



Douglas Partners

Geotechnics | Environment | Groundwater

Report on
Geotechnical Investigation

Building J03
Engineering Faculty, The University of Sydney

Prepared for
The University of Sydney

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Integrated Practical Solutions



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

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Report on Geotechnical Investigation

Building J03

Engineering Faculty, The University of Sydney

1. Introduction

This report presents the results of a geotechnical investigation undertaken for Building J03 (known as Electrical Engineering) at Engineering Faculty, The University of Sydney. The investigation was commissioned in an email dated 25 August 2016 by Charisma Nambiar of The University of Sydney and was undertaken in accordance with Douglas Partners' proposal SYD160938 dated 18 August 2016.

A geotechnical investigation was undertaken in September and October 2016 to provide information on the subsurface conditions on the site and included the drilling of two boreholes, laboratory testing and engineering analysis. Details of all the field work and comments relevant to design and construction are given in this report.

2. Site Description and Geology

The site is approximately rectangular in shape and measures about 70 m by 50 m. It is located within the grounds of the Engineering Faculty, The University of Sydney where the multi-storey Electrical Engineering Building currently is situated. It is surrounded by Maze Crescent to the west, Seymour Centre to the north and other university buildings to the south and east.

The site of the Electrical Engineering building is relatively level having been excavated into a gently rising slope to the west and possibly some filling on the eastern side.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale which typically comprises dark grey shale and laminite.

3. Field Work Methods

The field work for the geotechnical investigation included the drilling of two cored bores (Bores 1 to 2) at the locations shown on Drawing 1, in Appendix B. The bores were located on the outside of the building envelope as there were access difficulties in drilling in the courtyard of the building.

The bores were drilled to approximately 10 m depth using a bobcat mounted drilling rig. The bores were initially augered to the top of rock at depths of 2.5 m and 1.0 m for Bores 1 and 2 respectively and then advanced using NMLC-sized diamond core drilling equipment to obtain 50 mm diameter continuous samples of the rock for identification and strength testing purposes.

In addition, a dynamic cone penetrometer (DCP) test was carried out in the sunken garden to the south of the site.

The ground surface levels at the borehole locations were determined by hand held GPS methods.

4. Field Work Results

The subsurface conditions encountered in the bores are presented in the borehole logs in Appendix C. Notes defining descriptive terms and classification methods are included in Appendix C.

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

- Topsoil or Pavement – 100 mm to 300 mm of silty clay topsoils;
- Filling – clay, sandy clay with some gravel to depths between 0.5 m and 0.6 m;
- Clay – very stiff hard clay to a depth of 2.2 m in Bore 1;
- Shaly Clay – hard shaly clay to depths of 2.5 m and 1.3 m; and
- Rock – extremely low strength and very low strength laminite and shale grading into medium strength shale at depths of 7.3 m and 3.5 m. Medium and high strength sandstone and/or siltstone were below the shale. Some joints, dipping in the range of 30° to 85°, were observed in the core samples.

Table 1 summarises the levels at which different materials were encountered in the cored boreholes.

Table 1: Summary of Material Strata Levels and Rock Classifications

Bore	Level of Top of Stratum (m, AHD) in Bores				
	Ground Level	Stiff Clay	EL rock	MS Shale	Base of Bore
1	19.6	19.0	17.1	12.2	9.6
2	19.7	18.6	18.4	14.7	8.7

EL = extremely low strength, MS = medium strength

The DCP encountered refusal at about 1 m depth (RL15.5 m) which is generally interpreted as rock. This is below the surface level of extremely low strength rock encountered in the bores rock, but above the level of medium strength shale.

The subsurface profile is similar to the bores drilled for the PNR building in 2005 taking into account the area of cut.

Free groundwater was not observed during augering and the use of drilling fluid prevented groundwater observations during rotary wash-boring and coring.

5. Laboratory Testing

Selected samples from the better quality rock core were tested for axial point load strength index (Is_{50}).

The results ranged from 0.3 MPa to 2.6 MPa which correspond to low to high strength rock, respectively. These I_{s50} results suggest an unconfined compressive strength (UCS) in excess of 40 MPa for some of the high strength rock encountered during the investigation.

Two samples were tested for aggressivity and the results are summarised in Table 2.

Table 2: Summary of Acidity and Aggressivity

Bore	Depth (m)	Description	pH	Resistivity ($\mu\text{S}/\text{cm}$)	Cl (ppm)	SO ₄ (ppm)
1	1.45-1.5 m	clay	5.0	120	85	62
2	0.4-0.5 m	filling	8.1	120	<10	10

Notes: Where: Cl = Chloride SO₄ = Sulphate

These laboratory test results indicate that the soils tested are non-aggressive to steel and mild to concrete when assessed in accordance with AS 2159:1995 "Piling: Design and Installation", Tables 6.1 and 6.2.

6. Proposed Development

The project involves the demolition of the existing Electrical Engineering building and the construction of a ten level teaching facility building with an equivalent two basement levels under part of the building envelope. The lowest ground level is given as RL 12.5 m which is approximately 7 m below existing ground level.

The geotechnical issues considered relevant to the proposed development include excavation, excavation support, groundwater, foundations and earthquake provisions.

7. Comments

7.1 Excavation

Based on a lowest floor level of RL 12.5 m, excavation depths are expected to be of the order of 7 m. Following demolition of the existing building, excavation for the basement levels is expected to be required in filling, soil and rock of varying strength including high strength shale. Excavation in filling, soil and some of the lower strength rock should be readily achievable using conventional earthmoving equipment such as hydraulic excavators with bucket attachments.

Excavation in stronger rock will probably require the use of ripping equipment, rock hammers or rock saws.

The use of rock hammers will cause vibration which, if not controlled, could possibly result in damage to nearby structures and disturbance to occupants. Normally, it is suggested that vibrations be provisionally limited to a peak particle velocity (PPV) of 8 mm/s at the foundation level of the buildings

to protect the architectural features of the building and to reduce discomfort for the occupants. The contractor may have to select his equipment carefully so that the equipment used on site to remove high strength rock does not cause excessive vibrations. Generally, a vibration trial and monitoring during construction are carried out to control the vibration levels.

Given the presence of some weak zones and jointing within the recovered rock cores near the rock surface, it is considered essential that the rock faces are inspected every 1.5 m drop. The inspection by a geotechnical consultant during the excavation should aim to identify any zones or areas that require treatment.

7.2 Excavation Support

7.2.1 General

The overburden and the extremely to low strength rock will need to be temporarily supported until the permanent basement walls are constructed. Shoring support will therefore be required from the ground surface down to at least the top of consistent medium strength rock. Based on the bores, shoring may be required to depths ranging from about 7 m on the western side and 3.5 m on the eastern side. The depth of support required on the site probably makes battering impractical and therefore shoring will probably be required.

Soldier piles with reinforced shotcrete panel infills are commonly used to support excavations. The soldier piles would generally be spaced at about 2 m to 2.5 m centres and should be socketed into medium or high strength laminite below bulk excavation level. Shotcreting should be undertaken in maximum 1.5 m or 2.0 m 'drops' as excavation proceeds in order to reduce the risk of local slippages and collapse between soldier piles. Temporary ground anchors will be required to prevent excessive lateral deformation of shoring/retaining walls and also to support the toe of the pile if piles are not taken below bulk excavation depth. For the permanent situation, the basement structure usually provides the required lateral support to the perimeter excavation once the temporary anchors are de-stressed.

In Bore 2, there is some high strength rock and the piling rig used to install the shoring piles must be capable of drilling through the high strength rock.

7.2.2 Design

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of high strength rock. Table 3 outlines material and strength parameters that may be used for the design of excavation support structures.

It is probable that shoring of the basement excavation will need to incorporate more than one row of anchors, especially if the soldier piles are not taken to the full depth of the excavation. The lateral pressure distribution on a multi-anchored or braced wall is complex and for preliminary design purposes a uniform distribution with depth (i.e. rectangular) of $4 H$ (kPa) could be assumed where H is the height of retained material in metres.

Table 3: Typical Material and Strength Parameters for Excavation Support Structures

Material	Bulk Density (kN/m ³)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)
Filling	18	0.4	0.6
Clay	20	0.3	0.5
Extremely/very low strength rock	22	0.2	0.3
Medium strength rock	22	0	0

Lateral pressures due to surcharge loads from sloping ground surface, the nearby road, and construction machinery should be included where relevant. Hydrostatic pressure acting on the shoring walls should also be included in the design where adequate drainage is not provided behind the full height of the wall.

The pressures given in Table 3 should incorporate a suitable factor of safety to limit deflection. For rocks, jointing may be a controlling factor and should be considered.

Regular rock-face inspections will be required during excavation to determine whether any potentially unstable rock wedges are present requiring permanent support. This is a possibility as there are a number of steep joints logged in the rock cores. Additional anchors/bolts may be required if large wedges are observed during excavation.

7.2.3 Ground Anchors

Where necessary, the use of declined tie-back (ground) anchors is suggested for the lateral restraint of the perimeter pile walls. Such ground anchors should be declined below the horizontal to allow anchorage into the stronger bedrock materials at depth. The design of temporary ground anchors for the support of pile wall systems (and potentially unstable rock wedges) may be carried out using the typical average bond stresses at the grout-rock interface given in Table 4.

Table 4: Typical Allowable Bond Stresses for Anchor Design

Material Description	Allowable Bond Stress (kPa)	Ultimate Bond Stress (kPa)
Extremely low to low strength rock	50 - 150	75 - 300
Medium strength Shale	500	1000
High Strength Shale	1500	3000

Ground anchors should be designed to have an appropriate free length (minimum of 3 m) and have a minimum 3 m bond length. After installation they should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes.

The parameters given in Table 4 assume that the anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical specialist may allow increased bond stresses to be adopted during construction, subject to testing.

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

Care should be taken to avoid damaging buried services, pipes and subsurface structures during anchor installation.

7.3 Groundwater

It is anticipated that the groundwater ingress into the excavation will occur as seepage along the soil-rock interface and through joints and bedding planes in the rock, especially after wet weather. Based on similar sites within the university, it is anticipated that any seepage can probably be controlled using sump and pumps during the construction phase. Grouting of open joints and partings may be necessary if excessive water ingress is an issue during excavation. Pumps may also be needed to remove seepage from bored pile excavations prior to the placement of concrete, if bored piles are used for shoring support.

The NSW Office of Water guidelines for development are *“Groundwater shall not be pumped or extracted for any purpose other than temporary construction dewatering at the site identified in the development application.”* Therefore, in the long term, the basement will have to be tanked to comply with the guidelines. For design purposes, it is suggested that the basement is tanked to 0.5 m above the surface level of rock.

7.4 Foundations

Based on the borehole logs, it appears that the proposed bulk excavation level (BEL) of RL 12 m will expose at least medium strength shale overlying medium and high strength sandstone or siltstone.

At the base of the bulk excavation works, spread footings (i.e. pad or strip footings) should be suitable for supporting the proposed building loads. Outside the basement area, piles founded on the medium or higher strength rock are suggested to provide uniform founding conditions.

Due to the anticipated variation across the site, an initial allowable bearing pressure of 3 500 kPa is suggested for design purposes. Local variations in rock levels and rock type may occur across the site. Inspection of all foundations by an experienced geotechnical consultant is recommended as there may be extensively weathered or jointed zones where reduced pressures are advisable. Higher bearing pressures (6 000 kPa or more) are likely to be achievable on high strength rock subject to inspection and appropriate testing.

For piles, the design load capacity can be increased by adopting an allowable skin friction of 350 kPa in the medium strength shale.

The settlement of a footing is dependent on the loads applied to the footing and the foundation conditions below the footing. The total settlement of a spread footing designed using the parameters provided in this report should be less than 1% of the footing width upon application of the design load. Differential settlements between adjacent footings may be in the order of 50% of the value of total settlement.

All footings should be inspected by an experienced geotechnical professional to check the adequacy of the foundation material and proof drilled or spoon tested as appropriate.

7.5 Seismic Design

In accordance with the Earthquake Loading Standard, AS1170.4, 2007, the site has a hazard factor (z) of 0.08 and a site sub-soil class of rock (Be). Even though the structure will be founded on medium or high strength rock, consideration has to be taken of the sides of the excavation, hence the classification of Be.

7.6 Further Investigation

The current investigation is based on bores drilled outside the existing and proposed building envelopes because access was not otherwise available. When access is available, it would be prudent to carry out further investigation inside the proposed building envelope to confirm the current investigation findings and to enable excavation contractors to confidently price the work without having to allow a high risk contingency.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at Electrical Engineering Building in accordance with DP's proposal dated 18 August 2016 and acceptance received from The University of Sydney dated 25 August 2016. The work was carried out under the The University of Sydney Panel Conditions of Engagement. This report is provided for the exclusive use of The University of Sydney for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or 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 notes should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

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

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. 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.

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 / environmental / 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

About this Report

Douglas Partners



Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;

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

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

Reports

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

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

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

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

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

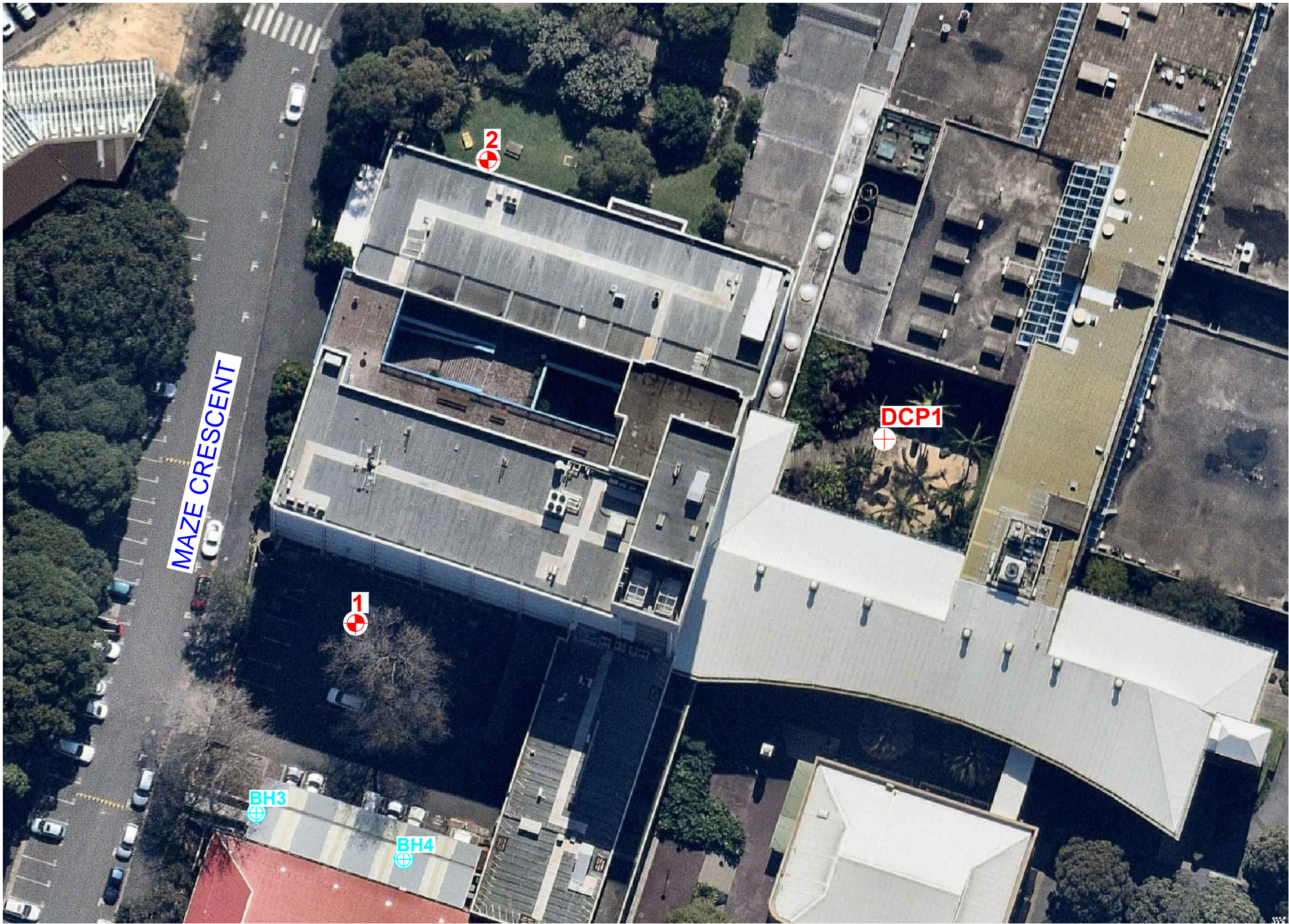
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

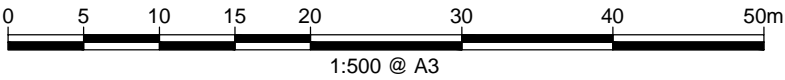
Appendix B

Plan



Locality Plan

NOTE:
1: Base image from Nearmap.com
2: Test locations are approximate only and were located using hand held GPS.



LEGEND

- Previous test bore (DP project 72278.03, 2005)
- Test bore location
- Dynamic cone penetrometer test location



Appendix C

Field Work Results

Results of Dynamic Penetrometer Tests

Client The University of Sydney
Project Building J03
Location Engineering Faculty

Project No. 85658.00
Date 30/09/16
Page No. 1 of 1

Test Location	1	2	3	4	5	6	7	8	9	10
RL of Test (AHD)	16.50									
Depth (m)	Penetration Resistance Blows/150 mm									
0 - 0.15	5									
0.15 - 0.30	8									
0.30 - 0.45	6									
0.45 - 0.60	6									
0.60 - 0.75	11									
0.75 - 0.90	25									
0.90 - 1.05	25/50									
1.05 - 1.20	B									
1.20 - 1.35										
1.35 - 1.50										
1.50 - 1.65										
1.65 - 1.80										
1.80 - 1.95										
1.95 - 2.10										
2.10 - 2.25										
2.25 - 2.40										
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										
3.00 - 3.15										
3.15 - 3.30										
3.30 - 3.45										
3.45 - 3.60										

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

Tested By
Checked By

Remarks Ref = Refusal, 24/110 indicates 25 blows for 110 mm penetration

Appendix D

Laboratory Results

Misc Inorg - Soil			
Our Reference:	UNITS	155366-3	155366-6
Your Reference	-----	BH1	BH2
	-		
Depth	-----	1.45-1.5	0.4-0.5
Date Sampled		5/10/2016	30/09/2016
Type of sample		SOIL	SOIL
Date prepared	-	18/10/2016	18/10/2016
Date analysed	-	18/10/2016	18/10/2016
pH 1:5 soil:water	pH Units	5.0	8.1
Electrical Conductivity 1:5 soil:water	µS/cm	120	120
Chloride, Cl 1:5 soil:water	mg/kg	85	<10
Sulphate, SO4 1:5 soil:water	mg/kg	62	10