GROUNDED
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Report on Previous Geotechnical Investigation

Proposed Redfern Place

600-660 Elizabeth Street, Redfern NSW

Prepared for Bridge Housing Ltd
C/-- Capella Capital

Project 99510.02

6 June 2024

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Report on Previous Geotechnical Investigation
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1. Introduction

This report prepared by Douglas Partners Pty Ltd (Douglas) presents the results of a previous geotechnical investigation undertaken by Douglas in 2020 at 600-660 Elizabeth Street, Redfern (Redfern Place). This report was commissioned by Bridge Housing Ltd and was undertaken in accordance with Douglas' proposal 99510.02.P.001.Rev0 dated 7 May 2024.

The purpose of this report is to address the Secretary's Environmental Assessment Requirements (SEARs) and to provide updated geotechnical advice applicable to the current proposed development with the aid of more recent borehole and groundwater level monitoring data completed by EI Australia Pty Ltd (EI).

This report accompanies a detailed State Significant Development Application (SSDA) that seeks approval for a mixed-use development at 600-660 Elizabeth Street, Redfern (Redfern Place). The development proposes four buildings comprising community facilities, commercial/office, affordable/social/specialist disability housing apartments and new public links and landscaping.

The project site comprises Lot 1 in DP 1249145. It has an area of approximately 10,834 m^2. Part of the site currently accommodates the existing Police Citizens Youth Club (PCYC) (to be demolished and replaced). The remaining portion of the site is vacant with remnant vegetation.

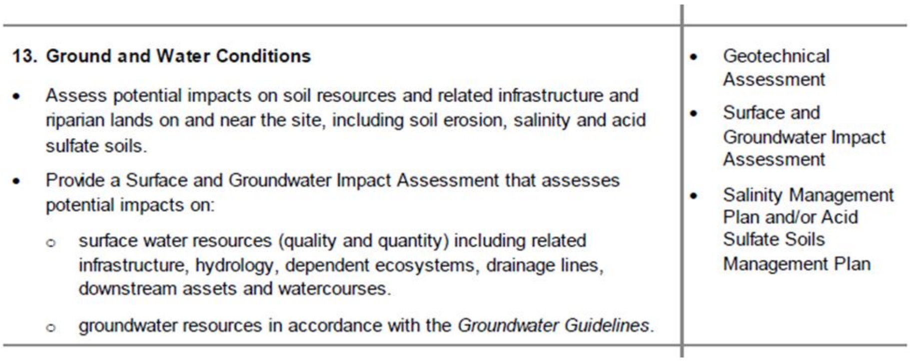
The SSDA seeks approval for redevelopment of the site, including:

- Demolition of existing buildings.
- Tree removal.
- Bulk earthworks including excavation.
- Construction of a community facility building known as Building S1.
- Construction of two residential flat buildings (known as Buildings S2 and S3) up to 14 and 10 storeys respectively, for social and affordable housing.
- Construction of a five-storey mixed use building (known as Building S4) comprising commercial uses on the ground level and social and specialist disability housing above.
- Construction of one basement level below Buildings S2, S3 and part of S4 with vehicle access from Kettle Street.
- Site-wide landscaping and public domain works including north-south and east-west pedestrian through-site link.

For a detailed project description refer to the Environmental Impact Statement prepared by Ethos Urban.

Further details of the proposed development are provided in Section 6.1 of this report.

The relevant sections of this report that address the SEARs requirements are shown below.

Item	SEARS Requirement	Relevant Section of this Report
1.0	 **13. Ground and Water Conditions** • Assess potential impacts on soil resources and related infrastructure and riparian lands on and near the site, including soil erosion, salinity and acid sulfate soils. • Provide a Surface and Groundwater Impact Assessment that assesses potential impacts on: ○ surface water resources (quality and quantity) including related infrastructure, hydrology, dependent ecosystems, drainage lines, downstream assets and watercourses. ○ groundwater resources in accordance with the *Groundwater Guidelines*. • Geotechnical Assessment • Surface and Groundwater Impact Assessment • Salinity Management Plan and/or Acid Sulfate Soils Management Plan	Geotechnical Assessment - All Sections of this Report. Surface and Groundwater Impact Assessment, Salinity Management Plan and Acid Sulfate Soils Management Plan - refer to reports by EI

The previous investigation by Douglas included the drilling of three rock-cored boreholes, six cone penetration tests (CPTs), groundwater dip measurements within the boreholes and CPT holes at the time of investigation, and laboratory testing of selected samples from the boreholes to assess the soil's aggressivity and plasticity/classification. The details of the field work and laboratory test results from the previous investigation are presented in this report, together with comments for design and construction.

2. Site Description

The site is a rectangular shape with a site area of approximately 10,834 m^2, as shown on Drawing 1 in Appendix B. The site is bounded by Kettle Street to the north, Walker Street to the east, Phillip Street to the south and Elizabeth Street to the west.

The southern third of the site is occupied by the PCYC, which generally includes one to two-storey brick buildings, a tennis court, soft-court playground, and asphaltic concrete (AC) on-grade car park. The remainder of the site comprises a parkland with a grassed surface and scattered mature trees. The park was formerly occupied by seven, apparently, single-storey residential buildings. Some building footings and buried services are expected to remain following the demolition of those buildings in 2013. Sewer main pipelines extend through the central area of the park.

The ground surface undulated throughout the park, with a generally mild slope down towards the south-east with reduced levels ranging between approximately RL 30 m and RL 31 m relative to the Australian Height Datum (AHD). Relatively large undulations in the order of 50 mm to 400 mm were observed in the road pavement, kerb/gutter and footpath along the western side of Walker Street, localised around the canopies of the large Melaleuca Guinquenervia (also known as ‘Paperbark’) trees. More information on the cause of the undulating ground is provided in Section 5 of this report.

3. Geology

Reference to the Sydney 1:100 000 Geological Sheet indicates that the site is located within an area underlain by Quaternary aged alluvium (marine sands), which typically comprise medium to fine-grained sand. The alluvium is underlain by Hawkesbury Sandstone, which is mapped further to the north-east. Hawkesbury Sandstone typically comprises medium to coarse-grained quartz sandstone with minor bands of shale.

The previous investigation confirmed the presence of alluvial soils underlain by Hawkesbury Sandstone.

The 1:25,000 Acid Sulphate Soil Risk map for Botany Bay indicates that the site does not lie within an area known for acid sulphate soils. The site also does not occur within an area mapped for known soil salinity issues.

4. Previous Investigation

Douglas has been granted permission from the Client, EMM Consulting Pty Ltd, of the previous geotechnical investigation report of 2020, to re-use the previous geotechnical data directly within this report. The details and findings of the previous investigation are reported as follows.

4.1 Field Work Methods

In 2020, Douglas completed a geotechnical investigation in areas that were readily accessible at that time and free of buried or overhead obstructions.

The field work included the drilling of three boreholes (BH301 to BH303) to depths of between 17.83 m and 25.65 m using a track-mounted drilling rig with 110 mm diameter continuous spiral flight augers/solid flight augers and rotary wash boring within the soil and NMLC (i.e. 50 mm diameter) diamond core drilling techniques in the bedrock. Standard penetration tests (SPT) were carried out at regular depth intervals to assess the soil strength and to collect samples for tactile assessment and laboratory testing.

It is noted that some of the SPT results appear suspect and are interpreted to be affected by problems associated with the rotary drilling method employed. It is likely that debris has fallen to the base of the borehole after removal of the drilling rods, prior to insertion of the SPT rods and that these tests have been performed on the loose debris instead of in-situ (undisturbed) soil at the base of the borehole. Based on correlation with the CPT data, it is interpreted that the SPT results between 9 m and 12 m depth may have been affected.

Cone penetration tests (CPTs) were undertaken at 6 locations (CPT304A, and CPT305 to CPT309) using a ballasted truck-mounted test rig to push a 35 mm diameter cone tipped probe into the soil with a hydraulic ram system. Continuous measurements were made of the end-bearing pressure on the cone tip and the friction on the sleeve located immediately behind the cone. Plots of the CPT results are produced with the interpretation of the soil type based on well-established correlations. Further information on CPT methods and interpretation of test results are given in the accompanying notes, included in Appendix C.

The location coordinates and surface RLs of the boreholes and CPTs were determined using a high precision differential Global Positioning System (dGPS), which has an accuracy of less than 0.1 m. Coordinates are in GDA94/MGA Zone 56 format (Geocentric Datum of Australia 1994 base with Map Grid of Australia projection) and RLs are relative to AHD. The test locations are shown on Drawing 1 in Appendix B.

All the field work was undertaken under the supervision of an experienced geotechnical engineer.

4.2 Field Work Results

The subsurface conditions encountered in the boreholes are described in the logs within Appendix C. Colour photographs of the recovered rock core are also included with the appropriate borehole in Appendix C. Notes defining descriptive terms and classification methods used are also included in Appendix C.

The results of the CPTs are also included in Appendix C together with the notes on the method and interpretation of the results. The inferred stratification based on the measured friction ratio is shown on each of the CPT results sheets.

4.2.1 Subsurface Profile

The sequence of materials encountered in the soil (and rock) profile across the site was generally uniform, both in terms of material type and strength/consistency/density.

The general sequence of subsurface materials encountered at the borehole and CPT locations is summarised in Table 1. Discussion on the selection of the geological 'Units' is provided in Section 6.2.

Table 1: Summary of Subsurface Profile

Material	Depth Range to Top of Unit (m)	RL Range of Top of Unit (m AHD)	Thickness Range (m)	Description
Fill	0	31.1 to 29.7	0.8 to 1.5	Fine to medium-grained sand with some fragments of gravel/brick/clay
Peat/Organic Clay	0.8 to 2.4	29.4 to 28.2	0.9 to 2.2	Dark grey, interbedded very soft to soft, with some organic materials and wood fragments
Sand (Generally Medium Dense)	2.7 to 3.4	28.8 to 26.7	2.0 to 5.0	Fine to medium-grained sand, typically medium dense and dense with interbedded soft to firm peat/silty clay bands

Material	Depth Range to Top of Unit (m)	RL Range of Top of Unit (m AHD)	Thickness Range (m)	Description
Peaty CLAY/SAND	5.8 to 6.8	23.8-24.5	2.9-8.2	Interbedded soft peaty clay with very loose to dense sand bands
Stiff to Very Stiff Clay	5.2 to 13.0	24.9 to 17.1	0.8 to 4.2	High plasticity, typically stiff to very stiff clay with sand and ironstone gravel (residual)
Medium to High Strength Sandstone	6.8 to 14.0	23.3 to 16.1	5.9 to >11.65	Medium to high strength sandstone with occasional extremely low strength bands

4.2.2 Groundwater

Groundwater was observed during auger drilling of BH301, BH302, and BH303. The use of water as a drilling fluid during the rotary wash-boring and core drilling of the boreholes precluded any further groundwater observations (i.e. below the depth of auger drilling). Groundwater was also measured in the CPT holes following the extraction of the rods.

A summary of the measured groundwater levels in the boreholes and CPT holes at the time of the investigation is provided in Table 2.

Table 2: Summary of Groundwater Measurements in Boreholes and CPT holes

Location ID	Surface RL (m AHD)	Depth to Groundwater (m)	Groundwater RL (m AHD)	Date Measured
BH301	31.1	3.5*	27.5	04.12.2019
BH302	30.5	1.6*	28.9	02.11.2019
BH303	30.1	3.5*	26.6	03.12.2019
CPT304A	30.6	1.6**	29.0	09.12.2019
CPT305	30.7	1.5**	29.2	09.12.2019
CPT306	30.4	1.7**	28.7	09.12.2019
CPT307	30.4	1.4**	29.0	09.12.2019
CPT308	30.0	1.4**	28.6	09.12.2019
CPT309	30.1	1.7**	28.4	09.12.2019

Notes: * Groundwater observed during auger drilling. The measurements are approximate, may be unstable levels and subject to fluctuations

** Water levels measured with tape within the open CPT holes. The measurements are approximate, may be unstable levels and subject to fluctuations

Groundwater dip-measurements indicated a groundwater table at depths of between 1.4 m and 3.5 m below ground level (i.e. at RL 26.6 to RL 29.2).

4.2.3 Laboratory Testing

Laboratory testing was undertaken on a selection of samples to determine the soil's aggressivity (pH, Electrical Conductivity, chloride ion content, sulphate ion content) for exposure classification of buried concrete and steel elements.

Laboratory testing was also undertaken on selected samples for Atterberg Limits, Linear Shrinkage and Field Moisture Content. The results of the laboratory aggressivity and Atterberg limits testing are included in Appendix D, with the results summarised in Table 3 and Table 4, respectively.

Table 3: Summary of Aggressivity Laboratory Test Results

Location ID	Material	Depth (m)	pH	Conductivity (µS/cm)	Cl (ppm)	SO_4 (ppm)	Resistivity (ohm.cm)[1]
BH301	SAND (SP) with interbedded peat bands	4.00-4.45	7.2	12	<10	<10	83,333
BH301	Silty Clay (CH) with sand	10.00-10.45	4.9	19	10	<10	52,632
BH302	Silty Clay (CH)	8.50-8.95	4.5	75	<10	74	13,333
BH303	SAND (SP) with clays and interbedded peat bands	5.50-5.95	5.3	88	<10	120	11,364

Notes: 1. Sample mixed 1(soil):5(water) prior to testing
2. Resistivity calculated as the inverse of conductivity

Table 4: Summary of Laboratory Test Results for Atterberg Limits and Moisture content

BH (Depth Range)	Description	WP (%)	WL (%)	PI (%)	LS (%)	Field Moisture Content (%)
BH303 (2.5-2.95 m)	Organic Clay (OH)	58	64	6	7.5	110
BH302 (1.1-1.4m)	SAND (SP)	Not Obtainable	Not Obtainable	Non-Plastic	Not Obtainable	6.1
BH303 (1.1-1.2m)	SAND (SP)	Not Obtainable	Not Obtainable	Non-Plastic	Not Obtainable	37.5
BH302 (1.4-1.45m)	PEAT/SAND	Not Obtainable	Not Obtainable	Non-Plastic	Not Obtainable	-

Notes: WP = plastic limit; WL = liquid limit; PI = plasticity index; LS = linear shrinkage; Iss = shrink-swell index

The point load strength index ($Is_{(50)}$) test results on rock cores were tested in-house, with the results shown on the borehole logs in Appendix C, at the respective test depths. The $Is_{(50)}$ values for the tested rock cores ranged from 0.55 MPa to 2.1 MPa, corresponding to a rock strength ranging from medium to high strength.

5. Background Information

5.1 Geotechnical Investigation in Walker Street by Douglas, 2009

Douglas previously undertook a geotechnical investigation for the City of Sydney Council in 2009 to assess the causes of the major damage to the Walker Street pavement between Kettle and Phillip Streets (i.e. along the eastern boundary of the subject site). Within the sandy soil, a very soft peat layer was generally identified between depths of 1.4 m and 2.4 m and underlain by very soft, organic clay typically between depths of 2.4 m and 3.2 m.

Shrink-Swell Index (I_{ss}) testing was carried out on samples of peat and organic clay to provide information on the soil reactivity and the field moisture content (FMC). The I_{ss} value provides an indication of the potential for volume change of the soil in response to variations in the soil moisture content. The Instability Index or Shrink-Swell Index (I_{ss}) for all the soils tested was very high, especially for the peat in the 'unaffected' area which was considered to have an extreme $I_{ss.}$

At no stage during the I_{ss} test did the peat or organic clay swell, the only observed movement was consolidation (i.e. shrinkage). It is noted that the peat had the ability to take on water while it consolidated, probably due to its organic structure. Consolidation can be explained as the settlement due to the drainage of pore water from a soil. As the pore water drains, the soil matrix becomes more compressed and the soil reduces in volume. The results of testing are summarised in Table 5.

In unaffected areas, the peat layer had an extremely high field moisture content (FMC) of 540% and shrink-swell index (I_{ss}) of 24% per ΔpF, whilst the organic clay had an FMC of 203% and I_{ss} of 11% per ΔpF. In affected areas, the peat had an FMC of 164% and I_{ss} of 12% per ΔpF, whilst the organic clay had an FMC of 96% and I_{ss} of 9% per ΔpF.

Due to the drought period which started circa 2000, together with the previous extraction of groundwater from the Botany sand aquifer, the regional water table lowered to within the peat layer or possibly below it. The "vertical striker roots" of the paperbark trees likely penetrated the peat and organic clay layer 'in search' of water. The trees dewatered and lowered the moisture content substantially in the peat and organic clay layers, leading to the consolidation of the highly compressible layers under the weight of the overburden soil pressure and tree weight.

Table 5: Results of Laboratory Shrink-Swell Index (I_{ss}) Testing

Borehole	Depth (m)	Description	FMC (%)	I_{ss} (% per ΔpF)
1	1.6 – 2.0	Peat – Unaffected Area	540	23.8
2	1.6 – 2.2	Peat – Affected Area	164	11.8
1	0.90-1.30	Organic Clay – Unaffected Area	203	10.7

Borehole	**Depth (m)**	**Description**	**FMC (%)**	**I_{ss} (% per ΔpF)**
2	0.50-0.80	Organic Clay – Affected Area	96	9.2

Notes: FMC – Field Moisture Content

The unaffected area (CPT1 and Borehole 1) is located under the centre of the road, away from the influence of the Melaleucas;

The affected area is directly below the Melaleucas and is influenced by them (all testing locations except CPT1 and Bore 1).

5.2 Other Consultants Reports

It is understood that the following geotechnical and contamination reports have more recently been prepared by EI for the proposed development:

- Additional Geotechnical Investigation (ref: Report E25947.G.04_Rev0, 15 March 2023);Additional Site Investigation (ref: Report E25947.E.03_Rev0, 31 March 2023);
- Groundwater Monitoring Report No.1 (ref: Report E25947.G11.01_Rev0, 15 November 2023);
- Acid Sulfate Soils Management Plan, (ref: E25947.E14_Rev1, 27 February 2024;
- Salinity Soils Management Plan (report pending);
- Groundwater Monitoring Report No.2 (ref: Report E25947.G11.02_Rev0, 3 April 2024);
- Groundwater Take Assessment (ref: Report E25947.G12_Rev2, 15 April 2024); and
- Remediation Action Plan (ref: Report E25947.E.06_Rev2, 16 May 2024).

The Groundwater Monitoring Report No.2 (ref: Report E25947.G11.02_Rev0, 3 April 2024) was made available to Douglas at the time of preparing this report and it describes the drilling of five boreholes at the current development site, which were drilled into bedrock with three groundwater monitoring wells installed.

The rock-cored boreholes (BH501 to BH505) encountered subsurface conditions that were generally consistent with those of Douglas' geotechnical investigation. Sandstone bedrock, logged as low, medium and high strength, was encountered below depths of between 7.6 m and 12.9 m or reduced levels (RLs) of between RL22.7 and RL17.6.

The groundwater level monitoring occurred for about one year from 2 March 2023 to 7 March 2024 from the three monitoring wells (BH502M, BH504M and BH505M) installed to depths of about 5.6 m to 6.0 m below ground surface and triangulated across the site. Wells at BH504M and BH505M are located in the central-northern area of the site and BH502M is at the southern site boundary. The locations of the wells installed by EI are shown on Drawing 1 in Appendix B.

The highest groundwater levels were approximately RL 28.7 m, RL 28.9 m and RL 29.4 m in wells BH502M, BH504M and BH505M, respectively. The lowest groundwater levels were measured at about RL28.4 m, RL 28.5 m and RL 29.1 m in wells BH502M, BH504M and BH505M, respectively. The groundwater levels fluctuate by about 0.3 m to 0.4 m with rainfall events, with a maximum daily rainfall of about 50 mm recorded during the monitoring period.

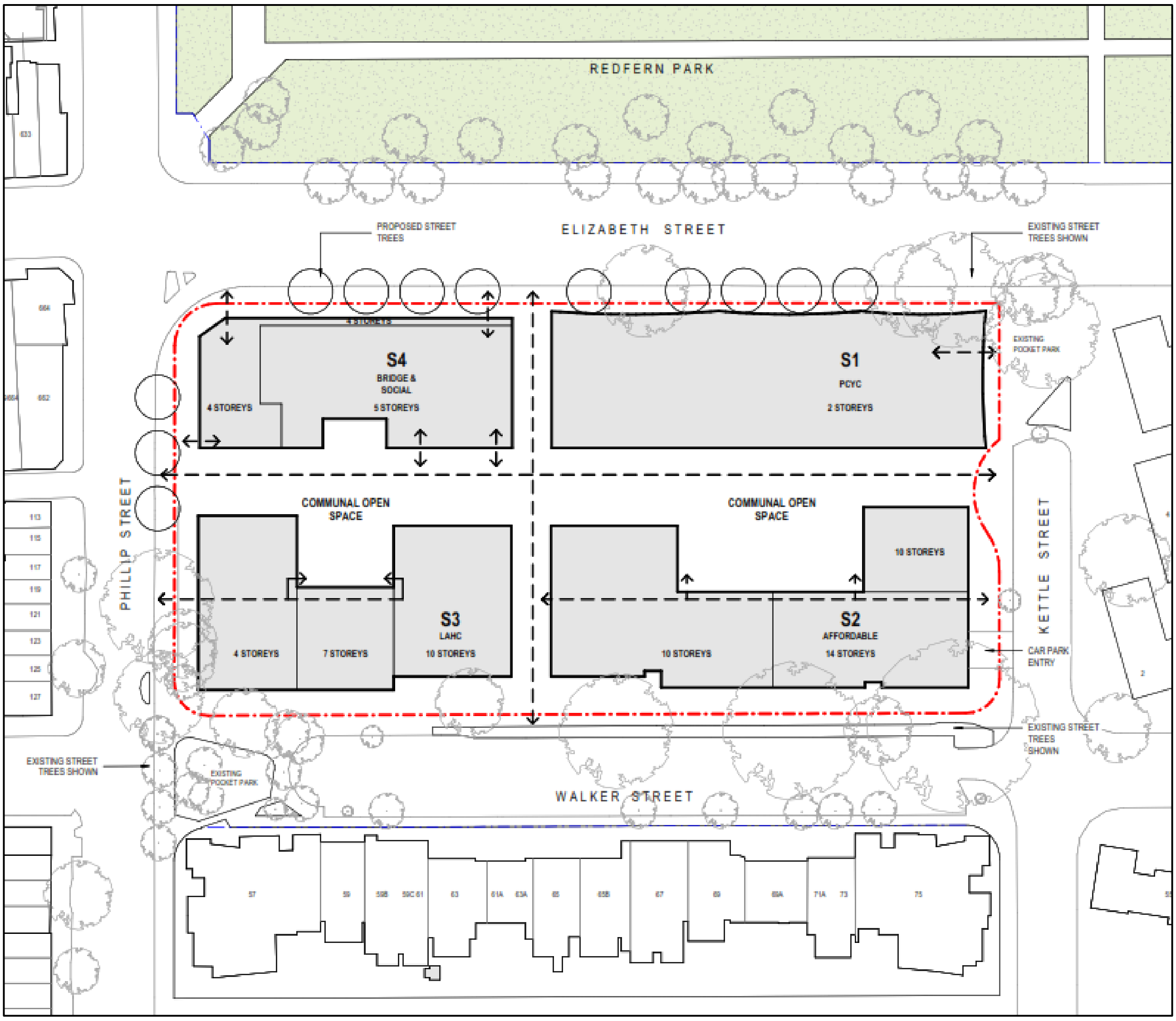
It is also understood that a previous site investigation for contamination was prepared by another consultant, EMM Consulting Pty Ltd (ref: Stage 2 Contamination Assessment, Report J190730 RP1, 29 May 2020) at the site.

6. Comments

6.1 Proposed Development

It is understood that the proposed development includes the demolition of the existing PCYC to allow for the construction of multiple buildings between two and fourteen storeys in height across the site, as well as a new PCYC with playing courts at the north-western corner of the site.

The locality of the proposed buildings and building numbers are shown in the image below. A selection of key architectural drawings of the proposed development are provided in Appendix B.



A one-level basement with finished floor level (FFL) mostly at RL 29.0 and partly to RL 28.07, extends across part of the site below proposed Buildings S2, S3 and S4. It is understood that a bulk excavation level (BEL) of RL28.4 is proposed for the single-level basement car park. Therefore, it is anticipated that a BEL of about RL27.4 would apply to the truck turning zone of the basement. The common basement is expected to require excavation to depths of 1 m to 3 m below current site levels, with localised deeper excavations for lift shafts, footings, etc.

The proposed PCYC (or S1) has a finished floor level at RL 31.5 for the building and playing courts. Approximately 1 m of fill is anticipated at the southern end of this area with the required fill height generally reducing towards the north.

The proposed S4 Building is also constructed over a basement “zone for additional flood storage”, fire storage tank and fire pump room with a FFL of RL 29.35.

Deep soil zones with landscaped areas are proposed around the basement footprints.

6.2 **Geotechnical Model**

The interpreted subsurface profile encountered at the boreholes and CPT locations has been grouped into six geotechnical units. Four geotechnical cross-sections (Section A-A’, B-B’, C-C’ and D-D’) showing the interpreted subsurface profile between the borehole and CPT locations are shown in Drawings 2, 3, 4 and 5, respectively, in Appendix B. The interpreted depth and RL at the top of the various strata boundaries at each test location is shown in Table 6. Reference should be made to the borehole logs and CPT test results for more detailed information and descriptions of the soil and rock profiles.

The interpreted strata boundaries shown on the cross-sections are diagrammatic only and should not be relied upon. The subsurface profile should only be considered accurate at the borehole or test locations and may vary away from and in between the test/bore locations. At the CPT locations, the depth to the top of rock was inferred from the CPT refusal depths, together with reference to the nearest cored borehole(s).

The site appears to be underlain by different depths of fill, sands and peat or clayey material overlying sandstone bedrock. The fill appeared to be variably compacted. The upper 5 m to9 m of the soil profile represents the most recent alluvial deposits overlain by varying depths of fill. Some of the clay and peat deposits were very soft and organic and may undergo long-term settlement or consolidation if subjected to surcharge loads. Planning and design should consider the presence of saturated, soft clays occurring at the proposed bulk excavation levels, and the impacts of soil consolidation on proposed and existing services and structures.

As noted, some of the SPT results (for the boreholes) below 9 m depth were discounted, and it was assumed that these low results (typically “N = 0”) are erroneous and caused by problems with the drilling method. This inference is based on the consistent and repeatable data obtained in the CPTs over the same depth interval. All CPTs located over the northern half of the site indicated the presence of stiff silty clay below 9 m depth. Notwithstanding this point, it is still possible that some zones of very soft organic clays are present in this 9 m to 12 m depth range. .

Mostly medium and high strength Hawkesbury Sandstone is expected below approximate levels of between RL16 and RL23. The top of rock is expected to generally dip down towards the north and the west, with a locally deeper area around BH303.

The previous laboratory test results indicated that the organic clay and peat samples are of high plasticity with an expected high potential for shrinkage.

There were also some parts of previously demolished building footings below the ground surface of the site, one of which was encountered while penetrating CPT 304. CPT304 was halted and filled, relocating the CPT rig by 0.5 m away and doing CPT304A to avoid obstruction from the existing old footing.

For detailed design and construction purposes, further investigation is recommended in the southern part of the site following the demolition of the existing PCYC building and playing courts. The 500 series boreholes by EI appear to be rock cored boreholes, however, the borehole logs in the Dewatering Management Plan show estimated rock strengths with the absence of point-load strength tests and/or unconfined compressive strength tests on the recovered rock core samples.

Table 6: Summary of Geotechnical Model

Material	**Depth (m) [Reduced Level (m AHD)] to Top of Each Unit**								
	BH301	**BH302**	**BH303**	**CPT 304A**	**CPT 305**	**CPT 306**	**CPT 307**	**CPT 308**	**CPT 309**
Fill	0.0 [31.1]	0.0 [30.5]	0.0 [30.1]	0.0 [30.6]	0.0 [30.7]	0.0 [30.4]	0.0 [30.4]	0.0 [30]	0.0 [30.1]
vs-s Peat/Organic Clay	1.3 [29.8]	1.4 [29.1]	1.2 [28.9]	0.8 [29.8]	1.3 [29.4]	1.2 [29.2]	1.0 [28.2]	1.5 [28.5]	1.9 [28.2]
l-md Sand with Peat Bands	2.7 [28.4]	2.7 [27.8]	3.4 [26.7]	1.7 [28.9]	3.1 [27.6]	2.8 [27.6]	3.2 [27.2]	3.3 [26.7]	3.2 [26.9]
Peaty CLAY/SAND	6.8 [24.2]	6.7 [24.2]	5.8 [24.3]	6.5 [23.9]	6.2 [24.5]	6.0 [24.4]	NE	NE	NE
st-vst Clay	12.3 [18.8]	NE	13.0 [17.1]	8.8 [21.8]	8.6 [22.1]	8.8 [21.6]	8.0 [22.4]]	8.3 [21.7	5.2 [24.9]
M and H Sandstone	13.1 [18.0]	11.6 [18.9]	14.0 [16.1]	11.7 [18.9]	12.6 [18.1]	12.8 [17.6]	10.2 [20.2]	11.4 [18.6]	7.1 [23.0]

Notes: vs-s = very soft to soft; st-vst = stiff to very stiff; l-md = loose to medium dense;
M = Medium Strength, H = High Strength. NE = Not Encountered

A relatively shallow groundwater table exists across the site.

In 2019, groundwater was measured at depths of between 1.4 m and 3.5 m and reduced levels of between RL 29.2 and RL 26.6.

In 2023/2024, the highest groundwater levels in the three wells by EI (BH502M, BH504M and BH505M) ranged between approximately RL 29.4 and RL28.4.

The long-term groundwater monitoring indicates that the groundwater flow direction is generally in a south-westerly direction and may fluctuate by at least 0.4 m following rainfall events.

It should be noted that groundwater levels are transient and that fluctuations may occur in response to climatic and seasonal conditions, such that groundwater levels may fluctuate more than the measured fluctuations to date.

6.3 **Dilapidation Surveys**

Dilapidation (building condition) reports should be undertaken on surrounding properties, utilities and infrastructure prior to commencing work on the site to document existing condition and defects so that any claims for damage due to construction-related activities can be accurately assessed. As a minimum, this should include adjacent Council property such as footpaths and roads which surround the site.

6.4 **Excavations**

Construction of the proposed basement will generally involve excavation to depths of about 1 m to 3 m below current site levels, with localised deeper excavations for lift shafts, footings etc. The BELs are expected to be within fill and organic soils, soft to very soft clayey soils. The basement excavation is also expected to be in the order of 0.5 m to 1 m below the groundwater table. The general sequence of materials to be removed from the proposed basement excavation is shown on the Interpreted Geotechnical Cross-Sections presented in Drawings 2 to 5, in Appendix B.

6.4.1 **Excavation Conditions**

Excavation to deeper than the proposed BELs may be required for the construction of a working platform from which tracked (piling) plant can operate given that very soft and soft and peaty soil is expected to remain below the BELs.

Excavations for the basements are likely to intersect pavements, fill and natural soil. Excavation of soil should be readily achieved using conventional earthmoving equipment, such as tracked excavators with bucket attachments. Excavation of existing pavements and any buried concrete footings is likely to require large excavators fitted with hydraulic rock hammers and/or rotary rock saws.

Prior to excavation, groundwater levels will need to be controlled within the basement area to a minimum of 1 m below the level of excavation for trafficability purposes (see Section 6.6 of this report). It should be noted that even when organic/peaty soil and sands have been dewatered, the excavated material will have high water content due to the remaining interstitial water. It is possible that some of the organic/peaty soil and sands will therefore require pre-treatment, such as spreading and drying and/or blending with drier materials to enable them to be readily removed using standard excavator attachments and loaded onto conventional dump trucks.

For temporary slopes in the existing fill and new engineered fill above the groundwater table and up to 2 m high, temporary batter slopes no steeper than 1.5:1 (Horizontal:Vertical) may be adopted provided the batters slopes are not surcharged directly behind their crest.

For permanent batter slopes in the existing fill and new engineered fill above the groundwater table and up to 2 m high, permanent batter slopes no steeper than 3:1 (Horizontal:Vertical) may be adopted provided the batters slopes are not surcharged directly behind their crest. Protection of permanent batter slopes from erosion should be provided by vegetation and suitable environmental mesh products, such as Jutemesh®.

Within very soft/soft peat/organic clay, even where groundwater is controlled below the excavation level, very shallow gradient batters would be required and are considered to be impractical and potentially unstable for the proposed basement excavation depths of up to 3 m. It will therefore be necessary to provide retaining support (i.e. shoring) for these soils for the basement construction.

With respect to trafficability, the existing fill, organic/peaty soil and sandy soils are likely to cause difficulties for the plant, particularly below the present groundwater table, where wet, “boggy” conditions could be expected even after dewatering. It will be necessary to form a rockfill working platform for piling/wall construction plant and for machinery required for the construction of foundations at the proposed BELs. Crushed (recycled) concrete, possibly sourced from the demolition of the existing structures or elsewhere, may be suitable to form a working platform following crushing to less than 70 mm maximum particle size and subject to environmental considerations. Geotextiles and geogrids could be incorporated to reduce the required thickness of granular bridging layers for working platforms.

For the shoring construction and any piling from the surface, it may be beneficial to leave the existing ground slabs and paving in place to provide a trafficable working surface. Due allowance should be made for the design and construction of suitable piling and general working platforms, both at the surface and at BELs. Consideration may be given to the incorporation of the working platform into the design of any raft slabs for the final basement structure.

6.4.2 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including “Waste Classification Guidelines” - 2014, New South Wales Environment Protection Authority (NSW EPA). This includes fill and natural materials that may be removed from the site. Reference should be made to Contamination Assessment reports and the Remediation Action Plan prepared by EI, for guidance on the off-site disposal of excavated materials.

6.5 Excavation Support

6.5.1 General

To reduce the dewatering requirements for construction of the basement, and to limit drawdown (i.e. lowering) of the groundwater table outside the basement during internal dewatering, the shoring wall should be relatively impermeable, installed around the full perimeter of the excavation and embedded a substantial depth below BEL. The embedment depth requirement is both to achieve the required passive restraint and to reduce groundwater inflow rates to practical volumes that can be pumped and managed to lower the groundwater table. It would be preferable for the shoring wall to be socketed at least 1.5 m into sandstone or consistent very stiff to hard clay, below the bulk excavation levels to reduce groundwater inflow rates. The shoring wall should not be terminated in the clayey sand/sandy clay layers at a higher level as this material is expected to have a higher permeability and may not provide an adequate barrier or 'cut-off' to groundwater seepage below the wall.

It is understood from the Groundwater Modelling and Take Assessment by EI, that for a sheet pile shoring wall installed 0.5 m into the residual clay stratum, an estimated inflow rate into the basement excavation of 0.01 m^3/day or 1.3 ML/year is estimated. Sheet pile walls with a 3 m and 6 m design socket length below BEL were also analysed. Estimated inflow rates in the order of 283 ML/year and 247 ML/year were predicted for sheet pile walls installed 3 m and 6 m below the BEL, respectively. Such inflow rates are clearly impractical to manage during construction.

Careful consideration should be given to the adoption of sheet piles to form the shoring walls for this project due to their potential for leakage and excessive lowering of the groundwater table that previous studies by Douglas (2009) have demonstrated is of critical importance due to the presence of soft organic clays and peats at shallow depth at the site and the surrounding area. For this reason, an impermeable shoring system such as diaphragm walls and secant pile shoring walls would be preferable to sheet piled walls, in areas where lowering of the groundwater level by more than 0.5 m below historically measured groundwater levels is required. Further discussion of this issue is given in the following sections of this report.

If the shoring walls are socketed into the sandstone then the shoring walls (unless sheet pile walls) could also be designed to support the multi-storey building loads.

Temporary lateral restraints such as anchors or internal props may be required; If so, they must be installed progressively as the excavation proceeds. It is anticipated that permanent lateral support of the basement retaining walls will eventually be provided by the structure of the completed buildings.

A relatively stiff retaining/shoring wall system may be required to limit the lateral deflection of shoring walls where basement excavations are close to the site boundaries and any existing infrastructure and buried services.

One of the controlling factors affecting the viability of basement construction through water-charged sandy soils is the capacity of ground anchors to restrain the upper part of the wall. The design and construction of ground anchors are discussed in Section 6.5.4.

It should be noted that it is not possible to totally eliminate lateral movement in an excavation. All walls move to some degree, depending on the magnitude of lateral restraint provided. The capacity of the adjacent road infrastructure and any services to withstand such movements should be considered as a part of wall selection and design.

It is suggested that survey targets be installed on the top of the shoring walls to monitor wall deflections and check that wall movements are as predicted.

6.5.2 Retaining/Shoring Wall Systems

The final basement structures should incorporate a watertight, tanked basement system given it is below the groundwater table in soft alluvium that are highly susceptible to consolidate in the long term if the groundwater table is lowered beyond historical groundwater fluctuations.. .

- Interlocking secant pile walls (temporary and permanent) – secant pile walls are typically formed by drilling alternate 'soft' grout or concrete piles and then installing 'hard' reinforced concrete piles by cutting into the previously drilled soft piles. This overlap typically ensures that piles are sealed, but even at relatively shallow depths, some misalignment can occur, and hence minor gaps sometimes appear in the wall. Drilling of piles into rock will also be problematic for secant piles and may result in decompression (or 'flighting') of the surrounding sands which can result in settlement and damage to adjacent infrastructure or utilities. The use of segmental casing and a high-powered (CFA) piling rig may be required to avoid issues associated with decompression.
- Diaphragm walls could also be used as the permanent basement wall. These walls are associated with lower risk but are relatively slow to construct and consequently more expensive. Diaphragm walls are constructed using a large grab, which excavates the soil and rock in panels which are supported by bentonite fluid. Each panel is then cast using concrete tremmied into the bentonite supported excavation, with reinforcement cages installed prior to the concrete being tremmied. The joints between the panels are sealed with a waterstop so that a completely water-tight wall is achieved.

It is understood that the more recent EI reports for the building Contractor, Hickory Construction Redfern Pty Ltd, consider steel, sheet pile shoring walls for the basement construction. Such walls are a possible alternative to the above shoring systems, however, provide a temporary shoring system that must have permanent, water-tight (tanked) basement walls constructed within it sheets. Sheet piles do experience leakage through their clutches and as such could effectively lower the groundwater table outside the basement perimeter during construction. Also, sheet piles can 'de-clutch' at their joints, or if misaligned, may form gaps leading to greater groundwater inflows than estimated by computer modelling and would need to be sealed as excavation proceeds. Penetrating the sheet piles for any ground anchor installations or bolting of internal props would also require specialised methods to prevent the loss of soil and groundwater through penetrations.

6.5.3 Basement Retaining Wall Design

It is suggested that preliminary design of shoring systems may be based on the earth pressure coefficients provided in Table 7. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_o) values should be used where the wall movement needs to be reduced. A triangular earth pressure distribution may be assumed where shoring walls are designed as cantilever walls or walls restrained by a single row of anchors or propping/bracing. (Cantilever walls should not be used for walls of more than 3 m in height or where deflection of the wall and retained ground must be limited).

Table 7: Parameters for Retaining Wall / Shoring Design

Material	Bulk Unit Weight (kN/m³)	Buoyant Unit Weight (kN/m³)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	Passive Earth Pressure*/ Coefficient
Fill	18	8	0.4	0.6	N/A
vs-s Peat/ Organic Clay	18	8	0.4	0.6	N/A
l-md Sand	21	11	0.33	0.45	K_p = 3.0
st-vst Clay	20	10	0.3	0.5	100 kPa

Notes: vs-s = very soft to soft; st-vst = stiff to very stiff; l-md = loose to medium dense;
*Ultimate values and only below bulk excavation level. May need to be reduced where batter slopes are located nearby

Hydrostatic pressure should be assumed to act on the full height of the basement walls to account for increases in groundwater levels caused by significant rainfall events and occasional flooding. Surcharge pressures from adjacent structures, construction machinery, stored materials and traffic should also be incorporated into the design of the wall as necessary.

Detailed design of the basement retaining wall should ideally be undertaken using a computer program such as PLAXIS, WALLAP or FLAC to model soil-structure interactions during different phases of construction. This detailed analysis could also be used to incorporate and model the effect of dewatering on the excavation and shoring and to assess the sensitivity of the proposed design to variations in the ground conditions.

6.5.4 Ground Anchors

Where necessary, the use of declined 'tie-back' (ground) anchors is suggested for the lateral restraint of the perimeter shoring walls. Such ground anchors should be declined below the horizontal to allow anchorage into the stronger materials at depth. The design of temporary ground anchors for the support of shoring wall systems may be carried out using the allowable average bond stress at the grout-soil interface given in Table 8.

Table 8: Allowable Bond Stresses for Anchor Design

Material Description	Allowable Bond Stress (kPa)
Medium dense sand or stiff clay (below 4 m depth)	25

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

Ground anchors should be designed to have a free length equal to their height above the base of the excavation and have a minimum of 3 m bond length. After installation, they should be proof loaded to 125% of the design Working Load and locked-off at no higher than 75% of the Working

Load. Periodic checks should be carried out during the construction phase to ensure that the Lock-Off Load is maintained and not lost due to creep effects or other causes.

The parameter given in Table 8 assume that the anchor holes are clean, with grouting and other installation procedures carried out carefully and in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical specialist may allow increased bond stresses to be adopted during construction, subject to testing.

In normal circumstances, the building will restrain the basement excavation over the longer term and therefore ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed on this site.

It will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services and pipes during anchor installation.

In general, the capacity of the upper soil profile is expected to be fairly poor for anchoring. Where high anchor loads are needed, it may be necessary to consider either specialist anchoring methods such as post-grouting or pressure grouting methods for sandy soils.

6.6 Groundwater and Dewatering

It is understood that a fully-tanked, watertight basement system will be adopted, such that dewatering will only be necessary for the temporary construction situation. Therefore, a secant pile shoring wall or diaphragm wall embedded into bedrock is recommended to cut off the flow of groundwater seepage into the basement. A sheet pile shoring/retaining wall could also possibly be considered, noting the additional risks and requirements associated with constructing a tanked basement. Leakage commonly occurs with sheet piles and due consideration of these risks is warranted.

Given the sensitivity of the peat and organic clay underlying the site and surrounds (at shallow depths), it will be particularly important to avoid significant lowering of the groundwater table during basement construction. It is understood from EI's Groundwater Modelling and Take Assessment that if a sheet pile shoring system was installed with a 3 m and 6 m socket design below BEL, the expected water level drawdown would be about 0.15 m and 0.12 m, respectively. For sheet pile walls installed 0.5 m into residual clay, EI suggest the drawdown would be negligeable. The nature of damage observed by Douglas and the Council in 2009 during a drought period, as described in Section 5 of this report, should be carefully considered in regards to the robustness of sheet pile, or other shoring systems in not significantly lowering the groundwater table in area surrounding the basement as this is likely to cause consolidation of soft peat/organic clays that would likely result in surface settlement and damage.

It is recommended that additional rock-cored boreholes are drilled around the proposed basement perimeter, so as to clearly define the required founding level of a 'cut-off' wall for design and construction purposes.

It is noted that PLAXIS 2D modelling by EI for sheet pile shoring walls installed to 3 m and 6 m below BEL predicted drawdown-induced groundwater settlement in the order of 15 mm and 12 mm, respectively. Predicted drawdown-induced groundwater settlement is not provided by

EI for sheet pile walls installed 0.5 m into residual clay, nor are any mounding effects reported for such a situation.

Within the basement excavation, it is suggested that the water level should be kept at least 1 m below the bulk excavation level to allow machinery to operate.

For a cut-off wall socketed into clay and rock, groundwater inflow into the excavation is expected to be primarily controlled by the watertightness of the cut-off walls and the more permeable zones below the floor of the basement.

In order to confirm that dewatering within the excavation zone does not also dewater zones outside the cut-off wall, such as through gaps in the cut-off wall, it is recommended that observation (standpipe) wells are installed outside the proposed basement area and monitored during dewatering until the development is completed. In this way, any significant groundwater drawdown outside the proposed excavation can be detected and addressed by varying pumping rates or jet-grouting zones of apparent leakage.

The potential to dewater and dispose of extracted groundwater off-site into the Council's stormwater system will depend on the contamination status of the groundwater and other groundwater properties. Reference should be made to reports by EI for groundwater management.

6.7 Piling and Foundations

6.7.1 General

Given the poor soils and high groundwater table indicated for the site, it is recommended that building loads are supported on piles founded within the underlying sandstone bedrock.

It is estimated that the design column working loads will be in the order of 7,000 kN for a 14-storey building, 5,000 kN for a 10-storey building and 2,700 kN for a five-storey building, based on an average column spacing of 8 m. There will also be some uplift loads present due to higher groundwater levels surrounding the basement walls. Considering the likely magnitude of column loads for the buildings, the development will need to be uniformly supported on piles founded within the underlying sandstone bedrock to reduce the potential issue of differential settlements. The use of rock-socketed piles may also be used to resist uplift (tension) loads on the basement floor slabs.

Bored pile excavation holes would not remain open in the sandy fill and natural sands, particularly below the groundwater table; therefore it is recommended that the piles be installed by continuous flight auger (CFA) methods. Continuous flight auger (CFA) concrete injected piles can be used to support the structural loads. The CFA rig would need to be powerful enough to drill a substantial socket into the underlying medium to high strength sandstone.

CFA piling is a 'blind' piling technique, and the piling contractor would need to be responsible for the assessment of whether suitable materials were encountered and whether available bearing capacities meet the design requirements. Additional cored boreholes could be drilled to prove the bearing stratum at key column locations across the site.

Soil decompression or 'flighting' can occur during CFA piling when a strong stratum is encountered and the penetration rate of the auger cannot be maintained. In this case, the augers continue to rotate but the rate or auger progression decreases and soil from around the auger is displaced upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and can lead to the damage of structures or utilities supported at high levels. Decompression should be avoided by monitoring auger speed and progression closely, using a suitable, experienced piling contractor with powerful, high-torque rigs.

6.7.2 Design

For the preliminary design of cased bored or CFA pile foundations, recommended maximum design pressures for the rock strata, for axial compression loading cases, are presented in Table 9. The shaft adhesion values for uplift (tension) piles may be taken as being equal to 70% of the values for compression.

Table 9: Recommended Design Parameters for Foundation Design (Piles or Pad Footings)

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure		Young's Modulus E (MPa)
	End Bearing⁽¹⁾ (kPa)	Shaft Adhesion⁽²⁾ (Compression) (kPa)	End Bearing⁽¹⁾ (kPa)	Shaft Adhesion⁽²⁾ (Compression) (kPa)	
M and H Sandstone	6,000	500	50,000	1,000	1,000

Notes: (1) End bearing pressures only applicable where socket extends at least one pile diameter into nominated founding stratum.

(2) Shaft adhesion applicable for the design of bored piers, uncased over rock socket length, where adequate sidewall cleanliness and roughness are achieved.

The settlement of a pile is dependent on the loads applied to the pile and the foundation conditions in the socket zone and below the pile toe. The total settlement of foundation piles designed using the 'allowable' parameters provided in Table 9 should be less than 1% of the pile diameter under the 'Working' or serviceability loading.

An appropriate geotechnical strength reduction factor should be applied when using the limit-state design approach for pile design as outlined in AS 2159 – 2009 Piling – Design and installation.

Based on the medium and high strength rock indicated by the boreholes, the drilling of long pile socket lengths may prove to be difficult and may cause decompression when using CFA piling methods. The construction of pile groups instead of single piles are likely for the anticipated column loads, particularly for the higher 10 and 14-storey buildings.

Additional rock-cored boreholes will be required to confirm the parameters given in Table 9 and to confirm rock socket levels across the broader site. The CFA method is a 'blind' piling method, so that a higher coverage of boreholes is warranted. Boreholes should be drilled at all key column locations and also across the proposed building footprint areas, to confirm founding levels.

6.7.3 Negative Skin Friction

It is recommended that allowance is made for the effects of negative skin friction on the shafts of piles. This is due to the potential effects of surface-induced loading (unsupported by piles) which will induce consolidation of the soft recent alluvium beneath the fill. Such friction-induced loads should be applied to the pile shaft length up to depths of approximately 16 m below the existing ground level.

The negative skin friction (τ) (in kPa per unit area of pile shaft) may be calculated as follows:

- for soft to firm (or softer) clay: $\tau = 0.15$ p’
- for loose to medium dense sand fill: $\tau = 0.20$ p’ (where p’ is the effective overburden pressure).

For piles at the BEL of the basements, negative skin friction considerations will generally not apply if the surcharge from the fill is removed. It is only where piles are outside the proposed basement and penetrate the soft peat and clay layers.

6.7.4 Slabs and Consolidation

Approximately 1-2 m of saturated, very soft/soft peaty/organic clay is expected to remain below BELs.

Consolidation of saturated, very soft/soft peaty/organic clay and very loose/loose sands is likely to occur with the placement of fill to raise surface levels for the proposed community facility building known as Building S1 or by applying a net surcharge pressure with basement or ground slabs on-grade.

For slab design, it is recommended that settlement analysis is undertaken to estimate the amount of consolidation or long-term settlement given the presence of very soft/soft clayey soils.

This would typically require piezocone testing to measure the excess pore water pressure dissipation rate in the very soft/soft clayey soils and to determine consolidation parameters for analysis.

Depending on the settlement tolerance of the slabs, it may be necessary for all basement and ground floor slabs to be fully-suspended on foundation piles, with any buried services tied or hung to the underside of the basement slabs to reduce the risk of the services moving with long-term differential settlement due to soil consolidation.

An alternative approach for the design of any slabs on grade would be to remove all of the soft organic clays and peat soils and replace with select (granular) engineering fill materials, with settlement analysis also completed.

6.7.5 Soil Aggressivity

Aggressivity to concrete piles was assessed using the laboratory test results from the previous investigation by Douglas in 2020. The exposure classification is assessed as being ‘mildly aggressive’ for steel piles, and ‘moderately aggressive’ for concrete piles in accordance with Australian Standard AS 2159 – 2009 *Piling – Design and installation*.

6.8 **Seismicity**

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class would be "Class D_e" based on the strengths of the materials encountered in the boreholes, including the presence of very soft and soft organic clays and peat materials. .

6.9 **Ground Vibrations**

Vibrations may be induced by a large number of site activities, including demolition of existing structures, shoring installation and anchoring, excavation, piling, and compaction works. Hence, particular care to avoid damaging adjacent buildings, utilities, or structures will generally be required.

Vibrations may cause densification of very loose sand layers and produce settlements in adjacent structures, pavements, or utilities founded at high levels.

The level of acceptable vibration is site-specific and 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.

The Australian Standard AS 2187.2 - 1993 "Explosives Code" recommends a maximum peak particle velocity (PPV) of 10 mm/sec to avoid structural damage to houses and low-rise residential or commercial buildings. Ground vibration arising from excavation plant is of a continuous nature, as opposed to transient nature such as with blasting events. More stringent vibration limits should generally apply for excavation plant than for blasting.

Douglas' experience indicates that vibration levels in the order of 5 to 7 mm/sec are sufficient to densify sands or cause damage in sensitive buildings or structures with pre-existing problems. Lower vibration levels have also, in a few cases, been known to cause densification in sands. Careful planning of excavation and earthworks adjacent to existing buildings or utilities will therefore be required. It is noted that the movement of heavy machinery around the site will also generate vibrations. It is recommended that a provisional (PPV) vibration limit of 5 mm/sec be adopted at the building line or adjacent buildings around the perimeter of the site, or at any utilities of concern.

It is recommended that a number of settlement monitoring points are established on the adjacent ground surface and road infrastructure, with regular surveying carried out in order to identify any settlement that may occur due to vibration or other construction activities. It should be noted that vibration-induced settlement in sands is not necessarily instantaneous, and the settlement may occur sometime (in the order of weeks) after vibrations have ceased.

It should also be noted that human perception of vibrations is much greater than that of buildings and consequently vibration levels considered insignificant for buildings may disturb humans.

Dilapidation reports should be undertaken on neighbouring properties prior to commencing work on the site to document any existing defects so that any claims for damage due to construction activities can be properly assessed.

Where vibrations are a concern for the operation of the plant at the site, consideration should be given to vibration trials at the commencement of work, which may indicate minimum setbacks from existing buildings or sensitive areas for a specific plant, and possibly the requirement for continuous vibration monitoring.

6.10 **Working Platforms**

Working platforms will be required where heavy loads such as from large piling or diaphragm wall rigs, or outrigger pads for mobile cranes are anticipated during construction, particularly in areas where poorly compacted fill and soft clay or loose sand is present. Such platforms typically require the use of additional layers of durable, high strength crushed rock, crushed recycled concrete, or similar. A working platform assessment specific to piling rigs/mobile cranes would be required at a later stage.

It is noted that failures of working platforms occur most frequently in the vicinity of poorly backfilled trenches and excavations. As these weaker ground conditions are localised, they may not be identified by borehole testing. It is therefore recommended that working platforms be proof-rolled using a 10-tonne roller (or similar) in the presence of a geotechnical engineer to detect any soft spots for remediation. Existing excavations within working platforms should be suitably backfilled to reduce the potential for working platform failures.

6.11 **Survey Monitoring**

The use of instrumentation to monitor existing adjacent roads and footpaths (and possibly buildings/structure) movements will be important for this development as the existing roads and streets are likely to be sensitive to differential foundation movement.

Precise survey points should be established on existing roads, buildings and structures adjacent to the proposed basement and services diversion excavations as well as on the shoring wall capping beam, prior to the commencement of any excavation works. Monitoring should be undertaken to an accuracy of at least ± 1 mm and should be continued throughout the construction phase until excavation faces are permanently supported by the new building structure, or in the case of the services diversion, until backfilled and completed.

Survey readings must be taken prior to the commencement of any excavation works to provide baseline readings. The frequency of survey monitoring should be at every 1.5 m drop in excavation or at least weekly.

A “trigger” or alarm level appropriate for the shoring system and based on expected movement, should be adopted for survey monitoring of existing buildings and the proposed shoring wall. A monitoring plan should be developed that includes trigger levels, hold points and actions by responsible parties, at which time the builder would be obliged to seek further advice from structural and geotechnical engineers.

6.12 Earthworks and Subgrade Preparation

The presence of the underlying soft peat and (organic) clay will mean that ongoing consolidation (settlement) is generally unavoidable. As such, any structures or pavements constructed above the soft clays will experience settlement-related damage over the long term.

Notwithstanding the above, it may be possible to construct a reasonable subgrade for lightly loaded slabs and pavements, provided that a minimum 800 mm thick layer of sand/gravelly sand (fill) is above the underlying soft peat/organic clay layer. The following general procedure is suggested for engineered fill construction at this site:

- Strip any topsoil, organic or root-affected material or other deleterious material down to a stable subgrade surface comprising loose (or better) sand or stiff clay, ensuring a minimum 800 mm thick 'bridging' layer of granular soil remains above the soft clay/peat material identified across the site;
- Proof roll the exposed surface using at least six passes of a minimum 8 tonne, smooth-drum roller, with the final test roll pass to be inspected by an experienced geotechnical practitioner to ensure that any soft or compressible materials are removed and replaced with 'select' rockfill (e.g. ripped sandstone), compacted in layers as described below;
- Place 'select' granular fill, if required, in near-horizontal layers whose thickness is appropriate to the machinery being used, but no thicker than 250 mm loose thickness. Fill should be approved, homogeneous, free of organic or other deleterious material, and have a maximum particle size of 75 mm;
- Place each layer of fill and compact horizontally in a cut and benched formation in accordance with AS 3798 where ground slopes are greater than 8H:1V;
- Compact each layer of fill to at least 98% Standard maximum dry density ratio; or 100% in the upper 0.3 m below the design subgrade level; and
- undertake 'Level 1' inspection and testing as detailed in AS 3798–2007 for new fill below pavements and where required for slabs or foundations.

The above method is generalised, and revision may be appropriate once further details are known on the proposed works, particularly if deeper fill is proposed. The risk and adverse impacts of long-term consolidation should also be considered for detailed design purposes, as described in Section 6.7.4 of this report. It may be necessary to design basement slabs as fully-suspended slabs.

6.13 Pavements

New pavements for any access roads or car parking should be designed as flexible pavements, which can be periodically remediated and repaired following settlement related damage. Concrete or block paving should be avoided as these pavements will be more difficult and costly to repair.

Provided the subgrade for all new pavements is controlled as described in Section 6.12, preliminary (flexible) pavement design could be based on a design CBR value of 2%, provided select engineered fill is used as a bridging layer. This design CBR value should be confirmed by future investigation of the CBR values of materials at the design subgrade levels and of the select engineered fill, and for any alternative material(s) proposed for use in the pavement subgrade.

It is Douglas' experience that the medium and long-term performance of pavements on sites such as this is often related to the drainage conditions, including surface and subsurface drainage, and at interfaces between pavement types. Careful attention should therefore be paid to the detailing of the new pavements, noting that pavement design based on design CBR assumes that the soils below the pavement remain at an equilibrium moisture content. Appropriate maintenance of the pavement surface, to limit the ingress of water through the pavement surface, will also be critical for its performance.

Given the presence of soft soils beneath the site, provision should be made for regular maintenance and pavement rehabilitation works.

6.14 **Further Investigations**

For detailed design and construction purposes, further investigation is recommended in the southern part of the site following the demolition of the existing PCYC building and playing courts. The 500 series boreholes by EI appear to be rock cored boreholes, however, the borehole logs in the Dewatering Management Plan show estimated rock strengths with the absence of point-load strength tests and/or unconfined compressive strength tests on the recovered rock core samples.

Additional rock-cored boreholes across the site to reduce data-gaps is also recommended to clearly define the required founding level of a 'cut-off' shoring wall for detailed design and construction purposes.

Piezocone testing for consolidation analyses is also recommended if slabs or pavements are designed to be supported on grade.

Groundwater wells should be installed beyond the basement periphery to allow groundwater level monitoring during the construction dewatering. Reference should be made to reports prepared by EI for any further investigations and monitoring, as required.

7. **Limitations**

Douglas Partners Pty Ltd (Douglas) has prepared this report for this project at 600-660 Elizabeth Street, Redfern NSW in accordance with Douglas' proposal 99510.02.P.001.Rev0 dated 7 May 2024 and acceptance received from Bridge Housing Ltd dated 14 May 2024. The work was carried out under Douglas' Engagement Terms. This report is provided for the exclusive use of Bridge Housing Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of Douglas, does so entirely at its own risk and without recourse to Douglas for any loss or damage. In preparing this report Douglas has necessarily relied upon information provided by the client and/or their agents.

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

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

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

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. Douglas 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 Douglas. This is because this report has been written as advice and opinion rather than instructions for construction.

This report provides specialist advice only and no part of it is considered a Regulated Design under the Design and Building Practitioner Act 2020 (NSW).

Appendix A

About this Report

About this Report

November 2023

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.

continued next page

About this Report

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

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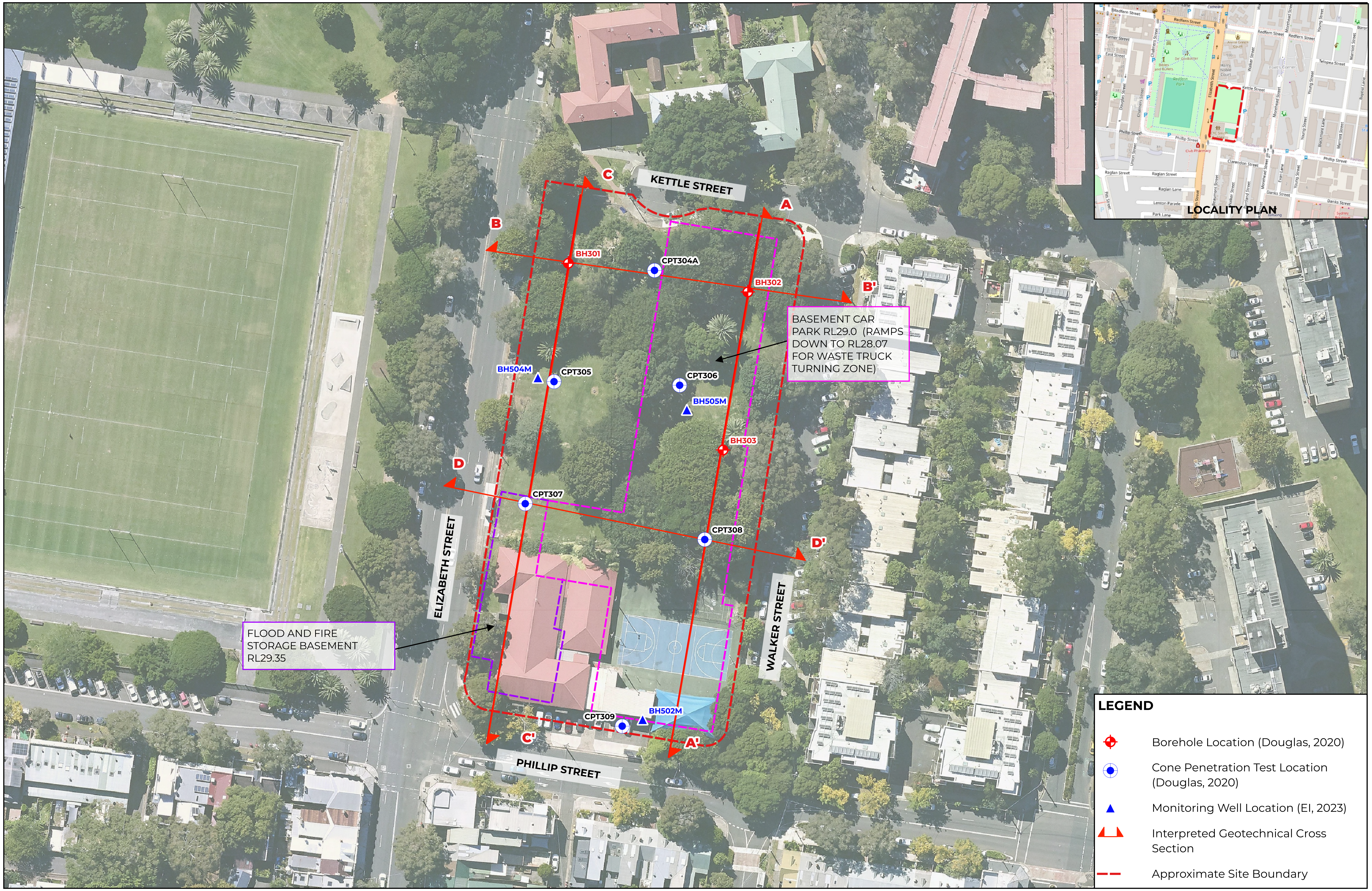
Appendix B

Drawings



LEGEND

⊕	Borehole Location (Douglas, 2020)
◉	Cone Penetration Test Location (Douglas, 2020)
▲	Monitoring Well Location (EI, 2023)
⊔	Interpreted Geotechnical Cross Section
– –	Approximate Site Boundary

REV	DESCRIPTION/COMMENT	DATE	DRAWN BY
0	Prelim Draft	04.06.2024	MN/EC

SCALE:
0 10 20 30 40 m
1:1000 @ A3

N

Douglas
PARTNERS
OFFICE: SYDNEY
96-98 Hermitage Rd, West Ryde NSW 2114
(02)9809 0666

CLIENT:
Bridge Housing Ltd

NOTE:
1: Basemap from Metromap (Dated 25.03.2024)
2: Site Boundary and Basement outline from Hayball, Project No. 2610, Drawing No. A02.00, Revision DRP4 (09.04.2024)

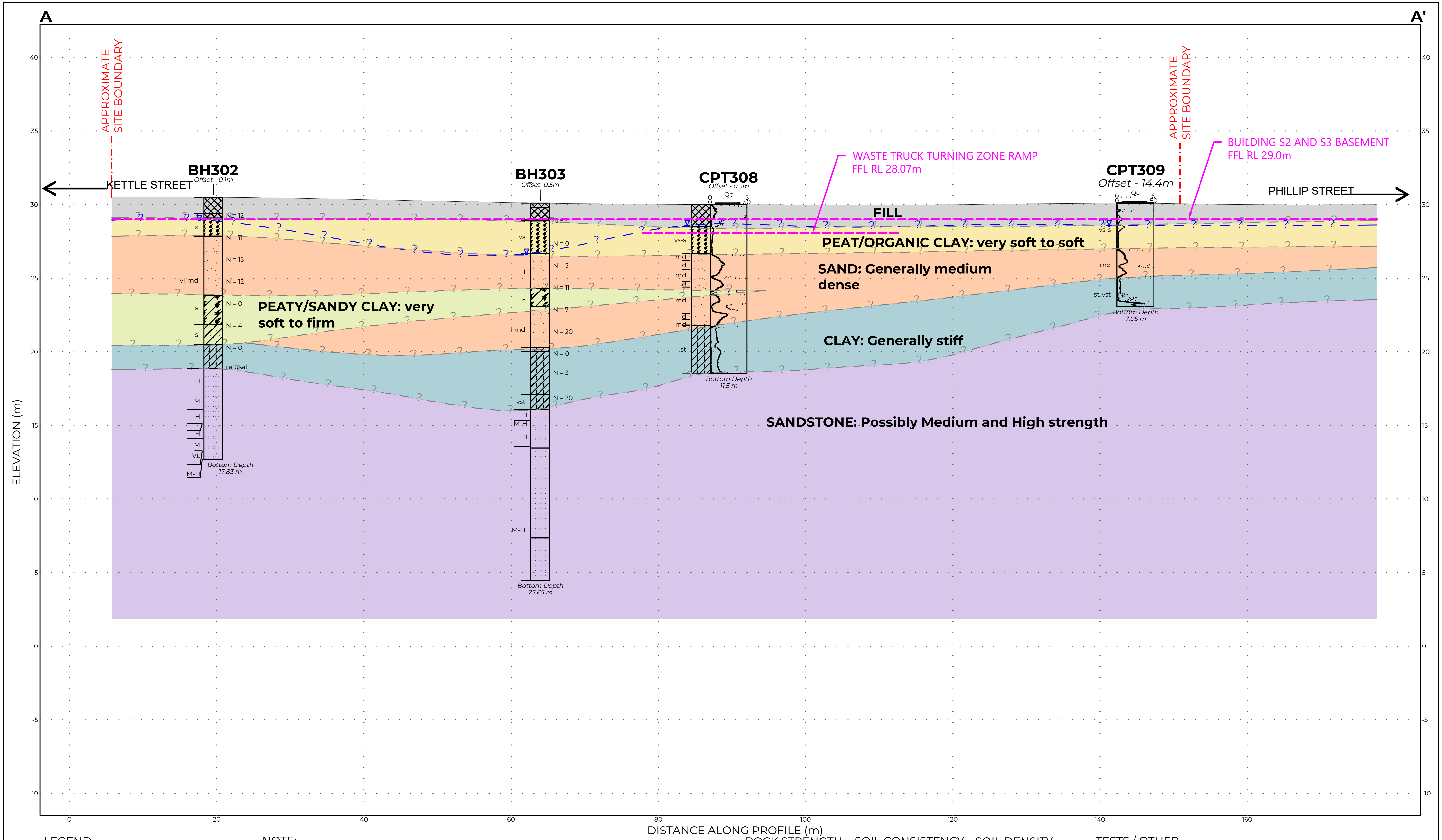
COORDINATE REFERENCE SYSTEM: GDA94 / MGA zone 56

PROJECT NAME:
Proposed Redfern Place

PROJECT ADRESS:
600-660 Elizabeth Street, Redfern

DRAWING TITLE:
TEST LOCATION PLAN

PROJECT NO: **99510.02**
DRAWING NO: **1**
REVISION: **0**



LEGEND

☒ Core Loss
▩ Filling
▥ Peat
▨ Peaty Clay
□ Sand
□ Sandstone
▨ Sandy Clay
▨ Silty Clay

NOTE:
1. Subsurface conditions are accurate at the borehole locations only.
Variations in subsurface conditions may occur between borehole locations.
Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs included only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal

ROCK STRENGTH
VL - Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

SOIL DENSITY
vl - Very Loose
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense

TESTS / OTHER
N - Standard penetration test value
— ? — — - Interpreted geotechnical boundary
▽ - Water level

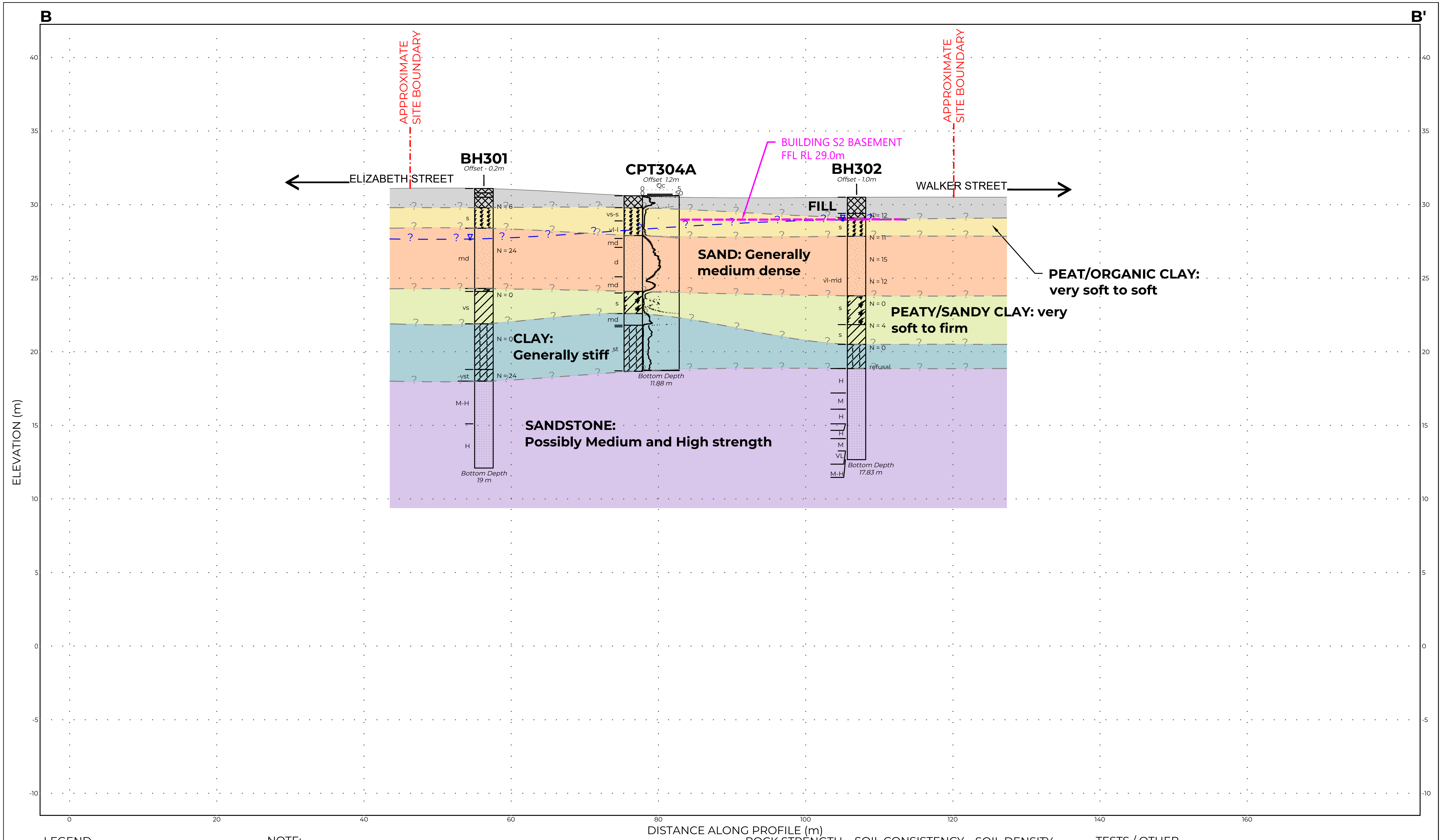
0 10
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas PARTNERS

CLIENT: Bridge Housing Ltd
OFFICE: Sydney
DRAWN BY: MN/EC
SCALE: 1:500 (H) 1:250 (V) @ A3
DATE: 06.06.2024

TITLE: **Interpreted Geotechnical Cross-Section A-A'**
Proposed Redfern Place
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.02
DRAWING No: 2
REVISION: 0



LEGEND
- ▨ Filling
- ▨ Peat
- ☐ Sand
- ▨ Peaty Clay
- ▨ Sandy Clay
- ▨ Silty Clay
- ☐ Sandstone

NOTE:
1. Subsurface conditions are accurate at the borehole locations only. Variations in subsurface conditions may occur between borehole locations. Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs included only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal

ROCK STRENGTH
VL- Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

SOIL DENSITY
vl - Very Loose
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense

TESTS / OTHER
N - Standard penetration test value
— ? — — - Interpreted geotechnical boundary
▽ - Water level

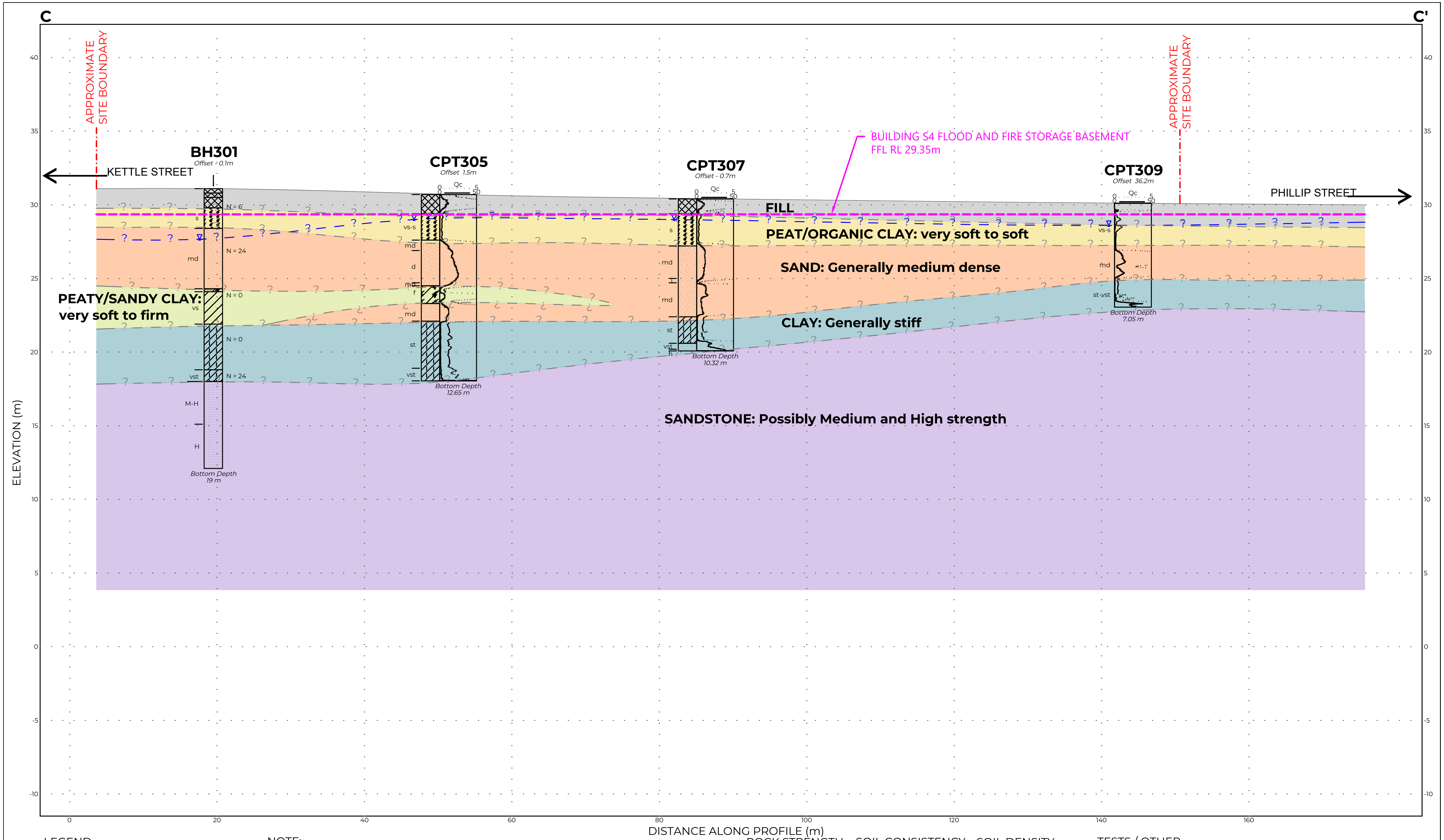
0 10
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas PARTNERS

CLIENT: Bridge Housing Ltd
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:500 (H) 1:250 (V) @ A3
DATE: 06.06.2024

TITLE: **Interpreted Geotechnical Cross-Section B-B'**
Proposed Redfern Place
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.02
DRAWING No: 3
REVISION: 0



LEGEND

☒ Core Loss
▩ Filling
▥ Peat
▨ Peaty Clay
░ Sand
░ Sandstone
▨ Sandy Clay
▩ Silty Clay

NOTE:
1. Subsurface conditions are accurate at the borehole locations only.
Variations in subsurface conditions may occur between borehole locations.
Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs included only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal

ROCK STRENGTH
VL- Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

SOIL DENSITY
vl - Very Loose
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense

TESTS / OTHER
N - Standard penetration test value
— ? — — - Interpreted geotechnical boundary
⊻ - Water level

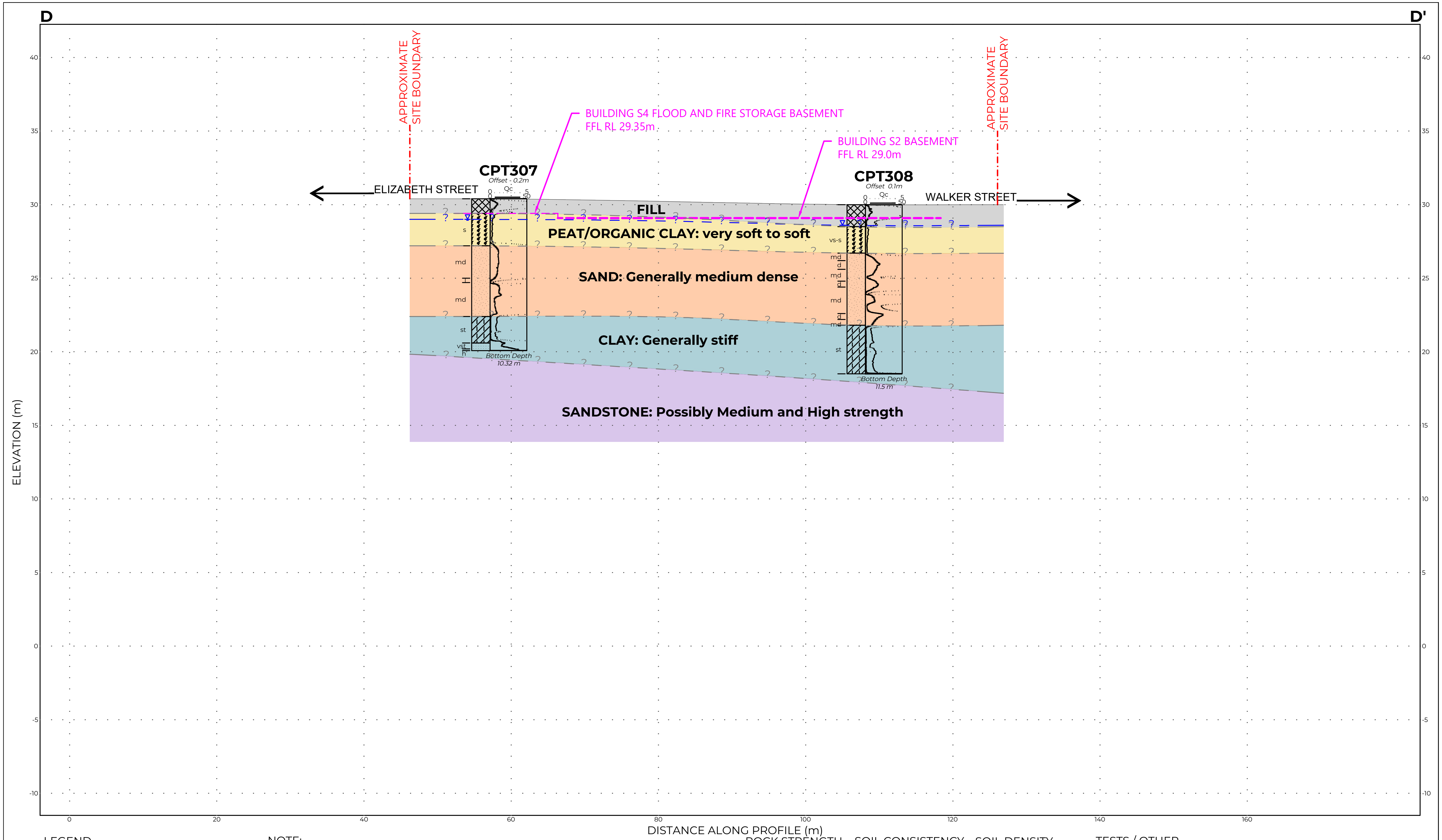
0 10
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas
PARTNERS

CLIENT: Bridge Housing Ltd
OFFICE: Sydney
DRAWN BY: MN/EC
SCALE: 1:500 (H) 1:250 (V) @ A3
DATE: 06.06.2024

TITLE: **Interpreted Geotechnical Cross-Section C-C'**
Proposed Redfern Place
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.02
DRAWING No: 4
REVISION: 0



LEGEND

▩ Filling	☐ Sandstone
▥ Peat	
⬚ Sand	
▨ Silty Clay	

NOTE:
1. Subsurface conditions are accurate at the borehole locations only.
Variations in subsurface conditions may occur between borehole locations.
Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs included only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal

ROCK STRENGTH
VL - Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

SOIL DENSITY
vl - Very Loose
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense

TESTS / OTHER
N - Standard penetration test value
— ? — — - Interpreted geotechnical boundary
▽ - Water level

0 10
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas PARTNERS

CLIENT: Bridge Housing Ltd
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:500 (H) 1:250 (V) @ A3
DATE: 06.06.2024

TITLE: **Interpreted Geotechnical Cross-Section D-D'**
Proposed Redfern Place
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.02
DRAWING No: 5
REVISION: 0

Project Title

ELIZABETH STREET, REDFERN
600-660 Elizabeth St, Redfern NSW 2016

haybball

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Canberra
Level 1,
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Canberra ACT 2601
T +61 2 9660 9329

ABN: 84006394261 NSW Nominated Architects: David Tordoff 8028

Drawn By JC
Checked By DT
Date Printed 12/04/2024 9:51:36 AM
Scale 1 : 250@ A1

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Rev	Date	Description
1	04/03/2024	Issued for review
2	05/03/2024	Issued for Information
DRP1	14/11/2023	Issued for DRP1
DRP2	29/01/2024	Issued for DRP2
DRP3	08/03/2024	Issued for DRP3
DRP4	09/04/2024	Issued for DRP4

Drawing Title
BASEMENT PLAN

Status
SKETCH DESIGN

Project No
2610

Drawing No
A02.00

Revision
DRP4

Verify all figured dimensions on site before undertaking any works. Do not scale dimensions off drawings.

Project Title

ELIZABETH STREET, REDFERN
600-660 Elizabeth St, Redfern NSW 2016

Melbourne
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ABN: 84006394261 NSW Nominated Architects: David Tordoff 8028

Drawn By JC
Checked By DT
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2	05/03/2024	Issued for Information
DRP1	14/11/2023	Issued for DRP1
DRP3	08/03/2024	Issued for DRP3
DRP4	09/04/2024	Issued for DRP4

Drawing Title
MASTERPLAN NEW

Status
SKETCH DESIGN

Project No
2610

Revision
DRP4

Drawing No
A02.01

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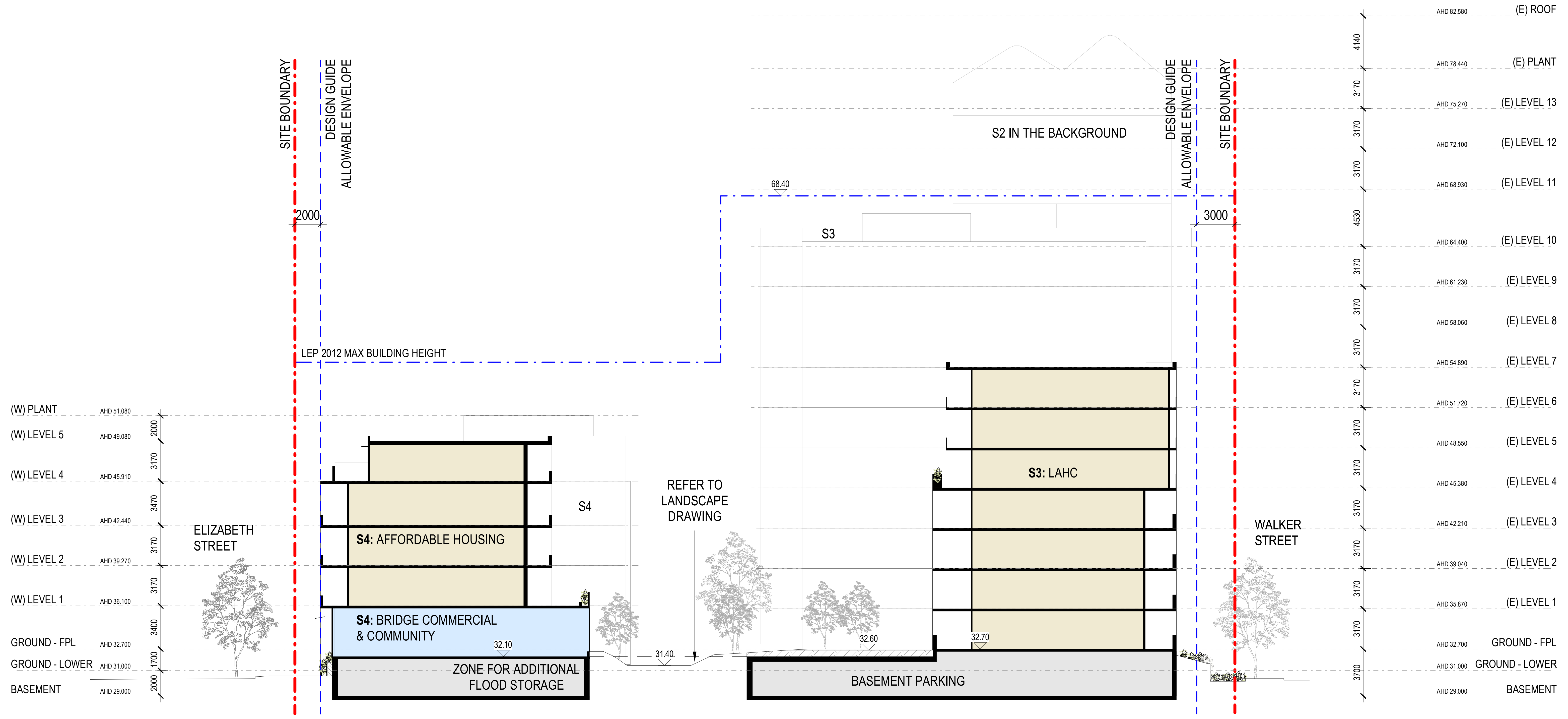

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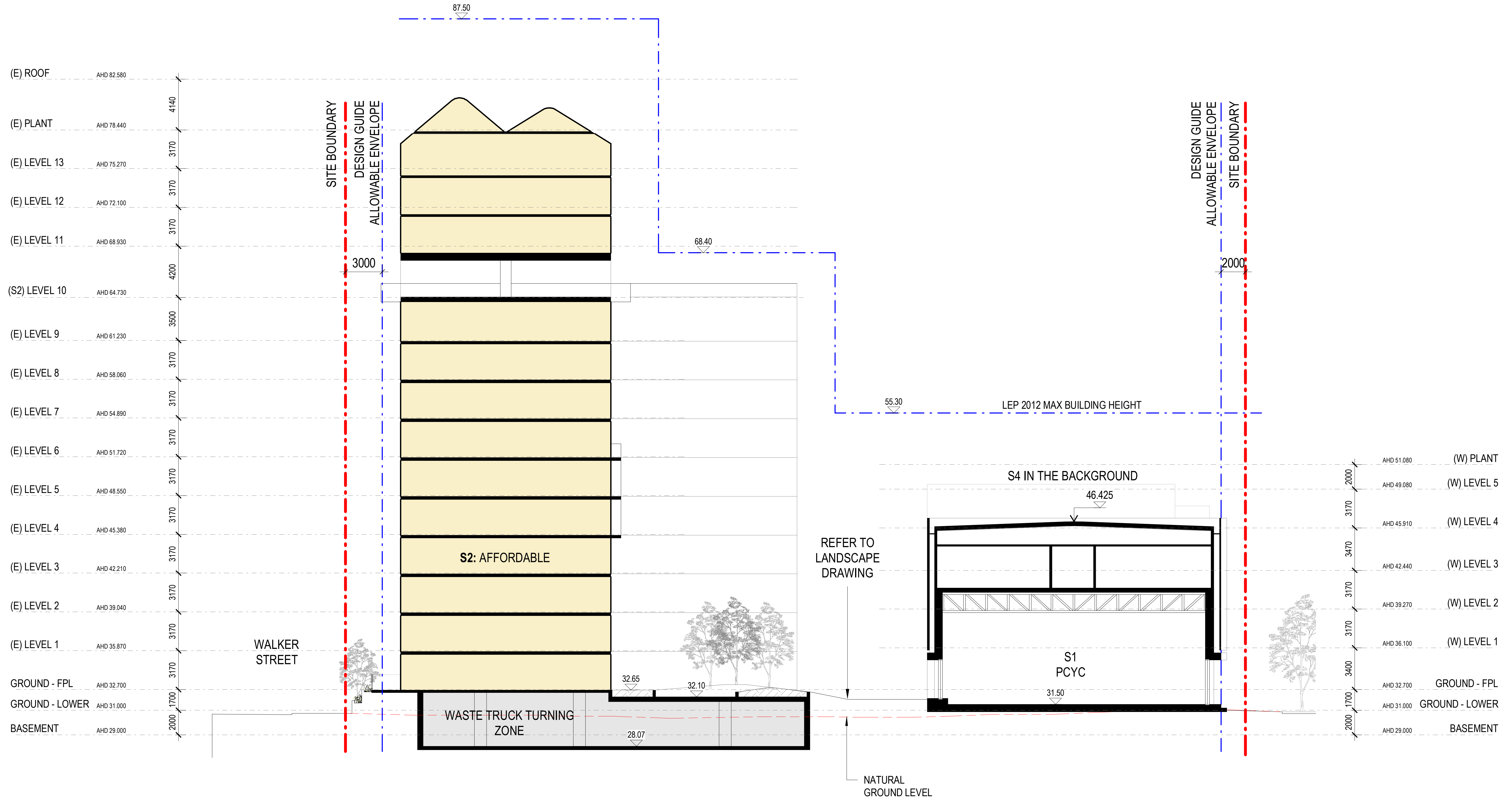

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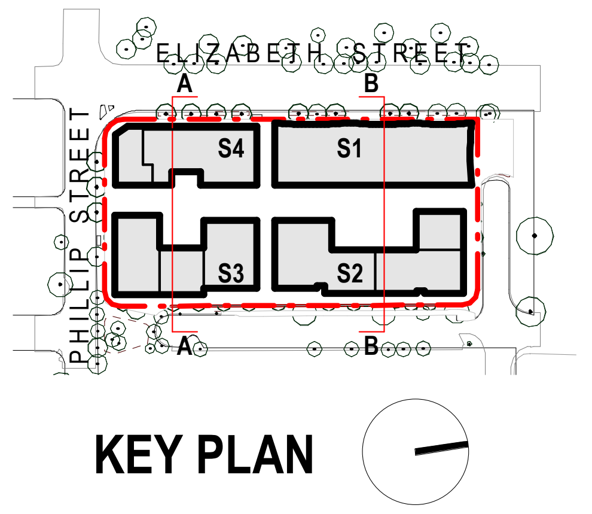

KEY PLAN

Project Title

ELIZABETH STREET, REDFERN
600-660 Elizabeth St, Redfern NSW 2016

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Sydney
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ABN: 84006394261 NSW Nominated Architects: David Tordoff 8028

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DRP2	29/01/2024	Issued for DRP2
DRP3	08/03/2024	Issued for DRP3
DRP4	09/04/2024	Issued for DRP4

Drawing Title
SITE SECTIONS

Status
SKETCH DESIGN

Project No
2610

Revision
DRP4

Drawing No
A06.01

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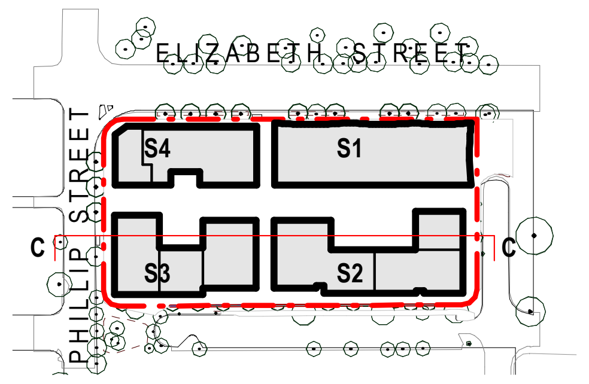

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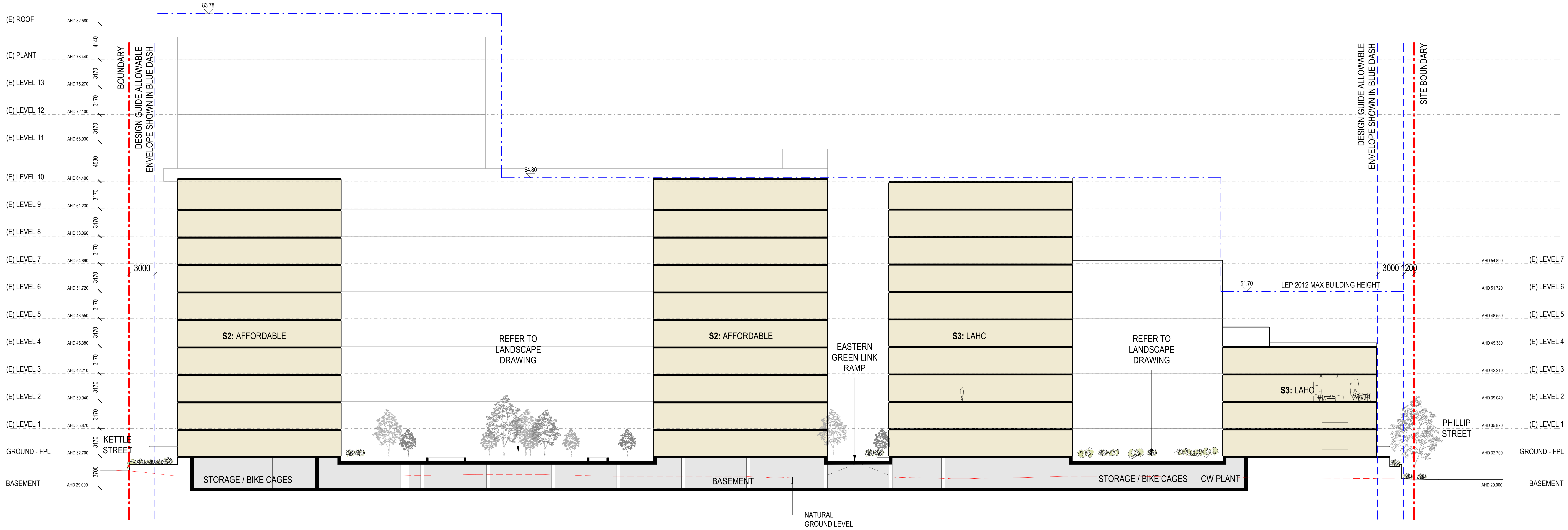

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Project Title

ELIZABETH STREET, REDFERN
600-660 Elizabeth St, Redfern NSW 2016

hayball

Melbourne
Level 1
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Melbourne VIC 3000
T +61 3 9699 3644

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ABN: 84006394261 NSW Nominated Architects: David Tordoff 8028

Drawn By JC
Checked By DT
Date Printed 12/04/2024 9:52:03 AM
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Rev	Date	Description
DRP1	14/11/2023	Issued for DRP1
DRP2	29/01/2024	Issued for DRP2
DRP3	08/03/2024	Issued for DRP3
DRP4	09/04/2024	Issued for DRP4

Drawing Title
SITE SECTIONS

Status
SKETCH DESIGN

Project No
2610

Revision
DRP4

Drawing No
A06.02

Appendix C

Results of Previous Field Work

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.

Description and Classification Methods

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

The soil group symbol classifications are given as follows based on two major soil divisions:

- Coarse-grained soils
- Fine-grained soils

Major Divisions				Description	
				Group Symbol*	Typical Name
COARSE-GRAINED SOILS	More than 65% by dry mass, (excluding that larger than 63 mm) is greater than 0.075 mm	GRAVEL	More than 50% of coarse grains are greater than 2.36 mm	GW	Well graded gravels and gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
		GRAVELLY SOILS		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clay gravels, gravel-sand-clay mixtures.
		SAND	More than 50% of coarse grains are less than 2.36 mm	SW	Well graded sands and gravelly sands, little or no fines.
				SP	Poorly graded sands and gravelly sands, little or no fines.
		SANDY SOILS		SM	Silty sand, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures.

* For coarse grained soils where the fines content is between 5% and 12%, the soil shall be given a dual classification eg GP-GM.

FINE-GRAINED SOILS	More than 35% by dry mass, (excluding that larger than 63 mm) is less than 0.075 mm	Liquid Limit less than 35%	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity
		35% <LL< 50%	CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
			Pt	Peat muck and other highly organic soils.

Soil Types

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

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

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

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

Definitions of grading terms used are:

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

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

In fine grained soils (>35% fines)

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

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

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

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

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

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

Soil Descriptions

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Origin

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

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

Moisture Condition – Coarse Grained Soils

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

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

Moisture Condition – Fine Grained Soils

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

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

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it 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 Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

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

* 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
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

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 occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

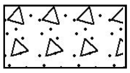
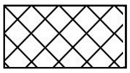
fg	fragmented
bnd	band
qtz	quartz

May 2017

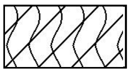
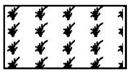
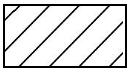
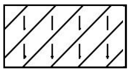
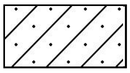
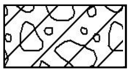
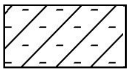
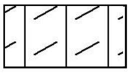
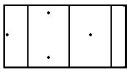
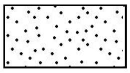
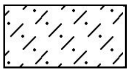
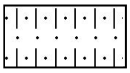
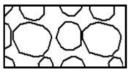
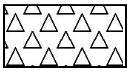
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

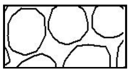
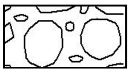
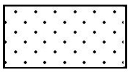
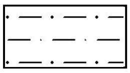
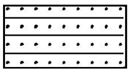
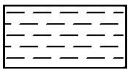
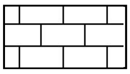
General

■	Asphalt
	Road base
	Concrete
	Filling

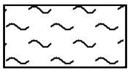
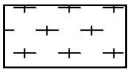
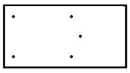
Soils

	Topsoil
	Peat
	Clay
	Silty clay
	Sandy clay
	Gravelly clay
	Shaly clay
	Silt
	Clayey silt
	Sandy silt
	Sand
	Clayey sand
	Silty sand
	Gravel
	Sandy gravel
	Cobbles, boulders
	Talus

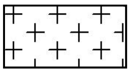
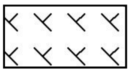
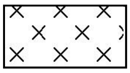
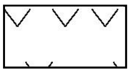
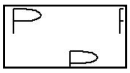
Sedimentary Rocks

	Boulder conglomerate
	Conglomerate
	Conglomeratic sandstone
	Sandstone
	Siltstone
	Laminite
	Mudstone, claystone, shale
■	Coal
	Limestone

Metamorphic Rocks

	Slate, phyllite, schist
	Gneiss
	Quartzite

Igneous Rocks

	Granite
	Dolerite, basalt, andesite
	Dacite, epidote
	Tuff, breccia
	Porphyry

Cone Penetration Tests

Introduction

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

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

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

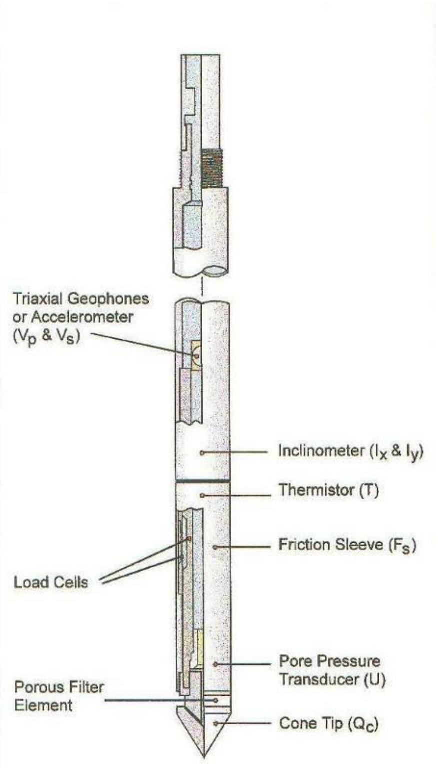


Figure 1: Cone Diagram

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

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

Figure 2: Purpose built CPT rig

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

Types of CPTs

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

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

Strata Interpretation

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

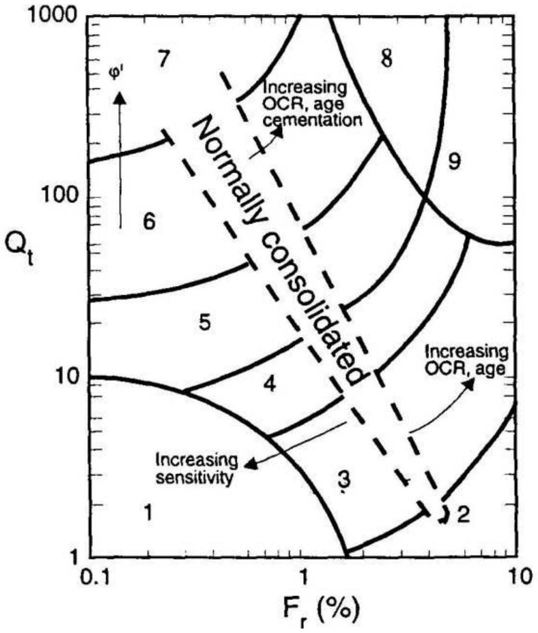


Figure 3: Soil Classification Chart

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

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

Engineering Applications

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

Settlement

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

Pile Capacity

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

Dynamic or Earthquake Analysis

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

Other Applications

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

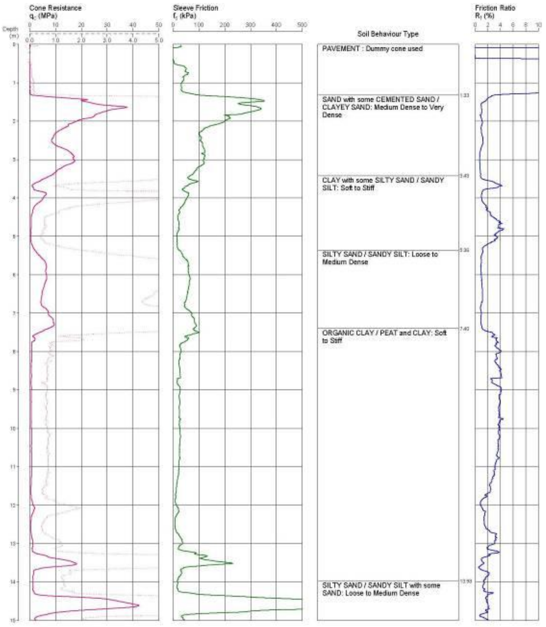


Figure 4: Sample Cone Plot

CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.6

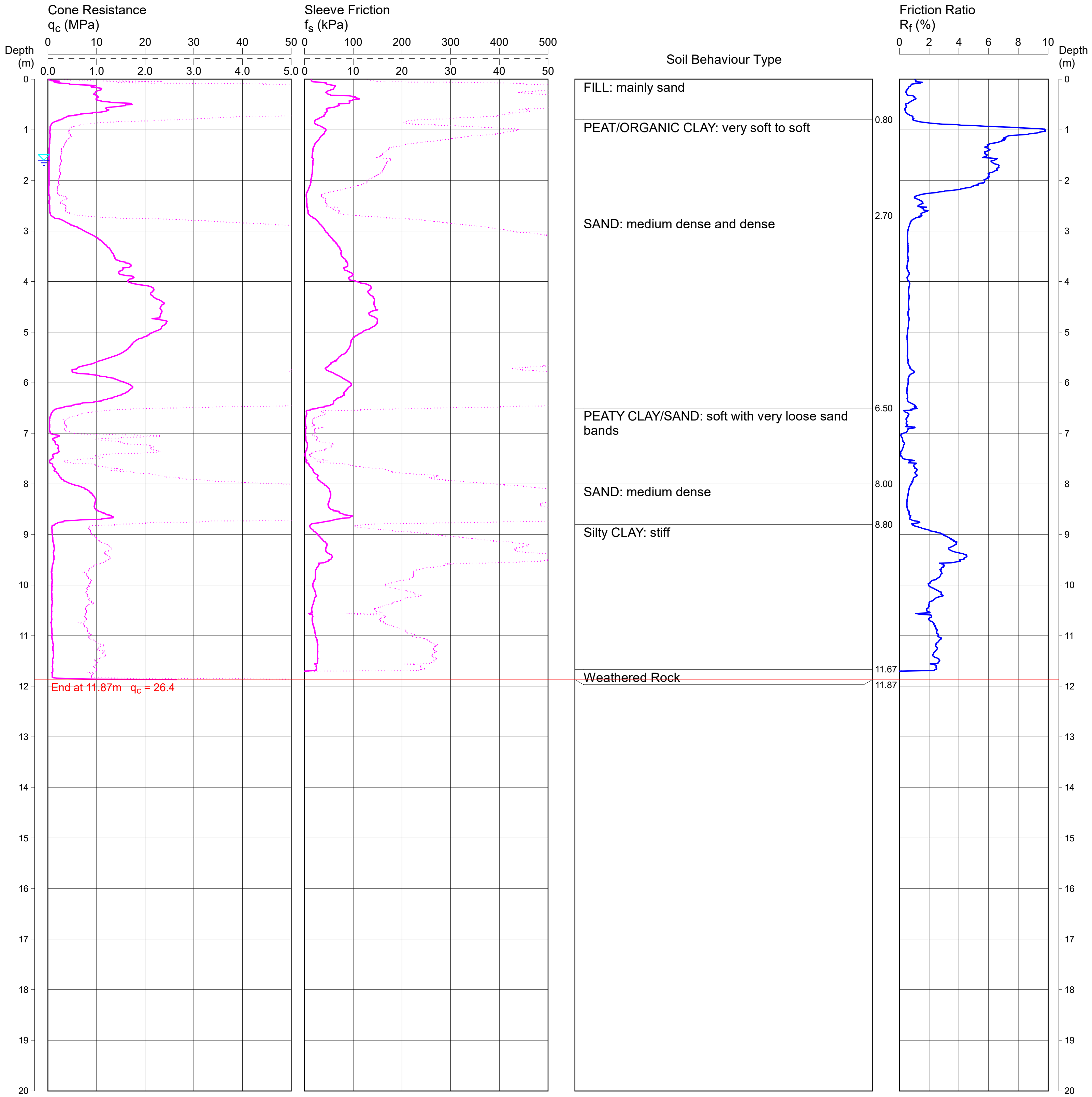
COORDINATES: 334250E 6248044N

CPT 304A

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Groundwater measured at 1.6m deep

Water depth after test: 1.60m depth (measured)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-304A.CP5
Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.7

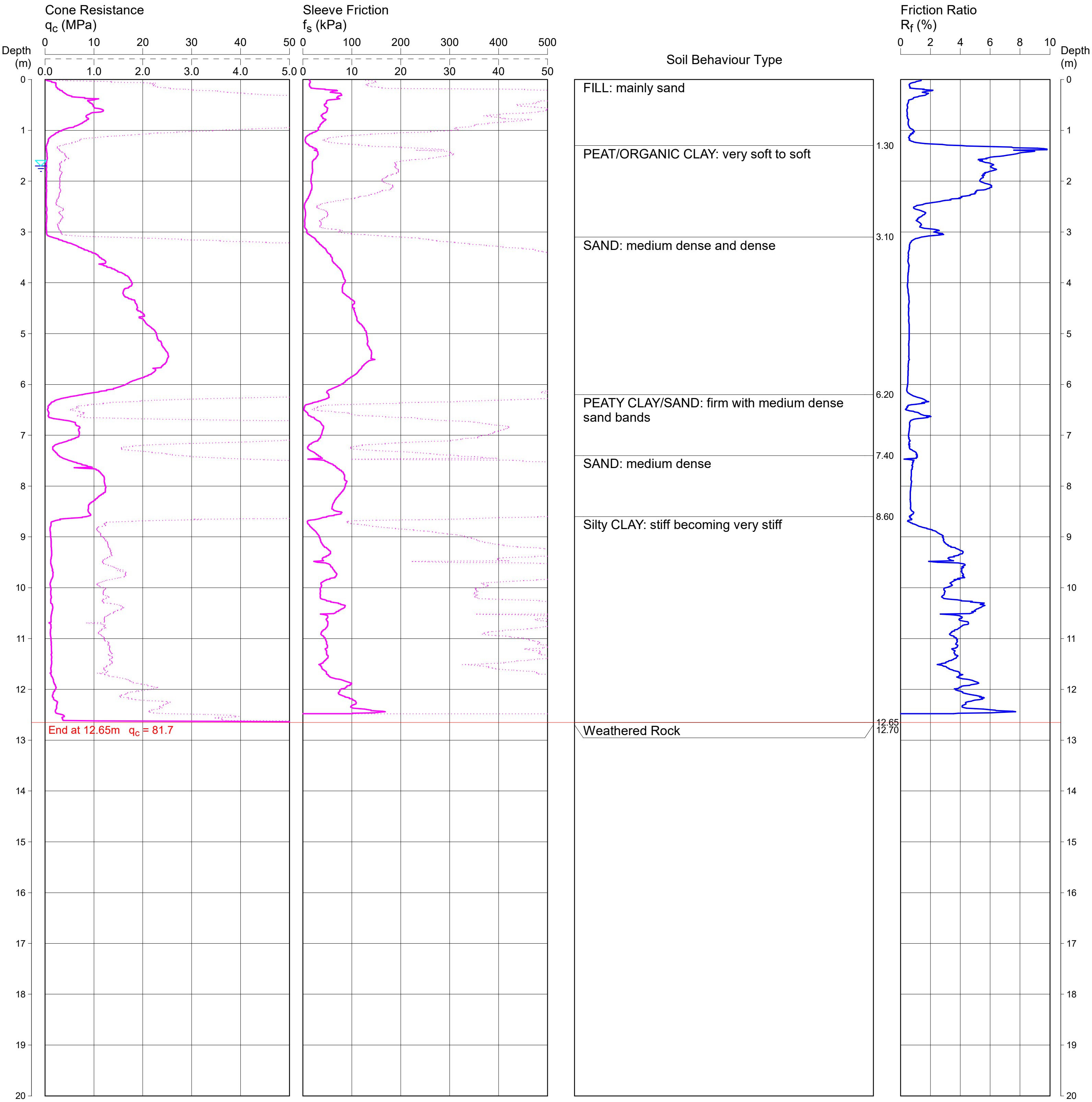
COORDINATES: 334222E 6248013N

CPT 305

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Groundwater measured at 1.7m deep

Water depth after test: 1.70m depth (measured)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-305.CP5

Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

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Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.4

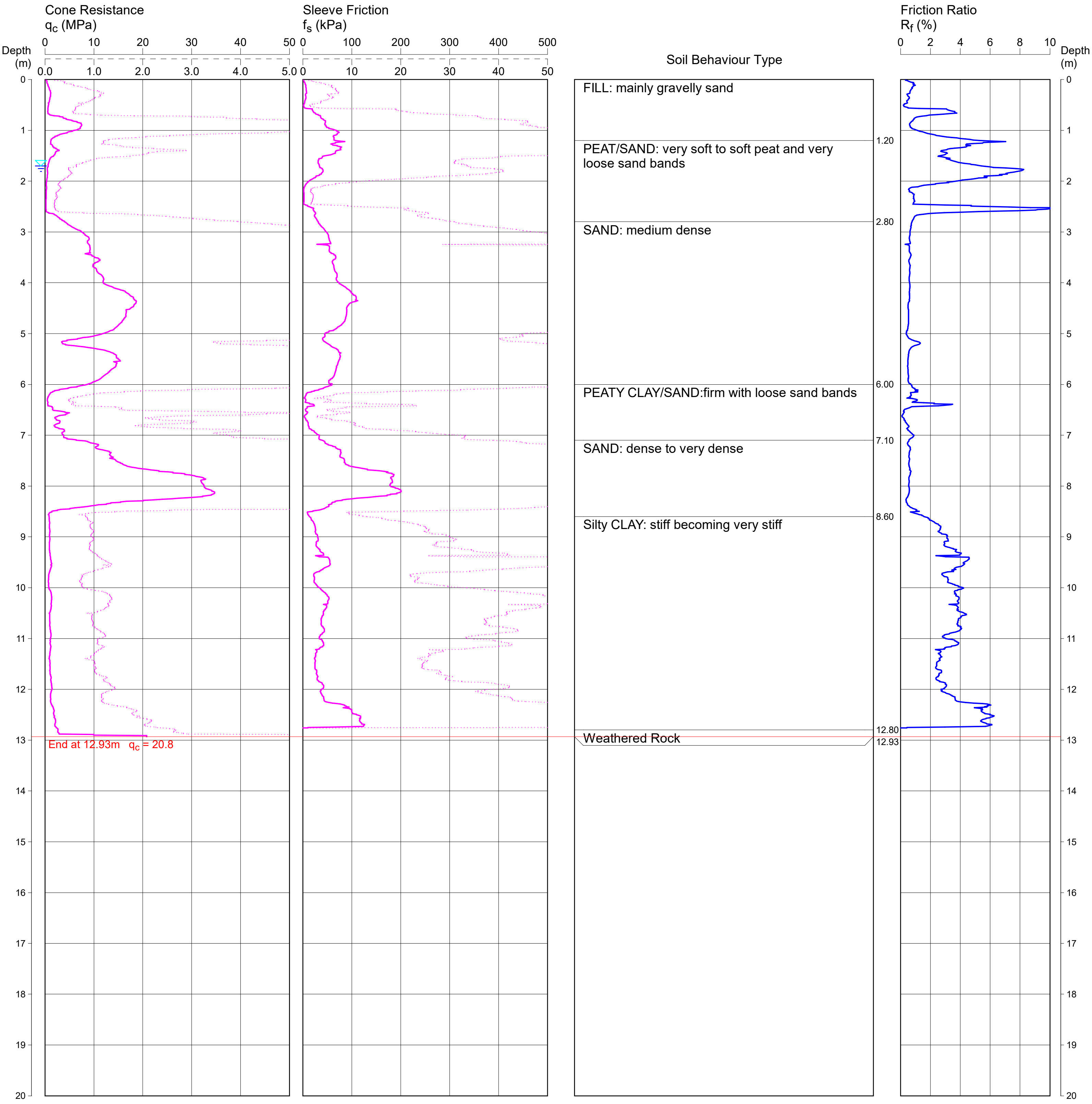
COORDINATES: 334257E 6248012N

CPT 306

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Groundwater measured at 1.7m deep

Water depth after test: 1.70m depth (measured)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-306.CP5
Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.4

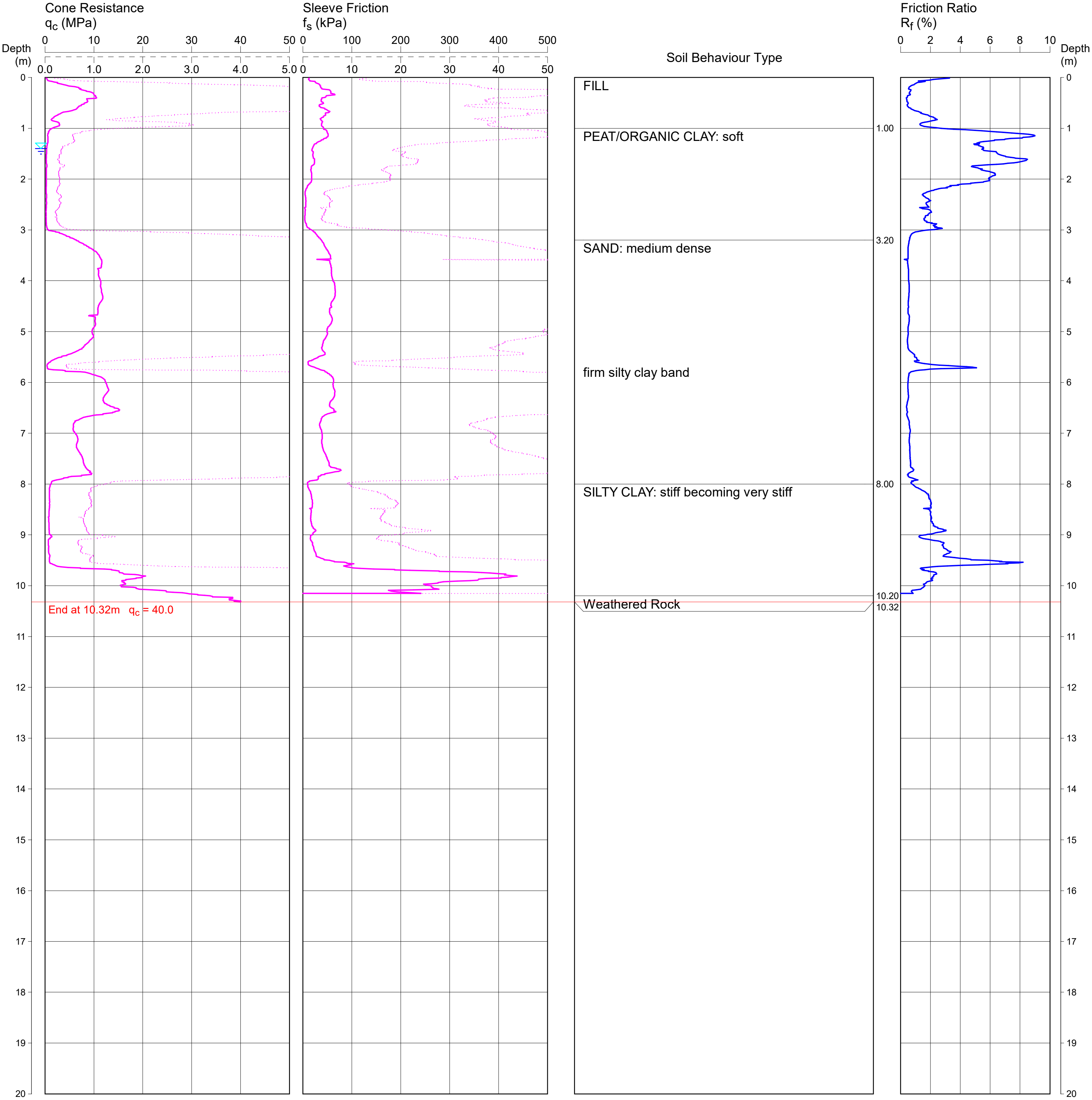
COORDINATES: 334214E 6247979N

CPT307

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Groundwater measured at 1.4m deep

Water depth after test: 1.40m depth (measured)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-307.CP5
Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.0

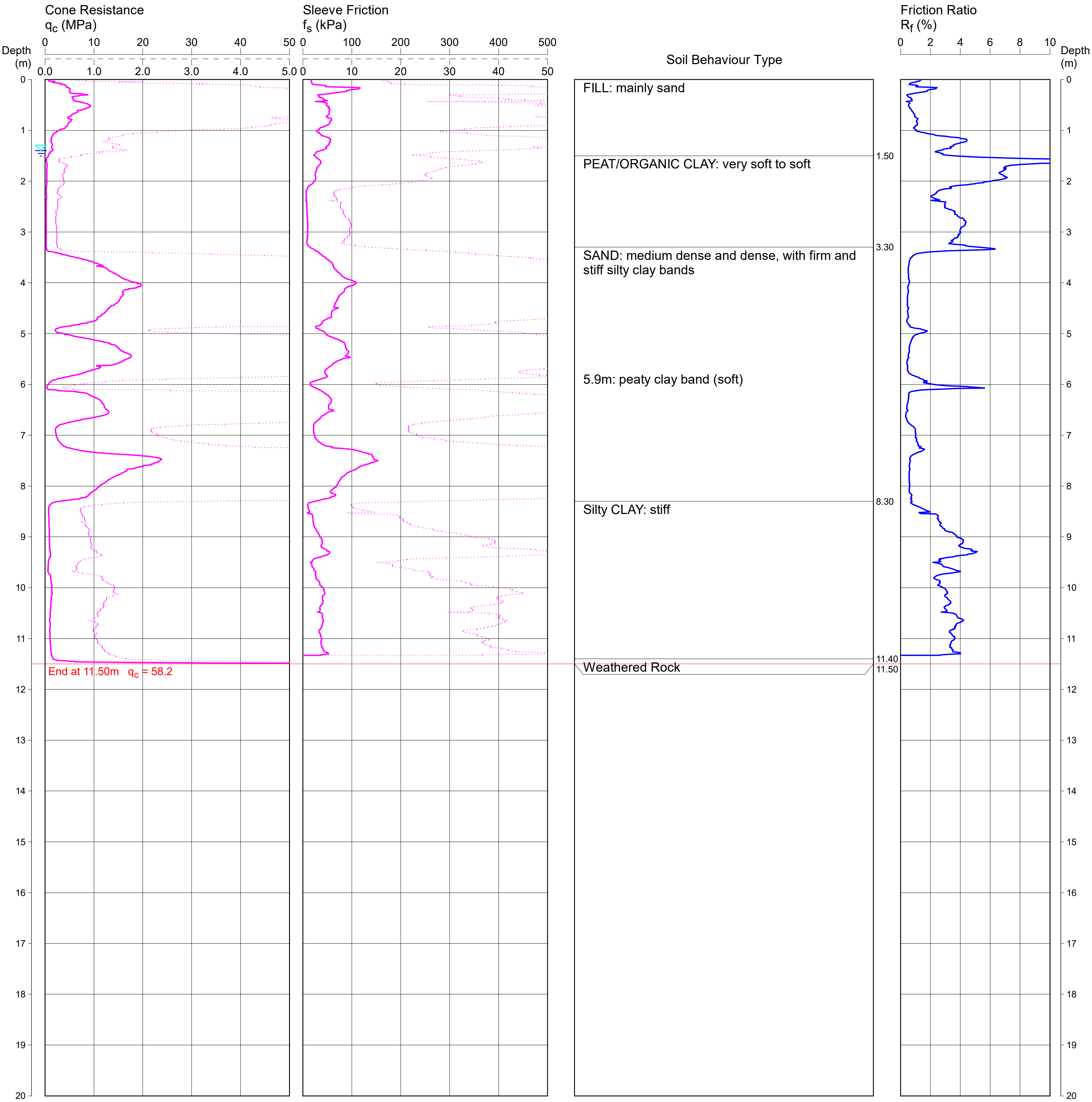
COORDINATES: 334264E 6247969N

CPT 308

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Groundwater mesured at 1.4m deep

Water depth after test: 1.40m depth (measured)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-308.CP5
Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.1

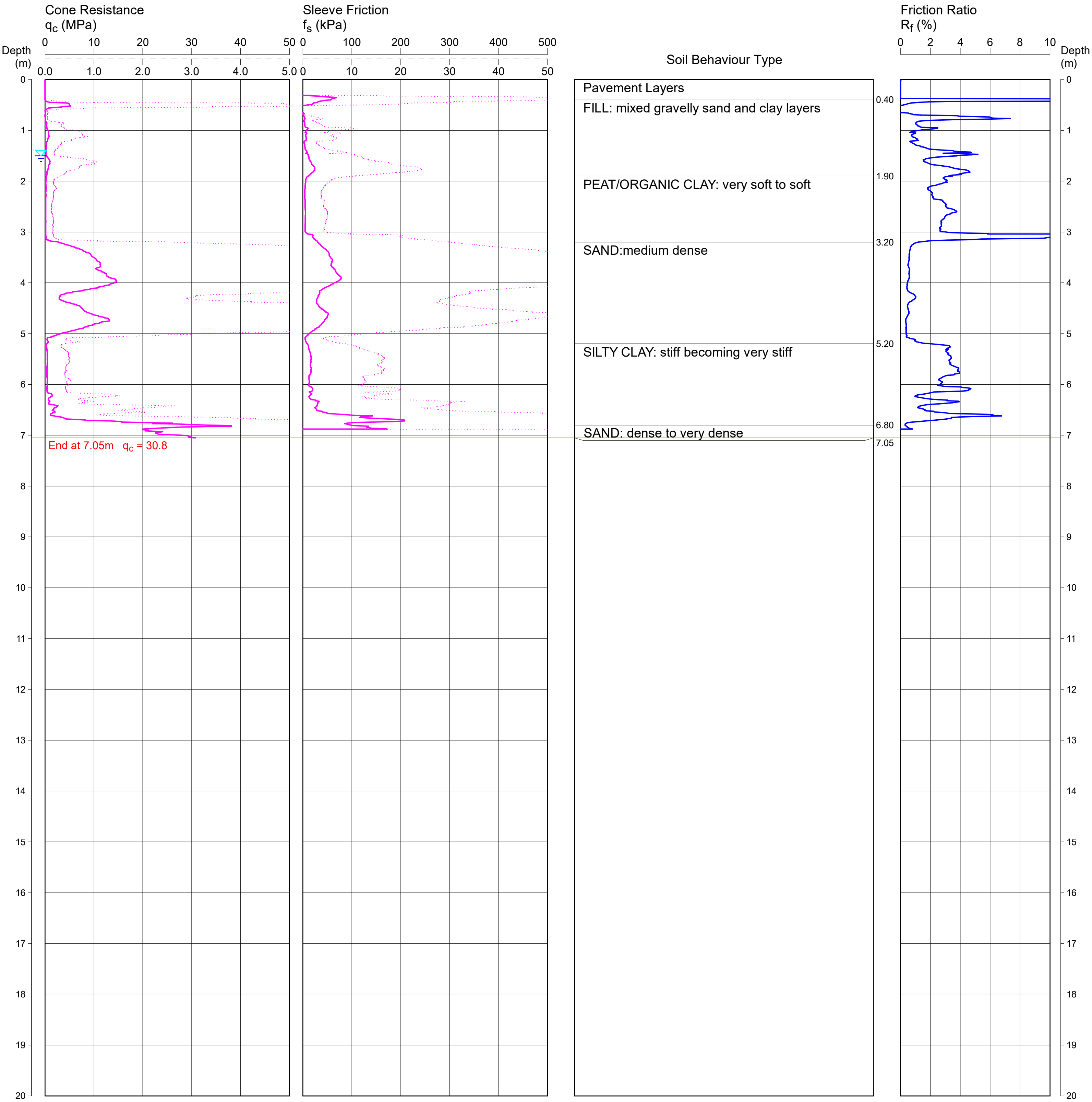
COORDINATES: 334240.8E 6247916.9N

CPT 309

Page 1 of 1

DATE 09/12/2019

PROJECT No: 99510.00



REMARKS: Dummy Cone to 0.4m deep. Test in Asphaltic Concrete Pavement.
Groundwater measured at 1.5m deep

Water depth after test: 1.50m depth (assumed)

File: P:\99510.00 - REDFERN, 600-660 Elizabeth Street, Geo\4.0 Field Work\4.2 Testing\CPTs\3- Cone Plot Files\99510 - CPT-309.CP5
Cone ID: Uni Newc **Type:** 2 Standard

ConePlot Version 5.9.2

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 31.1
EASTING: 334226
NORTHING: 6248046
DIP/AZIMUTH: 90°/--

BORE No: BH301
PROJECT No: 99510.00
DATE: 4/12/2019
SHEET 1 OF 2

Depth (m)	Description of Strata	Water	Type	Test Results & Comments
	FILL/SAND: fine to medium grained, pale brown, trace gravel, wet		A	
0.3	FILL/SAND: fine to medium grained, dark grey, trace gravel and brick fragments, wet		A	
0.6	FILL/SAND: fine to medium grained, pale brown, trace clay, wet		A S	2,4,2 N = 6 last spt number in peat layer
1.3	PEAT: dark grey, with organics and wood fragments, wet, soft, alluvial			
2.7	SAND (SP):fine to medium grained, pale brown, with interbedded peat bands, wet, medium dense, alluvial	▼	S	2,9,15 N = 24
6.8	PEATY CLAY: soft			
7.0	Sandy CLAY (CH): medium to high plasticity, grey, trace rootlets, w>LL		S	0,0,0 N = 0
9.2	Silty CLAY (CH): high plasticity, grey, with sand, w>LL, possibly residual			

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

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** RB **CASING:** HW to 11.5 m
TYPE OF BORING: Solid Flight Augering to 3.5 m, Rotary Drilling to 13.1 m, NMLC coing to 19.0 m
WATER OBSERVATIONS: 3.5 m
REMARKS: *Probably affected by drilling method

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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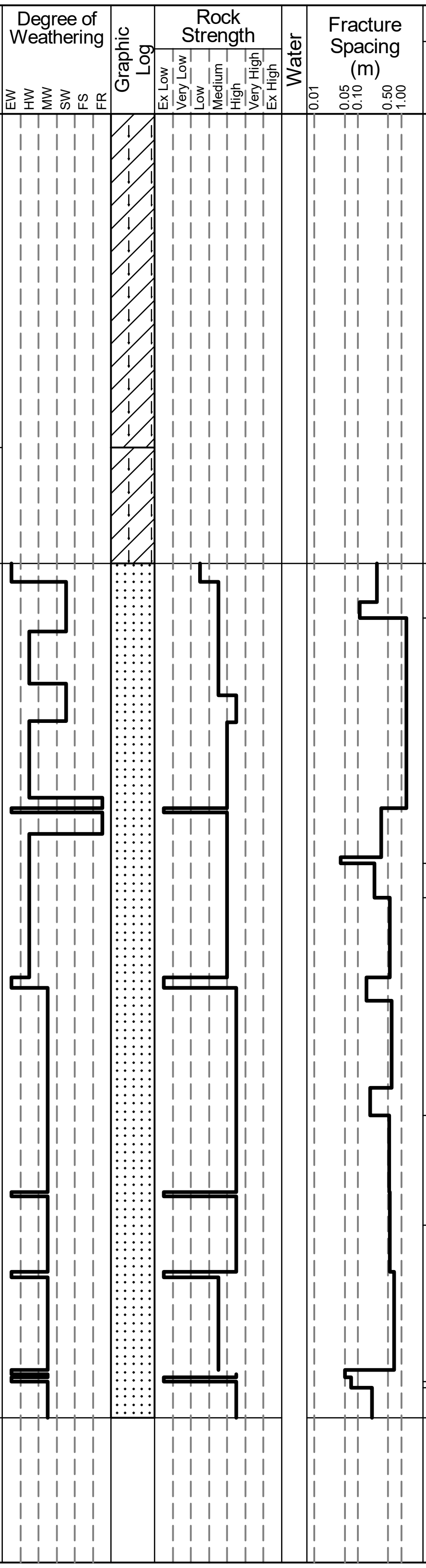
BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 31.1
EASTING: 334226
NORTHING: 6248046
DIP/AZIMUTH: 90°/--

BORE No: BH301
PROJECT No: 99510.00
DATE: 4/12/2019
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
21		Silty CLAY (CH): high plasticity, grey, with sand, w>LL, possibly residual *(continued)*		S			0,0,0 N = 0 suspect results*
20	11						
19	12						
	12.3	Silty CLAY (CH): high plasticity, grey, with sand, w>LL, very stiff, residual		S			7,10,14 N = 24
18	13						
	13.1	SANDSTONE: fine to medium grained, red brown pale brown and grey, high strength then medium to high strength and then high strength, highly weathered then moderately weathered, slightly fractured, Hawksebury sandstone	13.37m: J, 60°, pl, ro, cln 13.48m: B, 0°, pl, cly vn, fe	C	100	91	PL(D) = 0.2 PL(D) = 1.2
17	14		14.79-14.82m: Cs,30mm				PL(D) = 1.3 PL(D) = 0.6
16	15		15.13m: B, 5°, cu, fe, tight 15.17m: B, 15°, cu, fe, tight 15.41m: B, 10°, cu, fe, tight 15.96-16.03: Ds, 70mm				PL(D) = 1.2 PL(D) = 1.2
15	16		16.12m: J, 30°, pl, fe, cly vn 16.72m: B, 0°, pl, fe, tight 16.91m: B, 0°, pl, cly 4mm	C	100	89	
14	17		17.44-17.47m: Cs, 30mm 17.67-17.70m: Cs, 30mm 17.99-18.03m: Cs, 30mm				PL(D) = 1.3 PL(D) = 1
13	18		18.72-18.75m: Cs, 30mm 18.75m: J, 60°, pl, fe 18.79m: B, 15°, pl, fe, cly 2mm				PL(D) = 1 PL(D) = 1.5
12	19 19.0	Bore discontinued at 19.0m					

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** RB **CASING:** HW to 11.5 m

TYPE OF BORING: Solid Flight Augering to 3.5 m, Rotary Drilling to 13.1 m, NMLC coing to 19.0 m

WATER OBSERVATIONS: 3.5 m

REMARKS: *Probably affected by drilling method

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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BORE: 301 PROJECT: REDFERN DECEMBER 2019

13.1 m – 17.0 m

BORE: 301 PROJECT: REDFERN DECEMBER 2019

17.0 m – 19.0 m

BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 30.5
EASTING: 334276
NORTHING: 6248038
DIP/AZIMUTH: 90°/--

BORE No: BH302
PROJECT No: 99510.00
DATE: 2/12/2019
SHEET 1 OF 2

Depth (m)	Description of Strata	Water	Sampling Type	Test Results & Comments
0.0	FILL/Silty SAND: fine to medium grained, dark brown, with fine gravel and trace rootlets and brick fragments, wet		A	
			A	
			A	
1.1	FILL: SAND (SP): fine to medium grained, dark brown and grey, wet, medium dense, alluvial		S	5,7,5 N = 12 last spt number in peat layer
1.4	PEAT: dark grey, with organics and timber, wet, soft, alluvial 1.6 m: w>LL	▼		
2.65	SAND (SP):fine to medium grained, pale brown, with interbedded soft to firm peat bands, wet, medium dense, alluvial		S	1,5,6 N = 11
			S	3,7,8 N = 15
	5.7 to 5.8 m: Peat band		S	4,3,9 N = 12
6.7	PEATY CLAY/SAND: interbedded soft peaty clay and loose sand		S	3,0,0 N = 0
8.65	Silty CLAY (CH): high plasticity, grey, trace sand, w>LL, soft, possibly residual		S	3,2,2 N = 4
10.0				

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** ZH/RB **CASING:** HW to 4.4 m

TYPE OF BORING: Solid Flight Augering to 4.5 m, Rotary Drilling to 11.64 m, NMLC coing to 17.83 m

WATER OBSERVATIONS: 1.6 m

REMARKS: *Probably affected by drilling method
No Sample recovered from SPT at depth 11.5 m - 11.55 m.

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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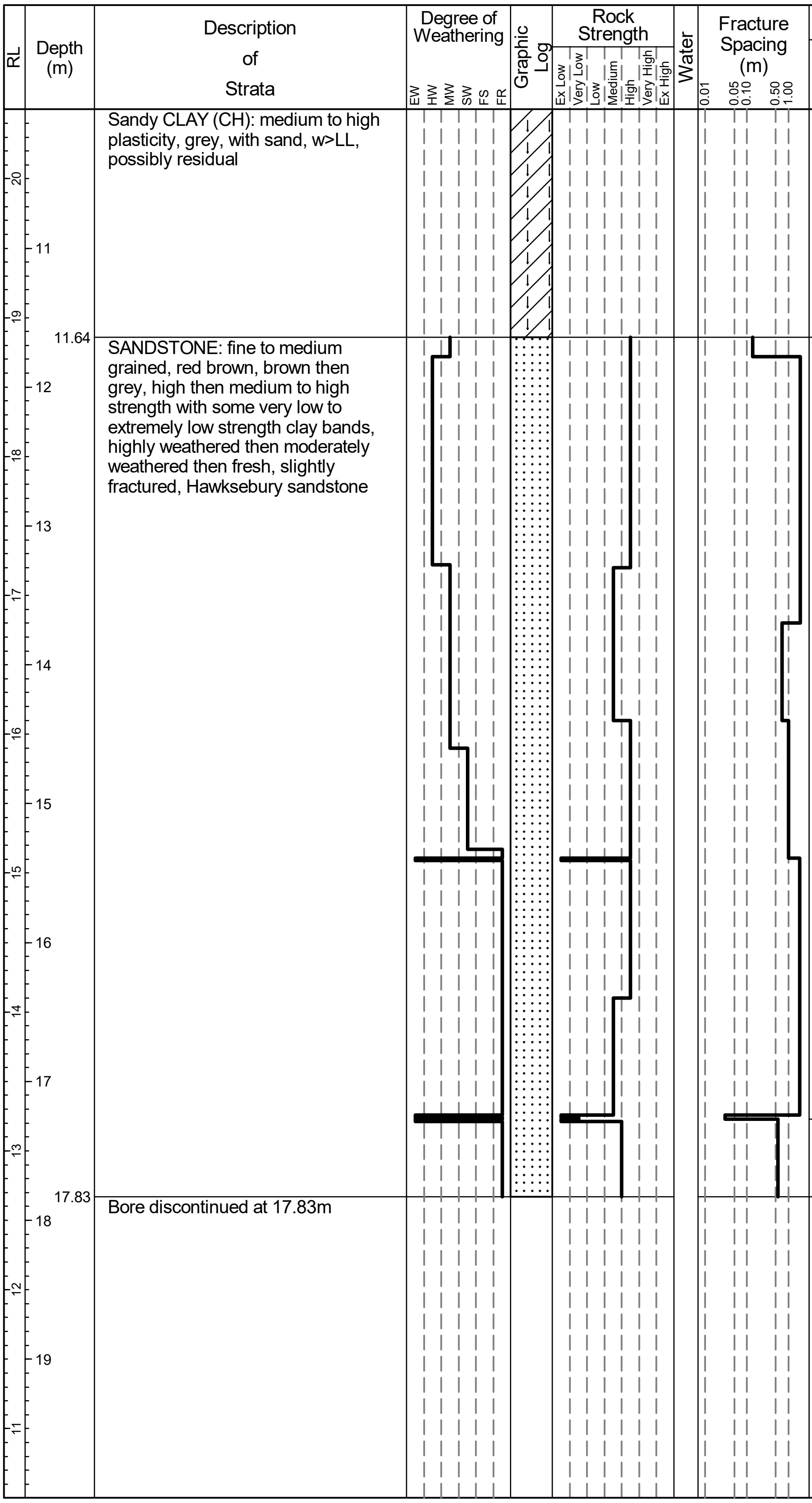
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 30.5
EASTING: 334276
NORTHING: 6248038
DIP/AZIMUTH: 90°/--

BORE No: BH302
PROJECT No: 99510.00
DATE: 2/12/2019
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
	Sandy CLAY (CH): medium to high plasticity, grey, with sand, w>LL, possibly residual		S			0,0,0 N = 0 suspect result*
			S			6/50 refusal
11.64	SANDSTONE: fine to medium grained, red brown, brown then grey, high then medium to high strength with some very low to extremely low strength clay bands, highly weathered then moderately weathered then fresh, slightly fractured, Hawksebury sandstone	11.78m: B, 0°, pl, ro, fe stn 13.7m: B, 5°, un, ro, cly vnr	C	100	100	PL(A) = 2 PL(A) = 1.2 PL(A) = 0.8 PL(A) = 1.1
		14.4m: B, 0°, cly 5mm, fe 15.39m: Cs, 20mm	C	100	99	PL(A) = 1 PL(A) = 1.4 PL(A) = 0.9 PL(A) = 1.1
		17.24m: Cs, 20mm 17.27m: Cs, 20mm	C	100	99	PL(A) = 1.2
17.83	Bore discontinued at 17.83m					

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** ZH/RB **CASING:** HW to 4.4 m

TYPE OF BORING: Solid Flight Augering to 4.5 m, Rotary Drilling to 11.64 m, NMLC coing to 17.83 m

WATER OBSERVATIONS: 1.6 m

REMARKS: *Probably affected by drilling method
No Sample recovered from SPT at depth 11.5 m - 11.55 m.

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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BORE: 302 PROJECT: REDFERN DECEMBER 2019

11.6 m – 16.0 m

BORE: 302 PROJECT: REDFERN DECEMBER 2019

16.0 m – 17.83 m

BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 30.1
EASTING: 334269.1
NORTHING: 6247994.1
DIP/AZIMUTH: 90°/--

BORE No: BH303
PROJECT No: 99510.00
DATE: 2 - 3/12/2019
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Water	Type	Test Results & Comments
30		FILL/Silty SAND: fine to medium grained, dark brown, with gravel, rootlets and brick fragments, wet		A	
	0.3	FILL/SAND: fine to medium grained, pale brown, wet brick fragments		A	
				A	
29	1.1	SAND (SP): fine to medium grained, pale brown, wet, loose, alluvial		S	2,2,2 N = 4
	1.2	PEAT: dark grey, with organics and timber, wet, very soft, alluvial			
28					
27				S	0,0,0 N = 0
	3.4	SAND (SP): fine to medium grained, pale brown, with interbedded peat bands, w>LL, loose to medium dense, alluvial	▼		
26				S	0,2,3 N = 5
25				S	5,3,8 N = 11
24	5.8	PEATY CLAY/SAND: interbedded soft peaty clay and loose sand			
23	7.0	SAND (SP): fine to medium grained, pale brown, with interbedded peat bands, w>LL, loose to medium dense, alluvial 7.5m: becoming dense		S	4,4,3 N = 7
22				S	8,13,7 N = 20
21					
	9.8	See description over page			

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** RB **CASING:** HW to 13 m
TYPE OF BORING: Solid Flight Augering to 3.5 m, Rotary Drilling to 14.0 m, NMLC coing to 25.65 m
WATER OBSERVATIONS: 3.5 m
REMARKS: *Probably affected by drilling method

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
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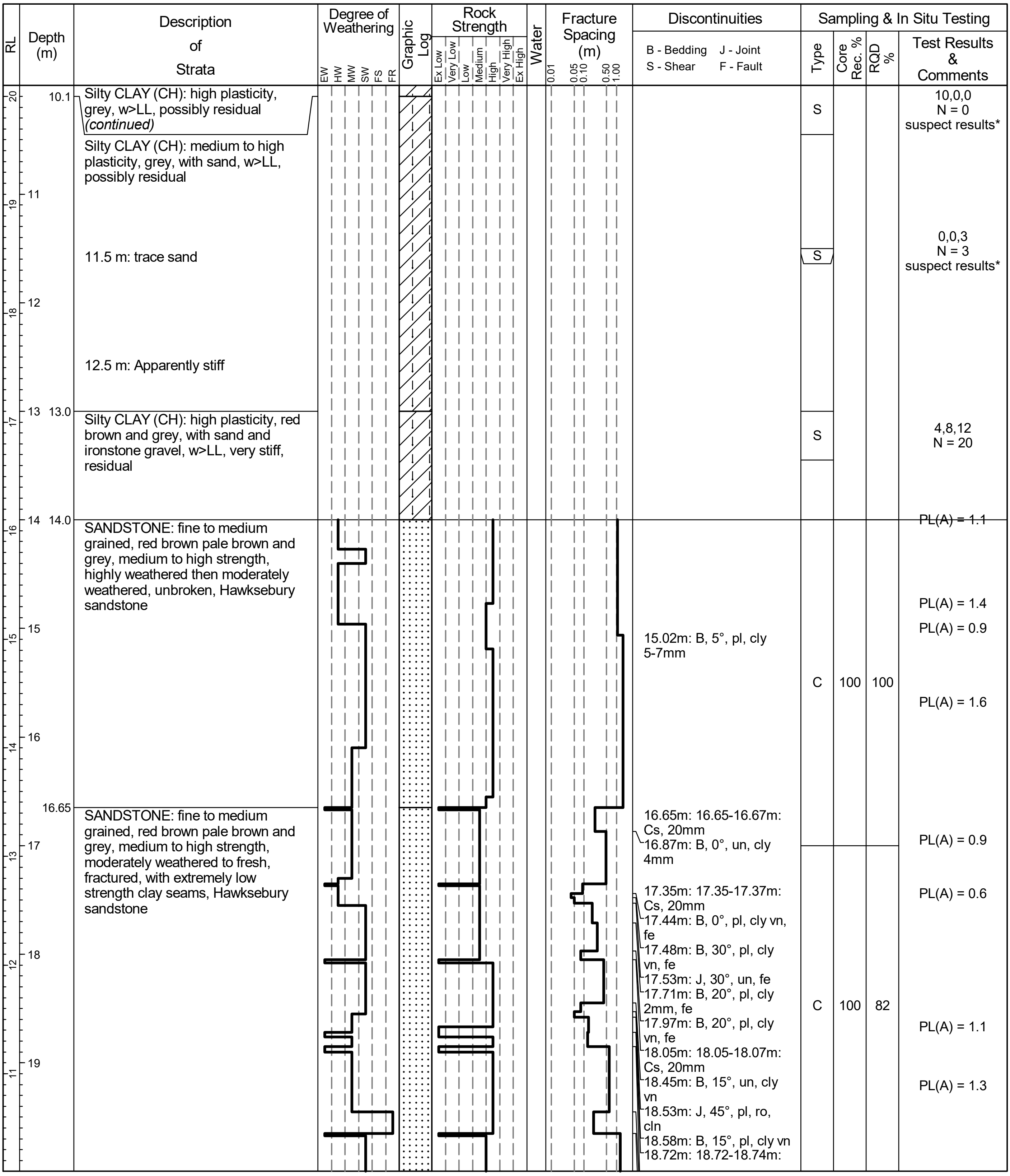
BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 30.1
EASTING: 334269.1
NORTHING: 6247994.1
DIP/AZIMUTH: 90°/--

BORE No: BH303
PROJECT No: 99510.00
DATE: 2 - 3/12/2019
SHEET 2 OF 3



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
10.1	Silty CLAY (CH): high plasticity, grey, w>LL, possibly residual *(continued)*		S			10,0,0 N = 0 suspect results*
	Silty CLAY (CH): medium to high plasticity, grey, with sand, w>LL, possibly residual 11.5 m: trace sand 12.5 m: Apparently stiff		S			0,0,3 N = 3 suspect results*
13.0	Silty CLAY (CH): high plasticity, red brown and grey, with sand and ironstone gravel, w>LL, very stiff, residual		S			4,8,12 N = 20
14.0	SANDSTONE: fine to medium grained, red brown pale brown and grey, medium to high strength, highly weathered then moderately weathered, unbroken, Hawksebury sandstone	15.02m: B, 5°, pl, cly 5-7mm	C	100	100	~~PL(A) = 1.1~~ PL(A) = 1.4 PL(A) = 0.9 PL(A) = 1.6
16.65	SANDSTONE: fine to medium grained, red brown pale brown and grey, medium to high strength, moderately weathered to fresh, fractured, with extremely low strength clay seams, Hawksebury sandstone	16.65m: 16.65-16.67m: Cs, 20mm 16.87m: B, 0°, un, cly 4mm 17.35m: 17.35-17.37m: Cs, 20mm 17.44m: B, 0°, pl, cly vn, fe 17.48m: B, 30°, pl, cly vn, fe 17.53m: J, 30°, un, fe 17.71m: B, 20°, pl, cly 2mm, fe 17.97m: B, 20°, pl, cly vn, fe 18.05m: 18.05-18.07m: Cs, 20mm 18.45m: B, 15°, un, cly vn 18.53m: J, 45°, pl, ro, cln 18.58m: B, 15°, pl, cly vn 18.72m: 18.72-18.74m:	C	100	82	PL(A) = 0.9 PL(A) = 0.6 PL(A) = 1.1 PL(A) = 1.3

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** RB **CASING:** HW to 13 m

TYPE OF BORING: Solid Flight Augering to 3.5 m, Rotary Drilling to 14.0 m, NMLC coing to 25.65 m

WATER OBSERVATIONS: 3.5 m

REMARKS: *Probably affected by drilling method

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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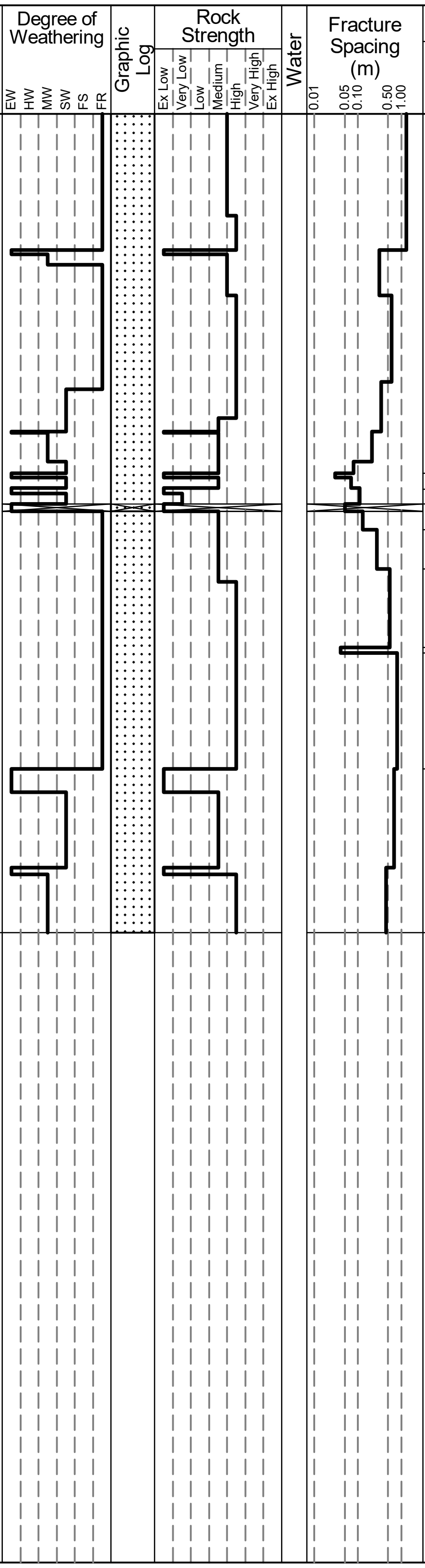
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: EMM Consulting Pty Limited
PROJECT: Proposed Mixed Use Development
LOCATION: 600-660 Elizabeth Street, Redfern

SURFACE LEVEL: 30.1
EASTING: 334269.1
NORTHING: 6247994.1
DIP/AZIMUTH: 90°/--

BORE No: BH303
PROJECT No: 99510.00
DATE: 2 - 3/12/2019
SHEET 3 OF 3



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

RL	Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
10		SANDSTONE: fine to medium grained, red brown pale brown and grey, medium to high strength, moderately weathered to fresh, fractured, with extremely low strength clay seams, Hawksebury sandstone *(continued)*	Cs, 40mm 18.85m: 18.85-18.90: Cs, 50mm 19.45m: B, 0°, pl, cly 8mm 19.65m: Ds, 20mm				PL(A) = 0.6
9	21		20.94m: 20.94-20.97m: Cs, 30mm 21.25m: B, 5°, pl, cbs	C	98	87	PL(A) = 1.2
8	22		21.85m: B, 5°, pl, cly 2mm 22.19m: Cs, 10mm 22.4m: B, 10°-20°, un, fe, cly 2mm 22.48m: 22.48-22.51m: Cs, 30mm 22.59m: 22.59-22.62m: Cs, 30mm				PL(A) = 0.95
	22.74		22.69m: CORE LOSS: 50mm				PL(A) = 0.6
7	23		22.87m: B, 5°, pl, cly 4mm 23.14m: B, 10°, cu, cly vn				PL(A) = 1.6
6	24		23.68m: B, 0°, pl, cly 7mm 23.72m: B, 0°, pl, cly 6mm 24.52m: 24.52-24.68m: Cs, 160mm	C	100	91	PL(A) = 1.2
5	25		25.2m: 25.20-25.25m: Cs, 50mm				PL(A) = 0.55 PL(A) = 2.1
	25.65	Bore discontinued at 25.65m					
4	26						
3	27						
2	28						
1	29						

RIG: Rig4 **DRILLER:** BG Drilling **LOGGED:** RB **CASING:** HW to 13 m
TYPE OF BORING: Solid Flight Augering to 3.5 m, Rotary Drilling to 14.0 m, NMLC coing to 25.65 m
WATER OBSERVATIONS: 3.5 m
REMARKS: *Probably affected by drilling method

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

BORE: 303 PROJECT: REDFERN DECEMBER 2019

14.0 m – 18.0 m

BORE: 303 PROJECT: REDFERN DECEMBER 2019

18.0 m – 23.0 m

BORE: 303 PROJECT: REDFERN DECEMBER 2019

18.0 m – 23.0 m

Appendix D

Results of Previous Laboratory Tests

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 232507

Client Details

Client	Douglas Partners Pty Ltd
Attention	Peter Valenti
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**99510.00, Redfern**
Number of Samples	4 Soil
Date samples received	06/12/2019
Date completed instructions received	06/12/2019

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	13/12/2019
Date of Issue	11/12/2019

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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

Nancy

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference		232507-1	232507-2	232507-3	232507-4
Your Reference	UNITS	BH301/4-4.45	BH301/10-10.45	BH302/8.5-8.95	BH303/5.5-5.95
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	09/12/2019	09/12/2019	09/12/2019	09/12/2019
Date analysed	-	09/12/2019	09/12/2019	09/12/2019	09/12/2019
pH 1:5 soil:water	pH Units	7.2	4.9	4.5	5.3
Electrical Conductivity 1:5 soil:water	µS/cm	12	19	75	88
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	74	120

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-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			09/12/2019	[NT]	[NT]	[NT]	[NT]	09/12/2019	[NT]
Date analysed	-			09/12/2019	[NT]	[NT]	[NT]	[NT]	09/12/2019	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	97	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	84	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	92	[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 (not SPOCAS); 60-140% for organics/SPOCAS (+/-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.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: lujia.wu@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Lujia Wu

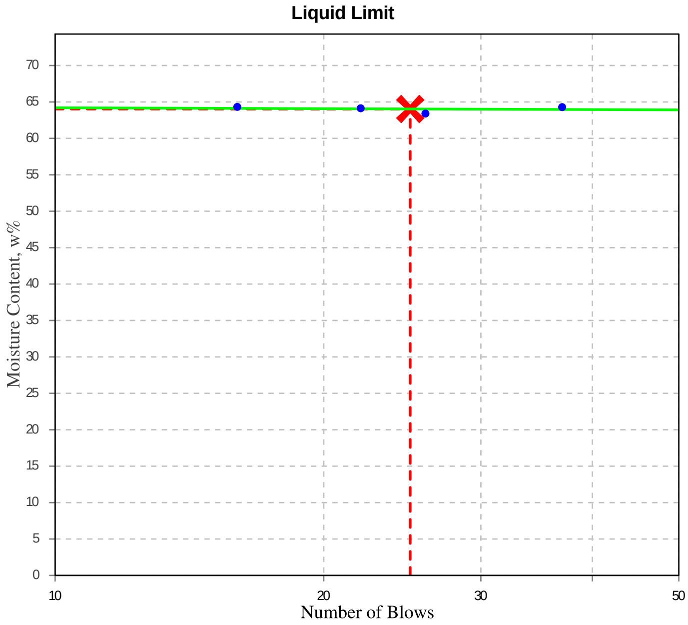
Approved Signatory: Lujia Wu
soil technician

NATA Accredited Laboratory Number: 828

Report Number: **99510.00-1**
Issue Number: 2 - *This version supersedes all previous issues*
Reissue Reason: *change description*
Date Issued: 16/01/2020
Client: EMM Consulting Pty Limited
Suite 1, Ground Floor, 20 Chandos Street, St Leonards NSW 2065
Contact: Anthony Davis
Project Number: 99510.00
Project Name: Proposed Mixed Use Development
Project Location: 600-660 Elizabeth Street, Redfern
Work Request: 5318
Sample Number: SY-5318A
Date Sampled: 05/12/2019
Dates Tested: 06/12/2019 - 17/12/2019
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: **BH303 (2.5-2.95m)**
Material: ORGANIC CLAY: high plasticity, dark grey, with organics and timber, wet, very soft, alluvial

Atterberg Limit (AS1289 3.1.1 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	64		
Plastic Limit (%)	58		
Plasticity Index (%)	**6**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	7.5		
Cracking Crumbling Curling	None		



Material Test Report

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Suite 1, Ground Floor, 20 Chandos Street, St Leonards NSW 2065
Contact: Anthony Davis
Project Number: 99510.00
Project Name: Proposed Mixed Use Development
Project Location: 600-660 Elizabeth Street, Redfern
Work Request: 5318
Sample Number: SY-5318B
Date Sampled: 05/12/2019
Dates Tested: 06/12/2019 - 12/12/2019
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: **BH302 (1.1-1.4m)**
Material: SAND (SP): fine to medium grained, dark brown and grey, wet, apparently loose, alluvial

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	Not Obtainable		
Plastic Limit (%)	Not Obtainable		
Plasticity Index (%)	**Non Plastic**		

Material Test Report

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Contact: Anthony Davis
Project Number: 99510.00
Project Name: Proposed Mixed Use Development
Project Location: 600-660 Elizabeth Street, Redfern
Work Request: 5318
Sample Number: SY-5318D
Date Sampled: 05/12/2019
Dates Tested: 06/12/2019 - 12/12/2019
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: **BH303 (1.1-1.2m)**
Material: SAND (SP): fine to medium grained, pale brown, wet, loose, alluvial

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NATA Accredited Laboratory Number: 828

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	Not Obtainable		
Plastic Limit (%)	Not Obtainable		
Plasticity Index (%)	**Non Plastic**		

Material Test Report

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Contact: Anthony Davis
Project Number: 99510.00
Project Name: Proposed Mixed Use Development
Project Location: 600-660 Elizabeth Street, Redfern
Work Request: 5318
Sample Number: SY-5318E
Date Sampled: 19/12/2019
Dates Tested: 17/12/2019 - 17/12/2019
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: **BH302 (1.4-1.45m)**
Material: PEAT/SAND: low plasticity, dark grey, with organics and timber, wet, soft, alluvial

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Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	Not Obtainable		
Plastic Limit (%)	Not Obtainable		
Plasticity Index (%)	**Non Plastic**		

Material Test Report

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Project Number: 99510.00
Project Name: Proposed Mixed Use Development
Project Location: 600-660 Elizabeth Street, Redfern
Work Request: 5318
Dates Tested: 06/12/2019 - 11/12/2019

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Moisture Content AS 1289 2.1.1

Sample Number	Sample Location	Moisture Content (%)	Material
SY-5318A	BH303 (2.5-2.95m)	110 %	ORGANIC CLAY: high plasticity, dark grey, with organics and timber, wet, very soft, alluvial
SY-5318B	BH302 (1.1-1.4m)	6.1 %	SAND (SP): fine to medium grained, dark brown and grey, wet, apparently loose, alluvial
SY-5318D	BH303 (1.1-1.2m)	37.5 %	SAND (SP): fine to medium grained, pale brown, wet, loose, alluvial