REPORT

TO
CAPITAL INSIGHT PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED NEW BUILDING AND REFURBISHMENT OF BUILDINGS W6A AND W6B

AT
MACQUARIE UNIVERSITY, NORTH RYDE, NSW

6 June 2017
Ref: 29807ZRrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5003
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as JK Geotechnics ABN 17 003 550 801

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Paul Roberts

Report prepared by:

Paul Roberts
Senior Associate I Engineering Geologist

Report reviewed by:

Agi Zenon
Principal I Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed refurbishment of the Macquarie University buildings W6A and W6B and the proposed new building to be constructed to the south of W6A. The location of the site within the Macquarie University campus is indicated on the attached Figure 1. The investigation was commissioned by Kevin Christesen of Woolacotts Consulting Engineers (Woolacotts) on behalf of Capital Insight Pty Ltd by signed 'Acceptance of Proposal' formed dated 4 October 2016. The commission was on the basis of our fee proposal (Ref. P43348ZR) dated 19 September 2016.

We have been provided with the following information:

- Architectural plans (Drawing Numbers SSDA-01 to 16 Issue 1, dated 18 May 2017) prepared by Budden Nangle Micheal & Hudson Architects.
- Architectural plans (SK-MU3 and SK-MU4, dated 1 September 2016) annotated by Woolacotts Consulting Engineers Pty Ltd (Woolacotts) and attached to their 'Geotechnical and Environmental Investigation Brief (Ref. 16-058).
- Survey plan (Project No. 30431, Job Ref. 34174, dated 13 July 2011) prepared by Lockley Land Title Solutions.
- Original structural drawings for the Arts and Social Sciences Building, now Buildings W6A & W6B (Drawing Numbers 1E, dated 10/8/66, 2E, dated 15/8/66, 3E, dated 25/7/66, 4E & 5E, dated 20/7/66, 6E, dated 29/7/66, 7E, dated 9/6/66, 8E, dated 16/6/66, 9E, dated 3/8/66, 10E, dated 29/9/66 and 21E/A, dated 21/2/67) prepared by Woolacott, Hale, Bond & Corlett.
- Structural drawings for stair access to Building W6A (Drawing Numbers S1 and S2, dated 5/8/87) prepared by Randall Jones & Associates Pty Ltd.
- Rail Corridor Impact Study drawing (Drawing Number SK0010 Rev. A, dated 28 April 2017) attached to a letter dated 28 April 2017, both prepared by Taylor Lauder Bersten Pty Ltd (TLB).

Based on a review of the provided information, we understand that the proposed works will include:

- Reconfiguration of Building W6A, including nine storey plant room extensions to the eastern and western ends of the building and a new nine storey central core that will accommodate the relocation of the lifts and stairs. Maximum vertical loads are expected to be 2,500kN (in compression) with tension loads of between 200kN and 700kN over the eastern and western ends of the building and the new lift core. New pile footings will be required, together with additional loads transferred to existing footings. Rock bolts are envisaged to be required to support tension loads acting on selected walls.

- Reconfiguration of Building W6B (comprising three buildings, each of three levels) and relocation of selected columns. Maximum vertical loads are expected to be 1,100kN (in compression) with tension loads of 150kN below new walls. New pile footings will be required. Rock bolts are envisaged to be required to support tension loads acting on selected walls.
- A new museum building to the south of W6A. The floor level of the new building will be at the same level as Building W6A (RL64.878m). Excavations to a maximum depth of about 5m will therefore be required to achieve design subgrade levels. A bridge link will be provided to connect the new Building to Building W6A. No structural loads have been provided for the new building and typical loads for this type of development have been assumed.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on demolition, excavation, retention, new pile footings, existing footings, relevant methods to support tension loads underneath the new walls and braces, on-grade floor slabs, earthquake sub soil class and soil aggression.

Environmental Investigation Services (EIS), our specialist environmental division have also completed a Preliminary Stage 1 Environmental Site Assessment report (Ref. E29807KRrpt rev1, dated 6 June 2017) and a Waste Classification Assessment report for the soil berm to the south of Buildings W6A and W6B (Ref. E29807KR2let-WC) dated 10 May 2017. The EIS reports should be read in conjunction with this report.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 27, 28 and 30 September 2016, and comprised:

- Where access was restricted, two boreholes (BH1 and BH2) were hand auger drilled to a refusal depth of 1.5m. The boreholes were then extended by wash boring and TT56 diamond coring techniques (with water flush), using our portable Melvelle drilling equipment, to final depths of 7.3m and 7.5m, respectively.
- Where access was unrestricted, nine boreholes (BH3 to BH11) were auger drilled to depths ranging from 4.27m (BH6) to 7.5m (BH5) using our truck mounted JK205, JK305 and JK308 drilling rigs. BH4, BH6, BH7 and BH9 were then extended by diamond core drilling using NMLC coring techniques (with water flush) to final depths of 7.42m, 7.3m, 8.6m and 8.78m, respectively.
- Where access was restricted, two environmental boreholes (BH12 and BH13) were hand auger drilled to 1m depth.

Prior to the commencement of the fieldwork, the test locations were electromagnetically scanned for buried services by a specialist subcontractor.

The test locations, as indicated on the attached Figure 2, were set out by taped measurements from existing site features. The approximate surface RLs at the test locations were interpolated between spot heights and contours indicated on the provided survey plan. The survey datum is the Australian Height Datum (AHD).

The state of compaction of the fill and the strength of the residual silty clay soils were assessed from the Dynamic Cone Penetrometer (DCP) test results and Standard Penetration Test (SPT) ‘N’ values, which were augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube. The strength of the augered bedrock profile was assessed from observation of drilling resistance when using a tungsten carbide (‘TC’) bit, examination of the recovered rock cuttings and subsequent correlation with laboratory moisture content test results. The strength of the cored portions of weathered bedrock was assessed by examination of the recovered rock core and correlation with subsequent Point Load Strength Index tests.

Groundwater observation were made in the boreholes during, and on completion of, auger and core drilling. Standpipes were installed in BH1, BH2, BH4, BH6, BH7 and BH9 to the following depths:

- BH1: 3m depth, with a response zone in the soil and bedrock from 0.4m to 7.3m depth.
- BH2: 3m depth, with a response zone in the soil and bedrock from 0.6m to 7.5m depth.
- BH4: 7.42m depth, with a response zone in the soil and bedrock from 1.5m to 7.42m depth.
- BH6: 7.3m depth, with a response zone in the sandstone bedrock from 2.8m to 7.3m depth.
- BH7: 8.6m depth, with a response zone in the sandstone bedrock from 3.9m to 8.6m depth.
- BH9: 8.78m depth, with a response zone in the sandstone bedrock from 1m to 8.78m depth.

The water in the standpipes was pumped out on completion of the fieldwork (i.e. purged). We returned to site on 6 October 2016 to record the standing water levels in the standpipes and obtain groundwater samples. No further longer term groundwater monitoring has been carried out.

The fieldwork was carried out under the direction of our geotechnical engineers (Tristan Piat, Arthur Kourtesis and Andrew Frost) who were present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, nominated in-situ testing and sampling and directed installation of the standpipes. The borehole logs (which include field test results and groundwater observations) and the DCP test results are attached, together with a glossary of logging terms and symbols used.

Selected soil and rock chip samples were returned to the Soil Test Services Pty Ltd (STS) NATA registered laboratory for moisture content, Atterberg Limit and linear shrinkage testing. The results are summarised in the attached STS Table A. The recovered rock core was also returned to STS where it was photographed and Point Load Strength Index tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are presented in the attached STS Table B.

Selected soil and water samples were also submitted under chain of custody to an alternate NATA registered laboratory (EnviroLab Services Pty Ltd) for pH, chloride and sulfate content testing. The test results are presented in the attached Appendix A.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently undulating topography on a hillside that sloped down to the north-west at about 6°. The site is situated over the central western section of the Macquarie University campus (see Figure 1).

At the time of fieldwork the site was occupied by Buildings W6A and W6B, with respective northern and eastern frontages onto the concrete paved, tree lined Wally's Walk and the asphaltic concrete (AC) paved Western Road (see Figures 1 and 2). The roads sloped down to the north at between about 2^o and 6^o. To the north of the western end of Wally's Walk a grass surfaced area sloped down to the north-west at between about 3° and 5°

Building W6A comprised a nine storey building and W6B comprised three buildings (each three storey) arranged in a 'horseshoe' shape in plan. The buildings were concrete frame and brick masonry construction with gravel surfaced courtyard surrounds containing stepped concrete paved walkways. A number of garden beds were also present within the surrounds to Buildings W6A and W6B and were supported by timber, brick and concrete retaining walls ranging between about 0.3m and 1.2m height. Numerous medium to large sized trees were present within the landscaped surrounds.

A concrete pedestrian walkway bridged over Western Road from the north eastern corner of W6B eastwards to the nearby multi-level concrete frame and brick masonry Building W5B.

To the west of Building W6B and to the north of the western end of Building W6A, was a single level framed building with synthetic walls and roof.

An AC paved walkway lined the southern side of Building W6A and sloped down to the west at about 2°. Two grass surfaced fill mounds, orientated east-west and bisected by concrete paved walkway (orientated north-south) were located immediately to the south of the walkway. The fill mounds were a maximum of about 5m high (western mound) and 4.3m high (eastern mound), and had side slopes between about 14° and 18°. The toe of the northern side of the western fill mound was supported by a timber retaining wall about 0.5m high with a bicycle shed immediately to the north.

An AC surfaced car park was situated to the south of the fill mounds and sloped down to the north-west at about 6°. The car park extended west to a two level concrete framed car park and extended south to Macquarie Drive (see Figure 1).

Based on a cursory inspection from within the site, the buildings, structures and paved surfaces within and adjoining the site generally appeared to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone close to the interface with the overlying Ashfield Shale. The investigation disclosed a generalised subsurface profile comprising fill over residual silty clay with weathered sandstone bedrock encountered at shallow to moderate depth. Groundwater was intermittently encountered at depth within the bedrock. Reference should be made to the attached borehole logs for detailed subsurface conditions at the specific locations. A graphical borehole summary over the footprint of the proposed museum building is presented in Figure 3. A summary of the pertinent subsurface conditions is presented below:

Paved Surfaces

An AC paved surface was encountered in BH4 to BH8 and the thickness ranged from 25mm (BH7) to 100mm (BH8). The AC surface was underlain by clayey gravel (BH4), gravelly sand (BH5 and BH8), or sandy gravel (BH6 and BH7) within thicknesses of 565mm (BH4), 150mm (BH5), 230mm

(BH7) and 300mm (BH8). The sands and gravels have been interpreted to represent road base materials.

Concrete paved surfaces were encountered in BH9, BH10 and BH11 with respective thicknesses of 160mm, 105mm and 100mm. In BH9 and BH10 the concrete contained reinforcement and was respectively underlain by gravelly sand (240mm thick) and sandy gravel (295mm thick) which has been interpreted to represent road base materials.

Fill

Silty clay fill with varying gravel content was encountered from surface level in BH1, BH2, BH12 and BH13. Sandy gravel fill was encountered from surface level in BH3. In BH1, BH2 and BH3, the fill extended to depths of 0.5m, 0.4m and 0.2m, respectively. BH12 and BH13 were terminated within the fill at 1m depth. Based on the DCP test results, the fill in BH1 and BH2 was assessed to be moderately to poorly compacted.

Residual Clay

Residual silty (occasionally sandy) clays were encountered below the fill or road base materials in all boreholes except BH12 and BH13. The silty clays were assessed to be of medium to high plasticity and the sandy clays were assessed to be of low to medium plasticity. The clays were typically very stiff or hard strength.

Weathered Sandstone Bedrock

Sandstone bedrock was encountered below the residual clays in all boreholes except BH12 and BH13. In BH9, a 1.95m band of interbedded sandstone and shale was encountered at 5.6m depth.

The bedrock was encountered at depths of 3.26m (BH1), 4.28m (BH2), 2m (BH3), 2.5m (BH4), 1.4m (BH5), 2.4m (BH6), 2.4m (BH7), 0.8m (BH8), 0.95m (BH9), 3.3m (BH10) and 0.8m (BH11). The bedrock surface appears to be stepping down to the north and south from BH6 (RL63.9m) and BH7 (RL65.3m) to the centre of the site (RL61.1m in BH3) and down towards the west (RL60.9m in BH1) and north-west (RL58.6m in BH10).

On first contact, the sandstone was assessed to be extremely to distinctly weathered and of extremely low to very low strength (occasionally low to medium) strength. The sandstone generally improved with depth to distinctly weathered and of variable strength; typically low to medium strength but occasionally very low to low or high strength. The ‘TC’ bit refusal encountered in BH3 (5.2m depth) and BH11 (6.5m depth) have been interpreted to represent sandstone bedrock of at least high strength.

In the cored sections of the boreholes, the following defects were recorded:

- Occasional sub-horizontal bedding partings dipping at between 0^{o} and 15^{o}.
- A number of sub-horizontal extremely weathered seams, clay seams, crushed seams and fractured bands ranging between about 2mm and 150mm thickness.
- A number of planar and undulating joints dipping at between approximately 22° and 90°.

The following core loss zones were also encountered:

- In BH1 at 1.61m depth: 0.88m thick, 3.15m depth: 0.11m thick, 4.66m depth: 0.59m thick and 6.1m depth: 0.08m thick.
- In BH2 at 1.8m depth: 0.31m thick, 3.03m depth: 0.61m thick, 4.48m depth: 0.46m thick, 5.21m depth: 0.32m thick and 6m depth: 0.18m thick, and
- In BH6 at 4.27m depth: 0.65m thick.

The core loss zones encountered in BH1 at 1.61m and in BH2 at depths of 1.8m and 3.03m, have been interpreted to represent residual silty clay and have been included in the depth ranges for the residual clays outlined above. The remaining core loss zones may be interpreted as representing clay seams, extremely weathered seams, fractured bands and/or crushed seams.

An indicative engineering classification of the bedrock (in accordance with Pells et al. 1998) has been carried out for the cored boreholes and is tabulated below:

	Approx. Surface RL(m) AHD	**Indicative Engineering Classification of Sandstone Bedrock Depths (m)**		
Borehole		**Class V (m)**	**Class IV (m)**	**Class III (m)**
1	64.2	3.26 - 6.18	6.18 - 7.3	-
2	64.0	4.28 – 6.18	6.18 – 7.5	-
3*	63.1	2.0 – 3.3	3.3-5.2	-
4**	64.2	2.5 – 3.3	3.3 – 6.2	6.20 – 7.42
5*	64.6	-	1.4 – 7.5	-
6**	64.6	2.4 - 4.92	4.92 - 7.30	-
7**	67.7	2.4 – 4.5 6.25 – 7.7	4.5 – 6.25 7.7 – 8.6	-
8*	63.3	0.8 – 5.4	5.4 – 7.0	-
9**	63.0	0.95 – 2.5 5.6 – 6.53	2.5 – 5.6 6.53 – 8.78	-
10*	61.9	3.3 – 4.3	4.3 - 5.2	-
11*	63.2	0.8 – 4.5	4.5 – 6.5	-

***: tentative classification based on augered borehole.**

****: Includes upper augered portion of borehole.**

Groundwater

Groundwater was encountered in the bedrock profile whilst auger drilling BH3, BH5 and BH8 at respective depths of 5.1m, 6.8m and 6m. On completion of auger drilling standing water levels were recorded at similar depths in these boreholes. A standing water level was recorded at 4m depth on completion of auger drilling BH6. The remaining auger drilled boreholes (and augered portions of boreholes) were ‘dry’ during, and on completion of, auger drilling.

In the cored boreholes, water flush levels were recorded on completion of coring at depths of 4.2m (BH4), 3m (BH6), 4m (BH7), and 3m (BH9). The water flush was pumped from the boreholes and water levels were recorded on 6 October 2016 at least one week following the initial purging when groundwater sampling was completed; standing water levels were recorded at depths of 4.2m (BH4), 3.4m (BH6), 4.2m (BH7), and 3.4m (BH9). We have interpreted these recorded standing water levels as representing groundwater levels within the bedrock associated with the hydrostatic head of groundwater within discrete water carrying defects.

Water flush returns of 100% were recorded whilst core drilling which indicates a relatively impermeable rock mass.

3.3 Laboratory Test Results

The Liquid Limit and Linear Shrinkage determinations confirmed our assessment of plasticity of the residual silty clays and indicated that they have a moderate to high potential for shrink/swell reactivity with changes in moisture content.

The results of the laboratory moisture content tests carried out on recovered rock cutting samples generally correlated well with our field assessment of bedrock strength.

The point load test results indicated that the rock within the cored portions of the boreholes was of low to very high strength with estimated Unconfined Compressive Strengths (UCS) ranging between 4MPa and 62MPa. However, the majority of the estimated UCS values ranged between 4MPa and 16MPa, i.e. low to medium strength. We note that the high and very high strength UCS values were associated with iron indurated sandstone bands within the sandstone.

A summary of the laboratory chemical test results is provided in the table below.

Location	Sample Depth (m)	Description	pH Units	Sulfate (mg/kg)	Chloride (mg/kg)
BH1	0.2 - 0.3	FILL: Silty clay	7.2	10	<10
BH2	0.4 - 0.5	Silty CLAY	8.3	32	34
BH4	0.6 - 1.05	Silty CLAY	6.3	140	<10
	4.2 - 4.4	Water	5.6	94	260
BH5	6.8 - 7.1	Water	5.2	99	310
BH6	3.4 - 3.6	Water	4.6	44	89
	6.0 - 6.5	Water	5.2	170	27
BH7	1.5 - 1.95	Silty CLAY	5.1	24	<10
	4.21-4.40	Water	5.2	170	27
BH8	0.5-0.75	Silty CLAY	5.1	51	<10
	6.0 - 6.5	Water	5.4	150	240
BH9	3.35 - 3.55	Water	5.2	110	31
BH10	0.5 - 0.95	Silty CLAY	5.4	62	<10
BH11	3.9 - 4.5	Water	5.0	100	160

4 COMMENTS AND RECOMMENDATIONS

4.1 Demolition and Excavation

4.1.1 General

To maintain the stability of the existing sections of Buildings W6A and W6B, careful demolition of selected sections of the buildings will be required. The structural engineer will need to determine the extent of temporary propping for the adjacent sections of existing buildings that will remain.

Excavation recommendations provided below should be complemented by reference to the Safe Work Australia Code of Practice ‘Excavation Work’, dated July 2015.

Excavations to a maximum depth of about 5m will be required to achieve design subgrade levels for the proposed new museum building to the south of W6A. The excavations will extend close to adjacent paved surfaces and will require removal of the existing fill mounds.

Following demolition of paved surfaces and any sections of existing landscape retaining walls, removal of trees and stripping of topsoil and root affected soils, the proposed excavations will encounter the soil profile and penetrate weathered sandstone bedrock.

We expect the excavations to be readily completed using tracked excavators. A bucket attachment to the excavator will be required to excavate the soil profile and extremely weathered bedrock. Any topsoil or root affected soils may be separately stockpiled for re-use in landscape areas as they are not suitable for reuse as engineered fill. In addition, rock breakers (attached to the excavator) may be required for demolition of existing concrete paved surfaces. Prior to commencement of the removal of paved surfaces, we recommend that a saw cut be provided where the paved surfaces abut paved surfaces outside the work area. This will assist in controlling potential damage associated with removal of the adjacent paved surfaces.

Excavation of low or higher strength sandstone bedrock may be achieved using rock breakers, rock grinders and ripping attachments to the tracked excavator. We expect that small to medium sized rock breaker attachments will be used. A medium to large sized dozer may also be used.

Care will be required to control ground vibrations associated with the use of rock breakers, such as the provision of rock saw cuts (see Section 4.1.2, below). Rock saws may also be used to create 'smooth' finishes on cut faces and aid in detailed excavation of footings, services trenches etc. Where rock breakers, rock saws and/or rock grinders are used, the resulting dust should be suppressed with water.

Tree root systems dry out the surrounding clayey soils and their removal will result in localised moisture recovery leading to swelling which may have a detrimental impact on the performance of nearby paved surfaces supported in the clayey soil profile. Therefore, trees should only be removed where absolutely necessary and as soon as practicable in order for the moisture content of the clayey subsoils to recover.

4.1.2 Potential Vibration and Ground Surface Movement Risks

The poorly compacted fill encountered in the investigation may extend beyond the work area of the proposed new museum building, and we therefore advise that sudden stop/start movements of tracked equipment should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of paved surfaces (and any buried services below) that will remain.

Care should be taken where rock breakers are used during demolition and for excavation of sandstone bedrock so that ground vibrations do not adversely affect nearby buildings, structures and paved surfaces. If there is any cause for concern then demolition and/or excavation should cease and further geotechnical advice sought.

While the rock breakers are being used to excavate bedrock, consideration should be given to continuous vibration monitoring of the adjacent university buildings to the east, west and north, to confirm that peak particle velocities (PPV) fall within acceptable limits. Subject to the results of the dilapidation reports (see Section 4.1.3, below), we would recommend that the PPV along the adjacent sides of nearby university buildings do not exceed 20mm/sec during bedrock excavation using rock breakers. Should higher vibrations be measured they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the vibration frequency. We note that this vibration limit will reduce the risk of vibration damage to the adjacent buildings and structures. However, these vibrations may still result in discomfort to occupants of the adjacent buildings. If excessive vibrations are confirmed, it will be necessary to use lower energy equipment such as smaller rock breakers and/or use rock saw cuts with the base of the slot maintained below the level at which the rock breaker is being used.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, and be operated one at a time and in short bursts only to reduce amplification of vibrations.

4.1.3 Dilapidation Surveys

If there are concerns regarding potential damage to adjacent university buildings, then consideration will need to be given to compiling detailed dilapidation reports prior to demolition and excavation commencing. The reports should be carefully reviewed to confirm that they present a fair record of existing conditions as the reports may assist the client should damage occur to any adjacent sections of university buildings.

4.1.4 Seepage

Seepage inflow may be expected within the excavations, particularly after periods of heavy rain within the soil profile close to, or at, the contact with the underlying bedrock. In addition, concentrated flows along defects within the rock mass may also be encountered. In general, we expect the inflows to be ephemeral, of small volume and managed by conventional sump and pump techniques.

Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed can be incorporated into the drainage design.

4.2 Temporary Batter Slopes and Retention

4.2.1 Temporary Batters and Retention

Temporary batter slopes through the clayey soil profile and extremely weathered sandstone bedrock no steeper than 1 Vertical (V) in 1 Horizontal (H) are considered to be appropriate. We expect that the above batter slopes will be achievable within the site geometry.

If battering is not preferred, a full depth engineered retention system installed prior to excavation commencing would be required. A soldier piled wall with shotcrete panels or a contiguous piled wall may be adopted, with permanent lateral support provided by the proposed structure

Over the deeper sections of the excavation, to reduce excavation induced ground movements, the retention system will need to be progressively anchored as excavation proceeds. Excavations must not extend beyond the nominated anchor point until the anchors have been installed, stressed and tested. However, where excavation depths are limited, i.e. around the toe areas of the existing fill mounds, we would expect the shoring piles to be designed as a cantilevered system with permanent lateral support provided by the proposed structure.

Bored piles would be suited to the site. Shoring piles should be embedded sufficient depth below bulk excavation level to satisfy stability considerations. Allowance will also need to be made for localised excavation below bulk excavation level such as for footings, lift pits etc. Piles extending below bulk excavation level (and penetrating bedrock) may also be incorporated into the footing system.

If soldier pile walls are proposed then their construction must be of high quality. The shotcrete infill panels must be completed without delay to reduce the shrinkage of clay soils immediately outside the excavation.

4.2.2 Sandstone Cut Face Stability

The proposed excavations will encounter sandstone bedrock. Class IV (or better) sandstone bedrock may be temporarily cut vertically, subject to geotechnical inspection. Geotechnical inspections should be completed by an experienced geotechnical engineer or engineering geologist at regular intervals of no more than 1.5m vertical excavation ‘lifts’.

The presence of potentially unstable wedges, clay seams and extremely weathered seams of sandstone and/or shale within the sandstone bedrock may adversely affect the stability of the cut faces and/or footings located close to the crests of cut faces. Such features may require shotcreting and rock bolting. Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

For conventional retaining walls founded at the crests of excavation faces, lateral restraint may be provided by starter bars drilled and grouted to a depth of at least 0.5m into the sandstone bedrock. The starter bars should be installed at a downward angle into the rock face and be provided with a vertical cogged length. Where cross bedded units within the sandstone bedrock are identified during geotechnical inspections and slope down into the excavation, then the starter bars may have to be extended to stabilise the potentially unstable cross bedded units.

4.2.3 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters should be adopted for the static design of the retention system, conventional retaining walls propped by the structure and landscape retaining walls:

- For design of conventional walls and cantilever piled walls that will be supported by the structure, we recommend the use of a triangular lateral earth pressure distribution with an ‘at rest’ earth pressure coefficient (k_o) of 0.55 for the soil profile and Class V bedrock, assuming a horizontal backfill surface.
- For progressively anchored or propped walls, where minor movements may be tolerated, we recommend the use of a uniform rectangular earth pressure distribution of 6H kPa for the retained profile, where ‘H’ is the retained height in metres; see Figure 4. If there are any adjacent areas of the site which are highly sensitive to lateral movement, a uniform rectangular earth pressure distribution of 8H kPa for the retained profile should be adopted.
- Where some minor movements of cantilever retaining walls may be tolerated (e.g. landscape walls), they may be designed using a triangular lateral earth pressure distribution and a coefficient of ‘active’ earth pressure, (k_a), of 0.3 for the soil, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the retained profile.
- Any surcharge affecting the walls (e.g. due to traffic loads, adjacent footings, construction loads, etc) should be allowed in the design using the appropriate earth pressure coefficient from above.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a

non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.

- Piled walls must be designed as permanently drained. For contiguous piled walls PVC pipes should be installed at nominal 1.2m horizontal spacing just above the adjacent floor level and just above the bedrock surface. Holes will need to be drilled to allow installation of the pipes and/or use of gaps between contiguous piles. The end of the pipe penetrating the retained profile behind the wall must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. Shotcrete infill panels would need to be provided with strip drains. The pipes and strip drains should discharge into the perimeter drainage system.
- For piles embedded into the underlying bedrock below bulk excavation level, an allowable lateral toe resistance of 200kPa may be adopted for sockets embedded a minimum depth of 1m into Class V sandstone bedrock. Increased allowable lateral toe resistances of 500kPa may be adopted for sockets embedded a minimum depth of 1m into Class IV (or better) sandstone. This value assumes excavation is not carried out within the zone of influence of the wall toe and the rock does not contain unfavourable defects, etc. The upper 0.3m depth of rock below the adjacent excavation level should not be taken into account in the socket design to allow for disturbance and tolerance effects during excavation.
- Lateral restraint of landscape walls founded in the soil profile below adjacent surface levels may be provided by the passive pressure of the soil below these levels. A ‘passive’ earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections. Localised excavations in front of the walls e.g. for buried services etc should also be taken into account in the design.
- Ground anchors, any rock bolts that may be required and starter bars providing lateral restraint for retaining walls at the crests of rock cut faces should be designed for an allowable bond strength of 200kPa assuming they are installed into Class IV (or better) sandstone bedrock.
- Rock bolts and dowels should be ‘nipped’ tight. All ground anchors should be proof tested to 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor. We recommend that only experienced contractors be considered for the rock bolt and anchor installation. Permanent rock bolts or ground anchors will need to be designed with due regard for long term corrosion protection.

4.3 Footings

4.3.1 Overview

For Building W6A, maximum vertical loads are expected to be 2,500kN (in compression) with tension loads of between 200kN and 700kN over the eastern and western ends of the building and the new lift core. New pile footings will be required, together with additional loads transferred to existing footings.

For Building W6B, maximum vertical loads are expected to be 1,100kN (in compression) with tension loads of 150kN below new walls. Woolacotts have assessed the existing pile footings and consider that they are not capable of supporting additional loads; new pile footings will therefore be required.

For both Buildings W6A and W6B, rock bolts are envisaged to be required to support tension loads acting on selected walls.

A review of the provided structural drawings for the existing buildings indicates the following:

- Building W6A is currently supported on individual piles and pile groups. The bored pile diameter is 20” and the column loads range between about 122 tons and 95 tons; equivalent to approximate bearing pressures of 5.4MPa and 4.1MPa, respectively. Drawing Number 1E notes that the piles were to be ‘driven to rock’.
- Building W6B is currently supported on bored ‘bell’ piles and designed for maximum bearing pressures of 2 tons per square foot (for clay) or 5 tonnes per square foot (for shale); equivalent to approximate bearing pressures of 190kPa and 480kPa, respectively.

Based on the results of our investigation, and with regard to the above bearing pressures, we note the following:

- Building W6A would need to have been founded in Class III sandstone assuming the piles were designed as end bearing only. Alternatively, the piles were founded in Class IV sandstone and the pile sockets also supported a portion of the loads.
- Building W6B has probably been founded in a combination of the very stiff to hard residual clays and Class V sandstone.

For Building W6A, without knowing the lengths of the existing piles, it is difficult to be certain of the class of sandstone present at the bases of the piles and thereby assess their ability to support additional loads. Assuming the piles are founded in Class IV sandstone, then any additional load carrying capacity will need to be assessed in relation to the advice provided in Section 4.3.2, below. In this regard, we note that the additional loads are likely to be of the order of 10% of the original design load, resulting in a maximum estimated additional bearing pressure of 440kPa. For this additional bearing pressure, additional settlements of the order of 1mm to 2mm would be expected below existing pile footings. We assume that the additional settlements would be within serviceability limits but this should be confirmed by Woolacotts.

To provide more information on the existing piles, low strain integrity testing of the piles could be undertaken to determine the pile lengths and review the results in relation to the current boreholes. The sides of the piles below the pile caps would need to be exposed in order to complete the low strain pile integrity testing (PIT). Results can be affected by concrete strength (which would have to be assumed), potential interference from the pile cap (which can usually be reduced by the PIT software), and a poor contact surface with the pile (the contact surface will need to be appropriately prepared).

If piles considerably deeper than the current borehole depths are demonstrated by the PIT, then it is likely that they have been founded in Class III sandstone, although additional deeper cored boreholes would be required to confirm this assessment. Based on the current investigation, Class III sandstone was only encountered in BH4 below 6.2m depth, with Class III sandstone expected to be encountered below the termination depths of the remaining boreholes, i.e. depths ranging from 5.2m to 7.5m (based on BH1 to BH6 adjacent to Building W6A).

4.3.2 Footing Design, Construction and Testing

We reiterate the indicative engineering classification of the bedrock (in accordance with Pells et al. 1998) provided in Section 3.2 above:

Borehole	**Approx. Surface RL(m) AHD**	**Indicative Engineering Classification of Sandstone Bedrock Depths (m)**		
		Class V (m)	**Class IV (m)**	**Class III (m)**
1	64.2	3.26 - 6.18	6.18 - 7.3	-
2	64.0	4.28 – 6.18	6.18 – 7.5	-
3*****	63.1	2.0 – 3.3	3.3 - 5.2	-
4**	64.2	2.5 – 3.3	3.3 – 6.2	6.20 – 7.42
5*	64.6	-	1.4 – 7.5	-
6**	64.6	2.4 - 4.92	4.92 - 7.30	-
7**	67.7	2.4 – 4.5 6.25 – 7.7	4.5 – 6.25 7.7 – 8.6	-
8*	63.3	0.8 – 5.4	5.4 – 7.0	-
9**	63.0	0.95 – 2.5 5.6 – 6.53	2.5 – 5.6 6.53 – 8.78	-
10*	61.9	3.3 – 4.3	4.3 - 5.2	-
11*	63.2	0.8 – 4.5	4.5 – 6.5	-

***: tentative classification based on augered borehole.**

****: Includes upper augered portion of borehole.**

Based on the indicative engineering classification, we recommend the following allowable bearing pressures be adopted for design of footings and assessment of the additional load carrying capacity of existing footings:

- Class V sandstone: 1,000kPa
- Class IV sandstone: 3,500kPa
- Class III sandstone (or better): 6,0000kPa

The load carrying portions of pile rock sockets may be designed using values of 10% or 5% of the above allowable bearing pressures in compression and tension, respectively. The design of rock sockets must take into account the presence of localised excavations for service trenches, lift pits etc.

Bored piles are expected to be suitable for the site. For piles, a maximum allowable bearing pressure of 3,500kPa should be adopted for Class IV (or better) sandstone as there would be practical difficulties in confirming better class of sandstone (and therefore a higher bearing pressure) in a deep pile drill hole.

We expect that a mix of residual clayey soils and variable quality sandstone bedrock will be exposed at bulk excavation level over the footprint of the proposed museum building. Where exposed, the sandstone, is expected to be Class V and Class IV, with sandstone locally a maximum of about

1.5m below bulk excavation level. A combination of pad and strip footings and bored piles founded in sandstone bedrock are therefore expected to be required to support the building loads, together with shoring piles (if selected).

For footings founded at the crest of cut faces in the new museum building, on Class IV (or better) sandstone bedrock, an allowable end bearing pressure of 1,000kPa may be adopted. However, geotechnical inspection of the cut face below the base of the footings would need to be carried out to check for the presence of any adversely orientated joints, extremely weathered seams etc. Any such features will require stabilisation using permanent rock bolts or underpins.

To confirm our assumptions regarding sandstone quality, we recommend that a geotechnical engineer inspect the high level footing bases and witness the drilling of bored piles.

All footings should be excavated/bored, inspected and poured with minimal delay. All footings should be free from all loose or softened materials prior to pouring. If water ponds in the base of the footing excavations or bored pile drill holes then it should be pumped dry and then re-excavated/over-drilled to remove all loose and softened materials prior to pouring. If a delay in pouring high level footings is anticipated consideration should be given to provision of a blinding layer of concrete to protect the base of the footing excavations in any Class V sandstone bedrock.

We recommend that competent piling contactors be used. The piling contractor should be provided with a copy of this geotechnical report, in order that appropriate piling rigs and equipment are brought to site. We note that for new piles required for Buildings W6A and W6B, access in places will be restricted and specialist difficult access piling rigs may be required.

4.3.3 Tension Loads

Tension loads below walls and the lift core may be resisted by the load carrying portion of pile rock sockets using the allowable shaft adhesion values provided above.

Alternatively, permanent rock anchors or rock bolts may be used and should be installed in Class IV (or better) sandstone bedrock and designed for an allowable bond strength of 200kPa. Appropriate long term corrosion protection will need to be provided. The drilling of the rock bolt/anchor holes should be witnessed by a geotechnical engineer to confirm the quality of the sandstone. We also recommend that all rock anchors and bolts be load tested, as outlined in Section 4.2.3, above.

4.4 Piling Rig Working Platform

We note that the piling contractor may require a working platform. The assessment of a working platform thickness would need to be completed by a geotechnical engineer based on the methodology outlined in BRE 2004 *'Working Platforms for Tracked Plant'*. Where a working platform is required it may need to be formed using engineered fill comprising durable granular material (such as DGB20 or similar as approved by the geotechnical engineer). Where engineered fill is to be placed to raise site surface levels this may comprise the working platform fill provided it meets the specification requirements. This should be taken into account when selecting engineered fill.

If certification of the working platform is required, then a geotechnical engineer should visit site to confirm that the thickness of the working platform has been achieved and to review the density test results carried out on the working platform material. We may then be in a position to certify the working platform, provided the thickness and minimum density requirements have been met; this certification would be more readily achievable if Level 1 control of fill placement and compaction, in accordance with AS3798-2007, is adopted.

4.5 Earthworks

4.5.1 Subgrade Preparation

The earthworks recommendations provided below should be complemented by reference to AS3798-2007.

Prior to construction of the on-grade floor slabs the areas of soil subgrade should be prepared as follows:

- Proof roll the subgrade using at least 6 passes of a minimum 5 tonne dead weight smooth drum static (non-vibration) roller to achieve a compaction of at least 98% of Standard Maximum Dry Density (SMDD). The aim of the proof rolling is to improve near surface compaction and to identify any unstable subgrade areas.
- Proof rolling should be closely monitored by the site supervisor or an experienced geotechnical engineer to detect soft or unstable areas which should be locally excavated down to a stiff base and replaced with engineered fill (as outlined in Section 4.5.3, below). Care should be taken not to over compact clayey subgrade areas.
- Sections of clay subgrade that contain shrinkage cracks should be watered and rolled until the shrinkage cracks disappear.

- Care will need to be exercised close to nearby existing structures, the new structure and any buried services as ground borne vibrations caused by the proof rolling may cause damage. If there any causes for concern during proof rolling, then further advice should be sought and/or the non-vibration (static) mode of the roller used.

4.5.2 Subgrade Drainage

The proposed museum building subgrade will comprise a combination of residual clay soils, and Class V and IV sandstone. The clays and Class V sandstone may be found to be unstable if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill. Trafficability may be improved by the use of a sacrificial surface layer of demolition rubble.

4.5.3 Engineered Fill

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the excavated clay soils and the weathered bedrock sourced from the bulk excavations may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller to within 2% of Standard Optimum Moisture Content (SOMC) and to at least 98% of SMDD. We note that use of clayey materials for engineered fill requires careful control of moisture contents and will likely have a low soaked CBR value.

To confirm the above specification has been achieved, density tests should be completed at the frequencies defined in AS3798. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

Alternatively, imported well graded granular material (ripped or crushed sandstone or building rubble) free of deleterious substances and having a maximum particle size of 40mm may be used as engineered fill and also backfill behind retaining walls. Such fill should be compacted in horizontal layers to a minimum density of 98% SMDD using the above mentioned roller. However, close to the rear of retaining walls a hand held plate compactor will most likely need to be used. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls. We warn that effective compaction may not be achieved behind retaining walls and post construction settlements may occur.

We note that if single sized granular material (or ‘no fines’ gravel) is used as backfill to retaining walls then only nominal compaction (with no compaction testing) will be required and would also act as the behind wall drainage. The behind wall drainage should be wrapped by a non woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion. Further, retaining wall backfill should be provided with a clay plug at surface level to reduce the likelihood of stormwater surcharging the retaining wall.

4.6 Building Floor Slab

We forewarn that the clayey soils and Class V sandstone at the proposed design subgrade level for the proposed museum building may be susceptible to weathering and degradation following exposure to the elements and in particular rainfall periods. We therefore recommend that good and effective drainage be provided during construction. The principal aim of the drainage is to promote run-off towards designated sumps by cross-falls and to reduce ponding. Any softened material should be scraped off prior to floor slab (including sub-floor drainage) construction.

Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

The building floor slab should be provided with at least a 100mm thick subbase of good quality, durable, single size, crushed rock (free of ‘fines’) such as ‘blue metal’ gravel, which will also act as underfloor drainage. The subbase will also provide more uniform slab support and will reduce the “pumping” of “fines” at joints.

The building floor slab underfloor drainage should include a sump and pump dewatering system. The retaining wall drains should be connected into the underfloor drainage system. Groundwater seepage monitoring should be carried out during the excavation prior to finalising the design of the pump out facility. The sump(s) should have an automatic level control pump to avoid flooding of the building. Outlets into the stormwater system will require Council approval.

4.7 Soil Aggression

Based on the advice provided in Table 4.8.1 of AS3600-2009 “Concrete Structures” we note that the laboratory chemical test results have indicated that an A1 and A2 Exposure Classification applies. We recommend that an A2 Exposure Classification is adopted.

For concrete pile footings, based on the advice provided in AS2159-2009 “Piling Design and Installation”, a ‘Non-Aggressive’ and ‘Mild’ Exposure Classification would apply (based on Table 6.4.2(C) of AS2159). We recommend that a ‘Mild’ Exposure Classification is adopted.

4.8 Earthquake Design Parameters

Based on the results of the investigation, the following design parameters should be adopted for earthquake design in accordance with AS1170.4-2007 (“Structural Design Actions, Part 4: Earthquake Actions in Australia”):

- Hazard Factor (Z) = 0.08
- Site Subsoil Class = Class B_e

4.9 Interaction With The Epping Chatswood Rail Tunnel (ECRL)

Based on a review of the provided letter and drawing prepared by TLB information, we confirm that the new footings for the proposed reconfiguration works for Building W6B will lie within the ‘Second Reserve Within Zone Of Influence’ rail protection reserve. However, the reconfiguration works for Building W6A and the new museum building lie outside the ‘Second Reserve Within Zone Of Influence’ rail protection reserve.

Reference to the ECRL Underground Infrastructure Protection Guideline (Report No. 20007300/PO-4532 Rev. 3, dated 16 May 2008) prepared by Transport Infrastructure Development Corporation indicates that the crown of the tunnel is at about RL30m. The new high level footings for the proposed reconfiguration works for Building W6B will be founded in Class V sandstone bedrock which is expected to be encountered at less than 2m below existing surface levels.

Based on our experience of 2D and 3D finite element analyses of similar basement excavations and new buildings constructed over, and adjacent to, road and rail tunnels, we expect that there will be negligible additional loads impacting the tunnel associated with footings for the proposed reconfiguration works for Building W6B. In addition, based on the locations of Building W6A and the new museum building (requiring maximum 5m excavation) outside the rail protection reserves, we also consider that:

- There will be negligible vertical deflections impacting the tunnel associated with proposed excavation, or visa versa.
- There will be negligible additional loads impacting the tunnel associated with footings for either the proposed building reconfiguration or new building.

We note that some use of rock breakers to excavate bedrock may be required to achieve the proposed museum building design subgrade level. However, the maximum depth of excavation will be down to about RL64.7m, and the tunnel crown level is at about RL30m. Considering the off-set of the proposed new building from the tunnel (i.e. outside the Second Reserve Within Zone Of Influence' rail protection reserve), in our opinion, any vibrations associated with rock breakers excavating bedrock will have attenuated to imperceptible levels at the tunnel.

4.10 Further Geotechnical Input

Provided below is a summary of additional geotechnical input outlined in the preceding sections of this report:

- Piling rig working platform thickness design.
- Pile Integrity testing.
- Additional cored boreholes to assist in interpretation of pile integrity test results.
- Dilapidation surveys.
- Quantitative vibration monitoring.
- Monitoring groundwater seepage into the excavation.
- Proof-rolling inspections.
- Density testing of all engineered fill.
- Witnessing drilling of bored pile footings and shoring piles.
- High level footing inspections.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between and below the completed boreholes and DCP tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Ref No:**	29807ZR
Project:	Proposed Refurbishment of Building W6A and W6B and Construction of Additional New Building at Macquarie University	**Report:**	A
		Report Date:	14/10/2016
Location:	Buildings W6A and W6B, Macquarie Park, NSW	**Page 1 of 1**	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
3	3.50-4.00	11.4				
3	5.10-5.20	5.0				
4	2.70-3.00	9.9				
4	3.50-4.00	4.5				
5	2.50-3.00	8.4				
5	5.00-5.50	7.3				
5	7.00-7.50	6.7				
6	2.40-2.70	7.7				
6	3.80-4.10	7.7				
7	0.50-0.95	22.9	54	21	33	15.0
7	3.50-4.00	6.5				
7	5.00-5.50	7.3				
8	2.80-3.00	8.5				
8	6.00-6.30	4.0				
8	6.80-7.00	4.6				
9	5.20-5.60	6.3				
10	3.50-4.00	13.2				
10	4.80-5.20	8.0				
11	0.50-0.80	12.7	46	19	27	12.0
11	2.50-3.00	14.4				
11	4.80-5.30	6.8				
11	6.20-6.50	6.6				

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 5/10/2016

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Refurbishment of Building W6A and W6B and Construction of Additional New Building at Macquarie University
Location: Buildings W6A and W6B, Macquarie Park, NSW

Ref No: 29807ZR
Report: B
Report Date: 5/10/2016
Page 1 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	3.41-3.44	0.7	14
	3.94-3.96	0.2	4
	4.27-4.29	0.3	6
	5.28-5.32	0.2	4
	5.85-5.88	0.2	4
	6.28-6.31	0.3	6
	6.73-6.77	0.6	12
	7.13-7.16	0.6	12
2	5.07-5.10	1.9	38
	5.57-5.61	1.1	22
	6.34-6.38	0.6	12
	6.92-6.95	0.3	6
	7.22-7.26	0.5	10
4	4.53-4.57	0.4	8
	4.88-4.93	0.9	18
	5.51-5.55	0.5	10
	6.28-6.33	0.5	10
	6.88-6.93	0.8	16
	7.25-7.30	0.8	16
6	5.06-5.09	0.5	10
	5.87-5.90	0.6	12
	6.18-6.21	0.5	10
	7.18-7.22	0.5	10
7	5.77-5.82	0.2	4
	6.08-6.11	0.5	10

NOTES: See Page 2 of 2

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Refurbishment of Building W6A and W6B and Construction of Additional New Building at Macquarie University
Location: Buildings W6A and W6B, Macquarie Park, NSW

Ref No: 29807ZR
Report: B
Report Date: 5/10/2016
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
7	6.75-6.79	3.1	62
	7.28-7.32	0.3	6
	7.90-7.94	0.3	6
	8.17-8.21	0.5	10
9	6.61-6.65	1.8	36
	7.17-7.20	0.4	8
	7.63-7.67	1.2	24
	8.26-8.30	1.2	24
	8.70-8.72	0.7	14

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: HAND AUGER
R.L. Surface: ~64.2 m
Date: 28/9/16
Datum: AHD
Plant Type:
Logged/Checked By: A.C.K./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING AND 6/10/16	■ ■			■ ■	REFER TO DCP TEST RESULTS	64		▩		FILL: Silty clay, low to medium plasticity, dark grey brown mottled brown, trace of fine to medium grained ironstone gravel.	MC>PL			APPEARS MODERATELY TO POORLY COMPACTED
	■			■				▨	CL-CH	FILL: Silty clay, medium to high plasticity, orange brown and dark grey brown, trace of fine grained sandstone gravel and fine grained ironstone gravel.	MC>>PL	VSt	300 300 250	RESIDUAL HP TESTING ON REMOULDED AUGER SAMPLE
				■		63	1	▨		SILTY CLAY: medium to high plasticity, orange brown and light brown mottled light grey, trace of fine grained ironstone gravel and fine grained sand.	MC<PL	H		TOO FRIABLE FOR HP TESTING
						62 61 60 59 58	2 3 4 5 6			REFER TO CORED BOREHOLE LOG				WASH BORE CASING 1.10m TO 1.61m

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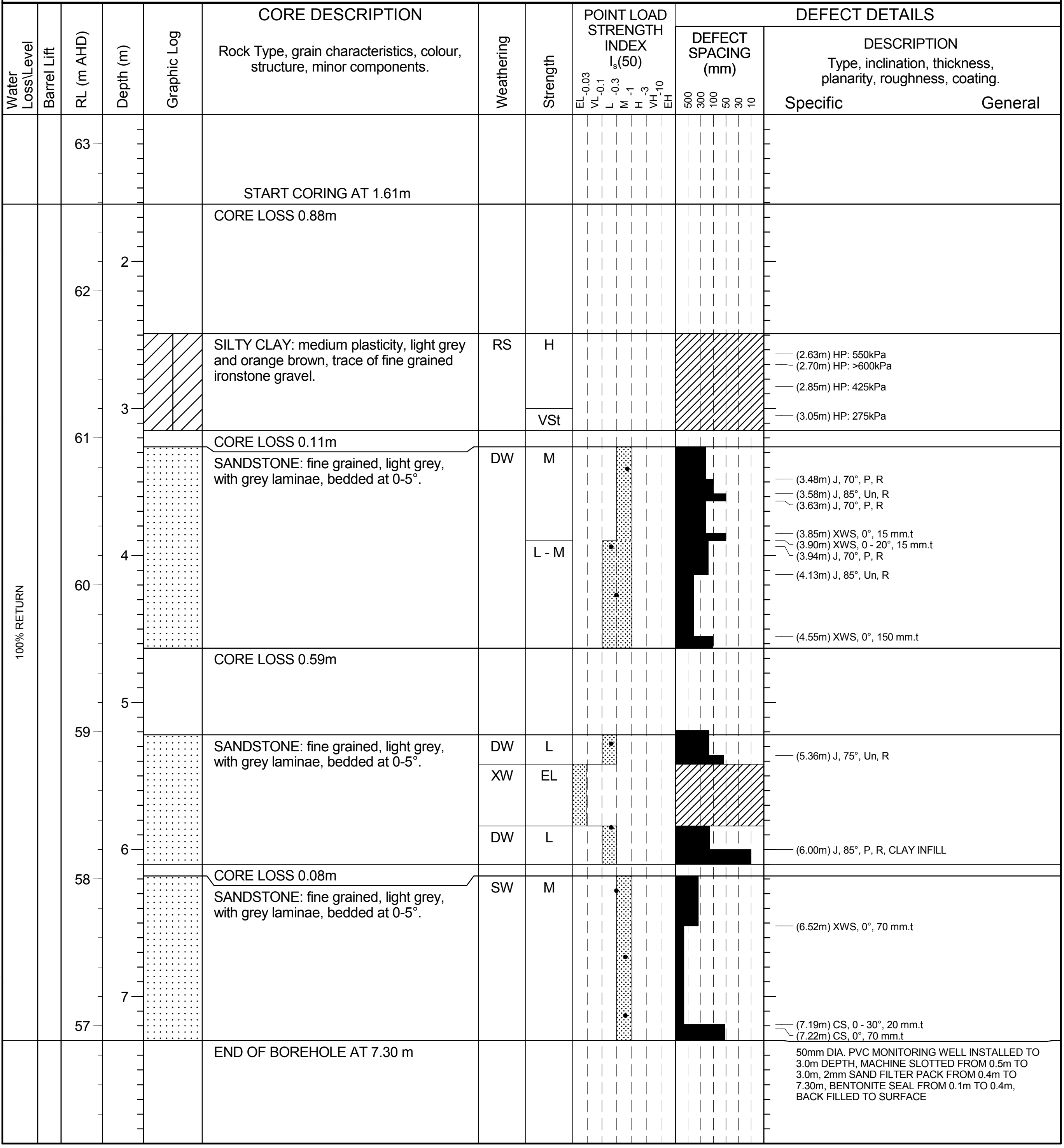
CORED BOREHOLE LOG

Borehole No.
1
2 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Core Size: TT56
R.L. Surface: ~64.2 m
Date: 28/9/16
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: A.C.K./P.R.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT SPACING (mm) 500, 300, 100, 50, 30, 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		63			START CORING AT 1.61m						
100% RETURN		62	2		CORE LOSS 0.88m						
			3		SILTY CLAY: medium plasticity, light grey and orange brown, trace of fine grained ironstone gravel.	RS	H VSt			(2.63m) HP: 550kPa (2.70m) HP: >600kPa (2.85m) HP: 425kPa (3.05m) HP: 275kPa	
		61			CORE LOSS 0.11m						
		60	4		SANDSTONE: fine grained, light grey, with grey laminae, bedded at 0-5°.	DW	M L - M			(3.48m) J, 70°, P, R (3.58m) J, 85°, Un, R (3.63m) J, 70°, P, R (3.85m) XWS, 0°, 15 mm.t (3.90m) XWS, 0 - 20°, 15 mm.t (3.94m) J, 70°, P, R (4.13m) J, 85°, Un, R (4.55m) XWS, 0°, 150 mm.t	
			5		CORE LOSS 0.59m						
		59			SANDSTONE: fine grained, light grey, with grey laminae, bedded at 0-5°.	DW XW DW	L EL L			(5.36m) J, 75°, Un, R (6.00m) J, 85°, P, R, CLAY INFILL	
		58	6		CORE LOSS 0.08m						
		57	7		SANDSTONE: fine grained, light grey, with grey laminae, bedded at 0-5°.	SW	M			(6.52m) XWS, 0°, 70 mm.t (7.19m) CS, 0 - 30°, 20 mm.t (7.22m) CS, 0°, 70 mm.t	
					END OF BOREHOLE AT 7.30 m					50mm DIA. PVC MONITORING WELL INSTALLED TO 3.0m DEPTH, MACHINE SLOTTED FROM 0.5m TO 3.0m, 2mm SAND FILTER PACK FROM 0.4m TO 7.30m, BENTONITE SEAL FROM 0.1m TO 0.4m, BACK FILLED TO SURFACE	

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29807ZR BH1 START CORING AT 1.61m
CORE LOSS: 0.88m
2
3
CL: 0.11m
4
CORE LOSS: 0.59m
5
6
CL: 0.08
7
END OF BORHOLE AT 7.30 m

Borehole No.
2
1 / 2

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: HAND AUGER
R.L. Surface: ~64.0 m
Date: 28/9/16
Datum: AHD
Plant Type:
Logged/Checked By: A.C.K./P.R.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING AND 6/10/16	■				REFER TO DCP TEST RESULTS			▩		FILL: Silty clay, low to medium plasticity, dark brown, trace of root fibres.	MC<PL			APPEARS MODERATELY COMPACTED
	■			■				▨	CL-CH	SILTY CLAY: medium to high plasticity, orange brown mottled light grey, trace of root fibres.	MC>PL	VSt	390 350 365	RESIDUAL HP TESTING ON REMOULDED AUGER SAMPLES
				■		63	1	▨		as above, but orange brown and light brown.	MC~PL	VSt - H	465 350 415	
				■				▨		SILTY CLAY: medium to high plasticity, light grey mottled orange brown, trace of fine to medium grained ironstone gravel.	MC<PL	H	465 500 410	HAND AUGER REFUSAL ON INFERRED IRONSTONE GRAVEL
						62	2			REFER TO CORED BOREHOLE LOG				WASH BORE CASING 1.5m TO 1.8m
						61	3							
						60	4							
						59	5							
						58	6							

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CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Core Size: TT56
R.L. Surface: ~64.0 m
Date: 28/9/16
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: A.C.K./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT SPACING (mm) 500, 300, 100, 50, 30, 10	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
					START CORING AT 1.80m						
100% RETURN		62	2		CORE LOSS 0.31m						
					SILTY CLAY: medium plasticity, light grey mottled orange brown, trace of fine grained ironstone gravel, root fibres.	RS	St - VSt			(2.16m) HP: 210kPa (2.23m) HP: 340kPa (2.41m) HP: 220kPa (2.59m) HP: 170kPa	
					SILTY CLAY: medium plasticity, light grey, trace of fine grained sand.						
		61	3		CORE LOSS 0.61m						
		60	4		SILTY CLAY: low to medium plasticity, light grey, trace of fine grained sand.	RS	St - VSt			(4.09m) HP: 380kPa (4.18m) HP: 180kPa	
					SANDSTONE: fine grained, light grey and red brown.	XW - DW	EL - VL			(4.31m) CS, 0°, 20 mm.t (4.34m) CS, 0°, 10 mm.t (4.44m) Cr, 5°, 40 mm.t	
					CORE LOSS 0.46m						
		59	5		SANDSTONE: fine grained, light grey with high strength iron indurated bands.	XW - DW	EL - VL			(4.98m) CS, 0°, 40 mm.t (5.13m) J, 30°, P, S, CLAY INFILL	
					CORE LOSS 0.32m						
					SANDSTONE: fine grained, grey, with high strength iron indurated bands, bedded at 0-5°.	DW	VL - L			(5.73m) J, 80°, Un, R (5.81m) XWS, 0 - 30°, 30 mm.t (5.91m) Be, 5°, P, R, CLAY INFILL (5.98m) Cr, 0°, 40 mm.t	
		58	6		CORE LOSS 0.18m						
					SANDSTONE: fine grained, light grey with grey laminae, and iron indurated seams and bands, bedded at 0-15°.	DW	M			(6.50m) Be, 15°, P, R (6.61m) XWS, 5°, 20 mm.t (6.72m) CS, 0 - 5°, 30 mm.t (6.75m) J, 90°, P, R (6.84m) J, 70°, P, R, CLAY INFILL	
		57	7								
					END OF BOREHOLE AT 7.50 m					50mm DIA. PVC MONITORING WELL INSTALLED TO 3.0m DEPTH, MACHINE SLOTTED FROM 1.0m TO 3.0m, 2mm SAND FILTER PACK FROM 0.6m TO 7.50m, BENTONITE SEAL FROM 0.15m TO 0.6m, BACK FILLED TO SURFACE	

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29807ZR BH2 START CORING AT 1.80m
CORE LOSS: 0.31m
2
3
CORE LOSS: 0.61m
4
CORE LOSS: 0.46m
5
CORE LOSS: 0.32
6
CL: 0.18m
7
END OF BOREHOLE AT 7.5m

BOREHOLE LOG

Borehole No.
3
1 / 1

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~63.1 m
Date: 27/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		63			FILL: Sandy gravel, fine to medium grained, igneous, dark grey.	D			IGNEOUS GRAVEL COVER
	N = 18 8,7,11			CL-CH	SILTY CLAY: medium to high plasticity, light grey mottled red brown, with ironstone gravel.	MC~PL	H	>600 580 400	RESIDUAL
	N > 16 13,16/ 150mm REFUSAL	62	1		as above, but low to medium plasticity.	MC<PL			TOO FRIABLE FOR HP TESTING
		61	2		SANDSTONE: fine grained, light grey, with iron indurated bands.	XW	EL		
		60 59	3 4		SANDSTONE: fine to medium grained, grey and dark grey, thinly bedded with medium strength iron indurated bands.	DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
ON COMPLETION		58	5				H		VERY HIGH RESISTANCE
		57	6		END OF BOREHOLE AT 5.20 m				'TC' BIT REFUSAL

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Borehole No.
4
1 / 2

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~64.2 m
Date: 27/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING							ASPHALTIC CONCRETE: 35mm.t	M			
			64				FILL: Clayey gravel, fine to medium grained, igneous, dark grey.				
		N = 17 5,7,10		1		CH	SILTY CLAY: high plasticity, red brown mottled light grey, with ironstone gravel.	MC>PL	VSt - H	340 540 400	RESIDUAL
		N = 34 15,15,19	63 62	2		CL	SILTY CLAY: medium plasticity, light grey mottled orange and red brown, with ironstone gravel.	MC<PL	H	>600 >600 >600	
				3		-	SANDSTONE: fine grained, light grey, laminae, bedded, with L strength iron indurated and clay bands.	XW - DW	EL - VL		
			61					DW	L - M		LOW 'TC' BIT RESISTANCE AND MODERATE TO HIGH BANDS
ON COMPLETION OF CORING AND ON 6/10/16			60	4			as above, but with high strength iron indurated bands.				4.2-4.4m WATER SAMPLE
			59 58	5 6			REFER TO CORED BOREHOLE LOG				

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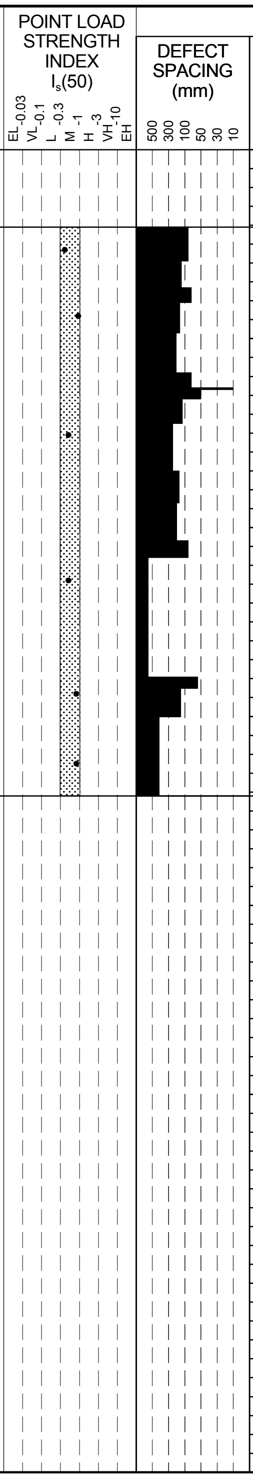
CORED BOREHOLE LOG

Borehole No.
4
2 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Core Size:
R.L. Surface: ~64.2 m
Date: 27/9/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK305
Bearing: N/A
Logged/Checked By: T.P./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		60			START CORING AT 4.41m				
100% RETURN		59 58	5 6		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10° and iron indurated bands.	DW	M	(4.50m) CS, 0 - 5°, 10 mm.t, HP: 220kPa (4.59m) CS, 3°, 2 mm.t (4.73m) CS, 0 - 5°, 8 mm.t (4.81m) CS, 4°, 4 mm.t (4.97m) CS, 0°, 60 mm.t (5.18m) XWS, 0 - 5°, 90 mm.t (5.26m) XWS, 1°, 2 mm.t (5.27m) XWS, 1°, 2 mm.t (5.32m) J, Un, R, CLAY INFILL, SUB VERTICAL (5.45m) CS, 0 - 5°, 25 mm.t, HP: 70kPa (5.70m) CS, 0°, 10 mm.t (5.87m) CS, 0 - 5°, 45 mm.t, HP:120kPa (6.07m) XWS, 0 - 5°, 6 mm.t (6.16m) CS, 0 - 5°, 90 mm.t, HP: 240, 320kPa	
					SANDSTONE: fine to medium grained, light grey with grey laminae, bedded at 5-15° and iron indurated bands.			(6.79m) XWS, 0 - 5°, 9 mm.t (6.85m) Be, 15°, P, R, IS	
		57	7		SANDSTONE: fine to medium grained, light grey.	SW		(7.00m) CS, 0°, 5 mm.t	
		56 55 54	8 9 10		END OF BOREHOLE AT 7.42 m			GROUNDWATER MONITORING WELL INSTALLED TO A DEPTH OF APPROXIMATELY 7.42m, CLASS 18 MACHINE SLOTTED PVC PIPE FROM 7.42m TO 3.42m, CASING FROM3.42m TO 0.1m, 2mm SAND FILTER PACK FROM 7.42m TO 1.5m, BENTONITE SEAL FROM 1.5m TO 0.2m, BACKFILLED WITH SAND TO THE SURFACE, COMPLETED WITH A STEEL GATIC COVER AND LOCKABLE CAP	



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JOB# 29807ZR BH4 START CORING AT 4.41m
4
5
6
7
EOBH AT 7.42m

Borehole No.
5
1 / 2

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~64.6 m
Date: 28/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			ASPHALTIC CONCRETE: 50mm.t				
		-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained igneous gravel.	M			
	N = 31 10,12,19	CL	SILTY CLAY: medium plasticity, light grey, with XW strength sandstone bands, and iron indurated bands.	MC<PL	H	>600 580 >600	RESIDUAL
	N > 8 11,8,0/ 0mm REFUSAL		SANDSTONE: fine grained, light grey, with medium strength iron indurated bands and VL bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			SANDSTONE: fine to medium grained, grey and dark grey, with medium strength iron indurated bands, thinly bedded.	DW	L		LOW RESISTANCE
							MODERATE RESISTANCE
							LOW TO MODERATE RESISTANCE
							MODERATE RESISTANCE
ON COMPLETION							6.8-7.1m WATER SAMPLE

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BOREHOLE LOG

Borehole No.
5
2 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~64.6 m
Date: 28/9/16 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** T.P./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■						SANDSTONE: fine to medium grained, grey and dark grey, with medium strength iron indurated bands, thinly bedded. *(continued)*	DW	M		MODERATE RESISTANCE
						57 56 55 54 53 52 51	8 9 10 11 12 13			END OF BOREHOLE AT 7.50 m				

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Borehole No.
6
1 / 2

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~66.3 m
Date: 28/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	■									ASPHALTIC CONCRETE: 70mm.t				
						66			-	FILL: Sandy gravel, fine to medium grained, dark grey and brown igneous, fine to coarse grained sand.	M			
	■			■	N = 17 6,6,11				CL	SANDY CLAY: low plasticity, light grey, orange brown and red brown, fine grained sand, with ironstone gravel.	MC<PL	(VSt - H)		RESIDUAL TOO FRIABLE FOR HP TESTING
				■		65	1				MC~PL	VSt		
				■	N > 16 6,16/ 100mm REFUSAL								250 400	
						64	2				MC<PL	(VSt - H)		
ON COMPLETION OF CORING				■					-	SANDSTONE: fine grained, light grey, with low to medium strength iron indurated bands.	XW - DW	EL - VL		VERY LOW TO LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
ON 6/10/16						63	3							
											DW	L		3.4-3.6m WATER SAMPLE LOW TO MEDIUM RESISTANCE
ON COMPLETION OF AUGERING				■			4							4.0-4.2m WATER SAMPLE
						62				REFER TO CORED BOREHOLE LOG				50mm DIA. PVC MONITORING WELL INSTALLED TO 7.3m DEPTH, MACHINE SLOTTED FROM 7.3m TO 3.3m, CASING FROM 3.3m TO 0.1m, 2mm SAND FILTER PACK FROM 7.3m TO 2.8m, BENTONITE SEAL FROM 2.8m TO 0.2m, AND COMPLETED WITH GATIC COVER AT SURFACE
						61	5							
						60	6							

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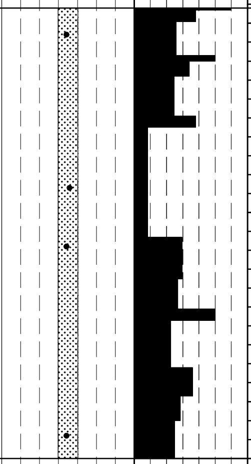
CORED BOREHOLE LOG

Borehole No.
6
2 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR	**Core Size:**	**R.L. Surface:** ~66.3 m
Date: 28/9/16	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** T.P./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
ON COMPLETION OF AUGERING					START CORING AT 4.27m						
		62			CORE LOSS 0.65m						
100% RETURN		61 60	5 6 7		SANDSTONE: fine grained, grey, with dark grey laminae, thinly bedded at 0-5°, and iron indurated bands.	DW	M			(4.93m) J, 45°, Un, R, IS (4.99m) J, 60°, P, S, IS (5.17m) J, SUBVERTICAL, P, R, IS (5.20m) Be, 0 - 5°, P, R, IS (5.28m) CS, 0 - 5°, 40 mm.t, HP: 200kPa (5.49m) CS, 0 - 5°, 40 mm.t, HP: 120kPa (5.55m) CS, 0°, 2 mm.t (6.13m) XWS, 0°, 18 mm.t (6.25m) Be, 0 - 5°, P, CLAY INFILL (6.35m) Be, 0°, P, S, CLAY INFILL (6.51m) J, SUBVERTICAL, Un, R, IS (6.55m) J, 22°, Un, R, IS (6.57m) Be, 0°, P, R, IS (6.82m) CS, 0 - 15°, 45 mm.t, HP: 170kPa (6.89m) XWS, 10°, 5 mm.t (6.97m) XWS, 0 - 5°, 30 mm.t (7.10m) CS, 0 - 5°, 18 mm.t, HP: 220kPa	
		59 58 57 56	8 9 10		END OF BOREHOLE AT 7.30 m						

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JOB# 29807ZR BH6 START CORING AT 4.27m
4
CORE LOSS 0.65m
5
6
7
END OF BH AT 7.30m

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Borehole No.
7
1 / 2

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~67.7 m
Date: 27/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING									-	ASPHALTIC CONCRETE: 25mm.t				ROADBASE
									CH	FILL: Sandy gravel, fine to medium grained igneous, blue grey and brown, fine to medium grained sand.	MC>PL	VSt		RESIDUAL
						67	1			SILTY CLAY: high plasticity, light grey mottled red brown and orange brown, trace of root fibres.			260 320 300	
						66	2			as above, but no root fibres and with ironstone gravel.	MC<PL	H	>600 580 >600	
ON COMPLETION OF CORING & AFTER 22 HRS ON 7/10/16						65 64	3 4		-	SANDSTONE: fine to medium grained, light grey, with low to medium strength iron indurated bands and clay bands.	XW	EL		VERY LOW TO LOW 'TC' BIT RESISTANCE 4.21-4.4m WATER SAMPLE
						63	5			SANDSTONE: fine to medium grained, light grey and grey, with low to medium strength iron indurated bands, thinly bedded.	DW	L		LOW TO MODERATE RESISTANCE
						62 61	6			REFER TO CORED BOREHOLE LOG				50mm DIA. PVC MONITORING WELL INSTALLED TO 8.6m DEPTH, MACHINE SLOTTED 8.6m TO 4.6m, CASING 4.6m TO 0.1m, 2mm SAND FILTER PACK 8.6m TO 3.9m, BENTONITE SEAL 3.8m TO 0.2m, COMPLETED WITH GATIC COVER AT SURFACE

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Borehole No.
7
2 / 2

CORED BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Core Size:
R.L. Surface: ~67.7 m
Date: 27/9/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK305
Bearing: N/A
Logged/Checked By: T.P./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03 VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		62			START CORING AT 5.70m						
100% RETURN			6		SANDSTONE: fine to medium grained, light grey and grey, thinly bedded at 0-10°, with iron indurated bands and dark grey laminae.	DW	VL M			(6.00m) CS, 0 - 5°, 25 mm.t (6.18m) XWS, 0 - 5°, 90 mm.t (6.24m) J, 65°, P, R, IS	
		61			SANDSTONE: fine to medium grained, light grey and grey.	XW	EL				
			7		very high strength iron indurated band 6.72m-6.81m.						
					low to moderate strength bands, 7.25m-7.34m, 7.52m-7.62m.					(7.49m) CS, 0 - 5°, 60 mm.t, HP: 210kPa (7.64m) J, 45°, Un, R	
		60	8		SANDSTONE: fine to medium grained, light grey and grey, thinly bedded at 0-10°, with dark grey laminae.	DW	L - M			(7.75m) CS, 0 - 10°, 20 mm.t, HP: 270kPa (8.03m) XWS, 0 - 5°, 110 mm.t (8.36m) XWS, 0 - 10°, 90 mm.t (8.44m) J, 45°, Un, R (8.50m) CS, 0 - 5°, 10 mm.t (8.57m) XWS, 0 - 5°, 55 mm.t	
		59	9		END OF BOREHOLE AT 8.60 m						
		58	10								
		57	11								
		56									

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JOB# 29807ZR BH7 START CORING AT 5.70
6
7
8
EOBH AT 8.60m

Borehole No.
8
1 / 1

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~63.3 m
Date: 27/9/16
Datum: AHD
Plant Type: JK305
Logged/Checked By: T.P./P.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
					ASPHALTIC CONCRETE: 100mm.t				
		63		-	FILL: Gravelly sand, fine to medium grained, brown, fine grained igneous gravel.	D			
	N > 18 8,18/ 100mm REFUSAL			CL	SANDY CLAY: low to medium plasticity,	MC<PL	(H)		RESIDUAL TOO FRIABLE FOR HP TESTING
			1		SANDSTONE: fine grained, light grey.	XW	EL		
		62 61	2		SANDSTONE: fine to medium grained, light grey, with iron indurated bands.	XW - DW	EL - VL		
		60 59 58	3 4 5		as above, but with XW bands.	DW	VL - L		
					SANDSTONE: fine to medium grained, light grey, with high to very high strength iron indurated bands.		L		LOW 'TC' BIT RESISTANCE
▼			6				M - H		MEDIUM TO HIGH RESISTANCE
		57					H - VH		HIGH TO VERY HIGH RESISTANCE 6.0-6.5m WATER SAMPLE

END OF BOREHOLE AT 7.00 m

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BOREHOLE LOG

Borehole No.
9
1 / 2

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~63.0 m
Date: 28/9/16 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** A.F./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING									CONCRETE: 160mm.t				8mm DIA. REINFORCEMENT, 30mm TOP COVER
	■							-	FILL: Gravelly sand, fine to coarse grained, brown, fine to medium grained igneous and sandstone gravel, trace of silt and brick fragments.	M			
	■			■	N = 20 5,7,13			CL	SANDY CLAY: medium plasticity, orange brown, fine grained sand, trace of ironstone gravel.	MC<PL	H	470 500 450	RESIDUAL
				■		62	1		SANDSTONE: fine to medium grained, red grey, with clay seams and M strength iron indurated bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE WITH LOW TO MODERATE BANDS
						61	2						
				■						DW	VL - L		
						60	3						
▼ on 6/10/16													3.35-3.55m WATER SAMPLE
						59	4						
						58	5						
				■					as above, but light grey and grey, without iron indurated bands.	SW	M		MODERATE RESISTANCE
						57	6		REFER TO CORED BOREHOLE LOG				50mm DIA. PVC MONITORING WELL INSTALLED TO 8.8m DEPTH, MACHINE SLOTTED 8.78m TO 2.8m, CASING 2.8m TO SURFACE, 2mm SAND FILTER PACK 8.8m TO 1m, BENTONITE SEAL 0.1m TO SURFACE, COMPLETED WITH STEEL GATIC COVER AT SURFACE

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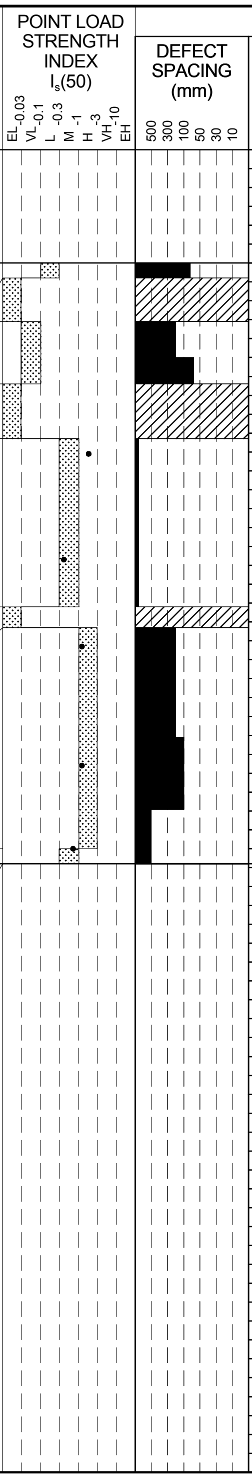
Borehole No.
9
2 / 2

CORED BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Core Size:
R.L. Surface: ~63.0 m
Date: 28/9/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: A.F./P.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				START CORING AT 5.60m				
100% RETURN		57	6	INTERBEDDED SANDSTONE AND SHALE: fine to medium grained, brown grey, bedded at 5-10°.	DW XW DW	L EL VL	(5.68m) Be, 5°, Un, R, IS (6.00m) FRACTURED BAND 50mm.t (6.10m) J, 85°, Un, R, IS (6.17m) J, 70°, Un, R, IS	
		56	7	as above, but grey and light grey, with high strengh seams.	XW DW XW	EL M EL	(7.52m) Be, 19°, Un, R, IS	
				SANDSTONE: fine grained, light grey, bedded at 5-10°.	SW	H	(7.77m) Be, 5°, Un, R, IS (7.82m) CS, 5°, 60 mm.t	
		55	8	as above, but bedded at 15°.		M	(8.11m) CS, 5°, 55 mm.t (8.20m - 8.46m) XWSx4, 15°, 3 mm.t	
		54	9	END OF BOREHOLE AT 8.78 m				
		53	10					
		52	11					



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JOB NO. 29807ZR
BH9
START CORING 5.60m
5
6
7
8
END OF HOLE AT 8.78m

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
10
1 / 1

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR **Method:** SPIRAL AUGER **R.L. Surface:** ~61.9 m
Date: 28/9/16 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** T.P./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION										CONCRETE: 105mm.t				7mm DIA. REINFORCEMENT, 35mm TOP COVER
									-	FILL: Sandy gravel, fine to medium grained igneous, dark grey, fine to coarse grained sand.	M			
									CH	SILTY CLAY: high plasticity, red brown mottled light grey, with ironstone gravel, trace of roots and root fibres.	MC>PL	VSt - H		RESIDUAL
					N = 13 5,6,7	61	1						440 390 450	
					N = 13 6,6,7	60	2						590 340 490	
												H		
					N > 20 13,20/ 150mm REFUSAL	59	3						450 450	
										SANDSTONE: fine grained, light grey, with low strength iron indurated bands.	XW	EL		
						58	4							
											DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						57	5							
										END OF BOREHOLE AT 5.20 m				
						56	6							
						55								

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
11
1 / 1

BOREHOLE LOG

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A & W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

Job No.: 29807ZR
Method: SPIRAL AUGER
R.L. Surface: ~63.2 m
Date: 30/9/16
Datum: AHD
Plant Type: JK205
Logged/Checked By: T.P./P.R.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			CONCRETE: 100mm.t				NO REINFORCEMENT OBSERVED
	N > 22 11,22/ 150mm REFUSAL	CL-CH	SILTY CLAY: medium to high plasticity, red brown mottled light grey, with ironstone gravel.	MC<PL	H	>600 >600	RESIDUAL
ON COMPLETION			SANDSTONE: fine grained, light grey, with iron indurated bands and clay bands.	XW - DW	EL - VL		3.2-3.4m VERY HIGH STRENGTH IRONSTONE BAND 3.9-4.5m WATER SAMPLE
			SANDSTONE: fine to medium grained, grey and dark grey, thinly bedded with medium strength iron indurated bands.	DW	M		
			END OF BOREHOLE AT 6.50 m				'TC' BIT REFUSAL

SAMPLES: ES, U50, DB, DS
RL (m AHD): 63, 62, 61, 60, 59, 58, 57
Depth (m): 1, 2, 3, 4, 5, 6

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client: CAPITAL INSIGHT PTY LTD
Project: PROPOSED NEW BUILDING & REFURBISHMENT OF BUILDINGS W6A&W6B
Location: MACQUARIE UNIVERSITY, NORTH RYDE, NSW

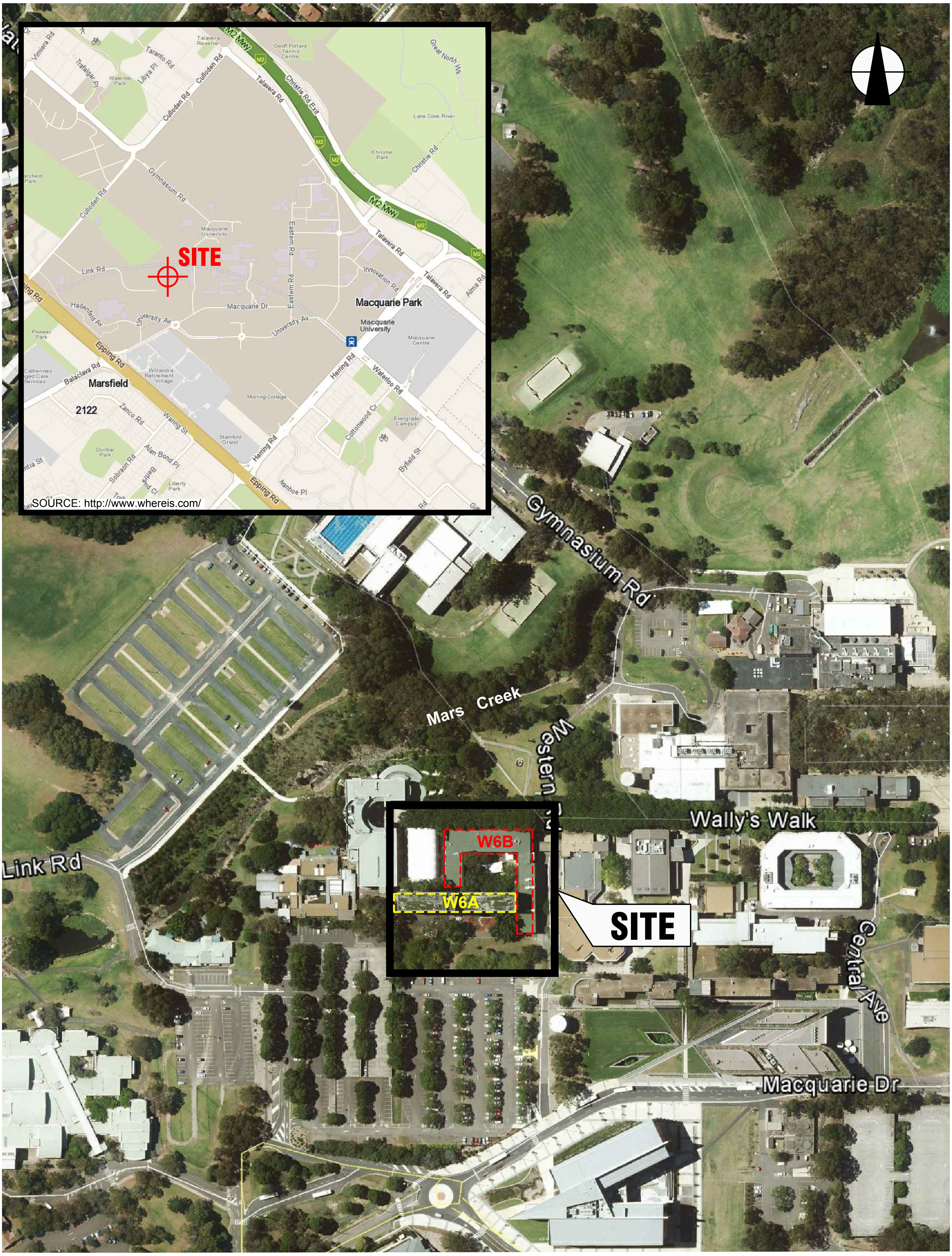
Job No. 29807ZR
Date: 28-9-16
Tested By: A.C.K.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Number of Blows per 100mm Penetration

Test Location	RL≈64.2m	RL≈64.0m					
Depth (mm)	**1**	**2**					
0 - 100	2	5					
100 - 200	2	7					
200 - 300	4	7					
300 - 400	7	7					
400 - 500	7	7					
500 - 600	7	6					
600 - 700	7	8					
700 - 800	7	7					
800 - 900	11	5					
900 - 1000	11	6					
1000 - 1100	12	4					
1100 - 1200	11	7					
1200 - 1300	10	15					
1300 - 1400	10	17					
1400 - 1500	10	13					
1500 - 1600	11	13					
1600 - 1700	9	16					
1700 - 1800	12	22					
1800 - 1900	13	23					
1900 - 2000	13	25					
2000 - 2100	13	REFUSAL					
2100 - 2200	11						
2200 - 2300	17						
2300 - 2400	14						
2400 - 2500	15						
2500 - 2600	18						
2600 - 2700	25/60mm						
2700 - 2800	REFUSAL						
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD



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This plan should be read in conjunction with the JK Geotechnics report.

Title:

SITE LOCATION PLAN

Location: BUILDING W6A & 6B
MACQUARIE UNIVERSITY, MACQUARIE PARK, NSW

Report No: 29807ZR

Figure No: 1

JK Geotechnics

PLOT DATE: 21/10/2016 5:17:40 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29807ZR MACQUARIE UNIVERSITY\CAD\29807ZR.DWG

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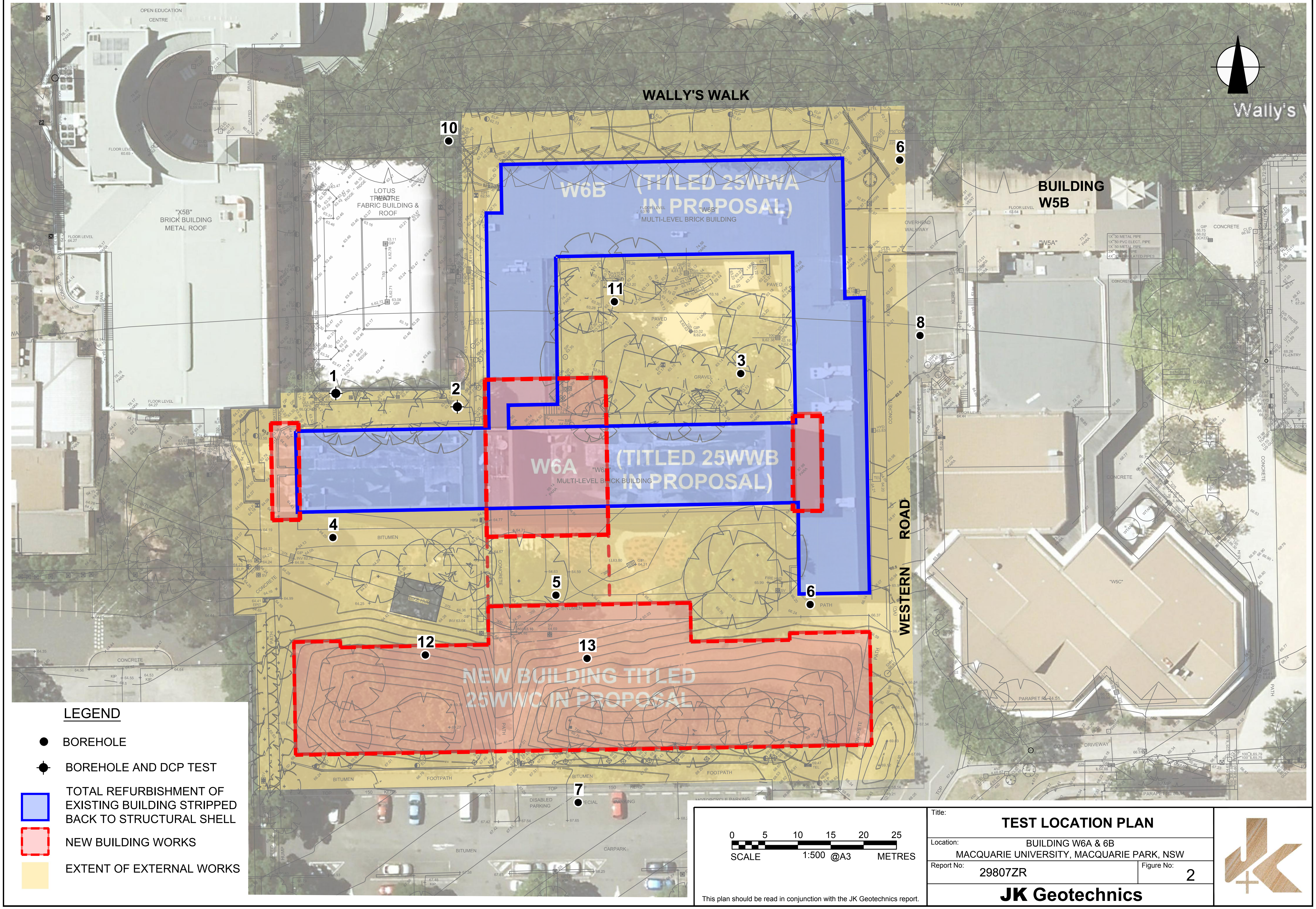

LEGEND

● BOREHOLE

⊕ BOREHOLE AND DCP TEST

🟦 TOTAL REFURBISHMENT OF EXISTING BUILDING STRIPPED BACK TO STRUCTURAL SHELL

🟥 NEW BUILDING WORKS

🟨 EXTENT OF EXTERNAL WORKS

0 5 10 15 20 25
SCALE 1:500 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:
TEST LOCATION PLAN

Location:
BUILDING W6A & 6B
MACQUARIE UNIVERSITY, MACQUARIE PARK, NSW

Report No: 29807ZR

Figure No: 2

JK Geotechnics

PLOT DATE: 5/06/2017 11:38:48 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29807ZR MACQUARIE UNIVERSITY\CAD\29807ZR.DWG

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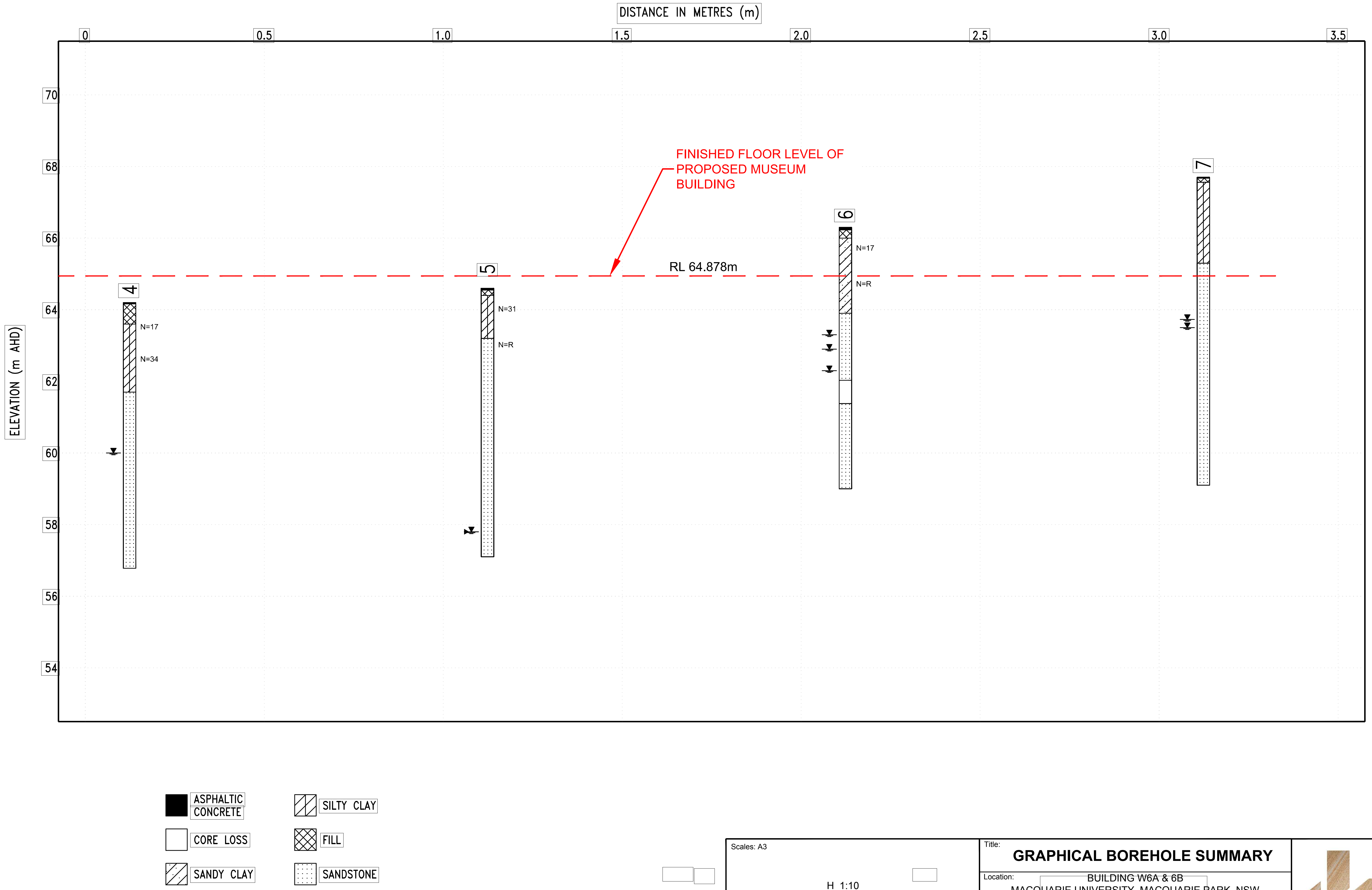

Scales: A3

H 1:10
V 1:100

This plan should be read in conjunction with the JK Geotechnics report.

Title:
GRAPHICAL BOREHOLE SUMMARY

Location: BUILDING W6A & 6B
MACQUARIE UNIVERSITY, MACQUARIE PARK, NSW

Report No: 29807ZR
Figure No: 3

JK Geotechnics

PLOT DATE: 5/06/2017 12:02:45 PM DWG FILE: S:\6 GEOTECHNICAL JOBS\29000'S\29807ZR MACQUARIE UNIVERSITY\CAD\29807ZR.DWG

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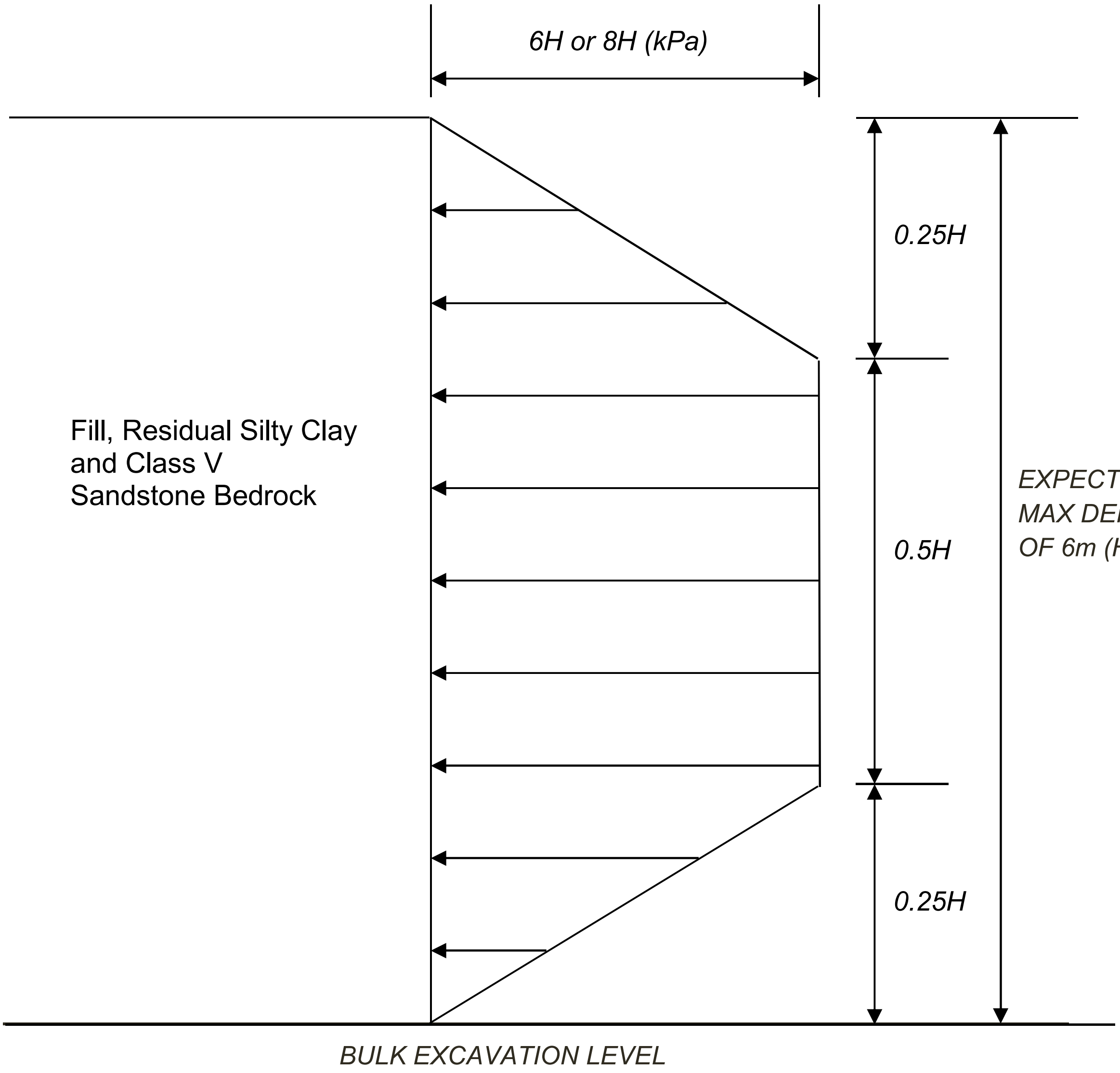


NOTE:

1 Use 4H for design of shotcrete infill panels where no movement sensitive structures or services are located within a horizontal distance equivalent to the depth of excavation.

2 Use 6H for design where no movement sensitive structures or services are located within a horizontal distance equivalent to the depth of excavation.

3 Use 6H for design of shotcrete infill panels where movement sensitive structures or services are located within a horizontal distance equivalent to the depth of excavation.

4 Use 8H for design where movement sensitive structures or services are located within a horizontal distance equivalent to the depth of excavation.

5 Surcharge and groundwater pressures must be added to the above, if applicable.

6 Vertical shale batters to be subject to progressive geotechnical inspections; anchoring of unstable rock wedges may be required.

7 Refer to text of report

RECOMMENDED DESIGN PRESSURES FOR ANCHORED OR PROPPED RETAINING WALLS – FULL DEPTH RETENTION

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No: 29807ZR Figure No 4

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained 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.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water 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 “feel” and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term ‘mud’ encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength 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 F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the ‘N’ value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm 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 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c” on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The 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.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

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

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than “straight line” variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it 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 and may not be the same at the time of construction.
- 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 be washed out of the hole or ‘reverted’ chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to 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 perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company 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 aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

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, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'* , published by the Institution of Engineers, Australia. Where information obtained from this investigation 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. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

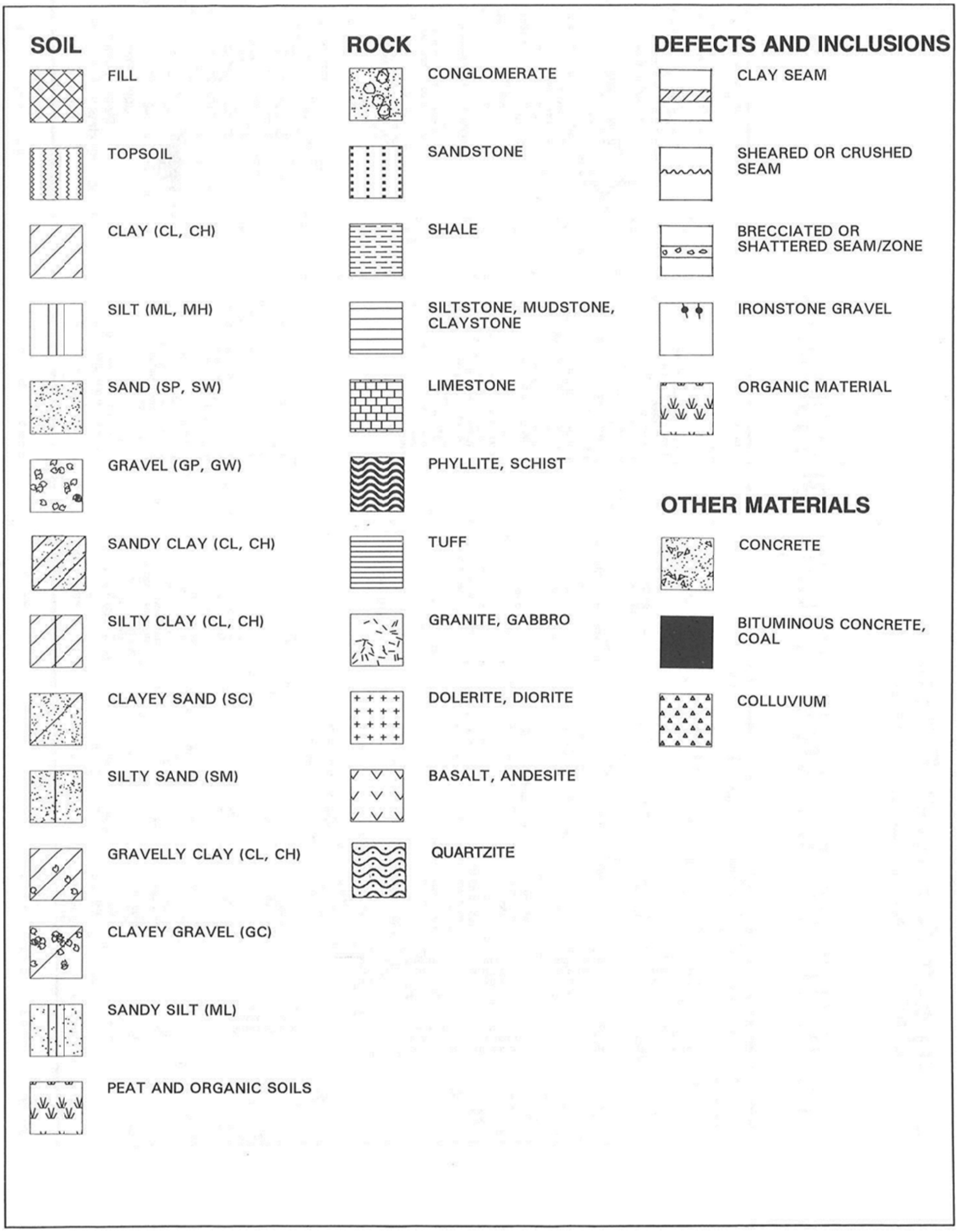
SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

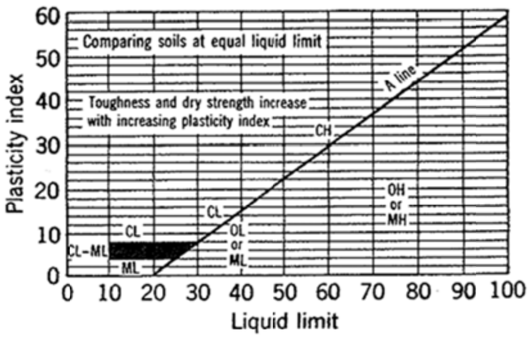
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria		
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for *GW*	
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures			Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines			$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for *SW*	
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits below "A" line with *PI* greater than 7	
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size										
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)						
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		 Plasticity chart for laboratory classification of fine grained soils	
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays				
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity				
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays				
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity				
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils				

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶—	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY – Runs freely through fingers.
	M	MOIST – Does not run freely but no free water visible on soil surface.
	W	WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – Unconfined compressive strength less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense >85 >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
----------------------	--------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
----------------------	--------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
----------------------	--------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
----------------------	--------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
----------------------	---------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
----------------------	---------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	

APPENDIX A

CERTIFICATE OF ANALYSIS 154914

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Tristan Piat

Sample log in details:

Your Reference:	**29807ZR, Macquarie Park**
No. of samples:	6 soils, 8 waters
Date samples received / completed instructions received	7/1016 / 7/10/16

This report replaces R01 due to changes in the sample depths.

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.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	14/10/16 / 2/11/16
Date of Preliminary Report:	Not Issued

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Accredited for compliance with ISO/IEC 17025 - Testing **Tests not covered by NATA are denoted with *.**

Results Approved By:

David Springer
General Manager

ACCREDITED FOR
TECHNICAL
COMPETENCE

Miscellaneous Inorganics						
Our Reference:	UNITS	154914-7	154914-8	154914-9	154914-10	154914-11
Your Reference	-------------	BH4	BH5	BH6	BH6	BH7
Depth	------------	4.2-4.4	6.8-7.1	4.0-4.2	3.4-3.6	4.21-4.4
Date Sampled		6/10/2016	28/09/2016	28/09/2016	6/10/2016	7/10/2016
Type of sample		WATER	WATER	WATER	WATER	WATER
Date prepared	-	07/10/2016	07/10/2016	07/10/2016	07/10/2016	07/10/2016
Date analysed	-	07/10/2016	07/10/2016	07/10/2016	07/10/2016	07/10/2016
pH	pH Units	5.6	5.2	4.9	4.6	5.2
Sulphate, SO4	mg/L	94	99	73	44	170
Chloride, Cl	mg/L	260	310	76	89	27

Miscellaneous Inorganics				
Our Reference:	UNITS	154914-12	154914-13	154914-14
Your Reference	-------------	BH8	BH9	BH11
Depth	------------	6.0-6.5	3.35-3.55	3.9-4.5
Date Sampled		30/09/2016	6/10/2016	30/09/2016
Type of sample		WATER	WATER	WATER
Date prepared	-	07/10/2016	07/10/2016	07/10/2016
Date analysed	-	07/10/2016	07/10/2016	07/10/2016
pH	pH Units	5.4	5.2	5.0
Sulphate, SO4	mg/L	150	110	100
Chloride, Cl	mg/L	240	31	160

Miscellaneous Inorg - soil						
Our Reference:	UNITS	154914-1	154914-2	154914-3	154914-4	154914-5
Your Reference	------------- -	BH1	BH2	BH4	BH7	BH8
Depth	------------	0.2-0.3	0.4-0.5	0.6-1.05	1.5-1.95	0.5-0.75
Date Sampled		29/09/2016	28/09/2016	27/09/2016	27/09/2016	30/09/2016
Type of sample		SOIL	SOIL	SOIL	SOIL	SOIL
Date prepared	-	11/10/2016	11/10/2016	11/10/2016	11/10/2016	11/10/2016
Date analysed	-	11/10/2016	11/10/2016	11/10/2016	11/10/2016	11/10/2016
pH 1:5 soil:water	pH Units	7.2	8.3	6.3	5.1	5.1
Sulphate, SO4 1:5 soil:water	mg/kg	10	32	140	24	51
Chloride, Cl 1:5 soil:water	mg/kg	<10	34	<10	<10	<10

Miscellaneous Inorg - soil		
Our Reference:	UNITS	154914-6
Your Reference	------------- -	BH10
Depth	------------	0.5-0.95
Date Sampled		28/09/2016
Type of sample		SOIL
Date prepared	-	11/10/2016
Date analysed	-	11/10/2016
pH 1:5 soil:water	pH Units	5.4
Sulphate, SO4 1:5 soil:water	mg/kg	62
Chloride, Cl 1:5 soil:water	mg/kg	<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-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 Miscellaneous Inorganics	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			07/10/2016	[NT]	[NT]	LCS-W1	07/10/2016
Date analysed	-			07/10/2016	[NT]	[NT]	LCS-W1	07/10/2016
pH	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-W1	102%
Sulphate, SO4	mg/L	1	Inorg-081	<1	[NT]	[NT]	LCS-W1	94%
Chloride, Cl	mg/L	1	Inorg-081	<1	[NT]	[NT]	LCS-W1	91%

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank
Date prepared	-			11/10/2016
Date analysed	-			11/10/2016
pH 1:5 soil:water	pH Units		Inorg-001	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	Dup. Sm#	Duplicate Base + Duplicate + %RPD	Spike Sm#	Spike % Recovery
Date prepared	-	[NT]	[NT]	LCS-1	11/10/2016
Date analysed	-	[NT]	[NT]	LCS-1	11/10/2016
pH 1:5 soil:water	pH Units	[NT]	[NT]	LCS-1	100%
Sulphate, SO4 1:5 soil:water	mg/kg	[NT]	[NT]	LCS-1	88%
Chloride, Cl 1:5 soil:water	mg/kg	[NT]	[NT]	LCS-1	88%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NR: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

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.

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.