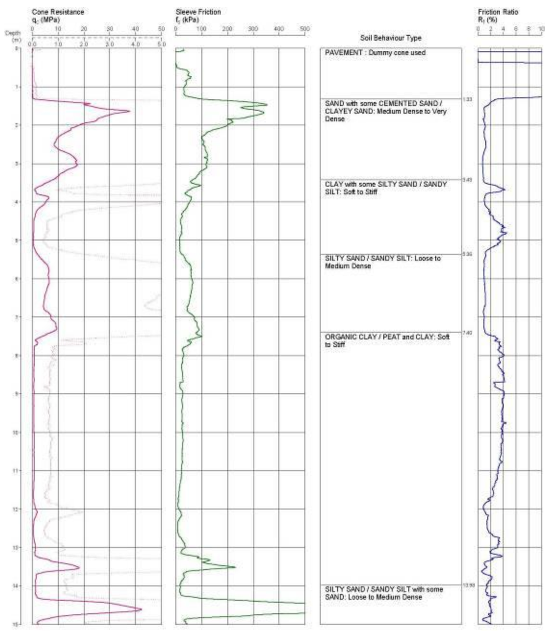


Figure 4: Sample Cone Plot

TEST PIT LOG

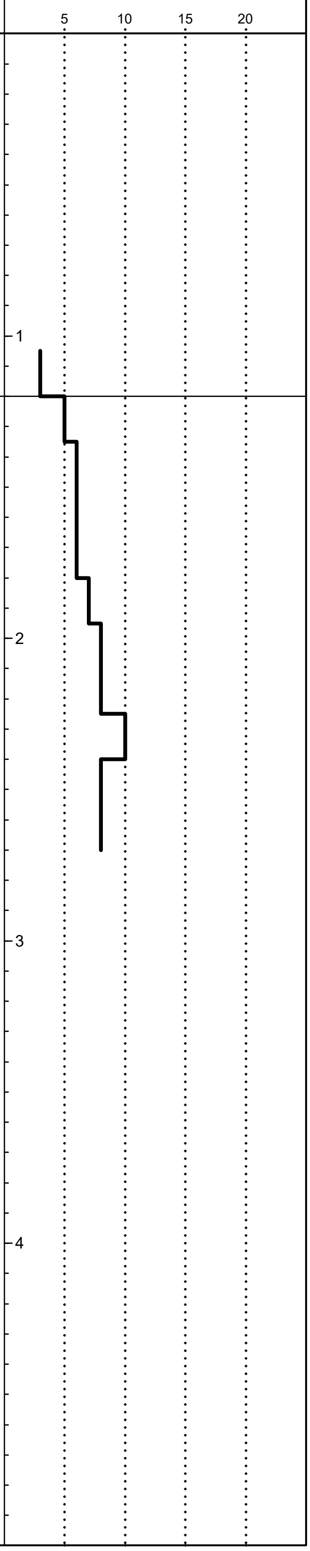
CLIENT: Lockhart-Krause Architects Pty Ltd
PROJECT: Bondi Surf Bathers' Life Saving Club Upgrade
LOCATION: Queen Elizabeth Drive, Bondi Beach

SURFACE LEVEL: 8.3 AHD
EASTING: 340750
NORTHING: 6248691

PIT No: TP3
PROJECT No: 99567.00
DATE: 30/1/2020
SHEET 1 OF 1

Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
0.04	BRICK Paving						
0.06 – 0.26	FILL/SAND: fine to medium, pale yellow, moist, appears generally moderately compacted						
0.28 – 0.46	FILL/Sandy GRAVEL: fine to medium gravel, fine sand, dark grey, moist, appears generally well compacted						
0.46	FILL/SAND: fine to medium, pale orange, moist, appears generally moderately compacted						
	FILL/SAND: fine to medium, pale grey, with brick and ceramic pipe fragments, moist, appears generally moderately compacted						
– 1.2	SAND SP: fine to medium, pale grey, moist, appears generally medium dense, aeolian Below 1.05m: medium dense						
1.2	Pit discontinued at 1.2m - target depth reached						

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand Tools to 1.2m
LOGGED: NB
SURVEY DATUM: MGA94

WATER OBSERVATIONS: No free groundwater observed during excavation

REMARKS: Base of concrete building footing at 0.91m depth

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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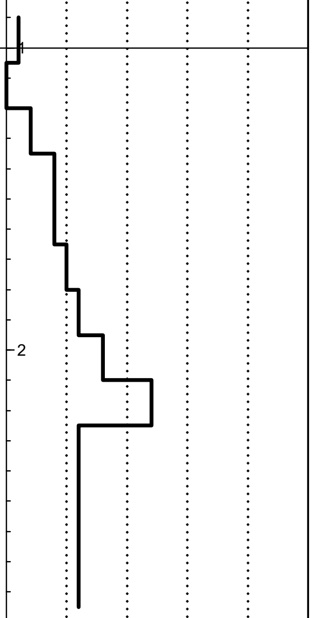
TEST PIT LOG

CLIENT: Lockhart-Krause Architects Pty Ltd
PROJECT: Bondi Surf Bathers' Life Saving Club Upgrade
LOCATION: Queen Elizabeth Drive, Bondi Beach

SURFACE LEVEL: 8.3 AHD
EASTING: 340771
NORTHING: 6248697

PIT No: TP4
PROJECT No: 99567.00
DATE: 30/1/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 150mm)
8		FILL/SAND: fine to medium, dark grey-brown, 10% silt fines with brick and concrete fragments, appears generally poorly compacted							
	0.7	SAND SP: fine to medium, pale grey, moist, appears generally loose, aeolian Below 0.9m: very loose and loose							
7	1.0	Pit discontinued at 1.0m - target depth reached Below 1.35m: medium dense							
6	2								
5	3								
4	4								



RIG: Hand Tools to 1.0m **LOGGED:** NB **SURVEY DATUM:** MGA94

WATER OBSERVATIONS: No free groundwater observed during excavation

REMARKS: Base of concrete building footing at 0.96m depth

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)		
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)		
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)		
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)		
D	Disturbed sample	▷	Water seep	S	Standard penetration test		
E	Environmental sample	▼	Water level	V	Shear vane (kPa)		

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Results of Dynamic Penetrometer Tests

Client	Bondi Surf Bathers' Life Saving Club Upgrade	**Project No.**	99567.00
Project	Lockhart-Krause Architects Pty Ltd	**Date**	30/01/2020
Location	Queen Elizabeth Drive, Bondi Beach	**Page No.**	1 of 1

Test Locations	TP3	TP4								
RL of Test (AHD)	8.3	8.3								
Depth (m)	**Penetration Resistance** Blows/150 mm									
0.00 – 0.15	E	E								
0.15 – 0.30	E	E								
0.30 – 0.45	E	E								
0.45 – 0.60	E	E								
0.60 – 0.75	E	E								
0.75 – 0.90	E	E								
0.90 – 1.05	E	1								
1.05 – 1.20	3	0								
1.20 – 1.35	5	2								
1.35 – 1.50	6	4								
1.50 – 1.65	6	4								
1.65 – 1.80	6	5								
1.80 – 1.95	7	6								
1.95 – 2.10	8	8								
2.10 – 2.25	8	12								
2.25 – 2.40	10	6								
2.40 – 2.55	8	6								
2.55 – 2.70	8	6								
2.70 – 2.85	D	6								
2.85 – 3.00		D								
3.00 – 3.15										
3.15 – 3.30										

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

Tested By NB
Checked By PAV

Remarks E = Excavated
D = Discontinued

CONE PENETRATION TEST

CLIENT: LOCKHART-KRAUSE ARCHITECTS PTY LTD

PROJECT: BONDI SURF BATHERS' LIFE SAVING CLUB UPGRADE

LOCATION: QUEEN ELIZABETH DRIVE, BONDI BEACH

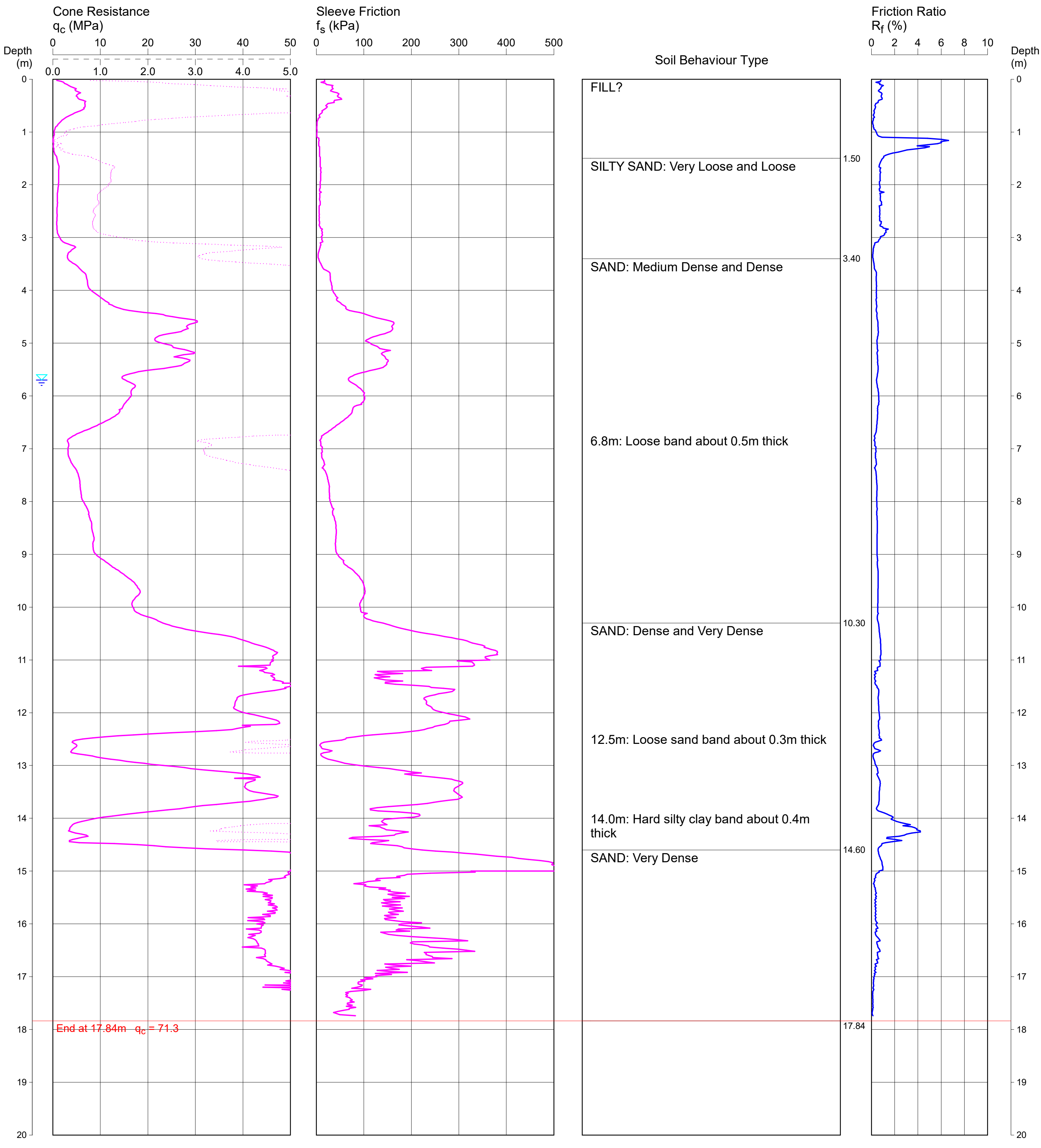
REDUCED LEVEL: 9.3

COORDINATES: 340737E 6248747N

CPT1

DATE 30/01/2020

PROJECT No: 99567



REMARKS: TEST DISCONTINUED DUE TO CONE TIP REFUSAL
GROUNDWATER MEASURED AT 5.7m AFTER REMOVAL OF RODS

Water depth after test: 5.7m

File: P:\99567.00 - BONDI BEACH, Queen Elizabeth Drive, Geo\4.0 Field Work\4.2 Testing\CPT1.CP5
Cone ID: 181002 **Type:** I-CFXY-10

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: LOCKHART-KRAUSE ARCHITECTS PTY LTD

PROJECT: BONDI SURF BATHERS' LIFE SAVING CLUB UPGRADE

LOCATION: QUEEN ELIZABETH DRIVE, BONDI BEACH

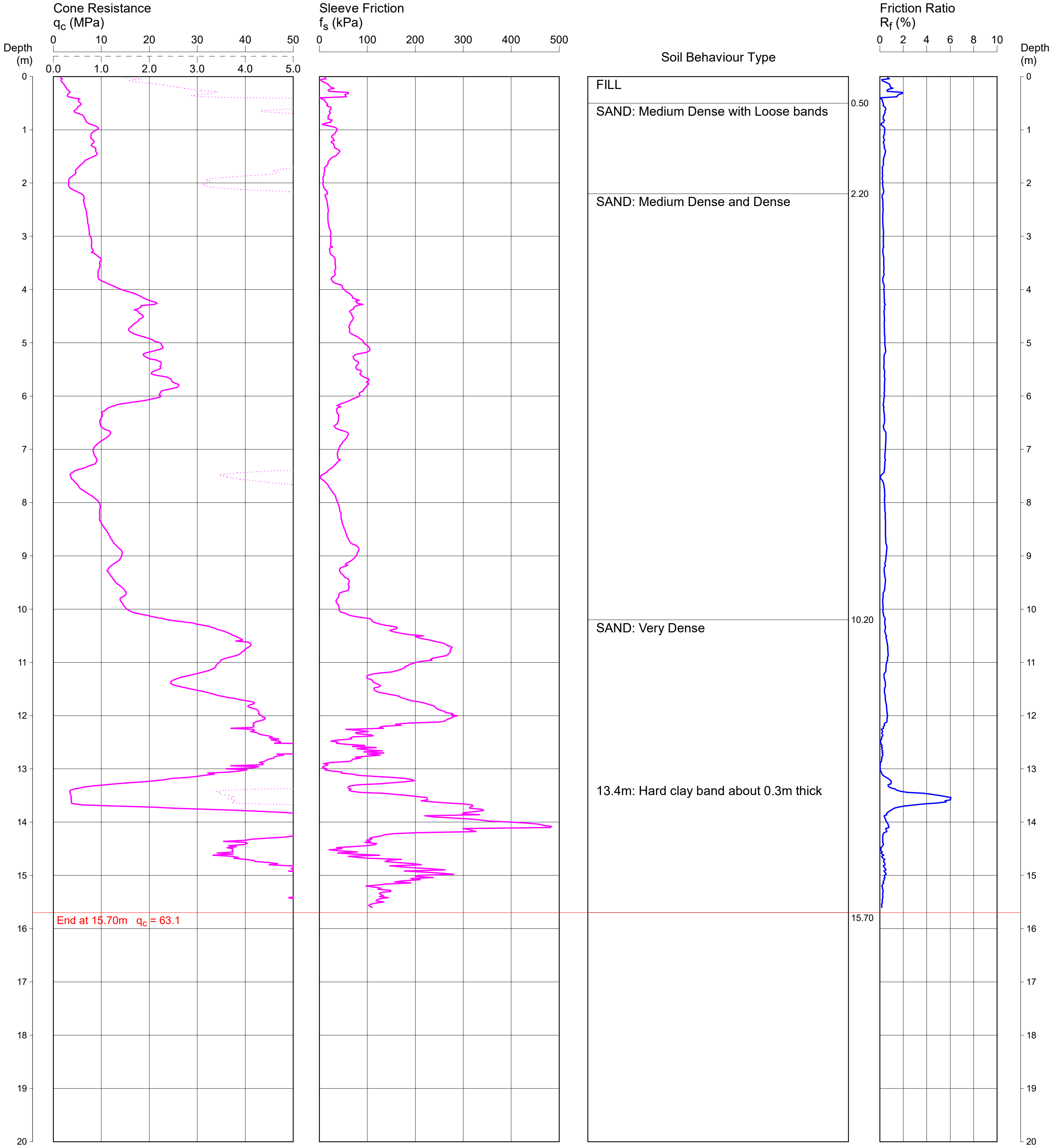
REDUCED LEVEL: 9.3

COORDINATES: 340767E 6248732N

CPT2

DATE 30/01/2020

PROJECT No: 99567



REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE ROD-BOWING NEAR REFUSAL
HOLE COLLAPSE MEASURED AT 5.2m AFTER REMOVAL OF RODS

File: P:\99567.00 - BONDI BEACH, Queen Elizabeth Drive, Geo\4.0 Field Work\4.2 Testing\CPT2.CP5
Cone ID: 181002 **Type:** I-CFXY-10

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: WAVERLEY COUNCIL

PROJECT: BONDI PAVILION UPGRADE

LOCATION: BONDI BEACH

REDUCED LEVEL: 10.2

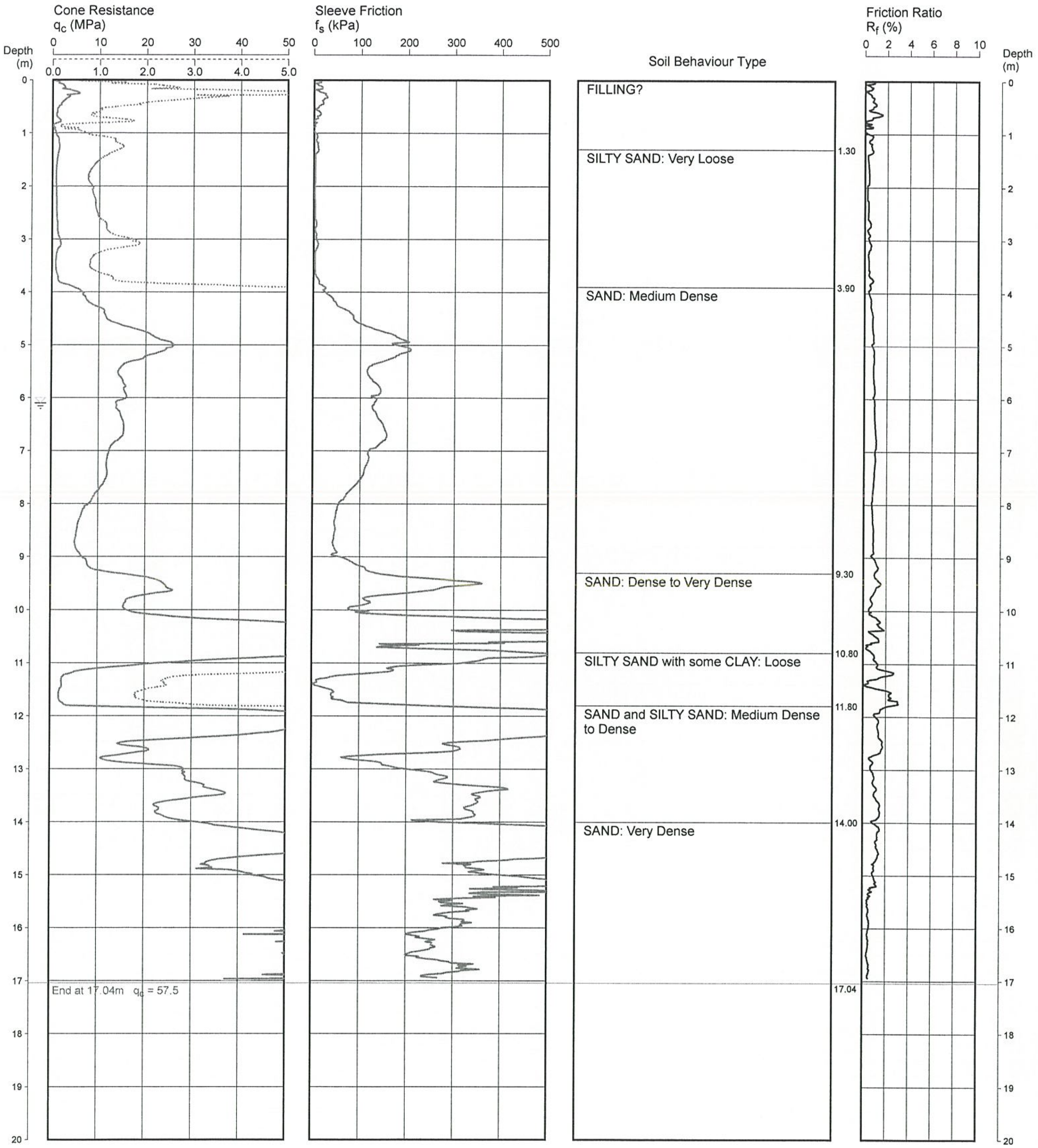
COORDINATES:

CPT5

Page 1 of 1

DATE 12/8/2015

PROJECT No: 84992.00



REMARKS: HOLE DISCONTINUED DUE TO EXCESSIVE ROD BOWING;
GROUNDWATER OBSERVED AT 6.1 m DEPTH AFTER WITHDRAWAL OF RODS.

Water depth after test: 6.10m depth (assumed)

File: P:\84992.00 - BONDI BEACH, Bondi Pavilion\4.0 Field Work\CPT Results\CPT5.CP5

Cone ID: 120631 **Type:** I-CFXY-10

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: WAVERLEY COUNCIL

PROJECT: BONDI PAVILION UPGRADE

LOCATION: BONDI BEACH

REDUCED LEVEL: 8.4

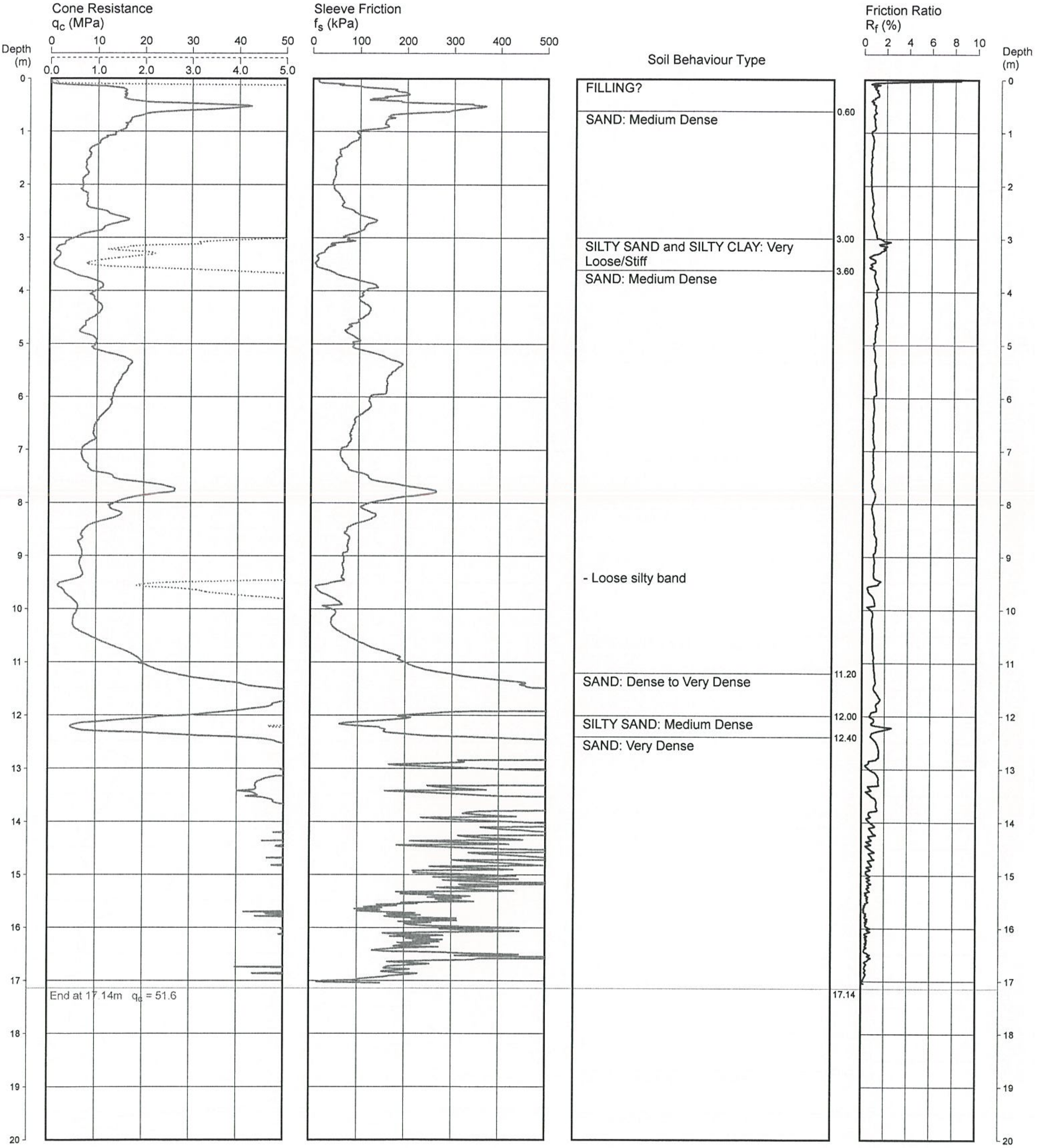
COORDINATES:

CPT6

Page 1 of 1

DATE 12/8/2015

PROJECT No: 84992.00



REMARKS: HOLE DISCONTINUED DUE TO CONE TIP REFUSAL;
HOLE COLLAPSE AT 5.2 m DEPTH AFTER WITHDRAWAL OF RODS.

File: P:\84992.00 - BONDI BEACH, Bondi Pavilion\4.0 Field Work\CPT Results\CPT6.CP5
Cone ID: 120631 **Type:** I-CFXY-10

ConePlot Version 5.9.2

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TEST PIT LOG

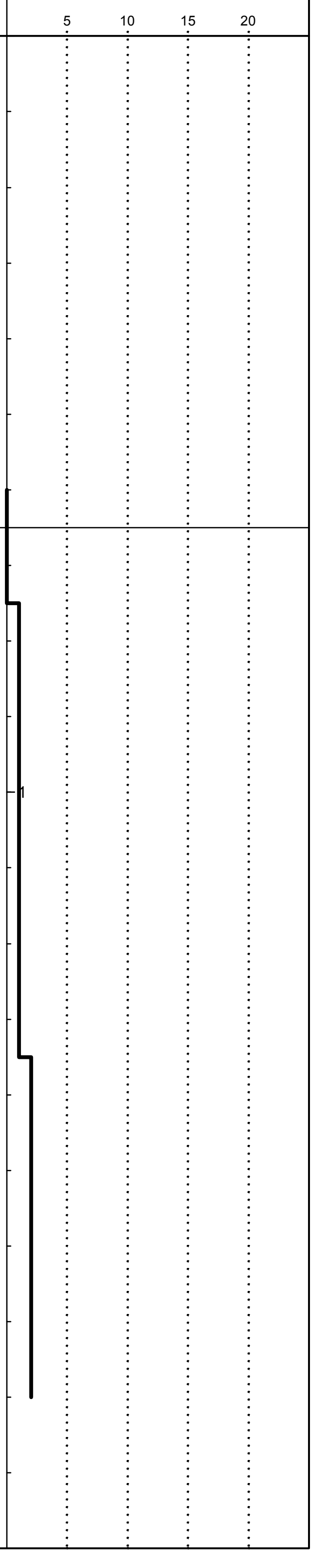
CLIENT: Waverley Council
PROJECT: Bondi Pavilion Upgrade
LOCATION: Bondi Beach

SURFACE LEVEL: 8.7 AHD
EASTING:
NORTHING:

PIT No: TP3
PROJECT No: 84992
DATE: 12/8/2015
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILLING - brown and grey, medium grained sand filling, damp						
	0.2	FILLING - light brown, medium grained sand filling with some bricks, plates, glass and other building materials, damp						
8	0.65	Pit discontinued at 0.65m - target depth reached						
	1							
7								

Dynamic Penetrometer Test (blows per 150mm)



RIG: 3.5t Excavator **LOGGED:** MB **SURVEY DATUM:** MGA94

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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TEST PIT LOG

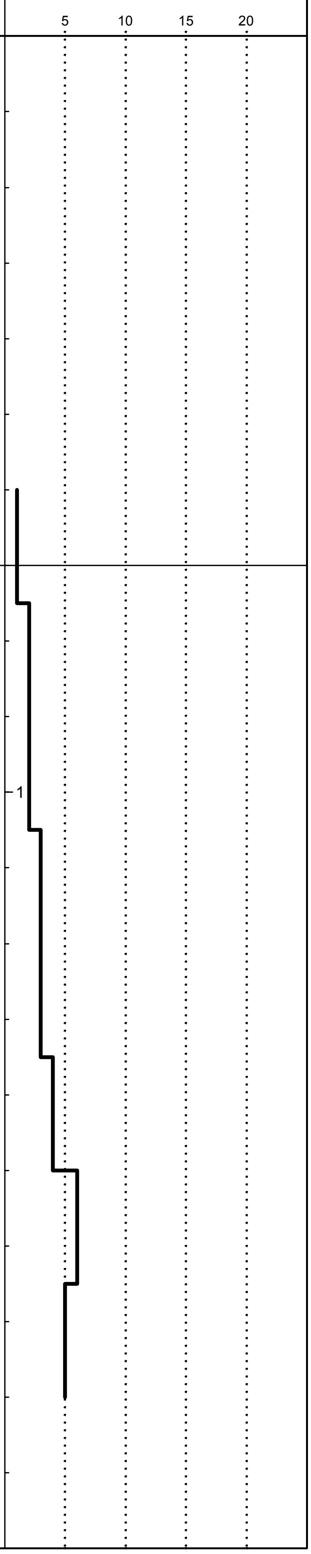
CLIENT: Waverley Council
PROJECT: Bondi Pavilion Upgrade
LOCATION: Bondi Beach

SURFACE LEVEL: 8.7 AHD
EASTING:
NORTHING:

PIT No: TP4
PROJECT No: 84992
DATE: 12/8/2015
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILLING - dark brown, medium grained sand topsoil filling, moist						
	0.1	FILLING - dark brown-grey, medium grained sand filling, damp 0.4m: becoming light brown with some bricks, glass bottles and terracotta fragments						
8	0.7	Pit discontinued at 0.7m - target depth reached						
	1							
7								

Dynamic Penetrometer Test (blows per 150mm)



RIG: 3.5t Excavator **LOGGED:** MB **SURVEY DATUM:** MGA94

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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