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Report on
Geotechnical and Groundwater Investigation

Proposed Mixed Use Development
19-27 Cross Street, Double Bay

Prepared for
Tri-Anta Pty Ltd

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Integrated Practical Solutions

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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.

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Report on Geotechnical and Groundwater Investigation
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1. Introduction

This report presents the results of a geotechnical and groundwater investigation undertaken for a proposed mixed use development at 19-27 Cross Street, Double Bay. The investigation was commissioned in an email dated 11 May 2018 by Savvas Hadjimichael of SDH & Associates on behalf of Tri-Anta Pty Ltd, developer for the project, and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal SYD180186 dated 10 May 2018.

It is understood that the proposed development comprises a new six (6) storey building over two basement levels.

The field investigation comprised the cone penetration testing, borehole drillings, installation and monitoring of groundwater wells. Selected samples from the boreholes were tested in a laboratory to determine chemical properties. Details of the field and laboratory work are provided in the report together with relevant comments on design and construction practice.

The geotechnical investigation was carried out concurrently with a preliminary contamination assessment and waste classification which have been reported separately.

Architectural drawings prepared by Luigi Rosselli for the development application were provided for the investigation.

2. Site Description and Regional Geology

The site, known as Lot 100 in DP 617017, is an approximate parallelogram shape maximum with dimensions of 42 m by 39 m and an area of 1334 m^2. It is bounded by Cross Street to the south, Transvaal Avenue to the east, a hotel to the west and commercial buildings to the north. The site is near flat and is currently occupied by a two storey brick shopping arcade and office buildings constructed on grade.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by filling over alluvial and estuarine sediments of the Quaternary Period. These sediments comprise silty to peaty quartz sand, silt, and clay. The Triassic aged Hawkesbury Sandstone underlies the Quaternary deposits. Hawkesbury Sandstone generally comprises medium to coarse grained quartz sandstone with minor shale and laminite lenses. An extract from the Geological Sheet is shown in Figure 1 (following page).

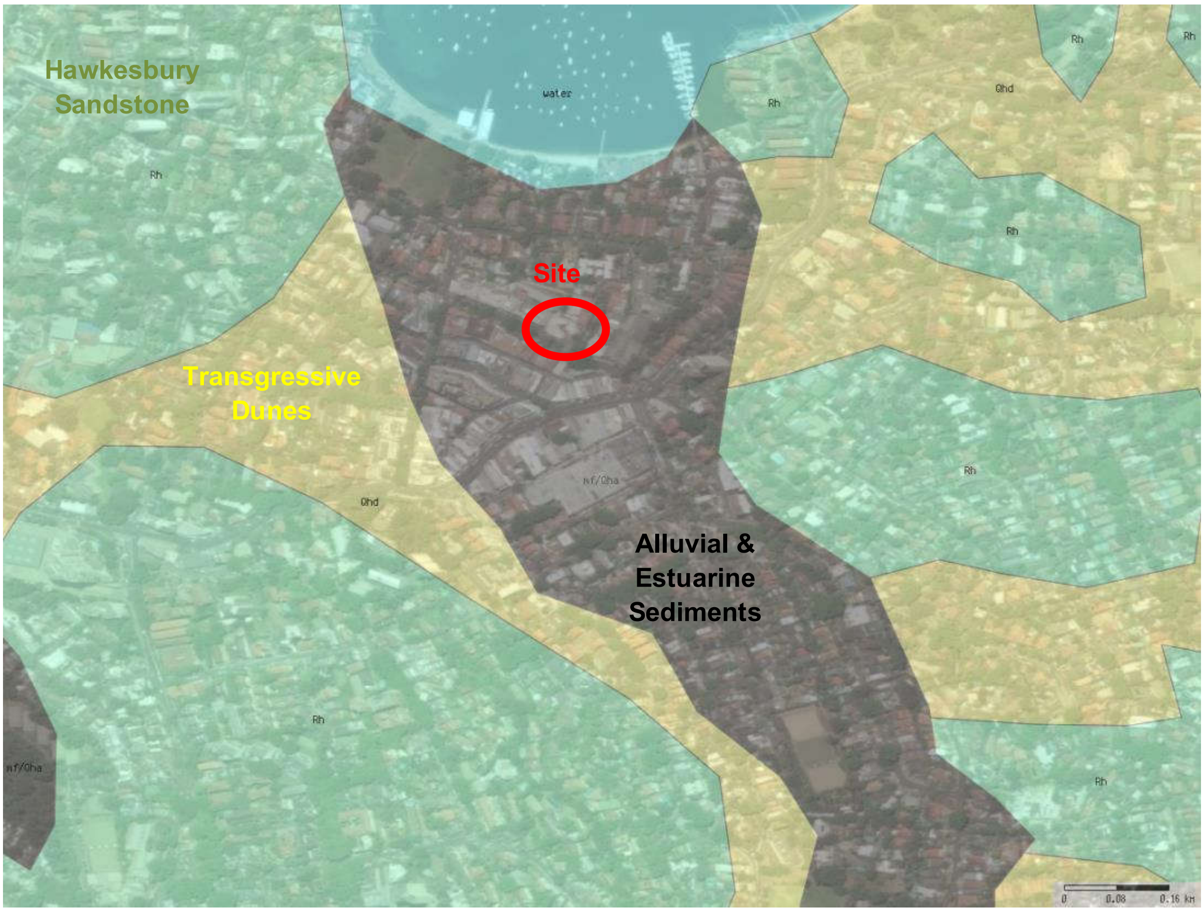


Figure 1: Extract from Sydney 1:100 000 Geological Series Sheet

3. Background

DP has previously investigated the adjacent site, 41 Cross Street, for the construction of the hotel which includes a two storey basement (Project 11525, dated 1989). The investigation included cone penetration tests (CPT)m boreholes and Marchetti dilotometer testing for the design of the basement diaphragm wall. Relevant borehole and CPT data from tests near the common boundary have been included in the analysis for the current development on 19-29 Cross Street.

4. Field Work

4.1 Methods

The field work for the geotechnical investigation included three CPTs (CPT1, CPT2 and CPT3) and boreholes at the CPT locations (BH1, BH2 and BH3). The current test locations are shown on Drawing 1, in Appendix A together with the locations of previous tests on the adjacent site near the western boundary.

Following a Dial-Before-You-Dig inquiry and search by an accredited locator, the test locations were positioned to be clear of underground services and accessible to the truck mounted CPT and drilling rigs and.

Initially, the paving and concrete surface at the test locations was dia-cored until the underlying soil subgrade was exposed. To confirm the locations were away from buried services, boreholes were drilled using 100 mm diameter augers to depths of up to 2.6 m. Samples were also taken for strata identification and laboratory testing. The boreholes were then backfilled with sand and CPT testing carried out to refusal at depths in the range 18.42 m to 20.96 m.

In the CPT, a 35 mm diameter cone with a following 130 mm long friction sleeve is attached to rods of the same diameter, and pushed continuously into the soil by hydraulic thrust from a ballasted truck mounted test rig. Strain gauges in the cone and sleeve measure resistance to penetration, and the results are displayed on a monitor and stored for interpretation, analysis and plotting. On withdrawal of the rods and cone, the remnant holes were dipped to measure, if possible, groundwater levels. Some additional notes describing the method of operation and interpretation of results precede the detailed test results given with the notes in Appendix A.

Following the CPT, boreholes were drilled at the same locations using a specialised truck mounted geotechnical drilling rig. The boreholes were drilled through the upper soil layers using solid flight augers down to groundwater level then extended using rotary wash boring techniques to depths in the range 8.95 m to 10.45 m. Standard penetration tests (SPT) were undertaken within the soil at regular intervals.

Monitoring wells were installed in all the boreholes facilitate measurement of groundwater levels in the longer term, water sampling for laboratory testing and permeability testing.

4.2 Field Work Results

The subsurface conditions encountered in the tests are presented in the CPT report sheets and borehole logs in Appendix A together with notes defining descriptive terms and classification methods. Sections along each boundary summarising the subsurface conditions from the CPTs and boreholes are shown in Drawing 2 – 5 in Appendix A.

The subsurface conditions encountered in the borehole and cone penetration tests on the site, as well as tests undertaken by DP in the near vicinity have been used to profile a geological model. The materials encountered in these tests below existing paving and concrete slabs can be described as follows:

CONCRETE:	concrete paving and concrete slabs to depths of 0.16 – 0.77 m;
FILLING:	gravelly sand filling with building rubble to depths of 0.25 – 0.70 m, overlying;
SAND:	generally medium dense to depths of 9 – 10 m, then very loose to loose to 13 m depth, increasing to medium dense to very dense with some weak clay and silt layers at depth.
BEDROCK:	Bedrock was inferred in all three CPTs at depths in the range 18.4 – 21.0 m. High strength sandstone was encountered at a depth of 26.75 m in BH10 previously drilled on the adjacent site near the common boundary

4.3 Hydrogeology

The results of the testing and previous experiences on numerous nearby sites indicate there is a shallow unconfined aquifer within the sand beneath the site. Groundwater was observed from 2.1 m depth below current surface levels and is expected to flow towards Double Bay which is located about 300 m to the north of the site. The results of groundwater measurements are summarised in Table 1.

Table 1: Results of Groundwater Monitoring

Borehole	Surface Level (mAHD)	Date							
		During Drilling 17-18 June 2018		20 June 2018		5-6 July 2018		27 August 2018	
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)
BH1	3.3	2.2	1.1	2.2	1.1	2.3	1.0	2.7	0.6
BH2	3.3	2.1	1.2	2.3	1.0	2.4	0.9	2.8	0.5
BH3	3.3	2.3	1.0	2.2	1.1	2.3	1.0	2.6	0.7

Groundwater monitoring on nearby sites carried out for longer periods using data loggers, indicated fluctuations in groundwater levels between RL 0.3 - 1.6 m AHD, consistent with the results of periodic monitoring presented in Table 1 for the subject site. The groundwater levels appeared to change as a result of rainfall and to a much lesser extent changes in tide levels in the nearby Sydney Harbour.

The results of the rising head permeability tests within the wells are provided in Appendix B and summarised in Table 2.

Table 2: Results of Permeability Tests

Borehole	Screened Depth of Well	Observation	Estimated Hydraulic Conductivity (k)	
			(m/sec)	(m/day)
BH1	0.7 – 8.3 m in sand	Water rise from 3.5 m to 2.7m in 120 sec	5×10^{-4}	43
BH2	0.7 – 8.5 m in sand	Water rise from 4.5 m to 3.0 m in 150 sec	5×10^{-4}	43
BH3	0.7 – 9.8 m in sand	Water rise from 3.4 m to 2.7 m in 60 sec	10×10^{-4}	86

The estimated hydraulic conductivity values are relatively high, typical of clean sand. Other sites in Double Bay typically have k values in the range of 10 – 20 m/day although lower values have been observed when the sand is mixed with significant proportions of silt and clay.

Water inputs to the aquifer are:

- Rainfall infiltration over the entire surface area of the aquifer; and
- Possible unnatural recharge from seepage from nearby leaking stormwater pipes.

Water outputs or losses from the aquifer are:

- Evapotranspiration from the vegetation across the surface of the aquifer;
- Nearby dewatering for irrigation of local gardens and of drained basements; and
- Discharge into the Double Bay.

A search of the NSW Office of Water groundwater database revealed that 35 registered groundwater bores are located within 500 m of the site. A review of the work summaries for the five registered bores closest to the site (GW112100, GW112103, GW106048, GW106047 and GW107680) did not reveal any water quality data. The closest bores (GW112100 and GW112103) were located about 70 m from the site and were authorised as monitoring bores. Bores GW106047 and GW106048 were authorised for domestic purposes and were installed to a depth of 6 m in sand. Bore GW107680 was used for dewatering and installed to a depth of approximately 5 m in sand and silt.

5. Laboratory Testing

5.1 Acid Sulphate Soils

Screening tests on soil samples were carried out by Envirolab Services Pty Ltd (Envirolab) to provide indications of actual acid sulphate soil (AASS) and potential acid sulphate soil (PASS). The natural field pH of each soil sample was measured after the addition of distilled water (pH_F), then the pH (pH_{FOX}) was measured following the addition of hydrogen peroxide and oxidisation for at least one hour. The results for the screening tests are summarised in Table 3.

Table 3: Summary of ASS and PASS Screening Test Results

Borehole	Depth (m)	Material Description	Natural pH_F	Oxidised pH_{FOX}	Change in pH	Reaction
BH1	0.3-0.4	Filling (gravelly sand)	7.8	7.6	0.2	Moderate
BH1	0.7-0.8	Sand	7.7	5	2.7	Slight
BH1	0.9-1.0	Sand	7.4	6	1.4	Slight
BH1	1.4-1.5	Sand	7.5	6.7	0.8	Slight
BH1	1.9-2.0	Sand	8.3	6.4	1.9	Slight
BH1	2.4-2.5	Sand	7.8	5.6	2.2	Slight
BH1	2.9-3.0	Sand	7.4	3	4.4	Slight
BH1	4.4-45	Sand	6.5	1.6	4.9	Moderate
BH2	1.0-1.1	Sand	9.8	10.1	-0.3	Slight

Borehole	Depth (m)	Material Description	Natural pH_F	Oxidised pH_{FOX}	Change in pH	Reaction
BH2	1.9-2.0	Sand	8.3	5.2	3.1	Slight
BH2	2.4-2.5	Sand	7.9	5	2.9	Slight
BH2	3.9-4.0	Sand	7.6	3	4.6	Moderate
BH2	5.4-5.5	Sand	7.2	1.9	5.3	Moderate
BH3	0.2-0.25	Filling (Sand)	9.1	8.8	0.3	Moderate
BH3	0.5-0.6	Sand	7	2.9	4.1	Slight
BH3	0.9-1.0	Sand	6.1	3.6	2.5	Slight
BH3	1.4-1.5	Sand	6.9	5.4	1.5	Slight
BH3	2.0-2.1	Sand	7.1	3.6	3.5	Slight
BH3	2.5-2.6	Sand	7.4	3.7	3.7	Slight
BH3	3.0-3.1	Sand	7.3	3.8	3.5	Slight
BH3	4.0-4.1	Sand	7.2	3.8	3.4	Slight
BH3	5.5-5.6	Sand	7.1	3.4	3.7	Moderate
BH3	10.0-10.45	Sand	6.9	1.7	5.2	High

Note: yellow highlight significant potential for exceedance of action criteria
red highlight are samples selected for chromium reducible sulfur testing

The screening test results were assessed for the possible presence of AASS or PASS on the basis of the following guidance indicators specified in the ASSMAC Guidelines:

- $pH_F \leq 4$ indicates oxidation has occurred in the past and that AASS are likely to be present;
- $4 < pH_F < 5.5$ indicates the soil is acidic. This may be as a result of limited oxidation of sulphides but may also be as a consequence of the presence of organic acids.
- $pH_{FOX} < 3$, plus a strong reaction with peroxide, plus a pH_{FOX} value of at least one pH unit below pH_F, strongly indicates a PASS. The higher the reaction, the lower the drop between pH_F and pH_{FOX}, and the lower the pH_{FOX} value, the higher the potential for PASS.
- $3 < pH_{FOX} < 4$ is less positive that the sample is PASS.
- $4 < pH_{FOX} < 5$ is neither positive nor negative, as some sulfides may be present in small quantities.
- $pH_{FOX} > 5$ and little or no drop from pH_F to pH_{FOX} indicate little net acid generating ability.

It should be noted that acid generation can be buffered by carbonate material in the samples (such as shell fragments). Also the pH change may be due to the oxidation of organic materials.

No samples provided positive indicators of AASS. Most of the samples provided positive indicators of PASS and four of these samples were tested for a Chromium Suite at Envirolab. The results of the analysis are summarised in Table 4 and compared with the action criteria specified in ASSMAC (1998) Guidelines. Full laboratory reports are attached in Appendix C.

Table 4: Results of Chromium Suite Testing

Borehole	Depth (m)	Description	pHKCl	Chromium Reducible Sulphur (%w/w)	Liming rate(1) (kg CaCO3/t)
BH1	4.4-45	Sand	5.1	0.02	1.3
BH2	5.4-5.5	Sand	5.7	0.01	<0.75
BH3	0.5-0.6	Sand	5.3	<0.005	<0.75
BH3	2-2.1	Sand	6.2	<0.005	<0.75
Action Criteria* (>1 tonne of sand soil disturbed)			<4	0.03	-

Notes: pHKCl = Non-oxidised pH
* Values above action criteria are indicators of PASS in accordance with ASSMAC
(1) Liming rate as reported by Envirolab

The results confirmed that none of the four samples tested were PASS soil above the action criteria.

5.2 Soil and Water Aggressivity

In addition to the natural field pH testing of soil samples using distilled water (pH_F) presented in Table 3, water samples from each borehole were tested in the field to determine the pH and electrical conductivity.

Table 5: Results of Water Aggressivity Testing

Borehole	pH	EC (µS/cm)
BH1	5.4	492
BH2	6.9	811
BH3	6.9	381

Notes: EC = electrical conductivity

The results, with reference to AS 2159 – 2009 Piling Design and Installation, suggest that the sand soil and groundwater is MODERATELY aggressive to concrete piles and MILDLY aggressive to steel piles.

6. Proposed Development

It is understood that the proposed development will include the demolition of the existing two storey commercial building and construction of a new six (6) storey mixed use building with a two level basement. The maximum depth of excavation required below existing ground surface level is expected to be about 6 m (~RL -4 m AHD) and will extend virtually to the boundaries on all sides.

7. Comments

7.1 Geotechnical Issues

Some of the primary geotechnical issues that need to be considered for development on the site are summarised below:

- Groundwater is relatively shallow at around 2 – 3 m depth and dewatering will be required for construction of basements;
- As low permeability material (bedrock) was encountered at depths of about 20 m below the site surface so construction of a deep cut off wall to limit groundwater into a drained basement will be very expensive. A tanked basement will therefore be required. The basement floor will need to be designed as a hydrostatic slab to resist the uplift pressure of the groundwater.
- Potential acid sulphate soils (PASS) are present beneath the site, but at levels below the ASSMAC Action Criteria;
- Perimeter walls will need to be designed to reduce inflow to control drawdown of water levels on adjacent sites as this has the potential to cause settlement and may also generate acidic conditions in the PASS;
- The site is underlain by layers of very loose to loose sand at depths of around 9 – 13 m. This may result in difficulties constructing the perimeter walls, as occurred on the adjacent hotel site, relatively poor founding and anchoring conditions, and significant settlement for raft footings.
- A diaphragm wall, ideally taken to bedrock, would be the best option for the basement retaining wall from a groundwater control view point, however given the rock depth of about 20 m the founding depth may be cost prohibitive. Diaphragm walls are very low permeability and have a good record for having only minor leakage when properly designed and constructed by a contractor prepared to adopt a strict quality control regime. Secant pile walls could be considered as a cheaper alternative, but they are more likely to be more permeable due to misalignment below depths of about 10 m. The risks associated with the various options will need to be considered in the conceptual design process.

7.2 Subsurface Conditions

7.2.1 Soil and Rock

As described in Section 4.2 of this report, the typical soil profile comprise shallow pavers, concrete and filling to depths of up to about 0.8 then natural sand. The sand is predominantly medium dense to depths of up 9 m (~RL -6 m AHD), with very loose to loose layers to about 13 m depth (RL -10 m AHD). The strength of the sand increases below this depth and is layered with silt and clay.

Inferred bedrock was at a depth of over 18 m (~RL -15 m AHD) near he north eastern corner and cored at nearly 27 m (~RL -15 m AHD) on the adjacent hotel site near the south western corner.

7.2.2 Groundwater

The Woollahra Municipal Council's Guidelines for Geotechnical and Hydrogeological Reports indicate that temporary changes in the water level during construction should not exceed 0.3 m, unless calculations based on site specific results can support a greater change, and that the development will not change the permanent water table by more than 0.2 m.

It is expected that a drawdown of less than 1.5 m would be within the range of historic low groundwater level fluctuations in Double Bay and therefore settlements due to drawdown of 1.5 m within the very loose to medium dense sands should be relatively minor (less than 20 mm). To further reduce the risk of adverse effects on surrounding properties, it is suggested that the proposed shoring and dewatering scheme should be designed to target a drawdown of no more than 1.0 m on surrounding properties and numerical modelling of the groundwater will be required as the design progresses.

Groundwater modelling for a similar, nearby site in Double Bay indicated that the drawdown would be approximately 1 m (without reinjection) near the excavation, reducing to approximately 0.5 m at a distance of about 60 m from the edge of the excavation. The groundwater modelling also indicated that with reinjection during the construction phase, the drawdown in groundwater was predicted to be less than 0.3 m. Re-injection together with a sufficiently deep, impermeable perimeter wall is suggested to control the drawdown and also to comply with the Woollahra Municipal Council's Guidelines.

A tanked basement will require no long term pumping and will therefore not drawdown the groundwater levels outside the site. Any permanent changes to the groundwater levels will be less than 0.2 m due to the following factors:

- The thickness of the saturated aquifer beneath the site is greater than 15 m with rock at depths below about 18 m;
- The perimeter walls will most likely terminate within the sands about 5 – 10 m above the base of the aquifer;
- The width of the proposed basement is small compared to the entire width of aquifer so groundwater flow can flow relatively easily around the basement;
- The proposed basement will be constructed directly adjacent to an existing two level basement on 41 Cross Street; and
- The aquifer is in highly permeable sand which will allow groundwater to flow relatively easily around and beneath the tanked basement.

In conclusion, the predicted effect of the tanked basement is considered to be within the requirements of the Woollahra Municipal Council guidelines.

7.3 Excavation Conditions and Batter Slopes

Once the existing building and pavements are removed, the perimeter wall installed and the groundwater levels reduced to at least 1 m below the lowest basement level, excavation will be carried out through filling and natural sands which should be readily removed using conventional earthmoving equipment such as tracked hydraulic excavators. Based on the measured groundwater levels, the bulk excavation to 6 m depth will be approximately 4 m below the groundwater level and temporary dewatering will be required to remove water already beneath the site and that which will flow under the perimeter wall as the excavation proceeds.

Based on previous experience, tracked plant and machinery will be required on the sandy soils during bulk earthworks. Once bulk excavation is complete trafficability, could be improved by placing working platform, such as a layer of compacted crushed concrete or similar, which may subsequently be used as sub-base below the basement floor slab.

During the bulk excavation phase, temporary batter slopes in sand within the perimeter walls above the groundwater level should not exceed 1.5H:1V (horizontal:vertical) in both filling and sand soils.

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA, 2014). Reference should be made to the preliminary contamination assessment and waste classification report for comments on the contamination status of the soils.

7.4 Dewatering and Tanking

For basement construction, it is assumed that a perimeter wall will be installed to below the bulk excavation level then site within the walls will progressively excavated and, when required, dewatered until design levels are reached and a tanked basement floor slab placed.

Depend on the design of the floor slab and footings, the bulk excavation for the basement will be to about 6 m depth. Typically the groundwater level should be lowered to at least 1 m below the bulk excavation level to allow machinery to operate and traverse the site. Therefore, the groundwater level measured at the time of the investigation may need to be temporarily lowered by approximately 5 m depth inside the shoring.

The loads due to a groundwater table rising to at least the ground surface should be considered in the basement and floor slab design.

In the long term, the uplift from the groundwater will usually be resisted by the weight of the buildings above once four to five levels are reached, and the detailing of the slab and foundations should be designed accordingly and the loads confirmed by the structural engineers.

7.4.1 Piping Failure

Piping failure through sandy soils at the base of the excavation may occur if not adequately dewatered during construction. Piping failures occur when the upward flow rates through the sand create uplift on the sand particles equal or greater than the effective weight of the soil. The risk of piping failure will generally be greatest if dewatering pumps fail when bulk excavation is below the water level outside the perimeter walls. It is recommended that the perimeter wall should have a minimum embedment of 7 m below the deepest bulk excavation level to control the risk of piping failure. Detailed analysis may indicate deeper embedment is required to reduce groundwater inflows to acceptable levels and drawdown on adjacent properties.

7.4.2 Method of Dewatering

Sump and pump dewatering methods will not be practical or effective for the high permeability sandy soils and spear-points installed at regular intervals within the confines of the excavation will be required. In this system, spears (slotted PVC pipes) are installed below the groundwater table and generally spaced at about 1 – 2 m centres around the perimeter of the excavation. Alternatively larger diameter spears can be used and positioned close to the centre of the site. The spears connect to a series of pumps and hoses which collect groundwater, usually in a sedimentation tank, prior to discharge off-site.

Based on the results of testing, the relatively clean sands underlying the site have a hydraulic conductivity (k) of between 5×10^{-4} – 10×10^{-4} m/sec. These values are consistent with typical values for sand soil and could be used for preliminary design of the temporary spear-point dewatering system for this site.

7.4.3 Drawdown and Settlement

The dewatering system is expected to be required to temporarily lower of the normal groundwater level by approximately 5 m depth, about 1 m belwo the base of the bulk excavation.

If detailed analysis indicates additional measures are required to control the drawdown outside the relatively impermeable perimeter shoring walls around the site to less than 0.3 m the following options, or combination of options, could be considered:

- use of recharge/reinjection wells to direct pumped water back into the ground outside the excavation perimeter to help maintain a more stable groundwater table. Usually vertical reinjection wells are installed outside the site where there is space or approval from neighbouring properties to do so. Where there is no access for vertical reinjection wells, the use of inclined reinjection wells installed through the shoring wall to below the adjacent areas could also be considered but would still require approval from the neighbouring property owners. Reinjection would generally be subject to approval from relevant authorities (i.e. DPI - Water).
- as socketing the perimeter walls into the relatively low permeability rock beneath the site is likely to be cost prohibitive, the walls into the interbedded sand and silt/clay below about 12 – 15 m depth to reduce seepage flows, however the effectiveness of this option cannot be easily assessed due the variability of this layered profile and potential for sandy channels.
- construction of a grout stabilised layer (or similar) below the bulk excavation level to reduce vertical flows. This is a specialised activity that is not routinely carried out for basement construction in the Sydney Region and would require further input from a specialist contractor and detailed analysis.

During construction, it is recommended that drawdown outside the excavation in the vicinity of the adjacent properties should be monitored by:

- Installing standpipes in accessible areas on adjacent properties (or roads) to monitor groundwater drawdown levels during dewatering;
- Measuring groundwater levels weekly for at least three weeks prior to operation of the dewatering system to establish pre-developed levels;
- Measuring groundwater levels twice daily during the first two days of dewatering, and then daily during the first week of dewatering and weekly until decommissioning of the dewatering pumps. The information should be provided to the geotechnical engineer on the day of measurement. A lesser frequency may be feasible once results are reviewed and assessed by the geotechnical engineer;

- Where drawdown levels exceed a 'trigger level' (to be set) below pre-developed groundwater levels, the reason for the change in groundwater level should be investigated and measures put in place to rectify the exceedance. These measures could include reduction of pumping rates or suspension of dewatering;

Design of the dewatering system will need to consider the effects of drawdown on adjacent properties and the dewatering of the site should be carried out by a contractor with demonstrated experience in similar conditions.

Numerical modelling should be carried out to assess the effectiveness of the proposed dewatering system and predict drawdown levels and associated settlements on adjacent properties. The modelling may indicate recharging of groundwater outside the basement excavation or installation of perimeter shoring to greater depths are required to reduce the risk of drawdown affecting adjacent properties. Groundwater modelling is generally carried out once details of the proposed shoring and dewatering system are available.

7.4.4 Groundwater Disposal

Groundwater that is removed from the site will require disposal. Generally, water from dewatering operations should be suitable for disposal by pumping to stormwater drains, subject to confirmation testing and approval from Council. Further testing and reporting may be required to determine appropriate disposal options, together with approval from relevant authorities.

7.5 Retaining Walls

Vertical excavations within the sandy soils will require retaining structures both during construction and as part of the final structure. It is anticipated that one to two rows of temporary anchors will be required to provide lateral restraint and limit wall movements. Alternatively top-down construction may be adopted, particularly if anchors cannot be used or if it is necessary to reduce wall movements.

7.5.1 Retaining Wall Design

Due to the depth of saturated sand to be retained and the proximity of adjacent buildings and infrastructure, it is possible that the shoring system may need to be supported by internal bracing to provide sufficient support and to reduce wall movements. Alternatively top down construction could be considered to further reduce risk and wall deflections. The use of temporary ground anchors to support the shoring walls will be difficult on all sides of the site due to the water charged sand soil for the full depth of the excavation.

Preferably, perimeter shoring walls should be founded well below the base of the bulk excavation at depths of 13 m (RL 10m AHD), below the very loose to loose sand layers, in medium dense or denser sand (possibly deeper to reduce water inflow) and to provide sufficient lateral restraint at the base of the excavation.

It is suggested that preliminary design of shoring systems may be based on the earth pressure coefficients provided in Table 6. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_o) values should be used where the wall movement needs to be minimal.

Where multiple rows of anchors or props are used it is suggested that preliminary design of shoring walls could be based on a trapezoidal earth pressure distribution where the maximum pressure acts over the central 60% of the wall, reducing to zero at the top and base.

Table 6: Retaining Wall Design Parameters

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient			Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (Ka)	At Rest (Ko)	Passive (Kp)		
Filling and Sand: very loose to loose	19	0.40	0.58	2.5	0	25
Sand: loose to medium dense	20	0.33	0.50	3.0	0	30
Sand: dense to very dense	22	0.27	0.43	3.7	0	35

The design of the shoring should allow for all surcharge loads, including building footings, inclined slopes behind the wall, traffic and construction related activities. Hydrostatic pressure acting on the shoring walls should also be considered in the design.

Detailed design of shoring should preferably be carried out using WALLAP, FLAC or other accepted computer analysis programs capable of modelling progressive excavation and anchoring and predicting potential lateral movements, stresses and bending moments.

7.5.2 Retaining Wall Systems

Perimeter wall shoring systems potentially suitable for the site in order of preference from a geotechnical and groundwater perspective could include:

- Diaphragm wall
- Secant pile wall
- Cement soil mixed (CSM) wall

A diaphragm wall system is likely to provide the least permeably retaining system for the site due to the geology and hydrogeology, however consideration will have to be given to the large equipment and plant required for construction that may present logistical challenges for the shoring contractor.

A secant pile wall comprising interlocking Continuous Flight Auger (CFA) piles or CFA piles with jet grouted columns between the piles could be suitable for the site. This shoring system can generally provide an effective seal to minimise sand loss and water inflow from behind the wall, and if adequately supported, minimise lateral deflections. The 'hard' (reinforced) piles can be incorporated into the vertical load carrying footing system and can generally form part of the basement structure. For CFA piles, care will be required to avoid decompression of the sandy soils during augering, which can lead to loosening of the foundations and damage to adjacent structures. It may be necessary to adopt temporary segmental casing to reduce the risk of decompression.

Soil mixed wall systems also provide a suitable alternative to the more conventional secant pile wall. These walls are constructed using specialised equipment to either blend cement with the in-situ soils to create a soil-cement mix. There are several different systems available and further advice should be obtained from the specialist piling contractor regarding the suitability of the wall system to this site. In particular, confirmation should be sought in relation to the consistency/strength of the soil mixed wall, the long term durability, permeability, potential issues with blending cement and joining the soil mixed wall with the tanked basement slab.

As a guide, well designed shoring walls in sand supported by anchors may experience lateral wall movements in the order of 1 mm to 2 mm for each metre of excavation height. The extent of movement will depend on the final design and construction methods used. A programme of precise survey monitoring should be adopted together with inclinometers installed in the wall during construction to assess movements of the shoring wall and adjacent buildings progressively during the excavation to ensure that tolerable limits are not exceeded and to provide an early indication of whether additional support is required.

Sheet piles are not recommended for this site as they are generally only suitable for shallower excavations above the water table and where there are no movement sensitive structures adjacent to the excavation.

A contiguous pile wall comprising closely spaced/touching CFA piles is also not recommended due to risks associated with seepage and sand loss in between the piles, particularly below the groundwater table.

7.5.3 Adjacent Foundations

Stabilising of the foundations beneath the neighbouring properties, which are currently expected to comprise shallow strip footings may also be considered (this may change with time due to future development). This would improve the strength of the sands and also help to reduce differential movements. This may be achieved through grout injection or chemical stabilisation. Further advice should be obtained from specialist contractors regarding the suitability of stabilisation at this site.

Not with standing this the, aim of the perimeter shoring system should be to control the lateral movements to acceptable levels. Previous experience indicates that suggested types of basements properly designed for water charged sands can be without any significant adverse effects on adjacent structures.

7.5.4 Ground Anchors

If top down construction is not carried out, temporary anchors or stiff propping will be required to control perimeter wall movements during the construction phase, with permanent support of walls provided by the final structure.

Design of temporary anchors within loose to dense (or denser) sand may be based on a friction angles (ϕ') of 30 - 35 degrees. Anchors should not be designed to bear in very loose sand. Trial anchors may be used to determine if higher friction angles/shaft adhesion values are achievable. The anchors should inclined at an angle no greater than 15° and have a free length extending to behind a line drawn up at 45° from the base of the excavation with a minimum length of at least 3 m, and lift-off tests should be carried out to confirm the anchor capacities. Post-grouting techniques may be used to achieve higher capacities. Vertical anchors to resist uplift loads during construction may also be required, possibly extending into the underlying medium to high strength sandstone for greater capacity. Suggested anchor design parameters for the carious materials underlying the site are provided in Table 7.

Table 7: Anchor Design Parameters

Material	Typical Internal Angle of Friction (ϕ°)	Unconfined Conpressive Strength (MPa	Ultimate Bond Stress (kPa)
Filling and Sand: very loose	25	-	-
Sand: loose	30	-	11d*
Sand: medium dense	33	-	13d*
Sand: dense to very dense	35	-	15d*
Sandstone: medium strength	-	10	1000
Sandstone: high strength	-	30	3000

* Estimated bas on bulk soil density = 20 kN/m^3, and d is depth below ground level to centre of bond length. These values should be halved below the water table

If temporary vertical anchors to rock are proposed, additional investigation of should be carried out to confirm the rock depth and strength across the site. The investigation would require cored boreholes extending to at least the proposed anchor depth with point load index strength testing of the recovered core samples to provide quantitative information for the optimisation of the anchor design.

The anchors will need to be carefully positioned and may need to be inclined at steeper angles to avoid adjacent services and building footings. It is noted that permission from property owners will be required prior to installing soil anchors beneath adjacent sites.

It is recommended that only reputable, specialist anchor contractors be engaged to design and/or install temporary anchors on this site.

7.6 Subgrade Preparation

It is expected that at the base of the bulk excavation level, the subgrade will be loose to medium dense sand. Following excavation to achieve design levels, the exposed soil surface should be rolled with a at least six passes of a minimum 12 tonne smooth drum roller. The final pass (test roll) should be inspected by a geotechnical engineer to help identify any weak or heaving areas.

If heavy plant is required to operate at the base of the excavation, a working platform should be constructed over the prepared subgrade. The platform should be constructed from good quality granular material with low fines, such as recycled concrete or high strength ripped sandstone. The thickness of the platform should be assessed once specific details of the heavy plant that will operate within the basement are known.

7.7 Foundations

7.7.1 Raft Slab

Individual pad and strip footings are not recommended for this site due to the potential for excessive and unpredictable differential settlement resulting from the very loose to loose sand underlying the site. A raft slab foundation may be feasible, however this will be subject to detailed review and analysis of bearing pressures and settlements as the design progresses and more specific details of the founding level, column layout and slab loadings become available. The presence of very loose top loose sand layers below the raft slab must considered in the design particularly for concentrated column and core loads.

Details of structural loads were not available at the time of preparing this report. Based on similar sized projects it is anticipated that a distributed slab load in the order of 50 kPa (after taking into account the mass of the soil excavated) may be applicable for the six (6) storey building. Preliminary settlement analyses have been carried out assuming a distributed slab load of 50 kPa, with a loaded area of 40 m by 40 m. The preliminary design of raft slabs to support column and floor loadings may be based on a modulus of subgrade reaction of 2 - 3 kPa/mm for the broadly loaded area. Settlements of between 15 – 25 mm could be expected under the assumed loads. It is noted that the modulus of subgrade reaction is not a fundamental soil parameter and is dependent on the load, the size of the loaded area, the rigidity of the raft system and the settlement characteristics of the subgrade materials.

A piled raft foundation may also be considered if the magnitude of the estimated settlements needs to be reduced.

7.7.2 Pile Foundations

The alternative to shallow foundations is to support the structural loads on piles founded within the dense to very dense sand which the CPTs encountered at depths of approximately 16 – 18 m below the existing surface level.

Continuous Flight Auger (CFA), concrete injected piles or cast-in-situ screwed pile types such as Atlas or Omega piles could be used at this site. These types of piles are all associated with relatively low levels of noise and vibration. Screwed cast in-situ piles leave a reinforced concrete screw shaped pile and involve lateral displacement of the soil during installation, more efficiently using the in-situ capacity of the soil.

It is expected that noise and vibration constraints at this site will preclude the use of driven pile types. Open bored piles will not be appropriate due to the potential for soil collapse and groundwater inflow and the relatively small site will preclude the use of bored piles being drilled under bentonite due to the size of the equipment required.

The design geotechnical strength of piles requires a geotechnical strength reduction factor (ϕ_g). This ϕ_g value, however, should be determined by the designer in accordance with the AS 2159 - 2009). The selection of ϕ_g is based on a series of individual risk ratings (IRR) which are weighted to give an average risk rating (ARR). The IRR values depend on factors such as the type and quality of testing, design method and parameter selection, pile installation control and monitoring, pile testing regime, and the redundancy in the foundation system.

7.8 Dilapidation Surveys

Dilapidation surveys should be undertaken on surrounding structures and pavements prior to commencing work on the site to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed. The appropriate extent of dilapidation surveys may be better assessed once details of the proposed development and construction methods have been confirmed.

7.9 Seismic Loading

In accordance with AS 1170 - 2007 Structural Design Actions, Part 4: Earthquake Actions in Australia the site has a hazard factor (Z) of 0.08 and subsoil Class Ce.

8. Conclusion

This report has discussed various geotechnical aspects of the proposed development and has outlined appropriate construction methods, monitoring requirements, and design parameters. Similar basements have been constructed in Sydney without significant impacts to surrounding properties. It is considered that the basement could be designed and constructed without significant adverse impacts to surrounding properties.

9. Limitations

Douglas Partners Pty Ltd has prepared this report for this project at 19-27 Cross Street Double Bay in accordance with DP’s proposal SYD180186 dated 10 May 2018 and acceptance received dated 11 May 2018. The work was carried out under DP’s Conditions of Engagement. This report is provided for the exclusive use of Tri-Anta 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 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.

This report must be read in conjunction with all of the attachments 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 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.

Asbestos has not been detected by observation or by laboratory analysis, either on the surface of the site, or in filling materials at the test locations sampled and analysed. Building demolition materials, such as concrete, brick, tile [list as appropriate to the field work findings], were, however, located in previous below-ground filling and/or above-ground stockpiles [as appropriate], and these are considered as indicative of the possible presence of hazardous building materials (HBM), including asbestos.

Although the sampling plan adopted for this investigation is considered appropriate to achieve the stated project objectives, there are necessarily parts of the site that have not been sampled and analysed. This is either due to undetected variations in ground conditions or to budget constraints (as discussed above), or to parts of the site being inaccessible and not available for inspection/sampling [where appropriate], or to vegetation preventing visual inspection and reasonable access [where appropriate]. It is therefore considered possible that HBM, including asbestos, may be present in unobserved or untested parts of the site, between and beyond sampling locations, and hence no warranty can be given that asbestos is not present.

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 and groundwater 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
Drawings 1 – 5
CPT Plots
Borehole Logs

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

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

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.

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.

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. 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	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

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

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

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	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

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;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

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

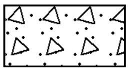
Symbols & Abbreviations

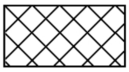
Graphic Symbols for Soil and Rock

General

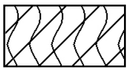
 Asphalt

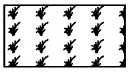
 Road base

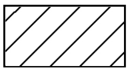
 Concrete

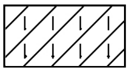
 Filling

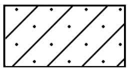
Soils

 Topsoil

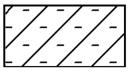
 Peat

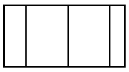
 Clay

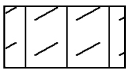
 Silty clay

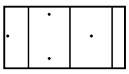
 Sandy clay

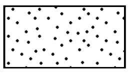
 Gravelly clay

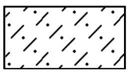
 Shaly clay

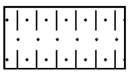
 Silt

 Clayey silt

 Sandy silt

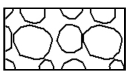
 Sand

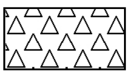
 Clayey sand

 Silty sand

 Gravel

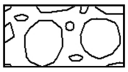
 Sandy gravel

 Cobbles, boulders

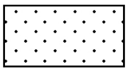
 Talus

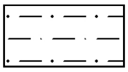
Sedimentary Rocks

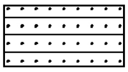
 Boulder conglomerate

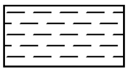
 Conglomerate

 Conglomeratic sandstone

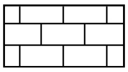
 Sandstone

 Siltstone

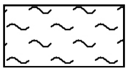
 Laminite

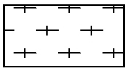
 Mudstone, claystone, shale

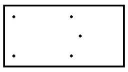
 Coal

 Limestone

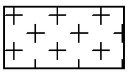
Metamorphic Rocks

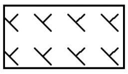
 Slate, phyllite, schist

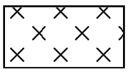
 Gneiss

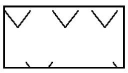
 Quartzite

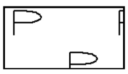
Igneous Rocks

 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

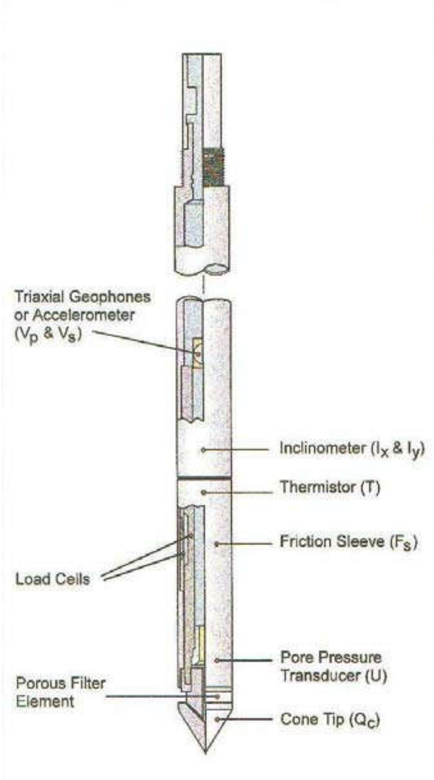


Figure 1: Cone Diagram

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)

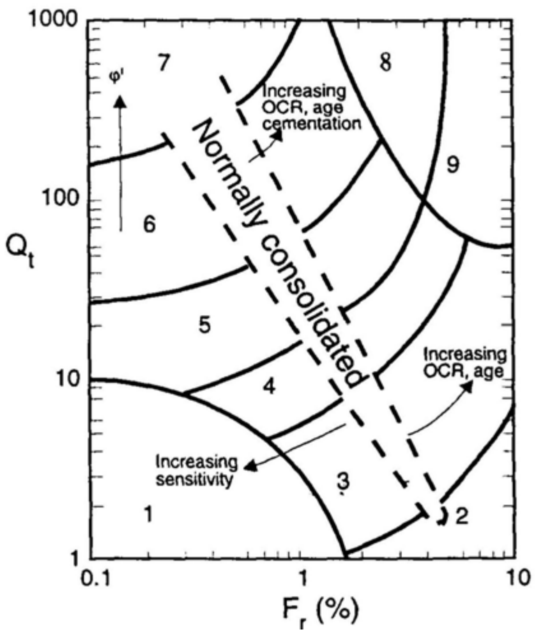


Figure 3: Soil Classification Chart

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.

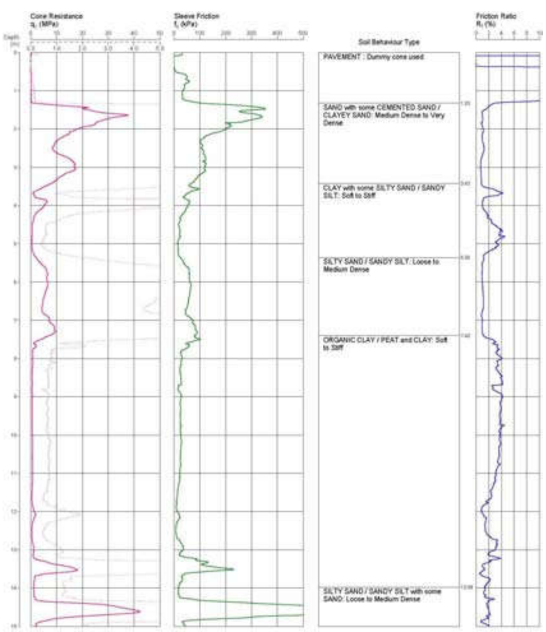
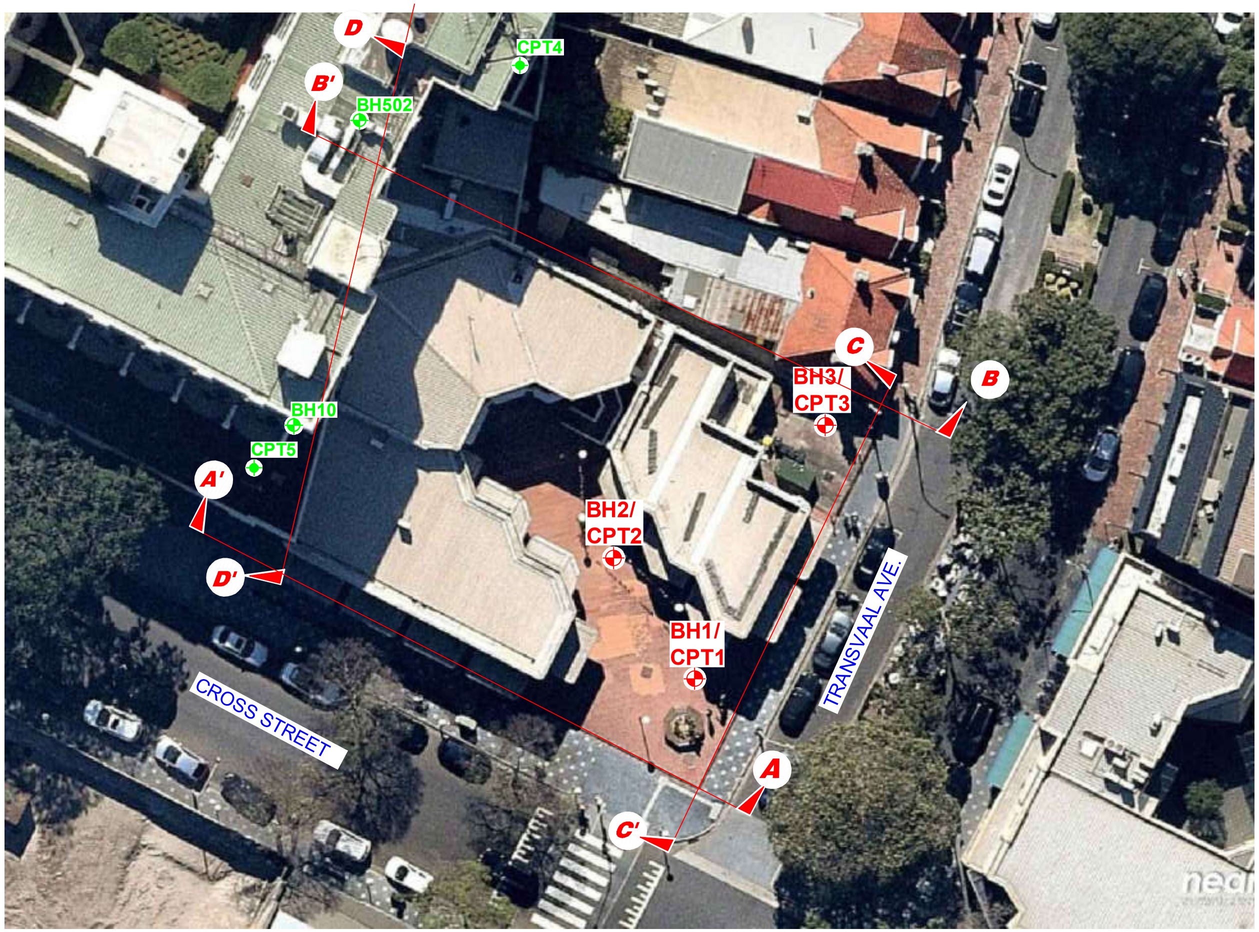
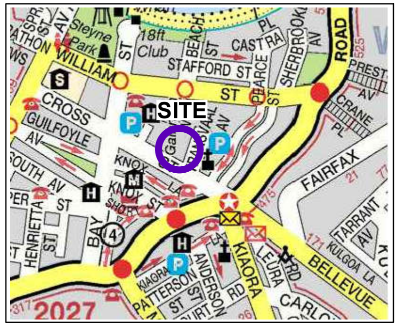


Figure 4: Sample Cone Plot





Locality Plan

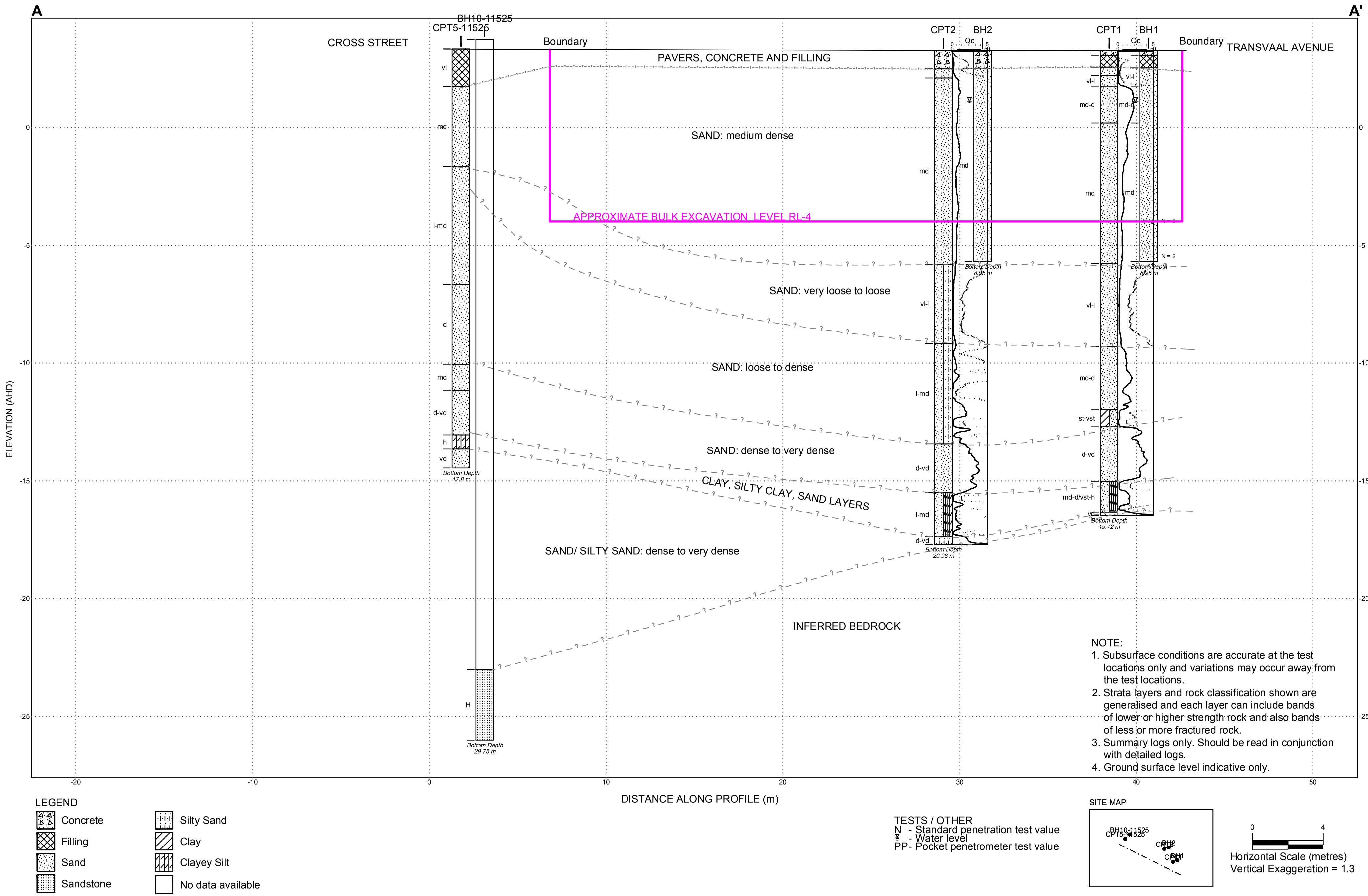
NOTE:
1: Base image from Nearmap.com (Dated 18.8.2018)
2: Test locations are approximate only and are shown with reference to existing features.

LEGEND

- Previous borehole location (1988)
- Previous CPT location (1988)
- Current borehole and CPT location
- A — A' Geotechnical Cross Section A-A'

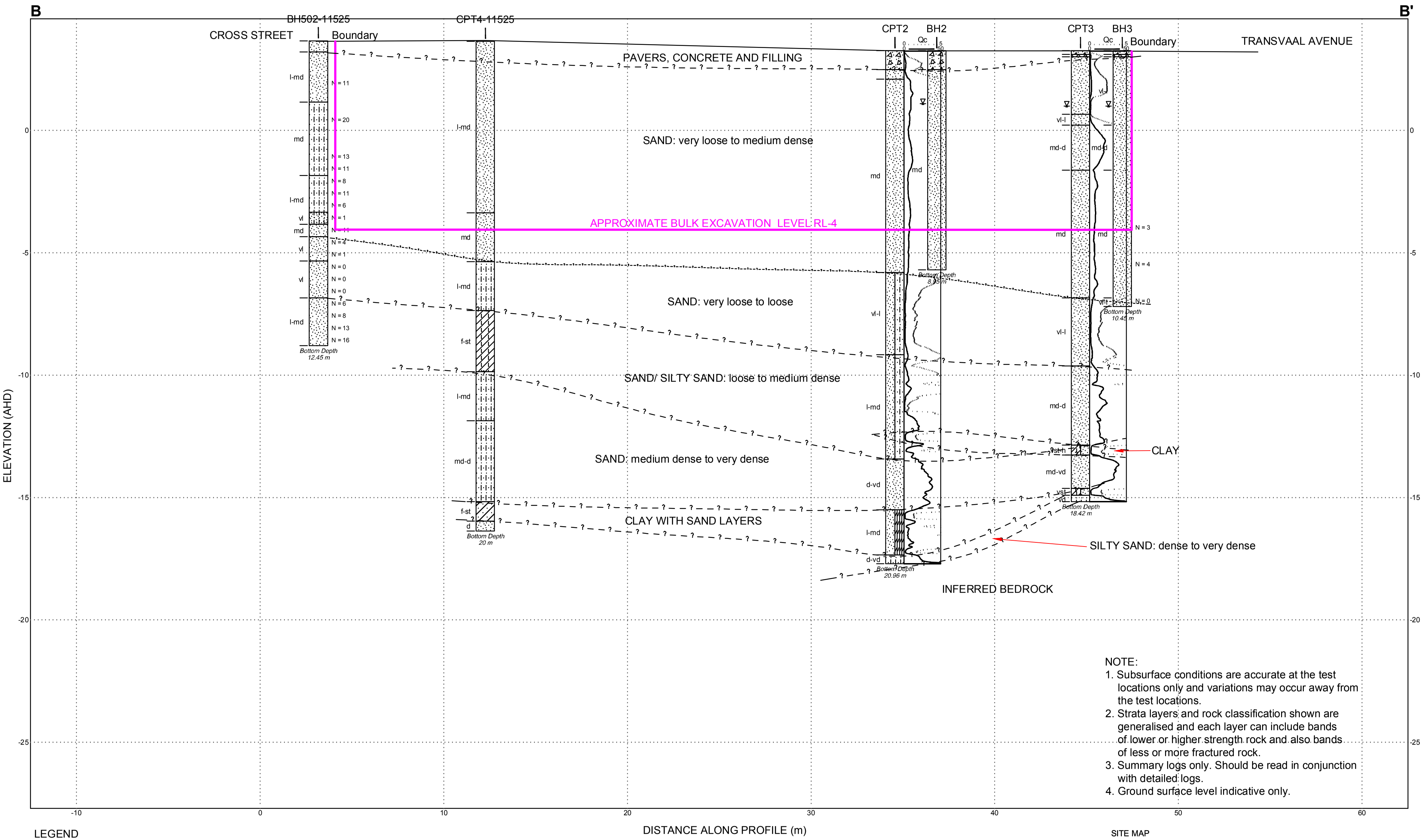
Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Tri-Anta Pty Ltd		TITLE: **Test & Cross Section Locations**
	OFFICE: Sydney	DRAWN BY: PSCH	**Proposed Mixed Use Development**
	SCALE: 1:400 @ A3	DATE: 12.9.2018	**19-27 Cross Street, DOUBLE BAY**

PROJECT No:	86397.00
DRAWING No:	1
REVISION:	0



NOTE:
1. Subsurface conditions are accurate at the test locations only and variations may occur away from the test locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.
4. Ground surface level indicative only.

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Tri-Anta Pty Ltd	TITLE: **Cross-section A-A'**	PROJECT No: 86397.00
	OFFICE: Sydney / DRAWN BY: KS	**Proposed Mixed Use Development**	DRAWING No: 2
	SCALE: 1:200 (H) 1:150 (V) @ A3 / DATE: 12.09.2018	**19-27 Cross Street, DOUBLE BAY**	REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the test locations only and variations may occur away from the test locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.
4. Ground surface level indicative only.

LEGEND

- Filling
- Sand
- Concrete
- Silty Sand
- Silty Clay
- Sandy Clay

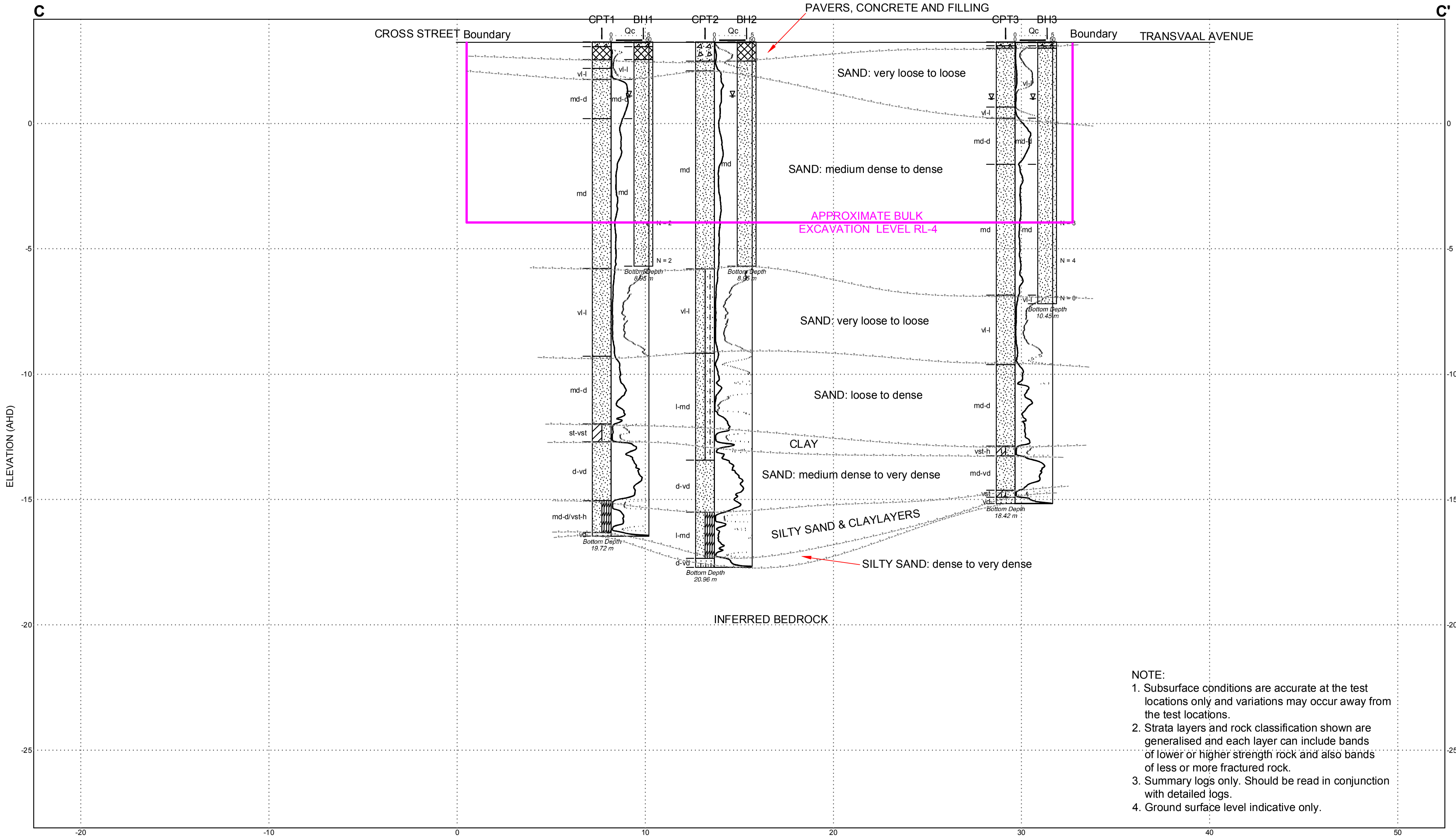
TESTS / OTHER
N - Standard penetration test value
- Water level
PP- Pocket penetrometer test value

SITE MAP

Horizontal Scale (metres)
Vertical Exaggeration = 1.3

Douglas Partners
Geotechnics / Environment / Groundwater

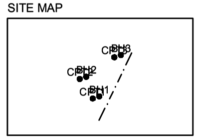
CLIENT: Tri-Anta Pty Ltd		TITLE: **Cross-section B-B'** **Proposed Mixed Use Development** **19-27 Cross Street, DOUBLE BAY**	PROJECT No: 86397.00
OFFICE: Sydney	DRAWN BY: KS		DRAWING No: 3
SCALE: 1:200 (H) 1:150 (V) @ A3	DATE: 12.09.2018		REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the test locations only and variations may occur away from the test locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.
4. Ground surface level indicative only.

TESTS / OTHER
N - Standard penetration test value
▽ - Water level
PP- Pocket penetrometer test value

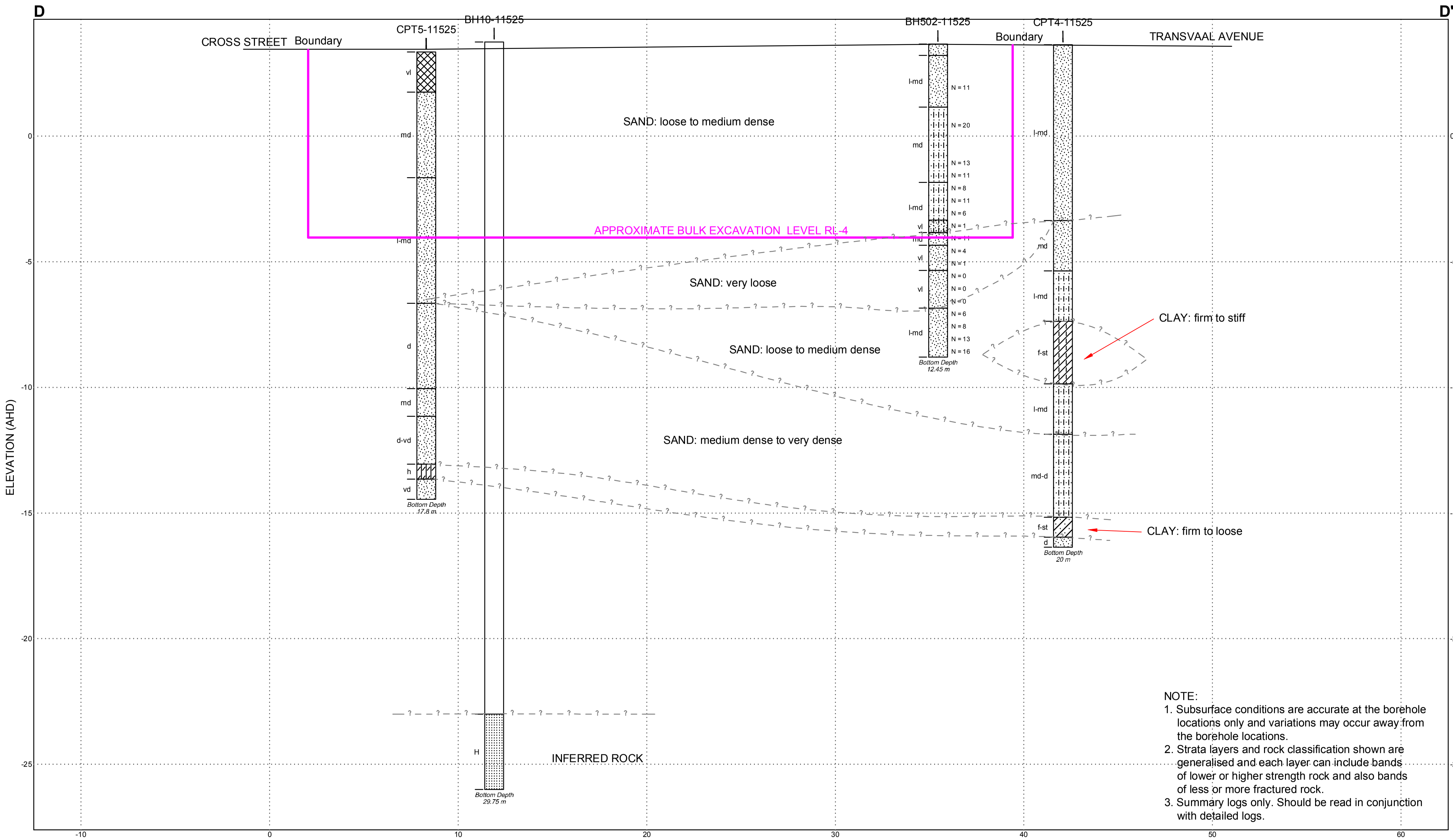
SITE MAP



Horizontal Scale (metres)
Vertical Exaggeration = 1.3

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Tri-Anta Pty Ltd		TITLE: **Cross-section C-C'**	PROJECT No: 86397.00
OFFICE: Sydney	DRAWN BY: KS	**Proposed Mixed Use Development**	DRAWING No: 4
SCALE: 1:200 (H) 1:150 (V) @ A3	DATE: 12.09.2018	**19-27 Cross Street, DOUBLE BAY**	REVISION: 0

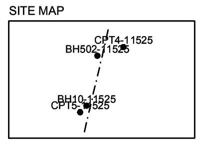


NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- Sandstone
- Sand
- Silty Sand
- Silty Clay
- Sandy Clay
- Filling
- Clayey Silt
- No data available

TESTS / OTHER
N - Standard penetration test value
Water level
PP- Pocket penetrometer test value



Horizontal Scale (metres)
Vertical Exaggeration = 1.3

CLIENT: Tri-Anta Pty Ltd		TITLE: **Cross-section D-D'** **Proposed Mixed Use Development** **19-27 Cross Street, DOUBLE BAY**	PROJECT No: 86397.00
OFFICE: Sydney	DRAWN BY: KS		DRAWING No: 5
SCALE: 1:200 (H) 1:150 (V) @ A3	DATE: 12.09.2018		REVISION: 0

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Tri-Anta Pty Ltd
PROJECT: Proposed Mixed Use Development
LOCATION: 19-27 Cross Street, Double Bay

SURFACE LEVEL: 3.25 AHD
EASTING: 337570
NORTHING: 6250157
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 86397.00
DATE:
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Results & Comments	Water	Well Construction Details
0.19	TILE / CONCRETE					Gatic Cover & Cap; Backfill; Bentonite Seal
0.7	FILLING: brown, gravelly sand filling with a trace of silt. Foam, glass, plastic, roof tile, fabric and cement fragments. Humid	A	0.3–0.4			Blank PVC Pipe
	SAND: very loose to loose, yellow, fine to medium grained sand. Humid	A	0.7–0.8			
	-0.9 m, becoming orange brown	A	0.9–1.0			
	-1.1 m, becoming light grey brown	A	1.4–1.5			
	-1.5 m, medium dense to dense, light grey, humid	A	1.9–2.0			Filter Sand
	-2.2 m, moist				17-06-18	
	-2.5 m, becoming brown	A	2.4–2.5			
		A	2.9–3.0			
	3.06 m, medium dense, brown, saturated					
		A	4.4–4.5			Slotted PVC Pipe
	-7.14 to 7.38 m, loose layer	S	7.0–7.45	1,1,1 N = 2		
	-8.0 m, grey, with a trace of carbonaceous materials					End Cap
8.95		S	8.5–8.95	1,1,1 N = 2		
	Bore discontinued at 8.95m					

RIG: Bobcat **DRILLER:** GM **LOGGED:** JY **CASING:** HQ to 5.5m

TYPE OF BORING: Hand auger to 1.1m, SFA (110mm diam) to 5.5m, rotary drilling to 8.5m.

WATER OBSERVATIONS: Groundwater observed at 2.2m on 17/6/2018

REMARKS: Location coordinates are in MGA94 Zone 56. Well installed to 8.3m, Gatic cover/backfill to 0.2m, Bentonite to 0.7 then gravel. Screen 0.7-8.3m

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)

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: TRI-ANTA PTY LTD

PROJECT: PROPOSED MIXED USE DEVELOPMENT

LOCATION: 19-27 CROSS ST, DOUBLE BAY

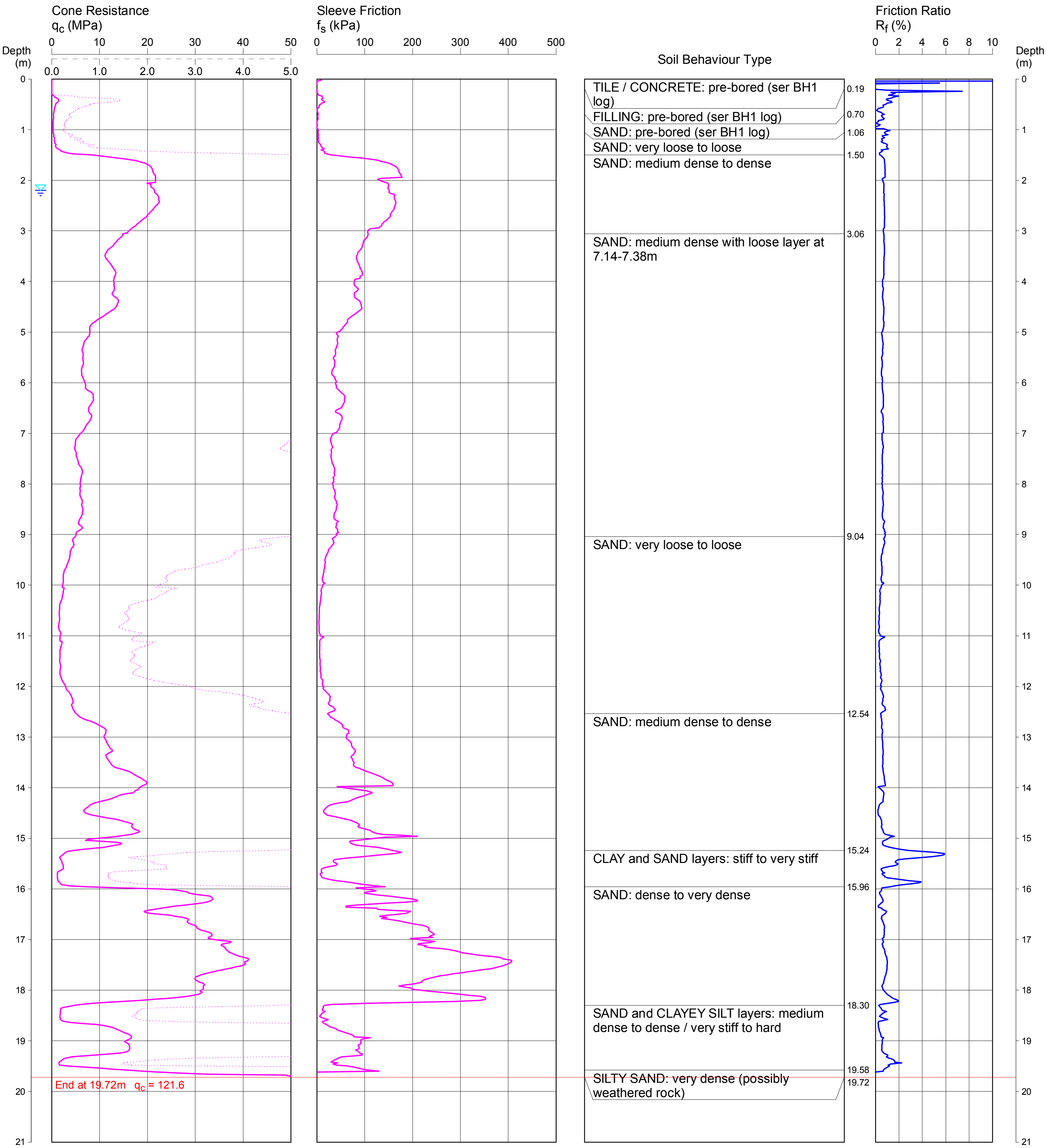
REDUCED LEVEL: 3.25

COORDINATES: 337567E 6250156N MGA94

CPT1

DATE 18/05/2018

PROJECT No: 86397



REMARKS: CONCRETE CORED, HAND AUGERED AND BACKFILLED TO 1.1 m DEPTH PRIOR TO TESTING; TEST DISCONTINUED DUE TO BENDING AT REFUSAL HOLE COLLAPSED AT 2.1 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.20m depth (measured)

File: P:\86397.00 - DOUBLE BAY 19-27 Cross Street\4.0 Field Work\4.2 Testing\CPTs\CPT1.CP5
Cone ID: 120618 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

CONE PENETRATION TEST

CLIENT: TRI-ANTA PTY LTD

PROJECT: PROPOSED MIXED USE DEVELOPMENT

LOCATION: 19-27 CROSS ST, DOUBLE BAY

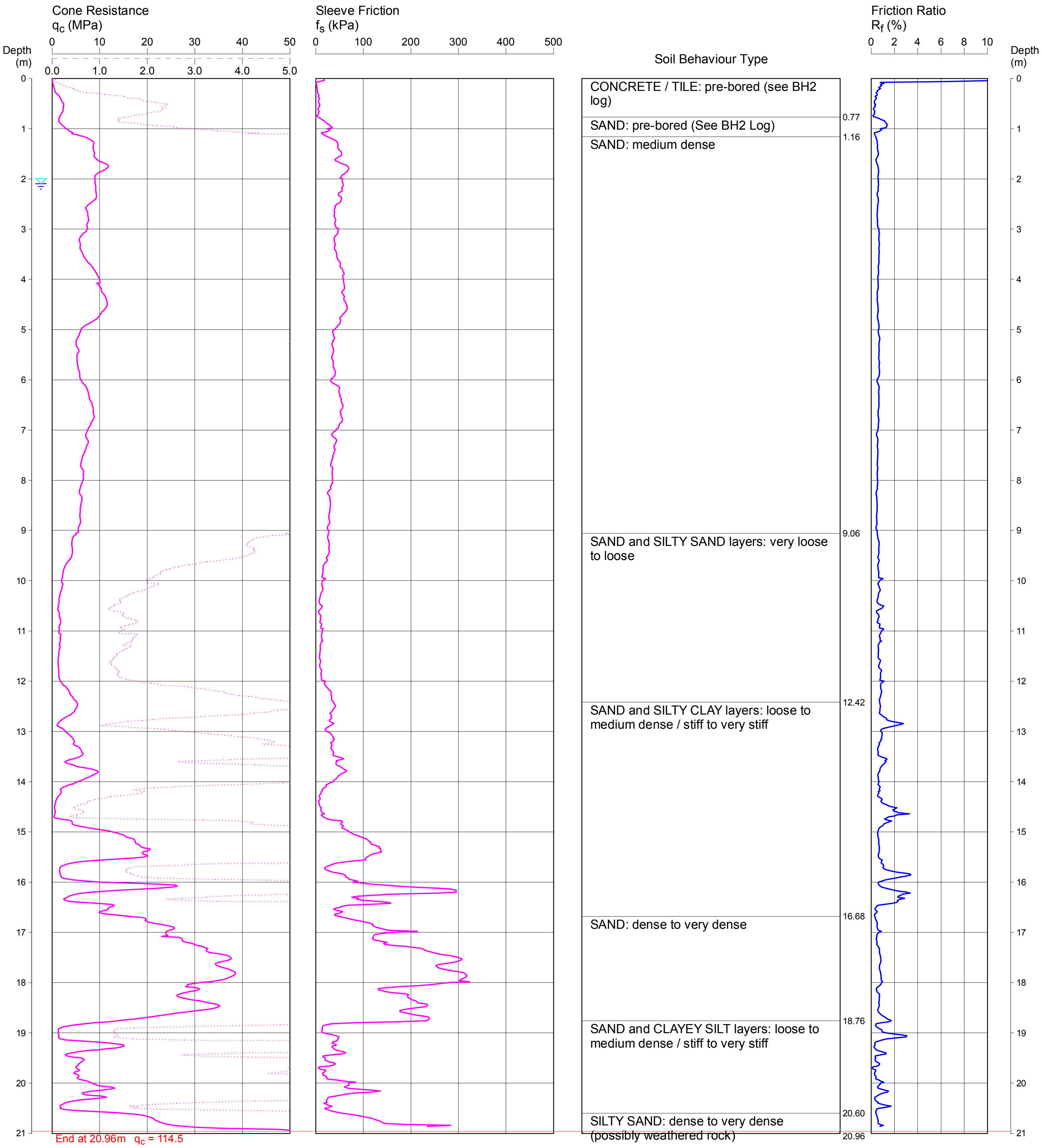
REDUCED LEVEL: 3.25

COORDINATES: 337561E 6250165N GDA94

CPT2

DATE 18/05/2018

PROJECT No: 86397



REMARKS: CONCRETE CORED, HAND AUGERED AND BACKFILLED TO 1.16 m DEPTH PRIOR TO TESTING; TEST DISCONTINUED AT REFUSAL
HOLE COLLAPSED AT SURFACE AFTER WITHDRAWAL OF RODS

Water depth after test: 2.10m depth (assumed)

File: P:\86397.00 - DOUBLE BAY 19-27 Cross Street\4.0 Field Work\4.2 Testing\CPTs\CPT2.CP5
Cone ID: 120618 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: TRI-ANTA PTY LTD

PROJECT: PROPOSED MIXED USE DEVELOPMENT

LOCATION: 19-27 CROSS ST, DOUBLE BAY

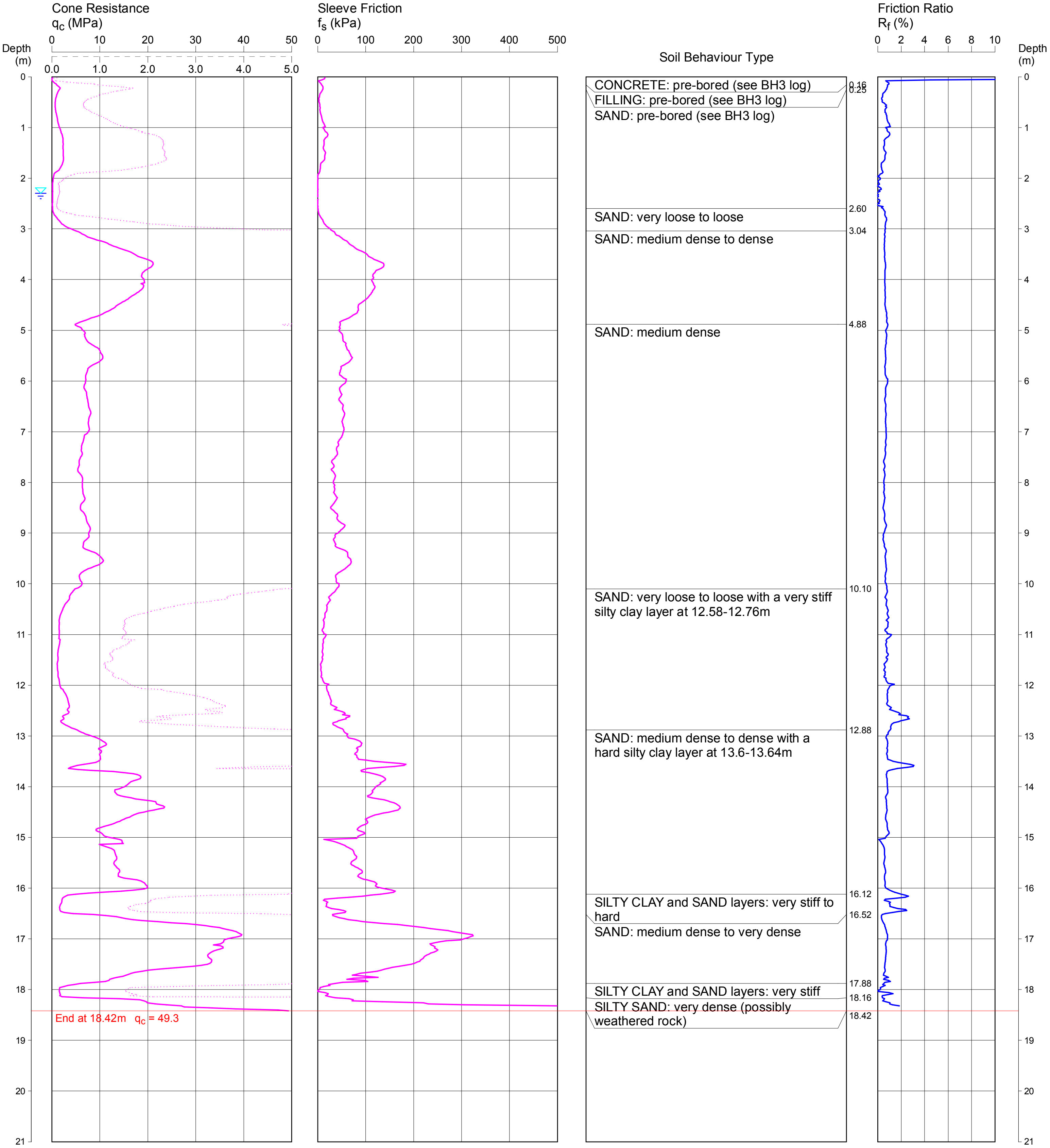
REDUCED LEVEL: 3.25

COORDINATES: 337576.848072984E 6250175.40173591N GDA94

CPT3

DATE 18/05/2018

PROJECT No: 86397



REMARKS: CONCRETE CORED, HAND AUGERED AND BACKFILLED TO 2.6 m DEPTH PRIOR TO TESTING; TEST DISCONTINUED DUE TO BENDING NEAR REFUSAL
GROUNDWATER MEASURED AT 2.3 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.30m depth (measured)

File: P:\86397.00 - DOUBLE BAY 19-27 Cross Street\4.0 Field Work\4.2 Testing\CPTs\CPT3.CP5
Cone ID: 120618 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: Tri-Anta Pty Ltd
PROJECT: Proposed Mixed Use Development
LOCATION: 19-27 Cross Street, Double Bay

SURFACE LEVEL: 3.25 AHD
EASTING: 337564
NORTHING: 6250166
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 86397.00
DATE:
SHEET 1 OF 1

Depth (m)	Description of Strata	Sampling Type	Sampling Depth	Sample	Results & Comments	Water	Well Construction Details
0.0	CONCRETE / TILE						Gatic Cover & Cap; Backfill; Bentonite Seal; Blank PVC Pipe
0.77	SAND: medium dense, light grey, fine to medium grained sand. Damp	A	1.0–1.1				
	-1.2 m, moist						
		A	1.9–2.0			17-06-18	Filter Sand
	-2.1 m, grey brown, wet	A	2.4–2.5				
	-4.0 m, trace black carbonaceous material	A	3.9–4.0				
	-5.0 m, saturated						Slotted PVC Pipe
		A	5.4–5.5				
		A	7.0–7.45		2,2,2 N = 4		
		A	8.5–8.95		1,2,1 N = 3		End Cap
8.95	Bore discontinued at 8.95m						

RIG: Bobcat **DRILLER:** GM **LOGGED:** JY **CASING:** HQ to 5.5m

TYPE OF BORING: Hand auger to 1.16m, SFA (110mm diam) to 5.5m, rotary drilling to 8.5m.

WATER OBSERVATIONS: Groundwater observed at 2.1m on 17/6/2018

REMARKS: Location coordinates are in MGA94 Zone 56. Well installed to 8.5m, Gatic cover/backfill to 0.2m, Bentonite to 0.7 then gravel. Screen 1.0-8.5m

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)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

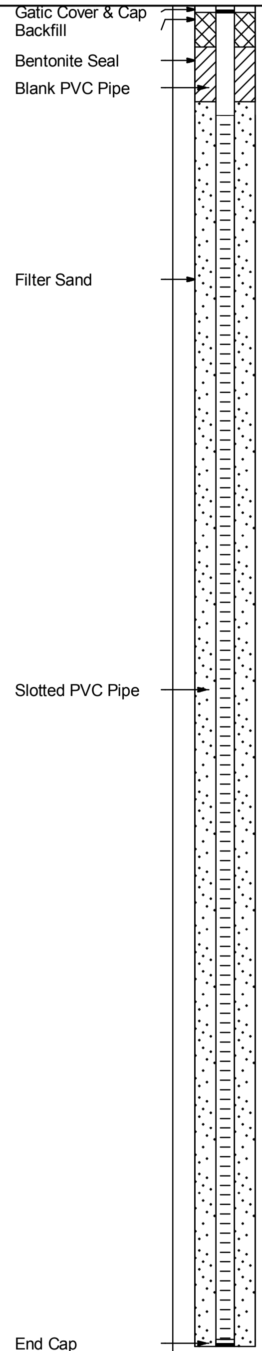
CLIENT: Tri-Anta Pty Ltd
PROJECT: Proposed Mixed Use Development
LOCATION: 19-27 Cross Street, Double Bay

SURFACE LEVEL: 3.25 AHD
EASTING: 337580
NORTHING: 6250176
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 86397.00
DATE:
SHEET 1 OF 1

Depth (m)	Description of Strata	Sampling Type	Sampling Depth	Results & Comments	Water
0.16	CONCRETE				
0.25	FILLING: yellow brown, fine to medium grained sand. Damp	A	0.2–0.25		
	SAND: medium dense, grey brown, fine to medium grained sand. Damp	A	0.5–0.6		
		A	0.9–1.0		
		A	1.4–1.5		
	2.0 m, moist	A	2.0–2.1		
	2.3 m, wet				18-06-18
	2.6 m, very loose to loose	A	2.5–2.6		
	3.04 m, medium dense to dense	A	3.0–3.1		
		A	4.0–4.1		
	4.88 m, medium dense				
		A	5.5–5.6		
		S	7.0–7.45	2,1,2 N = 3	
		S	8.5–8.95	1,2,2 N = 4	
	10.0 m, grey	S	10.0–10.45	2,0,0 N = 0	
10.45	10.1 m, very loose to loose				
	Bore discontinued at 10.45m				

Well Construction Details:



RIG: Bobcat **DRILLER:** GM **LOGGED:** JY **CASING:** HQ to 5.5m

TYPE OF BORING: Hand auger to 2.6m, SFA (110mm diam) to 5.5m, rotary drilling to 10.0m.

WATER OBSERVATIONS: Groundwater observed at 2.3m on 18/6/2018

REMARKS: Location coordinates are in MGA94 Zone 56. Well installed to 9.8m, Gatic cover/backfill to 0.3m, Bentonite to 0.7 then gravel. Screen 0.8-9.8m

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)

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix B

Results of Laboratory Testing

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
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customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 192237

Client Details

Client	Douglas Partners Pty Ltd
Attention	Konrad Schultz, Jeremie Young
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86397.00, 19-27 Cross St, Double Bay**
Number of Samples	11 Soil
Date samples received	22/05/2018
Date completed instructions received	22/05/2018

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	29/05/2018
Date of Issue	29/05/2018

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

sPOCAS field test						
Our Reference		192237-1	192237-2	192237-3	192237-4	192237-5
Your Reference	UNITS	BH1	BH1	BH1	BH1	BH2
Depth		0.3-0.4	0.7-0.8	0.9-1	1.4-1.5	1-1.1
Date Sampled		17/05/2018	17/05/2018	17/05/2018	17/05/2018	18/05/2018
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	22/05/2018	22/05/2018	22/05/2018	22/05/2018	22/05/2018
Date analysed	-	22/05/2018	22/05/2018	22/05/2018	22/05/2018	22/05/2018
pH_F (field pH test)*	pH Units	7.8	7.7	7.4	7.5	9.8
pH_{FOX} (field peroxide test)*	pH Units	7.6	5.0	6.0	6.7	10.1
Reaction Rate*	-	Moderate	Slight	Slight	Slight	Slight

sPOCAS field test						
Our Reference		192237-6	192237-7	192237-8	192237-9	192237-10
Your Reference	UNITS	BH3	BH3	BH3	BH3	BH3
Depth		0.2-0.25	0.5-0.6	0.9-1.0	1.4-1.5	2-2.1
Date Sampled		18/05/2018	18/05/2018	18/05/2018	18/05/2018	18/05/2018
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	22/05/2018	22/05/2018	22/05/2018	22/05/2018	22/05/2018
Date analysed	-	22/05/2018	22/05/2018	22/05/2018	22/05/2018	22/05/2018
pH_F (field pH test)*	pH Units	9.1	7.0	6.1	6.9	7.1
pH_{FOX} (field peroxide test)*	pH Units	8.8	2.9	3.6	5.4	3.6
Reaction Rate*	-	Moderate	Slight	Slight	Slight	Slight

sPOCAS field test		
Our Reference		192237-11
Your Reference	UNITS	BH3
Depth		2.5-2.6
Date Sampled		18/05/2018
Type of sample		Soil
Date prepared	-	22/05/2018
Date analysed	-	22/05/2018
pH_F (field pH test)*	pH Units	7.4
pH_{FOX} (field peroxide test)*	pH Units	3.7
Reaction Rate*	-	Slight

Method ID	Methodology Summary
Inorg-063	pH- measured using pH meter and electrode. Soil is oxidised with Hydrogen Peroxide or extracted with water. Based on section H, Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004. To ensure accurate results these tests are recommended to be done in the field as pH may change with time thus these results may not be representative of true field conditions.

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

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www.envirolab.com.au

CERTIFICATE OF ANALYSIS 192237-C

Client Details

Client	Douglas Partners Pty Ltd
Attention	Konrad Schultz, Jeremie Young
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86397.00, 19-27 Cross St, Double Bay**
Number of Samples	11 Soil
Date samples received	22/05/2018
Date completed instructions received	15/06/2018

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	22/06/2018
Date of Issue	21/06/2018

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

Chromium Suite			
Our Reference		192237-C-7	192237-C-10
Your Reference	UNITS	BH3	BH3
Depth		0.5-0.6	2-2.1
Date Sampled		18/05/2018	18/05/2018
Type of sample		Soil	Soil
Date prepared	-	20/06/2018	20/06/2018
Date analysed	-	20/06/2018	20/06/2018
pH_{kcl}	pH units	5.3	6.2
s-TAA pH 6.5	%w/w S	<0.01	<0.01
TAA pH 6.5	moles H^+/t	<5	<5
Chromium Reducible Sulfur	%w/w	<0.005	<0.005
a-Chromium Reducible Sulfur	moles H^+/t	<3	<3
S_{HCl}	%w/w S	<0.005	<0.005
S_{KCl}	%w/w S	<0.005	<0.005
S_{NAS}	%w/w S	<0.005	<0.005
ANC_{BT}	% $CaCO_3$	<0.05	<0.05
s-ANC_{BT}	%w/w S	<0.05	<0.05
s-Net Acidity	%w/w S	<0.005	<0.005
a-Net Acidity	moles H^+/t	<5	<5
Liming rate	kg $CaCO_3$/t	<0.75	<0.75
a-Net Acidity without ANCE	moles H^+/t	<5	<5
Liming rate without ANCE	kg $CaCO_3$/t	<0.75	<0.75
s-Net Acidity without ANCE	%w/w S	<0.005	<0.005

Method ID	Methodology Summary
Inorg-068	Chromium Reducible Sulfur - Hydrogen Sulfide is quantified by iodometric titration after distillation to determine potential acidity. Based on Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004.

QUALITY CONTROL: Chromium Suite					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			20/06/2018	[NT]	[NT]	[NT]	[NT]	20/06/2018	[NT]
Date analysed	-			20/06/2018	[NT]	[NT]	[NT]	[NT]	20/06/2018	[NT]
pH $_{kcl}$	pH units		Inorg-068	[NT]	[NT]	[NT]	[NT]	[NT]	94	[NT]
s-TAA pH 6.5	%w/w S	0.01	Inorg-068	<0.01	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
TAA pH 6.5	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	90	[NT]
Chromium Reducible Sulfur	%w/w	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Chromium Reducible Sulfur	moles H^+/t	3	Inorg-068	<3	[NT]	[NT]	[NT]	[NT]	101	[NT]
S_{HCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{KCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{NAS}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
ANC_{BT}	% $CaCO_3$	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-ANC_{BT}	%w/w S	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity without ANCE	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate without ANCE	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity without ANCE	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions	
NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions	
Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

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Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

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Measurement Uncertainty estimates are available for most tests upon request.

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customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 194329

Client Details

Client	Douglas Partners Pty Ltd
Attention	Konrad Schultz
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86397.00, Double Bay**
Number of Samples	12 SOIL
Date samples received	19/06/2018
Date completed instructions received	19/06/2018

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	26/06/2018
Date of Issue	22/06/2018

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

sPOCAS field test						
Our Reference		194329-1	194329-2	194329-3	194329-4	194329-5
Your Reference	UNITS	BH1	BH1	BH1	BH1	BH2
Depth		1.9-2	2.4-2.5	2.9-3	4.4-45	1.9-2
Type of sample		SOIL	SOIL	SOIL	SOIL	SOIL
Date prepared	-	21/06/2018	21/06/2018	21/06/2018	21/06/2018	21/06/2018
Date analysed	-	21/06/2018	21/06/2018	21/06/2018	21/06/2018	21/06/2018
pH_F (field pH test)*	pH Units	8.3	7.8	7.4	6.5	8.3
pH_{FOX} (field peroxide test)*	pH Units	6.4	5.6	3.0	1.6	5.2
Reaction Rate*	-	Slight	Slight	Slight	Moderate	Slight

sPOCAS field test						
Our Reference		194329-6	194329-7	194329-8	194329-9	194329-10
Your Reference	UNITS	BH2	BH2	BH2	BH3	BH3
Depth		2.4-2.5	3.9-4.0	5.4-5.5	3.0-3.1	4.0-4.1
Type of sample		SOIL	SOIL	SOIL	SOIL	SOIL
Date prepared	-	21/06/2018	21/06/2018	21/06/2018	21/06/2018	21/06/2018
Date analysed	-	21/06/2018	21/06/2018	21/06/2018	21/06/2018	21/06/2018
pH_F (field pH test)*	pH Units	7.9	7.6	7.2	7.3	7.2
pH_{FOX} (field peroxide test)*	pH Units	5.0	3.0	1.9	3.8	3.8
Reaction Rate*	-	Slight	Moderate	Moderate	Slight	Slight

sPOCAS field test			
Our Reference		194329-11	194329-12
Your Reference	UNITS	BH3	BH3
Depth		5.5-5.6	10.0-10.45
Type of sample		SOIL	SOIL
Date prepared	-	21/06/2018	21/06/2018
Date analysed	-	21/06/2018	21/06/2018
pH_F (field pH test)*	pH Units	7.1	6.9
pH_{FOX} (field peroxide test)*	pH Units	3.4	1.7
Reaction Rate*	-	Moderate	High

Method ID	Methodology Summary
Inorg-063	pH- measured using pH meter and electrode. Soil is oxidised with Hydrogen Peroxide or extracted with water. Based on section H, Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004. To ensure accurate results these tests are recommended to be done in the field as pH may change with time thus these results may not be representative of true field conditions.

Result Definitions

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CERTIFICATE OF ANALYSIS 194329-A

Client Details

Client	Douglas Partners Pty Ltd
Attention	Paul Gorman, Konrad Schultz, Jeremie Young
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86397.00, Double Bay**
Number of Samples	12 SOIL
Date samples received	19/06/2018
Date completed instructions received	25/06/2018

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	02/07/2018
Date of Issue	02/07/2018

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Chromium Suite			
Our Reference		194329-A-4	194329-A-8
Your Reference	UNITS	BH1	BH2
Depth		4.4-4.5	5.4-5.5
Type of sample		SOIL	SOIL
Date prepared	-	26/06/2018	26/06/2018
Date analysed	-	26/06/2018	26/06/2018
pH_{kcl}	pH units	5.1	5.7
s-TAA pH 6.5	%w/w S	<0.01	<0.01
TAA pH 6.5	moles H^+/t	<5	<5
Chromium Reducible Sulfur	%w/w	0.02	0.01
a-Chromium Reducible Sulfur	moles H^+/t	13	6
S_{HCl}	%w/w S	<0.005	<0.005
S_{KCl}	%w/w S	<0.005	<0.005
S_{NAS}	%w/w S	<0.005	<0.005
ANC_{BT}	% $CaCO_3$	<0.05	<0.05
s-ANC_{BT}	%w/w S	<0.05	<0.05
s-Net Acidity	%w/w S	0.028	0.010
a-Net Acidity	moles H^+/t	17	6.3
Liming rate	kg $CaCO_3$/t	1.3	<0.75
a-Net Acidity without ANCE	moles H^+/t	17	6.3
Liming rate without ANCE	kg $CaCO_3$/t	1.3	<0.75
s-Net Acidity without ANCE	%w/w S	0.028	0.010

Method ID	Methodology Summary
Inorg-068	Chromium Reducible Sulfur - Hydrogen Sulfide is quantified by iodometric titration after distillation to determine potential acidity. Based on Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004.

QUALITY CONTROL: Chromium Suite					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			26/06/2018	[NT]	[NT]	[NT]	[NT]	26/06/2018	[NT]
Date analysed	-			26/06/2018	[NT]	[NT]	[NT]	[NT]	26/06/2018	[NT]
pH $_{kcl}$	pH units		Inorg-068	[NT]	[NT]	[NT]	[NT]	[NT]	93	[NT]
s-TAA pH 6.5	%w/w S	0.01	Inorg-068	<0.01	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
TAA pH 6.5	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	85	[NT]
Chromium Reducible Sulfur	%w/w	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Chromium Reducible Sulfur	moles H^+/t	3	Inorg-068	<3	[NT]	[NT]	[NT]	[NT]	97	[NT]
S_{HCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{KCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{NAS}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
ANC_{BT}	% $CaCO_3$	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-ANC_{BT}	%w/w S	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity without ANCE	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate without ANCE	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity without ANCE	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
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