Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Glebe Mid-Rise Project
31 Cowper Street and 2A-2D Wentworth Park Road,
Glebe

Prepared for
New South Wales Land and Housing Corporation

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

	Signature	Date
Author		15 May 2020
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Report on Geotechnical Investigation
Glebe Mid-Rise Project
31 Cowper Street and 2A-2D Wentworth Park Road, Glebe

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed multi-storey development ('Glebe mid-rise project') at 31 Cowper Street and 2A-2D Wentworth Park Road, Glebe. The investigation was commissioned by New South Wales Land and Housing Corporation via a Letter of Agreement dated 29 January 2020, and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal SYD191235 dated 11 December 2019.

The investigation was undertaken in support of a planning proposal for the site and for preliminary design by New South Wales Land and Housing Corporation (LAHC). It was carried out concurrently with a preliminary site investigation for contamination (PSI: reported separately). At the time of the investigation, the two parts of the site were occupied by either a two-level residential building with central courtyard (31 Cowper Street: 'South Site'), or a two-level residential 'townhouse-style' building (2A-2D Wentworth Park Road: 'North Site'), separated by a City of Sydney Council road known as 'Park Lane'.

Drawings for the proposed development by Johnson Pilton Walker Pty Ltd (Project 19001, Rev00, dated 1 May 2020) show that the proposed re-development for the site is to include:

- demolition of both existing residential buildings and associated brick boundary walls and fences;
- excavation for basement carparking: two levels beneath the South Site, and one level beneath part of the North Site; and
- construction of two new mixed-use, multi-storey buildings with up to eight above-ground levels (i.e. Lobby and Levels 01 to 07), close to the site boundaries.

The geotechnical investigation scope of work included drilling a total of seven boreholes at the site, dynamic cone penetrometer testing at two locations, and installation and follow-up measurement of one standpipe piezometer in a completed borehole. The investigation was undertaken to provide information on the subsurface profile, groundwater levels, excavation conditions, and to provide comments on design and construction issues. Details of the geotechnical field work and laboratory testing carried out are given in this report. Analytical results and a discussion on acid sulfate soils is presented within the PSI report.

2. Site Description

The South Site and North Site, known respectively as Lots 17 and 18 in DP 244897, are both roughly trapezoidal-shaped fenced parcels of land which are separated by a north-west to south-east trending road known as Park Lane. Both sites are bounded by Mitchell Lane to the west and Cowper Street to the east. The South Site is bounded by Wentworth Street (ie to the south), and the North Site is bounded by Wentworth Park Road. The two sites are shown on Drawing 1 in Appendix C. It is noted that Wentworth Park is present nearby to the north, and Blackwattle Bay is present some 500 m north-west of the site.

As shown on the survey drawing provided, the South Site (ie Lot 17) has maximum plan dimensions of some 37 m by 40 m and an area of 1163 m^2 (refer Drawing 1 of Mepstead and Associates Pty Ltd, Project 5743, Rev B, dated 27 February 2019). The survey drawing provided shows that the North Site (ie Lot 18) has maximum plan dimensions of some 30 m by 28 m and an area of 626 m^2 (refer Sheet 1 of Veris Australia Pty Ltd, Project 201704, Issue 1, dated 19 August 2019).

Both the South Site and North Site are relatively flat, with a slight slope down to north-east, towards Wentworth Park. In accordance with the provided survey drawings, site levels fall from approximately RL3.2 m along Wentworth Street to RL2.5 m at the Wentworth Park Road property boundary (relative to the Australian Height Datum: AHD). Site photographs taken during the field work period are presented in Appendix B.

It is noted from a historical 'Parish of Petersham' map for the Municipality of The Glebe (dated 1890) that both an area of 'reclaimed land' and the historical high-water mark for the nearby Blackwattle Bay are approximately co-incident with the southern property boundary of Lot 18 and Park Lane. This historical high-water mark has been included on Drawing 1.

Nearby land uses include:

- South Site (Lot 17):
    - a playground to the west;
    - low and medium density residential dwellings to the south; and
    - mixed commercial and high density residential to the east.
- North Site (Lot 18):
    - low and medium density residential dwellings to the west;
    - open spaces to the north; and
    - mixed commercial and high density residential to the east.

3. Geological Setting

Reference to the Sydney 1:100 000 Geological Series Sheet (Herbert, 1983) indicates that the site is underlain by man-made fill over Quaternary alluvial and estuarine sediment (Qha), underlain by Hawkesbury Sandstone. Alluvium is generally silty to peaty quartz sand, silt and clay. The Hawkesbury Sandstone typically comprises horizontally bedded and vertically jointed, massive and cross-bedded, medium grained quartz sandstone with a few shale interbeds.

A parallel series of intrusive igneous dykes are indicated on the geological map, traversing approximately north-west to south-east about 50 m north of the site, and approximately parallel with Wentworth Park Road. Igneous dykes typically weather to form deep residual soils.

Reference to the Soil landscapes of the Sydney 1:100 000 sheet (Chapman and Murphy, 1989) indicates that the site is underlain by disturbed terrain ("extensively disturbed by human activity").

Reference to the Botany Bay 1:25 000 Acid Sulfate Soil Risk Map (Murphy, 1997) indicates that the site is located near to an area of disturbed terrain ('X2').

Based on a review of Crown drawings for the site, reclamation / levelling of the site had occurred prior to the year 1890.

The drilling confirmed the presence of filling materials and estuarine alluvial sediments, underlain by Hawkesbury Sandstone.

4. Field Work Methods

The field work was undertaken between 21 January 2020 and 24 January 2020, and included:

- scanning for buried services using a scanning sub-contractor;
- drilling of seven boreholes (Boreholes BH1 to BH7), to depths ranging between 0.7 m and 15.38 m, with two of the boreholes drilled using hand tools (ie BH3 and BH7), and the other five boreholes drilled using track-mounted drilling rigs;
- completion of dynamic cone penetrometer (DCP) tests at two locations (ie adjacent to Boreholes BH3 and BH7), taken to depths of 0.62 m and 2.35 m;
- installation of one standpipe piezometer within Borehole BH4, screened within soil (refer borehole log for standpipe construction details);
- groundwater observations during auger drilling; and
- development of, and groundwater sampling from, the standpipe piezometer on 29 January 2020.

It is noted that the surface pavers and concrete in boreholes BH2 and BH4 were dia-cored. The boreholes were drilled within soils (and between 0.1 m to 0.3 m into weathered sandstone within Boreholes BH1, BH2 and BH6) using auger drilling methods. With the exception of Boreholes BH3 and BH7, five of the boreholes were extended into the underlying rock using rotary diamond core drilling techniques.

Selected soil samples obtained during auger drilling were submitted to an analytical laboratory, for analysis of soil pH, electrical conductivity, sulfate and chloride concentrations. Additional analyses were carried out for the assessment of typical contaminants of concern and the potential for acid sulfate soil conditions: the results of these tests are included in the PSI report.

All field work was carried out under the full-time supervision of a geotechnical engineer, with logging of the soil undertaken generally in accordance with Australian Standard AS 1726 (2017).

Coordinates and surface levels for the test locations were obtained using either a differential Global Positioning System receiver (dGPS: Borehole BH4), or interpolated based on site measurements and the site survey drawings provided. The co-ordinates and surface level of Borehole BH4 are considered to have an accuracy of 0.1 m in both plan and elevation, whereas the co-ordinate accuracy for the other boreholes is considered to be about 0.5 m in plan and 0.1 m in elevation. Coordinates are in GDA94 / MGA Zone 56 format (Geocentric Datum of Australia 1994 base, with Map Grid of Australia projection) and elevations are measured relative to the Australian Height Datum (AHD). The test locations are shown on Drawing 1 in Appendix C.

The weather on two of the field days was mostly fine and sunny, however, periods of rain occurred on 21-22 January 2020, and 24 January 2020.

Further details of the methods and procedures employed during the site investigation are presented in the Notes About This Report, in Appendix A.

5. Field Work Results

The subsurface conditions encountered within the boreholes are presented on the attached logs in Appendix D, along with standard notes defining the descriptive terms and the classification methods used.

The subsurface conditions encountered during the investigation can be summarised as:

- *Within the North Site*
    - FILL – sand and gravelly sand fill up to 1.0 m depth, over sandy clay filling with some brick, sandstone gravel, ash, plastic, charcoal, slag, glass and ceramic tile to 1.9 m depth (elevation down to RL0.8 m), generally in a loose or medium dense condition; over
    - ALLUVIUM – very soft to stiff and loose to medium dense, orange-brown, grey or red-brown clayey sand, low to medium plasticity sandy clay or high plasticity clay, moist then wet (moisture content for cohesive soils greater than the plastic limit below an elevation of RL0.3 m); over
    - RESIDUAL – dense, pale grey clayey sand, wet (possibly extremely weathered sandstone: Borehole BH5 only); over
    - SANDSTONE – medium to coarse grained, very low to low strength becoming high strength sandstone.
- *Within the South Site*
    - FILL – sand, gravel, gravelly sand, and clayey sand fill up to 2.4 m depth (to an elevation of RL1.0 m), including some inclusions of brick, sandstone gravel, ash, plastic, charcoal, slag, glass and ceramic tile, generally in a loose condition, and some surface concrete slabs and gravelly sand ‘roadbase’ materials associated with footpaths and car parking areas (Boreholes BH2 and BH4 only); over
    - ALLUVIUM – very soft to stiff, mottled orange-brown and grey, grey and dark grey low to medium plasticity sandy clay or high plasticity clay, moisture content equal to or greater than the plastic limit below an elevation of between RL0.2 m and RL0.7 m); over
    - RESIDUAL – dense, pale grey-brown clayey sand, wet (possibly extremely weathered sandstone: Borehole BH4 only); over

- SANDSTONE – medium to coarse grained, very low strength becoming high strength sandstone, with dark grey, very low strength and highly to slightly weathered carbonaceous shale encountered in Borehole BH1 between 10.08-10.70 m depth.

Core loss within cored borehole BH5 is interpreted to represent extremely low strength bedrock, washed away during the coring process.

A summary of the surface levels and depths at which various strata were encountered during the investigation is presented in Table 1. It is noted that the top of rock is indicated to reduce in 'steps' towards the north-east and Wentworth Park, with a 2 m difference in elevation between the levels for the top of rock in boreholes BH5 and BH6.

Table 1: Surface levels and Summary of Subsurface Profile at Test Locations

Test ID	Surface RL (AHD) and Top of Concrete / Filling Materials	Top of Alluvium		Top of Extremely Low to Very Low Strength Sandstone		Top of Medium or High Strength Sandstone[2]	
		Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)
BH1	3.2	1.8	1.4	5.2	-2.0	7.1	-3.9
BH2	3.5	2.4	1.1	5.1	-1.6	5.3	-1.8
BH3	3.5	>0.7	<2.8	*ne*	*ne*	*ne*	*ne*
BH4	2.9	1.9	1.0	6.3	-3.4	6.9	-4.0
BH5	2.7	1.8	0.9	*ne*	*ne*	6.0	-3.3
BH6	2.7	1.9	0.8	8.0	-5.3	8.2	-5.5
BH7	3.5	>1.5	<1.9	*ne*	*ne*	*ne*	*ne*

Notes: 1. 'ne' denotes the material was not encountered.
2. Taken as consistent medium or high strength sandstone, below thick clay seams.

It should be noted that sub-horizontal clay-coated bedding plane defects or seams of clay (up to 200 mm thick) were present in the upper 1.7 m of core. High strength rock was typically encountered below these defects, with joint defects within the medium or high strength sandstone observed to be widely-spaced and either steeply-dipping (70-90 degrees) or moderately dipping (45-60 degrees).

Groundwater was observed within the five boreholes drilled through the alluvium (ie either during auger drilling or within the standpipe piezometer in Borehole BH4), with groundwater observed within the alluvium at an elevation of between RL0.1 m and RL0.3 m. The water level within the standpipe piezometer (Borehole BH4) was measured both 9 and 16 days following standpipe installation, at depths of 2.84 m and 2.64 m (respectively) within sandy clay alluvium (lower than was observed during the drilling).

During development, about 90 litres of water was pumped from the standpipe using a low flow pump. Following the pumping, it was noted that the water level within the standpipe was similar to the pre-pumping level, indicating a high permeability of the surrounding soil and a rapid rate of groundwater recharge.

6. Laboratory Testing

6.1 Rock Core

Laboratory testing was completed on selected samples of rock core from each of the cored boreholes for Point Load Strength Index (Is_{50}) testing (a total of 45 tests). The results are presented on the borehole logs and show Is_{50} values in the range 0.2 - 3.1MPa, with the indicative rock strength ranging from very low, up to high to very high strength. The estimated unconfined compressive strengths (UCS) from point load strength test results, using a conversion factor of 15 to 20, are up to about 60 MPa.

6.2 Soil Samples – Chemical Analysis

Four soil samples from the boreholes were tested in a NATA-accredited analytical laboratory to determine soil aggressivity (pH, electrical conductivity, sulfate and chloride ion concentrations). Analysis of soil samples was also carried out for common contaminants of concern, and both screening and quantitative tests for assessment of potential acid sulfate soil conditions. The contamination and acid sulfate soil test results are presented and discussed in the PSI report, and are not further discussed in this report.

The soil aggressivity results are summarised in Table 2, with the laboratory test report included in Appendix E.

Table 2: Laboratory Test Results for Aggressivity to Buried Concrete and Steel

Sample ID	Sample Description	Elevation of Sample[1] (RL m)	pH	EC[2] (µS/cm)	Chloride (mg/kg)	Sulfate (mg/kg)
BH1, 0.9-1.0m	Gravel and Sand (Fill)	2.3	8.2	130	10	26
BH1, 2.5-2.95m	Clay (Alluvium)	0.7	6.2	260	140	280
BH4, 2.5-2.6m	Sandy Clay (Alluvium)	0.4	7.1	170	29	200
BH7, 1.2-1.3m	Sandy Clay (Fill)	2.3	8.4	100	20	10

Notes: (1) Elevation quoted is for the 'top' of the sample.
(2) EC = Electrical Conductivity. (3) Analysed soil was tested as a 1:5 mixture of soil:water.

7. Proposed Development

The architectural drawings prepared by Johnson Pilton Walker Pty Ltd (Project 19001, Revision 00, dated 1 May 2020) show that the proposed development is to include two buildings separated by Park Lane, known as the North Site (i.e. 2A-2D Wentworth Park Road) and the South Site (i.e. 31 Cowper Street). Drawings A-1000 and A-1001 show that the South Site is to have two basement levels for car parking and machine rooms, and that the North Site has one level of basement car parking (within the southern portion of the development footprint).

8. Geotechnical Model

The geotechnical model for the site is a 2 m - 3.5 m thick layer of fill (mixtures of sand, gravel, clayey sand, and sandy clay), generally in a loose condition and with inclusions of brick, sandstone gravel, ash, plastic, charcoal, slag, glass and ceramic tile, then very soft to stiff sandy clay or clay alluvium (2.7 m - 5.9 m thick, below elevations in the range RL0.8 m to RL1.4 m, decreasing to the north-east), overlying either dense residual clayey sand or sandstone. Below the 'top of rock', the sandstone in the boreholes is either very low or medium strength. It appears to 'step down' in level towards the north-east, from an elevation of RL-1.6 m in the south-east of the site (Borehole BH1) to RL-5.3 m within the North Site (Borehole BH6: refer Drawings 2 and 3).

The site's location along the former, undeveloped limit of Blackwattle Bay is indicated by Park Lane's position sub-parallel to the historical high-water mark (circa 1890: refer Drawing 1), and the historical naming of Wentworth Street (east of the site) as 'Water Street'.

The cross-sections show the interpreted geotechnical divisions of underlying soil and rock. The interpreted boundaries shown on the sections are accurate at the borehole locations only and layers shown diagrammatically on these drawings are inferred strata boundaries only. Reference should be made to the borehole logs for more detailed information and descriptions of the soil and rock. In particular, the bedrock profile beneath Park Lane and the North Site are expected to step down in a series of small cliff lines and benches. The linear representation of the interpreted top of rock shown is diagrammatic only and is likely to be mis-leading in terms of actual rock levels, at least in some areas of the site.

The rock encountered within the cored boreholes drilled as part of the current investigation has been classified in accordance with the procedures given in Pells et. al. (1998), which use a combination of rock strength and fracture spacing to divide the rock into five classes ranging from Class I (medium to high strength and very few defects) to Class V (extremely low to very low strength and/or highly fractured). The interpreted depth and Reduced Level (RL) at the top of the rock classes are shown in Table 3.

It should be noted that closely-fractured zones, weak seams or bands and core 'loss' zones can occur within higher strength rock, and as such the classification may be downgraded in these areas. Some zones of higher strength rock were 'down-rated' due to the presence of closely spaced defects observed in the rock cores. It is possible that some of the core loss and fractured zones are drilling induced.

It is inferred from the soil electrical conductivity test results, and the measured water electrical conductivity results (during water sampling) that the groundwater observed within the alluvial sediments is not seawater or brackish water, such as from the nearby tidal Blackwattle Bay (about 500 m from the site). Groundwater levels and flow rates are likely to vary over time, depending on rainfall and downslope drainage conditions.

Table 3: Summary of Depths (and Reduced Levels) to Top of Rock Classes

Bore hole	Surface RL (m AHD)	Depth and Reduced Levels (m AHD) to top of Rock Classes[1]									
		Class V		Class IV		Class III		Class II		Class I	
		Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)
BH1	3.2	5.2	-2.0	ne[2]	ne[2]	7.1[3]	-3.9[3]	ne	ne	ne	ne
BH2	3.5	5.1	-1.6	5.3	-1.8	ne	ne	ne	ne	6.9	-3.4
BH3	3.5	>0.7	<2.8	ne	ne	ne	ne	ne	ne	ne	ne
BH4	2.9	ne	ne	6.3	-3.4	7.6	-4.7	ne	ne	8.4	-5.5
BH5	2.7	ne	ne	ne	ne	6.0	-3.3	ne	ne	6.3[4]	-3.6[4]
BH6	2.7	8.0	-5.3	ne	ne	8.2	-5.5	ne	ne	11.0	-8.3
BH7	3.5	>1.3	<2.2	ne	ne	ne	ne	ne	ne	ne	ne

Notes: (1) Rock classification is based on Pells et al., 1998.

(2) "ne" denotes that this class of rock was Not Encountered.

(3) An interval of carbonaceous shale was encountered within this interval.

(4) A 150 mm interval of core loss and fragmented core was encountered at an elevation of RL-7.6 m, the designer should consider whether this rock class is appropriate if deep foundations are being considered in this part of the site.

9. Comments

9.1 Geotechnical Issues

Some of the primary geotechnical issues that need to be considered for the development include:

- Groundwater is shallow, and dewatering will be required for the construction of basements;
- Potential acid sulfate soils (PASS) are likely to be encountered within the alluvial soils (discussed further within a separate PSI report);
- Shoring walls for basements will need to be designed to reduce groundwater inflow to the basements and to control drawdown of water levels on adjacent sites: drawdown has the potential to cause ground settlement and to generate acidic conditions in any discharge water;
- The shoring will need to be socketed into competent rock, which can be problematic for some shoring systems and can result in decompression and loosening of the surrounding soils;
- If cut-off walls into rock are successfully constructed to reduce inflow and drawdown of groundwater then it is technically feasible to construct a tanked basement, otherwise it will be necessary to use a partially drained basement. All this, however, will be subject to review and approval by both The City of Sydney Council and by Water NSW;
- For low long-term impact on the groundwater level, a 'tanked' basement should be constructed to reduce the need for long-term collection and removal of groundwater inflows. A tanked basement will need to be designed for hydrostatic uplift.

9.2 Site Classification

Filling materials were encountered within the boreholes at the site, interpreted to occur to depths of up to 2.4 m below the current ground surface. Based on visual observations and SPT testing, the fill materials appear typically to be in a loose condition. Some inclusions of brick, sandstone gravel, ash, plastic, charcoal, slag, glass and ceramic tile were observed within the filling. Compaction or density testing records of this material, inferred to have been placed during raising / levelling of the site (including prior to 1890), were not available and this material is therefore considered to be ‘uncontrolled’.

As there is more than 0.8 m of uncontrolled sand filling, the site is classified as Class P, when assessed in accordance with Australian Standard AS2870 (2011). It is noted that trees are present near to the site boundaries and within footpaths nearby.

9.3 Site Preparation and Trafficability

9.3.1 General

Site preparations are likely to include:

- the demolition and removal of the existing residential buildings and associated retaining walls (including the possible need for selective removal of trees from the site);
- the creation of working platforms for machinery such as piling rigs / excavators and mobile cranes; and
- installation of basement shoring and cut-off walls along the perimeter of the basement footprint(s).

The creation of access for machines between the North Site and South Site (ie across Park Lane) should also be considered (such as via a road closure over the construction period).

The encountered near-surface fill materials at the site varied between granular and cohesive materials. Sandy clay fill is likely to be exposed within the North Site following surface stripping, which may become slippery and difficult to traverse following periods of rain. Some rutting / surface damage of this material should be expected, particularly if traversed following periods of prolonged rainfall. It is anticipated that tracked machines would be able to safely travel over the site and work upon the fill materials while they are exposed, but larger machines may not be able to work on the existing filling without a granular working platform, possibly including a layer of crushed rock or concrete of at least 300 mm thickness and maybe with some additional geogrid reinforcement, in conjunction with some rolling with a 5-tonne smooth drum roller.

For support of mobile crane outriggers and piling equipment, an assessment of the required thickness of working platform should be made once this equipment has been selected.

For the construction of pavements, the following site preparation measures are recommended:

- Excavate to the pavement subgrade level within the pavement footprint(s);
- Remove any vegetation-affected fill materials and any other deleterious materials below design / bulk excavation levels;

- Test roll the exposed surface using a minimum 12-tonne smooth drum roller in non-vibration mode. The surface should be rolled a minimum of six times with the last two passes observed by an experienced geotechnical engineer to detect any 'soft spots', in accordance with the project Specification;
- Any heaving materials identified during proof rolling should be removed as directed by the geotechnical engineer; and
- Placement of suitable filling materials up to design levels, and density testing of the compacted layers, should be undertaken in accordance with the project Specification.

9.3.2 Dilapidation Surveys

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

9.4 Groundwater

Groundwater was encountered during drilling within all five of the machine-drilled boreholes, or subsequently measured within the standpipe piezometer screened within the soil (ie Borehole BH4), at elevations in the range RL0.2 m to RL0.3 m. The high volume of water able to be extracted from the standpipe piezometer screened within the soil, and the observed rapid rate of recharge, indicate an unconfined aquifer with a steady-state piezometric surface of around RL0.3 m.

Allowing for some variability, the groundwater observations and measurements indicate that excavations for either one- or two-level basements within the northern and southern parts of the site will intersect the regional groundwater table. To eliminate the need for continual pumping and disposal of groundwater from the basements, and to prevent drawdown of the groundwater beneath nearby properties, a cut-off wall into the underlying rock / low permeability layer will be required for the full height of each basement. The potential remains for groundwater inflows to occur through the base of the excavations through the rock, which may require a water-tight or 'tanked' basement to be constructed. Further discussion on cut-off walls and basement tanking is presented in Section 9.8.1.

In accordance with the survey plans provided, ground surface levels for the southern and northern parts of the site vary between RL3.2 m and RL2.5 m. On the basis of the measured water levels and allowing for increases in water levels due to heavy rainfall, flooding and construction of new basements, it is recommended that a design groundwater level for the site of RL3.2 m be adopted.

9.5 Excavation Conditions

Bulk excavations for the proposed development would be about 3.5 m for the North Site and 6.2 m for the South Site, with localised deepening for a car lift over-run.

The final finished level at the base of the bulk excavations range between about RL3 m and RL0 m (ie for the South Site and North Site, respectively), and will be carried out through fill materials and clayey alluvial soils. Excavations within the South Site are also likely to encounter extremely low strength sandstone, over high strength, slightly fractured sandstone.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment. Excavation of medium or high strength rock may require moderate to heavy ripping with a large bulldozer. High strength rock will probably require hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material.

Detailed excavation for footings and services, or side trimming, within medium or high strength rock will generally require the use of a rotary rock saw or milling head, possibly in conjunction with a hydraulic rock hammer or vibrating rock ripper. Rock saws or milling heads may also be required to reduce vibrations near existing structures for human comfort and to reduce the potential for causing damage to such structures.

9.6 Disposal of Excavated Material

Off-site disposal of excavated material will require assessment for re-use or classification in accordance with *Waste Classification Guidelines* (NSW EPA, 2014), prior to disposal to an appropriately licensed landfill or receiving site. This includes filling and virgin excavated natural materials (VENM), such as may be removed from this site.

It is noted that chemical analysis for commonly occurring contaminants was carried out on a groundwater sample from the standpipe piezometer, and on selected soil samples obtained from boreholes. The results of these tests are presented in a PSI report, which is being prepared under separate cover (refer to DP Report 99554.01.R.001, in preparation). Subject to the recommendations given in that report, further environmental testing may need to be carried out to classify excavated spoil prior to disposal. The type and extent of testing to be undertaken will depend on the final use or destination of the spoil, and the requirements of the receiving site. The results of the environmental testing are not further discussed herein.

9.7 Vibrations

During excavation, construction vibration may be generated which, if not controlled, could possibly result in damage to nearby structures and underground services (eg closer than 20 m). Therefore it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of structure (eg reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the structure, and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/sec peak particle velocity (PPV), which is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian / International Standard AS/ISO 2631.2 (2014) indicates that a PPV of 8 mm/sec at the ground level of nearby structures is below the normal building damage threshold.

Some of the nearby buildings are most likely to be supported on high level footings founded on firm alluvial clay or loose sandy fill materials. Vibrations have the potential to induce settlements which may result in damage to adjacent buildings. For this reason, it is suggested that a tentative maximum PPV of 3 mm/sec (applicable at the foundation level of existing buildings) be employed at this site for both architectural and human comfort considerations. A higher limit of up to 8 mm/sec may be adopted for buildings founded on dense sand or rock, however, this will be subject to further geotechnical review and site vibration testing.

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

9.8 Dewatering and Tanking

9.8.1 General and Seepage Rates

The current investigations have indicated groundwater levels at about RL0.1 m and RL0.3 m across the site, with the groundwater surface apparently falling slightly to the north-east. Groundwater levels will vary and may temporarily rise by at least 1 m following prolonged heavy rainfall. In the absence of more detailed and long-term measurements it is suggested that a water level 2 m above the current measured levels should be considered for basement design (ie within about 0.5 m of the current surface level). Flood levels should also be considered if applicable to this site.

The proposed bulk excavations will extend to a depth below the measured groundwater table by between about 3.3 m for the South Site and 0.3 m for the North Site. If site dewatering results in excessive drawdown (lowering of the water level) beneath surrounding sites then this has potential to induce settlement, and also to expose PASS which may result in generation of acidic soil and groundwater conditions. Existing groundwater contamination on the site, if applicable, should also be considered in the planning.

To reduce groundwater flows into the basement and thereby reduce potential impacts to the surrounding groundwater, potential acid sulfate soils, and neighbouring buildings/pavements, a relatively water-tight “cut-off” wall should be formed around the perimeter of the basement excavation. As a guide, it is suggested that the cut-off wall should be socketed at least 2 m into consistent medium strength or stronger rock. It may be possible to justify a socket in lower strength rock to avoid installing shoring through several metres of weathered rock, however, this will be subject to further geotechnical review and probably groundwater modelling.

Some of the upper weathered rock profile includes fractured zones and lower strength bands. It is possible that relatively high seepage flows may still occur through fractured zones in the rock, if exposed in vertical rock excavations below the cut-off wall. The seepage could be controlled/reduced by grouting of the fractured rock (eg ‘primary’, ‘secondary’, and ‘tertiary’ grouting to control inflows), however, this is difficult to carry out when seepage is flowing into the excavation.

An alternative approach could be to install the cut-off walls into slightly weathered to fresh, slightly fractured and unbroken, high strength rock below the bulk excavation level. This option would be expected to significantly reduce seepage flows, as it will only occur though the relatively low permeability medium or high strength rock below the basement floor. This option may effectively reduce inflow rates into the basement to the extent that a drained basement may be justified without significant impact on groundwater levels on surrounding sites.

Flow through rock is typically governed by joint defects: if highly fractured areas of rock are exposed in the excavation floor then pressure grouting may need to be undertaken to reduce inflows.

Further detailed investigations and groundwater modelling would be required to predict seepage rates and drawdown in both the short and long term. Modelling would also be required to assess whether a cut-off wall into rock below the bulk excavation may be used to allow a drained basement. However, a drained basement will be subject to review and approval by both The City of Sydney Council and by Water NSW.

If a drained basement is not possible then a water-tight ‘tanked’ basement will be required for the permanent basement structure. A tanked basement would need to be designed to resist uplift forces associated with (hydrostatic) groundwater pressures, for which preliminary design could be based on a groundwater level 2 m above the current measured levels.

9.8.2 Drawdown and Settlement

It is suggested that the design and construction of the basement should be carried out to target a drawdown on adjacent properties of less than 1 m. As a minimum, this will require perimeter cut-off walls into rock, and possibly installed into rock below the bulk excavation level to cut off horizontal flows through rock into the excavation. Further modelling may indicate that a tanked basement is required, to reduce long-term drawdown to acceptable levels. A drawdown of 1 m would be expected to be within the range of previous groundwater level fluctuations and therefore settlements due to drawdown of 1 m within the soils may be relatively minor (ie possibly less than 5 mm).

During construction, it is recommended that groundwater drawdown outside of both the excavation and within the vicinity of the nearby properties should be monitored, in general accordance with the following procedure:

- Install standpipes in accessible areas on adjacent properties (or roads) to monitor groundwater drawdown levels during dewatering;
- Measure groundwater levels on a weekly basis for three weeks prior to operation of the dewatering system to establish pre-construction levels;
- Measure groundwater levels twice per day during the first two days of dewatering, daily during the first week of dewatering, and weekly until decommissioning of the dewatering pumps, or until a lesser frequency is advised by the geotechnical engineer;
- The measured values are to be provided to the geotechnical engineer on the day of measurement for review;
- Where drawdown levels exceed a pre-determined ‘trigger level’ (to be set) below pre-construction groundwater levels, the reason for the change in groundwater level should be investigated and measures put in place to rectify the exceedance. These measures could include reduction of pumping rates or suspension of dewatering.

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

9.8.3 Groundwater Disposal

The groundwater removed from the site will require disposal. Reference should be made to the companion PSI report for advice on the contamination status of the groundwater and treatment requirements. DP can carry out testing of groundwater quality and can review this aspect and provide advice, if required.

9.9 Excavation Support

Shoring will be required around the perimeter of the site. It may be possible to terminate shoring on competent rock above the bulk excavation in some areas (ie South Site), followed by vertical unsupported excavation in rock, however, this will need to consider potential groundwater inflows and impacts as discussed in Section 9.8.1. It may be necessary or beneficial to install cut-off walls into rock below the bulk excavation level.

9.9.1 Retaining Wall Systems

The final basement structure should incorporate a water-tight ‘tanked’ basement 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 some mis-alignment can occur even at relatively shallow depths and hence minor gaps can appear in the wall. The potential for mis-alignment and therefore seepage and soil 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 soft soils which can result in damage to adjacent buildings. The use of segmental casing would 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. Universal column sections are “plunged” into the “wet” panel at regular intervals along the wall, to provide bending stiffness, however, past experience with DSM/CSM walls has indicated that the mixing consistency, and consequently the permeability and durability of the wall, needs to be carefully considered particularly within clayey soils and rock. This option is unlikely to be suitable in the clayey soils and may not achieve an effective seal at the rock interface.

9.9.2 Retaining Wall Design

The shoring may need to be supported by internal bracing and/or ground anchors to control deflections, particularly for the South Site where a deeper basement is being considered. The use of temporary ground anchors to support the shoring walls would most likely require steeply-inclined anchors bonded into rock, for which permission from Council and other stakeholders may be required.

Shoring walls should be founded in rock at least 1.0 m below the base of the bulk excavation level (possibly deeper to reduce water inflow) in order to provide lateral restraint at the base of the excavation and to avoid the risk of adversely inclined joints or wedges undermining the base of the shoring. It may be possible to terminate the shoring walls within unsupported medium strength or stronger sandstone above the bulk excavation level. In this case, it will be important for a geotechnical professional to assess the stability of the rock directly beneath each pile. The toe of the shoring walls which terminate above bulk excavation level will also need to be restrained with rock bolts or anchors.

It is suggested that preliminary design of shoring systems may be based on the earth pressure coefficients provided in Table 4. '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.

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

Table 4: Recommended Design Parameters for Shoring Systems

Material	**Unit Weight (kN/m^3)**	**Earth Pressure Coefficient**		**Effective Cohesion c' (kPa)**	**Effective Friction Angle (Degrees)**
		Active (K_a)	**At Rest (K_0)**		
Fill, and Very soft to firm clays and sandy clays, and loose clayey sand (Alluvium)	18	0.4	0.6	0	20
Stiff sandy clay and clay (Alluvium)	18	0.25	0.4	0	25
Medium dense and dense clayey sand (Alluvium and Residual)	18	0.3	0.45	0	35
Extremely Low to Low Strength Sandstone or Carbonaceous Shale	22	0.1	0.2	100	25
Medium Strength or stronger Sandstone	24	0*	0*	300	40

Note * subject to geotechnical inspection

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

Passive resistance for shoring founded in rock below the base of the bulk excavation (including allowance for services or footings) may be based on the ultimate passive restraint values provided in Table 5. These ultimate values represent the pressure mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety of say 2 to limit wall movement (may be higher if the rock is fractured). The top 0.5 m of the socket should be ignored due to possible disturbance and over-excavation.

Table 5: Recommended Passive Resistance Values

Foundation Stratum	Ultimate Passive Pressure (kPa)
Very low to low strength sandstone	2,000
Medium strength or stronger sandstone	4,000

Detailed design of shoring should preferably be carried out using WALLAP, FLAC or other accepted computer analysis programs, which are capable of modelling progressive excavation and anchoring, and predicting potential lateral movements, stresses and bending moments. FLAC (or similar) would be required if it is necessary to assess ground movements on surrounding properties, as WALLAP will only assess wall movements.

9.9.3 Vertical Rock Excavation

The low to medium strength or stronger sandstone will generally be stable when cut vertically, provided that adversely-oriented joints or other defects are not present. All vertical faces in rock should be inspected by a geotechnical professional at regular depth intervals, to check for both adversely-oriented joints and to assess whether additional stabilisation measures are required (such as rock bolts or shotcrete).

Given that the typical main joint sets within Hawkesbury Sandstone in the Sydney region are oriented sub-parallel to some of the proposed excavation faces, it is expected that some narrow wedges will be formed where these near-vertical joints intersect the excavation faces. Therefore, some rock bolts may be required to stabilise these wedges.

9.9.4 Ground Anchors / Rock Bolts

The design of temporary and permanent ground anchors for the support of shoring systems may be carried out using the maximum bond stresses given in Table 6. The anchors should preferably have their bond length within medium strength and stronger rock. Anchors taken to rock may need to be more steeply-inclined (such as for the North Site where the depth to rock is greater).

Table 6: Recommended Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Very low to low strength rock	50	100
Medium strength rock	500	1000
High strength or stronger rock	1500	3000

The parameters given in Table 6 assume that the drilled anchor holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 33 degrees in soil and 60 degrees in rock, from the base of the shoring or the top of free-standing medium strength or stronger rock. “Lift-off” tests should be carried out to confirm the anchor capacities, and it is suggested that ground anchors should be proof loaded to 125% of the design Working load and locked-off at no higher than 80% of the Working load. 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.

In normal circumstances the building will restrain the basement excavation over the long-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 at 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, and possible conflicts with neighbouring piled footings or existing basements (ie to the east). It is suggested that legal advice should be sought before penetrations into adjacent properties are proposed. Anchoring should only be carried out by an experienced contractor with demonstrated experience in similar ground conditions.

Vertical anchors for uplift support could also be designed using the parameters given in Table 6. The designer should check the cone pull-out failure mechanism by assuming a 90-degree cone for the soil and rock.

9.10 Foundations

Based upon the investigation results and the number of basement levels (as indicated on Drawing 2 and Drawing 3), excavations to the proposed bulk excavation levels will expose high strength sandstone within the South Site, and very soft / loose sandy clay or clayey sand alluvium within the North Site. The very soft alluvial soils are considered to have an allowable bearing pressure of 10 kPa and are likely to be subject to excessive settlements if loaded.

The new buildings for both the South Site and North Site should be uniformly founded on sandstone bedrock. Pad footings may be suitable where rock is exposed subject to loads and settlement tolerances. Piles may also be required in other areas to reach more competent medium strength or stronger rock, to achieve higher bearing capacities.

Pad footings and piles may be designed using the maximum pressures for the various rock strata presented in Table 7 on the following page. Shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the values for compression. Alternatively, if a lower allowable bearing pressure of 3.5 MPa is adopted then testing during construction could be limited to inspection of foundations.

Where footings are located within the zone of influence of adjacent excavations, drawn upward at 45 degrees from the toe of the excavation (such as lift shafts or tanks), the allowable bearing pressure should be reduced by 25% and the excavation floor carefully inspected for adversely oriented joints. Alternatively, the footings may be taken deeper, below the zone of influence.

The design is likely to be governed by serviceability considerations such as settlement criteria and performance, and the ultimate bearing pressures provided in Table 7 will probably need to be lowered in order to limit settlements to an acceptable amount.

Table 7: Recommended Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure		Field Elastic Modulus (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
Sandstone – Class V	1,000	75	3,000	150	50
Sandstone – Class IV	1,000	100	4,000	250	100
Sandstone – Class III	3,500	350	20,000	800	350
Sandstone – Class II	6,000	600	60,000	1,500	900
Sandstone – Class I	10,000	600	120,000	3,000	2,000

To use a bearing pressure value for design of 10 MPa, 100% of the footings should be spoon tested to a depth equivalent to 1.5 times the footing width and additional boreholes should be drilled to 3 m below bulk excavation level. Spoon testing involves drilling a 50 mm diameter hole below the base of the footing, to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak/clay bands. The amount of proving of the founding material of the footings could be reduced to spoon testing 33% of the footings if the bearing pressure is reduced to 6 MPa. (Note that further drilling should be carried out to confirm the rock strength before the suggested bearing pressures can be adopted.) If weak seams are detected then footings may need to be taken deeper to reach suitable foundation material.

For limit state design, selection of the geotechnical strength reduction factor (ϕ_g) in accordance with Australian piling code AS 2159 (2009) is based on a series of individual risk ratings (IRR), which are weighted on numerous factors and lead to an average risk rating (ARR). Therefore, it is recommended that an appropriate geotechnical strength reduction factor be calculated by the pile designer. Preliminary design could be based on a ϕ_g of 0.4, and refined as the design progresses. Footing settlements may be calculated for assessment of the serviceability limiting state using the elastic modulus values given in Table 7.

Foundations proportioned on the basis of the allowable bearing pressures in Table 7 would be expected to experience total settlements of less than 1% of the footing width / pile diameter under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

All pad footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters, and proof drilled or spoon tested as appropriate. Spoon testing should be carried out in at least one third of the footings which are designed for an allowable end bearing pressure of greater than 3,500 kPa.

9.11 Soil / Groundwater Aggressivity to Concrete and Steel Structures

Aggressivity to buried elements was assessed with reference to the results of the soil laboratory tests (ie pH, electrical conductivity, chloride and sulfate content) and Australian Standard AS 2159 (2009). The exposure classification for buried steel is assessed as being Mild, whereas the classification for buried concrete is assessed as being non-aggressive. Consideration should also be given to the PASS and its possible impact on the exposure classification and durability. This will be influenced by the shoring system and basement construction adopted, together with the extent of drawdown of water levels during construction.

9.12 Seismic Design

In accordance with the Earthquake Loading Standard, AS1170.4 (2007), the site has a hazard factor (z) of 0.08. The site sub-soil class for each site would be generally Class D_e, due to the loose / soft alluvial soils (SPT value less than 6) encountered to depths of less than 10 m on the site.

10. References

AS 1170.4:2007, *Structural design actions, Part 4: Earthquake actions in Australia,* Standards Australia.

AS 1726:2017, *Geotechnical Site Investigations,* Standards Australia.

AS 2159:2009, *Piling – Design and Installation*, Third edition, Standards Australia.

AS/ISO 2631.2:2014, *Mechanical vibration and shock – Evaluation of human exposure to whole-body vibration – Vibration in buildings (1 Hz to 80 Hz)*, Standards Australia / International Standards Organisation.

AS 2870:2011, *Residential Slabs and Footings*, Standards Australia.

AS 4678:2002, Earth-retaining Structures, Standards Australia.

Bertuzzi, R. and Pells, P.J.N (2002), *Geotechnical parameters of Sydney Sandstone and Shale*, Australian Geomechanics Journal, Vol. 37, No. 5.

Chapman G.A., and Murphy C.L. (1989), "Soil landscapes of the Sydney 1:100 000 sheet". Soil Conservation Service of New South Wales, Sydney.

Herbert C. (1983), *Sydney 1:100 000 Geological Sheet 9130, 1st edition*. Geological Survey of New South Wales, Sydney.

Murphy C.L. (1997), *Botany Bay 1:25 000 Acid Sulfate Soil Risk Map Sheet, 2nd edition.* Department of Land and Water Conservation, Sydney.

NSW Environment Protection Authority (NSW EPA: 2014), *Waste Classification Guidelines*.

Pells, PJN., Mostyn, G., and Walker, B.F. (1998), *Foundations on Sandstone and Shale in the Sydney region.* Australian Geomechanics Journal, Vol. 33, No. 3.

11. Limitations

Douglas Partners (DP) has prepared this report for this project at 31 Cowper Street and 2A-2D Wentworth Park Road, Glebe, in accordance with DP's proposal SYD191235 dated 11 December 2019 and a 'Letter of Agreement to undertake LAHC 2019/608' dated 29 January 2020. The work was carried out under a modified New South Wales Land and Housing Corporation contract. This report is provided for the exclusive use of New South Wales Land and Housing Corporation for this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

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

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

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

Asbestos has not been detected by observation or by laboratory analysis, either at the surface of the site, or within filling materials at the test locations sampled and analysed, as outlined in a separate environmental report which is presented under separate cover. Building demolition materials, such as concrete, brick, ceramic tile, timber, glass, ash, slag, charcoal and plastic, were, however, located in previous below-ground filling, and these are considered as indicative of the possible presence of hazardous building materials (HBM), including asbestos.

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

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

Douglas Partners Pty Ltd

Appendix A

About This Report

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

Site Photographs

Photograph 1 – View west along Wentworth Street. The approximate position of Borehole BH1 along Mitchell Lane is indicated as shown.

Photograph 2 – View north along Mitchell Lane. The approximate position of Borehole BH1 is indicated as shown.

Douglas Partners *Geotechnics / Environment / Groundwater*	**Site Photographs**	PROJECT:	99554.00
	31 Cowper Street & 2A-2D Wentworth Park Road	PLATE No:	1
	Glebe	REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

Photograph 3 – View north-east at the location of Borehole BH1.

Photograph 4 – View west across Cowper Street. The approximate position of Borehole BH2 is indicated as shown.

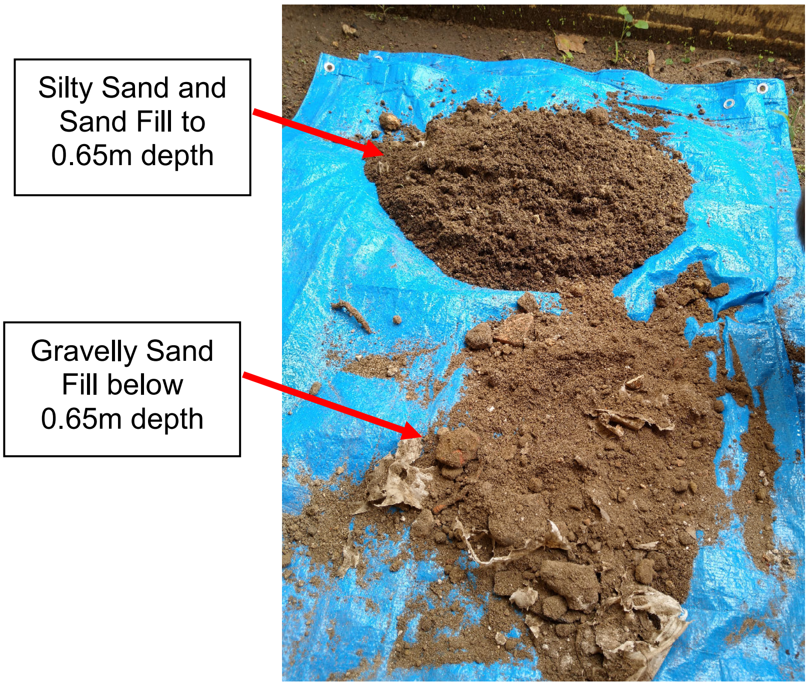
Douglas Partners *Geotechnics / Environment / Groundwater*	**Site Photographs**	PROJECT:	99554.00
	31 Cowper Street & 2A-2D Wentworth Park Road	PLATE No:	2
	Glebe	REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

Photograph 5 – View north-west across Cowper Street. The approximate position of Boreholes BH2 and BH6 are indicated as shown.

Photograph 6 – View north along Cowper Street at Borehole BH2.

Douglas Partners *Geotechnics / Environment / Groundwater*	**Site Photographs** **31 Cowper Street & 2A-2D Wentworth Park Road** **Glebe**	PROJECT:	99554.00
		PLATE No:	3
		REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

Photograph 7 – View south within the 31 Cowper Street property. The position of Borehole BH3 is indicated as shown.



Photograph 8 – View of drilling spoil obtained from Borehole BH3.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs** **31 Cowper Street & 2A-2D Wentworth Park Road** **Glebe**	PROJECT:	99554.00
		PLATE No:	4
		REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

Photograph 9 – View west within the 31 Cowper Street property, with Park Lane in the background. The approximate position of Borehole BH4 is indicated as shown.

Photograph 10 – View west along Park Lane. The approximate positions of Boreholes BH4 to BH7 are indicated as shown.

Douglas Partners
Geotechnics / Environment / Groundwater

Site Photographs

31 Cowper Street & 2A-2D Wentworth Park Road

Glebe

CLIENT:	New South Wales Land and Housing Corporation
PROJECT:	99554.00
PLATE No:	5
REV:	0
DATE:	12/02/2020

Photograph 11 – View south within the rear of the 2D Wentworth Park Road property, with Park Lane in the background. The approximate position of Borehole BH5 is indicated as shown.

Photograph 12 – View north within the rear of the 2B Wentworth Park Road property. The approximate position of Borehole BH6 is indicated as shown.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs** **31 Cowper Street & 2A-2D Wentworth Park Road** **Glebe**	PROJECT:	99554.00
		PLATE No:	6
		REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

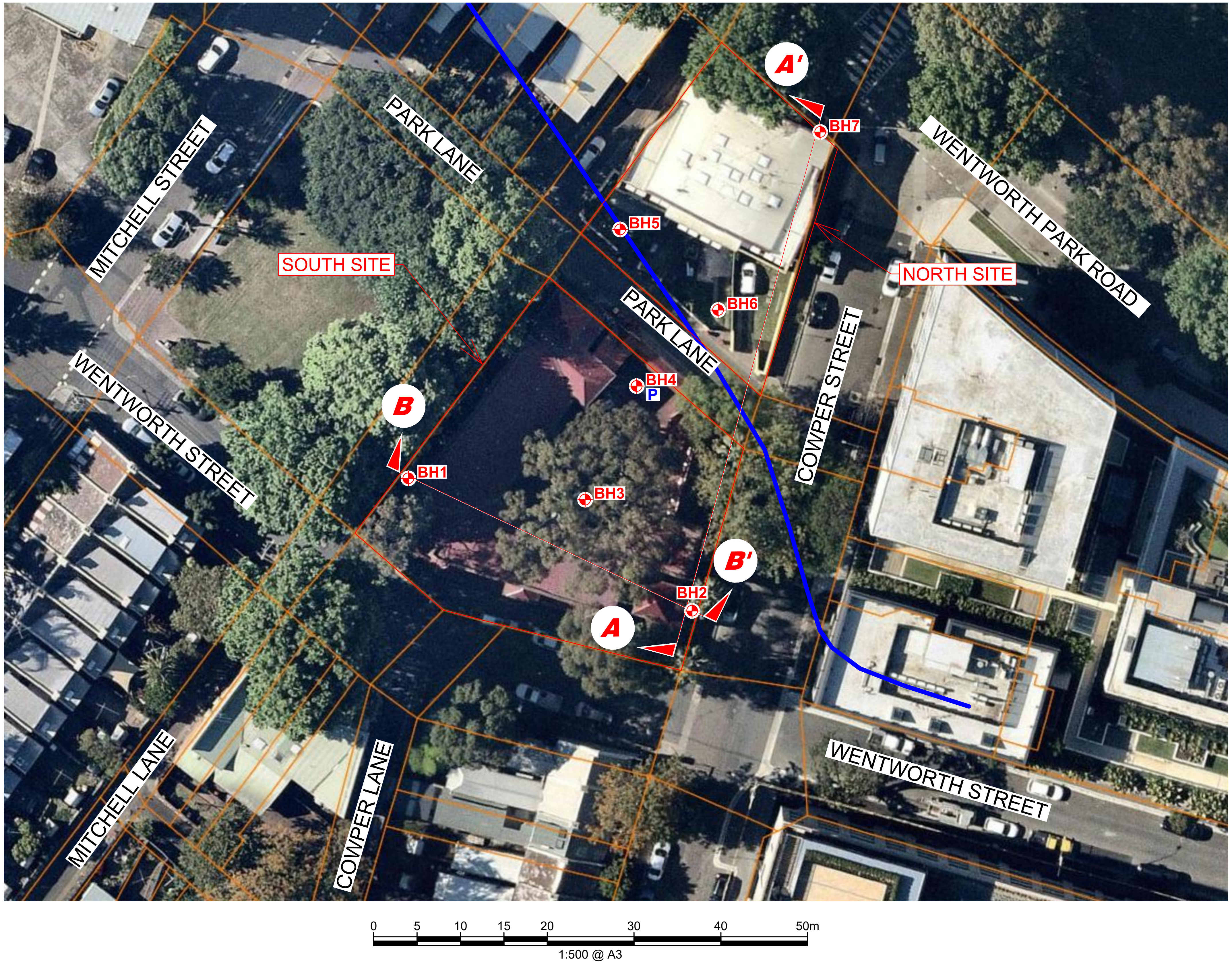
Photograph 13 – View south across Wentworth Park Road towards the position of Borehole BH7, which is indicated as shown.

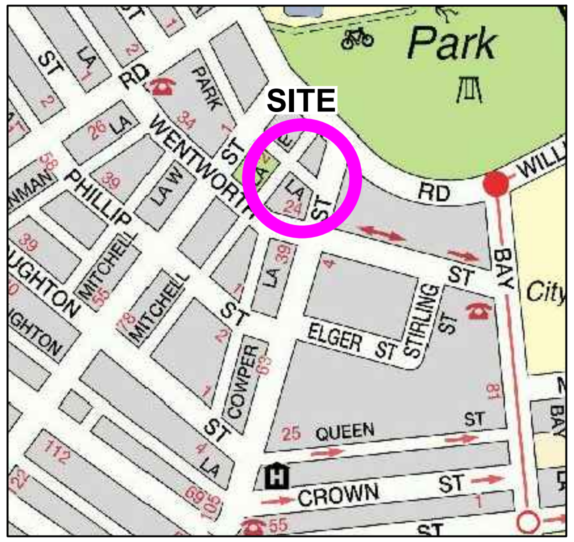
Photograph 14 – View of drilling spoil obtained from Borehole BH7.

Douglas Partners *Geotechnics / Environment / Groundwater*	**Site Photographs** **31 Cowper Street & 2A-2D Wentworth Park Road** **Glebe**	PROJECT:	99554.00
		PLATE No:	7
		REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

Appendix C

Drawings





Locality Plan

LEGEND

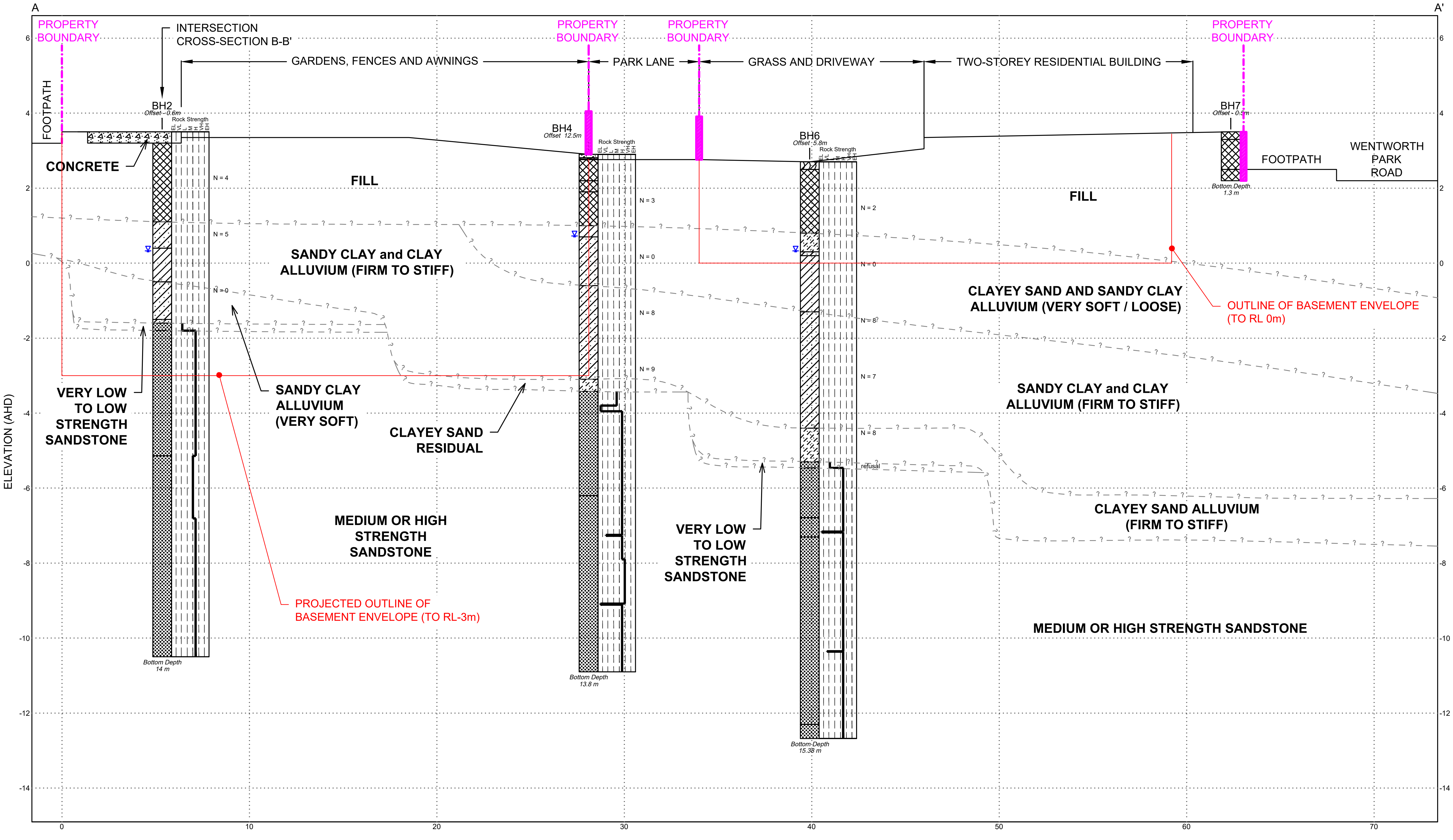
- ⌖ Borehole Location
- **P** Standpipe Piezometer
- A — A' Geotechnical Cross Section A-A'
- ▬ Historical High Water Mark and Inferred Southern Limit of Reclaimed Land (1890 Map from Parish of Petersham)

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

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: New South Wales Land and Housing Corporation		TITLE: **Site and Test Location Plan**	PROJECT No: 99554.00
OFFICE: Sydney	DRAWN BY: IT/HDS	**Glebe Mid-Rise Project**	DRAWING No: 1
SCALE: 1:500 @ A3	DATE: 11.05.2020	**31 Cowper St and 2A-2D Wentworth Park Rd, Glebe**	REVISION: 1

N



LEGEND

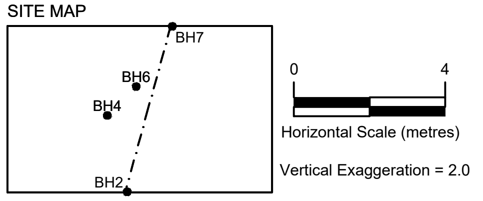
▨	Concrete	▨	Clay
▨	Fill	▨	Sandy Clay
▨	Fill / Topsoil	▨	Clayey Sand
▨	Fill / Roadbase	▨	Sandstone

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

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

TESTS / OTHER
N - Standard penetration test value
▼ - Water level
— ? — — - Inferred geotechnical boundary

SITE MAP



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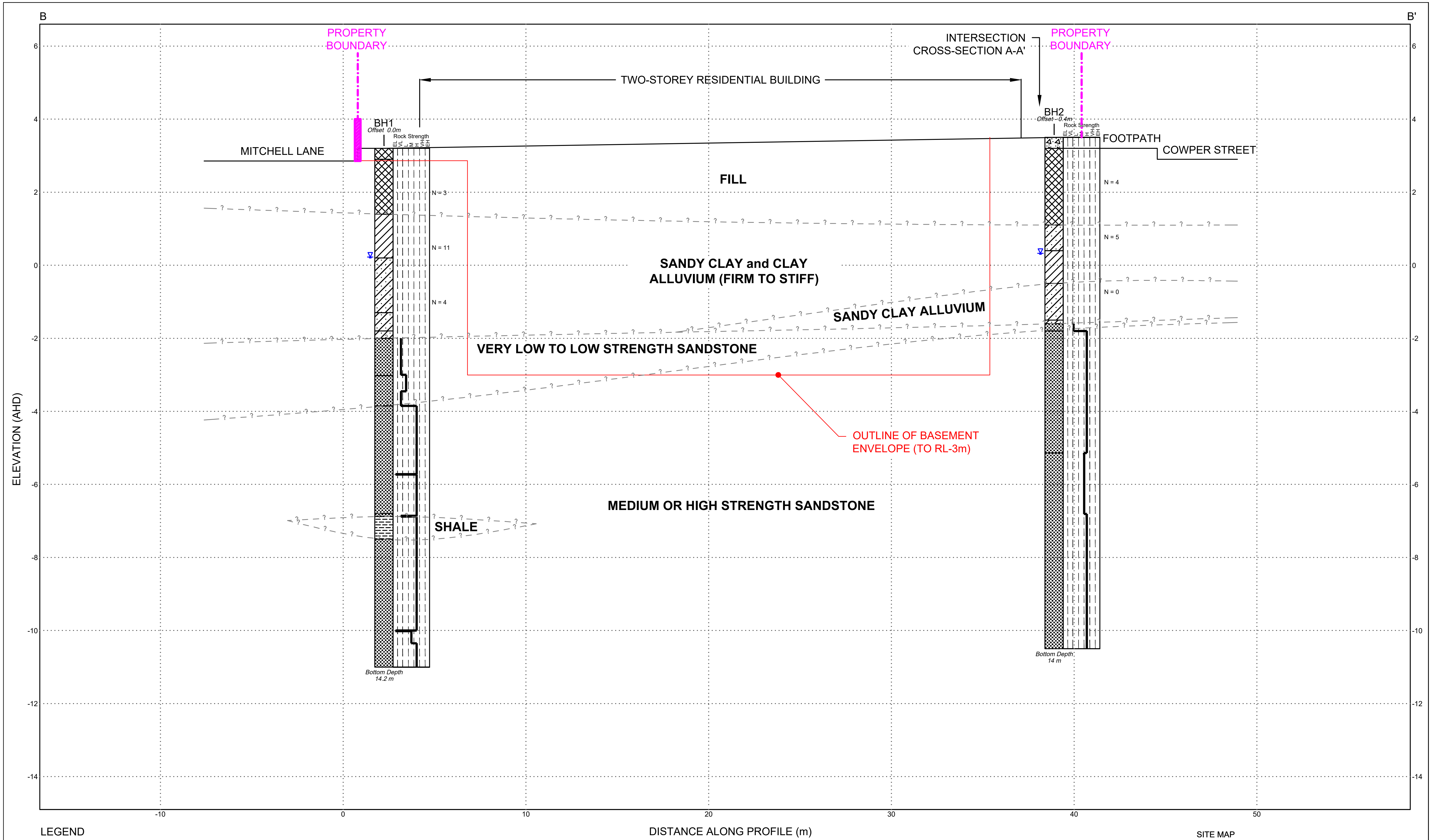
CLIENT: New South Wales Land and Housing Corporation
OFFICE: Sydney
DRAWN BY: IT/HDS
SCALE: 1:200 (H) 1:100 (V) @ A3
DATE: 11.05.2020

TITLE: **Inferred Geotechnical Cross-Section A-A'**
Glebe Mid-Rise Project
31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

PROJECT No: 99554.00
DRAWING No: 2
REVISION: 1



LEGEND

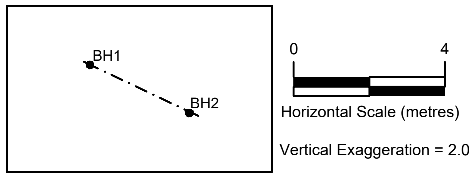
▦ Concrete
▩ Fill
▨ Clay
▨ Sandy Clay
▤ Shale
▩ Sandstone

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

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

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

SITE MAP



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CLIENT: New South Wales Land and Housing Corporation
OFFICE: Sydney
DRAWN BY: IT/HDS
SCALE: 1:200 (H) 1:100 (V) @ A3
DATE: 11.05.2020

TITLE: **Inferred Geotechnical Cross-Section B-B'**
Glebe Mid-Rise Project
31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

PROJECT No: 99554.00
DRAWING No: 3
REVISION: 1

Appendix D

Field Work Results

Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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

Description and Classification Methods

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

Soil Types

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

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

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

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

Definitions of grading terms used are:

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

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

In fine grained soils (>35% fines)

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

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

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

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

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

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

Soil Descriptions

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Origin

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

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

Moisture Condition – Coarse Grained Soils

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

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

Moisture Condition – Fine Grained Soils

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

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

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

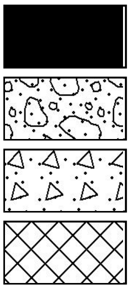
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

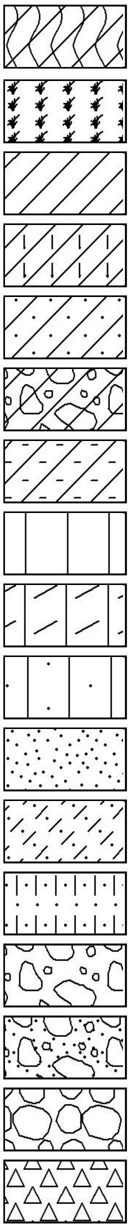
Graphic Symbols for Soil and Rock

General



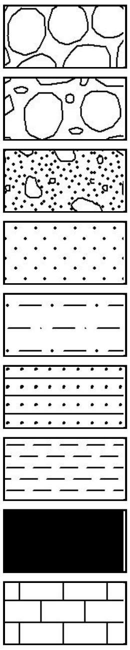
- Asphalt
- Road base
- Concrete
- Filling

Soils



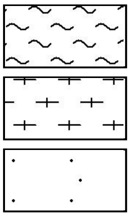
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



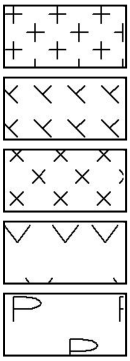
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks



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

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.2 AHD
EASTING: 332849
NORTHING: 6249728
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 99554.00
DATE: 21/01/2020
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Water	Type	Test Results & Comments
3		FILL/SAND: fine to medium, dark grey, trace silt, organic debris, gravel and charcoal fragments, moist, appears generally in a loose condition		A/E	PID=3
	0.3	FILL/GRAVEL and SAND: medium to coarse gravel, angular to subangular and rectangular, flaky, mostly ripped sandstone gravel, fine to medium sand, with glass, ceramic tile and timber, trace ash, slag and charcoal, moist, appears generally in a loose condition		A/E*	PID=2
				A/E	PID=4
2	1			S	1,1,2 N = 3 PID=2
	1.8	CLAY CH: high plasticity, mottled brown and pale grey, trace silt and ironstone gravel, w=PL, stiff, alluvial		A/E	PID=3
1	2				
				S	5,6,5 N = 11 PID=2
0	3 3.0	Sandy CLAY CL: low plasticity, pale brown, 30% fine sand, w~PL, stiff, alluvial	21-01-20	A/E	PID=1
-1	4			S	2,2,2 N = 4 PID=1
	4.5	Sandy CLAY CL: low plasticity, pale grey and brown, fine to medium, 35% clay/silt, w>PL, stiff, alluvial			
	5.0				

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
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: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC to 14.2m

WATER OBSERVATIONS: Free groundwater at 3.0m whilst augering

REMARKS: *BD2 210120 replicate of sample 0.4-0.5m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinate obtained using Nearmap and site measurements.

SAMPLING & IN SITU TESTING LEGEND

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

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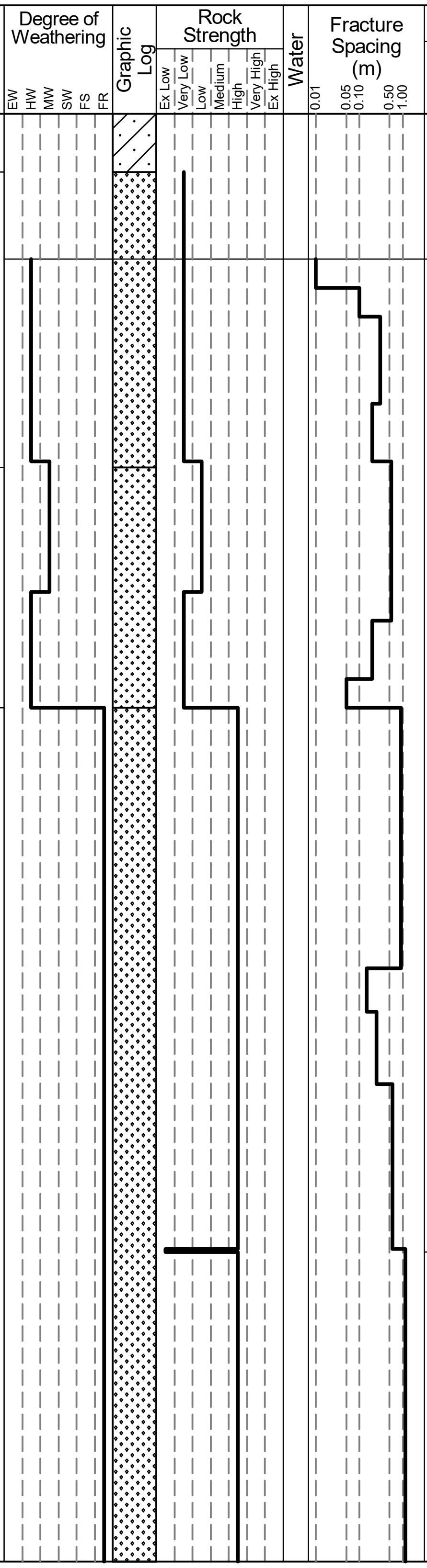
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.2 AHD
EASTING: 332849
NORTHING: 6249728
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 99554.00
DATE: 21/01/2020
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
		Sandy CLAY CL: refer previous page					
-2	5.2	SANDSTONE: fine to medium grained, brown, very low strength, highly weathered, Hawkesbury Sandstone					
			5.5 to 5.6m: fg, fe	C	100	50	
			5.7m: J45°, pl, ro, fe				PL(A) = 0.1
	6		6.0 & 6.2m: J(x2) 70°, pl, ro, cln				
-3	6.22	SANDSTONE: medium grained, pale grey and brown, low to medium strength with very low strength bands, slightly then highly weathered, slightly fractured, Hawkesbury Sandstone					
							PL(A) = 0.3
			6.7m: Ds, 200mm	C	100	68	
			6.95m: J50°, pl, ro, cly				
	7 7.05	SANDSTONE: medium to coarse grained, pale grey, thinly bedded and cross bedded (5° to 25°), high strength, fresh, slightly fractured to unbroken, Hawkesbury Sandstone	7.05m: J45°, pl, ro, cly				
-4							PL(A) = 1.4
	8		7.95m: B0°, pln, ro, cly co, 2mm				
			8.1 & 8.35m: B(x2) 10°, pln, ro, cly co, 2mm	C	100	100	
-5							PL(A) = 1.7
	9		8.92m: B0°, cly 5mm 8.93 to 9.3m: J70° to 90°, cu, he, cly 5mm				
-6		Below 9.3m, unbroken					
				C	100	52	PL(A) = 1.2
	10.0						

RIG: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 5.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC to 14.2m
WATER OBSERVATIONS: Free groundwater at 3.0m whilst augering
REMARKS: *BD2 210120 replicate of sample 0.4-0.5m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinate obtained using Nearmap and site measurements.

SAMPLING & IN SITU TESTING LEGEND

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

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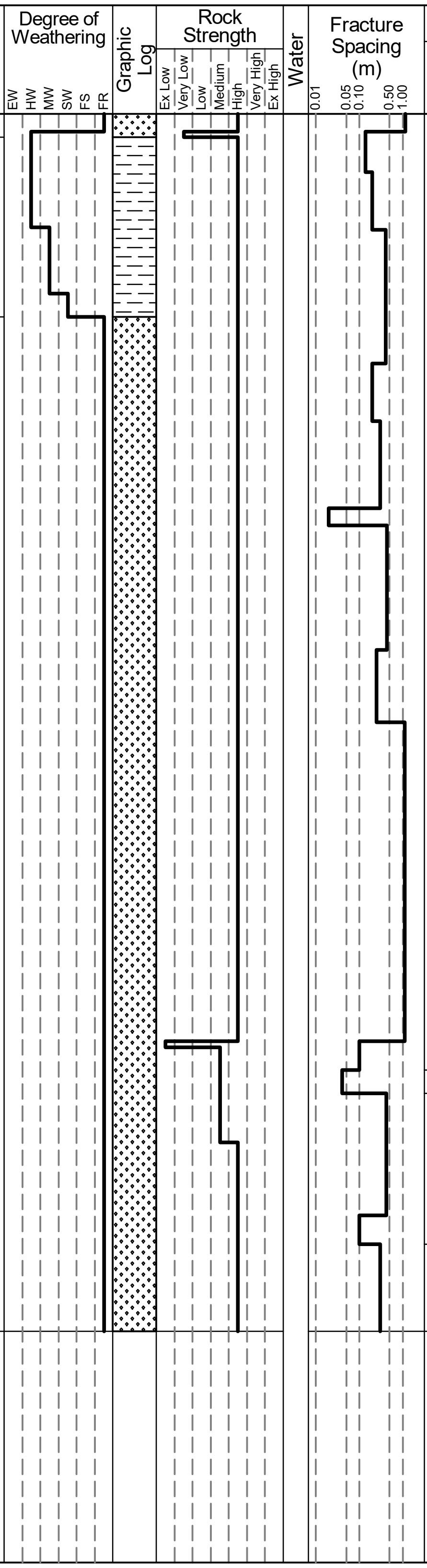
BORHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.2 AHD
EASTING: 332849
NORTHING: 6249728
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 99554.00
DATE: 21/01/2020
SHEET 3 OF 3



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

RL	Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE: refer previous page					
-7	10.08	CARBONACEOUS SHALE: dark grey, with irregular clayey zones, very low strength, highly to slightly weathered, highly fractured, Hawkesbury Shale	10.06m: J35°, un, ro, cly 10mm 10.2m: J45°, pl, sm, cly 10.4m: J45°, pl, sm, cln	C	100	52	PL(A) = 0.2
-8 -9	10.7 11 12	SANDSTONE: medium grained, pale grey, thinly bedded and cross bedded (10°-20°) with carbonaceous flakes and laminations between 10.7-12.4m, high strength, fresh, slightly fractured to unbroken, Hawkesbury Sandstone	11.06m: B10°, cly co, 2mm 11.36m: J45°, pl, ro, cly 5mm 12.1m: J45°, pl, ro, cly 3mm	C	100	96	PL(A) = 2.6 PL(A) = 1.1
-10	13 14		13.2m: J35°, pl, ro, cly 15mm 13.3m: J30°, pl, ro, cly 10mm 13.38m: B5°, cly 2mm 13.8m: B20°, pl, ro, cly vn 13.9m: J45°, pl, ro, cln	C	100	94	PL(A) = 0.7 PL(A) = 1.6
-11	14.2	Bore discontinued at 14.2m - Target depth reached					

RIG: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC to 14.2m

WATER OBSERVATIONS: Free groundwater at 3.0m whilst augering

REMARKS: *BD2 210120 replicate of sample 0.4-0.5m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinate obtained using Nearmap and site measurements.

SAMPLING & IN SITU TESTING LEGEND

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

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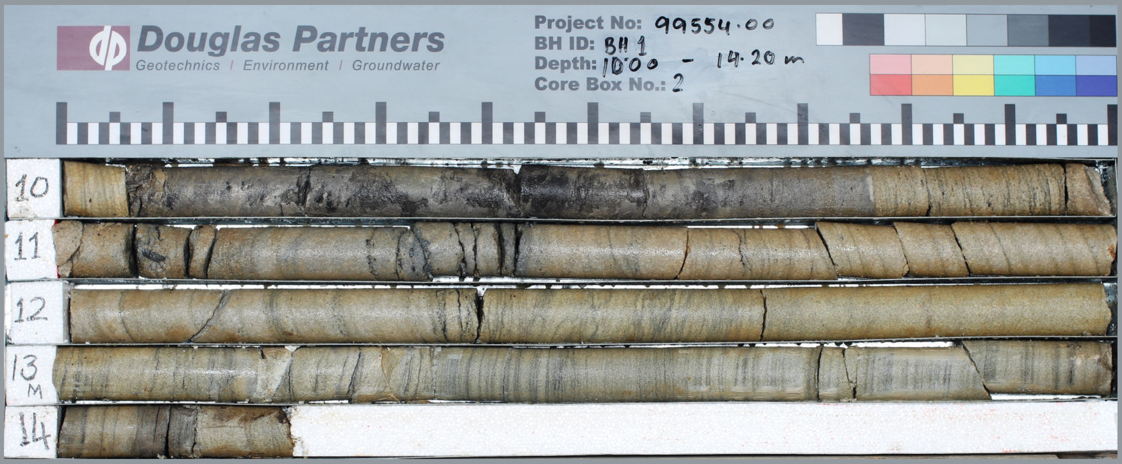
BORE: 1 PROJECT: GLEBE JANUARY 2020

5.50 – 10.00m

BORE: 1 PROJECT: GLEBE JANUARY 2020



10.00 – 14.20m

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.5 AHD
EASTING: 332882
NORTHING: 6249712
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 99554.00
DATE: 22/01/2020
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Water	Type	Test Results & Comments
		CONCRETE			
	0.3	FILL/Sandy CLAY: low plasticity, dark grey, fine to medium, with fine to coarse, angular to subangular igneous gravel and clay, trace ash, slag, glass, brick, rootlets, ripped sandstone gravel and wire, w~PL, appears generally in a loose condition			
3				A/E	PID<1
	1			A/E	PID<1
				S	1,2,2 N = 4 PID<1
2					
	2			A/E	PID<1
	2.4	Sandy CLAY CL: low plasticity, brown, fine to medium, w~PL, firm, alluvial		A/E	PID<1
1				S	4,2,3 N = 5 PID<1
	3				
	3.1	CLAY CH: high plasticity, dark grey, trace fine to medium ironstone gravel and fine sand, w>PL, firm, alluvial	▼ 22-01-20		
0				A/E	PID<1
	4 / 4.0	Sandy CLAY CL-CI: low to medium plasticity, grey-brown, fine to medium sand, w>PL, very soft, alluvial		A/E	PID<1
				S	1,0,0 N = 0 PID=2
-1					
	5.0			A/E	PID=3

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
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: XC100 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.5m

TYPE OF BORING: Diatube (250mm) to 0.3m, Solid flight auger (TC-bit) to 5.2m, NMLC coring to 14.0m

WATER OBSERVATIONS: Free groundwater observed at 3.2m whilst augering

REMARKS: Surface level obtained from Mepstead and Associates Pty Ltd, drawing 5743 dated 18/12/2018. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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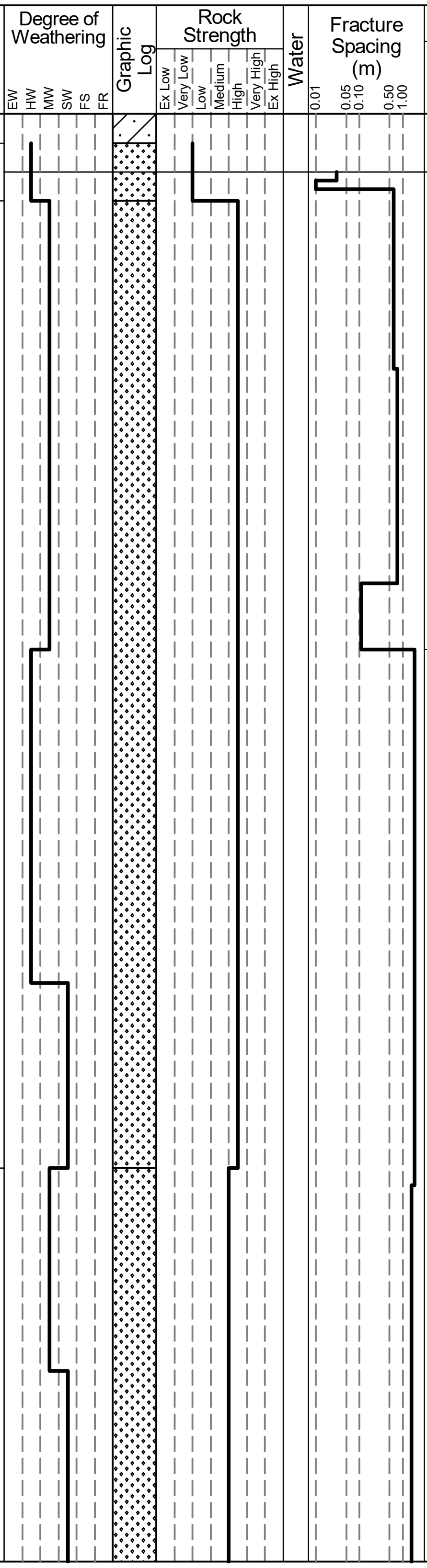
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.5 AHD
EASTING: 332882
NORTHING: 6249712
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 99554.00
DATE: 22/01/2020
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
		Sandy CLAY CL-CI: as per previous page					
	5.1	SANDSTONE: fine to medium grained, pale grey, very low strength, highly weathered, Hawkesbury Sandstone	5.23m: Cs, 30mm				
	5.3	SANDSTONE: medium to coarse grained, orange-brown and pale grey, thinly bedded and cross bedded, high strength, moderately weathered, slightly fractured, Hawkesbury Sandstone					PL(A) = 1.4
			5.88m: B5°, un, ro, fe stn	C	100	96	
							PL(A) = 2
			6.62m: B5°, pl, ro, fe stn				
			6.75m: B5°, pl, ro, cbs vn				
		Below 6.85m, red-brown, highly weathered then slightly weathered, unbroken	6.85m: B0°, pl, sm, cly co				
				C	100	100	PL(A) = 2.3
							PL(A) = 2.2
	8.64	SANDSTONE: medium to coarse grained, pale grey and orange-brown, thinly bedded and cross bedded, with carbonaceous flakes and laminations, medium to high then high strength, moderately weathered then slightly weathered, unbroken, Hawkesbury Sandstone	8.7m: B5°, un, ro, fe	C	100	100	
							PL(A) = 0.9
				C	100	100	

RIG: XC100 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.5m

TYPE OF BORING: Diatube (250mm) to 0.3m, Solid flight auger (TC-bit) to 5.2m, NMLC coring to 14.0m

WATER OBSERVATIONS: Free groundwater observed at 3.2m whilst augering

REMARKS: Surface level obtained from Mepstead and Associates Pty Ltd, drawing 5743 dated 18/12/2018. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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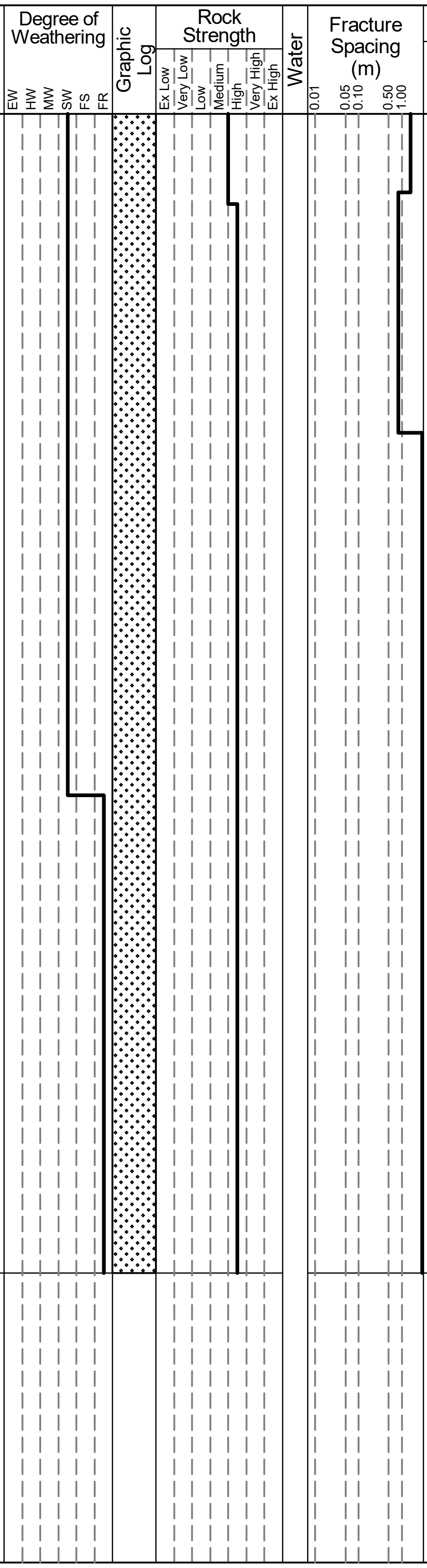
BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.5 AHD
EASTING: 332882
NORTHING: 6249712
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 99554.00
DATE: 22/01/2020
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
-7		SANDSTONE: medium to coarse grained, pale grey and orange-brown, thinly bedded and cross bedded, with carbonaceous flakes and laminations, medium to high then high strength, moderately weathered then slightly weathered, unbroken, Hawkesbury Sandstone *(continued)*	10.27m: B0-5°, pl, sm, cly co	C	100	100	PL(A) = 2.1
	11		11.1m: B0°, pl, sm, cly co				
-8		Below 11.5m, with occasional thin bands of fine gravel		C	100	100	PL(A) = 2.2
	12	Below 12.35m, fresh					PL(A) = 2.4
-9	13			C	100	100	PL(A) = 1.4
-10							
	14 14.0	Bore discontinued at 14.0m - Target depth reached					
-11							

RIG: XC100 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.5m

TYPE OF BORING: Diatube (250mm) to 0.3m, Solid flight auger (TC-bit) to 5.2m, NMLC coring to 14.0m

WATER OBSERVATIONS: Free groundwater observed at 3.2m whilst augering

REMARKS: Surface level obtained from Mepstead and Associates Pty Ltd, drawing 5743 dated 18/12/2018. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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BORE: 2 PROJECT: GLEBE JANUARY 2020

5.20 – 9.00m

BORE: 2 PROJECT: GLEBE JANUARY 2020

9.00 – 14.00m

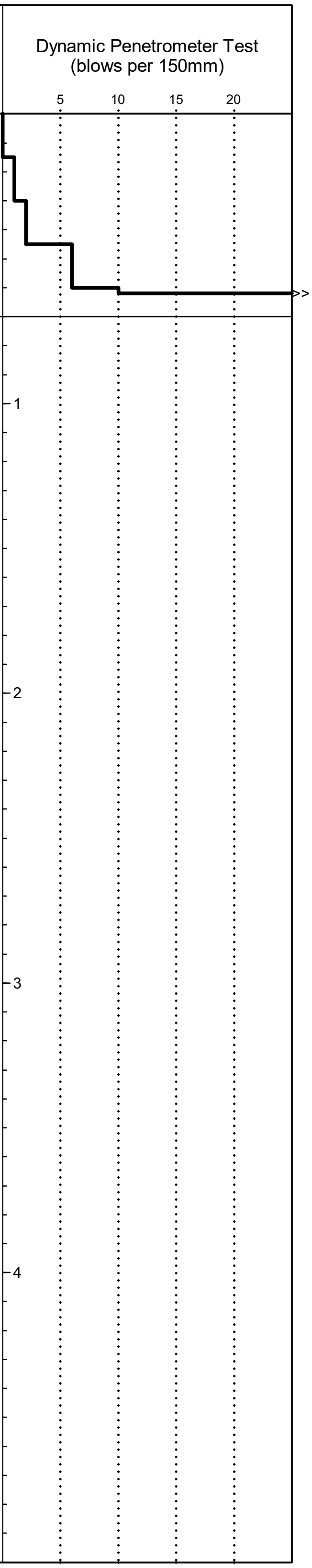
BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.5 AHD
EASTING: 332870
NORTHING: 6249725
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 99554.00
DATE: 20/01/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILL/Silty SAND: find and medium, dark brown, trace gravel and fine roots, dry to moist, appears generally in a loose to very loose condition	▨	A/E	0.0 0.1		PID<1	
	0.32	FILL/SAND: fine and medium, pale brown, trace gravel and silt, moist, appears generally in a loose condition	▨	A/E*	0.4 0.5		PID<1	
	0.65		▨					
	0.7	FILL/Gravelly SAND: fine to coarse sand, pale brown, dark brown and orange, medium to coarse gravel, trace brick fragments and plastic, moist, appears generally in a dense condition						
		Bore discontinued at 0.7m - Refusal on inferred tree root within fill						



RIG: Hand Tools **DRILLER:** HDS **LOGGED:** HDS **CASING:** Uncased

TYPE OF BORING: Hand Auger to 0.7m, within garden bed.

WATER OBSERVATIONS: No free groundwater observed

REMARKS: *BDA 200120 replicate of sample 0.4-0.5m. Surface level obtained from Mepstead and Associates Pty Ltd, drawing 5743 dated 18/12/2018. Co-ordinates from Nearmap & site measurements

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

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.9 AHD
EASTING: 332875.4
NORTHING: 6249737.4
DIP/AZIMUTH: 90°/--

BORE No: BH4
PROJECT No: 99554.00
DATE: 20/01/2020
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Water	Sampling Type	Test Results & Comments
	0.09 0.12 0.14	CONCRETE: grey, fine and medium igneous aggregate, trace voids, 8mm diameter steel reinforcement at 0.05m		A/E	PID=3
		FILL/ROADBASE: Gravelly SAND, fine to medium, angular to subangular gravel, fine to coarse sand, moist to wet, appears well compacted		A/E	PID=1
	0.7	CONCRETE: grey			
2		FILL/SAND: fine to coarse, dark grey, with ash, trace gravel, slag, ceramic tiles, glass, timber, moist, appears generally in a loose condition		A/E*	PID=4
	1.0	FILL/Clayey SAND: fine to medium, dark grey and brown, trace gravel, ash, slag and glass, moist, appears generally in a loose condition		S	1,1,2 N = 3
		FILL/SAND: fine to coarse, dark grey, with fine to coarse angular to sub-angular gravel and glass, trace silt, ash, slag, ceramic tile, and clay nodules, moist, appears generally in a loose condition			
1	1.9	CLAY CL: low plasticity, brown, trace sand and silt, w<PL, firm, alluvial (possible fill)		A/E	PID=1
	2.2	Sandy CLAY CL: low plasticity, pale grey to grey, fine to medium, w=PL, very soft, alluvial	20-01-20	A/E	
0			29-01-20	S	0,0,0 N = 0 PID=2
	3.5	Sandy CLAY CI: medium plasticity, mottled brown and pale grey, 30% fine sand, 5% ironstone gravel, firm to stiff, w~PL, alluvial			
-1				S	2,3,5 N = 8 PID=2
-2					

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

RIG: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 6.32m

TYPE OF BORING: Diatube (250mm) to 0.09m, Solid flight auger (TC-bit) to 6.32m, NMLC Coring 6.32 to 13.8m

WATER OBSERVATIONS: Free groundwater at 2.2m whilst augering

REMARKS: *BD1 200120 replicate of sample 0.9-1.0m. Standpipe installed: Solid PVC to 0.1-1.5m, screened PVC 1.5-6.5m with end cap, backfill 0.2-0.5m, Bentonite 0.5-1.0m, gravel 1.0-6.5m, bentonite 6.5-7.0m, gravel backfill 7.0-13.8m, gatic cover at the surface.

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)

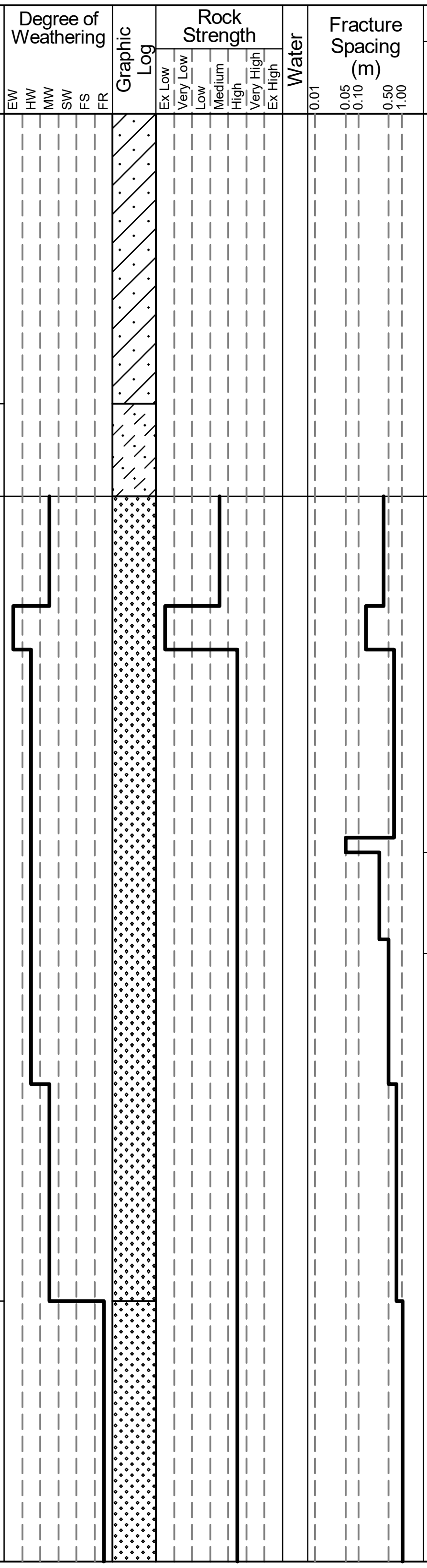
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.9 AHD
EASTING: 332875.4
NORTHING: 6249737.4
DIP/AZIMUTH: 90°/--

BORE No: BH4
PROJECT No: 99554.00
DATE: 20/01/2020
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
		Sandy CLAY CI: medium plasticity, mottled brown and pale grey, 30% fine sand, 5% ironstone gravel, firm to stiff, w~PL, alluvial *(continued)*		S			4,4,5 N = 9 PID=2
-3	6 6.0	Clayey SAND SC: fine to medium, pale grey-brown, 30% clay, apparently dense, wet, residual (possibly extremely weathered sandstone)					
	6.32	SANDSTONE: medium grained, pale grey, orange-brown and yellow-brown, thinly bedded and cross bedded with zones of iron cementation, medium then high strength, moderately then highly weathered, fractured to slightly fractured, Hawkesbury Sandstone	6.7m: Cs, 150mm	C	100	85	PL(A) = 0.9
-4	7		7.5m: B20°, fe 7.55m: B25°, pl, ro, fe				PL(A) = 2
-5	8		7.85-7.88m: fg 7.9m: B25°, fe, he	C	100	100	
		Below 8.35m, moderately weathered, unbroken	8.35m: B10°, fe				PL(A) = 1.9
-6	9						
	9.1	SANDSTONE: medium to coarse grained, pale grey, thinly bedded and cross bedded, trace siltstone flakes, high strength, fresh, unbroken, Hawkesbury Sandstone	9.1m: B10°, fe	C	100	98	PL(A) = 1.6
-7							

RIG: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 6.32m

TYPE OF BORING: Diatube (250mm) to 0.09m, Solid flight auger (TC-bit) to 6.32m, NMLC Coring 6.32 to 13.8m

WATER OBSERVATIONS: Free groundwater at 2.2m whilst augering

REMARKS: *BD1 200120 replicate of sample 0.9-1.0m. Standpipe installed: Solid PVC to 0.1-1.5m, screened PVC 1.5-6.5m with end cap, backfill 0.2-0.5m, Bentonite 0.5-1.0m, gravel 1.0-6.5m, bentonite 6.5-7.0m, gravel backfill 7.0-13.8m, gatic cover at the surface.

SAMPLING & IN SITU TESTING LEGEND

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

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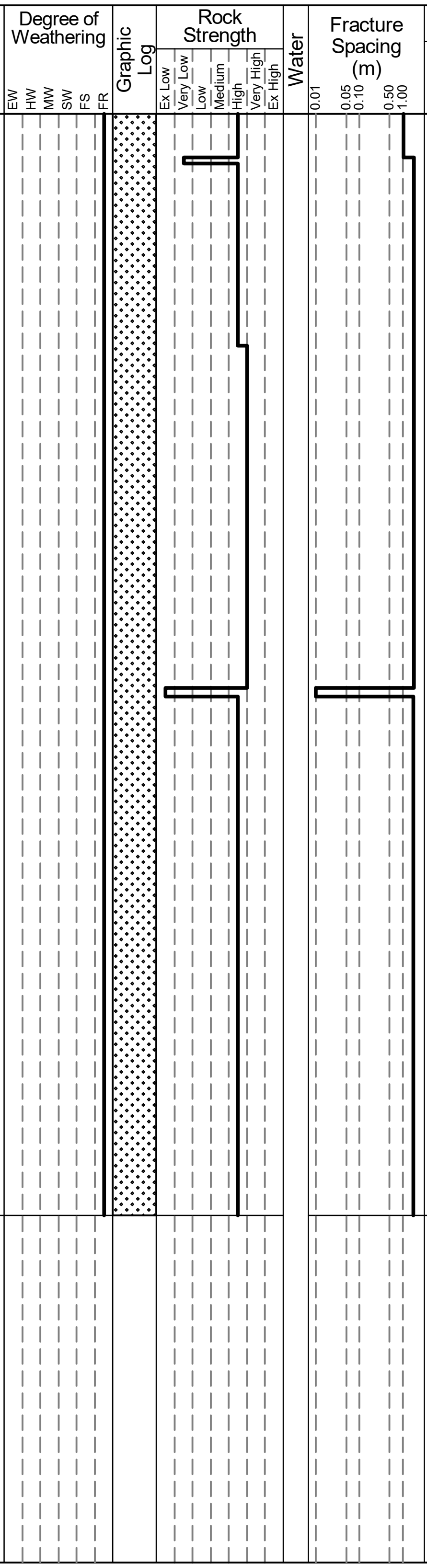
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.9 AHD
EASTING: 332875.4
NORTHING: 6249737.4
DIP/AZIMUTH: 90°/--

BORE No: BH4
PROJECT No: 99554.00
DATE: 20/01/2020
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
		SANDSTONE: medium to coarse grained, pale grey, thinly bedded and cross bedded, trace siltstone flakes, high strength, fresh, unbroken, Hawkesbury Sandstone *(continued)*	10.15m: B0°, cly, fg 10mm	C	100	98	PL(A) = 1.8
-8	11	Between 10.8-11.98m, high to very high strength		C	100	98	PL(A) = 3.1
-9	12		11.98m: Cs, 30mm				PL(A) = 1.9
-10	13			C	100	100	PL(A) = 1.6
	13.8	Bore discontinued at 13.8m - Target depth reached. Standpipe piezometer installed	13.75m: B5°, cly co 2mm				
-11	14						
-12							

RIG: XC100 **DRILLER:** FF **LOGGED:** SI **CASING:** HW to 6.32m

TYPE OF BORING: Diatube (250mm) to 0.09m, Solid flight auger (TC-bit) to 6.32m, NMLC Coring 6.32 to 13.8m

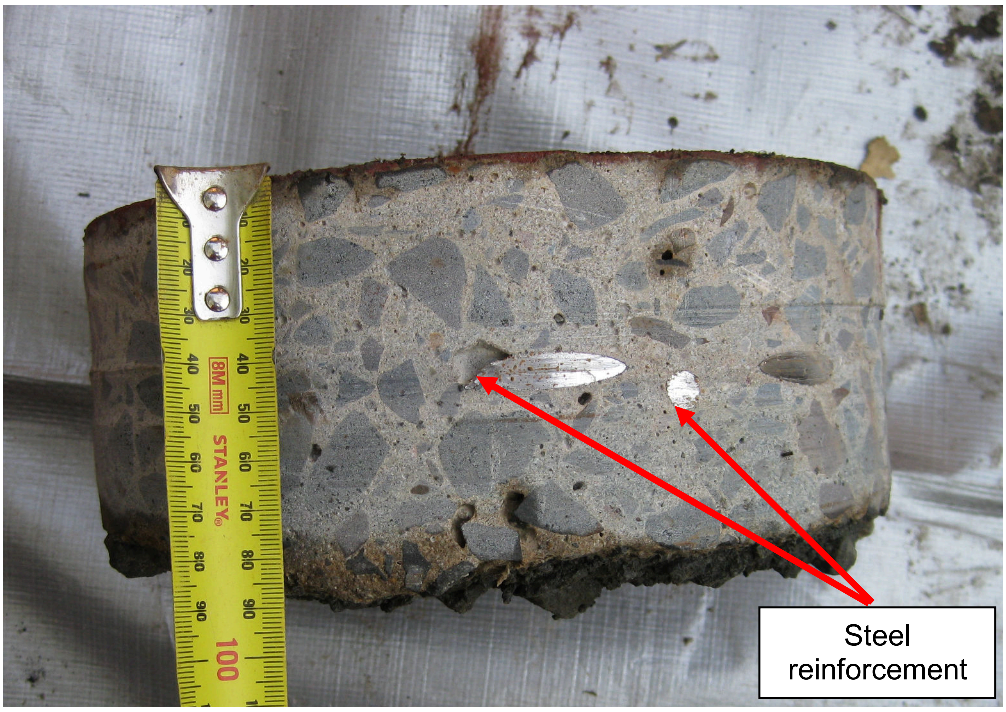
WATER OBSERVATIONS: Free groundwater at 2.2m whilst augering

REMARKS: *BD1 200120 replicate of sample 0.9-1.0m. Standpipe installed: Solid PVC to 0.1-1.5m, screened PVC 1.5-6.5m with end cap, backfill 0.2-0.5m, Bentonite 0.5-1.0m, gravel 1.0-6.5m, bentonite 6.5-7.0m, gravel backfill 7.0-13.8m, gatic cover at the surface.

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)

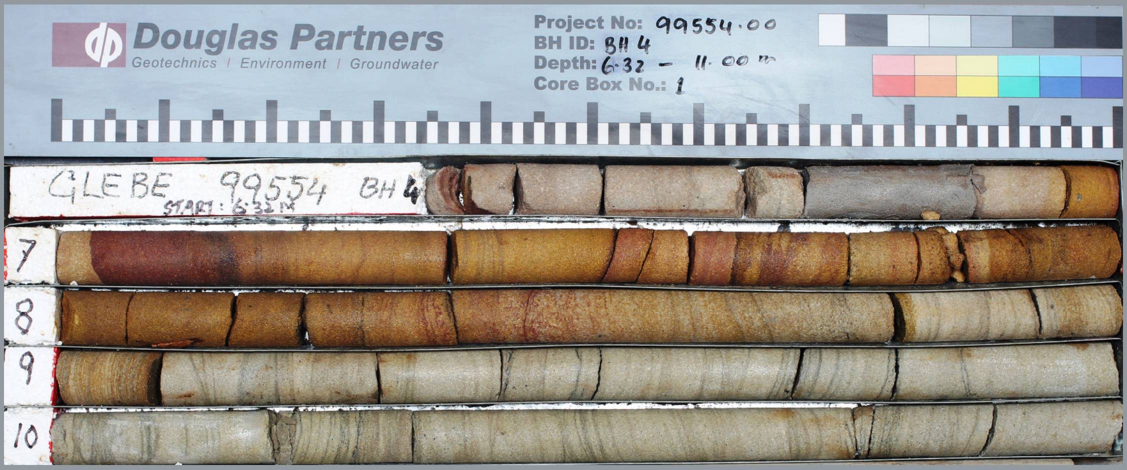
Douglas Partners
Geotechnics / Environment / Groundwater



Photograph D1 – View of concrete core from surface of Borehole BH4, with steel reinforcement indicated as shown.

Douglas Partners *Geotechnics / Environment / Groundwater*	**Detailed Photographs** **31 Cowper Street & 2A-2D Wentworth Park Road** **Glebe**	PROJECT:	99554.00
		PLATE No:	D1
		REV:	0
	CLIENT: New South Wales Land and Housing Corporation	DATE:	12/02/2020

BORE: 4 PROJECT: GLEBE JANUARY 2020



6.32 – 11.00m

BORE: 4 PROJECT: GLEBE JANUARY 2020

11.00 – 13.80m

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332874
NORTHING: 6249756
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 99554.00
DATE: 23/01/2020
SHEET 1 OF 4

Depth (m)	Description of Strata	Water	Sampling Type	Test Results & Comments
	FILL/TOPSOIL: SAND, fine to medium, brown, with silt, trace ash, slag, gravel and rootlets, dry, appears generally in a loose condition		A/E	PID<1
0.2	FILL/Sandy CLAY: low plasticity, dark grey, fine to medium sand, with fine to coarse, angular to subangular igneous gravel, trace ash, slag, glass, brick, rootlets, ripped sandstone gravel, w~PL, appears generally in a loose condition		A/E	PID<1
			A/E	PID<1
			S	2,2,2 N = 4 PID<1
1.8	Clayey SAND SC: fine to coarse, orange-brown, moist, medium dense, alluvial		A/E*	PID<1
2.4	Sandy CLAY CL-CI: low to medium plasticity, dark grey and red-brown, fine to medium, w>PL, firm to stiff, alluvial	23-01-20	A/E	PID<1
			S	1,4,4 N = 8 PID<1
3.0	CLAY CH: high plasticity, dark grey and red-brown, trace fine sand, w>PL, firm to stiff, alluvial		A/E	PID<1
			A/E	PID<1
			A/E	PID<1
			S	2,4,4 N = 8 PID<1
4.6	Sandy CLAY CL: medium plasticity, pale grey and red-brown, fine to medium, w>PL, firm to stiff, alluvial		A/E	PID<1
5.0				

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.7m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC coring to 15.3m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: *BD3 230120 replicate of sample 1.9-2.0m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

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)

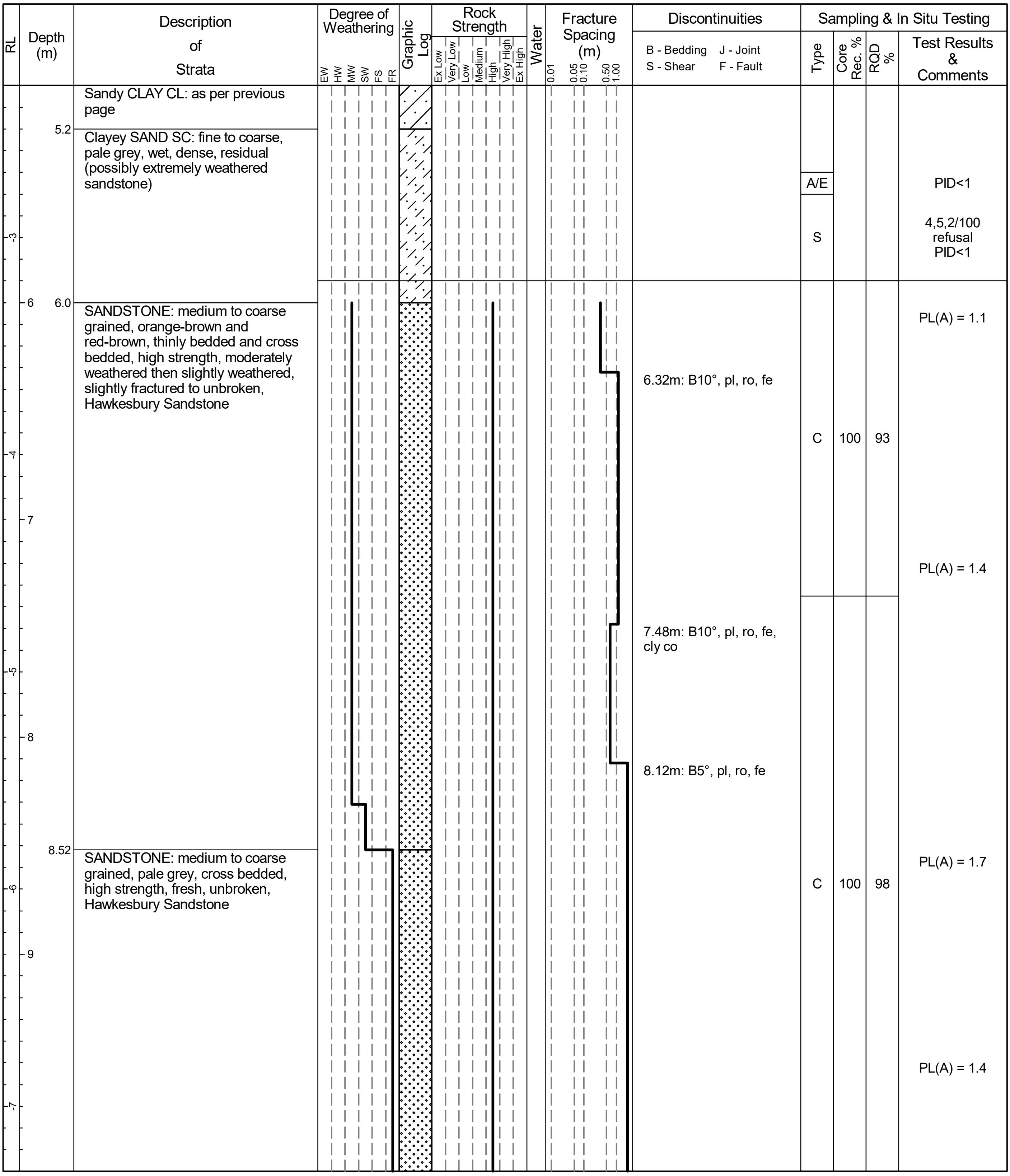
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332874
NORTHING: 6249756
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 99554.00
DATE: 23/01/2020
SHEET 2 OF 4



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	Sandy CLAY CL: as per previous page					
5.2	Clayey SAND SC: fine to coarse, pale grey, wet, dense, residual (possibly extremely weathered sandstone)		A/E			PID<1
			S			4,5,2/100 refusal PID<1
6.0	SANDSTONE: medium to coarse grained, orange-brown and red-brown, thinly bedded and cross bedded, high strength, moderately weathered then slightly weathered, slightly fractured to unbroken, Hawkesbury Sandstone	6.32m: B10°, pl, ro, fe 7.48m: B10°, pl, ro, fe, cly co 8.12m: B5°, pl, ro, fe	C	100	93	PL(A) = 1.1 PL(A) = 1.4
8.52	SANDSTONE: medium to coarse grained, pale grey, cross bedded, high strength, fresh, unbroken, Hawkesbury Sandstone		C	100	98	PL(A) = 1.7 PL(A) = 1.4

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.7m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC coring to 15.3m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: *BD3 230120 replicate of sample 1.9-2.0m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

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

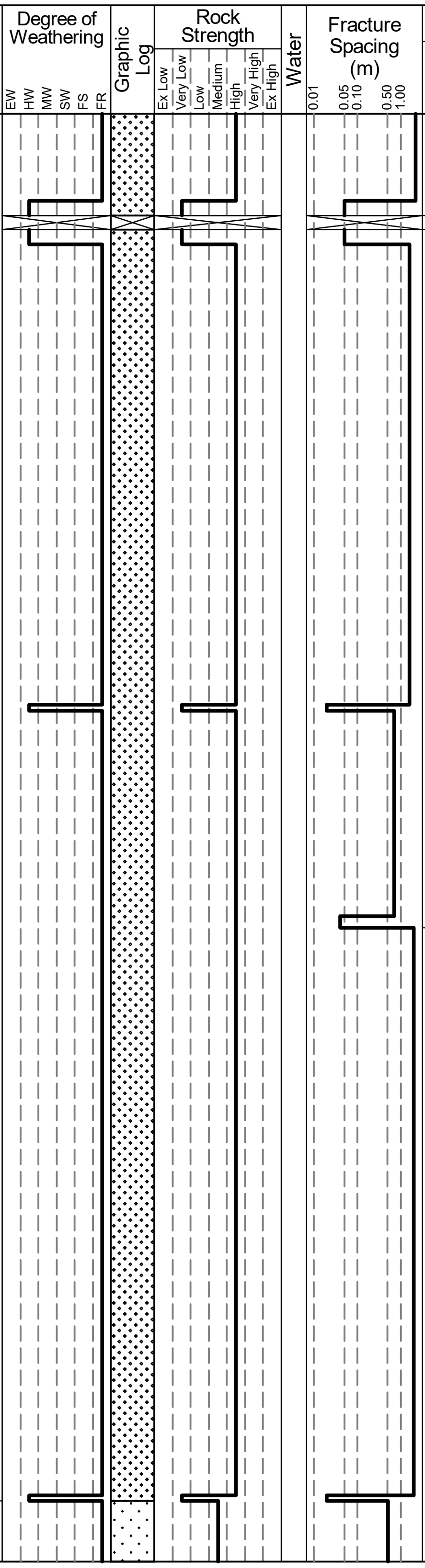
BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332874
NORTHING: 6249756
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 99554.00
DATE: 23/01/2020
SHEET 3 OF 4

RL	Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE: medium to coarse grained, pale grey, cross bedded, high strength, fresh, unbroken, Hawkesbury Sandstone *(continued)*		C	100	98	
	10.4		10.3m: fg, 50mm 10.35m: CORE LOSS: 50mm 10.4m: fg, 50mm				
-8							PL(A) = 1.9
	11						
-9				C	98	95	PL(A) = 1.7
	12		12.04m: fg, 20mm, cly vn				
-10			12.77m: B0°, pl, sm, cly co 12.81m: B0°, pl, ro, cly vn				
	13						PL(A) = 1.7
-11							PL(A) = 1.5
	14			C	100	99	PL(A) = 1
-12							
	14.79	SANDSTONE: refer following page	14.77m: fg, 20mm, cly co				
	15.0						



RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.7m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC coring to 15.3m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: *BD3 230120 replicate of sample 1.9-2.0m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332874
NORTHING: 6249756
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 99554.00
DATE: 23/01/2020
SHEET 4 OF 4

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
		SANDSTONE: fine to medium grained, grey, massive, medium strength, fresh, unbroken, Hawkesbury Sandstone	FR	⁘	Medium		1.00		C	100	99	PL(A) = 0.9
	15.3	Bore discontinued at 15.3m - Target depth reached										
-13												
	16											
-14												
	17											
-15												
	18											
-16												
	19											
-17												

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 5.7m

TYPE OF BORING: Solid flight auger (TC-bit) to 5.5m, NMLC coring to 15.3m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: *BD3 230120 replicate of sample 1.9-2.0m. Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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BORE: 5 PROJECT: GLEBE JANUARY 2020

5.9 – 10.0m

BORE: 5 PROJECT: GLEBE JANUARY 2020

10.0 – 15.0m

BORE: 5 PROJECT: GLEBE JANUARY 2020

15.0 – 15.3m

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332885
NORTHING: 6249747
DIP/AZIMUTH: 90°/--

BORE No: BH6
PROJECT No: 99554.00
DATE: 24/01/2020
SHEET 1 OF 4

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
		FILL/TOPSOIL: SAND, fine to medium, dark brown, trace silt, gravel, brick, clay nodules, charcoal, ash, slag and rootlets, moist, appears generally in a loose condition							A/E			PID<1
	0.2	FILL/Sandy CLAY: low plasticity, dark grey, fine to medium, with clay and fine to coarse, angular to subangular igneous gravel, trace ash, slag, glass, brick, rootlets, ripped sandstone gravel, w~PL, appears generally in a loose condition							A/E			PID<1
2									A/E			PID<1
	1								S			2,1,1 N = 2 PID<1
1												
	1.9	Clayey SAND SC: fine to medium, orange-brown, moist, loose, alluvial							A/E			PID<1
	2											
	2.4	Sandy CLAY CL-CI: low to medium plasticity, grey, fine to medium sand, w>PL, very soft, alluvial				▼ 24-01-20			A/E			PID<1
	2.5	CLAY CI-CH: medium to high plasticity, grey and red-brown, trace fine to medium sand, w>PL, very soft, alluvial							S			0,0,0 N = 0 PID<1
0												
	3								A/E			PID<1
									A/E			PID<1
-1												
									A/E			PID<1
	4 4.0	Sandy CLAY CL-CI: low to medium plasticity, red-brown and pale grey, fine to coarse, w>PL, firm, alluvial							S			2,3,5 N = 8 PID<1
-2												
									A/E			PID<1

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 8.4m

TYPE OF BORING: Solid flight auger (TC-bit) to 8.1m; NMLC coring to 15.38m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

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)

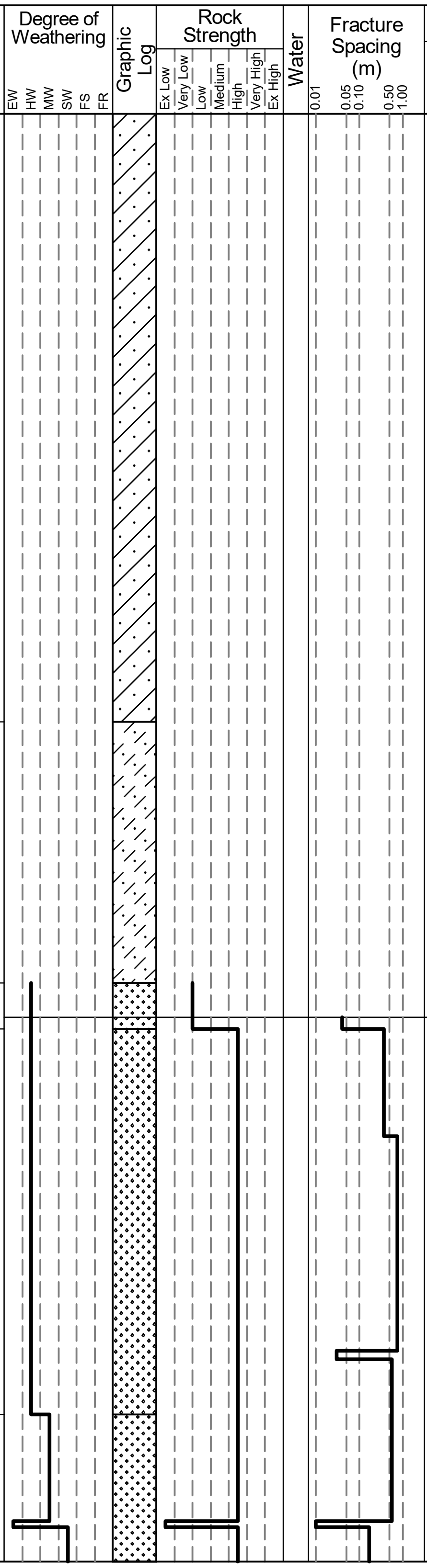
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332885
NORTHING: 6249747
DIP/AZIMUTH: 90°/--

BORE No: BH6
PROJECT No: 99554.00
DATE: 24/01/2020
SHEET 2 OF 4



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		Sandy CLAY CL-CI: low to medium plasticity, red-brown and pale grey, fine to coarse, w>PL, firm, alluvial *(continued)*					
				A/E			PID<1
-3				S			3,4,3 N = 7 PID<1
	6						
				A/E			PID<1
-4							
	7						
	7.1	Clayey SAND SC: fine to coarse, red-brown, wet, loose, alluvial		S			2,4,4 N = 8 PID<1
-5							
	8 8.0	SANDSTONE: fine to medium grained, red-brown, very low to low strength, highly weathered, Hawkesbury Sandstone		S			10/20,B refusal PID<1
	8.16	SANDSTONE: medium to coarse grained, red-brown and orange-brown, thinly bedded and cross bedded, high strength, highly weathered, slightly fractured, Hawkesbury Sandstone	8.16m: B5°, pl, ro, cly vn, fe stn				PL(A) = 1.1
			8.53m: B0°, pl, sm, cly co				
-6							
	9			C	100	96	
			9.27-9.3m: J45°, pl, ro, cly vn				
	9.49	SANDSTONE: medium to coarse grained, pale grey, thinly bedded and cross bedded, with carbonaceous flakes and laminations, high strength, moderately weathered to fresh, slightly fractured					PL(A) = 1.4
-7			9.86m: Cs, 20mm				
	10.0						

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 8.4m

TYPE OF BORING: Solid flight auger (TC-bit) to 8.1m; NMLC coring to 15.38m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

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)

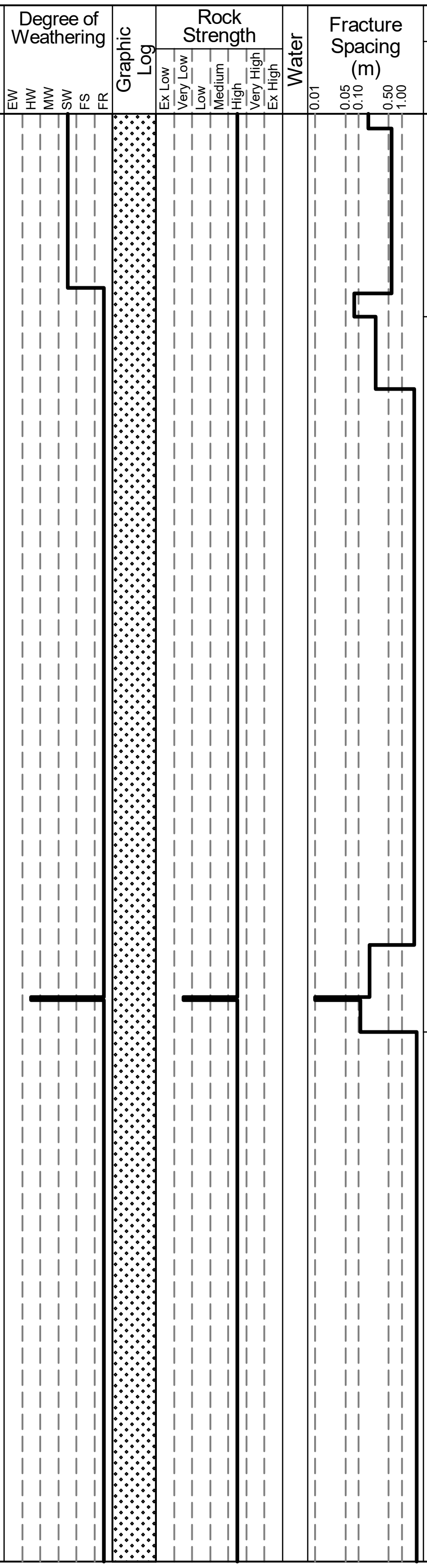
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332885
NORTHING: 6249747
DIP/AZIMUTH: 90°/--

BORE No: BH6
PROJECT No: 99554.00
DATE: 24/01/2020
SHEET 3 OF 4



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE: medium to coarse grained, pale grey, thinly bedded and cross bedded, with carbonaceous flakes and laminations, high strength, moderately weathered to fresh, slightly fractured, Hawkesbury Sandstone	10.05m: B0°, pl, ro, cly vn	C	100	96	
							PL(A) = 1.1
			10.62-10.7m: J60°, pl, ro, cly vn				
-8							
			10.7m: B5°, un, sm, cbs co				
	11	Below 10.95m, unbroken	10.95m: B0-10°, un, sm, cly co				
							PL(A) = 2
-9							
				C	100	97	
	12						
							PL(A) = 2.5
-10							
			12.87m: B0°, pl, sm, cly co				
	13						
			13.05m: fg, 10mm, cly co				
			13.17m: B0°, pl, sm, cly co				
							PL(A) = 1.2
-11							
	14						
				C	100	100	
							PL(A) = 1.6
-12							
	15.0						

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 8.4m

TYPE OF BORING: Solid flight auger (TC-bit) to 8.1m; NMLC coring to 15.38m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

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: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 2.7 AHD
EASTING: 332885
NORTHING: 6249747
DIP/AZIMUTH: 90°/--

BORE No: BH6
PROJECT No: 99554.00
DATE: 24/01/2020
SHEET 4 OF 4

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
		SANDSTONE: refer previous page							C	100	100	PL(A) = 1.3
	15.38	Bore discontinued at 15.38m - Target depth reached										
-13												
	16											
-14												
	17											
-15												
	18											
-16												
	19											
-17												

RIG: Comacchio Geo 205 **DRILLER:** Terratest **LOGGED:** IT **CASING:** HW to 8.4m

TYPE OF BORING: Solid flight auger (TC-bit) to 8.1m; NMLC coring to 15.38m

WATER OBSERVATIONS: Free groundwater observed at 2.4m whilst augering

REMARKS: Surface level obtained from Veris Australia Pty Ltd, drawing number 201704 dated 15/08/2019. Co-ordinates obtained using Nearmap & site measurements

SAMPLING & IN SITU TESTING LEGEND

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

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BORE: 6 PROJECT: GLEBE JANUARY 2020

8.12 – 12.00m

BORE: 6 PROJECT: GLEBE JANUARY 2020

12.00 – 15.38m

BOREHOLE LOG

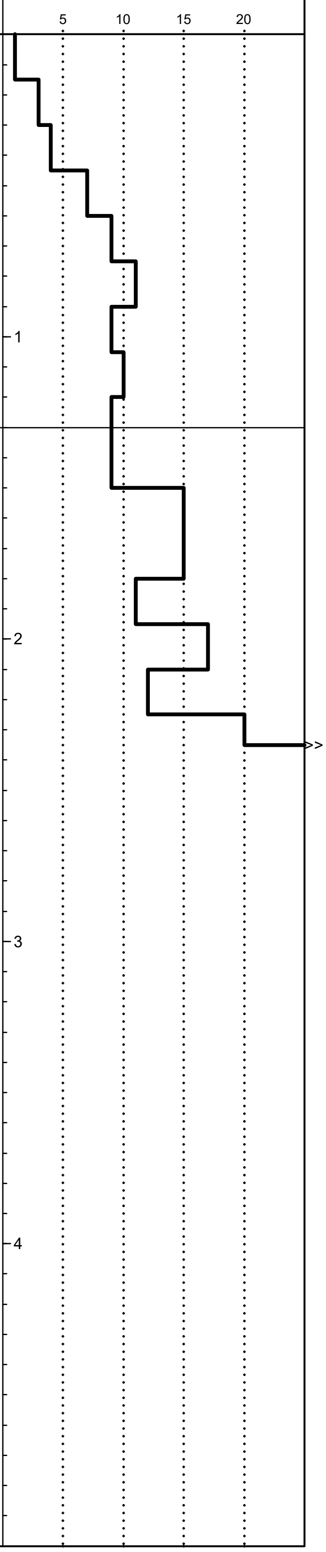
CLIENT: New South Wales Land and Housing Corporation
PROJECT: Glebe Mid-Rise Project
LOCATION: 31 Cowper St and 2A-2D Wentworth Park Rd, Glebe

SURFACE LEVEL: 3.5 AHD
EASTING: 332897
NORTHING: 6249767
DIP/AZIMUTH: 90°/--

BORE No: BH7
PROJECT No: 99554.00
DATE: 20/01/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILL/SAND: fine and medium, dark brown, trace silt, brick fragments and gravel, moist to wet, appears generally in a loose to medium dense condition	☒	A/E	0.0 0.2		PID<1	
	0.2	FILL/Gravelly SAND: fine and medium, dark brown and brown, fine and medium gravel (brick, sandstone), trace ash, plastic, charcoal, glass and tile, moist, appears generally in a medium dense condition At 0.54 m, layer of white fabric and green glass	☒	A/E	0.2 0.3		PID<1	
3				A/E	0.5 0.6		PID<1	
				A/E	0.8 0.9		PID<1	
	1.0	FILL/Sandy CLAY: low plasticity, pale brown to brown, fine and medium, trace rusted metal objects, silt, ash and charcoal, w<PL, appears generally in a stiff condition	☒	A/E	1.0 1.1		PID<1	
				A/E	1.2 1.3		PID<1	
	1.3	Bore discontinued at 1.3m - Refusal in fill on coarse gravel						
2	1							
1	2							
0	3							
-1	4							

Dynamic Penetrometer Test
(blows per 150mm)



RIG: Hand Tools **DRILLER:** HDS **LOGGED:** HDS **CASING:** Uncased

TYPE OF BORING: Hand Auger to 1.3m

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Within garden box, 0.65 m above street level and 0.52m back from the inside face of the brick retaining wall.

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

Results of Dynamic Penetrometer Tests

Client Department of Land and Housing Corporation
Project Glebe Mid-Rise Project
Location 31 Cowper Street and 2A to 2D Wentworth Park Road, Glebe
Project No. 99554.00
Date 24/01/2020
Page No. 1 of 1

Test Locations	BH3	BH7				
RL of Test (AHD)	3.5	3.5				
Depth (m)	**Penetration Resistance** Blows/150 mm					
0.00 – 0.15	0	1				
0.15 – 0.30	1	3				
0.30 – 0.45	2	4				
0.45 – 0.60	6	7				
0.60 – 0.75	10/20	9				
0.75 – 0.90	Ref	11				
0.90 – 1.05		9				
1.05 – 1.20		10				
1.20 – 1.35		9				
1.35 – 1.50		9				
1.50 – 1.65		15				
1.65 – 1.80		15				
1.80 – 1.95		11				
1.95 – 2.10		17				
2.10 – 2.25		12				
2.25 – 2.40		20/100				
2.40 – 2.55		Ref				

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

Tested By: HS/SI
Checked By: HS

Remarks Ref = Refusal

Appendix E

Laboratory Test Results

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

CERTIFICATE OF ANALYSIS 235035-A

Client Details

Client	Douglas Partners Pty Ltd
Attention	Huw Smith
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**99554.01, Glebe**
Number of Samples	Additional Testing on 3 Soils
Date samples received	22/01/2020
Date completed instructions received	23/01/2020

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	31/01/2020
Date of Issue	31/01/2020

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Soil Aggressivity				
Our Reference		235035-A-2	235035-A-5	235035-A-7
Your Reference	UNITS	1/0.9-1	4/2.5-2.6	7/1.2-1.3
Date Sampled		21/01/2020	21/01/2020	21/01/2020
Type of sample		Soil	Soil	Soil
pH 1:5 soil:water	pH Units	8.2	7.1	8.4
Electrical Conductivity 1:5 soil:water	µS/cm	130	170	100
Chloride, Cl 1:5 soil:water	mg/kg	10	29	20
Sulphate, SO4 1:5 soil:water	mg/kg	26	200	10

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.

Client Reference: 99554.01, Glebe

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	103	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	106	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	93	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	101	[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.

Envirolab Ref: 235035A

Due: 31/1/20

Douglas Partners
Geotechnics / Environment / Groundwater

CHAIN OF CUSTODY DESPATCH SHEET

Project No: 99554.00
Project Name: Mixed-Use Development
Project Manager: HDS
Emails: huw.smith@douglaspartners.com.au
Date Required: Same day ☐ 24 hours ☐ 48 hours ☐ 72 hours ☐ Standard ☐
Prior Storage: ☐ Esky x Fridge ☐ Shelved

Suburb: GLEBE
Order Number
Sampler: NLE/SI

To: Envirolab Services
12 Ashley Street, Chatswood 2067
Attn: Aileen Hie
Phone: 02 9910 6200
Email: Ahie@envirolab.com.au

Do samples contain 'potential' HBM? Yes ☐ No ☐ (If YES, then handle, transport and store in accordance with FPM HAZID)

Sample ID	Lab ID	Date Sampled	Sample Type	Container Type	Analytes								Notes/preservation
			S - soil W - water	G - glass P - plastic	Aggressivity (pH, EC, SO4, Chloride)								
1/0.9-1.0	2	21/01/20	S	G, P	X								Soil Samples already at Laboratory under Envirolab job number 235035
4/2.5-2.6	5	21/01/20	S	G	X								
7/1.2-1.3	7	21/01/20	S	G	X								
													Please analyse and report separately to contamination testing
PQL (S) mg/kg													**ANZECC PQLs req'd for all water analytes** ☐

PQL = practical quantitation limit. If none given, default to Laboratory Method Detection Limit

Lab Report/Reference No:

Metals to Analyse: Nil

Total number of samples in container:

Relinquished by: NLE

Transported to laboratory by: Already at laboratory

Send Results to: Douglas Partners Pty Ltd

Address 96 Hermitage Road, West Ryde

Phone: 02 9809 0666 **Fax:**

Signed:

Received by: AH

Date & Time: 23/1/20

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

CERTIFICATE OF ANALYSIS 235396-A

Client Details

Client	Douglas Partners Pty Ltd
Attention	Huw Smith
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**99554.01, Glebe**
Number of Samples	23 Soil
Date samples received	28/01/2020
Date completed instructions received	28/01/2020

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	04/02/2020
Date of Issue	03/02/2020

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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

Soil Aggressivity		
Our Reference	UNITS	235396-A-1
Your Reference		1
Depth		2.5-2.95
Date Sampled		21/01/2020
Type of sample		Soil
pH 1:5 soil:water	pH Units	6.2
Electrical Conductivity 1:5 soil:water	µS/cm	260
Chloride, Cl 1:5 soil:water	mg/kg	140
Sulphate, SO4 1:5 soil:water	mg/kg	280

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.
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Client Reference: 99554.01, Glebe

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Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	96	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	102	[NT]

Result Definitions

NT	Not tested
NA	Test not required
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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.

Douglas Partners
Geotechnics / Environment / Groundwater

CHAIN OF CUSTODY DESPATCH SHEET

Project No:	99554.00	**Suburb:**	GLEBE	**To:**	Envirolab Services
Project Name:	Mixed-Use Development	**Order Number**			12 Ashley Street, Chatswood 2067
Project Manager:	HDS	**Sampler:**	NLE/SI	**Attn:**	Aileen Hie
Emails:	huw.smith@douglaspartners.com.au			**Phone:**	02 9910 6200
Date Required:	Same day ☐ 24 hours ☐ 48 hours ☐ 72 hours ☐ Standard ☐			**Email:**	Ahie@envirolab.com.au

Prior Storage: ☐ Esky x Freezer ☐ Shelved Do samples contain 'potential' HBM? Yes ☐ No ☐ (If YES, then handle, transport and store in accordance with FPM HAZID)

Sample ID	Lab ID	Date Sampled	Sample Type S - soil W - water	Container Type G - glass P - plastic	Analytes Aggressivity (pH, EC, SO4, Chloride)								Notes/preservation
1/2.5-2.95	①	21/01/20	S	G, P	X								Please analyse and report separately to contamination testing
PQL (S) mg/kg													

ANZECC PQLs req'd for all water analytes ☐

667

Envirolab Services
12 Ashley St
Chatswood NSW 2067
Ph: (02) 9910 6200
Job No: 235396-A
Date Received: 28·01·2020
Time Received: 12:28
Received by: TC.
Temp: [✓ Cool]/Ambient
Cooling: Ice/[✓ Icepack]
Security: [✓ Intact]/Broken/None

PQL = practical quantitation limit. If none given, default to Laboratory Method Detection Limit

Metals to Analyse: Nil

Lab Report/Reference No: 235396-A.

Total number of samples in container:

Relinquished by: NLE

Transported to laboratory by:

Send Results to: Douglas Partners Pty Ltd

Address: 96 Hermitage Road, West Ryde

Phone: 02 9809 0666

Fax:

Signed:

Received by: ELS, Trudead Gmallet.

Date & Time: 28.1.20