Douglas Partners
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Report on
Geotechnical Assessment

Proposed 132/33 kV Substation
21 Waterloo Road, Macquarie Park

Prepared for
Ausgrid

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

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Report on Geotechnical Assessment
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1. Introduction

This report presents the results of a geotechnical assessment undertaken for a proposed 132/33kV Substation to be constructed adjacent to an existing substation at 21 Waterloo Road, Macquarie Park. The investigation was commissioned in an email dated 28 June 2018 by Mr Paul Hurst of Ausgrid and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal SYD180586 dated 22 June 2018.

The proposed substation footprint is between Waterloo Road and an existing operational substation to the north-east.

It is understood that the proposed development will include:

- shallow excavations for a transformer yard, with 2 transformer bays;
- shallow excavations for a control room, on the northern side of the development footprint;
- excavations for a single-level cable basement below a switchroom, with the basement floor level understood to be no deeper than RL45.5 m, relative to the Australian Height Datum (m AHD);
- shallow excavations for cable trenches, to link into an existing 11 kV cable pit (south-west of the proposed substation); and
- construction of a 132/33 kV switchroom over the cable basement footprint, and construction of the control room, with the switchroom and control room proposed to have a design floor level of approximately RL49.5 m.

Site investigation (including hand augered and machine drilled boreholes) was carried out to provide detailed sub-surface information on the depth to rock within the development footprint, and to obtain soil and rock samples for logging, testing and waste classification purposes. The objective of the geotechnical assessment is to confirm the site geotechnical model.

The investigation included borehole drilling, installation of a standpipe piezometer, and laboratory testing of selected groundwater, soil and rock samples. The field work was conducted in conjunction with a Preliminary Site Investigation and Limited Stage 2 Contamination Investigation, and Preliminary Waste Classification, which are both reported separately (DP reports 86471.01.R.001.Rev0 and 86471.01.R.002.Rev0, dated August 2018).

The details of the field work and laboratory testing are presented in this report, together with comments and recommendations relating to design and construction practice.

2. Previous Investigations

Documents supplied indicate that previous geotechnical and environmental site investigations were completed at the site in 1997 and 2000 by Jeffery and Katauskas Pty Ltd (J&K), and in 2000 by Environmental Investigation Services Pty Ltd (EIS: a division of J&K) and Golder Associates Pty Ltd (Golder). It is noted that the drilling results from the 1997 geotechnical investigation were included as an appendix within the J&K report prepared for Enerserve (Report reference 15400SLrpt, dated 9 September 2000).

The previous geotechnical and environmental investigations completed for the wider substation development site are understood to have included a total of 19 boreholes by J&K (including three boreholes cored into the underlying rock: Boreholes 1, 3 and 7), 5 boreholes by EIS, and 22 test pits by Golder. The boreholes and test pits completed in close proximity to the proposed substation development footprint are shown on Drawing 1.

The 2000 J&K report Section 3.1 indicates that previous cut and fill earthworks at the site resulted in a 'benched' profile, with temporary batters ranging up to 4 m high and grades of around 1(H):1(V). Excavations for a cable trench, a pit with sloping side batters, and exposed sandstone which were noted on several of the J&K test location plans (e.g. Figure 7) were not observed during the current field work. It is assumed that the "3m deep pit" shown on the Golder site plan was for the cable pit excavation, which is located adjacent to the southern property boundary along Waterloo Road.

It is understood that two trenches have recently been excavated by Ausgrid (to about 3 m depth), using non-destructive digging (NDD) methods, to expose the buried high voltage cables. The locations of both trenches are shown on Drawing 1. It is assumed that these trenches were terminated within the filling, with logs for these trenches not available at the time of reporting. It is noted that logs for five boreholes completed by EIS were not included in their report (i.e. Boreholes BH A to BH E), and that the log for test pit TP17 was omitted from the Golder report.

The Golder test pits (excavated with a trailer-mounted backhoe) generally reached refusal to the backhoe bucket at or up to 0.3 m below the top of sandstone. The 2000 J&K report Section 3.4 notes that the Golder logs were for "environmental investigation purposes and therefore they do not show the detail that would be expected with geotechnical logging", that the Golder test pit logs "did not have any surface reduced levels", and that the Golder test location plan is "not to scale and therefore the locations of these test pits can only be approximated".

3. Site Description and Geology

The site, identified as Lot 1 in DP 1006960, includes an existing, active substation (Macquarie Park Substation, ZN 8000), with street address 21 Waterloo Road, Macquarie Park. Industrial and commercial properties are located adjacent to the site. A review of aerial imagery indicates that the site was developed into a substation around 2003.

The proposed substation footprint is within a grass-covered area, on the eastern side of an existing concrete driveway and locked access gate and south-west of the existing substation control room and live transformer yards (refer Plate 1 in Appendix C). It is understood that this area was filled to the current surface levels using site-won materials, after completion of the previous geotechnical reports (i.e. after the year 2000), with these filling materials placed over existing filling materials, which are inferred to also have been sourced from the site.

The proposed development area has plan dimensions of approximately 25 m (parallel to Waterloo Road) by 40 m. The western part of the site is relatively flat, with a gradual slope down to the south and east. To the east the ground surface slopes towards a small car parking area and a concrete driveway (refer Plate 2). The provided site survey drawing indicates that surface levels within the development footprint vary between RL49.9 m to RL47.0 m (north-west to south-east corners, respectively).

A buried stormwater service transects the proposed footprint, with a buried fire hydrant pipe adjacent to the north-western corner of the development footprint.

Reference to the Sydney 1:100 000 Geological Series Sheet (Geological Survey of NSW: Reference 1) indicates that the site is underlain by Hawkesbury Sandstone of Triassic age, which typically comprises medium to coarse grained quartz sandstone, very minor shale and laminite lenses.

A review of the NSW Acid Sulfate Soil Risk Map for Prospect/Parramatta River (Reference 2) indicates that the site is not located within an area of known acid sulfate soil risk.

4. Field Work Methods

The field work was undertaken over two days between 10 and 11 July 2018, and included:

- scanning for buried services using a scanning sub-contractor;
- excavation using non-destructive digging (NDD) techniques of two 'L'-shaped test trenches adjacent to proposed machine-drilled borehole locations;
- reinstatement of the NDD trenches using certified clean sand backfilling, compacted in layers using hand compaction equipment (i.e. 'wacker packer');
- excavation of five hand augered boreholes HA1 to HA5, to depths of 0.4 – 0.6 m;
- drilling of two boreholes (BH01 and BH02) using a track-mounted drilling rig, to depths of 7.33 m and 7.0 m (respectively);
- installation of a standpipe piezometer within Borehole BH01, screened within the sandstone (refer to borehole log for well construction details);
- groundwater observations during NDD excavations and drilling;
- sampling of soils during drilling for geotechnical, contamination and waste classification purposes;
- measurement of the groundwater level and developing of the groundwater well on 17 July 2018; and
- measurement of the groundwater level and sampling of the groundwater well on 24 July 2018.

The two machine-drilled boreholes were advanced within the soils and the upper 0.3 m to 0.7 m of the underlying extremely low to very low strength sandstone using auger drilling methods, with both boreholes deepened into the underlying extremely low and higher strength rock using NMLC diamond core drilling techniques. The boreholes drilled using hand tools encountered shallow refusal on coarse inclusions within the filling, such as gravel, cobbles and boulders (0.4 m to 0.6 m total drilled depths).

Logging of the soil and rock materials within the boreholes was undertaken in general accordance with Australian Standard AS 1726 - 2017 (Reference 3), with observations made during NDD excavation utilised to supplement observations made during borehole drilling.

All field work was carried out under the full-time supervision of a geotechnical engineer. Co-ordinates and surface levels of the test locations were obtained using a high-precision differential GPS, and checked using measurements relative to surveyed site features. The inferred accuracy of the co-ordinates is 0.1 m in both plan and elevation. The positions of the boreholes, and the scaled locations of previous boreholes and test pits from the 1997 and 2000 site investigations, are shown on Drawing 1, Appendix B.

Further details of the methods and procedures employed in the investigation are presented in the attached Notes About This Report.

5. Field Work Results

The subsurface conditions encountered in the boreholes are presented on the attached borehole logs in Appendix D, along with standard notes defining the descriptive terms and the classification methods used, and core photographs from boreholes BH01 and BH02. Site photographs are presented in Appendix C. A cross-section through the development footprint is presented as Drawing 2, in Appendix B.

The subsurface conditions encountered in the boreholes can be summarised as:

FILLING:	silty clay topsoil with some rootlets (0.1 m thick), over silty clay, sandy clay and clay filling with concrete and brick fragments, and cobbles and boulders of sandstone and siltstone, to 3.5 m depth (elevation of RL46.0 m to RL46.9 m), damp to moist, generally firm to stiff (inclusive of filling placed after the completion of the previous geotechnical site investigations in the year 2000); over
RESIDUAL CLAY:	stiff, brown, slightly silty clay with trace ironstone gravel, damp, 0.3 m thick (to an elevation of RL45.7 m, possibly filling: encountered at the base of a former pit shown on Drawing 7 of the J&K report); over
SANDSTONE:	extremely low to very low strength, medium grained sandstone, becoming high strength and moderately weathered within 0.95 m below the top of rock. It is noted that intervals of extremely low and low strength sandstone (0.4 – 0.6 m thick) were encountered within both cored boreholes.

It is understood that records of earthworks compaction and/or density testing records for the filling material were not available at the time of reporting. The variability of the encountered materials suggests that the filling material across the site is "uncontrolled filling", in that it has not been placed and compacted with Level 1 earthworks inspections and testing, as defined in Australian Standard AS 3798 – 2007 "Guidelines on earthworks for commercial and residential developments" (Reference 4).

The surface levels and depths at which various materials were encountered in the current boreholes during the investigation are summarised in Table 1. Depths of various materials from selected historical site investigations (assuming surface levels for previous investigations are within about 0.3 m below the current ground surface levels) are summarised in Table 2. Details for Bore E and Test Pit TP17 are not included in Table 2, as no investigation logs were included in the various supplied reports.

Table 1: Borehole Surface levels and Summary of Subsurface Profile

Borehole	Surface RL (m AHD)	Top of Residual Clay		Top of Weathered Sandstone	
		Depth (m)	RL (AHD)	Depth (m)	RL (AHD)
BH01	49.5	3.5	46.0	3.8	45.7
BH02	49.6	ne	ne	2.7	46.9
HA1	48.5	> 0.4	< 48.1	ne	ne
HA2	47.6	> 0.55	< 47.0	ne	ne
HA3	49.7	> 0.5	< 49.2	ne	ne
HA4	48.2	> 0.6	< 47.6	ne	ne
HA5	49.6	> 0.4	< 49.2	ne	ne

Notes: "ne" indicates Not Encountered

Table 2: Summary of Historical Site Investigation Data

Historical Test Location	Historical Surface Level (m AHD)	Top of Residual Clay (m AHD)		Top of Weathered Sandstone (m AHD)		Interpolated Current Surface Level (m AHD)
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)	
TP1	48.5[3]	0.8	47.7[3]	1.4	47.1[3]	48.0
TP2	49.1[3]	1.2	47.9[3]	2.0	47.1[3]	49.3
TP3	50.0[3]	1.5	48.5[3]	2.2	47.8[3]	50.2
TP15	49.7[3]	1.5	45.9[3]	2.4	47.3[3]	49.8
TP16	47.4[3]	0.8	46.6[3]	1.4	46.0[3]	47.5
TP18	49.9[3]	2.4	47.5[3]	3.0	46.9[3]	49.9
TP19	50.0[3]	2.0	48.0[3]	2.6	47.4[3]	50.1
Bore 1	48.5	1.6	46.9	2.0	46.5	48.6
Bore 2	49.7	1.7	48.0	2.3	47.4	49.9
Bore 6	44.5	ne[1]	ne	1.5	43.0	45.0
Bore 7	46.5	0.5	46.0	0.8	45.7	45.4
Bore 201	46.3	ne[1]	ne	0.3	46.0	46.4

Historical Test Location	Historical Surface Level (m AHD)	Top of Residual Clay (m AHD)		Top of Weathered Sandstone (m AHD)		Interpolated Current Surface Level (m AHD)
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)	
Bore 206	50.0	ne[1]	ne	1.4	48.6	50.1
Bore 207	49.8	1.5	48.3	2.5	47.3	50.1
Bore 208	49.7	1.2	48.5	3.3[2]	46.4	49.9
Bore 209	49.1	ne[1]	ne	1.0	48.1	49.4

Notes: (1) "ne" indicates Not Encountered. (2) Extremely low strength bands above this depth within the residual clay. (3) Level inferred.

Groundwater was not observed within the recent boreholes during auger drilling or NDD excavation, and the use of water as a drilling fluid during coring of the rock prevented further groundwater observations. Measurement of the standpipe piezometer installed within Borehole BH01 was completed on 17 July 2018 and 24 July 2018 (i.e. between one and two weeks following installation and development of the piezometer). The water level measurements are presented in Table 3.

Table 3: Groundwater Measurements in Standpipe Piezometer

Borehole	Surface RL (m AHD)	Groundwater Measurements				Stratum Notes for Groundwater
		17 July 2018		24 July 2018		
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)	
BH01	49.5	6.0	43.5	5.0	44.5	Within sandstone. Closely spaced clay seams and rock defects between 5.3 - 5.9 m depth

Only two of the nearby historical site investigations encountered groundwater during auger drilling. The measurements within these two boreholes (Bore 2 and Bore 207) are presented in Table 4.

Table 4: Reported Groundwater Measurements from Historical Boreholes, 13 May 1997

Borehole	Surface RL (m AHD)	Measurements		Groundwater Notes from Borehole Log
		Depth (m)	RL (m AHD)	
Bore 2	49.7	2.6	47.1	Within sandstone. Level measured 3.75 hours following completion of auger drilling, inflow at 4.8 m depth during drilling
Bore 207	49.8	3.6	46.2	Within sandstone. Level measured on completion of the borehole, with inflow at 4.5 m depth during drilling

6. Laboratory Testing

6.1 Rock Core

Laboratory testing was completed on selected rock core specimens from the boreholes for rock strength (Point Load Strength Index: Is_{50}). The results, which are presented on the borehole logs, generally indicate Is_{50} values of 0.3 MPa to 2.3 MPa, indicating rock ranging from medium to high strength classification. To obtain inferred unconfined compressive strengths (UCS) from point load strength test results, a conversion factor of 15 to 20 is often used, indicating a UCS of up to about 46 MPa.

6.2 Soil Samples – Chemical Analysis

Two soil samples selected from the boreholes were submitted for analysis at a chemical laboratory. The analytes included soil aggressiveness to buried concrete and steel elements (pH, electrical conductivity, sulfate and chloride concentration). Further chemical testing was undertaken for a waste classification, which has been reported under separate cover (DP Report 86471.01.R.001.Rev0, dated August 2018), and is not discussed further herein.

The soil aggressivity results are summarised in Table 5, with the laboratory test reports included in Appendix E.

Table 5: Laboratory Test Results for Soil Aggressiveness to Buried Concrete and Steel

Sample ID	Sample Description	Elevation of Sample[1] (RL m)	pH	EC[3] (µS/cm)	Cl [3] (mg/kg)	SO_4 [3] (mg/kg)
BH01, 2.5-2.95 m	Clay (Filling)	47.0	8.9	110	10	43
HA3, 0.5 m	Silty Clay (Filling)	49.2	7.8	28	10	<10

Notes: (1) Elevation quoted is for the 'top' of the samples. (2) EC = Electrical Conductivity, Cl – Chloride, SO_4 = sulfate. (3) Each analyte was tested as a 1:5 mixture of soil:water.

In accordance with AS 2159 - 2009 (Reference 5), the results of the chemical laboratory testing indicate that the filling tested is non-aggressive to buried concrete and steel.

6.3 Soil Samples – Mechanical Analysis

One soil sample collected from Borehole BH02 was tested to determine the compaction properties and California bearing ratio (CBR – 4 day soak) at a NATA-accredited soils laboratory. The results of the testing are summarised in Table 6 below. The laboratory test reports are included in Appendix E.

Table 6: Summary of Compaction Properties, CBR, and Moisture Content

Borehole ID	Depth (m)	Material Description	FMC (%)	OMC (%)	MDD (t/m^3)	Swell (%)	Curing (hr)	CBR (%)	Over size (%)

Borehole ID	Depth (m)	Material Description	FMC (%)	OMC (%)	MDD (t/m^3)	Swell (%)	Curing (hr)	CBR (%)	Over size (%)
BH02	0.5 – 1.5	Sandy clay (Filling)	14.6	11.5	1.91	0.5	172	15	5.7

Notes: FMC = Field Moisture Content, OMC = Optimum Moisture Content, MDD = Maximum Dry Density, CBR = California Bearing Ratio, Oversize = material retained on the 19 mm sieve.

7. Proposed Development

It is understood that the proposed development on the southern part of the existing substation site is to include a transformer yard with two transformer bays, a new 132/33 kV switchroom, and an adjoining control room. The development includes shallow excavations for a single level cable basement beneath the switchroom (basement design floor level understood to be no deeper than RL45.5 m), with a cable trench to link into an existing 11 kV cable pit.

Given the site topography, it is anticipated that excavations for the cable basement will require a 4 m deep excavation to achieve the design floor level, with further deepening of the excavation to accommodate floor slabs and footings.

8. Geotechnical Model

8.1 Historical Geotechnical Model

The historical geotechnical model for the proposed switchroom and transformer yard footprint (based on the J&K report and borehole logs near the proposed development footprint) comprised three geotechnical units, given below in order of increasing depth below the ground surface:

MATERIAL A: Silty clay or sandy clay filling, with sandstone gravel, to depths of between 1 – 2.5 m;

MATERIAL B: Residual silty clay, with bands of iron cementation or extremely weathered sandstone, between 0 – 2.2 m thick; over

MATERIAL C: generally described (e.g. in boreholes 208 and 209) as red-brown and grey to brown, fine to medium grained, extremely low to very low strength sandstone.

8.2 Current Geotechnical Model

Past cut/fill construction activities were noted within a historical geotechnical report (J&K: 2000), with the original site levels having apparently been extensively modified (i.e. topsoil and residual soils almost completely removed). The origin of the filling used to level the site at that time was not included in the report, however, it is likely that site re-profiling utilised site-won material.

Historical and current investigations completed in close proximity to each other (i.e. Borehole BH02 and Test Pit TP15) indicate some variability in the encountered soil profile, however, elevations for the top of rock are similar. Elsewhere, the elevation of the top of rock as encountered in the current boreholes is similar to the inferred contours for the top of rock presented in Figure 4 of the J&K Report. These contours indicate the elevation of the top of rock reduces in a southerly direction, in a series of broad, shallow 'steps'.

Comparison of historical spot survey levels and surface levels of boreholes with current survey data supplied by Ausgrid indicates that minimal stripping of filling across the site has occurred. Further, the survey data indicates that the surface level within a former pit / excavation within the current development footprint has been raised by about 2.5 m: refer Drawing 2. The origin of the 'new' filling (i.e. post-year 2000) is likely to be from site-won material, sourced from excavations for the substation which was reportedly constructed circa 2003. Descriptions of filling materials placed prior to and after the year 2000 are similar, with the exception that the new filling appears to have an increased relative proportion of sand and demolition rubble (i.e. concrete and brick inclusions). The thickness of filling and residual soils over weathered sandstone appears to decrease towards the southern side of the development footprint.

Based on the current and historical geotechnical information for the site, the current geotechnical model for the proposed development area is considered to comprise three geotechnical units, as follows (in order of increasing depth below the ground surface: refer to Drawing 2 in Appendix B):

UNIT 1: Silty Clay, Sandy Clay and Clay filling (including topsoil), with brick fragments, and cobbles and boulders of sandstone and siltstone, to depths of between 2 – 3.5 m; over

UNIT 2: Clay residual soil with ironstone gravel (up to about 0.5 m thick); over

UNIT 3: Extremely low to very low strength sandstone at depths of between 1.5 – 3.5 m, becoming high strength within 1 m of the top of rock.

The installed standpipe piezometers, and historical groundwater observations, indicate that groundwater seepage flow is occurring within the underlying sandstone, likely from rock defects. Groundwater levels and flow rates are likely to vary over time, depending on downslope drainage and climatic conditions.

9. Comments

9.1 Site Classification

Uncontrolled filling materials were encountered within the boreholes and test pits at the site, interpreted to occur to depths of up to 3.5 m below current surface levels. Based on visual observations and SPT testing, the filling appears to be generally firm to stiff. Some brick and concrete fragments were observed within the filling in five of the seven test locations (i.e. Boreholes BH01, BH02, and HA1 to HA3).

As there is more than 0.4 m of clay filling on the site, the site will be classified as Class P, when assessed in accordance with the requirements of Australian Standard AS 2870 - 2011 "Residential Slabs and Footings" (Reference 6). It is noted that one small tree is present near to the proposed development site, with another small tree within the development footprint.

The Standard does, however, allow for an alternative site classification, provided the site is assessed using engineering principles. Due to the relatively shallow depth to rock, and the proposed design finished levels for the proposed switchroom and cable basement (i.e. at or close to the top of rock), these structures are likely to be founded on weathered sandstone, for a Class A site. Therefore, the site classification in the area of the proposed switchroom is considered to be Class P, with an alternative classification of Class A following site preparation and earthworks.

9.2 Site Preparation and Trafficability

Bulk excavations to around 4 m depth for the cable basement will be within clay and sandy clay filling with cobbles and boulders of sandstone and siltstone (with brick and concrete inclusions), and through a thin layer of residual clay. A limited amount of vertical excavation is expected within extremely low to low strength sandstone near bulk excavation level (BEL: assumed to be at RL45.5 m).

For the construction of pavements and floor slabs the following site preparation measures are recommended:

- Excavate to bulk excavation levels within the pavement and building footprints;
- Remove any vegetation or organic filling and any other deleterious materials below design / bulk excavation levels;
- Where rock is not exposed, 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 ‘weak spots’, in accordance with the project Specification;
- Any heaving materials identified during test 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.

Access to the proposed development, such as for concrete trucks and other vehicles, is likely to be via an existing concrete pavement driveway on the western side of the site.

The filling materials are likely to remain trafficable during the construction period under the applied loading of vehicles with tyres, although some rutting / surface damage is to be expected if traversed following periods of prolonged rainfall. It may be difficult for machines to access the eastern side of the site, due to the side slope (approximate grade 4(H):1(V) - refer Photo 4 in Appendix C). It is suggested a layer of crushed rock or recycled concrete working platform at least 300 mm thick be placed and compacted on the upper part of the site to improve all-weather trafficability. Consideration could be given to incorporating this layer of crushed rock or concrete into the final surface layer of the transformer yard.

For support of mobile crane outriggers on filling materials, thicker working platforms comprising compacted crushed rockfill are likely to be required. An assessment of the required platform thickness should be made once mobile crane equipment has been selected. The suitability of the concrete pavement to take mobile crane outrigger loads will also need to be assessed.

9.3 Excavation

9.3.1 Methods

Excavations for the proposed new switchroom, control room and cable basement are expected through silty clay and sandy clay filling (with cobbles and boulders of sandstone and siltstone), and within extremely low to very low strength sandstone (to a maximum of approximately 0.5 m below the top of rock). Minimal excavations are proposed for the transformer yard. Based upon the assumed cable basement BEL of RL45.5 m, the depth of excavation will be up to about 4 m below existing surface levels.

The clay filling materials cannot be expected to stand vertically for any length of time. Clay filling and extremely low to very low strength sandstone is expected to be exposed at the cable basement bulk level. It is noted that existing buried services (e.g. fire-fighting and stormwater services) are within the proposed building footprint, and may need to be re-located.

Excavations for proposed cable trenches (to link into the existing cable trench: a further approximately 2.3 m depth of excavation) are expected through sandy clay filling (with cobbles and boulders of sandstone and siltstone), and then through sand filling (cable trench backfilling). It is anticipated that clay filling will be exposed along most of the trench length, with the thickness of clay filling and the depth to the base of the cable trench reducing towards Waterloo Road.

It is considered that excavation of the filling, including of the cobbles and boulders, can be readily carried out using conventional earthmoving equipment. Some additional allowance should be made for the on-site handling of these large sandstone pieces. Excavations into the underlying extremely low and very low strength sandstone for bulk and detailed excavations, such as for footing excavations or deepening of trenches, are also likely to be readily carried out using conventional earthmoving equipment. Excavation contractors should make their own assessment of likely productivity depending on their equipment capabilities and operator skills.

9.3.2 Vibration Control

The use of rock hammers or ripping of low and higher strength rock, such as may be encountered at the site below an elevation of RL44.8 m, will cause vibrations which, if not controlled, could possibly result in damage to nearby structures and underground services (e.g. closer than 20 m).

It is assumed that the foundation systems of the nearby substation buildings are founded on low and medium strength sandstone. It is suggested that vibrations be provisionally limited to a peak particle velocity (PPV) of 8 mm/s at the ground level of the neighbouring buildings to protect architectural features, and at cable level within identified underground service trenches. This level complies with AS/ISO 2631.2 - 2014 (Reference 7) and is well below the normal building damage threshold level.

It is suggested that the client assess whether the proposed vibration limit will have a serviceability impact on the existing cables nearby. This provisional limit may need to be modified depending on the result of such assessments. A site specific vibration monitoring trial may be required to determine vibration attenuation once excavation plant and methods have been finalised.

9.3.3 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: Reference 8), 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 was carried out on selected soil and groundwater samples obtained from boreholes and standpipe piezometers, and that a waste classification has been carried out based on these test results (refer to DP Report 86471.01.R.002.Rev0, dated August 2018). 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 undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site. The results of the environmental testing and waste classification are not further discussed herein.

9.3.4 Groundwater

Groundwater was encountered in the current and previous investigation at depths of 2.6 – 6 m (RL47.1 – 43.5 m). It is likely that the proposed excavation for the cable trenches and cable basement will not intersect the regional groundwater table (within the sandstone), however, intermittent groundwater seepage into the cable basement and footing excavations through the soil at the soil-rock interface below RL44 m, during and after periods of wet weathered, is possible. Some de-watering during construction will probably be required, possibly prior to placing concrete in foundation excavations.

Based on experience, it is anticipated that seepage volumes into the cable basement excavation will be small. If groundwater seepage into the basement is not tolerable then the cable basement will need to be waterproofed.

If some seepage into the basement is tolerable (i.e. the basement designed for drained conditions), it can probably be controlled over the long-term via a sub-floor drainage and collection system for seepage removal and to safeguard against uplift pressures. This could comprise a minimum 100 mm thick, durable, open-graded, crushed rock layer with subsurface drains and sumps. It is normally necessary to incorporate provision for regular flushing and cleaning of iron oxide sludge in the maintenance and design of the sub-floor drainage system.

Groundwater entering excavations and post-construction accumulation of groundwater below the basement floor will need to be disposed of in accordance with the Protection of the Environment Operations Act 1997 (POEO Act). Ultimately, this requires that any water discharged into the natural environment should comply with the Australian and New Zealand Guidelines for Fresh and Marine Water Quality, Australian and New Zealand Environment and Conservation Council (ANZECC) and Agricultural and Resource Management Council of Australia and New Zealand.

The above water quality guideline criteria include trigger values for pH, turbidity, nutrients, dissolved oxygen and faecal coliforms (unlikely to be present in excavation water). An appropriate strategy would be to carry out testing of groundwater samples during construction to assess its compliance with the ANZECC water quality guidelines. If the tested water quality complies with the guidelines, it can normally be pumped directly into the stormwater system, subject to regulatory and Council approvals, who normally also have limits on dissolved iron. Alternatively, the pumped groundwater seepage would require on-site treatment such as sedimentation and dosing to improve the quality of water to a sufficient level to comply with the ANZECC requirements before disposal into stormwater, again subject to appropriate approvals. In some circumstances, if groundwater is substantially contaminated, it may be necessary to dispose of it off-site as liquid waste or through the sewer via a trade waste agreement.

9.3.5 Excavation Slopes

It is understood that excavations for the proposed cable basement will be required beneath the footprint of the switchroom, with a transformer yard proposed to be constructed to the west. Based on the supplied buried services drawings ("S23336 Reduced file size.pdf"), services are indicated to occur parallel to and within the Waterloo Road property boundary. On the assumption that the filling soils beneath the transformer yard are not to be excavated, these documents indicate that there will be insufficient space between the proposed buildings and transformer yard (western side), and the buried services along the property boundary (southern side) to enable the excavation faces to be battered to a safe angle during construction and which will therefore need to be supported. Sufficient space will exist on the other two sides of the excavation to enable slope batters to be formed, which will require a mid-height bench with width no less than 1 m.

Suggested temporary and permanent maximum batter slope angles for slopes not exceeding 2 m in height are presented in Table 7.

Table 7: Recommended Temporary and Permanent Maximum Batter Slopes

Material	Batter Slope Ratio (H:V)	
	Short-term (Temporary)	Long-term (Permanent)
Filling, firm to stiff condition	1 : 1	1.5 : 1[a]
Residual Clay, stiff condition	1 : 1	1.5 : 1[a]
Extremely low to very low strength sandstone	0.75 : 1	1 : 1

Notes: (a) 1.5(H):1(V) slope is equivalent to a 34 degree slope, 3(H):1(V) slope if vegetation and maintenance of the batters is required.

Vertical excavations for cable trenches will likely require shoring boxes. Material stockpiles and machinery / equipment should not be stored at the crest of unsupported excavations.

9.4 Excavation Support

9.4.1 General

As insufficient space exists to batter the western and southern sides of the 4 m deep cable basement excavation to the recommended slope angles, the excavation will require both temporary and permanent lateral support, to ensure that excavation stability is maintained.
The shoring / support system considered suitable to retain the soil slope on the western and southern sides of the cable basement is bored concrete soldier piles, with the 'gap' between piles supported

using steel mesh-reinforced shotcrete, with short (i.e. 1 m long) steel soil nails or dowels installed into the filling material, and incorporated into the shotcrete using 'spider plates'.

Geotechnical inspections must be carried out during excavation, with further stabilisation and drainage measures to be implemented as required (e.g. strip drains) to maintain appropriate excavation stability.

9.4.2 Shoring Design and Retaining Walls

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of medium strength rock, which for the current site is the full excavation depth. Retaining walls are also subjected to earth pressures for their full height.

Material and strength parameters that may be used for preliminary design of excavation support structures and retaining walls (if required) are presented in Table 8 and Table 9. A triangular earth pressure distribution may be adopted on the rear of the shotcrete. Unless positive drainage measures can be incorporated to prevent water pressure build up behind the shoring walls, full hydrostatic head should be allowed for in design while, at the same time, allowing for the soil density to reduce to the buoyant condition.

The values of active earth pressure coefficient, K_a, to be used for estimating soil pressures, are for a level ground surface and a flexible wall allowing for some lateral movement. To minimise movement of adjacent footings, the soil and weathered rock below the foundations should be designed using an "at rest" lateral earth pressure coefficient (K_o) – refer Table 8.

Table 8: Typical Material and Strength Parameters for Excavation Support Structures – Earth Pressures

Material	Bulk Density (γ: kN/m^3)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	Ultimate Passive Earth Pressure (kPa)
Filling	20	0.3	0.6	-
Residual Clay	20	0.25	0.45	-
Sandstone: EL to VL	22	0.1	0.15	300

Notes: Strength descriptors: EL = Extremely low, VL = Very low

Table 9: Typical Material and Strength Parameters for Excavation Support Structures

Material Type	Effective Friction Angle (φ')	Effective Modulus E' (MPa)	Poisson's Ratio (ν')
Filling (Clay)	25	4	0.35
Residual Clay	25	8	0.35
Sandstone: EL to VL	30	75	0.15

Notes: Strength descriptors: EL = Extremely low, VL = Very low

Design of retaining walls should allow for lateral pressures from surcharge loads above the wall, such as from sloping ground, traffic loading, or arising from construction plant. The ultimate passive pressure given in Table 8 should incorporate a suitable factor of safety to limit deflection.

9.5 Foundations

Based upon the results of the investigations, the following materials are anticipated to be encountered at bulk excavation level:

- Cable basement (BEL: RL45.5 m) – residual clay and extremely low to very low strength sandstone;
- Control room (BEL: RL49.0 m) – clay filling; and
- Transformer yard (BEL: RL49.0 m) – clay filling.

It is recommended that all footings be taken to sandstone, via driven or bored piles for the control room and transformer yard, or shallow pad footings for the switchroom and cable basement. The piles should be socketed into a uniform founding stratum such as low or higher strength sandstone: Borehole BH01 encountered consistent high strength sandstone at a depth of 1.9 m below the “top of rock”. An allowance for temporary steel casing may be required for the bored piles, to prevent the cobbles and boulders from falling into pile holes and creating other challenges. The extremely low to very low strength sandstone stratum could be used for shallow footings (at basement level), with an allowable bearing pressure of 1,000 kPa.

Recommended maximum allowable (and ultimate) bearing pressures, shaft adhesions and modulus values for the rock encountered in boreholes at the site are presented in Table 10. These parameters apply to the design of socketed bored piles, for the support of axial compression loadings. They can be adopted on the assumption that the excavations are clean and free of loose debris, with pile sockets free of smear and adequately roughened immediately prior to concrete placement. For piles driven to refusal on rock, the structural capacity of the piles is likely to govern design.

Table 10: Recommended Design Parameters and Moduli for Foundation Design

Foundation Stratum[1]	Allowable End Bearing (MPa)	Ultimate End Bearing (MPa)	Allowable Shaft Adhesion (kPa)	Ultimate Shaft Adhesion (kPa)	Field Elastic Modulus (MPa)
Sandstone – Extremely low to very low strength	1.0	3.0	75	150	50
Sandstone – Low Strength	3.5	20	350	800	350
Sandstone – High Strength	6.0	60	600	1500	900

Notes: (1) Based on Pells et. al (1998).
(2) Shaft adhesion applicable to the design of bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved

Foundations proportioned using the allowable parameters would be expected to settle less than 1% of the footing width (or pile diameter) under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value. An experienced geotechnical professional should inspect all bored pile excavations prior to the placement of concrete and steel, to check the adequacy of the foundation material and to undertake spoon testing as appropriate.

Whilst the allowable bearing pressure is not likely to be critical to the design, pile footings taken down into consistent high strength sandstone could potentially be designed for an allowable bearing pressure of 6,000 kPa and possibly up to 12,000 kPa, subject to additional investigation or spoon testing during construction. If higher bearing pressures are used in design, however, then additional testing will be required in the form of cored boreholes and/or spoon testing of footings, to ensure there are no defects beneath footings. Alternatively, if a lower allowable bearing pressure of 3,500 kPa is adopted then testing during construction could be limited to inspection of foundations.

9.6 Materials Re-Use

The provided drawings indicate that placement of filling elsewhere on the site is not proposed as part of the proposed substation development, and that limited additional new areas of pavement are proposed.

The materials anticipated to be excavated from within the building footprint (e.g. clay filling, residual clay and weathered sandstone) are considered to be suitable from a geotechnical perspective for re-use at the site, however, sieving and segregation / stockpiling of the coarse material (e.g. boulders) from the filling is likely to be required. Re-use could be considered for new pavement areas, provided the materials are moisture conditioned prior to compaction, and as general filling for landscaping, although past experience with these materials indicates the potential for low soil fertility. Cross-fall to suitable drainage will be required to prevent saturation, waterlogging and softening of the clay.

Further advice should be sought when further details are known, on issues including placement and test rolling methodologies, required compaction densities and recommended layer thicknesses.

9.7 Floor Slab and Pavement Design Parameters

The floor at basement level can be designed as a slab on ground. The final rock surface (at BEL) should be trimmed and scraped clean of debris, and the stiff residual clay or clay filling (where it occurs at BEL) compacted using a smooth drum roller.

Based upon CBR test results from the filling and on residual clay samples from nearby sites, and allowing for some variability, it is suggested that a design CBR value for the subgrade material and reworked filling material not exceed 5%. If imported material is used to level the site and form subgrade levels, the design CBR value will depend on the type of imported material.

The design CBR value is based on the provision of adequate surface and subsoil drainage to maintain the subgrade as close to the optimum moisture content as possible. Subsoil drainage should be installed adjacent to pavement edges abutting lawns or garden areas.

It will be necessary to provide under-floor drainage to safeguard against uplift pressures if the basement floor and walls are designed for drained conditions. This could comprise a minimum 100 mm thick, durable open-graded crushed rock with subsurface drains and sumps.

9.8 Seismic Design

In accordance with the Earthquake Loading Standard, AS1170.4 – 2007 (Reference 9), the site has a hazard factor (z) of 0.08 and a site sub-soil class of shallow soil (C_e), being underlain by materials with a compressive strength less than 0.8 MPa, and with a surface layer of less than 25 m depth of stiff cohesive soil.

10. References

1. Herbert C., 1983, Sydney 1:100 000 Geological Sheet 9130, 1st edition. Geological Survey of New South Wales, Sydney.
2. The Department of Land and Water Conservation, 1997 (Edition 2). 1:25 000 Acid Sulphate Soil Risk map for Prospect / Parramatta River.
3. Australian Standard AS1726-2017, "Geotechnical Site Investigations", Standards Australia.
4. Australian Standard AS3798-2007, "Guidelines on Earthworks for commercial and Residential Developments", March 2007, Standards Australia.
5. Australian Standard AS2159-2009, "Piling – Design and Installation", third edition, 2009, Standards Australia.
6. Australian Standard AS2870-2011, "Residential Slabs and Footings", April 2011, Standards Australia.
7. Australian / International Standard 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)".
8. NSW Environment Protection Authority (EPA), 2014, "Waste Classification Guidelines".
9. Australian Standard AS1170.4 – 2007, "Structural design actions, Part 4: Earthquake actions in Australia".

11. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at Macquarie Park Substation, 21 Waterloo Road, Macquarie Park, in accordance with DP's proposal SYD180586 dated 22 June 2018 and acceptance received from Mr Paul Hurst dated 28 June 2018. The work was carried out under the amended Master Services Agreement – Design and Related Services Panel (Purchase Order Number 4500978730). This report is provided for the exclusive use of Ausgrid 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, with Preliminary Site Investigation and Limited Stage 2 Contamination Investigation, and Preliminary Waste Classification reports produced under separate covers. 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 containing material was not detected by observation from boreholes or at the ground surface, at the test locations sampled and analysed. Building demolition materials, such as bricks and concrete fragments, were also located within 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). 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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

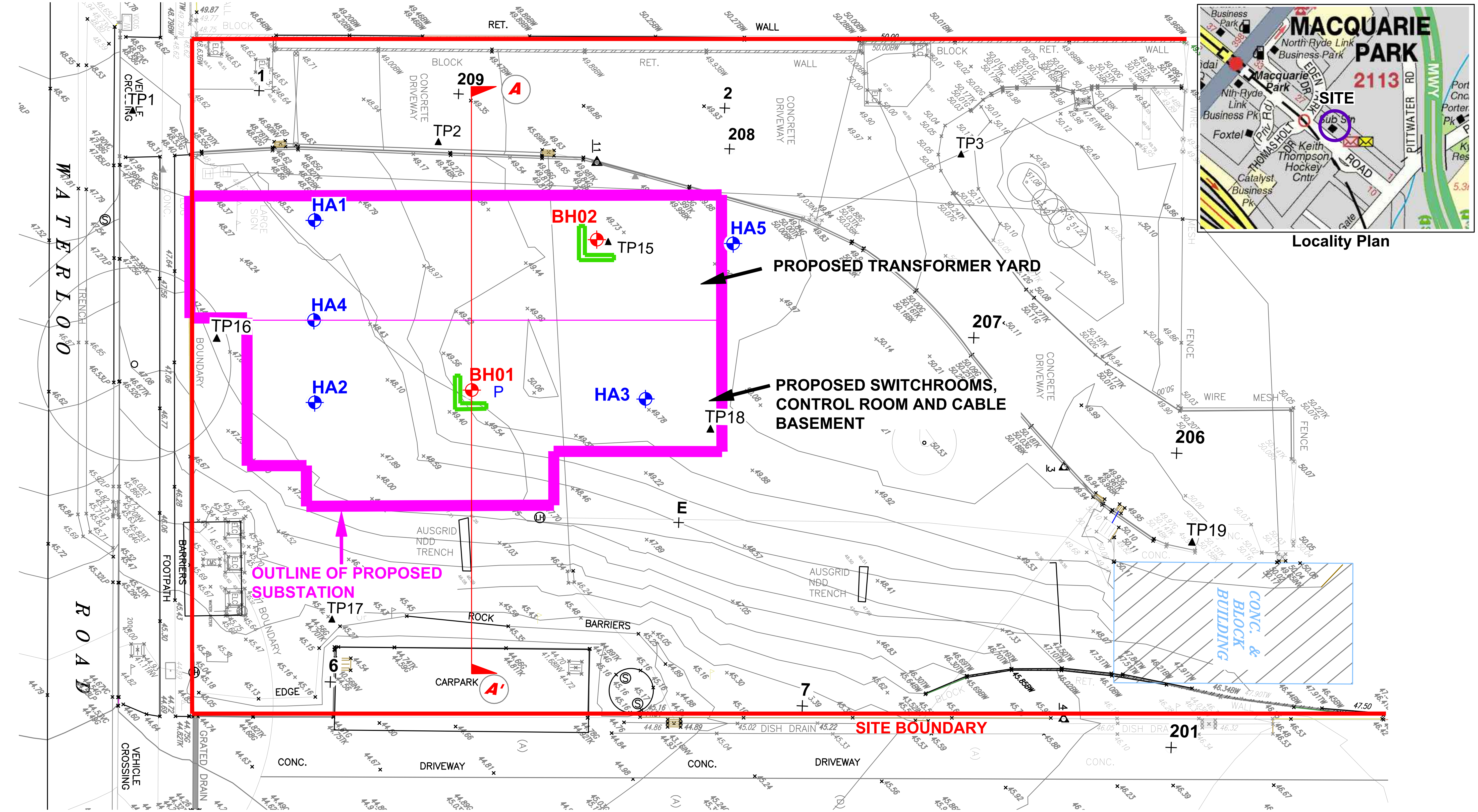
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

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

Appendix B

Drawings



Locality Plan

NOTE:
1: Base drawing from Degotardi Smith & Partners Pty Ltd (Dwg S 23336,dated 12.1.2018)
2: Test locations are approximate only and are shown with reference to existing features
3: Survey levels are relative to the Australian Height Datum.

0 1 2 4 6 8 10 15 20 30m

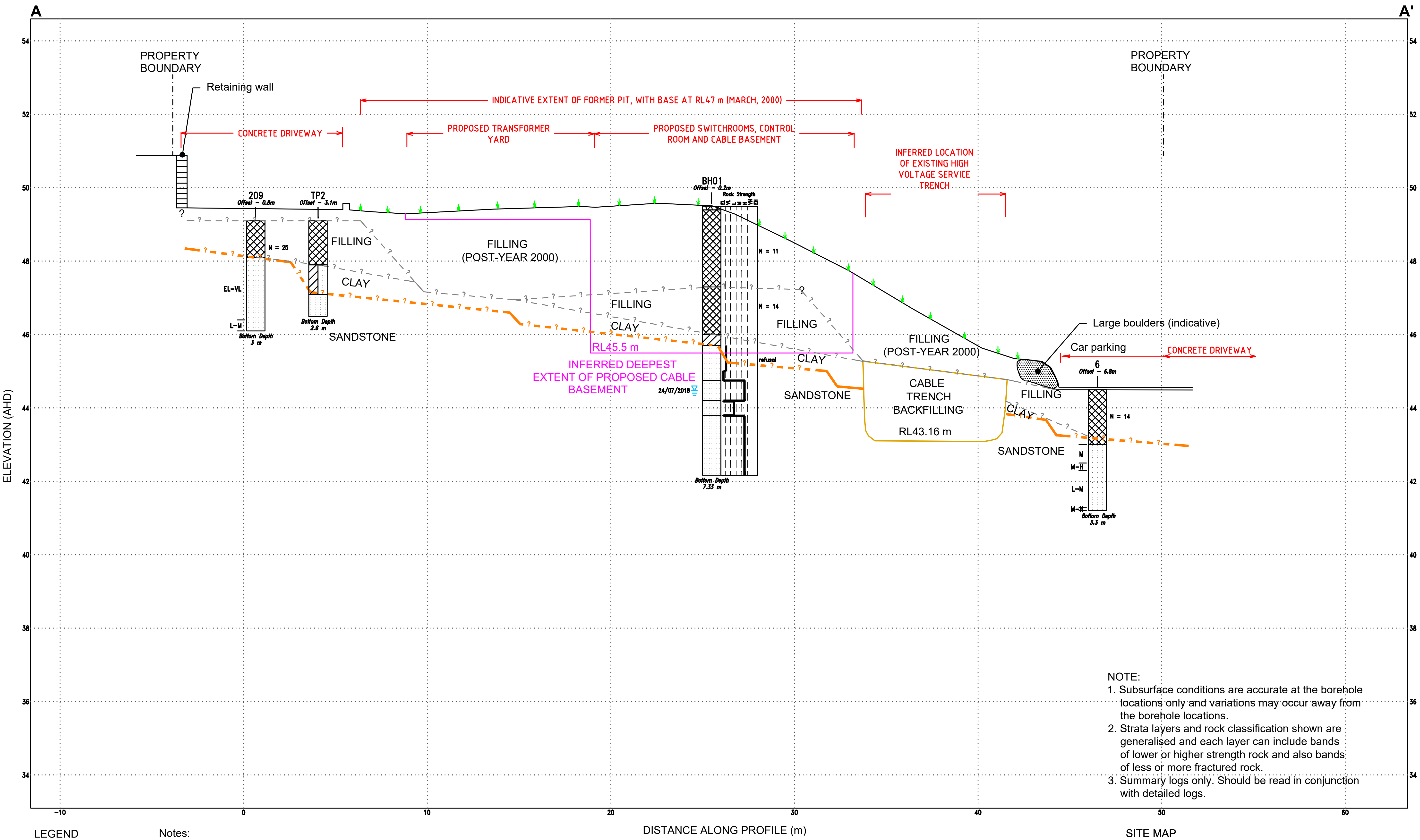
1:300 @ A3

LEGEND

- ▲ Historical test pit (Golders)
- + Historical borehole (J&K / EIS)
- Cored borehole
- Hand augered borehole
- NDD trench
- A — A' Geotechnical cross-section
- P Standpipe Piezometer

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: AusGrid		TITLE: **Test Location Plan**	PROJECT No: 86471.00
OFFICE: Sydney	DRAWN BY: PSCH	**Proposed 132/33kV Substation**	DRAWING No: 1
SCALE: 1:300 @ A3	DATE: 4.7.2018	**21 Waterloo Road, MACQUARIE PARK**	REVISION: 0



LEGEND

- Topsoil
- Filling
- Clay
- Sandstone

Notes:

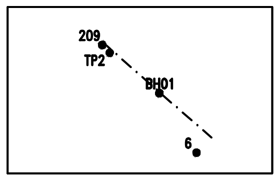
1. Vertical and horizontal scales are not equal
2. Profile of top of rock for an inferred pit based on J&K Report 15400SL Figure 7, dated September 2000.
3. Summarised boreholes and borehole surface levels taken from J&K Report (September 2000).
4. Level of base of existing high voltage cable trench taken from Enerserve Drawing 128302 (Rev1), dated November 2000.

TESTS / OTHER

N - Standard penetration test value

- Water level

SITE MAP



0 4

Horizontal Scale (metres)

Vertical Exaggeration = 2.0

Douglas Partners

Geotechnics / Environment / Groundwater

CLIENT: Ausgrid		TITLE: **Geotechnical Cross-Section A-A'**	PROJECT No: 86471.00
OFFICE: Sydney	DRAWN BY: HDS	**Macquarie Park Zone Substation**	DRAWING No: 2
SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 27.07.2018	**21 Waterloo Road, MACQUARIE PARK**	REVISION: 0

Appendix C

Site Photographs

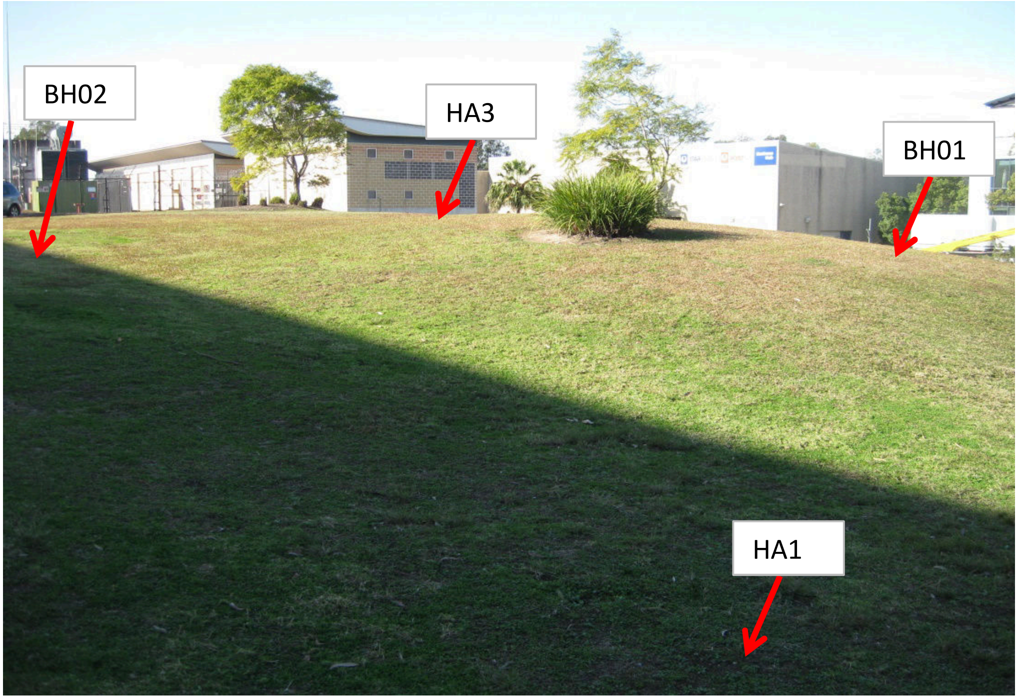


Photo 1 - View north-east towards the existing substation and across the proposed substation footprint. The approximate locations of the tests are indicated as shown.

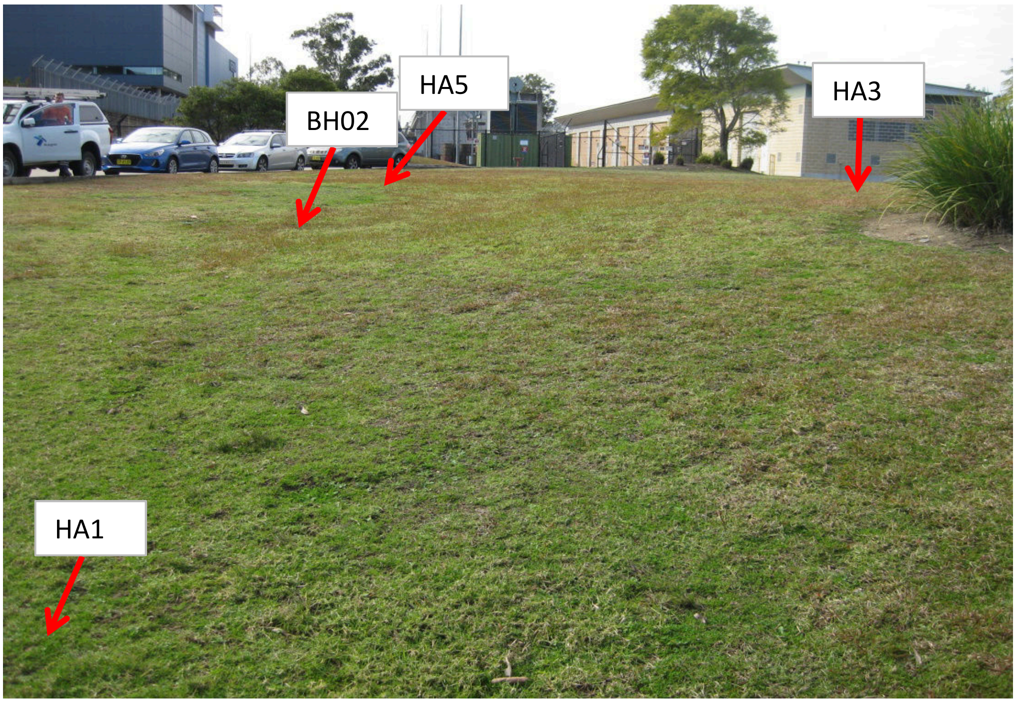


Photo 2 - View north-east towards the existing substation and across the proposed substation footprint. The approximate locations of the tests are indicated as shown.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT: 86471.00
	Geotechnical Assessment	PLATE No: 1
	21 Waterloo Rd, Macquarie Pk	REV: 0
	CLIENT: Ausgrid	DATE: 31-Jul-18

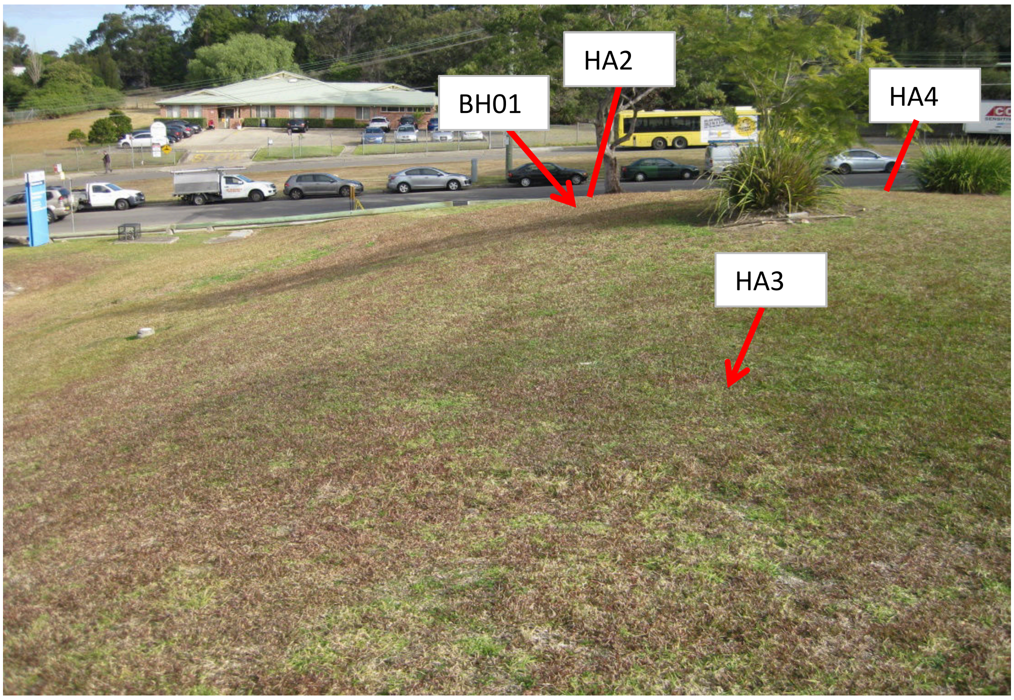


Photo 3 - View south-west across the proposed substation footprint, with Waterloo Road in the background. Approximate locations of test positions are as shown.

Photo 4 - View north-east towards the existing substation buildings, east of the proposed substation footprint.

Douglas Partners Geotechnics / Environment / Groundwater	Site Photographs	PROJECT: 86471.00
	Geotechnical Assessment	PLATE No: 2
	21 Waterloo Rd, Macquarie Pk	REV: 0
	CLIENT: Ausgrid	DATE: 31-Jul-18

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.

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

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

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

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

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

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

Rock Descriptions

Douglas Partners

Rock Strength

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

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

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

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Degree of Fracturing

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

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

Rock Descriptions

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

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

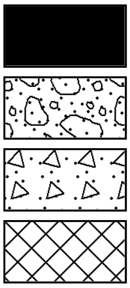
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

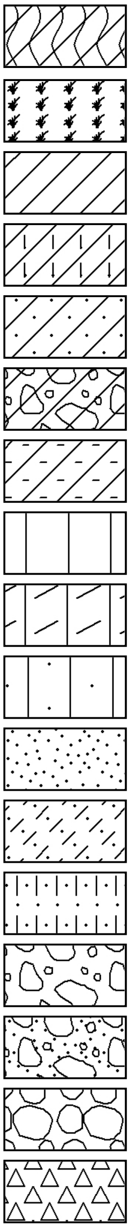
Graphic Symbols for Soil and Rock

General



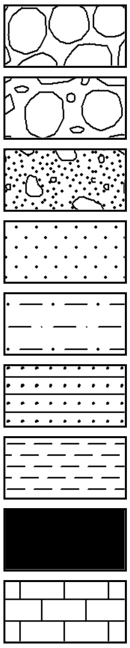
- Asphalt
- Road base
- Concrete
- Filling

Soils



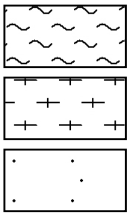
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



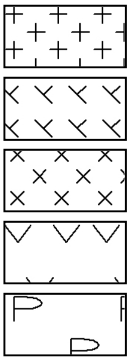
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks



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

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 49.5 AHD
EASTING: 327069
NORTHING: 6259841
DIP/AZIMUTH: 90°/--

BORE No: BH01
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Sampling Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL: dark grey and grey, silty sand topsoil filling, trace clay.		A/E			
	FILLING: brown to dark brown, sandy clay filling, with some sandstone, siltstone, brick and concrete gravel and cobbles, moist, generally in a stiff condition.		A/E			
			A/E*			
			S			3,6,5 N = 11
2.2	FILLING: grey to dark brown, clay filling, with some sandstone and siltstone gravel and cobbles, trace sand, moist, generally in a stiff to very stiff condition.		A/E			
			S			4,3,11 N = 14
3.5	CLAY: stiff, brown, slightly silty clay, with trace ironstone gravel, damp (possibly filling).					
3.8	SANDSTONE: extremely low to very low strength, extremely weathered, pale grey, medium grained sandstone with ironstone bands.		A/E			
			S			16,19,6/60 refusal
		4.50-4.75m: Cs, 250mm	C	100	86	
4.75	SANDSTONE: high strength, highly weathered, fractured to slightly fractured, pale grey to red-brown, medium grained sandstone.					PL(A) = 1.2
5.3	SANDSTONE: low strength, highly weathered, fractured to slightly fractured, pale grey, medium grained sandstone.	5.30m: Cs, 20mm 5.36m: B, 0-10°, pl, ro, cly co 5.47m: J, 40-50°, pl, ro, cly co				
5.71	SANDSTONE: high strength, moderately weathered, slightly fractured to unbroken, pale brown to red-brown, medium grained sandstone.	5.7m: B, 0-10°, pl, ro, cly co 5mm 5.91m: B, 0-10°, ro, cln				PL(A) = 1.8
						PL(A) = 1.5
						PL(A) = 1.5
7.33	Bore discontinued at 7.33m - Target depth reached.					

RIG: Hanjin D8 **DRILLER:** BG **LOGGED:** AT **CASING:** HQ to 4.5m

TYPE OF BORING: Solid flight auger to 4.5m, NMLC coring to 7.33m.

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: *TR1110718 and TR2110718 taken at 0.9-1.0m.
Standpipe installed, blank 0-4.33m; screen 4.33-7.33m; backfill 0-3.5m; bentonite plug 3.5-4.0m; gravel 4.0-7.33m, gatic cover at 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

BORE: BH01 PROJECT: MACQUARIE PARK STS AUGUST 2018

4.5m – 7.33m

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 49.6 AHD
EASTING: 327066
NORTHING: 6259856
DIP/AZIMUTH: 90°/--

BORE No: BH02
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL: dark grey and grey, silty sand topsoil filling, trace clay.		A/E			
	FILLING: brown to dark brown, sandy clay filling, with some sandstone, siltstone, brick and concrete gravels and cobbles, moist, generally in a firm to stiff condition.		A/E			
			E* B			
			S			3,2,4 N = 6
2.7	SANDSTONE: very low strength, highly weathered, red-brown, medium grained sandstone with some ironstone bands.		S			19,20/100 refusal
3.24	SANDSTONE: low to medium strength, highly weathered, fractured, pale grey to red-brown, medium grained sandstone with some extremely low strength clay bands.	3.18m: Cs, 20mm 3.5m: B, 0-10°, ro, cln 3.63m: Cs, 25mm 3.9m: Cs, 20mm 4.14m: B, 0-10°, ro, fe stn	C	100	74	PL(A) = 0.27 PL(A) = 0.41
4.4	SANDSTONE: extremely low strength, extremely to highly weathered, highly fractured, pale grey, medium grained sandstone.	4.4m: B, 0-10°, ro, cly vn 4.41m: B, 0-10°, ro, cln 4.56m: B, 0-10°, ro, cln 4.66m: B, 0-10°, ro, cln 4.76m: B, 0-10°, ro, cln 4.83m: B, 0-10°, ro, cln 4.89m: B, 0-10°, ro, cln 4.94m: B, 0-10°, ro, cln	C	100	56	
5.0	SANDSTONE: high strength, moderately weathered, slightly fractured and unbroken, pale brown, medium grained sandstone.	5.39m: Cs, 10mm 6.46m: Cs, 40mm 6.55m: B, 0-10°, ro, cln				PL(A) = 1.2 PL(A) = 2.3
7.0	Bore discontinued at 7.0m - Target depth reached.					

RIG: Hanjin D8 **DRILLER:** BG **LOGGED:** AT **CASING:** HQ to 3.0m
TYPE OF BORING: Solid flight auger to 3.0m, NMLC coring to 7.0m.
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *TR6110718 and TR7110718 taken at 0.9-1.0m.

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: BH02 PROJECT: MACQUARIE PARK STS AUGUST 2018

3.0m – 7.0m

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 48.5 AHD
EASTING: 327051
NORTHING: 6259840
DIP/AZIMUTH: 90°/--

BORE No: HA1
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 0mm)
	0.1	TOPSOIL: brown, silty clay topsoil filling, some rootlets.		A/E	0.1				
	0.4	FILLING: brown, silty clay filling, slightly gravelly and sandy, gravels are sub-angular to sub-rounded and fine to coarse igneous, sandstone, siltstone and brick, with some cobbles up to 80mm diameter, damp.		A/E*	0.4				
		Bore discontinued at 0.4m - Hand auger refusal on possible cobble/boulder.							

RIG: Hand tools **DRILLER:** RMM **LOGGED:** RMM **CASING:** None

TYPE OF BORING: Hand auger (100mm diameter) to 0.4m.

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: *TR4110718 and TR5110718 taken at 0.4m.

☐ 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
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BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 47.6 AHD
EASTING: 327062
NORTHING: 6259831
DIP/AZIMUTH: 90°/--

BORE No: HA2
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 0mm)
	0.1	TOPSOIL: brown, silty clay topsoil filling, some rootlets.		A/E	0.1				
47	0.55	FILLING: brown then grey, silty clay filling, slightly gravelly, gravels are sub-angular to sub-rounded and fine to coarse, mostly siltstone with igneous, sandstone and brick, with some siltstone cobbles up to 100mm diameter, damp to humid.		A/E	0.5				
		Bore discontinued at 0.55m - Hand auger refusal in gravelly filling.							

RL scale: 47, 46, 45, 44, 43, 42, 41, 40. Depth scale: 1, 2, 3, 4, 5, 6, 7. Dynamic Penetrometer scale: 5, 10, 15, 20.

RIG: Hand tools **DRILLER:** RMM **LOGGED:** RMM **CASING:** None
TYPE OF BORING: Hand auger (100mm diameter) to 0.55m.
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

☐ 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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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 49.7 AHD
EASTING: 327078
NORTHING: 6259851
DIP/AZIMUTH: 90°/--

BORE No: HA3
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 0mm)
	0.1	TOPSOIL: brown, silty clay topsoil filling, some rootlets.		A/E	0.1				
	0.5	FILLING: brown, silty clay filling, slightly gravelly and sandy, gravels are sub-angular to sub-rounded and fine to coarse, mostly sandstone with some igneous and brick, damp to humid.		A/E	0.5				
		Bore discontinued at 0.5m - Hand auger refusal on possible cobble/boulder.							

RL scale: 49, 48, 47, 46, 45, 44, 43, 42. Depth scale: 1, 2, 3, 4, 5, 6, 7. DCP scale: 5, 10, 15, 20.

RIG: Hand tools **DRILLER:** RMM **LOGGED:** RMM **CASING:** None
TYPE OF BORING: Hand auger (100mm diameter) to 0.5m.
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

☐ 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

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 48.2 AHD
EASTING: 327057
NORTHING: 6259835
DIP/AZIMUTH: 90°/--

BORE No: HA4
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 0mm)
48	0.1	TOPSOIL: brown, silty clay topsoil filling, some rootlets.							
	0.6	FILLING: brown then grey, silty clay filling, slightly gravelly, gravels are sub-angular and fine to coarse, mostly siltstone with some sandstone, damp to humid.		A/E	0.5				
		Bore discontinued at 0.6m - Hand auger refusal in gravelly filling.							

RIG: Hand tools **DRILLER:** RMM **LOGGED:** RMM **CASING:** None
TYPE OF BORING: Hand auger (100mm diameter) to 0.6m.
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

☐ 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

BOREHOLE LOG

CLIENT: Ausgrid
PROJECT: Macquarie Park Zone Substation
LOCATION: 21 Waterloo Road, Macquarie Park

SURFACE LEVEL: 49.6 AHD
EASTING: 327073
NORTHING: 6259864
DIP/AZIMUTH: 90°/--

BORE No: HA5
PROJECT No: 86471.00
DATE: 11/7/2018
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 0mm)
	0.1	TOPSOIL: brown, silty clay topsoil filling, some rootlets.							
	0.4	FILLING: brown, silty clay filling, slightly gravelly and sandy, gravels are sub-angular to sub-rounded, fine to coarse, mostly sandstone, siltstone and igneous, damp to humid.							
		Bore discontinued at 0.4m - Hand auger refusal in gravelly filling.							

RIG: Hand tools **DRILLER:** RMM **LOGGED:** RMM **CASING:** None
TYPE OF BORING: Hand auger (100mm diameter) to 0.4m.
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

☐ 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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Groundwater Field Sheet

Bore Volume = casing volume + filter pack volume
$= \pi h_1 d_2^2/4 + n(\pi h_1 d_1^2/4 - \pi h_2 d_2^2/4)$

Where: $\pi = 3.14$
n = porosity (0.3 for most filter pack material)
h_1 = height of water column
d_1 = diameter of annulus
h_2 = length of filter pack
d_2 = diameter of casing

Bore Vol Normally: 7.2*h

Project and Bore Installation Details	
Bore / Standpipe ID:	~~86471.01~~ BH01
Project Name:	Macquarie Park Substation
Project Number:	86471.01
Site Location:	
Bore GPS Co-ord:	
Installation Date:	11.7.18
GW Level (during drilling):	m bgl
Well Depth:	m bgl
Screened Interval:	m bgl
Contaminants/Comments:	
Bore Development Details	
Date/Time:	
Purged By:	
GW Level (pre-purge):	m bgl
GW Level (post-purge):	m bgl
PSH observed:	Yes / No (interface / visual). Thickness if observed:
Observed Well Depth:	m bgl
Estimated Bore Volume:	L
Total Volume Purged:	(target: no drill mud, min 3 well vol. or dry)
Equipment:	
Micropurge and Sampling Details	
Date/Time:	17/3
Sampled By:	MW
Weather Conditions:	Fine 6.0m (estimated)
GW Level (pre-purge):	? m bgl Wasn't getting tone from dip meter,
GW Level (post sample):	? m bgl despite it working correctly from other water sources.
PSH observed:	Yes / No (interface / visual). Thickness if observed:
Observed Well Depth:	7.04 m bgl
Estimated Bore Volume:	L
Total Volume Purged:	L
Equipment:	Geosub / Bailer

Water Quality Parameters

Time / Volume	DO (mg/L)	EC (μS or mS/cm)	pH	Redox (mV)	Temp (°C)
Stabilisation Criteria (3 readings)	**+/- 0.3 mg/L**	**+/- 3%**	**+/- 0.1**	**+/- 10 mV**	**0.1°C**
12:55	2.30	705 μS	6.17	70	19.3
12:57	2.14	708	6.16	69	19.5
12:59	2.06	706	6.16	67	20.0
Additional Readings Following stabilisation:	DO % Sat	SPC	TDS		
	209ppm				

All readings taken from single bailer in bucket due to insufficient water in well for continuous flow

Sample Details

Sampling Depth (rationale):	6.5 m bgl,
Sample Appearance (e.g. colour, siltiness, odour):	Very silty, no odour
Sample ID:	MW1
QA/QC Samples:	metals, PAH
Sampling Containers and filtration:	Sampling conducted from bailer as couldn't get flow from Geosub.

Appendix E

Laboratory Test Results

Material Test Report

Report Number:	**86471.00-1**
Issue Number:	1
Date Issued:	24/07/2018
Client:	Ausgrid
	Level 1/9-13 Carter Street, Lidcombe NSW 2141
Contact:	Paul Hurst
Project Number:	86471.00
Project Name:	MACQUARIE PARK Zone Substation
Project Location:	Macquarie Park Substation, Macquarie Park
Work Request:	3477
Sample Number:	18-3477A
Date Sampled:	11/07/2018
Sampling Method:	Sampled by Engineering Department
Sample Location:	**BH02 (0.5 - 1.5m)**
Material:	Sandy Clay Filling

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Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: mick.gref@douglaspartners.com.au

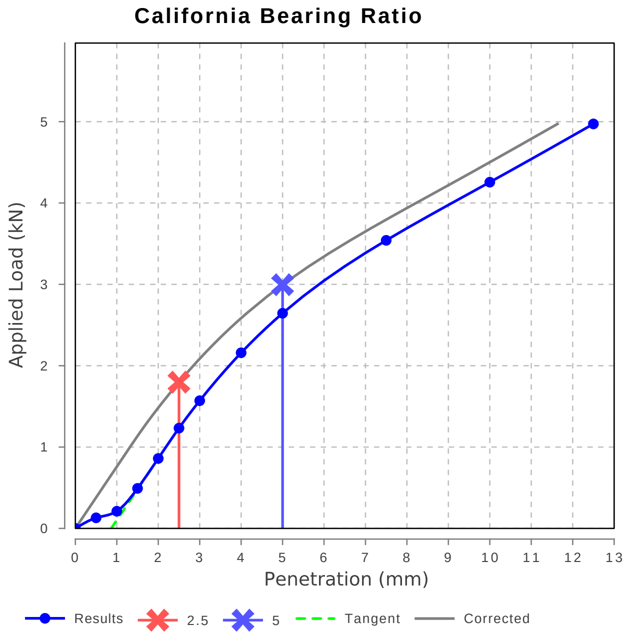
NATA WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Michael Gref

NATA Accredited Laboratory Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**15**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)	1.91		
Optimum Moisture Content (%)	11.5		
Laboratory Density Ratio (%)	100.0		
Laboratory Moisture Ratio (%)	100.0		
Dry Density after Soaking (t/m^3)	1.90		
Field Moisture Content (%)	14.6		
Moisture Content at Placement (%)	11.6		
Moisture Content Top 30mm (%)	14.2		
Moisture Content Rest of Sample (%)	13.9		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	172		
Swell (%)	0.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	5.7		



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CERTIFICATE OF ANALYSIS 196123-A

Client Details

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

Sample Details

Your Reference	**86471.00, Geotech. Investigation, Macquarie Park**
Number of Samples	11 Soil
Date samples received	12/07/2018
Date completed instructions received	12/07/2018

Analysis Details

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

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

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

Report Details

Date results requested by	20/07/2018
Date of Issue	19/07/2018

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Soil Aggressivity			
Our Reference		196123-A-2	196123-A-11
Your Reference	UNITS	BH01	HA3
Depth		2.5-2.95	0.5
Date Sampled		11/07/2018	11/07/2018
Type of sample		Soil	Soil
pH 1:5 soil:water	pH Units	8.9	7.8
Electrical Conductivity 1:5 soil:water	µS/cm	110	28
Resistivity by calculation	ohm m	91	360
Chloride, Cl 1:5 soil:water	mg/kg	10	10
Sulphate, SO4 1:5 soil:water	mg/kg	43	<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-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	196123-A-1
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	2	8.9	8.9	0	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	2	110	120	9	106	[NT]
Resistivity by calculation	ohm m	0.1	Inorg-002	<0.1	2	91	86	6	[NT]	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	10	10	0	111	106
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	43	45	5	115	113

Result Definitions

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

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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CHAIN OF CUSTODY DESPATCH SHEET

Project No:	86471.00	**Suburb:**	Macquarie Park	**To:**	Envirolab Services
Project Name:	Geotechnical Investigation	**Order Number**			12 Ashley St Chatswood 2067
Project Manager:	Huw Smith	**Sampler:**	AT/RMM	**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 ☑ Fridge ☐ 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: Agressivity										Notes/preservation
BH01/2.5-2.95	2	11.07.18	s	g	×										
HA3/0.5	11	11.07.18	s	g	×										
															Envirolab Services 12 Ashley St Chatswood NSW 2067 Ph: (02) 9910 6200 Job No: 196123-A
															Date Received: 12/7/18 Time Received: 15:30 Received By: MT/YH Temp: Cool/Ambient Cooling: Ice/Icepack Security: Intact/Broken/None
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: 196123-A

Metals to Analyse:

Total number of samples in container: **Relinquished by:** NLE **Transported to laboratory by:** Bonds

Send Results to: Douglas Partners Pty Ltd **Address:** 96 Hermitage Road West Ryde NSW 2114 **Phone:** (02) 4271 1836 **Fax:** (02) 4271 1897

Signed: 12/7 **Received by:** MT EU **Date & Time:** 12/7/18 1030