Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Commercial Development
2b - 6 Hassall Street, Parramatta

Prepared for
The Trustee for CHOF5 Hassall Street Trust

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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.

	Signature	Date
Author		27 March 2019
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Table of Contents

Report on Geotechnical Investigation
Proposed Commercial Development
2b - 6 Hassall Street, Parramatta

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed commercial development at 2b - 6 Hassall Street, Parramatta. The investigation was commissioned in an email dated 16 November 2018 by Thomas Lay of Solutions Consulting Australia, on behalf of Charter Hall Direct Property Management Limited and was undertaken in accordance with the email proposal by Douglas Partners Pty Ltd (DP) dated 16 November 2018.

It is understood that the proposed development will include the demolition of existing structures and construction of a 19-storey commercial building with one basement. The proposed building will cover the majority of the site area.

The geotechnical investigation included the drilling of three rock cored boreholes and the installation of one groundwater monitoring well. Four boreholes were drilled on the site previously for a Preliminary Site Contamination Investigation (PSI), Report 86451.00.R.001. One of the boreholes recovered rock core and two groundwater monitoring wells were installed.

Details of the current and previous field work are given in the report, together with comments on design and construction issues.

This report supersedes the previous geotechnical assessment report undertaken by DP (Report 86415.01.R.001.Rev1 dated 16 July 2018).

2. Site Description and Geology

The site comprises three adjacent land parcels at Parramatta (Nos. 2b, 4 and 6 Hassall Street), covers approximately 2650 m^2 (refer to Drawing 1 in Appendix A) and has a gentle fall to the south-east. The site is currently occupied by a two storey office building (No. 2b), a vacant block (No. 4) and a three storey residential apartment building (No. 6).

The site is bound by Hassall Street to the south, the NSW Police headquarters to the east and north-east, Lancer Barracks to the north and the Commercial Hotel to the west.

Clay Cliff Creek is located approximately 120 m to the south-east of the site and is contained in a concrete culvert. The base of the creek is at approximately RL 7 m, relative to Australian Height Datum (AHD).

As shown on Figure 1, the Sydney 1:100 000 Geology Sheet indicates that the site is underlain by Ashfield Shale which comprises black to dark-grey shale and laminite. Stream and alluvial sediment related to Clay Cliff Creek are located approximately 20 m to the south-east.

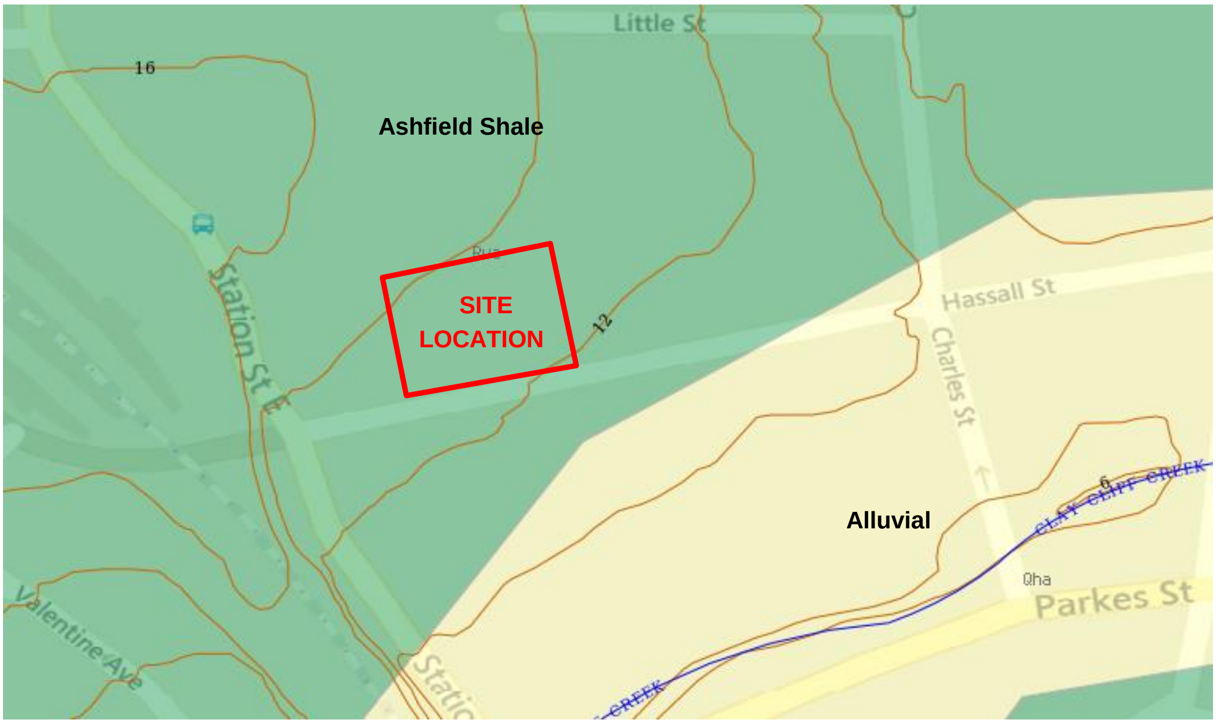


Figure 1: Geology Sheet

As shown on Figure 2, the Sydney 1:100 000 Soils Landscape Sheet indicates that the site underlain by the 'Blacktown' soil landscape. Blacktown soils typically consist of silty clay to about 1.0-1.5 m depth over mottled clays derived from the underlying shale. Alluvial soils ('Birrong' soil landscape) related to Clay Cliff Creek are located approximately 30 m to the south-east of the site.

Figure 2: Soil Sheet

3. Previous Investigations

DP has previously prepared the following reports for this project on this site:

- Preliminary Site Investigation for Contamination – 86415.00.R.001.Rev1 dated 17 July 2017
- Geotechnical Assessment – 86415.01.R.001.Rev1 dated 16 July 2017
- Detailed Site Contamination Investigation – 86415.02.R.001.Rev1 dated 14 November 2018

The cored boreholes and groundwater monitoring wells from the above investigations have been included in this report.

In addition, DP has undertaken geotechnical investigations on neighbouring properties to the east, south-east and south of the site. The locations of boreholes from these previous investigations are shown on Drawing 1 in Appendix B, with some of the boreholes being used in forming the geotechnical model for the site as described in Section 7.

4. Field Work Methods

The field work for the current investigation included three boreholes (BH101, BH102 and BH103) which were drilled using a truck and track-mounted rigs. The borehole locations are shown on Drawing 1 in Appendix B.

The boreholes were drilled into weathered rock to depths of 1.2 m to 1.6 m using solid flight augers and rotary drilling methods, and then continued to depths of 15.8 m to 16 m using diamond core drilling equipment to obtain continuous core samples of the bedrock.

The boreholes were logged and sampled by a geotechnical engineer. The rock cores recovered from the boreholes were photographed, followed by Point Load Strength Index (Is_{50}) testing on selected samples.

A groundwater monitoring well was installed in BH101 to 16 m depth. Water levels were measured in the well installed in BH101 along with the wells previously installed in BH1 and BH3.

Rising head permeability tests were conducted in the three wells. For each test, water within the well was pumped out and then the rise in the water level was measured at regular intervals using an automatic logger as the water level recharged. For BH1 the well was effectively dry and no water could be removed, thus no permeability test was undertaken.

The coordinates and ground surface levels at each borehole location were surveyed using a differential GPS, accurate to 0.1 m.

5. Field Work Results

Details of the subsurface conditions encountered in the current investigation are given in the borehole logs in Appendix C, together with colour photographs of the rock core and notes defining classification methods and descriptive terms.

5.1 Boreholes

The sequence of subsurface materials encountered within the boreholes, in increasing depth order, may be summarised as follows:

Pavement, Filling:	Asphalt and roadbase in BH1 and BH101 to depths of 0.2 – 0.4 m. Filling comprising silty clay with some sandstone gravel and tile fragments to a maximum depth of 0.6 m; over
Clay:	Stiff to very stiff clay, generally increasing to a hard shaly clay with depth; over
Shale, Siltstone and Laminite:	Extremely low and very low strength shale below about 0.9 – 1.2 m depth, becoming low strength below about 3.6 – 5.5 m depth. Medium strength siltstone below about 5.5 – 7.5 m depth. Some very high strength bands were encountered in BH101 and BH103. Below 13 – 14 m depth high strength laminite was encountered.

No free groundwater was observed during augering of the boreholes to maximum depths of 1.5 m and the introduction of water during the drilling process precluded the measurement of groundwater below this depth.

Depths to groundwater measured in the monitoring wells are shown in Table 1.

Table 1: Measured Groundwater Levels

Date	Event	Depth to Groundwater, m *(RL, m AHD)*		
		BH1	BH3	BH101
12/6/18	Before development	5.96 *(6.4)*	7.75 *(5.1)*	-
18/6/18	PSI sampling	7.76 *(4.6)*	8.81 *(4.0)*	-
22/11/18	Before development (new well)	-	-	14.20 *(-1.1)*
20/11/18	Before permeability testing	7.78* *(4.6)*	8.82 *(4.0)*	15.20 *(-2.1)*
30/11/18	After permeability testing	7.72* *(4.7)*	9.13 *(3.7)*	14.62 *(-1.5)*

Note: * Well effectively dry as the bottom ~300 mm of water in the well could not be extracted to allow for permeability testing

Maximum and minimum water levels measured in the monitoring wells are shown on the interpreted geotechnical cross section on Drawing 2 in Appendix B.

5.2 Permeability Test

Rising head permeability tests were carried out in the monitoring wells in BH3 and BH101. The results of the test are included in Appendix C.

Monitoring well BH3 was pumped to 9.34 m depth and the rise in water level then measured at regular intervals using an automatic data logger. The water level recharged to 8.99 m depth after 200 minutes. The hydraulic conductivity was assessed to be 1.9×10^{-7} m/s.

Monitoring well BH101 was pumped to 15.99 m depth and the rise in water level then measured at regular intervals using an automatic data logger. The water level recharged to 14.43 m depth after 200 minutes. The hydraulic conductivity was assessed to be 1.2×10^{-7} m/s.

6. Laboratory Testing

Four samples (one rock and three soil) were submitted for chemical analyses (pH, sulphate, chloride, electrical conductivity) to an external laboratory (Envirolab Services Pty Ltd), for assessment of soil aggressivity to buried structural elements (e.g. concrete and steel). The results of the chemical analyses are summarised in Table 2 and detailed results are presented in Appendix E.

Table 2: Summary of Laboratory Chemical Analysis Results

Borehole No. (Sample Depth)	Description	pH	Sulphate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm.cm)
BH101 (0.5m)	Clay	5.4	70	<10	20,000
BH102 (1.0m)	Clay	4.9	130	20	12,000
BH103 (0.5m)	Clay	5.1	250	10	6,800
BH103 (4.0m)	Shale	6.4	56	23	16,000

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the borehole logs at the appropriate depth. The Is_{50} values for the rock ranged from 0.09 MPa to 2.3 MPa, indicating that the rock samples tested raged from very low to high strength. Three point load tests were conducted on very high strength bands in BH101 and BH103, with Is_{50} values of 5.1 MPa up to 7.8 MPa.

7. Geotechnical Model

A geotechnical cross-section (Section A-A') showing the interpreted subsurface profile is presented as Drawing 2 in Appendix B. This section shows the interpreted geotechnical divisions of underlying soil and rock. The interpreted boundaries shown on the section are accurate at the borehole locations only and layers shown diagrammatically on this drawing are inferred strata boundaries only. Reference should be made to the borehole logs for more detailed information and descriptions of the soil and rock.

Maximum and minimum water levels measured in the monitoring wells are also shown on the cross section. It is expected that the permanent groundwater table would be below the proposed bulk excavation, with levels similar to those encountered in the monitoring well in BH101. The water levels encountered in the wells installed in BH1 and BH3 are expected to be a result of seepage through the rock mass (these wells were installed to a shallower depth than the well in BH101). Seepage can be expected to flow across the soil-rock interface and through fractures within the rock mass.

It is important to note that some bands of very high strength, iron rich siltstone were identified in BH101 and BH103.

8. Proposed Development

It is understood that the proposed development will include the demolition of existing structures and construction of a 19-storey commercial building with one basement. The basement level is at RL 7.3 m AHD and is expected to require excavation up to about 4.5 m depth. The lift pit is understood to require excavation to RL 1.6 m, which is approximately 11 m below existing surface levels

9. Comments

9.1 Site Preparation and Earthworks

9.1.1 Excavation Conditions

It is expected that the basement will require the excavation of soils and extremely low to very low strength rock to average depths of 2 – 3 m and then medium to high strength, slightly fractured and unbroken shale, siltstone and laminite. Some thin bands of very high strength rock are also expected within the deeper rock profile.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment. It is anticipated that excavation of medium and high strength rock will require moderate to heavy ripping with a large bulldozer or hydraulic rock breakers in conjunction with heavy ripping. Some of the high to very high strength laminite will be effectively unrippable.

The excavation rate that can be achieved, particularly within the medium to high strength rock varies considerably and is dependent upon the degree of jointing in the rock, the rock strength, the type of machinery being used and the skill of the operator. It is suggested that bulk excavation tenderers be required to make their own assessment of the equipment required to carry out the work. Contractors may inspect the rock core samples at the DP office in West Ryde prior to submitting final tenders (rock cores are generally kept for 6 months after drilling unless longer holding times are requested).

9.1.2 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings and pavements that may be affected by the basement construction. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

9.1.3 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the recommendations presented in DP's DSI Report (86415.02.R.001.Rev1 dated 14 November 2018) and with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014).

9.1.4 Acid Sulphate Soils

Reference to regional mapping and the results of the boreholes indicate that the site is underlain by residual soils and potential acid sulphate soils (PASS) are not expected on this site.

9.2 Excavation Support

It is anticipated that the excavation will be cut vertically and will require temporary shoring during construction. Temporary batter slopes may be possible within the basement excavation footprint, e.g. for access ramps. Providing that the recommendations in this Section 9.2 are adopted along with good design and construction practices, adjacent property and/or infrastructure should not be adversely impacted by ground movement.

9.2.1 Batter Slopes

Suggested maximum temporary batter slopes for unsupported excavations up to a maximum height of 3 m are shown in Table 3. If surcharge loads are applied near the crest of the slope then further geotechnical review and probably flatter batters or stabilisation using rock bolts or soil nails may be required.

Table 3: Recommended Safe Batter Slopes for Exposed Material

Exposed Material	Maximum Temporary Batter Slope (H : V)
Clay: stiff to very stiff	1.5 : 1
Shale: extremely to very low strength	1 : 1
Shale/Siltstone: low and medium strength	0.5 : 1*

Notes: * Subject to assessment of jointing in rock and also shoring design to account for potential rock wedges

Any soil batter slopes that are exposed over the long term should be covered with mesh reinforced shotcrete which is pinned to the face with dowels. Drainage will need to be installed behind the shotcrete to intercept any seepage or groundwater. A minimum shotcrete thickness of 80 mm is recommended, unless stability issues dictate a greater thickness is required.

9.2.2 Retaining/Shoring Walls

Vertical excavations within the soils Shale/Siltstone will require both temporary and permanent lateral support during and after excavation. A bored soldier pile shoring wall with shotcrete infill panels would be suitable where there are no movement sensitive structures in close proximity to the excavation. Typically, soldier piles are spaced at approximately 2 m to 3 m centres, however, closer spaced piles may be required to reduce wall movements, or prevent collapse of infill materials, where pavements, structures or services are located in close proximity to the excavation.

Preferably, shoring piles should be founded at least 1.0 m below the base of the bulk excavation level (or any perimeter drainage trenches or footings) in order to provide lateral restraint at the base of the excavation and to avoid the risk of adversely inclined joints or wedges undermining the bases of the piles. Note that this may require the drilling of piles through thin bands of very high strength siltstone. The toe of piles that are terminated above bulk excavation level will need to be restrained with rock bolts or anchors.

It is anticipated that at least one row of anchors may be required to provide lateral restraint to shoring piles for the basement excavation. Shoring will need to be designed to support earth pressures and surcharge loads.

9.2.3 Earth Pressure Design

Design for lateral earth pressures may be based on the parameters given in Table 4. For situations where only minor lateral movements are acceptable, such as the support of sensitive structures or services, an increased pressure based on "at-rest" conditions should be adopted, depending on the level of restraint required. A uniform pressure of 10 kPa should be adopted for the support of medium strength or stronger rock between soldier piles and/or anchors to account for minor joint wedges that may become mobilised.

All surcharge loads should be allowed for in the shoring design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 1 m to 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to the stormwater drainage system.

Table 4: Recommended Design Parameters for Shoring Systems

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (K_a)	At Rest (K_o)		
Filling and Residual Clay	20	0.3	0.5	5	20
Shale: extremely to very low strength	21	0.2	0.3	10	25
Shale/Siltstone: low and medium strength	22	10 kPa uniform	10 kPa uniform	20	25

9.2.4 Rock Wedge Design

Steeply dipping (60° to 90°) joints were encountered in the rock core samples and the design of temporary and possibly permanent support will also need to consider the possibility that steeply dipping joints in the shale/siltstone will daylight near the base of the shoring wall leading to wedges of rock which need to be supported by the temporary and permanent retaining structures.

The support system would typically comprise anchors spaced at 2 m to 3 m centres over the rock face, preferably through the shoring piles. These anchors should have their bond lengths formed in rock behind a line projected up at 45 degrees from bulk excavation level (including services trenches and footing excavations).

As a guide, it is suggested that the anchor capacity (working load) of the support system should be designed for an anchor force per unit width of 4.2 H^2 (kN) where H is the height of the excavation in metres. This approximation of the anchor force required to support a 45 degree wedge is based on an anchor inclination of 10 degrees below horizontal, an average bulk weight of 22 kN/m^3, and friction angle of 25 degrees and cohesion of 0 kPa along the failure plane. Given that there is a very low probability that a joint would run the full length and height of the excavation it suggested that this aspect of the design may be carried out using a factor of safety of 1.0.

Should there be a requirement to increase the angle of installation of the anchor to steeper than 10 degrees (i.e. to reach stronger rock) then the anchor capacity would need to be increased as shown in Table 5.

Inspection of the cut faces during the excavation phase should be carried out by an experienced engineering geologist or geotechnical engineer to check the adequacy of the design. The mapping of all actual joints and faults will also allow the re-calculation of the horizontal force required to restrain the actual joint wedges present for final support design, for the permanent basement structure.

Table 5: Increased Capacity Requirement for Steeper Anchors

Angle of Installation (below horizontal)	Required Percentage Increase in Capacity
10°	0%
15°	5%
20°	14%
25°	22%
30°	27%

9.2.5 Passive Resistance

Passive resistance for piles founded below the base of the bulk excavation (including allowance for services or footings) may be based on the ultimate passive restraint values provided in Table 6. These ultimate values will need to incorporate a factor of safety to limit the wall movement that is required to mobilise the full passive resistance. The top 0.5 m of the socket should be ignored due to possible disturbance (e.g. over-excavation) and tolerance effects. The passive restraint adopted in the design must not exceed the shear capacity of the pile.

Table 6: Passive Resistance Values

Foundation Stratum	Ultimate Passive Pressure (kPa)
Shale/Siltstone: low strength	1,500
Siltstone/laminite: medium strength	3,000

9.2.6 Ground Anchors

The design of temporary and permanent ground anchors for the support of excavations and/or shoring systems may be carried out on the basis of the maximum bond stresses given in Table 7.

Table 7: Recommended Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Shale: extremely to very low strength	100	200
Shale/Siltstone: low and medium strength	200	400
Siltstone/Laminite: high strength	500	800

The parameters given in Table 7 assume that the drilled holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring, and "lift-off" tests should be carried out to confirm the anchor capacities. It is suggested that ground anchors should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load.

It is anticipated that the building will support the basement excavation over the long term and therefore the ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection including full column grouting and the use of an internal corrugated sheathing over the full length of the anchor. A detailed specification would need to be prepared for the installation and stressing of permanent anchors.

9.2.7 Excavation Induced Ground Movements

There is a possibility that horizontal movements due to stress relief will occur during the excavation works. Based on published literature, the lateral deflections for vertical excavations supported by shoring could be in the order of 0.05% to 0.1% of the excavation height, which corresponds with approximately 2 – 5 mm for a 4 – 5 m depth of excavation. Unsupported vertical excavations can be expected to experience lateral deflections of 0.1% to 0.2% of the excavation height.

It is unlikely to be practicable to provide restraint for the relatively high in-situ horizontal stresses associated with stress relief movements. Therefore it is recommended that appropriate allowance be made for movements of this order in construction and planning.

9.3 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of building structure (e.g. reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVi for human comfort.

Based on the experience of DP and with reference to AS2670, it is suggested that a maximum PPVi of 8 mm/s (applicable at the foundation level of existing adjacent buildings) be employed at this site for both architectural and human comfort considerations.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

9.4 Groundwater and Seepage

The water levels measured in BH1 and BH3 are likely a result of seepage through the rock mass and along the soil/rock interface and not a reflection of the regional water table. The levels measured in BH101 (at just below RL 0 m AHD) are expected to be the regional groundwater table.

Based on the water level monitoring undertaken in November 2018, the regional groundwater table is expected to be below the proposed lift pit bulk excavation (RL 1.6 m). However, groundwater seepage through defects in the rock mass can be expected and some pumping of water may be required during construction.

No significant groundwater changes are expected to occur from the proposed works that would adversely impact surrounding property and/or infrastructure.

During construction and in the long term, it is anticipated that seepage into the excavation should be readily controlled by perimeter drains connected to a "sump-and-pump" system. A drained basement will require permanent subfloor drainage below the basement floor slab to direct seepage to the stormwater drainage system. Consideration could be given to constructing the lift pit as a watertight ‘tanked’ structure to account for fluctuations in the regional groundwater table, particularly after periods of prolonged rainfall.

It is possible that iron oxides will precipitate from any seepage, possibly leading to a build-up of an iron-oxide sludge. Allowance for periodic cleaning of such sludge should be made in the long-term maintenance requirements.

Excavations for pile foundations may encounter minor seepage inflows and allowance should be made to ‘tremie’ pour/pump concrete to the base of the pile excavations.

9.5 Foundations

It is expected that bulk excavation level for the basement will be in low and possibly medium strength shale/siltstone and therefore the column loads can be supported on pad or strip footings. If large column spacings are proposed, then higher column loads could be supported on pile footings socketed into high strength laminite.

Recommended foundation design parameters for the various rock strata are presented in Table 8 and assume that the footing excavations are clean and free of loose debris and water.

Table 8: Recommended Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable		Maximum Ultimate		Young's Modulus E (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
Shale LS	2,000	150	5,000	300	300
Shale/Siltstone MS	3,500	300	10,000	400	500
Laminite HS	8,000	650	40,000	1000	2,000

Notes: LS = low strength, MS = medium strength, HS = high strength
Shaft adhesion applicable for the design of bored piers, uncased over rock socket length, where adequate sidewall cleanliness and roughness is achieved.

Foundations proportioned on the basis of the allowable bearing pressures in Table 8 would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

Pad footings designed for allowable end bearing pressures of greater than 3500 kPa will require spoon testing in 30% to 50% of footings to check for weak seams below the base of individual footings. If weak seams are encountered, the footings will either need to be deepened or the allowable bearing capacity reduced.

All footing excavations should be inspected by a geotechnical engineer prior to the placement of steel and concrete.

9.6 Soil Aggressivity

The laboratory test results for soil aggressivity (see Table 2) were compared with the exposure classifications outlined in Australian Standard AS 2159 – 2009 *Piling – Design and installation*. The results indicate that the soils are non-aggressive to buried steel elements and mildly aggressive to buried concrete elements.

9.7 Pavements / Slab on Grade

It is anticipated that the basement subgrade will generally include very low to medium strength shale, which will be suitable for the basement car park slab on grade. Car park live loads of between 5 – 10 kPa can expect settlement of less than 5 mm.

It is suggested that site preparation and placement of engineered filling for lightly loaded pavements and slabs on ground at basement level and at ground level should incorporate the following:

- stripping of unsuitable material (e.g. vegetation and organic topsoil). This material can be reused on site for the purpose of landscaping only. Reuse of material should also consider the contamination status of the soil and may require further assessment;
- rolling of the exposed subgrade (soil only) using an 8-tonne minimum deadweight smooth drum roller with the final pass (proof roll) inspected by a geotechnical engineer to detect any soft or heaving areas. Any soft spots detected during proof rolling would generally need to be stripped to a stiff base or maximum depth of 0.5 m and replaced with engineered filling;
- engineered filling for replacing soft spots or raising site levels should be placed in layers of 250 mm maximum loose thickness and compacted to a dry density ratio of between 98% and 102% relative to Standard compaction with moisture contents strictly within ±2% of standard optimum moisture content (OMC). The density ratio should be increased to between 100% and 102% relative to standard compaction within 300 mm of the subgrade surface. Ripped siltstone and sandstone on site should generally be suitable for re-use as engineered filling provided it has a maximum particle size of 70 mm and moisture content within 2% of OMC;
- density testing of each layer of filling should be undertaken in accordance with AS 3798-2007 “Guidelines for Earthworks for Commercial and Residential Developments” to verify that specified density ratios have been achieved.

9.8 Seismic Loading

In accordance with AS1170-2007 “Structural Design Actions, Part 4: Earthquake Actions in Australia” a hazard factor (Z) of 0.08 and a site subsoil Class B_e is considered to be appropriate for the site.

10. Limitations

Douglas Partners (DP) has prepared this report for this project at 2-6 Hassall Street, Parramatta in accordance with DP’s email proposal dated 16 November 2018 and acceptance received on 16 November 2018 by Thomas Lay of Solutions Consulting Australia, on behalf of Charter Hall Direct Property Management Limited. The work was carried out under DP’s Conditions of Engagement.

This report is provided for the exclusive use of Charter Hall Direct Property Management Limited for this project only and for the purposes as described in the report. It should not be used by third parties or relied upon for other projects or other sites. 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 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 the PSI and previous investigations near the site. 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 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 investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Reference should be made to DP's Detailed Contamination Site Investigation Report (86415.02.R.001.Rev1 dated 14 November 2018).

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

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

Copyright

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.

Site Anomalies

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

Information for Contractual Purposes

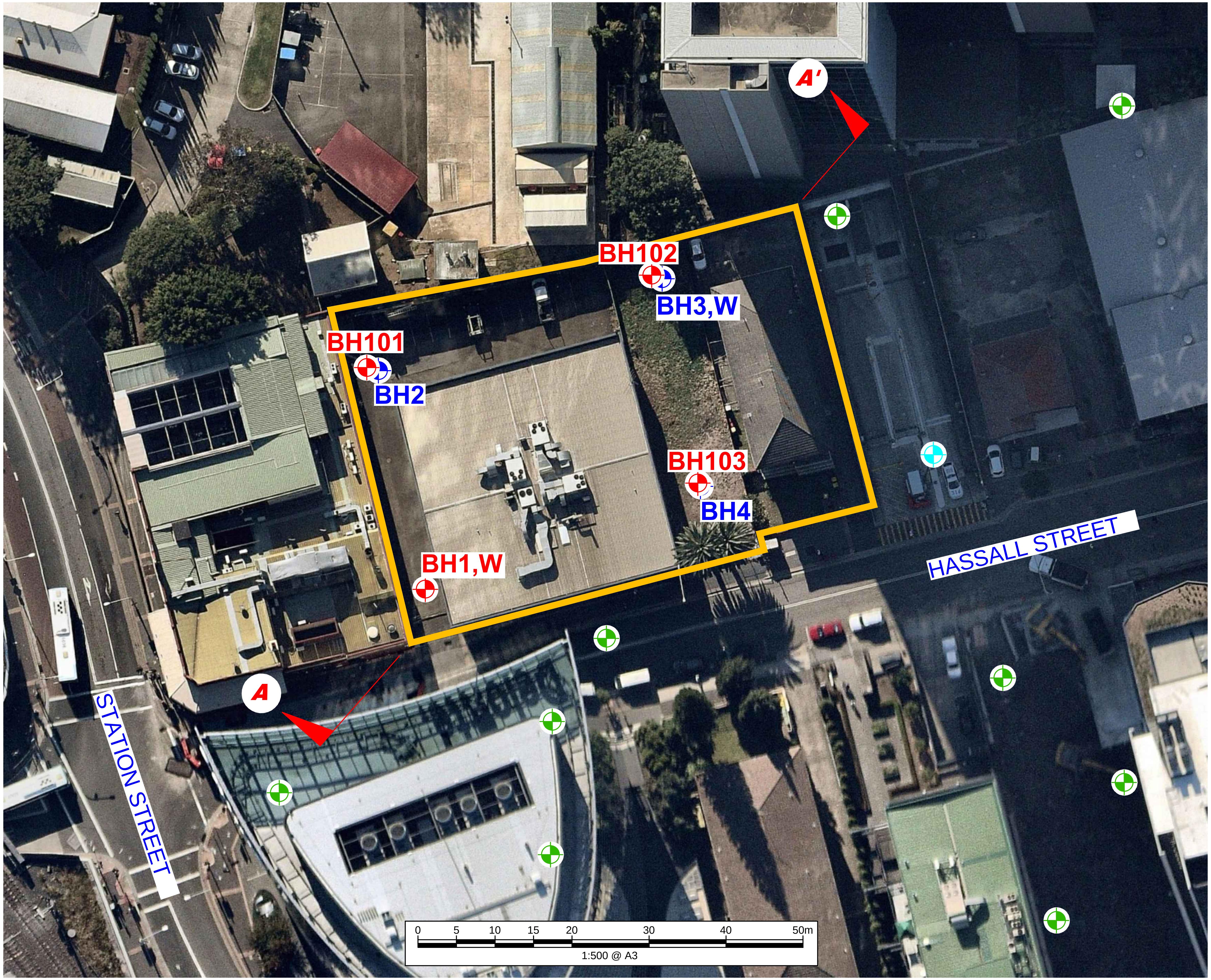
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

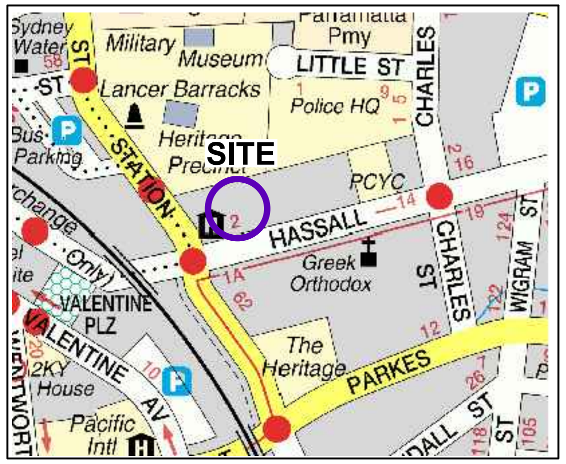
Site Inspection

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

Appendix B

Drawings 1 and 2





Locality Plan

LEGEND

- Current cored borehole
- Current augered borehole
- W Groundwater monitoring well
- Previous cored borehole
- Previous augered borehole
- Site boundary
- A A' Interpreted geotechnical cross section

NOTE:
1: Base image from Nearmap.com
(Dated 28.5.2018)
2: Test locations are approximate only and
are shown with reference to existing features.

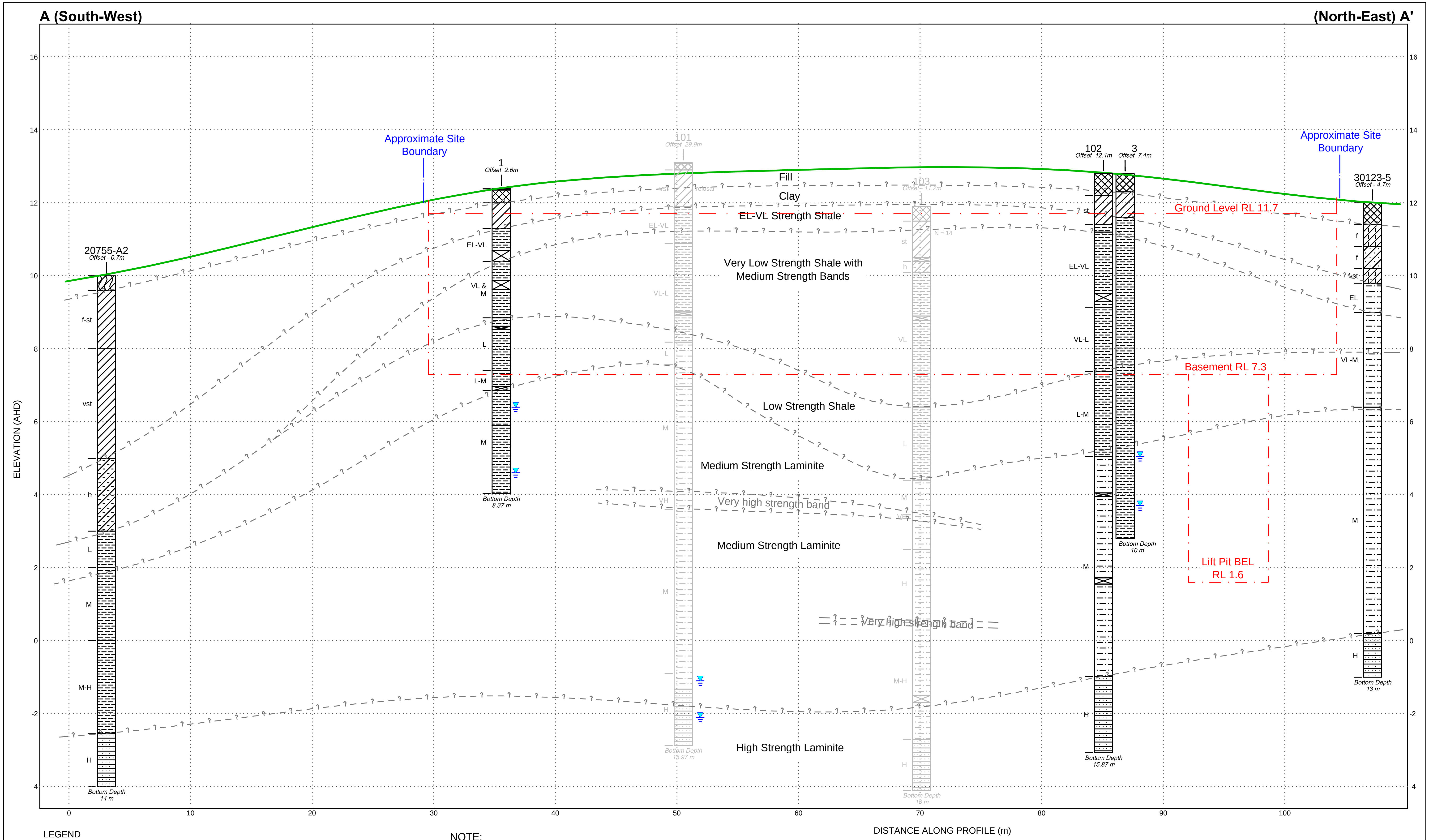
Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: The Trustee for CHOF5 Hassall Street Trust
OFFICE: Sydney
DRAWN BY: LJH
SCALE: 1:500 @ A3
DATE: 28.11.2018

TITLE: **Site Plan**
Geotechnical Investigation
2-6 Hassall Street, PARRAMATTA

N

PROJECT No: 86415.03
DRAWING No: 1
REVISION: 0



LEGEND

☒ Core Loss

■ Asphaltic Concrete

▨ Clay

☒ Filling

▤ Laminite

▨ Sandy Clay

▤ Shale

▨ Shaly Clay

▤ Siltstone

▨ Silty Clay

▨ Topsoil

NOTE:

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

ROCK STRENGTH
EL - Extremely low
VL - Very low
L - Low
M - Medium
H - High

SOIL STRENGTH/CONSISTENCY
f - Firm
st - Stiff
vst - Very stiff
h - Hard
l - Loose
md - Medium dense
d - Dense
vd - Very dense

TESTS / OTHER
N - Standard penetration test value
▽ - Water level
- ? - - Inferred Geological Boundary
-·-·- - Proposed Floor Levels

0 6
Horizontal Scale (metres)

Vertical Exaggeration = 3.0

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Geotechnics | Environment | Groundwater

CLIENT:	The Trustee for CHOF5 Hassall Street Trust
OFFICE: Sydney	DRAWN BY: LJH
SCALE: 1:300 (H) 1:100 (V) @ A3	DATE: 27.03.2019

TITLE: **Interpreted Geotechncial Cross Section A-A'**
Geotechnical Investigation
2-6 Hassall Street, PARRAMATTA

PROJECT No:	86415.03
DRAWING No:	2
REVISION:	1

Appendix C

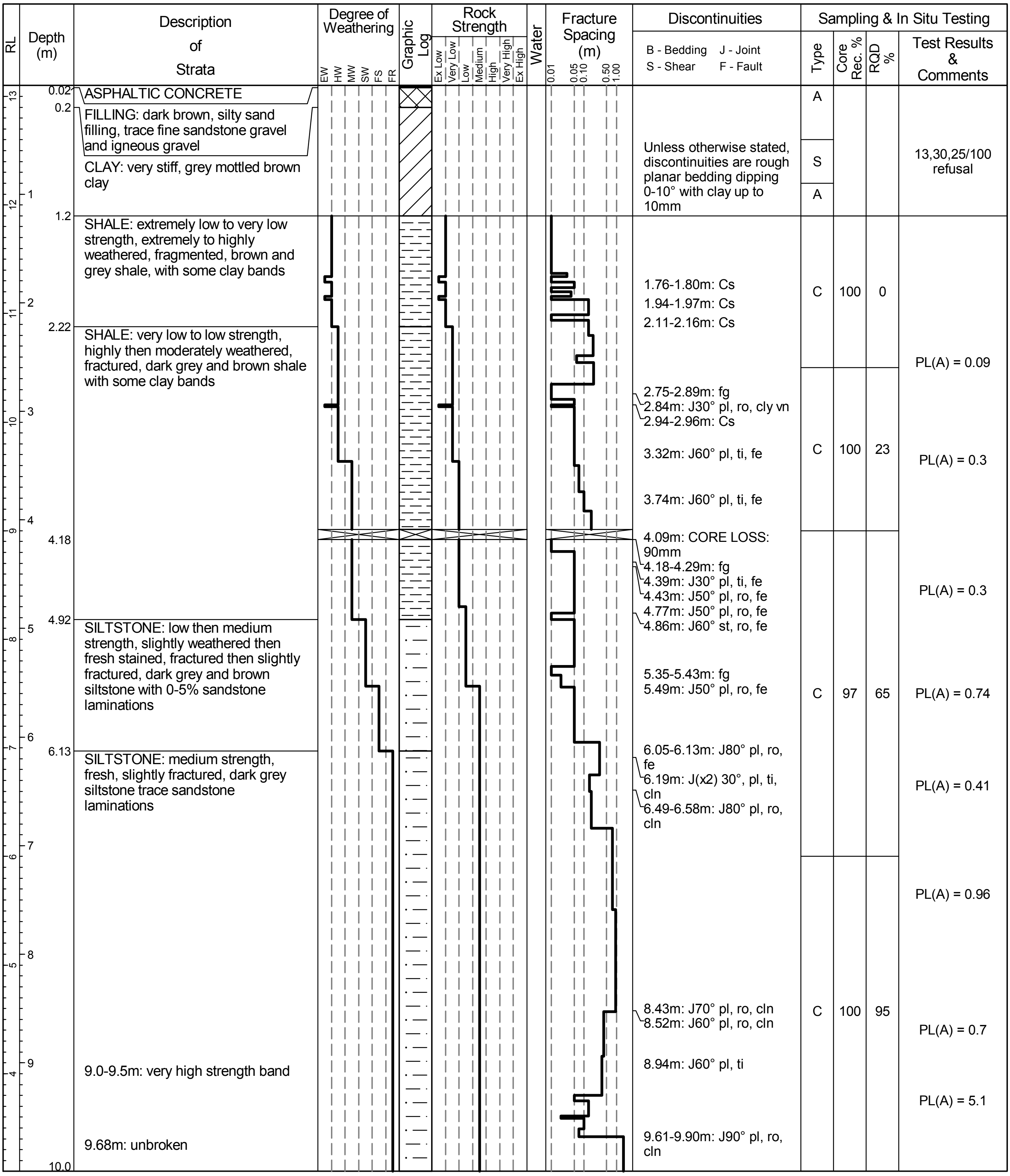
Borehole Logs

BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 13.1 AHD
EASTING: 315552
NORTHING: 6256255
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 86415.03
DATE: 22-11-2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALTIC CONCRETE		A			
0.2	FILLING: dark brown, silty sand filling, trace fine sandstone gravel and igneous gravel					
	CLAY: very stiff, grey mottled brown clay	Unless otherwise stated, discontinuities are rough planar bedding dipping 0-10° with clay up to 10mm	S			13,30,25/100 refusal
			A			
1.2	SHALE: extremely low to very low strength, extremely to highly weathered, fragmented, brown and grey shale, with some clay bands	1.76-1.80m: Cs 1.94-1.97m: Cs 2.11-2.16m: Cs	C	100	0	
2.22	SHALE: very low to low strength, highly then moderately weathered, fractured, dark grey and brown shale with some clay bands	2.75-2.89m: fg 2.84m: J30° pl, ro, cly vn 2.94-2.96m: Cs 3.32m: J60° pl, ti, fe 3.74m: J60° pl, ti, fe	C	100	23	PL(A) = 0.09 PL(A) = 0.3
4.18		4.09m: CORE LOSS: 90mm 4.18-4.29m: fg 4.39m: J30° pl, ti, fe 4.43m: J50° pl, ro, fe 4.77m: J50° pl, ro, fe 4.86m: J60° st, ro, fe	C	97	65	PL(A) = 0.3
4.92	SILTSTONE: low then medium strength, slightly weathered then fresh stained, fractured then slightly fractured, dark grey and brown siltstone with 0-5% sandstone laminations	5.35-5.43m: fg 5.49m: J50° pl, ro, fe				PL(A) = 0.74
6.13	SILTSTONE: medium strength, fresh, slightly fractured, dark grey siltstone trace sandstone laminations	6.05-6.13m: J80° pl, ro, fe 6.19m: J(x2) 30°, pl, ti, cln 6.49-6.58m: J80° pl, ro, cln				PL(A) = 0.41
						PL(A) = 0.96
		8.43m: J70° pl, ro, cln 8.52m: J60° pl, ro, cln	C	100	95	PL(A) = 0.7
	9.0-9.5m: very high strength band	8.94m: J60° pl, ti				PL(A) = 5.1
	9.68m: unbroken	9.61-9.90m: J90° pl, ro, cln				
10.0						

RIG: Geo 305 **DRILLER:** LC **LOGGED:** SLB **CASING:** HW to 1.2m

TYPE OF BORING: SFA (TC-Bit) to 0.8m, R to 1.2m, NMLC-coring to 16.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Well installed. Screen 4-16m. Blank 0-4m. Bentonite 2.0-3.5m.

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)

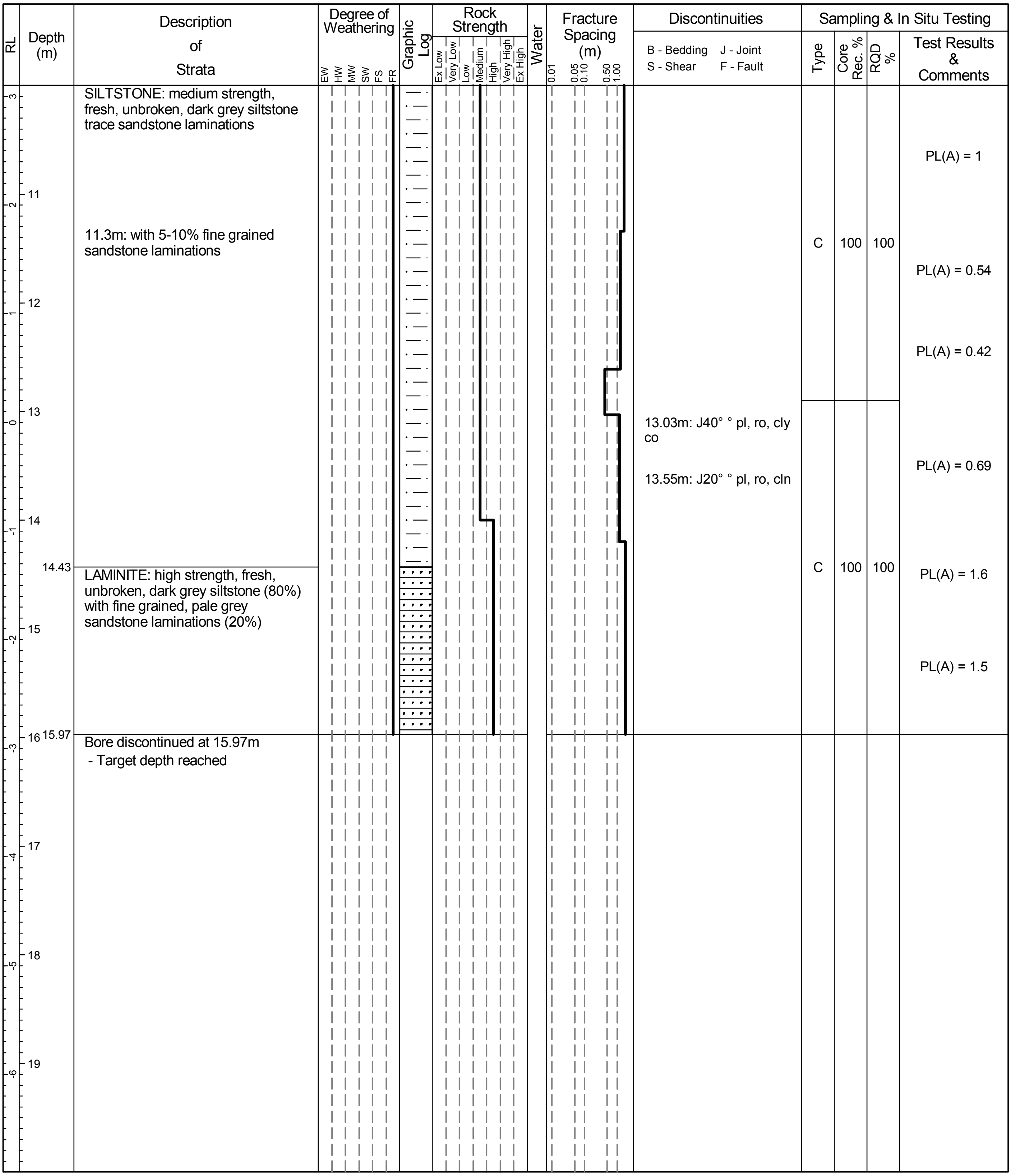
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 13.1 AHD
EASTING: 315552
NORTHING: 6256255
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 86415.03
DATE: 22-11-2018
SHEET 2 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	SILTSTONE: medium strength, fresh, unbroken, dark grey siltstone trace sandstone laminations					PL(A) = 1
	11.3m: with 5-10% fine grained sandstone laminations		C	100	100	PL(A) = 0.54
						PL(A) = 0.42
		13.03m: J40° ° pl, ro, cly co				
		13.55m: J20° ° pl, ro, cln				PL(A) = 0.69
14.43	LAMINITE: high strength, fresh, unbroken, dark grey siltstone (80%) with fine grained, pale grey sandstone laminations (20%)		C	100	100	PL(A) = 1.6
						PL(A) = 1.5
15.97	Bore discontinued at 15.97m - Target depth reached					

RIG: Geo 305 **DRILLER:** LC **LOGGED:** SLB **CASING:** HW to 1.2m

TYPE OF BORING: SFA (TC-Bit) to 0.8m, R to 1.2m, NMLC-coring to 16.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Well installed. Screen 4-16m. Blank 0-4m. Bentonite 2.0-3.5m.

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)

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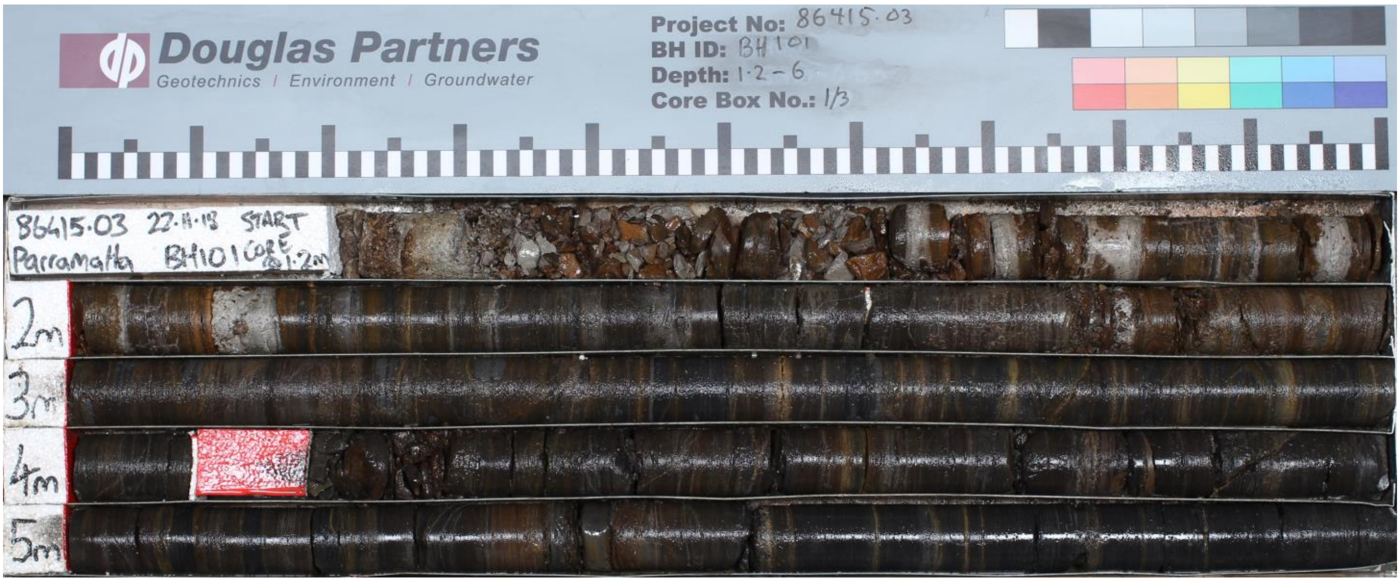
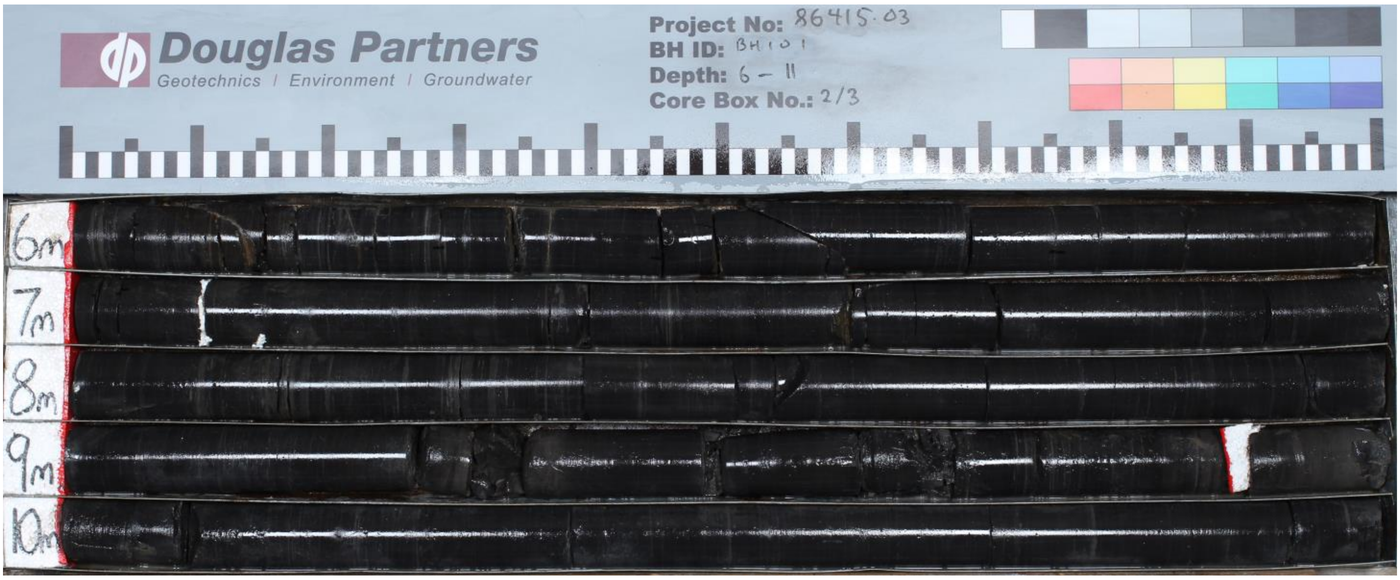

Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH101
Depth: 1.2 - 6
Core Box No.: 1/3
86415.03 22.11.18 START
Parramatta BH101 CORE @1.2m
2m
3m
4m
5m


Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH101
Depth: 6 - 11
Core Box No.: 2/3
6m
7m
8m
9m
10m

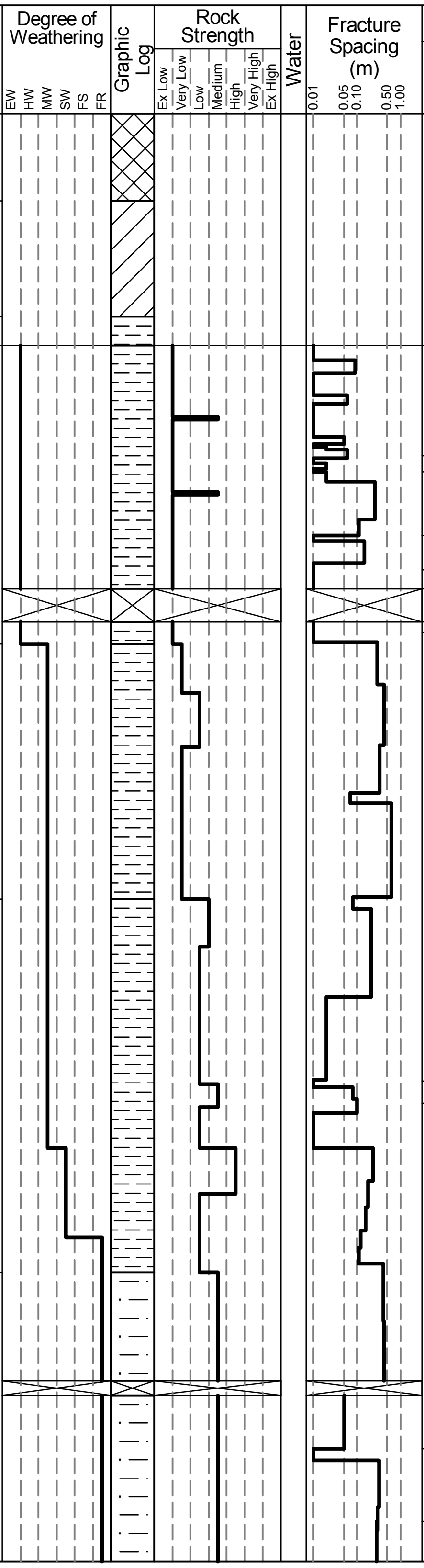

Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH101
Depth: 11 - 15.95
Core Box No.: 3/3
11m
12m
13m
14m
15m

BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 12.8 AHD
EASTING: 315589
NORTHING: 6256267
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 86415.03
DATE: 20-11-2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	FILLING: dark brown, silty clay filling with some sand, trace of sandstone gravel (~10mm) and rootlets, moist		A			
0.6	CLAY: stiff, orange mottled red and grey clay, damp	Unless otherwise stated, discontinuities are rough planar bedding dipping 0-10° with clay up to 10mm	A A S			4,6,12 N = 18
1.4	SHALE: extremely low to very low strength, grey and orange shale					
1.6	SHALE: extremely low to very low strength, extremely to highly weathered, fragmented, brown and grey shale with extremely low strength clay bands and some medium strength ironcemented bands	1.60-1.70m: fg 1.96-1.98m: Cs 2.32m: J45° pl, ro, fe 2.36-2.41m: Cs 2.47-2.49m: Cs 2.80-2.84m: J90, st, ro, cly co 2.91-2.95m: Cs 3.15-3.23m: fg 3.28m: CORE LOSS: 230mm	C	90	0	
3.51		3.51-3.58m: fg 3.58-3.64m: Cs				
3.66	SHALE: very low and low strength, moderately weathered, highly fractured, dark grey shale	3.94m: J30° pl, ro, cly co	C	97	30	PL(A) = 0.18 PL(A) = 0.58 PL(A) = 0.25
5.42	SHALE: low to medium strength, moderately weathered, fractured, dark grey shale	5.49m: J30 ° pl, ro, cly co 6.37m: J30° pl, ro, fe 6.55-6.76m: J60-90° un, ro, fe 6.68m: J40° pl, ro, fe 6.83m: J40° pl, ro, cly vn	C	100	85	PL(A) = 0.26 PL(A) = 0.48
8.0	SILTSTONE: medium strength, fresh, slightly fractured, dark grey shale with 0-5% sandstone laminations	8.75m: CORE LOSS: 100mm				PL(A) = 0.38
8.85		9.12-9.22m: J70-90° un, ro, fe 9.22-9.30m: fg 9.62m: J70° st, ro, fe 9.71-9.93m: J60-90° un,	C	100	96	PL(A) = 0.6

RIG: Geo 305 **DRILLER:** SS **LOGGED:** SLB **CASING:** HW to 1.0m
TYPE OF BORING: SFA (TC-Bit) to 1.0m, R to 1.6m, NMLC-coring to 16.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

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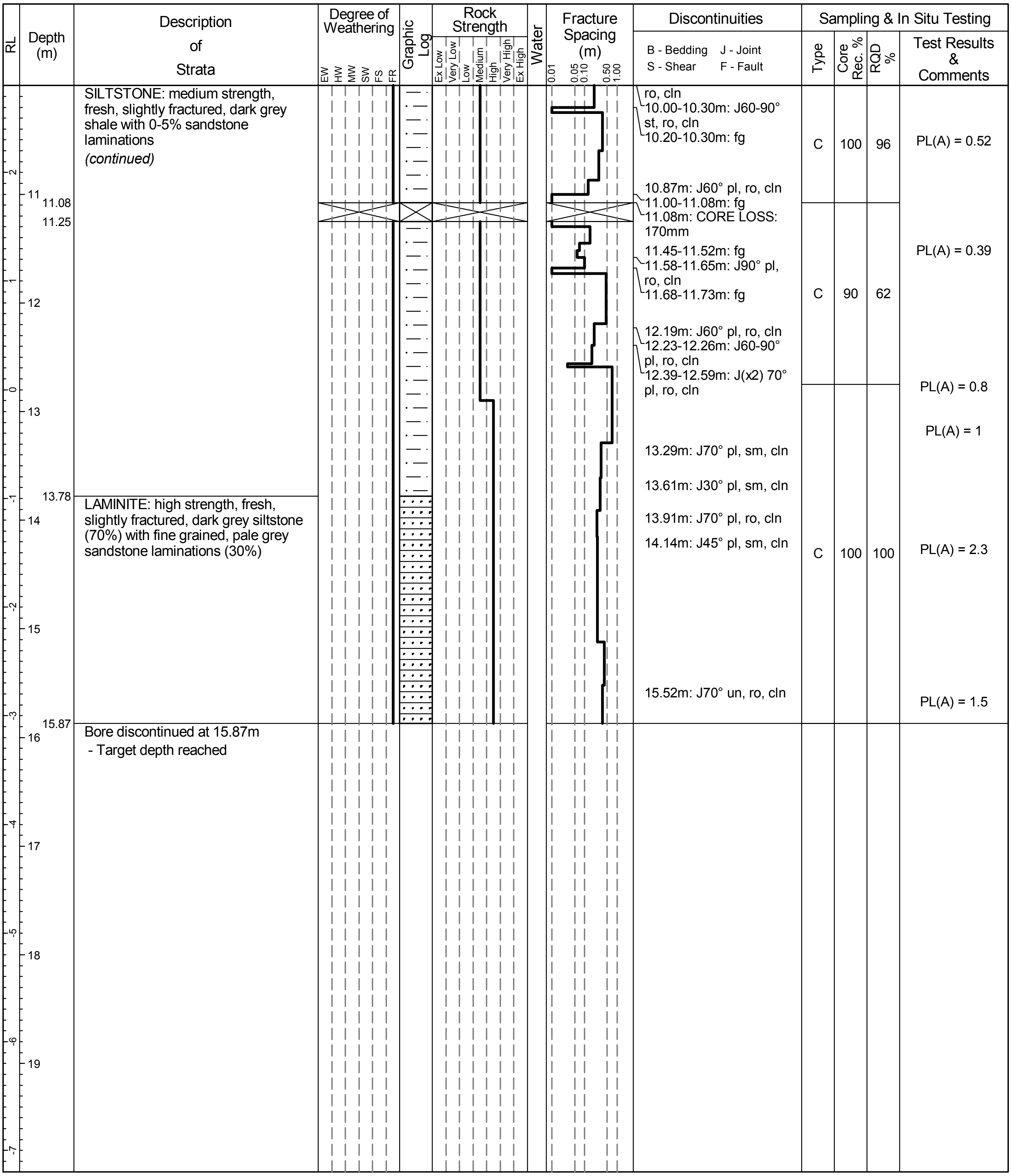
BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 12.8 AHD
EASTING: 315589
NORTHING: 6256267
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 86415.03
DATE: 20-11-2018
SHEET 2 **OF** 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	SILTSTONE: medium strength, fresh, slightly fractured, dark grey shale with 0-5% sandstone laminations *(continued)*	ro, cln 10.00-10.30m: J60-90° st, ro, cln 10.20-10.30m: fg 10.87m: J60° pl, ro, cln 11.00-11.08m: fg	C	100	96	PL(A) = 0.52
11.08		11.08m: CORE LOSS: 170mm				
11.25		11.45-11.52m: fg 11.58-11.65m: J90° pl, ro, cln 11.68-11.73m: fg 12.19m: J60° pl, ro, cln 12.23-12.26m: J60-90° pl, ro, cln 12.39-12.59m: J(x2) 70° pl, ro, cln	C	90	62	PL(A) = 0.39
		13.29m: J70° pl, sm, cln 13.61m: J30° pl, sm, cln	C	100	100	PL(A) = 0.8 PL(A) = 1
13.78	LAMINITE: high strength, fresh, slightly fractured, dark grey siltstone (70%) with fine grained, pale grey sandstone laminations (30%)	13.91m: J70° pl, ro, cln 14.14m: J45° pl, sm, cln 15.52m: J70° un, ro, cln				PL(A) = 2.3 PL(A) = 1.5
15.87	Bore discontinued at 15.87m - Target depth reached					

RIG: Geo 305 **DRILLER:** SS **LOGGED:** SLB **CASING:** HW to 1.0m
TYPE OF BORING: SFA (TC-Bit) to 1.0m, R to 1.6m, NMLC-coring to 16.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

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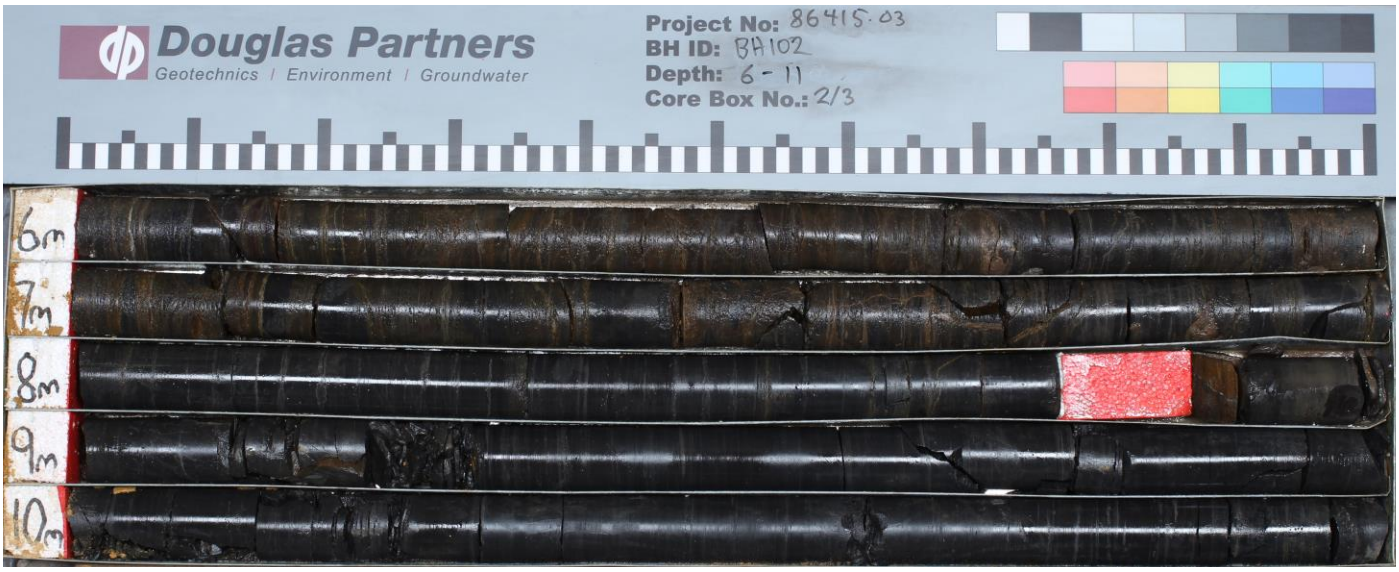
Project No: 86415.03
BH ID: BH102
Depth: 1.6-6.0
Core Box No.: 1/3

86415.03 20.11.18
Parramatta BH102 START CORE @ 1.60m


Project No: 86415.03
BH ID: BH102
Depth: 6-11
Core Box No.: 2/3

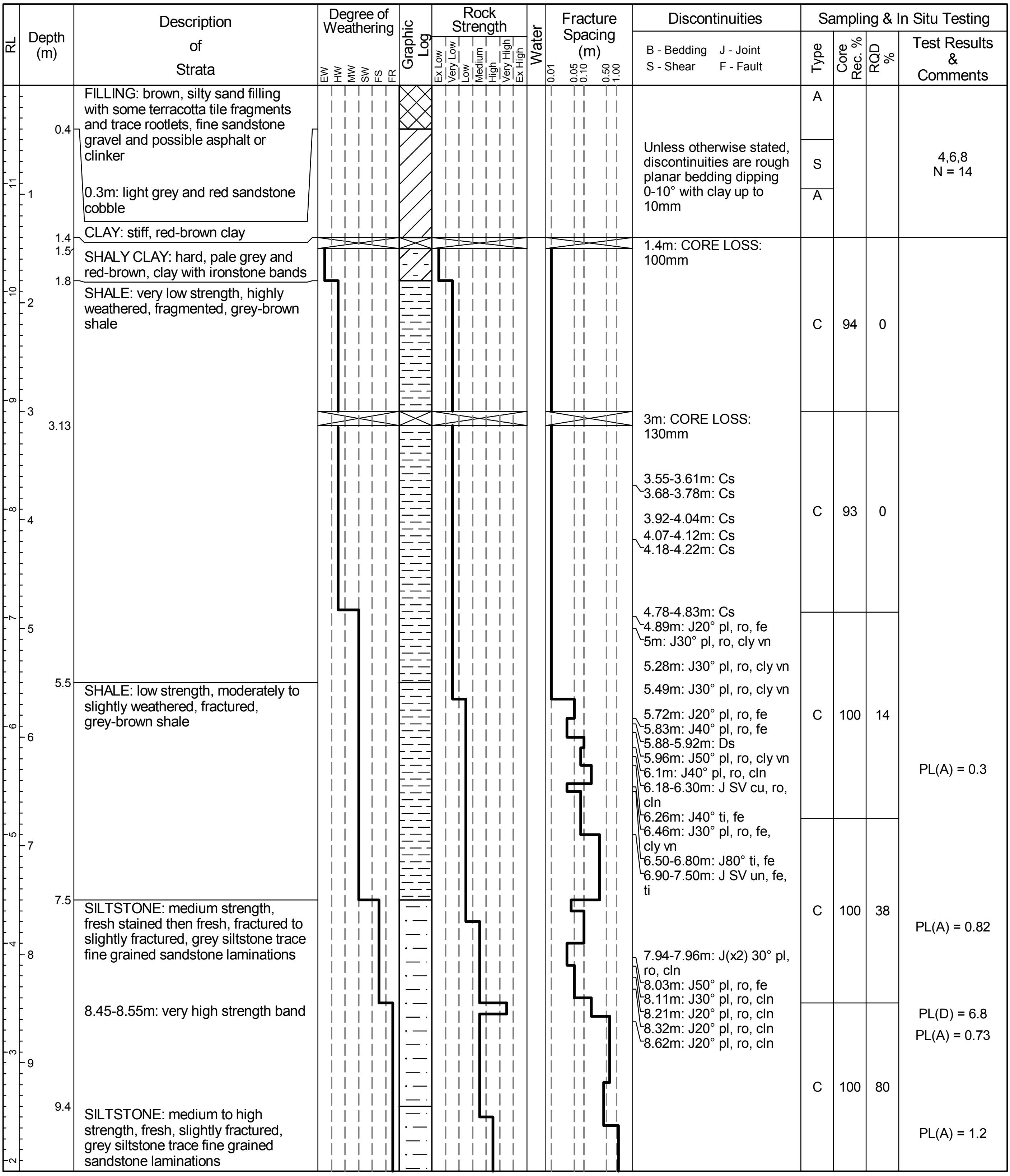

Project No: 86415.03
BH ID: BH102
Depth: 11-15.9
Core Box No.: 3/3

BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 11.9 AHD
EASTING: 315595
NORTHING: 6256240
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 86415.03
DATE: 21-11-2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	FILLING: brown, silty sand filling with some terracotta tile fragments and trace rootlets, fine sandstone gravel and possible asphalt or clinker	Unless otherwise stated, discontinuities are rough planar bedding dipping 0-10° with clay up to 10mm	A			
0.4	0.3m: light grey and red sandstone cobble		S			4,6,8 N = 14
			A			
1.4	CLAY: stiff, red-brown clay					
1.5	SHALY CLAY: hard, pale grey and red-brown, clay with ironstone bands	1.4m: CORE LOSS: 100mm	C	94	0	
1.8	SHALE: very low strength, highly weathered, fragmented, grey-brown shale					
3.13		3m: CORE LOSS: 130mm 3.55-3.61m: Cs 3.68-3.78m: Cs 3.92-4.04m: Cs 4.07-4.12m: Cs 4.18-4.22m: Cs	C	93	0	
		4.78-4.83m: Cs 4.89m: J20° pl, ro, fe 5m: J30° pl, ro, cly vn 5.28m: J30° pl, ro, cly vn 5.49m: J30° pl, ro, cly vn				
5.5	SHALE: low strength, moderately to slightly weathered, fractured, grey-brown shale	5.72m: J20° pl, ro, fe 5.83m: J40° pl, ro, fe 5.88-5.92m: Ds 5.96m: J50° pl, ro, cly vn 6.1m: J40° pl, ro, cln 6.18-6.30m: J SV cu, ro, cln 6.26m: J40° ti, fe	C	100	14	PL(A) = 0.3
		6.46m: J30° pl, ro, fe, cly vn 6.50-6.80m: J80° ti, fe 6.90-7.50m: J SV un, fe, ti				
7.5	SILTSTONE: medium strength, fresh stained then fresh, fractured to slightly fractured, grey siltstone trace fine grained sandstone laminations	7.94-7.96m: J(x2) 30° pl, ro, cln 8.03m: J50° pl, ro, fe 8.11m: J30° pl, ro, cln	C	100	38	PL(A) = 0.82
	8.45-8.55m: very high strength band	8.21m: J20° pl, ro, cln 8.32m: J20° pl, ro, cln 8.62m: J20° pl, ro, cln				PL(D) = 6.8 PL(A) = 0.73
			C	100	80	
9.4	SILTSTONE: medium to high strength, fresh, slightly fractured, grey siltstone trace fine grained sandstone laminations					PL(A) = 1.2

RIG: Geo 305 **DRILLER:** LC **LOGGED:** LJH **CASING:** HW to 1.2m

TYPE OF BORING: SFA (TC-Bit) to 1.0m, R to 1.4m, NMLC-coring to 16.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

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

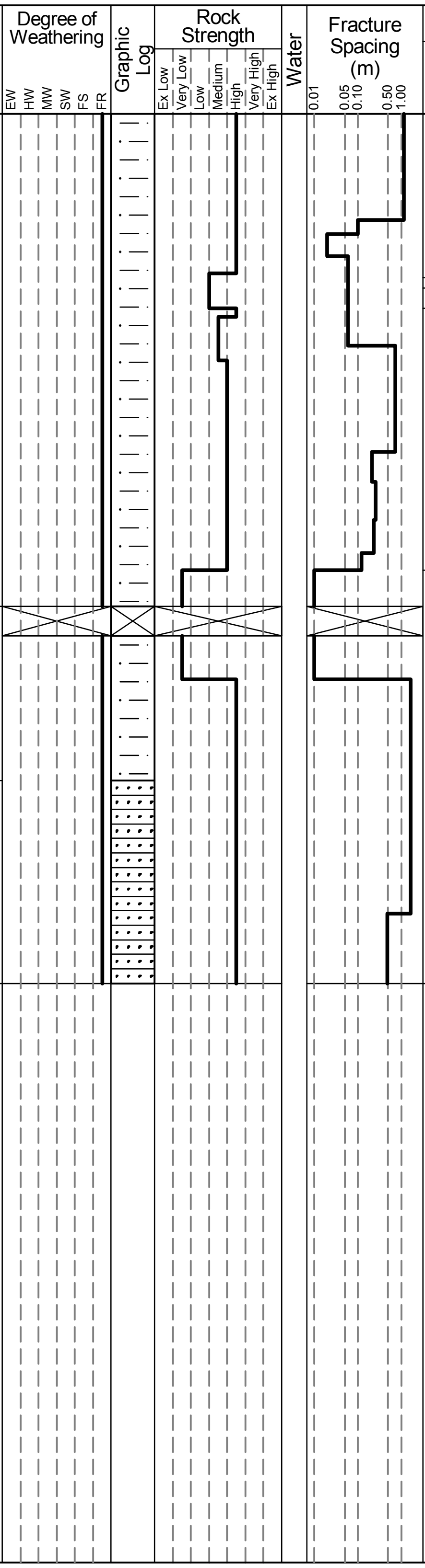
BOREHOLE LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 11.9 AHD
EASTING: 315595
NORTHING: 6256240
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 86415.03
DATE: 21-11-2018
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		SILTSTONE: medium to high strength, fresh, slightly fractured, grey siltstone trace fine grained sandstone laminations *(continued)*		C	100	80	PL(A) = 1.2
1	11	10.50m: with 5-10% fine grained sandstone laminations	10.83-10.98m: J(x7) 30-40° ti, 10-20mm spacing				
		11.34-11.40m: very high strength band	11.13m: J30° pl, ro, 1mm cly 11.20-11.24m: Ds 11.34-11.40m: J80° pl, ro, cln				PL(A) = 7.8
		11.60-13.10m: laminations at 20°					PL(A) = 1.1
0	12			C	91	67	PL(A) = 1
-1	13		12.8m: J20° st, ro, cln				
		13.15-13.90m: sheared zone	13.03m: J40° pl, ro, cln 13.15-13.90m: SZ				
	13.4 13.6		13.4m: CORE LOSS: 200mm				
-2	14						
	14.6	LAMINITE: high strength, fresh, slightly fractured, grey siltstone (80%) with pale grey, fine grained sandstone laminations (20%)		C	100	88	PL(A) = 1.2
-3	15		15.52m: J70° pl, ro, cln				PL(A) = 1.8
-4	16 16.0	Bore discontinued at 16.0m - Target depth reached					
-5	17						
-6	18						
-7	19						
-8							



RIG: Geo 305 **DRILLER:** LC **LOGGED:** LJH **CASING:** HW to 1.2m
TYPE OF BORING: SFA (TC-Bit) to 1.0m, R to 1.4m, NMLC-coring to 16.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

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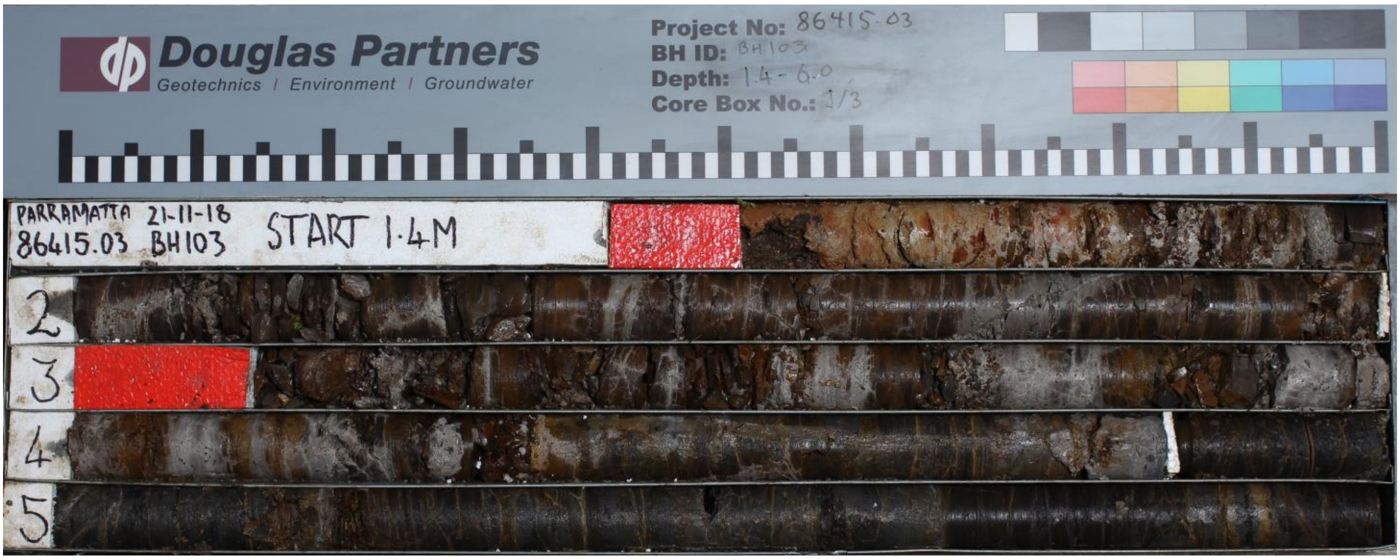

Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH103
Depth: 1.4-6.0
Core Box No.: 1/3
PARRAMATTA 21-11-18
86415.03 BH103
START 1.4M
2
3
4
5


Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH103
Depth: 6 – 11
Core Box No.: 2/3
6
7
8
9
10


Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.03
BH ID: BH103
Depth: 11– 16
Core Box No.: 3/3
11
12
13
14
15

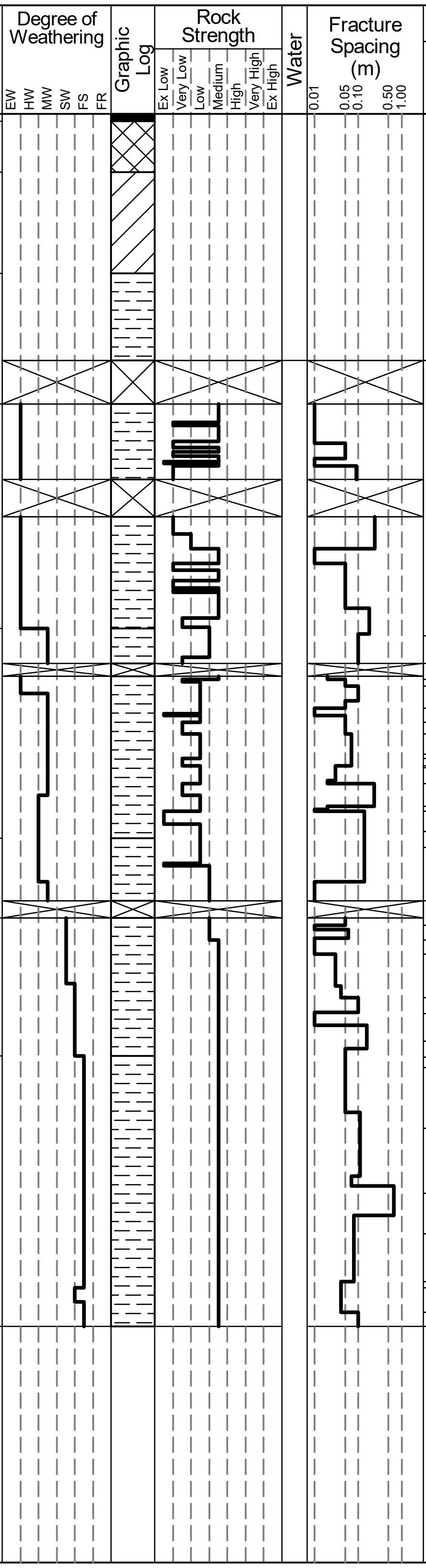
BOREHOLE LOG

CLIENT: Charter Hall Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: 2-6 Hassall Street, Parramatta

SURFACE LEVEL: 12.4 AHD
EASTING: 315560
NORTHING: 6256226
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 86415.00
DATE: 1/6/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.05	ASPHALTIC CONRETE
0.4	FILLING - grey, silty sand filling with some fine to medium sandstone gravel 0.3m: wet
1.1	CLAY - red mottled yellow-grey, clay, damp 0.9m: becoming shaley clay
2.0	SHALE: extremely low to very low strength, extremely to highly weathered, fragmented, grey and brown shale
2.78	SHALE: very low and medium strength banded, extremely to highly weathered, fragmented, pale grey and brown shale
3.55	
3.88	SHALE: low strength, highly to moderately weathered, fractured and fragmented, dark grey shale
5.0	SHALE: low to medium strength, highly to moderately weathered, fractured, dark grey shale
5.55	SHALE - medium strength, slightly weathered then fresh stained, fragmented then fractured, dark grey shale
6.5	SHALE: medium strength, fresh stained, slightly fractured, dark grey shale
8.37	Bore discontinued at 8.37m - Target depth reached



Discontinuities
B - Bedding J - Joint
S - Shear F - Fault

Note: Unless otherwise stated, rock is fractured along smooth, planar bedding dipping 0-10° with iron staining or clay coating
1.7m: CORE LOSS: 300mm
2.52m: CORE LOSS: 260mm
3.41m: J50° pl, ro, fe
3.59m: J20° pl, ro, cly 1mm
3.6m: J35° pl, ro, cly 3mm
3.79m: CORE LOSS: 90mm
3.90m: Ds 20mm
3.95-4.05m: J(x2) 40-50° pl, ro, fe
4.10-4.15m: J(x2) 15-30° pl, ro, cly 2mm
4.20m: Ds 80mm
4.28m: J15° pl, ro, fe
4.42m: J45°, st, ro, fe
4.45m: Ds 50mm
4.50m: J70° pl, ro, fe
4.51-4.60m: J(x5) 20°, pl, ro, fe
4.62m: Ds 80mm
4.78m: J70° pl, ro, fe
4.81m: Ds 90mm
4.95m: J15° pl, ro, cly 1mm
5.06-5.34m: J85°, pl, ro, fe
5.43m: CORE LOSS: 120mm
5.60-5.63m: J(x2) 25°, pl, ro, fe
5.70-5.79m: J(x2) 80°, pl, ro, fe
5.8m: J45° pl, ro, fe
6.10m: J(x2) 20° pl, ro, fe
6.21-6.29m: J(x5) 20-45° pl, ro, fe
6.40m: J40°, pl, ti, fe
6.51m: J45° pl, ro, fe
6.58m: J40°, pl, ro, fe
7.00-7.25m: J (x3) 60° pl, ro, cln
7.45m: J45°, st, ro, fe
7.73m: J50° pl, ro, fe
8.06m: J20°, pl, ro, fe
8.10-8.18m: J(x3) 60° pl, ro, fe
8.27m: J65°, pl, ro, fe

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A/E			PID <1ppm
A/E			PID <1ppm
A/E			PID <1ppm
A/E*			PID <1ppm
A/E			PID <1ppm
C	66	0	
C	79	0	PL(A) = 0.9
C	100	21	PL(A) = 0.2
C	94	17	PL(A) = 0.2 PL(A) = 0.7
C	100	60	PL(A) = 0.7 PL(A) = 0.4

RIG: Bobcat **DRILLER:** GM **LOGGED:** NW / LS **CASING:** HQ to 1.5m
TYPE OF BORING: SFA (TC-bit) to 1.5m; NMLC Coring to 8.37m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *BD1/20180601 taken at 0.9-1.0m
Well installed. Screen 1.5-8.37m. Blank 0.0-1.5m. Gravel Bentonite 1.0-1.5m.

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)

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Project No: 86415.00
BH ID: BH1
Depth: 1.7 - 6.0m
Core Box No.: 1 of 2
PARRAMATTA 86415 BH1 01-06-18 1of2 Boxes START 1.7m
CORELOSS 0.30m
2
Core Loss 0.25m
3
Coreloss 0.07m
4
5
CORELOSS 0.11m


Douglas Partners
Geotechnics / Environment / Groundwater
Project No: 86415.00
BH ID: BH1
Depth: 6.00 - 8.37m
Core Box No.: 2 of 2
6
7
8
END 8.37m

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

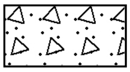
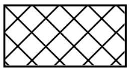
Other

fg	fragmented
bnd	band
qtz	quartz

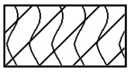
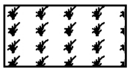
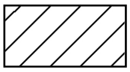
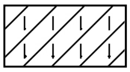
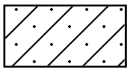
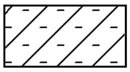
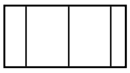
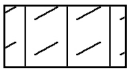
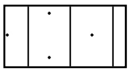
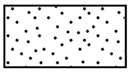
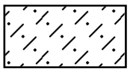
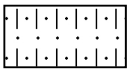
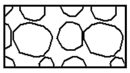
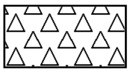
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

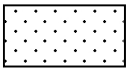
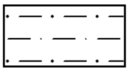
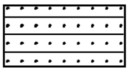
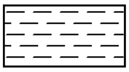
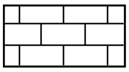
General

■	Asphalt
	Road base
	Concrete
	Filling

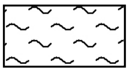
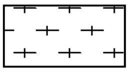
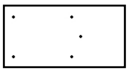
Soils

	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

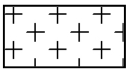
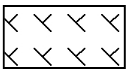
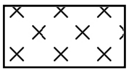
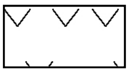
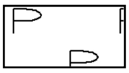
Sedimentary Rocks

	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

Soil Descriptions

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.

Sampling Methods

Douglas Partners

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.

Rock Descriptions

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:

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

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

Appendix D

Well Logs and Permeability Test Results

WELL LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 13.1 AHD
EASTING: 315552
NORTHING: 6256255
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 86415.03
DATE: 22-11-2018
SHEET 1 OF 1

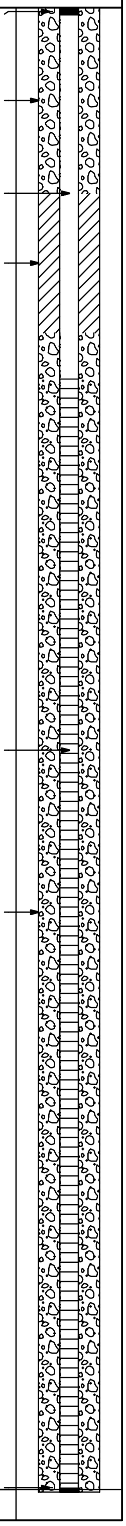
RL	Depth (m)	Description of Strata
13	0.02	ASPHALTIC CONCRETE
	0.2	FILLING: dark brown, silty sand filling, trace fine sandstone gravel and igneous gravel
12	1.2	CLAY: very stiff, grey mottled brown clay
		SHALE: extremely low to very low strength, extremely to highly weathered, fragmented, brown and grey shale, with some clay bands
11	2.22	SHALE: very low to low strength, highly then moderately weathered, fractured, dark grey and brown shale with some clay bands
9	4.18	
8	4.92	SILTSTONE: low then medium strength, slightly weathered then fresh stained, fractured then slightly fractured, dark grey and brown siltstone with 0-5% sandstone laminations
7	6.13	SILTSTONE: medium strength, fresh, slightly fractured, dark grey siltstone trace sandstone laminations 9.0-9.5m: very high strength band 9.68m: unbroken
3	10.0	SILTSTONE: medium strength, fresh, unbroken, dark grey siltstone trace sandstone laminations 11.3m: with 5-10% fine grained sandstone laminations
-1	14.43	LAMINITE: high strength, fresh, unbroken, dark grey siltstone (80%) with fine grained, pale grey sandstone laminations (20%)
-3	15.97	Bore discontinued at 15.97m - Target depth reached

RL scale: 13, 12, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1, 0, -1, -2, -3
Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16

Sampling & In Situ Testing

Type	Depth	Sample	Results & Comments	Water
A	0.1			
S	0.5 0.9		13,30,25/100 refusal	
A	1.0			
C	1.2 2.55		PL(A) = 0.09	
C	2.6 3.45 4.1		PL(A) = 0.3	
C	4.65 5.6 6.45 7.1		PL(A) = 0.3 PL(A) = 0.74 PL(A) = 0.41	
C	7.45 8.7 9.35 10.0		PL(A) = 0.96 PL(A) = 0.7 PL(A) = 5.1	
C	10.65 11.7 12.45 12.9		PL(A) = 1 PL(A) = 0.54 PL(A) = 0.42	
C	13.5 14.5 15.35 15.97		PL(A) = 0.69 PL(A) = 1.6 PL(A) = 1.5	

Well Construction Details



Depth	Well Construction Details
	Gatic cover
1	Backfill
2	Blank 0.0-4.0m
3	Bentonite plug 2.0-3.5m
8	Slotted PVC Screen 4.0-16.0m
10	Gravel 3.5-16.0m
16	End cap

RIG: Geo 305 **DRILLER:** LC **LOGGED:** SLB **CASING:** HW to 1.2m
TYPE OF BORING: SFA (TC-Bit) to 0.8m, R to 1.2m, NMLC-coring to 16.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Well installed. Screen 4-16m. Blank 0-4m. Bentonite 2.0-3.5m.

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)

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WELL LOG

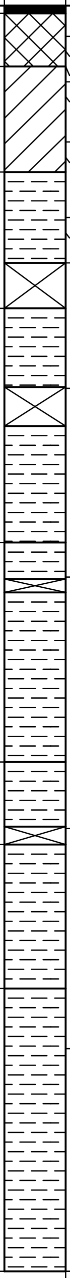
CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 12.4 AHD
EASTING: 315560
NORTHING: 6256226
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 86415.03
DATE: 1-6-2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.05	ASPHALTIC CONRETE
0.4	FILLING - grey, silty sand filling with some fine to medium sandstone gravel 0.3m: wet
	CLAY - red mottled yellow-grey, clay, damp 0.9m: becoming shaley clay
1.1	SHALE: extremely low to very low strength, extremely to highly weathered, fragmented, grey and brown shale
1.7	
2.0	SHALE: very low and medium strength banded, extremely to highly weathered, fragmented, pale grey and brown shale
2.52	
2.78	
3.55	SHALE: low strength, highly to moderately weathered, fractured and fragmented, dark grey shale
3.79	
3.88	
5.0	SHALE: low to medium strength, highly to moderately weathered, fractured, dark grey shale
5.43	
5.55	SHALE - medium strength, slightly weathered then fresh stained, fragmented then fractured, dark grey shale
6.5	SHALE: medium strength, fresh stained, slightly fractured, dark grey shale
8.37	Bore discontinued at 8.37m - Target depth reached

Graphic Log

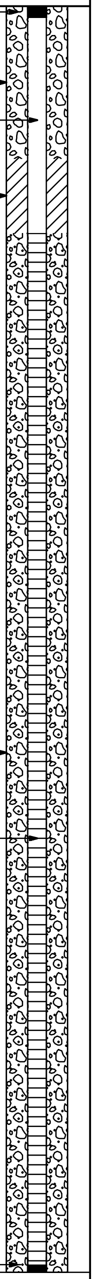


Sampling & In Situ Testing

Type	Depth	Sample	Results & Comments
A/E	0.2		PID <1ppm
A/E	0.3		PID <1ppm
	0.4		
A/E	0.5		PID <1ppm
	0.6		
A/E*	0.9		PID <1ppm
	1.0		
A/E	1.4		PID <1ppm
	1.5		
	1.7		
C			
	2.53		
C			
	3.37		PL(A) = 0.9
	3.78		
	4.06		PL(A) = 0.2
C			
	5.44		
	5.56		PL(A) = 0.2
C			
	6.34		PL(A) = 0.7
	6.9		
C	7.63		PL(A) = 0.7
	8.0		PL(A) = 0.4
	8.37		

Well Construction Details

Gatic cover
Backfill
Blank 0.0-1.5m
Bentonite plug 1.0-1.5m
Gravel 1.5-8.37m
Slotted PVC Screen 1.5-8.37m
End cap



RIG: Bobcat **DRILLER:** GM **LOGGED:** NW / LS **CASING:** HQ to 1.5m
TYPE OF BORING: SFA (TC-bit) to 1.5m; NMLC Coring to 8.37m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *BD1/20180601 taken at 0.9-1.0m
Well installed. Screen 1.5-8.37m. Blank 0.0-1.5m. Gravel Bentonite 1.0-1.5m.

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)

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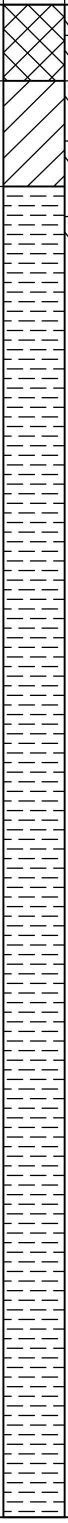
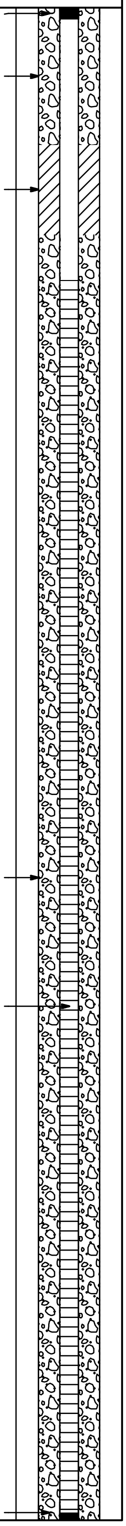
WELL LOG

CLIENT: The Trustee for CHOF5 Hassall Street Trust
PROJECT: Proposed Commercial Development
LOCATION: 2B-6 Hassal Street, Parramatta

SURFACE LEVEL: 12.8 AHD
EASTING: 315590
NORTHING: 6256267
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 86415.03
DATE: 8-6-2018
SHEET 1 OF 1





RL	Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water	Well Construction Details
		FILLING: dark brown, silty clay filling with some sand, sandstone gravel (10-20mm) and trace rootlets, charcoal and fragments of glass and terracotta.	A	0.0 0.1		PID < 1 ppm		Gatic cover
			A	0.2 0.3		PID = 9 ppm		Backfill
	0.5	0.2m: Slag and hydrocarbon odour 0.4m: with some orange clay	A*	0.5 0.6		PID < 1 ppm		
12	1	CLAY: orange mottled red-grey clay with trace charcoal	A	0.9 1.0		PID < 1 ppm		1
	1.2	SHALE: very low strength, extremely weathered, grey mottled orange shale 1.5m: becoming grey 1.7m: very low strength with medium strength bands (inferred from auger refusal and surrounding geology)	A	1.4 1.5		PID < 1 ppm		Bentonite plug 0.9-1.5m
11	2							2
10	3							3
9	4							4
8	5							5
		5.4m: medium strength (inferred from rock roller refusal and drill cuttings)						
7	6							Gravel/Sand 1.5-10.0m 6
								Slotted PVC Screen 1.8-10.0m
6	7							7
5	8							8
4	9							9
3	10.0							End cap

Bore discontinued at 10.0m - Target depth reached

RIG: Bobcat **DRILLER:** GM **LOGGED:** NW **CASING:** HQ to 4.6m

TYPE OF BORING: SFA (TC-Bit) to 1.7m; Rock Roller to 5.4m; Poly-chrystalline Diamond bit to 10.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: *BD1/20180608 taken at 0.5-0.6m. Auger refusal at 1.7m.
Well installed. Screen 1.8-10.0m. Blank 0.0-1.8m. Gravel Bentonite 0.9-1.5m.

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)

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Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising or Falling Head Test Report

Client: The Trustee for CHOF5 Hassall Street Trust
Project: Proposed Commercial Development
Location: 2b - 6 Hassall Street, Parramatta

Project No: 86415.03
Test date: 22-Nov-18
Tested by: LJH

Test Location
Description: Borehole
Material type: Shale

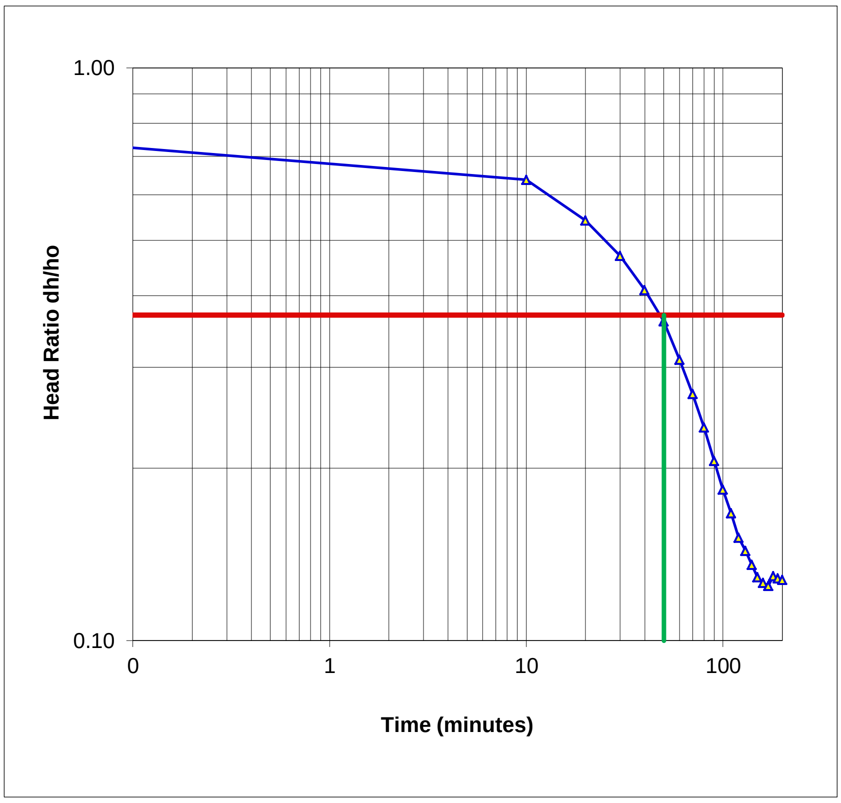
Test No. BH101
Easting: m
Northing m
Surface Level: m AHD

Details of Well Installation
Well casing diameter (2r) 76 mm
Well screen diameter (2R) 76 mm
Length of well screen (Le) 12 m

Depth to water before test 14.2 m
Depth to water at start of test 15.99 m

Test Results

Time (min)	Depth (m)	Change in Head: dH (m)	dH/Ho
0	15.99	1.79	1.000
10	15.34	1.14	0.638
20	15.17	0.97	0.542
30	15.04	0.84	0.470
40	14.93	0.73	0.409
50	14.85	0.65	0.361
60	14.75	0.55	0.309
70	14.68	0.48	0.269
80	14.62	0.42	0.236
90	14.57	0.37	0.206
100	14.53	0.33	0.184
110	14.50	0.30	0.167
120	14.47	0.27	0.151
130	14.46	0.26	0.144
140	14.44	0.24	0.136
150	14.43	0.23	0.129
160	14.43	0.23	0.126
170	14.42	0.22	0.125
180	14.43	0.23	0.130
190	14.43	0.23	0.128
200	14.43	0.23	0.128



To = 50 mins
3000 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity **k =** **1.2E-07** m/sec
= **0.042** cm/hour

Permeability Testing - Rising or Falling Head Test Report

Client:	The Trustee for CHOF5 Hassall Street Trust	Project No:	86415.03
Project:	Proposed Commercial Development	Test date:	20-Nov-18
Location:	2b - 6 Hassall Street, Parramatta	Tested by:	SLB

Test Location

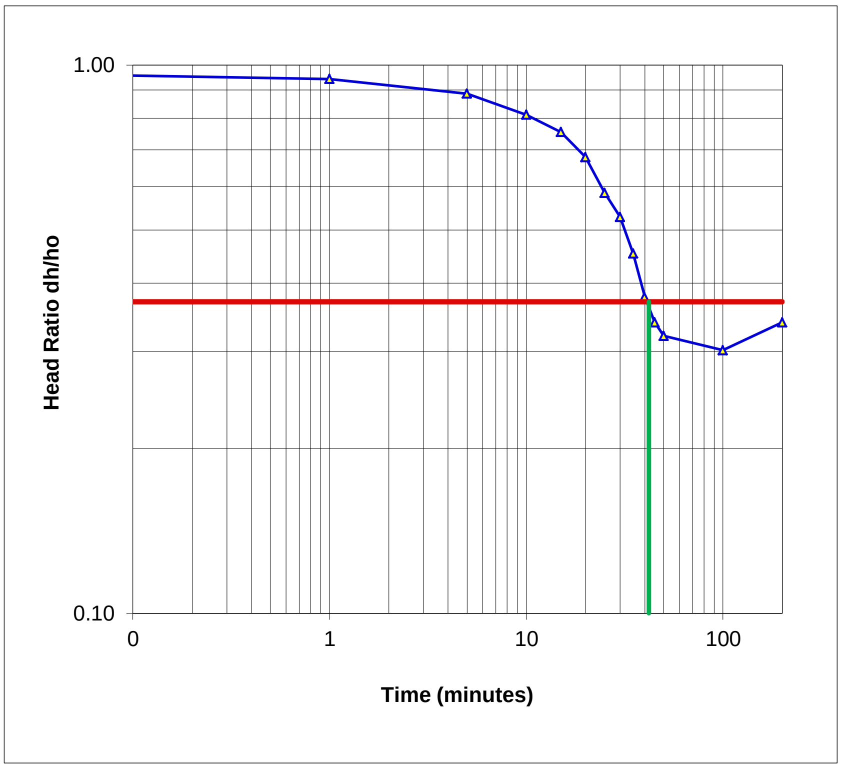
Description:	Borehole	**Test No.**	BH3	
Material type:	Shale	Easting:		m
		Northing		m
		Surface Level:		m AHD

Details of Well Installation

Well casing diameter (2r)	76	mm	Depth to water before test	8.81	m
Well screen diameter (2R)	76	mm	Depth to water at start of test	9.34	m
Length of well screen (Le)	8.2	m			

Test Results

Time (min)	Depth (m)	Change in Head: dH (m)	dH/Ho
0	9.34	0.53	1.000
1	9.31	0.50	0.943
5	9.28	0.47	0.887
10	9.24	0.43	0.811
15	9.21	0.40	0.755
20	9.17	0.36	0.679
25	9.12	0.31	0.585
30	9.09	0.28	0.528
35	9.05	0.24	0.453
40	9.01	0.20	0.377
45	8.99	0.18	0.340
50	8.98	0.17	0.321
100	8.97	0.16	0.302
200	8.99	0.18	0.340



To = 42 mins
2520 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**1.9E-07**	m/sec
	=	**0.068**	cm/hour

Appendix E

Laboratory Test Results

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 206901

Client Details

Client	Douglas Partners Pty Ltd
Attention	Luke James-Hall
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86415.03, Parramatta**
Number of Samples	4 SOIL
Date samples received	29/11/2018
Date completed instructions received	29/11/2018

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	06/12/2018
Date of Issue	04/12/2018

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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

Soil Aggressivity					
Our Reference		206901-1	206901-2	206901-3	206901-4
Your Reference	UNITS	BH101	BH102	BH103	BH103
Depth		0.5	1.0	0.5	4.0
Date Sampled		21/11/2018	21/11/2018	21/11/2018	21/11/2018
Type of sample		SOIL	SOIL	SOIL	SOIL
pH 1:5 soil:water	pH Units	5.4	4.9	5.1	6.4
Electrical Conductivity 1:5 soil:water	µS/cm	49	85	150	64
Resistivity by calculation	ohm m	200	120	68	160
Chloride, Cl 1:5 soil:water	mg/kg	<10	20	10	23
Sulphate, SO4 1:5 soil:water	mg/kg	70	130	250	56

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Soil Aggressivity						Duplicate			Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	5.4	5.5	2	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	1	49	45	9	99	[NT]
Resistivity by calculation	ohm m	0.1	Inorg-002	<0.1	1	200	220	10	[NT]	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	97	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	70	64	9	96	[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.

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: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

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.