Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

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

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1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for a proposed mixed-use development 600-660 Elizabeth Street, Redfern. The investigation was commissioned by EMM Consulting Pty Limited and was undertaken in accordance with DP's proposal SYD191128.P.002.Rev0 dated 29 October 2019.

It is understood that the proposed development may include multiple buildings up to about nine storeys in height, with one 19-storey building to be located in the north-east corner of the site. Although not confirmed at this stage, it is expected that the development will incorporate a two to three-level basement across the majority of the site (i.e. excavation depths of 6 m to 9 m are anticipated).

The investigation included the drilling of three rock-cored boreholes, groundwater measurement during drilling, six cone penetration tests (CPTs), and laboratory testing of selected samples from the boreholes to assess the soil's aggressivity and plasticity. The details of the field work and laboratory test results are presented in this report, together with comments for design and construction.

It is understood that a contamination assessment was being undertaken by EMM for this project at the same time as this investigation. The contamination report prepared by EMM should be read in conjunction with this report. It is also understood that groundwater data will continue to be collected by EMM.

2. Site Description

The site is a rectangular shape with approximate dimensions of 148 m (north-south) by 55 m (east-west) and a site area of approximately 8,200 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 Police Citizens Youth Club (PCYC) South Sydney, 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 park with a grassed surface and 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 in 2013. Sewer main pipelines extend through the central area of the park.

The ground surface undulated throughout the park, with a general 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 quinquenervia (also known as 'Paperbark') trees. More information on the cause of the undulating ground is provided in Section 4 of this report.

3. Geology

Reference to the Sydney 1:100 000 Geological Sheet indicates that the site is located within 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.

Field work for this 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

DP 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. 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 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). I_{ss} 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 1.

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 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 1: 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
2	0.50-0.80	Organic Clay – Affected Area	96	9.2

Notes:

1. FMC – Field Moisture Content
2. The unaffected area (CPT1 and Borehole 1) is located under the centre of the road, away from the influence of the Melaleucas;
3. The affected area is directly below the Melaleucas and is influenced by them (all testing locations except CPT1 and Bore 1).

5. Field Work Methods

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 have 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 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.

6. 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.

6.1 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 2. Discussion on the selection of the geological ‘Units’ is provided in Section 9.

Table 2: 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 MD)	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
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
St-VSt 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)
M-H 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

Note: SANDS, VL = Very Loose, MD = Medium Dense; CLAYS, VS = Very Soft, S = Soft, St = Stiff, VSt = Very Stiff

6.2 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 measured at depths of between 1.4 m (RL 30.0 m) and 3.5 m (RL 31.1 m). A summary of the measured groundwater levels is provided in Table 3.

Table 3: Summary of Groundwater Measurements

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

* 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 measurements indicated a groundwater table at depths between 1.4 m and 3.5 m below ground level (i.e. at RL 26.6 to 29.2). It should be noted that groundwater levels are transient and that fluctuations may occur in response to climatic and seasonal conditions.

7. 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,

The results of the laboratory aggressivity and Atterberg limits testing are included in Appendix D, with the results summarised in Table 4 and Table 5, respectively.

Table 4: 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 5: Summary of Laboratory Test Results for Atterberg Limits and Moisture content

BH (Depth Range)	Description	W_P (%)	W_L (%)	PI (%)	LS (%)	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: W_P = plastic limit; W_L = liquid limit; PI = plasticity index; LS = linear shrinkage; I_{ss} = 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.

8. Proposed Development

Based on the concept design drawings provided by EMM, the proposed development may include multiple buildings up to about nine storeys in height, with one 19-storey building to be located in the north-east corner of the site. Although not confirmed at this stage, it is expected that the development will incorporate a two to three-level basement across the majority of the site (i.e. excavation depths of 6 m to 9 m are anticipated). The PCYC will be demolished with the removal of trees to make way for the new development.

It is estimated that the design column working loads will be in the order of 7000 kN for 19-storey building and 3000 kN for nine-storey buildings, based on an average column spacing of 8 m.

9. Comments

9.1 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 units 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 site appears to be underlain by different depths of fill, sands and clayey material overlying sandstone bedrock. The fill appeared to be variably compacted. The upper 5.2-8.8 m of the soil profile (Units 1, 2, and 3) represents the most recent alluvial deposits overlain by varying depths of fill. Some of the clay and peat deposits were very soft and organic.

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, which generally coincides with the proposed bulk excavation level (for a three-level basement). Planning and design should consider the possibility of some areas of soft clays occurring at the bulk excavation level.

The previous laboratory test results indicated that the organic clay and peat samples are of high plasticity with a 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.

The interpreted boundaries shown are accurate only at the test locations and are diagrammatic only. They 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.

Further investigation would be required especially in the southern part of the site where no geotechnical investigation could be done due to the presence of existing PCYC building. In particular, the CPT encountered practical refusal at 7.1 m depth (RL 23 m AHD). Although interpreted to be caused by dense/very dense cemented sand, it is possible that refusal was caused by bedrock (i.e. sandstone). It will be important to confirm the presence of bedrock above the proposed basement level at the southern end of the site, prior to the finalisation of design and before bulk excavation.

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]
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]
VL-MD Sands 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 [17.1]	8.8 [21.8]	8.6 [22.1]	8.8 [21.6]	8 [22.4]]	8.3 [21.7	5.2 [24.9]
M-H Sandstone	13.1 [18]	11.64 [18.86]	14 [16.1]	11.7 [18.9]	12.6 [18.1]	12.8 [17.6]	10.2 [20.2]	11.4 [18.6	SAND(1)

Notes: SAND, VL = Loose, L = Loose, MD = Medium Dense, D = Dense, VD = Very Dense;
CLAY, S = Soft, F = Firm, St = Stiff, Vst = Very Stiff, H = Hard
ROCK, VLS = Very Low Strength, LS = Low Strength, MS = Medium Strength
N.E = Not Encountered

(1) CPT 309 encountered practical refusal on dense cemented sand or possibly sandstone bedrock

9.2 Dilapidation Surveys

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

9.3 Excavations

Construction of the proposed basement will generally involve excavation to a maximum depth of about 9 m below current site levels which is expected to be within filling, organic soils, soft to very soft clayey soils and medium dense to dense, natural sand. 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.

9.3.1 Excavation Conditions

Excavation to deeper than 9 m may be required for the construction of the basement (raft) slab, which is likely to be required due to the high uplift pressures expected on the slab. Deeper excavation still may be required for the provision of a working platform from which tracked (piling) plant can operate. Therefore, excavation to depths of about 10 m below existing ground levels may be required.

Excavations for the basement are likely to intersect pavements, filling, natural soil and possibly bedrock of variable strength at the southern end of the site. Excavation of soil and extremely low to very low strength bedrock should be readily achieved using conventional earthmoving equipment, such as tracked excavators with bucket attachments. Removal of low strength or stronger bedrock will require relatively large excavators fitted with hydraulic rock hammers and rotary rock saws. Excavation of existing pavements is also likely to require similar plant and equipment. For the productive excavation of low strength or stronger rock within large areas, ripping of rock with large dozers should be considered.

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 (see Section 9.5). Groundwater and Dewatering for details). It should be noted that even when 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 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 sands within the excavation zone, where groundwater is controlled below the excavation level, batters no steeper than 1.5 (H): 1.0 (V) may be adopted. This is for batters up to 3 m in height and assumes that no surcharge load is at the back of the batter. If there is insufficient room for batters, then it will be necessary to provide retaining support (i.e. shoring) for the soils and any weak rock.

With respect to trafficability, the sandy soils and filling may cause difficulties for the plant, particularly below the present groundwater table where wet, “boggy” conditions could be expected even after dewatering. It will generally be necessary to form a working platform at the surface for piling/wall construction plants and for plants required for the construction of foundations at the proposed BEL. Crushed concrete, 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. 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 working platforms, both at the surface and at bulk excavation level. Consideration may be given to the incorporation of the working platform into the design of any raft slabs for the final basement structure.

9.3.2 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 filling and natural materials that may be removed from the site. Reference should be made to EMM’s Contamination Assessment report for guidance on the off-site disposal of excavated materials.

9.4 Excavation Support

9.4.1 General

The construction of the proposed two to three-level basement close to the site boundaries will pose a significant challenge, particularly with the high groundwater table at the site. Care will need to be taken during design and construction to ensure that a suitable structure and construction methodology is adopted for the basement.

To reduce the dewatering requirements for construction of the basement, and to limit drawdown outside the basement during internal dewatering, the shoring wall should be relatively impermeable and installed around the full perimeter of the excavation and keyed into very stiff to hard clay or rock. It is suggested that the shoring wall should be socketed at least 1.5 m into sandstone or consistent very stiff to hard clay, below the bulk excavation level. The shoring wall should not be terminated in the clayey sand/sandy clay layer 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.

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 understood that permanent lateral support of the basement retaining walls will eventually be provided by the structure of the completed building.

A relatively stiff retaining/shoring wall system will be required at this site in order to limit lateral deflections.

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 9.4.4.

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 buildings and infrastructure to withstand such movements should be considered as a part of wall selection and design.

Particular attention should be paid during the shoring wall installation to determine whether each part intercepts a sandstone ledge or buried cliff line. Where a shoring wall does span across an irregular bedrock surface particular care should be taken to ensure the full length of the wall is adequately socketed into uniform clay or rock. This will reduce the potential for seepage below the toe of the shoring wall.

It is suggested that inclinometers be installed around the site boundary, along Elizabeth Street Avenue, Phillip Street, Walker Street, and Kettle Street in order to monitor wall deflections.

9.4.2 Retaining/Shoring Wall Systems

The final basement structure should incorporate a watertight retaining wall system around the basement perimeter.

The following options may be considered:

- Diaphragm walls may 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.
- Interlocking secant pile wall (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. The potential for misalignment and therefore seepage and sand loss through gaps in deep secant pile walls is very high. Drilling of piles into rock will also be problematic for secant piles and may result in decompression of the surrounding sands which can result in damage to adjacent buildings, infrastructures or utilities. The use of segmental casing may be required to avoid issues associated with decompression.
- Deep soil mix (DSM) or cutter soil mix (CSM) wall (temporary) – DSM/CSM walls involve blending or mixing of grout with the site soils in situ to form cement stabilised soil panels with universal column sections “plunged” into the “wet” panel at regular intervals along the wall to provide bending stiffness. However, experience with the DSM/CSM walls has indicated that the mixing consistency, and consequently the permeability and durability of the wall need to be carefully considered, particularly within clayey/peaty soils and rock. This option is unlikely to be suitable in the clayey and peaty soils and may not achieve an effective seal at the rock interface.

9.4.3 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. For preliminary design of shoring walls, a uniform distribution of 4H is suggested for earth pressure where some ‘inward’ lateral wall deflection is acceptable, increasing to 6H to 8H, where wall (and retained ground) movements must be kept to a minimum. In all cases, ‘H’ is the depth of proposed excavation (e.g. wall ‘height’).

Table 7: Parameters for Retaining Wall / Shoring Design

Material	Bulk Unit Weight (kN/m^3)	Bouyant Unit Weight (kN/m^3)	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 Clays	18	8	0.4	0.6	N/A
st-vst Clays	20	10	0.3	0.5	100 kPa
l-md Sands	21	11	0.33	0.45	K_p = 3.4
M-H Sandstone	24	14	0.25	1.0	4000 kPa

Notes: vs-s = very soft to soft; st-vst = stiff to very stiff; l-md = loose to medium dense; M-H = medium to high strength

*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 flooding. Surcharge pressures from adjacent structures, construction machinery 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.

9.4.4 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 stresses at the grout-rock/soil interface given in Table 8.

Table 8: Allowable Bond Stresses for Anchor Design

Material Description	Allowable Bond Stress (kPa)
Medium dense sand (below 4 m depth)	25
Medium to high strength Rock	500

It is unlikely that conventional anchors will have sufficient capacity unless they are installed in the bedrock. Secondary-grouted anchors could be used in the filling 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 fairly 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 parameters given in Table 7 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.

9.5 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. Given the sensitivity of the peat and organic clay underlying the site and surrounds, it will be particularly important to avoid significant lowering of the groundwater table during basement construction. Otherwise, the nature of damage observed by DP and the Council in 2009, as described in Section 4, could be expected. For this reason, it will be important that additional rock-cored boreholes are drilled around the proposed basement perimeter, so as to clearly define the required founding level of the ‘cut-off’ wall for design and construction purposes.

Although probably less expensive, the secant pile method can suffer possible misalignments, particularly for basements of more than two-levels, which might cause water inflows. Diaphragm walls are generally more robust and trusted in terms of construction, but are more expensive and involve heavy construction facilities. CSM walls are not suggested as the excavation facilities for this method might not be able to provide sufficient socket depth necessary for the shoring wall and the presence of substantial peat and organic clays would likely result in a poor ‘soil-mixed’ concrete wall, with low strength and poor durability.

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. On this basis, the normal groundwater level may need to be temporarily lowered by 9 – 10 m (i.e. lowered to about RL 19.0 to RL 20.0).

For the 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 presence of defects or more permeable zones in the clay and rock mass underlying the basement (i.e. below the floor of the basement). Significantly higher rock permeability (and therefore groundwater inflows) may occur if a geological feature crosses the site although this has not been identified by the boreholes. Some groundwater inflows will inevitably occur through retaining/shoring walls and up through the floor of the excavation.

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. Contamination testing on stormwater runoff seepage has not been completed by DP to date, and this should be completed with further soil/groundwater contamination assessments.

9.6 Piling and Foundations

9.6.1 General

It is estimated that the design column working loads will be in the order of 7000 kN for the19-storey building and 3000 kN for the nine-storey buildings, based on an average column spacing of 8 m. There will also be relatively high uplift loads present due to high 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 or pad footing founded within the underlying sandstone bedrock to reduce the potential issue of differential settlements. The expected high uplift (or tension) loads will probably also dictate the use of rock-socketed piles.

Bored pile excavation holes would not remain open in the sandy filling 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 can occur during CFA piling when a strong stratum is encountered. In this case, the augers continue to rotate but the rate or auger progression decreases and soil from around the auger is displaced upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and 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.

9.6.2 Design

Recommended maximum design pressures for the rock strata, for axial compression loading cases, are presented in Table 9. For piles shaft adhesion values for uplift (tension) 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-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 bored or CFA 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 strength of the rock indicated by the bores and the anticipated column loads it is expected that although piles with rock sockets in the order of 2-3 m will cater for (Working) loads of up to 7,000 kN, groups of piles will be required for supporting the proposed column loads for the 19-storey building.

9.6.3 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 filling. 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: τ = 0.15 p'
- for loose to medium dense sand filling: τ = 0.20 p'
  (where p' is the effective overburden pressure)

For piles at the bulk excavation (i.e. basement) level, negative skin friction considerations will generally not apply. It is only where piles are outside the proposed basement and penetrate the soft peat and clay layers.

9.6.4 Soil Aggressivity

Aggressivity to concrete piles was assessed using the laboratory test results. 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*.

9.7 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.

9.8 Ground Vibrations

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

9.9 Working Platforms

Working platforms may 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 filling and soft clay or loose sand is present. Such platforms typically require the use of additional layers of durable, high strength crushed rock 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.

9.10 Survey Monitoring

The use of instrumentation to monitor existing adjacent roads and footpaths and 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.

9.11 Earthworks and Site Preparation

If the proposed basement excavation extends around the full perimeters of the site, it is unlikely that any form of earthworks construction will be required. If, however, basement and hardstand or carpark areas are required at or near the existing ground surface, 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 clays will experience settlement-related damage over the long term.

Notwithstanding the above, it may be possible to construct a reasonable subgrade for relatively light loading, 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 top soil, 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;
- 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 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 a deep fill is proposed.

9.12 Pavements

New pavements for 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 9.11, preliminary (flexible) pavement design could be based on a design CBR value of 2%. This value should be confirmed by future investigation, and for any alternative material(s) proposed for use in the pavement subgrade.

It is DP's 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.

10. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 600-660 Elizabeth Street, Redfern in accordance with DP's proposal SYD191128.P.002.Rev0 dated 29 October 2019 and acceptance received from EMM Consulting Pty Ltd (EMM) dated 22 November 2019. The work was carried out under EMM's Terms and Conditions, with mutually agreed amendments to some clauses (ref: email from Anthony Davis dated 13 December 2019). This report is provided for the exclusive use of EMM 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 sites or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client.

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

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

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

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

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

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

Site Anomalies

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

Information for Contractual Purposes

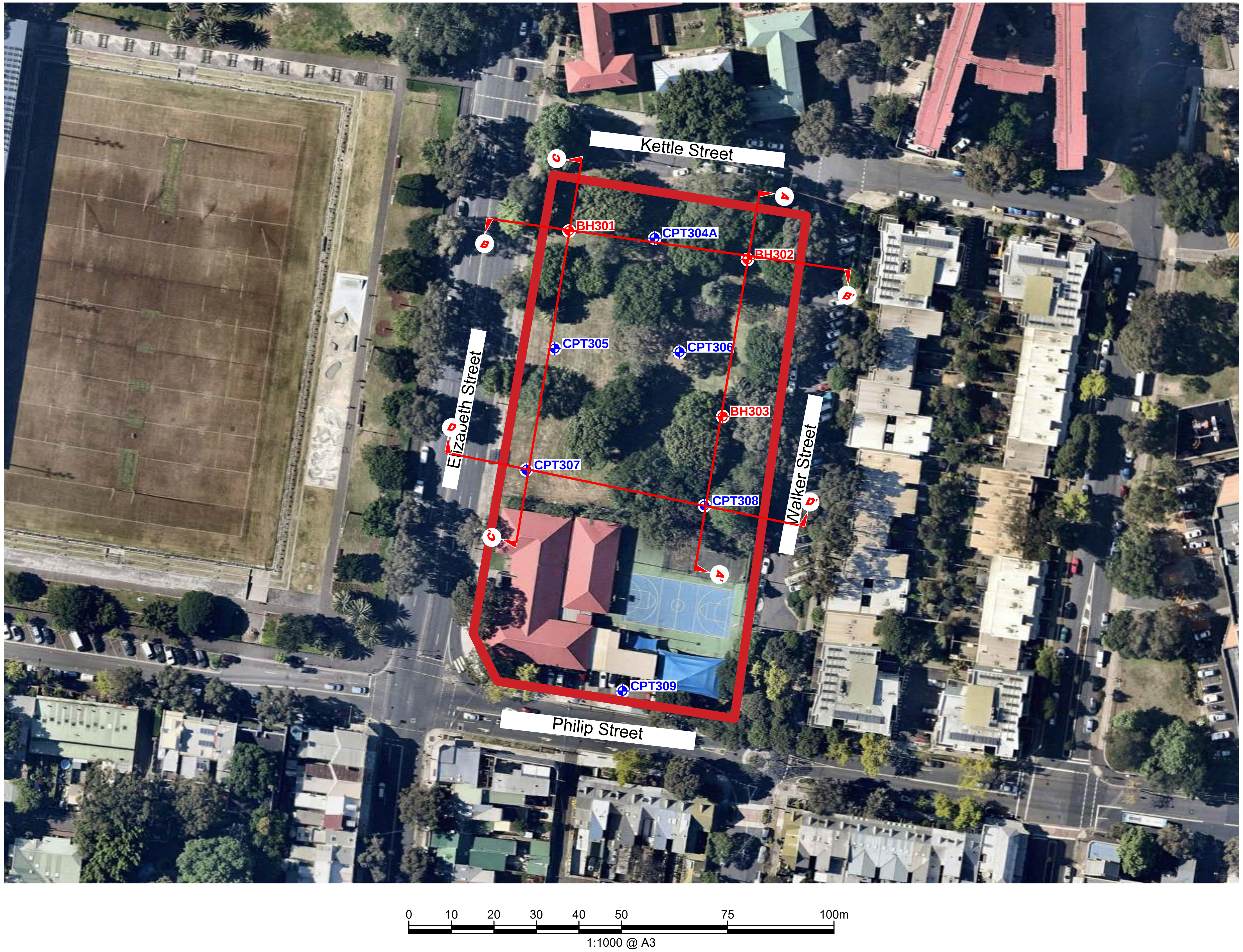
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

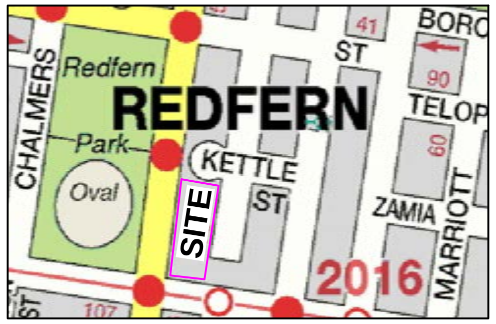
Site Inspection

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

Appendix B

Drawings





Locality Plan

LEGEND

- ⯐ Borehole Location
- ⯐ CPT Location

NOTE:
1: Base image from Nearmap.com (Dated 22.10.2019)

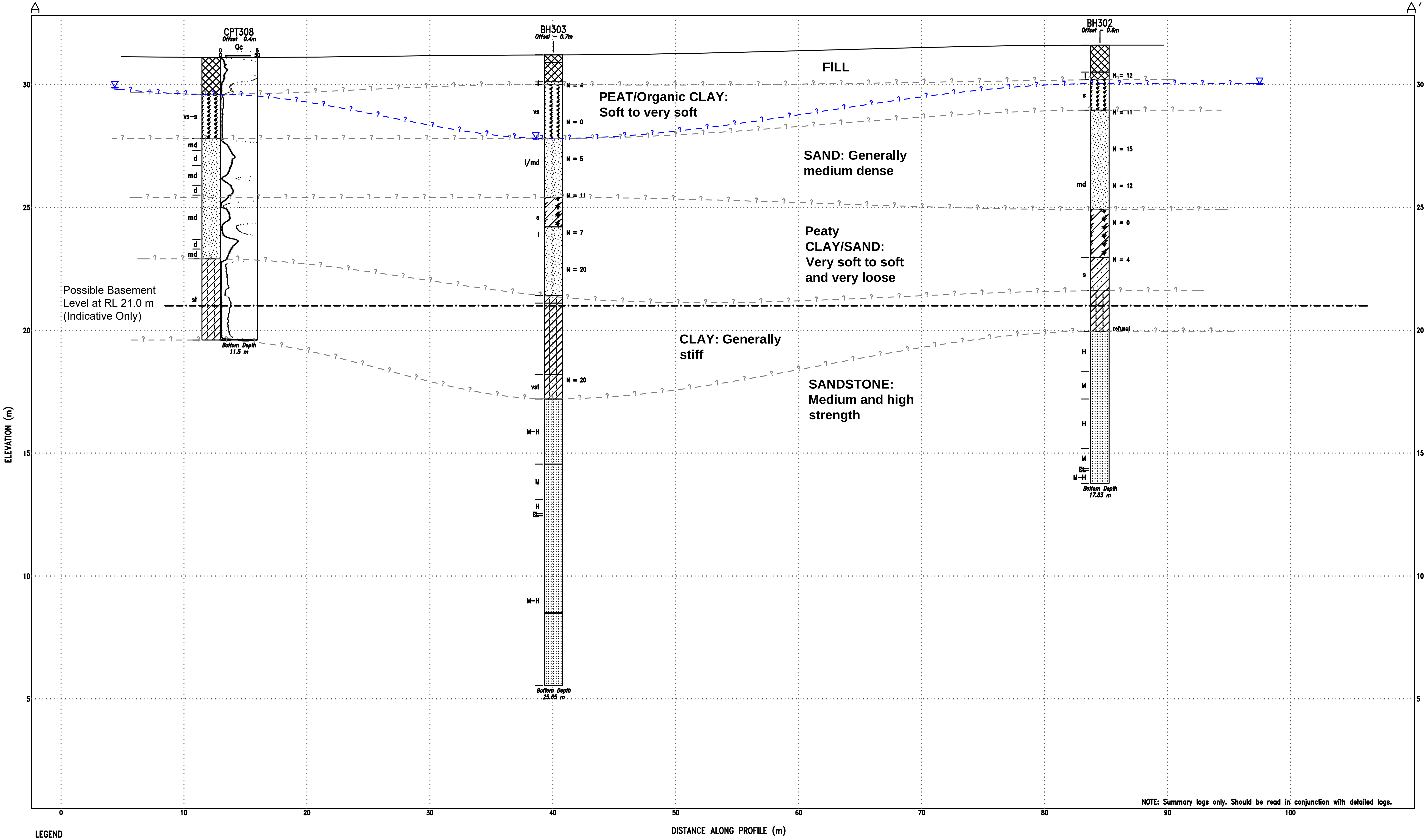
Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: EMM Consulting Pty Ltd
OFFICE: Sydney
DRAWN BY: ZH
SCALE: 1:1000 @ A3
DATE: 15.01.2020

TITLE: **Test Location Plan**
Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

N

PROJECT No: 99510.00
DRAWING No: 1
REVISION: 0



NOTE: Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ▩ Filling
- ▥ Peat
- ░ 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.

SOIL STRENGTH

l	- Loose	f	- firm
md	- Medium Dense	st	- Stiff
d	- Dense	vst	- Very Stiff
vd	- Very Dense	h	- Hard

ROCK STRENGTH

VL - Very low
L - Low
M - Medium
H - High
VH - Very high

TESTS / OTHER

— ? — — - Interpreted geological boundary

N - Standard penetration test value

▽ ? — — - Measured groundwater table in Boreholes and CPT holes

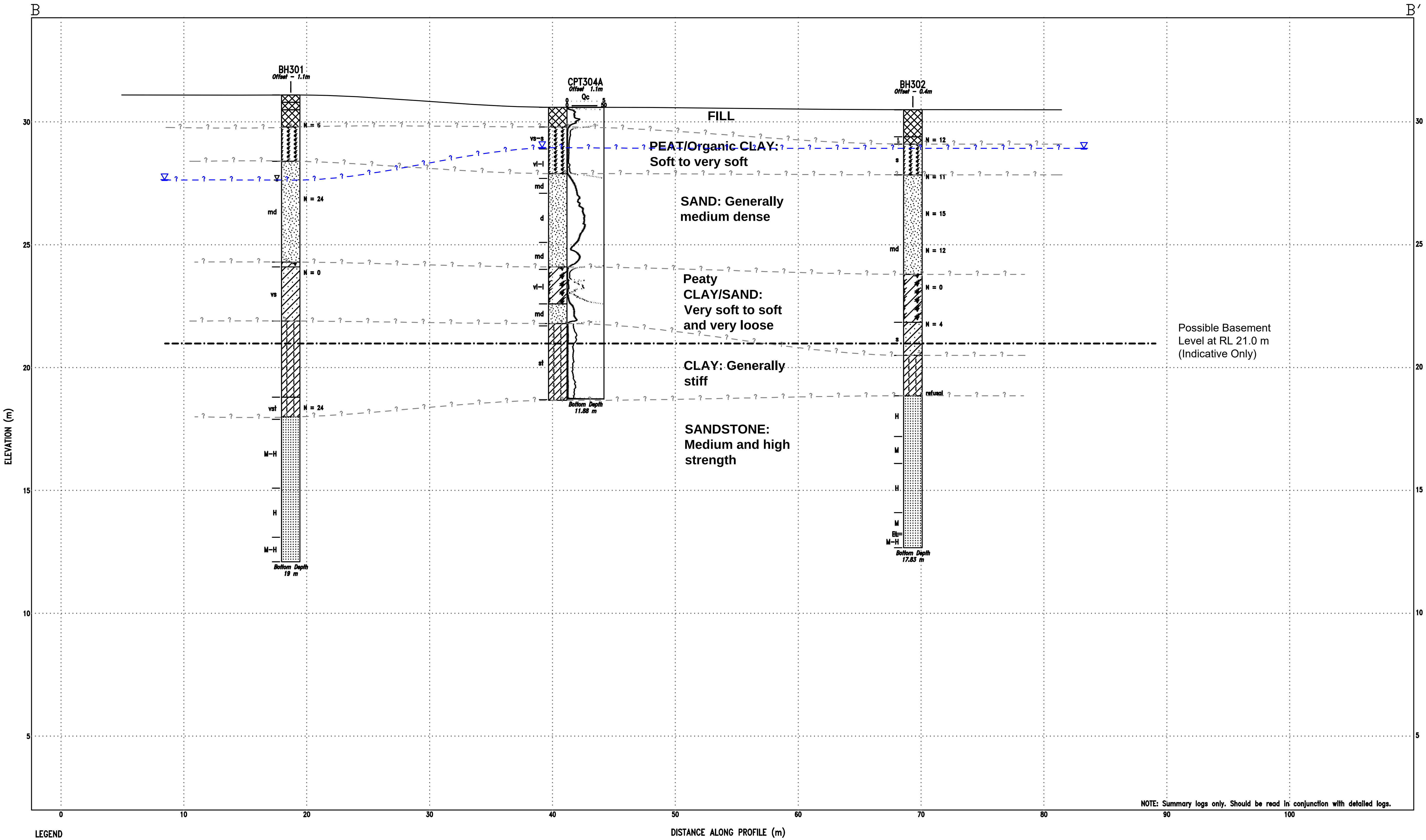
0 6
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: EMM Consulting Pty Limited
OFFICE: Sydney
DRAWN BY: ZH
SCALE: 1:300 (H) 1:150 (V) @ A3
DATE: 15.01.2020

TITLE: Interpreted Geotechnical Cross-Section A-A'
Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.00
DRAWING No: A-2
REVISION:



NOTE: Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ▩ Filling
- ▥ Peat
- ░ 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.

SOIL STRENGTH
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense
f - firm
st - Stiff
vst - Very Stiff
h - Hard

ROCK STRENGTH
VL - Very low
L - Low
M - Medium
H - High
VH - Very high

TESTS / OTHER
— ? — — - Interpreted geological boundary
N - Standard penetration test value
⊻ ? — — - Measured groundwater table in Boreholes and CPT holes

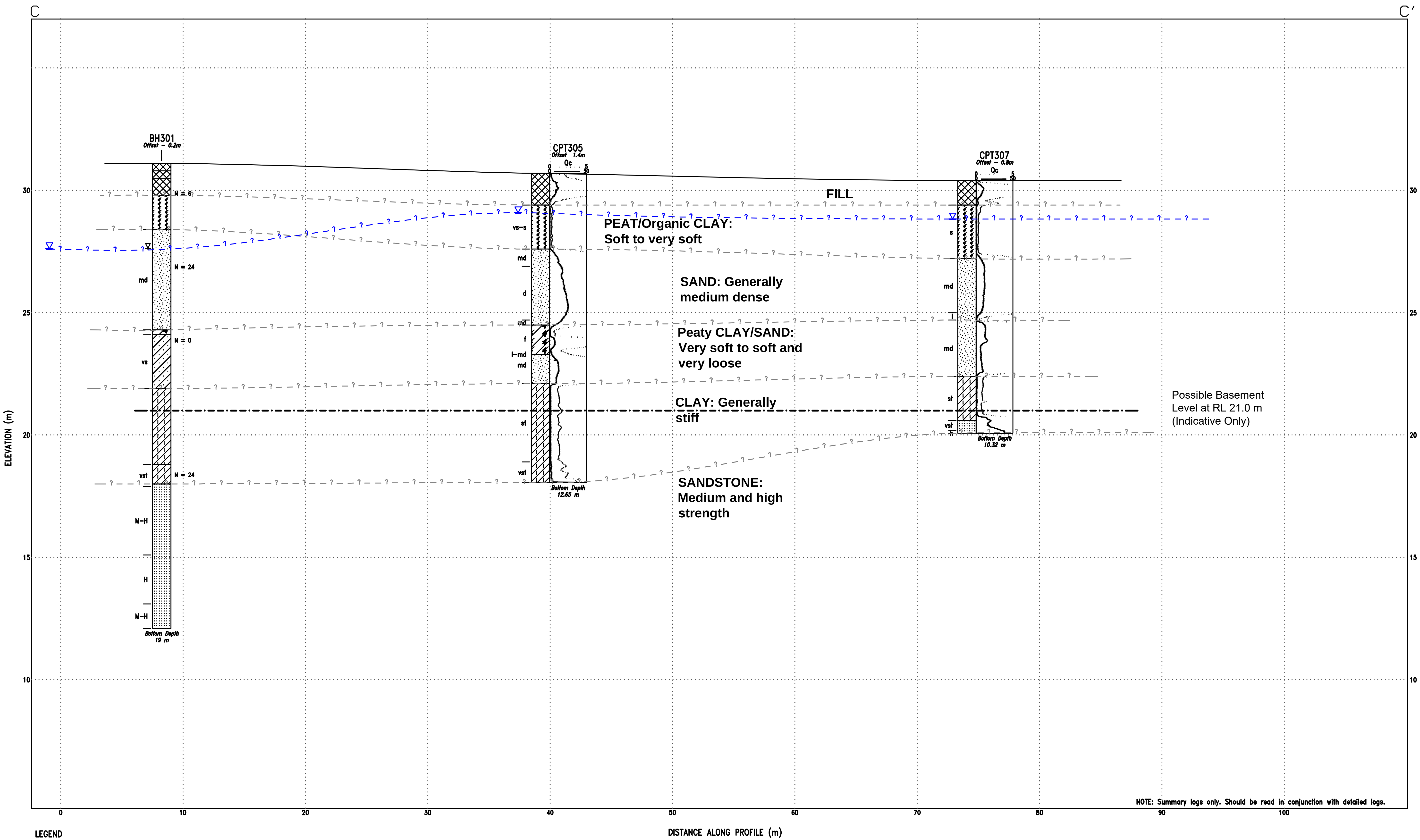
0 6
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT:	EMM Consulting Pty Limited		
OFFICE:	Sydney	DRAWN BY:	ZH
SCALE:	1:300 (H) 1:150 (V) @ A3	DATE:	15.01.2020

TITLE: Interpreted Geotechnical Cross-Section B-B'
Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.00
DRAWING No: A-3
REVISION:



NOTE: Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ▩ Filling
- ▥ Peat
- ⬚ 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.

SOIL STRENGTH

l	- Loose	f	- firm
md	- Medium Dense	st	- Stiff
d	- Dense	vst	- Very Stiff
vd	- Very Dense	h	- Hard

ROCK STRENGTH

VL	- Very low
L	- Low
M	- Medium
H	- High
VH	- Very high

TESTS / OTHER

— ? — — - Interpreted geological boundary

N - Standard penetration test value

▽ ? — — - Measured groundwater table in Boreholes and CPT holes

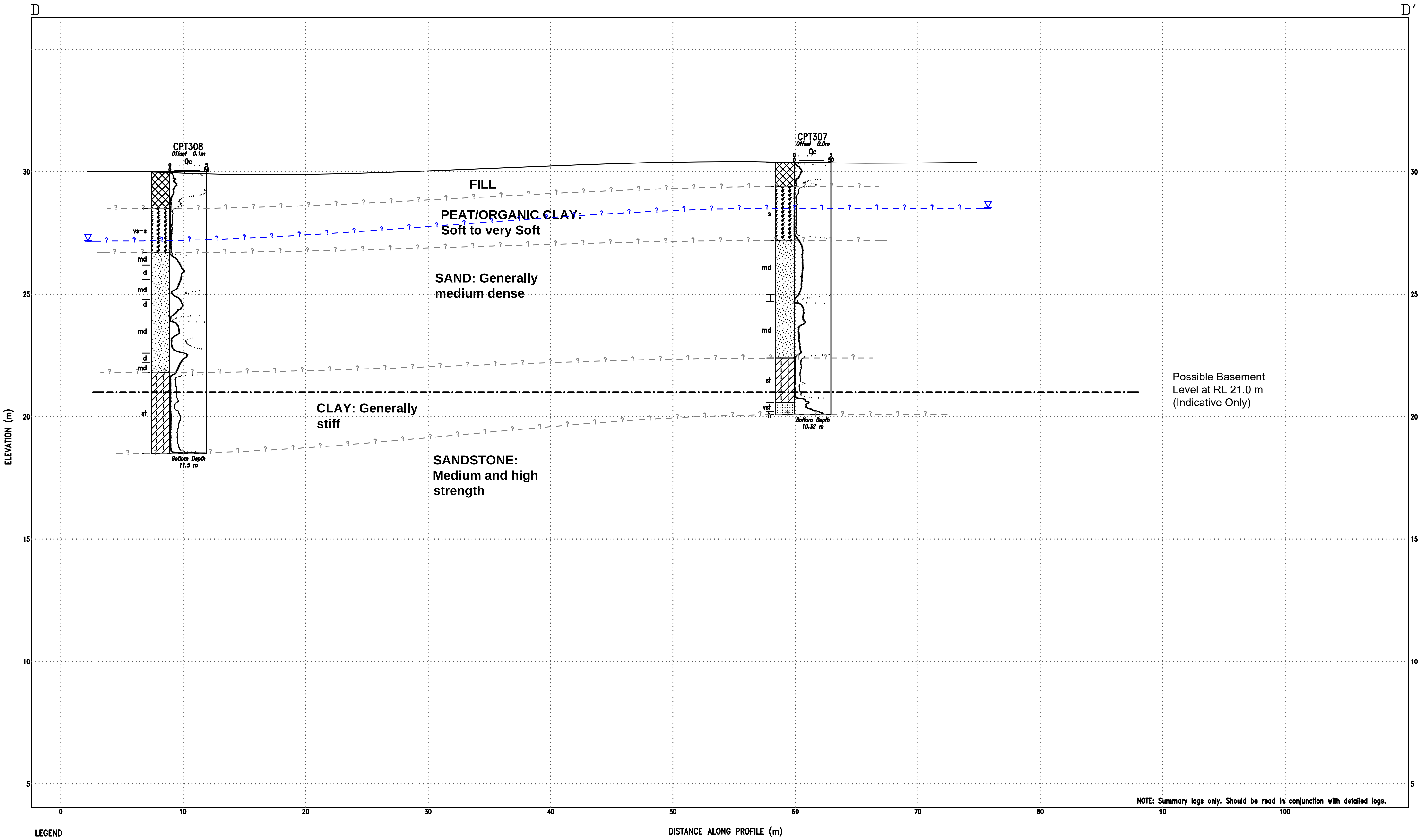
0 6
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT:	EMM Consulting Pty Limited		
OFFICE:	Sydney	DRAWN BY: ZH	
SCALE:	1:300 (H) 1:150 (V) @ A3	DATE:	15.01.2020

TITLE: Interpreted Geotechnical Cross-Section C-C′
Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

PROJECT No:	99510.00
DRAWING No:	A-4
REVISION:	



LEGEND

☒	Core Loss
▩	Filling
▥	Peat
⛆	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.

SOIL STRENGTH
l - Loose
md - Medium Dense
d - Dense
vd - Very Dense
st - Stiff
vst - Very Stiff
h - Hard

ROCK STRENGTH
VL - Very low
L - Low
M - Medium
H - High
VH - Very high

TESTS / OTHER
— ? — — - Interpreted geological boundary
N - Standard penetration test value
▽ ? — — - Measured groundwater table in Boreholes and CPT holes

0 6
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: EMM Consulting Pty Limited
OFFICE: Sydney
DRAWN BY: ZH
SCALE: 1:300 (H) 1:150 (V) @ A3
DATE: 15.01.2020

TITLE: Interpreted Geotechnical Cross-Section D-D'
Proposed Mixed Use Development
600-660 Elizabeth Street, Redfern

PROJECT No: 99510.00
DRAWING No: A-5
REVISION:

Appendix C

Results of Field Work

Sampling Methods

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Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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

Soil Descriptions

Description and Classification Methods

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

Soil Types

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

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

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

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

Definitions of grading terms used are:

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

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

In fine grained soils (>35% fines)

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

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

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

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

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

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

Soil Descriptions

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Origin

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

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

Moisture Condition – Coarse Grained Soils

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

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

Moisture Condition – Fine Grained Soils

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

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

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:

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

where 'sound' rock is assessed to be rock of low strength or 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

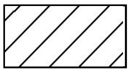
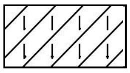
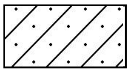
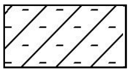
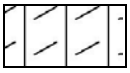
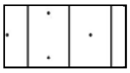
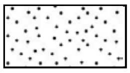
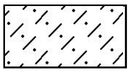
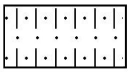
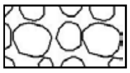
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

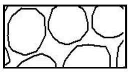
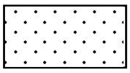
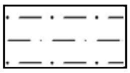
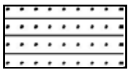
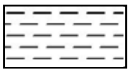
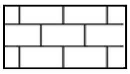
General

■	Asphalt
	Road base
	Concrete
	Filling

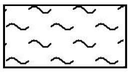
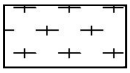
Soils

	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

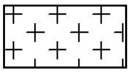
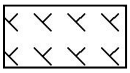
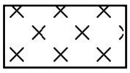
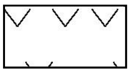
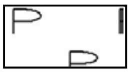
Sedimentary Rocks

	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

May 2017

Cone Penetration Tests

Douglas Partners

Introduction

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

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

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

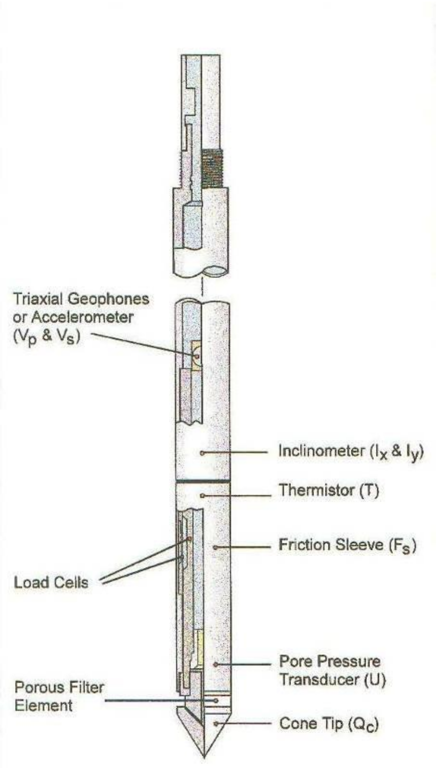


Figure 1: Cone Diagram

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

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

Figure 2: Purpose built CPT rig

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

Types of CPTs

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

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

Strata Interpretation

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

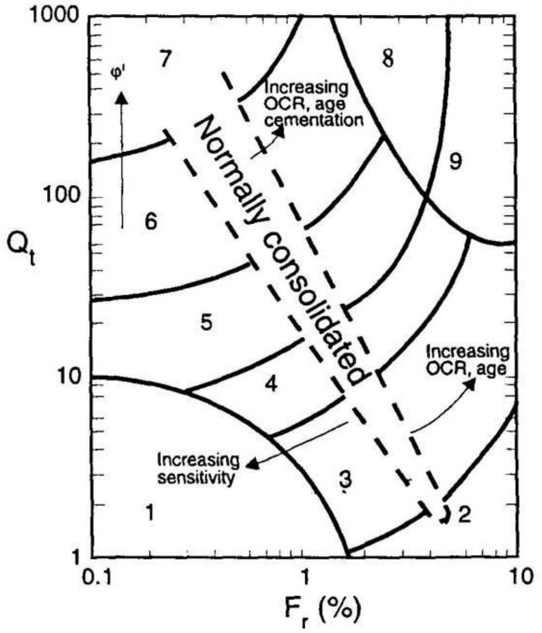


Figure 3: Soil Classification Chart

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

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

Engineering Applications

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

Settlement

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

Pile Capacity

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

Dynamic or Earthquake Analysis

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

Other Applications

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

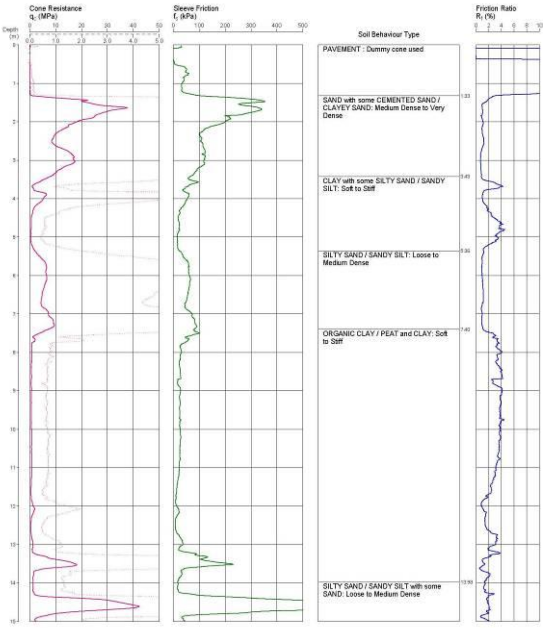


Figure 4: Sample Cone Plot

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

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
31		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										

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)

Douglas Partners
Geotechnics / Environment / Groundwater

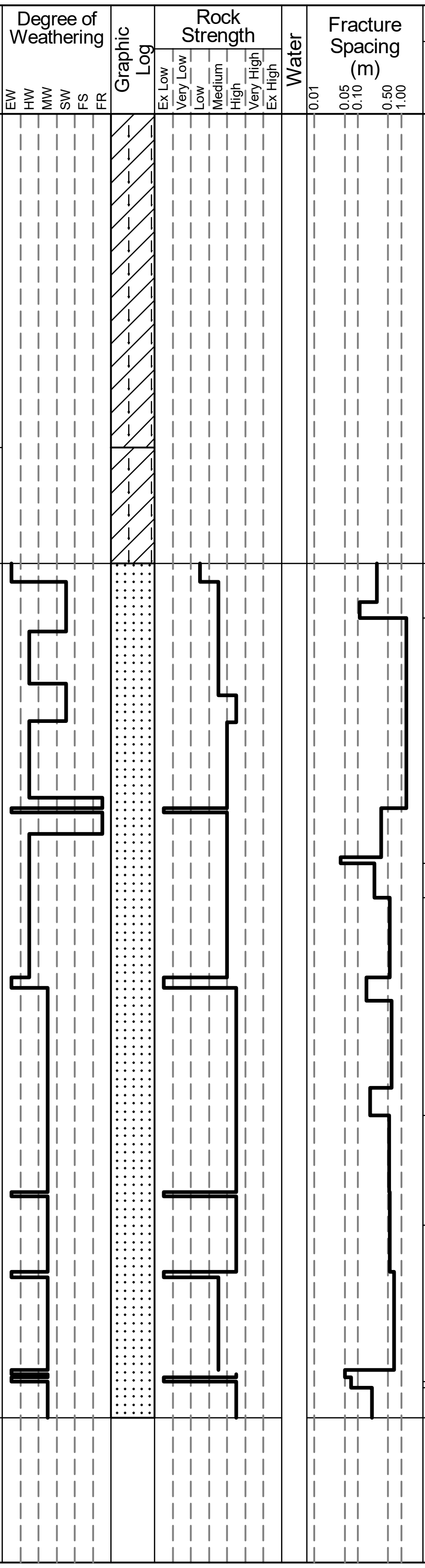
BOREHOLE LOG

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 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 20 19	11 12	Silty CLAY (CH): high plasticity, grey, with sand, w>LL, possibly residual *(continued)*		S			0,0,0 N = 0 suspect results*
	12.3 13	Silty CLAY (CH): high plasticity, grey, with sand, w>LL, very stiff, residual		S			7,10,14 N = 24
18 17 16 15	13.1 14 15 16	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 14.79-14.82m: Cs,30mm 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	C	100	91	PL(D) = 0.2 PL(D) = 1.2 PL(D) = 1.3 PL(D) = 0.6 PL(D) = 1.2 PL(D) = 1.2
14 13	17 18		16.12m: J, 30°, pl, fe, cly vn 16.72m: B, 0°, pl, fe, tight 16.91m: B, 0°, pl, cly 4mm 17.44-17.47m: Cs, 30mm 17.67-17.70m: Cs, 30mm 17.99-18.03m: Cs, 30mm 18.72-18.75m: Cs, 30mm 18.75m: J, 60°, pl, fe 18.79m: B, 15°, pl, fe, cly 2mm	C	100	89	PL(D) = 1.3 PL(D) = 1 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)

Douglas Partners
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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 & In Situ Testing: 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				

Column headings: RL; Depth (m); Description of Strata; Degree of Weathering (EW, HW, MW, SW, FS, FR); Graphic Log; Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High); Water; Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00); Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault); Sampling & In Situ Testing (Type, Core Rec. %, RQD %, Test Results & Comments)

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)

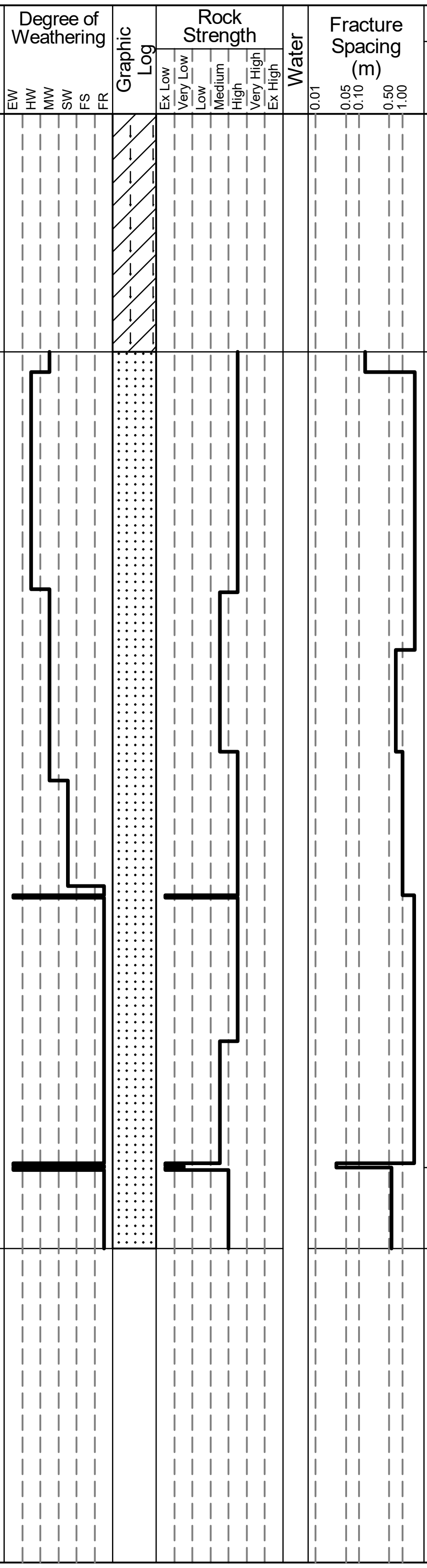
Douglas Partners
Geotechnics / Environment / Groundwater

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	C	100	100	PL(A) = 2 PL(A) = 1.2
		13.7m: B, 5°, un, ro, cly vnr				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)

Douglas Partners
Geotechnics / Environment / Groundwater

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	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	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 1.1	SAND (SP): fine to medium grained, pale brown, wet, loose, alluvial							S			2,2,2 N = 4
28 27	1.2 2 3	PEAT: dark grey, with organics and timber, wet, very soft, alluvial							S			0,0,0 N = 0
26 25	3.4 4 5	SAND (SP): fine to medium grained, pale brown, with interbedded peat bands, w>LL, loose to medium dense, alluvial				▼			S S			0,2,3 N = 5 5,3,8 N = 11
24	5.8 6	PEATY CLAY/SAND: interbedded soft peaty clay and loose sand										
23 22 21	7 7.0 8 9	SAND (SP): fine to medium grained, pale brown, with interbedded peat bands, w>LL, loose to medium dense, alluvial 7.5m: becoming dense							S S			4,4,3 N = 7 8,13,7 N = 20
	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)

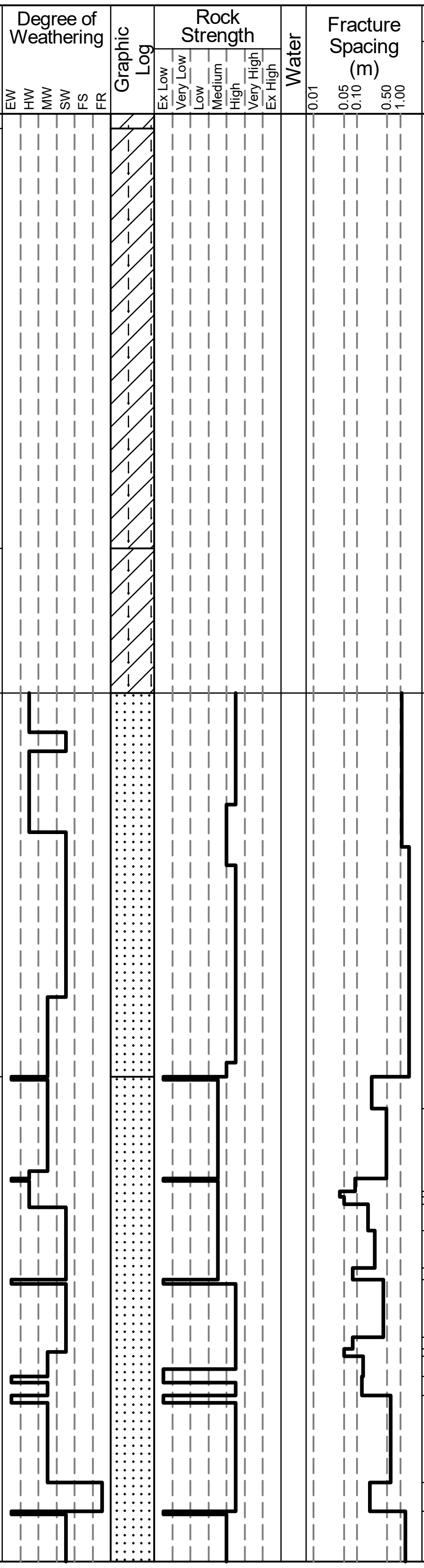
Douglas Partners
Geotechnics / Environment / Groundwater

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 2 OF 3



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
20	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					
19	11	11.5 m: trace sand		S			0,0,3 N = 3 suspect results*
18	12	12.5 m: Apparently stiff					
17	13 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
16	14 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		C	100	100	PL(A) = 1.1 PL(A) = 1.4
15	15		15.02m: B, 5°, pl, cly 5-7mm				PL(A) = 0.9 PL(A) = 1.6
14	16						
	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				
13	17		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				PL(A) = 0.9 PL(A) = 0.6
12	18		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) = 1.1
11	19						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)

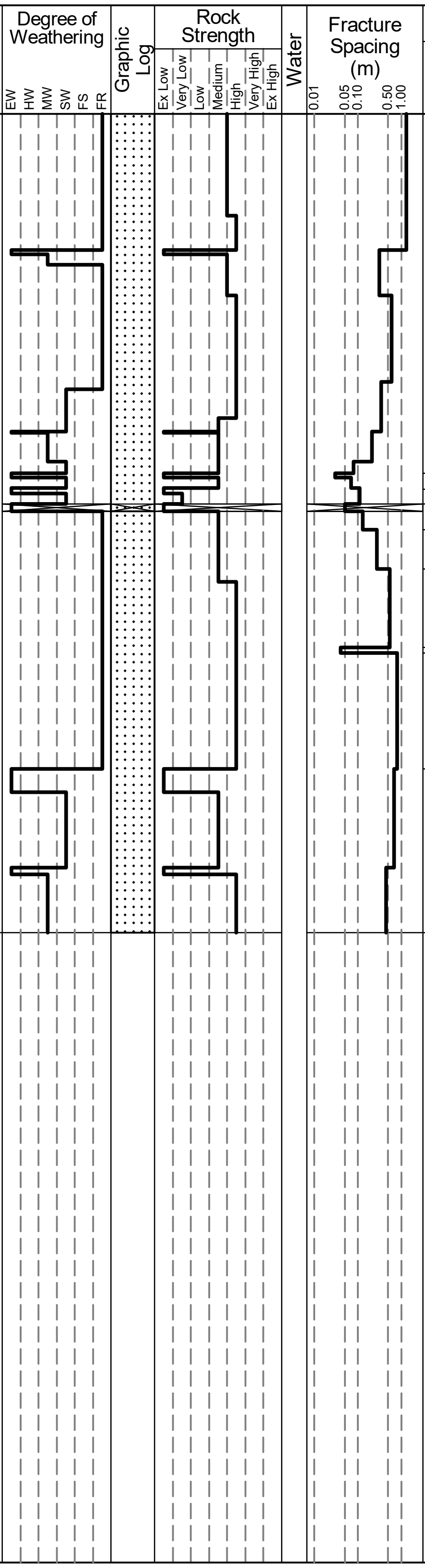
Douglas Partners
Geotechnics / Environment / Groundwater

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



RL	Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
10 – 7	21 – 22.74	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 20.94m: 20.94-20.97m: Cs, 30mm 21.25m: B, 5°, pl, cbs 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 22.69m: CORE LOSS: 50mm	C	98	87	PL(A) = 0.6 PL(A) = 1.2 PL(A) = 0.95 PL(A) = 0.6
7 – 5	22.74 – 25.65		22.87m: B, 5°, pl, cly 4mm 23.14m: B, 10°, cu, cly vn 23.68m: B, 0°, pl, cly 7mm 23.72m: B, 0°, pl, cly 6mm 24.52m: 24.52-24.68m: Cs, 160mm 25.2m: 25.20-25.25m: Cs, 50mm	C	100	91	PL(A) = 1.6 PL(A) = 1.2 PL(A) = 0.55 PL(A) = 2.1
4 – 1	25.65 – 29	Bore discontinued at 25.65m					

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

Project No: 99510.00
BH ID: BH303
Depth: 23.0m - 25.65m
Core Box No.: 3/3

18.0 m – 23.0 m

CONE PENETRATION TEST

CLIENT: EMM Consulting Pty Ltd

PROJECT: Proposed Mixed-Use Development

LOCATION: 600-660 Elizabeth Street, Redfern

REDUCED LEVEL: 30.6

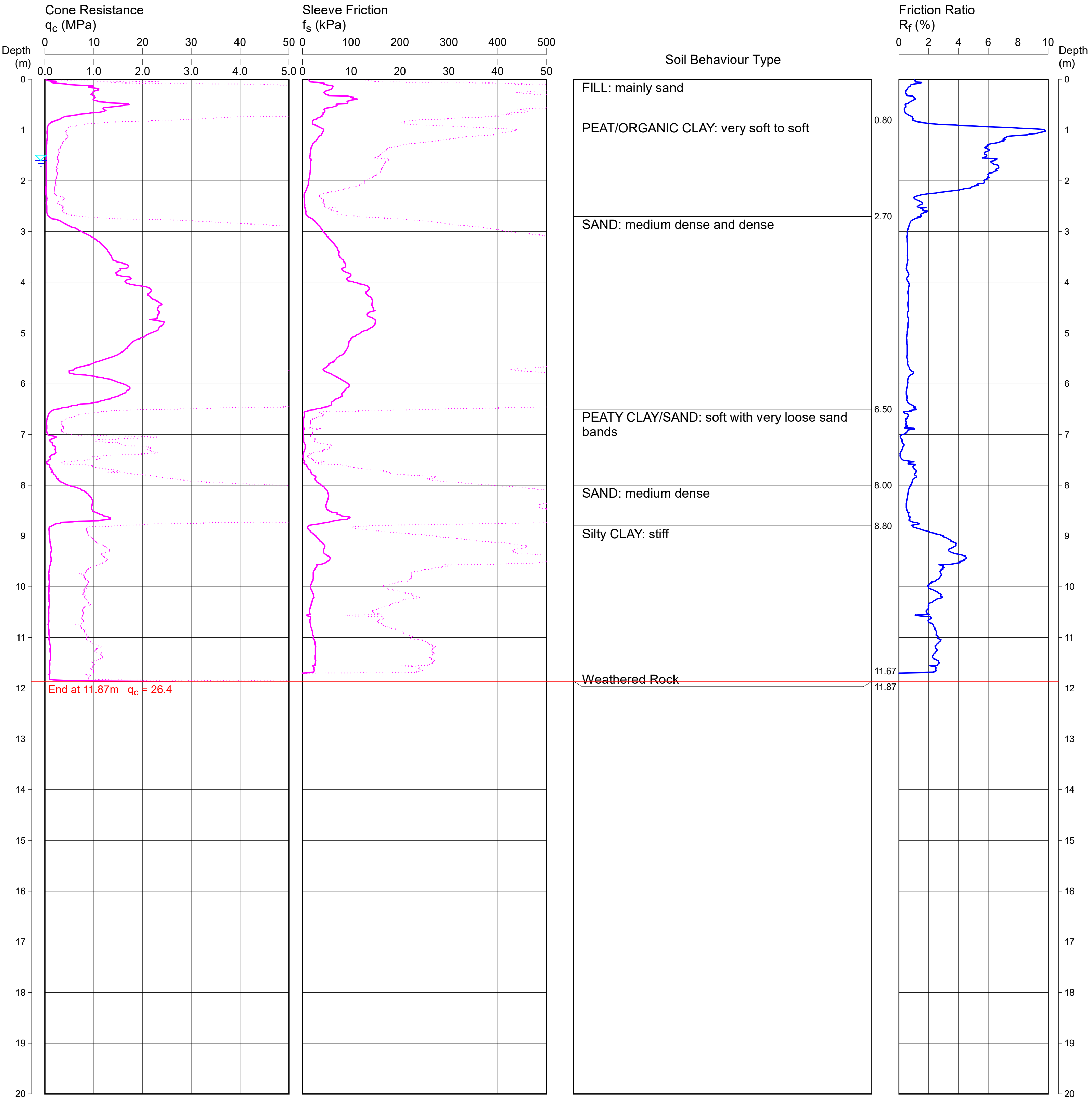
COORDINATES: 334250E 6248044N

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

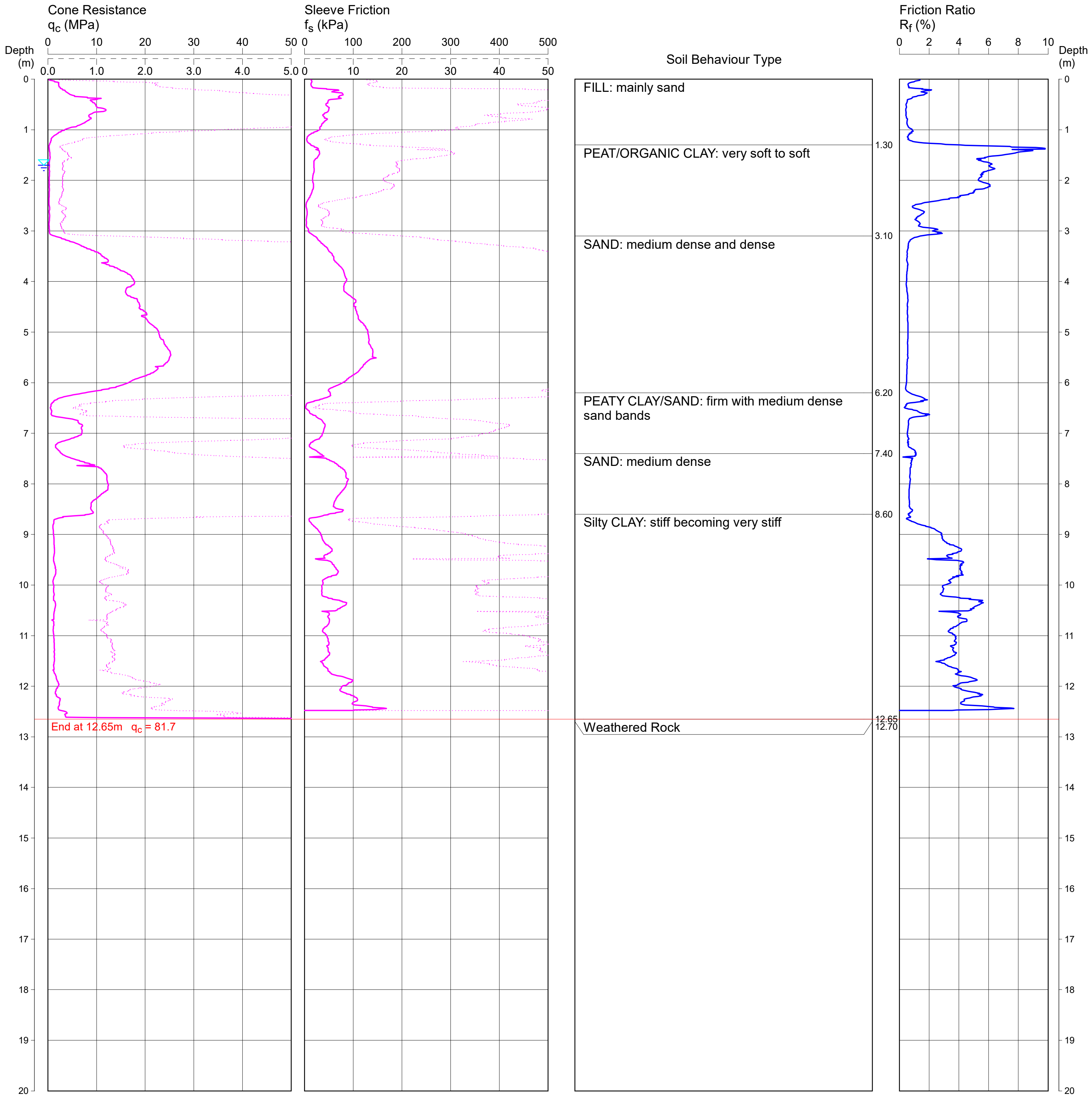
COORDINATES: 334222E 6248013N

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

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: 30.4

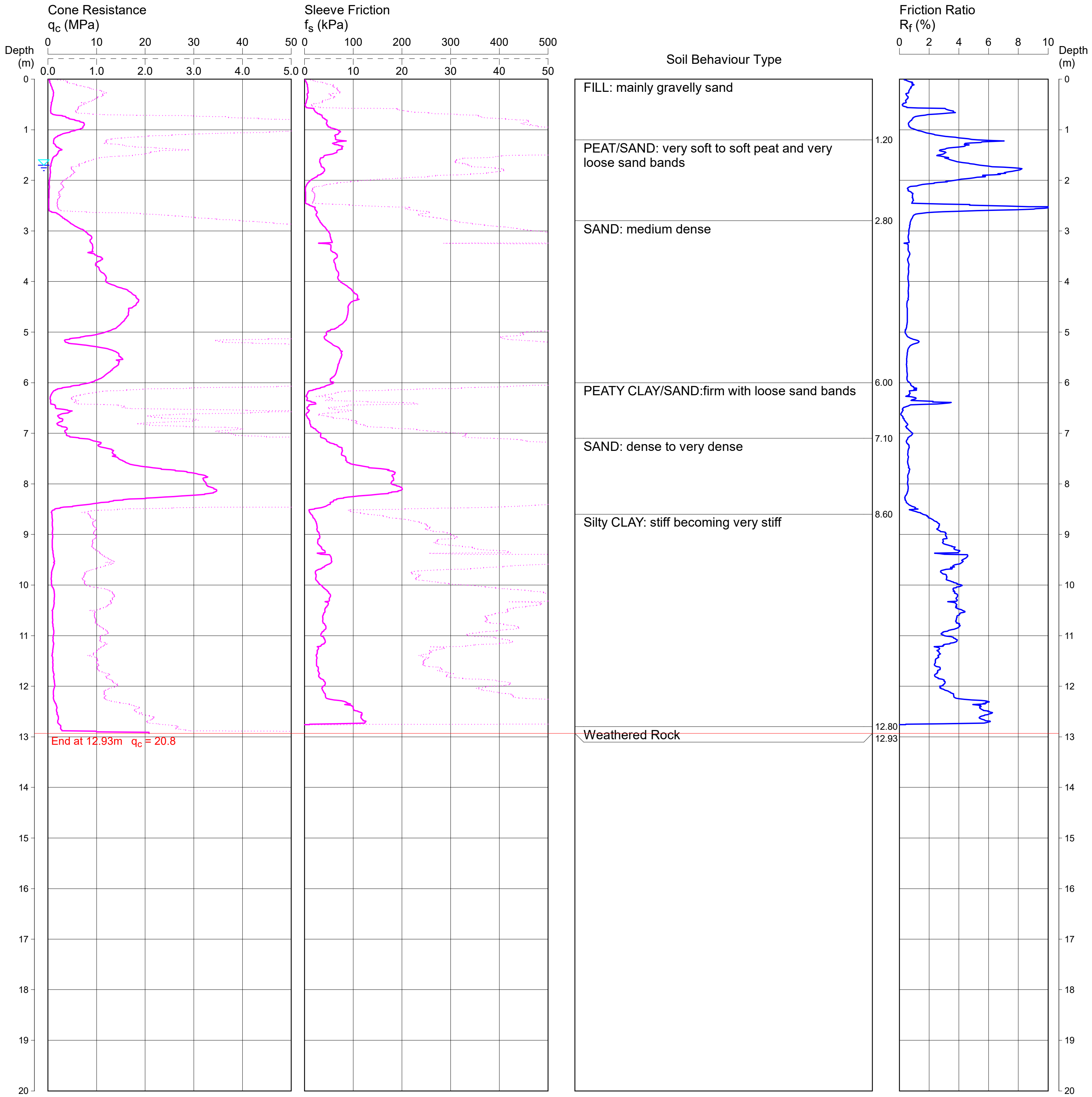
COORDINATES: 334257E 6248012N

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

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: 30.4

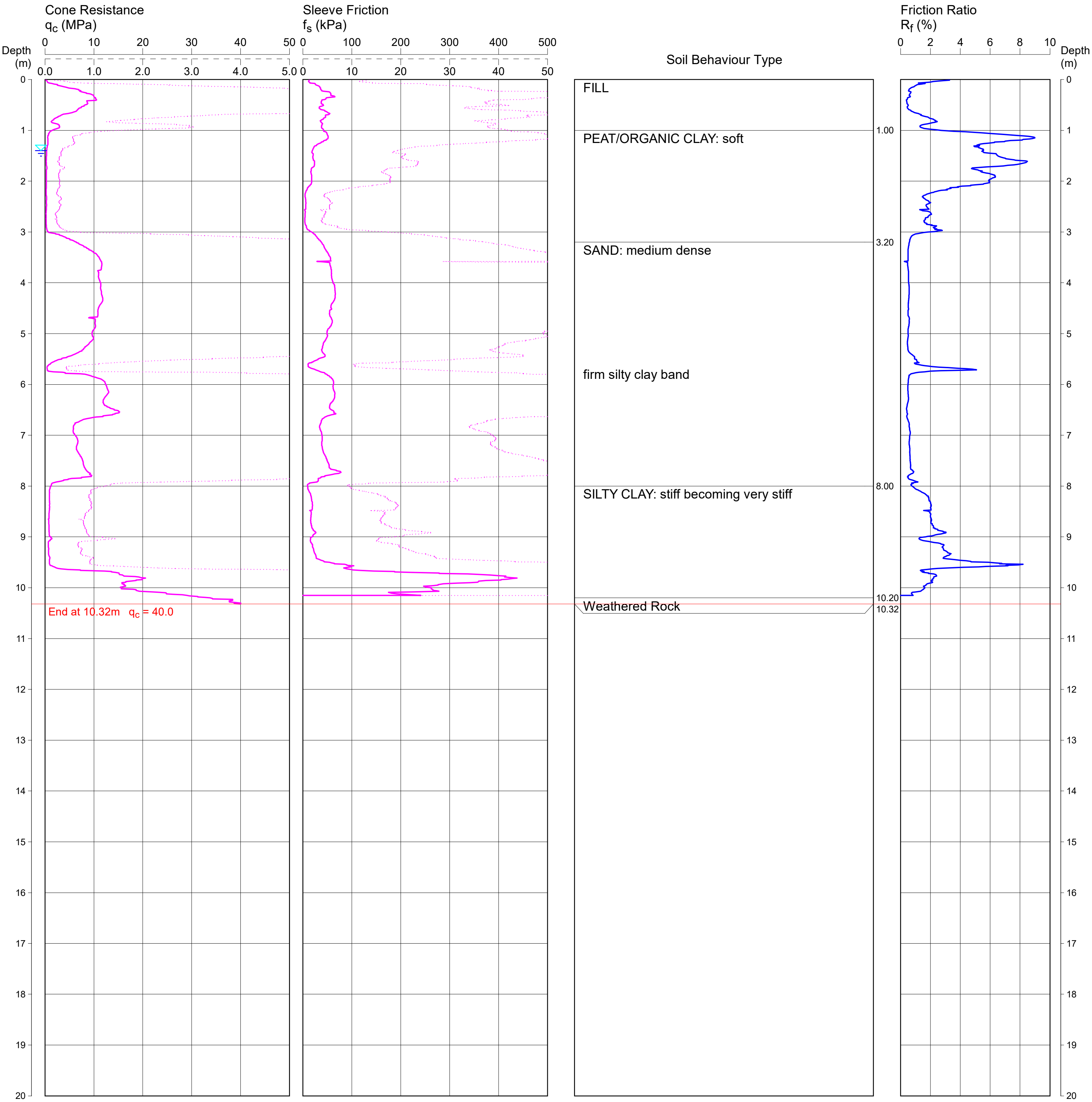
COORDINATES: 334214E 6247979N

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

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: 30.0

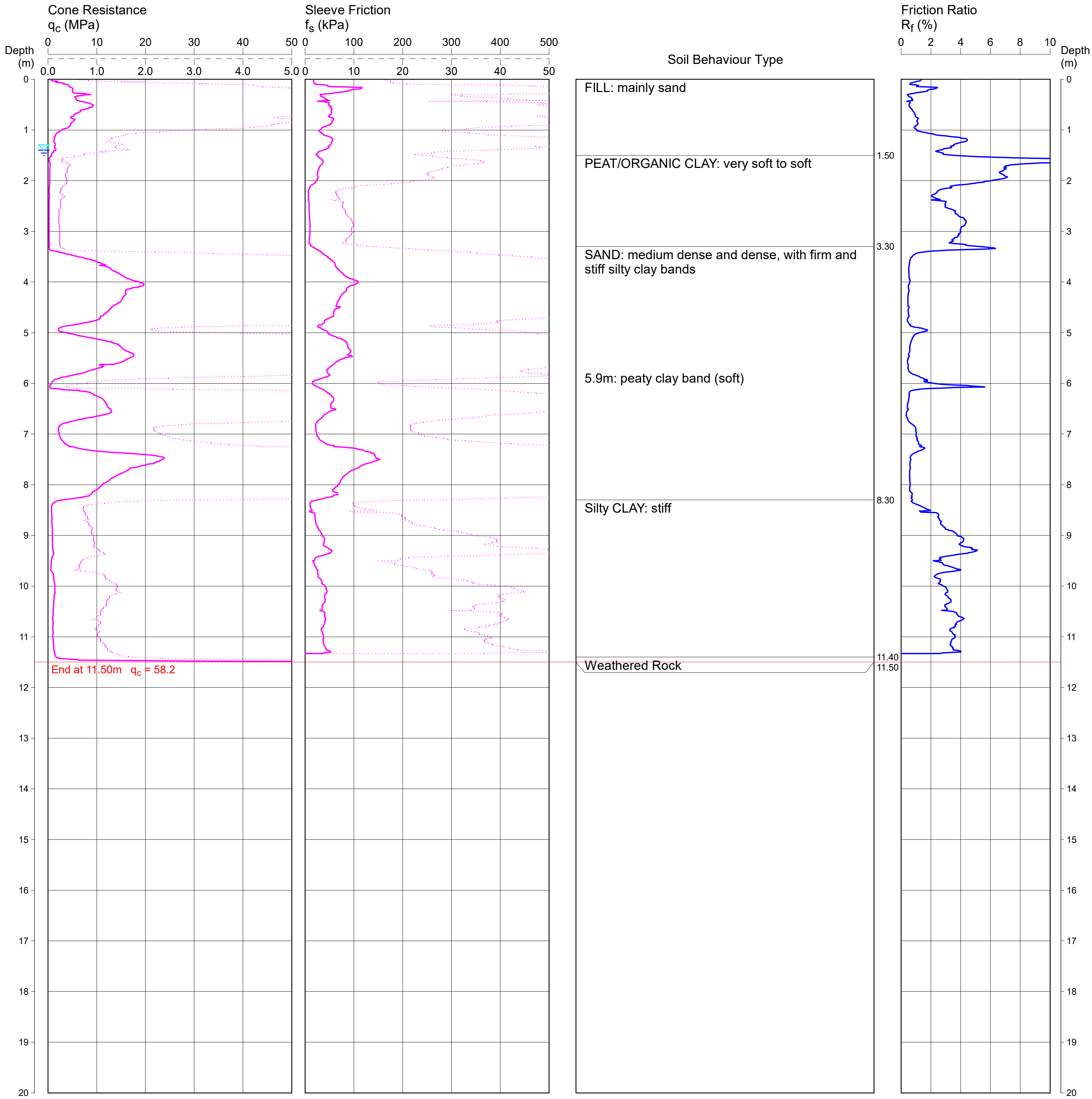
COORDINATES: 334264E 6247969N

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

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: 30.1

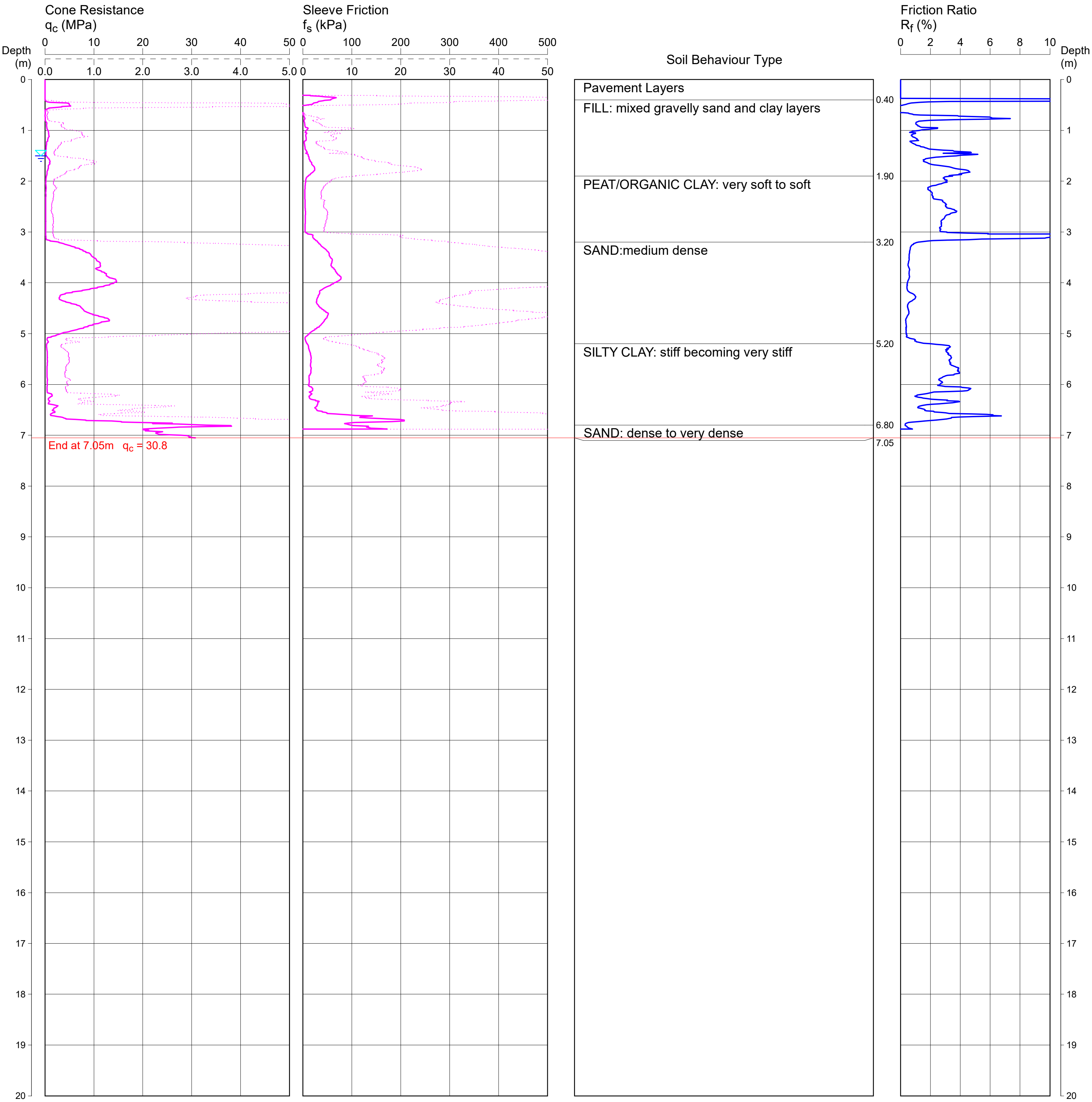
COORDINATES: 334240.8E 6247916.9N

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

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix D

Results of Laboratory Tests

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

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

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing

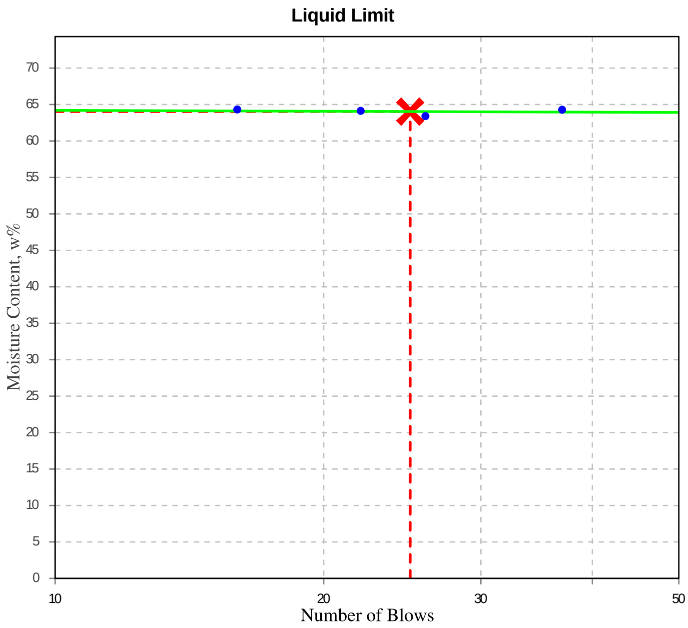
Lujia Wu

Approved Signatory: Lujia Wu
soil technician

NATA Accredited Laboratory Number: 828

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

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

WORLD RECOGNISED ACCREDITATION

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-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

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-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

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

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
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Phone: (02) 9809 0666
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Email: lujia.wu@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

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-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

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

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

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
Dates Tested: 06/12/2019 - 11/12/2019

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing

Lujia Wu

Approved Signatory: Lujia Wu
soil technician
NATA Accredited Laboratory Number: 828

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

Envirolab Services Pty Ltd
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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 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.