REPORT TO

ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE

AT

CULLODEN ROAD AND GYMNASIUM ROAD, MACQUARIE PARK, NSW

Date: 20 October 2020
Ref: 33484BJrpt

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Jarett Mones

Jarett Mones
Associate | Geotechnical Engineer

Report reviewed by:

Daniel Bliss
Principal | Geotechnical Engineer

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

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33484BJrpt	Final Report	20 October 2020

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:
a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.
If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

At the Company's discretion, JKG may send a paper copy of this report for confirmation. In the event of any discrepancy between paper and electronic versions, the paper version is to take precedence. The USER shall ascertain the accuracy and the suitability of this information for the purpose intended; reasonable effort is made at the time of assembling this information to ensure its integrity. The recipient is not authorised to modify the content of the information supplied without the prior written consent of JKG.

Table of Contents

ATTACHMENTS

STS Table A: Four Day Soaked California Bearing Ratio Test Report
STS Table B: Moisture Content, Atterberg Limits & Linear Shrinkage Test Report
JKG Table C: Point Load Strength Index Test Report (2 Pages)
Envirolab Services Certificate of Analysis No. 251823
Borehole Logs BH1 to BH8 Inclusive (with Core Photographs for Cored Boreholes)
Figure 1: Site Location Plan
Figure 2: Borehole Location Plan
Vibration Emission Design Goals
Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Royal Institute for Deaf and Blind Children (RIDBC) - Centre of Excellence located at the corner of Culloden Road and Gymnasium Road, Macquarie Park, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Antonio Galasso of M Projects Pty Ltd (M Projects) on behalf of the RIDBC by signed 'Acceptance of Proposal', dated 1 September 2020. The commission was on the basis of Option 2 of our fee proposal dated 27 August 2020, Ref. P52447BJRev1.

In order to complete this geotechnical investigation, we were supplied with the following relevant documents:

- Draft architectural drawings by WMK Architecture (Project Number: 19181, drawing numbers shown on Cover Sheet and Location Plan, Drawing No. A000, dated 25 September 2020).
- Request for Fee Proposal with scope of work by M Projects, dated 13 August 2020.
- Plan showing proposed borehole locations on a WMK architectural drawing (pdf file labelled: 'Topography', received from M Project on 13 August 2020).
- Site Services: Potable Cold Water, Fire Hydrants, Natural Gas, High Voltage Electricity, Sewer Drainage & Stormwater Drainage by DBA Hydraulics (Project No. 3046, Revision: Model, dated 18 May 2018 and pdf file labelled 'COMBINED 180518-1047').
- Survey drawings by LTS Lockley (Ref: 30431 222DT, Sheets 1 to 5, date of survey 10 September 2020).

Based on the supplied information, we understand that following demolition of the concrete footpath and landscaping walls and the removal of trees, the proposed development will comprise construction of a two-level administration and medical facility over a single basement level within the eastern portion of the site (Zone 1) and a single level pre-school and primary school building within the western portion of the site (Zone 2). The basement below the proposed Zone 1 building is proposed at RL72m, which will require excavation to a maximum depth of about 5m. The floor level of the Zone 2 building is proposed at RL78.5m, which will require only minor excavation to a maximum depth of about 0.5m at the western end of the site. However, fill will be required for the majority of this building to a maximum depth of about 6m within the southern portion of the site. Based on discussions with the BG&E (structural engineer) we understand that the filling for the Zone 2 building will be permanently supported by a retaining wall. An access road and parking area are proposed to the north-west of the Zone 2 building adjacent to Culloden Road. We assume that this will be constructed at about the existing surface level.

The purpose of the investigation was to assess the subsurface conditions at eight borehole locations, and to use this as a basis for providing comments and recommendations on excavation, groundwater, earthworks, retention, footings, pavements, and slabs-on-grade.

In addition, our environmental division, JK Environments (JKE) has undertaken a review of readily available information relating to acid sulfate soils and salinity at the site, which has been included herein.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was completed on 15 and 16 September 2020 and comprised the following:

- The drilling of eight boreholes (BH1 to BH8) using our track mounted JK305 drill rig.
- BH1, BH5, BH6 and BH8, were initially auger drilled to depths of 5.7m, 7.25m, 4.2m and 5.7m, respectively and were extended to depths of 8.4m, 10.23m, 8.67m and 11.29m, respectively, by diamond rock coring techniques using an NMLC triple tube core barrel with water flush.
- BH2, BH3, BH4 and BH7 were auger drilled only to depths of 9m, 8.3m, 9m and 10.4m, respectively.
- Installation of groundwater monitoring wells in BH5 and BH6 for longer term groundwater monitoring.

The borehole locations, as shown on Figure 2, were set out by taped measurements from existing surface features. The surface RLs, as shown on the borehole logs, were estimated by interpolation between spot levels and contours shown on the supplied LTS survey drawings and are therefore approximate only. The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and strength of the subsurface soils were assessed from the Standard Penetration Test (SPT) 'N' values, augmented by the results of hand penetrometer tests on cohesive samples obtained from the SPT split tube sampler. Within the augured portion of the boreholes, the strength of the underlying weathered bedrock was estimated by observation of the auger penetration resistance of a Tungsten Carbide ('TC') bit attached to the base of the auger string, examination of the drill rock cuttings and subsequent correlation with laboratory moisture content test results. Strengths assessed in this manner are approximate only and variations of one strength order should not be unexpected.

The strength of the cored bedrock was assessed by examination of the recovered rock core and subsequent correlation with Point Load Strength Index ($I_{S(50)}$) testing. The results of the Point Load Strength Index tests are presented in the attached Table C and on the cored borehole logs. The Unconfined Compressive Strengths (UCS) of the rock core were estimated from the Point Load Strength Index tests and are also summarised in Table C. Photographs of the recovered core are presented with the borehole logs.

Groundwater observations were made in the boreholes during, on completion of augering, and on completion of coring. Subsequent groundwater readings were made within the wells installed in BH5 and BH6, 6.5 hours (BH5) and 28 hours (BH6) following installation and on 21 September and 1 October 2020 (about 1 or 2 weeks following installation). We note that water is introduced into the borehole during core drilling and therefore the water levels after coring are likely to be artificially higher than actual levels. In this regard on 21 September 2020 we pumped out the groundwater wells to remove the coring water. A summary of the groundwater levels is provided below in Section 3.2. Groundwater measurements are also shown on the borehole logs.

The fieldwork was completed in the full-time presence of our geotechnical engineer, Mr Baki Abdul, who set out the borehole locations, nominated the sampling and testing and prepared the borehole logs. The borehole logs, with core photographs, are attached with this report, together with a set of explanatory notes which provide further details of the investigation techniques adopted, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd (Envirolab), both NATA accredited laboratories, for testing to determine moisture contents, Atterberg limits, linear shrinkages and 4-day soaked CBR values (by STS) and pH, sulphate contents, chloride contents and resistivity (by Envirolab). The results are presented in the attached STS Tables A and B and Envirolab Certificate of Analysis No. 251823. Contamination testing of the site soils or groundwater was outside the scope of this geotechnical investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within the Macquarie University campus, to the north-west of the Sport and Aquatic Centre Facility and on the corner of Culloden Road and Gymnasium Road. The site is located within gently undulating topography, with an east to west trending ridgeline running across the high side of the site. The majority of the site is located on the southern side of the ridge and slopes down to the south at generally 3° to 6°. Surface levels within the site range from about RL79m on the crest of the ridge in the western corner to about RL71m at the southern end.

The site comprises a vegetated hillside, with large sized trees in the northern and southern portions, and grass in between. A concrete footpath winds through the site, between the intersection of Culloden Road and Gymnasium Road and the road on the south-eastern side adjacent to the Sport and Aquatic Centre. At the southern end of the footpath are timber retaining walls supporting the ground surface above the footpath typically for a height of about 0.5m.

Along the south-eastern boundary the ground surface steepens to about 15° to 20°, sloping down to the access road to the south-east of the site.

The site is bound by Culloden Road to the north-west, Gymnasium Road to the north-east, an internal access road and then the Sport and Aquatic Centre facility to the south-east and carparks to the south and south-west.

3.2 Geology and Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet indicates the site is underlain by Ashfield Shale, but near the contact with the underlying Hawkesbury Sandstone, both of the Wianamatta Group. The Ashfield Shale is described as 'black to dark-grey shale and laminite' whereas the Hawkesbury Sandstone is described as 'medium to coarse grained quartz sandstone, very minor shale and laminite lenses'.

In summary, the boreholes generally encountered silty clay fill covering residual silty clay that graded into weathered siltstone bedrock at depths ranging from 0.9m to 2.3m. In some boreholes, sandstone bedrock was encountered at depth. The more pertinent features encountered are described below. For a more detailed descriptions of the materials encountered in each borehole reference should be made to the attached borehole logs.

Fill

Silty clay fill was encountered in all boreholes to depths of 0.2m (BH3), 0.4m (BH2, BH5 and BH6), 0.5m (BH1 and BH7), 0.6m (BH4) and 1.3m (BH8). The silty clay fill was assessed to be of medium plasticity. Traces of fine to medium grained ironstone and siltstone gravel, root fibres and other organic material, slag and ash were noted within the fill. Within BH8, where testing was carried out in the fill, the fill was assessed to be moderately compacted.

Residual Silty Clay

The residual silty clay was assessed to be of medium to high plasticity and generally of hard strength. However, in BH7 and BH8, the residual clay was assessed to be of very stiff to hard strength. Traces of fine to medium grained ironstone and siltstone gravel, ash and organic material were encountered within the residual silty clay.

Weathered Bedrock

Weathered siltstone was encountered at depths of 0.9m (BH5), 1.0m (BH6), 1.1m (BH1), 1.2m (BH2), 1.3m (BH3 and BH4), 1.8m (BH8) and 2.3m (BH7).

In BH3 to BH6 and BH8 the siltstone was initially assessed to be extremely weathered with properties of a soil (hard clay). This extremely weathered zone was between 0.1m and 0.7m thick, and below that the siltstone was assessed to be distinctly weathered and of very low to low or low strength. In BH1, BH2 and BH7 the siltstone on initial contact was assessed to be distinctly weathered and of very low to low strength. Generally, the siltstone improved with depth to low strength, medium strength and high strength, with some very high strength bands. However, some lower strength bands were encountered in some boreholes as follows:

- BH1 at a depth range of 5.7m to 6.45m the siltstone bedrock was assessed to be of very low to low strength.
- BH6 at a depth of 4.2m, there was a 0.42m thick extremely weathered zone (hard silty clay) over very low strength siltstone from a depth range of 4.64m to 5.4m.
- BH8 at a depth of 7.3m, there was a 0.3m thick extremely weathered zone (hard silty clay).

We note that BH2 to BH4 and BH7 were augered only so it is possible that some extremely weathered or lower strength zones are present in the assessed higher strength bedrock that could not be identified during angering. To confirm if this is the case cored boreholes would be required.

Sandstone bedrock was encountered with depth in some of the boreholes in the south-eastern portion of the site as follows:

- BH6 at a depth of 5.4m to the termination depth of 8.67m. The sandstone bedrock was assessed to initially be highly weathered and of medium strength, then improving to be fresh and of high, high to very high or very high strength.
- BH7 at a depth of 9.8m to the 'TC' bit refusal depth of 10.4m. The sandstone bedrock was assessed to be fresh and of high strength.
- BH8 at a depth of 10.77m to the termination depth of 11.29m. The sandstone bedrock was assessed to be fresh and of high to very high strength.

Defects encountered within the rock mass in the cored boreholes included bedding partings, extremely weathered and clay seams (between 12mm to 110mm thick) and joints inclined at 30° to 90°.

The rock within the boreholes has been classified in general accordance with Pells et al. "Classification of Sandstones and Shales in the Sydney Region: A Forty Year Review", Australian Geomechanics, June 2019 and is summarised in the table below. We note that although sandstone was encountered in some boreholes the classification adopted has generally been taken as "Shale" as the sandstone was only encountered for limited depths. Class III Shale was only able to be classified in the boreholes where coring of the rock was carried out (BH1, BH5, BH6 and BH8). Such quality of rock was probably encountered within the remaining boreholes but since they were auger drilled only, an assessment of the defects within the rock cannot be made and so classification beyond Class IV Shale is not possible.

Borehole	Depth and Approximate RL (AHD) to the Start of each Rock Class					
	Class V		Class IV		Class III	
	Depth	Approx RL	Depth	Approx RL	Depth	Approx RL
1	1.1m	77.4m	1.1m	77.4m	6.6m	71.9m
2	1.2m	76.8m	1.2m	76.8m	N/A	
3	1.3m	73.3m	1.9m	72.7m	N/A	
4	1.3m	75.8m	2.2m	74.9m	N/A	
5	0.9m	76.4m	1.0m	76.3m	7.4m	69.9m
6	1.0m	70.8m	1.7m	70.1m	5.4m	66.4m
7	2.3m	73.4m	2.3m	73.4m	N/A	
8	1.8m	72.2m	2.0m	72.0m	8.4m	65.6m

Groundwater

No groundwater seepage was encountered during auger drilling, but in BH7 groundwater was measured on completion of augering at a depth of 6.8m. Within the groundwater monitoring wells, following removal of drilling water, groundwater was measured on 21 September and 1 October 2020 at the depths and levels given in the table below.

Borehole	Groundwater Measured on 21 September 2020		Groundwater Measured 1 October 2020	
	Depth	Approx RL (AHD)	Depth	Approx RL (AHD)
5	4.6m	72.7m	6.4m	70.9m
6	3.8m	68.0m	3.7m	68.1m

3.3 Laboratory Test Results

The 4-day soaked CBR tests returned values of 5% for the silty clay fill (BH1, depth range of 0.2m to 0.5m) and 4.0% for the residual silty clay (BH2, depth range of 0.4m to 1.2m). Swelling of 0.5% and 1.0% were recorded for these samples from BH1 and BH2, respectively.

Based on the results of the Atterberg Limits and linear shrinkage test results, the samples tested are of medium plasticity and are assessed to have a moderate shrink/swell potential with changes in moisture content.

The moisture content and point loads strength index test results on samples of the bedrock showed reasonably good correlation with our field assessment of rock strength. Point Load Strength Index ($I_{s\,(50)}$) tests ranged from 0.08MPa to 5.8MPa. Estimated unconfined compressive strength (UCS), based on the relationship of UCS = 20 x ($I_{s\,(50)}$), ranged from 1.6MPa to 116MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below and are also presented in the attached Envirolab Certificate of Analysis No. 251823. Refer to Section 4.10 for an interpretation of the results.

Summary of Envirolab Results

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH4	0.6-0.95	Silty Clay (Residual)	4.8	36	45	15,000
BH4	1.5-1.95	Extremely Weathered Siltstone: Silty Clay	4.4	10	57	14,000
BH8	0.5-0.95	Fill: Silty Clay	5.8	53	10	22,000

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Excavation recommendations provided below should be complemented by reference to the latest version of the Code of Practice ‘Excavation Work’, Code of Practice prepared by Safe Work Australia.

Excavation for the basement level of the Zone 1 building, to RL72m, will be required to a maximum depth of about 5m, although there may be locally deeper areas of excavation for OSD tanks, lift core ‘over runs’, services, etc. We have outlined the approximate footprint of the basement for the Zone 1 building on Figure 2. Excavation for the Zone 2 building, to RL78.5m, will only be minor be to a maximum depth of about 0.5m in the western corner of the site.

Excavation to these depths will encounter silty clay fill, residual silty clay and weathered siltstone bedrock within the Zone 1 basement excavation. We anticipate that the bedrock will generally be of very low to low strength, but some higher strength bedrock may be encountered.

Excavation of the soils and bedrock of very low strength or less, should be able to be completed using conventional earthmoving equipment, such as medium sized excavators (say 15 to 20 tonnes) with buckets with “tiger teeth” attached.

Where the bedrock is of low strength or greater strength, “hard rock” excavation conditions are expected. Excavation of such rock will require the use of rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders, rock saws, or rock splitting.

Hydraulic rock hammers must be used with care due to the risk of damage to existing structures or vibration sensitive services due to the vibrations generated by such equipment. The presence of any vibration sensitive services should be assessed prior to the start of excavation. The closest structure to the proposed excavation is the Sports and Aquatic Centre, but it is about 30m away from the south-eastern edge of the proposed basement so will be further away from where rock hammers will be used. Nevertheless, we consider it would be prudent to undertaken some vibration monitoring at the commencement of rock hammer use to assess if the vibrations transmitted to the structure are within acceptable limits. If the vibrations are not of concern then further monitoring is unlikely to be required, but if the vibrations are close to or above acceptable limits then further monitoring should be carried out. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive it would be necessary to change to alternative excavation equipment, such as a smaller rock hammer or non-percussive excavation equipment, such as ripping hooks, rotary grinders, rocks saws or rock splitting.

4.2 Groundwater

Following stabilisation of water levels in the installed wells (BH5 and BH6), the groundwater level was found to fall between the two wells from RL70.9m to RL68.1m, sloping down to the south. Due to the variable measured groundwater levels, we consider that they represent flow through the defects within the bedrock profile rather than a standing groundwater table.

The measured groundwater levels are below the proposed level for the Zone 1 basement of RL72.0m and therefore, groundwater is unlikely to be a significant issue for the proposed excavation. However, groundwater seepage may occur along the soil/rock interface and through joints and bedding partings within the rock, particularly during and after periods of rainfall. Any seepage that does occur should be able to be controlled using gravity drainage and conventional sump and pump techniques.

In the long term, drainage should be provided behind all retaining walls and possibly below the basement slab. The completed excavation should be inspected by the hydraulic consultant to confirm that the designed drainage system is adequate for the actual seepage flows.

4.3 Retention

For shallow excavations of no more than about 3m temporary batters could be used, but for the deeper excavations a retention system should be installed prior to the start of excavation. Since the formation of temporary batters will require the excavated material to be stockpiled on site and then used for backfill following construction of the permanent retaining walls, it may be more practical to install a retention system around the entire basement perimeter.

Where space permits, temporary batters no more than about 3m in height of 1 Vertical (V) in 1 Horizontal (H) may be used, provided the excavation is well away from existing surcharge loads including roadways and services. As a guide, surcharge loads should be no closer than twice the batter height from the top of any batter.

Permanent batters, if required, may be formed at slopes of 1V:2H providing these batters are protected from erosion and degradation. This protection may comprise covering with shotcrete and mesh or by establishing vegetation across the batters with thick root systems. However, flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation.

A suitable retention system to support the excavation for the proposed Zone 2 basement would be a soldier pile wall with shotcrete infill panels. The pile wall should be founded with sufficient embedment below bulk excavation level to satisfy stability and founding considerations; however, anchors or props will still be required to provide lateral support in addition to pile embedment. Walls retaining no more than about 3m may be designed as cantilevered walls, but walls retaining greater heights will require anchors or props in order to limit deflections and to satisfy stability criteria.

Permission will need to be obtained from the owners of any adjacent properties prior to installation of anchors below those properties. The design of anchors should also consider the location and depth of any services that surround the basement excavation so they can be avoided.

Cantilevered walls supporting a height of no more than about 3m may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.3 and a bulk unit weight of $20kN/m^3$. This assumes that some resulting ground movements are tolerable and no structure or movement sensitive services are located behind the wall. Where movements are to be kept low or walls are propped by other structural elements the trapezoidal pressure distributions given below should be used.

Anchored or propped walls should be designed on the basis of a trapezoidal lateral pressure distribution with a maximum lateral pressure of 4H kPa, where H is the depth of excavation in metres. The full lateral pressure should be applied over the central 50% of the trapezoidal pressure distribution. For boundary areas that are highly sensitive to lateral movement, such as services, buildings, etc. consideration should be given to using a trapezoidal lateral pressure distribution of magnitude 8H kPa to reduce deflections.

The above coefficient and lateral pressures are based on horizontal backfill surfaces and any inclined backfill should be taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the walls.

Alternatively, a more refined analysis of the retaining walls could be conducted by an engineer who can interpret the data in the borehole logs in relation to the other assumptions in the analysis method adopted, and has a good understanding of soil-structure interaction. The following parameters could be used initially for such analysis using say Wallap or Plaxis. However, the designer must satisfy themselves that the final parameters adopted are appropriate for their design. Some form of sensitivity analysis of the design parameters should be carried out as part of the design process.

Strata	Unit Weight	Drained Cohesion	Friction Angle	Elastic Modulus
Existing (Uncontrolled/Unengineered) Fill	19kN/m^3	2kPa	25°	10MPa
Engineered Fill and Residual Soil (very stiff to hard)	20kN/m^3	5kPa	28°	30MPa
Extremely Weathered Siltstone	22kN/m^3	5kPa	30°	60MPa
Very Low Strength Siltstone	23kN/m^3	30kPa	32°	300MPa

Where soldier piles are spaced more than about 1.8m apart, the reinforced shotcrete between the soldier piles should be designed as withstanding lateral pressure from the soil and extremely weathered rock, and distributing the loads back to the piles. During the excavation, reinforced shotcrete panels should be sprayed progressively as the excavation progresses to support the soil and weathered rock between the piles, such that there is no more than 1.8m vertical face of material exposed below the shotcrete at any time. It will be necessary to install strip drains behind each panel of shotcrete to dissipate pore pressures behind the shotcrete. While the strip drain will dissipate the pore pressures immediately behind the shotcrete, it may not significantly affect the pore pressures a short distance behind the excavation face, and as such there will be hydrostatic pressures on the shoring system. Therefore, these hydrostatic pressures must still be considered in the shoring design.

The toe socket of the piles may be designed based an allowable lateral pressure of 250kPa for siltstone of at least very low to low strength, though the upper 0.5m of socket below the bulk excavation, or any localised excavation such as for footings or services, should be ignored in the design of the pile toe capacity.

Where temporary anchors are used to support the excavation or permanent anchors for walls supporting filling below the Zone 2 building, they should be designed with minimum free lengths and bond lengths of 4m and 3m respectively, and the bond zone should be formed entirely behind a line drawn upward at 1V in 1H from the toe of the excavation/wall. Anchors bonded into the distinctly weathered siltstone of at least very low to low strength may be provisionally designed for maximum bond stress not exceeding 250kPa. All anchors should be proof loaded to at least 130% of their working load in the presence of a geotechnical or structural engineer independent of the contractor before locking off at about 80% of their working load. Lift-off tests should be completed on all anchors following lock-off, and at least 10% of anchors should be subjected to further lift-off testing two to four days after lock-off to confirm that the anchors are holding their load. If these anchors are showing any significant loss of load, all anchors should then be subject to lift-off testing to ascertain whether further lateral support is required. The anchors should preferably be designed as a design and construct tender to allow innovation in the design, and so the contractor is directly responsible for the performance of the anchors and contractual issues do not arise if anchors fail to hold their design loads.

We assume the final support of the shoring system for the Zone 1 building will be provided by bracing from the floor slabs of the proposed structure.

4.4 Subgrade Preparation and Filling

We understand that fill will be placed below the proposed Zone 2 building to raise the surface levels. This fill will be quite deep, possibly to maximum depths of about 6m and as such if the floor slab is supported on the fill significant differential settlement may occur between the portion of the slab supported on the residual soils and the portion supported on deep fill. Even if the fill is placed as controlled, engineered fill some consolidation settlement of the fill will occur, which must be allowed for in the design. Alternatively, the floor slab of the building could be designed as a fully suspended slab supported on the footing system founded within the underlying bedrock.

The following recommendations should be followed where the proposed fill is to support floor slab loads or for preparation of proposed pavement areas. Where fully suspended slabs are adopted, no particular subgrade preparation would be required other than stripping of vegetation and root affected soils.

Following demolition of the existing timber retaining walls and removal of trees and constructing temporary batters/shoring, areas where new pavements and buildings are proposed should be excavated to the design subgrade level. Where this does not include removal of the grass, vegetation and topsoil these should also be fully stripped. We anticipate that the subgrade will mostly be silty clay fill within Zone 2 and in the proposed access road and parking to the north-west of Zone 2, and a combination of silty clay fill, residual silty clay and siltstone bedrock at Zone 1.

The existing fill was generally between a thickness of about 0.2m and 0.6m at the investigation locations, but there is a localised thicker area, about 1.3m, around BH8 and possibly other areas. We are unaware of records that document the manner of placement, compaction specification and control of the fill. Hence, in the absence of records, the fill is considered to be “uncontrolled” and should not be relied upon to provide suitable foundation support to on-ground floor slabs or footings.

Within the proposed building areas where floor slabs are not suspended all existing fill should be fully removed and replaced with controlled, engineered fill. If a fully suspended system is adopted, void formers should be placed below the suspended slabs to reduce the risk of swelling clays placing upward pressure on the slabs. At this stage we suggest a void former of at least 40mm thickness below all suspended slabs and footing beams. Within pavement areas, the existing fill may remain in place provided it performs satisfactorily during proof rolling as recommended below.

Following excavation of fill in the building areas and stripping of vegetation and root affected soils within pavement areas, the exposed subgrade should be proof rolled with at least 8 passes of a minimum 8 tonne dead weight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and in the presence of a geotechnical engineer to detect any weak subgrade areas.

Any weak areas detected during proof rolling should be locally excavated to a sound base and the excavated material replaced within engineered fill, or as recommended by the geotechnical engineer during the proof roll inspection. Following any required treatment of the subgrade, engineered fill may be placed as recommended in Section 4.5.

4.5 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated silty clay fill, residual silty clay and weathered siltstone bedrock may be reused as engineered fill provided it is free of deleterious materials, particles in excess of 75mm in size, and can be appropriately moisture conditioned. Any clay fill should be compacted to a density strictly between 98% and 102% of SMDD and at a moisture content within 2% of Standard Optimum Moisture Content (SOMC). It is likely that additional moisture will be required to achieve the moisture requirement.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 2,500m^2 or three tests per visit, whichever requires the most tests. The frequency of testing for service trench backfill should be at least one test per two layers per 40 linear metres of trench.

We recommend that at least Level 2 control of fill compaction, as defined in AS3798-2007 (or latest standard at the time of testing), be adhered to on this site. Where structures will be supported on newly placed, i.e. engineered fill, we recommend that Level 1 control of fill placement and compaction be carried out. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.6 Site Classification

Due to the presence of uncontrolled fill (up to 1.3m depth) encountered within the boreholes, we classify the site in its present condition as a Class 'P' site in accordance with AS2870-2011. Following completion of the proposed excavation and filling, the site classification would still remain as Class ‘P’ due to the variable conditions that will be present ranging from areas of deep fill to exposed bedrock. Based on the size of the proposed development, the standard footing designs given in AS2870-2011 do not apply for this development. The fill and residual clays will be subject to shrink/swell movements, but the extent will depend on the nature of the material used for fill. As a guide, movements similar to a class M site may be expected for the residual silty clay, but if clay is used as engineered fill the movement may increase to be similar to a Class H1 or H2 site.

4.7 Footings

Since siltstone bedrock will be encountered within the basement excavations and is expected to be encountered within the footing excavations in the western corner of the site, all footings should be founded within the bedrock to provide uniform support and reduce the risk of differential settlements. Where slabs are sensitive to surface movements consideration should be given to adopting fully suspended slabs as discussed in Section 4.4.

Pad or strip footings would be appropriate where rock is exposed or is at depths of less than about 1m, but where rock is deeper the use of piles would be more practical. However, even where rock is exposed piles may be adopted in order to reach better quality rock.

Bored piers would be appropriate for this site. However, some groundwater seepage may be encountered, requiring the use of pumps and/or tremie concreting techniques.

Reference should be made to the table in Section 3.2 for the depths and levels where each class of rock was encountered in the boreholes. The parameters given in the following table may be used for the design of footings founded within the rock. Piles should be nominally socketed at least 0.3m into the design class of rock. Where piles extend below a nominal 0.3m socket into rock the allowable shaft adhesions may be adopted for compressive and tensile (uplift) loads. This assumes that the rock socket is suitably roughened.

The serviceability parameters are based on settlement of less than 1% of the footing width or pile diameter. The ultimate parameters may be used for limit state design on the understanding that settlement of the footings may be up to 5% of the footing width or pile diameter. Differential settlement of about half the total settlements would be expected. The designer may use the modulus values given below to estimate the settlement of particular footings.

Rock Class	Allowable End Bearing Pressure	Allowable Shaft Adhesion in compression	Ultimate End Bearing Pressure	Ultimate Shaft Adhesion in compression	Elastic Modulus
Class V	700kPa	70kPa	3000kPa	100kPa	70MPa
Class IV	1000kPa	100kPa	3000kPa	150kpa	300MPa
Class III	3500kPa	350kPa	20,000kPa	500kPa	1000MPa

For the design of piles in uplift, shaft adhesions of half the above shaft adhesions in compression may be used.

Ultimate values must be used in conjunction with an appropriate geotechnical reduction factor (ϕ_g) which must be calculated in accordance with the methodology outlined in AS2159-2009 'Piling Design and Installation'.

For footings founded within Class V or Class IV rock at least the initial stages of footing excavation or pile drilling should be inspected by a geotechnical engineer to confirm that the suitable foundation material has been encountered. Where footings are founded within Class III rock we recommend that all footing excavations and the drilling of all piles be inspected by a geotechnical engineer.

4.8 Pavement Design Parameters

An access road and parking area are proposed to the north-west of the Zone 2 building as shown on Figure 2. We anticipate that some minor excavations may be required to grade the access road at the entry and exit to Culloden Road, but otherwise levels will be close to the existing.

Provided the subgrade is prepared as recommended in Section 4.4 above, we recommend that the proposed pavement be designed based on a soaked CBR of 4%, or an estimated modulus of subgrade reaction of 28kPa/mm (750mm plate).

Where fill is used to raise site levels, or replace unsuitable subgrade by the appropriate depth, pavement design may reflect the thickness and four day soaked CBR value of the imported material.

Surface and subsoil drainage should be provided around the perimeter of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 300mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain. In addition, the surface of the adjacent pavement subgrade should be provided with a uniform cross fall towards the subsoil drain to assist with drainage.

Concrete pavements should have a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (latest version) unbound base material (or similar good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

4.9 Slabs-On-Grade

Where a suspended slab system is not adopted for the building, and where the proposed floor slabs will directly overlie bedrock we recommend that an underfloor drainage blanket be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints. We also recommend that the slab be designed with top and bottom reinforcement and be made rigid enough to withstand shrink/swell movements of the residual clays and any clay fill and long term settlement of any deep fill.

4.10 Aggressivity

Based on the results of the pH, sulphate content, chloride content and resistivity testing, the soils and weathered siltstone would have an exposure classification of 'Moderate' when assessed in accordance with the criteria of concrete piles given in Table 6.4.2 (C) of AS2159-2009 "Piling Design and Installation''. Any concrete exposed to these conditions should have a characteristic concrete strength and cover as recommended in Table 6.4.3 of the standard.

For steel piles the soils and weathered siltstone would have an exposure classification of 'Non-Aggressive' when assessed in accordance with the criteria given in Table 6.5.2 (C) AS2159-2009. Any steel exposed to these conditions should have a uniform corrosion allowance as recommended in Table 6.5.3.

4.11 Acid Sulfate Soil

A review of the Department of Land and Water Conservation (1997) 1:25,000 Acid Sulfate Soil Risk Map for Prospect / Parramatta River indicates the site is in an area of "no known occurrence" for acid sulfate soils. A review of the City of Ryde Acid Sulfate Soil Planning Maps indicates that the site is not mapped as being in a risk Class 1, 2, 3, 4 or 5 area. It is also noted that the site is located well above RL5m and is underlain by residual soils and relatively shallow siltstone bedrock. Based on this information, the risk of encountering acid sulfate soil materials at the site or in the immediate vicinity is considered to be negligible. An acid sulfate soil management plan is not considered to be required.

4.12 Salinity

The site is not located within the area of Western Sydney included in the Salinity Potential Map. However, based on the topography and considering the geological landscape, soil and groundwater salinity may occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects.

Reference should be made to the aggressivity information in Section 4.10 on design of buried structural elements.

4.13 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Inspection of temporary batters.

- Proof roll inspections and inspection of bulk excavation materials.
- Inspection of all footings, including pile drilling, to confirm the design allowable bearing pressures and socket lengths (as applicable to the design).
- Compaction testing of any engineered fill placed.

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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes 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 is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten 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) could be expected. We strongly recommend that this requirement 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
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33484BJ
Project:	Proposed Royal Institute for Deaf & Blind Children - Centre of Excellence	**Report:**	A
		Report Date:	28/09/2020
Location:	Culloden & Gymnasium Roads, Macquarie Park, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	BH 1	BH 2
DEPTH (m)	0.20 - 0.50	0.40 - 1.20
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.52 STD	1.63 STD
Optimum Moisture Content (%)	24.6	21.4
Moulded Dry Density (t/m^3)	1.48	1.59
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	99	98
Moisture Contents		
Insitu (%)	19.8	16.9
Moulded (%)	24.5	20.9
After soaking and After Test, Top 30mm(%)	30.8	30.0
Remaining Depth (%)	28.7	23.3
Material Retained on 19mm Sieve (%)	9*	0
Swell (%)	0.5	1.0
C.B.R. value: @2.5mm penetration	5	4.0

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 17/09/2020.
- * Denotes not used in test sample.

28/09/2020
Authorised Signature / Date
(D. Treweek)

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

MOISTURE CONTENT, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Royal Institute for Deaf & Blind Children - Centre of Excellence
Location: Culloden & Gymnasium Roads, Macquarie Park, NSW

Ref No: 33484BJ
Report: B
Report Date: 30/09/2020
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 %**
1	2.50 - 3.00	5.7	-	-	-	-
2	0.50 - 0.95	18.7	46	23	23	11.0
3	2.00 - 3.00	8.0	-	-	-	-
4	0.60 - 0.95	18.7	50	24	26	11.0
4	2.20 - 3.00	7.2	-	-	-	-
4	5.50 - 6.00	7.4	-	-	-	-
5	5.00 - 6.00	7.0	-	-	-	-
6	2.00 - 3.00	6.7	-	-	-	-
6	3.50 - 4.00	7.4	-	-	-	-
7	4.00 - 4.50	8.0	-	-	-	-
7	8.50 - 9.00	4.5	-	-	-	-

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: 21/09/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

30/09/2020
Authorised Signature / Date
(D. Treweek)

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Royal Institute for Deaf & Blind Children (RIDBC)	**Ref No:**	33484BJ
Project:	Proposed RIDBC Centre of Excellence	**Report:**	C
Location:	Culloden & Gymnasium Roads, Macquarie Park, NSW	**Report Date:**	18/09/20

Page 1 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	5.85 - 5.89	0.3	6	A
	6.24 - 6.28	0.1	2	A
	6.63 - 6.67	2	40	A
	7.12 - 7.14	1.7	34	A
	7.51 - 7.54	1.7	34	A
	7.86 - 7.89	1.3	26	A
	8.18 - 8.21	1.1	22	A
5	7.41 - 7.45	1.1	22	A
	7.81 - 7.84	1.6	32	A
	8.05 - 8.09	1.6	32	A
	8.79 - 8.83	1	20	A
	9.18 - 9.22	1.1	22	A
	9.47 - 9.49	1	20	A
	9.86 - 9.89	1	20	A
	10.07 - 10.10	0.9	18	A
6	4.52 - 4.56	0.08	1.6	A
	5.60 - 5.62	0.4	8	A
	5.85 - 5.88	5.8	116	A
	6.25 - 6.29	3.1	62	A
	6.77 - 6.81	2.2	44	A
	7.33 - 7.36	3.4	68	A
	7.83 - 7.87	2.7	54	A
	8.12 - 8.16	2.4	48	A
	8.48 - 8.52	1.9	38	A
8	6.37 - 6.40	0.3	6	A

NOTE: SEE PAGE 2

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Royal Institute for Deaf & Blind Children (RIDBC)	**Ref No:**	33484BJ
Project:	Proposed RIDBC Centre of Excellence	**Report:**	C
Location:	Culloden & Gymnasium Roads, Macquarie Park, NSW	**Report Date:**	18/09/20

Page 2 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
8	6.97 - 7.00	0.6	12	A
	7.11 - 7.15	0.5	10	A
	8.40 - 8.42	2	40	A
	8.63 - 8.65	3.3	66	A
	8.84 - 8.88	2.7	54	A
	9.12 - 9.15	2.2	44	A
	9.40 - 9.43	3.9	78	A
	9.83 - 9.86	4.2	84	A
	10.35 - 10.39	3.9	78	A
	10.87 - 10.90	3.1	62	A
	11.18 - 11.21	2.4	48	A

NOTES

1. In the above table testing was completed in test direction A for the Axial direction, D for the Diametral direction, B for the block test and L for the Lump test.
2. The above strength tests were completed at the 'as received' moisture content
3. Test Method: RMS T223.
4. For reporting purposes, the IS(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 IS (50)

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

CERTIFICATE OF ANALYSIS 251823

Client Details

Client	JK Geotechnics
Attention	Baki Abdul
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33484BJ, Macquarie Park**
Number of Samples	3 Soil
Date samples received	22/09/2020
Date completed instructions received	22/09/2020

Analysis Details

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

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

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

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

Report Details

Date results requested by	29/09/2020
Date of Issue	25/09/2020

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

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

Results Approved By
Diego Bigolin, Team Leader, Inorganics

Authorised By

Nancy Zhang, Laboratory Manager

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Misc Inorg - Soil				
Our Reference		251823-1	251823-2	251823-3
Your Reference	UNITS	BH4	BH4	BH8
Depth		0.6-0.95	1.5-1.95	0.5-0.95
Date Sampled		16/09/2020	16/09/2020	15/09/2020
Type of sample		Soil	Soil	Soil
Date prepared	-	23/09/2020	23/09/2020	23/09/2020
Date analysed	-	23/09/2020	23/09/2020	23/09/2020
pH 1:5 soil:water	pH Units	4.8	4.4	5.8
Chloride, Cl 1:5 soil:water	mg/kg	45	57	10
Sulphate, SO4 1:5 soil:water	mg/kg	36	10	53
Resistivity in soil*	ohm m	150	140	220

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			23/09/2020	1	23/09/2020	23/09/2020		23/09/2020	[NT]
Date analysed	-			23/09/2020	1	23/09/2020	23/09/2020		23/09/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	4.8	4.9	2	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	45	44	2	95	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	36	39	8	100	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	150	150	0	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

Samples #3 were out of the recommended holding time for this analysis pH in soil.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~78.5 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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										FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and siltstone gravel, trace of ash.	w<PL			
					N = 22 6,9,13	78	1		CH	Silty CLAY: high plasticity, brown and yellow brown, trace of fine to medium grained ironstone and siltstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
						77	2		-	SILTSTONE: grey and brown.	DW	VL - L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						76 75 74	3 4 5					M		MODERATE RESISTANCE
						73				as above, but grey and dark grey.				
						72	6			REFER TO CORED BOREHOLE LOG				

COPYRIGHT

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
2 / 2

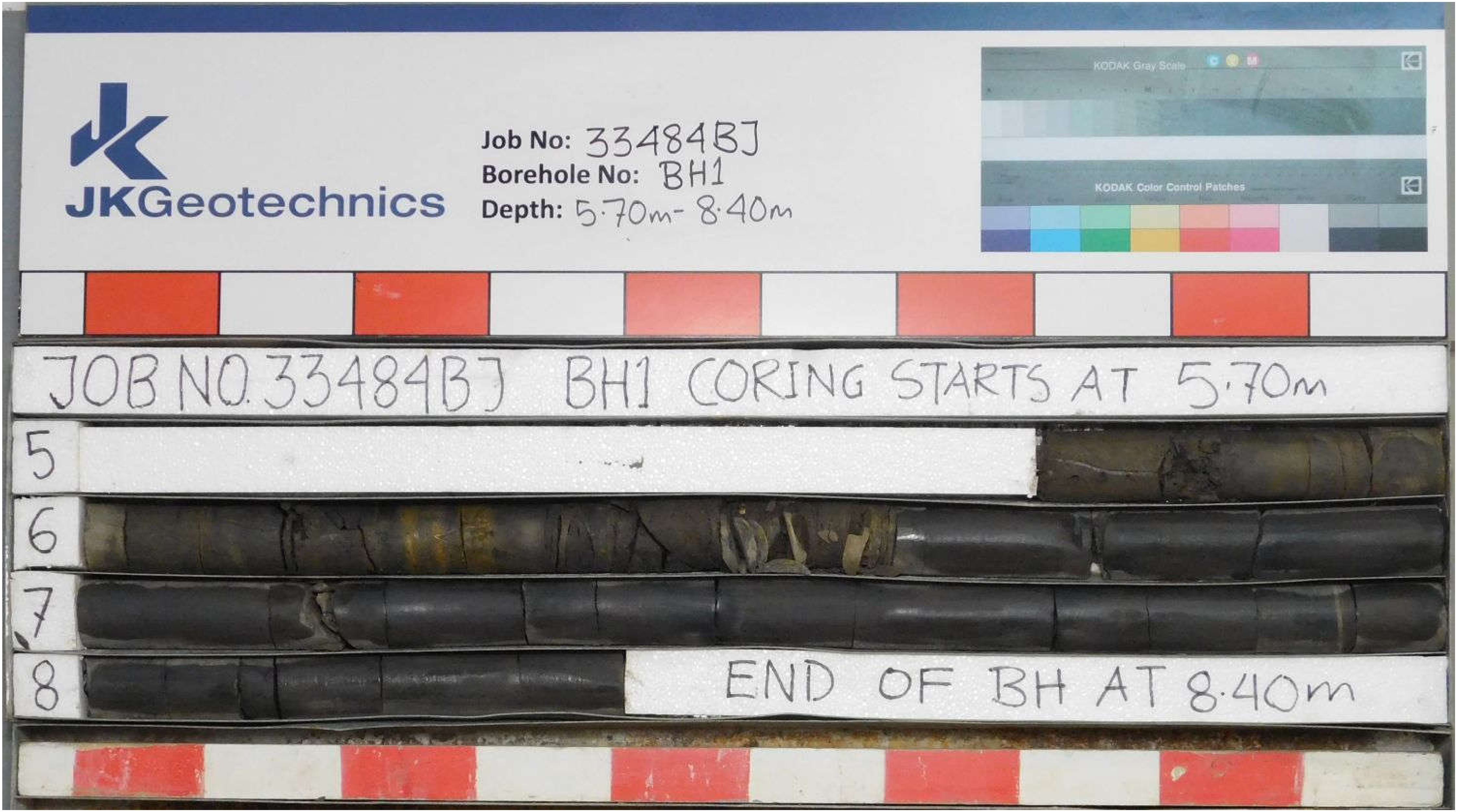
Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ | **Core Size:** NMLC | **R.L. Surface:** ~78.5 m
Date: 16/9/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** B.A./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
		73			START CORING AT 5.70m						
100% RETURN			6		SILTSTONE: grey and brown, indistinct rock fabric.	HW	VL - L	0.30 0.10		(5.70m) XWS, 0°, 100 mm.t (5.80m) CS, 0°, 30 mm.t (6.45m) J, 50°, P, S, Fe Sn	Ashfield Shale
		72			SILTSTONE: grey and dark grey, indistinct rock fabric..	FR	H	2.0 1.7 1.7 1.3 1.1		(6.75m) Be, 2°, P, S, Cn (6.86m) Be, 2°, P, S, Cn (7.18m) J, 40°, P, S, Cn	
		71	7								
		70	8		END OF BOREHOLE AT 8.40 m						
		69	9								
		68	10								
		67	11								

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33484BJ MACQUARIEPARK.GPJ <<DrawingFile>> 20/10/2020 11:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 33484BJ
Borehole No: BH1
Depth: 5·70m- 8·40m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~78.0 m
Date: 16/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

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										FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and siltstone gravel, trace of organic material.	w<PL			
					N = 23 6,10,13				CI	Silty CLAY: medium plasticity, brown and orange brown, trace of fine to medium grained ironstone and siltstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
						77	1							
									-	SILTSTONE: grey, dark grey and brown, with medium strength siltstone bands.	DW	VL - L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						76	2							
						75	3							
						74	4							
						73	5							
												M		MODERATE RESISTANCE
						72	6							
										as above, but grey and dark grey.				

BOREHOLE LOG

Borehole No.
2
2 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~78.0 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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
						70	8		-	as above, but grey and dark grey. *(continued)*	DW	M		MODERATE RESISTANCE
										SILTSTONE: dark grey.	FR	H		HIGH RESISTANCE
						69	9			END OF BOREHOLE AT 9.00 m				
						68	10							
						67	11							
						66	12							
						65	13							

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~74.6 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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							FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone gravel, trace of root fibres.				
		N = 19 5,8,11	74	1		CH	Silty CLAY: high plasticity, light grey, brown and orange brown, trace of fine to medium grained siltstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
		N > 28 8,16,12/ 80mm REFUSAL	73			-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey and yellow brown, with very low to low strength siltstone bands.	XW	Hd		ASHFIELD SHALE
			72 71 70 69 68	2 3 4 5 6			SILTSTONE: grey, dark grey and brown, with medium strength bands.	DW	VL - L		LOW 'TC' BIT RESISTANCE
							SILTSTONE: grey and dark grey.		M		MODERATE RESISTANCE

COPYRIGHT

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
2 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~74.6 m
Date: 16/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

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										
				■		67		▥	-	SILTSTONE: grey and dark grey. *(continued)*	DW	M		MODERATE RESISTANCE
				■			8	▥		SILTSTONE: grey, with fine to medium grained sandstone bands.	SW	H		HIGH RESISTANCE
						66 65 64 63 62 61	9 10 11 12 13			END OF BOREHOLE AT 8.30 m				'TC' BIT REFUSAL

COPYRIGHT

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~77.1 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained siltstone gravel, trace of ash and root fibres.	w<PL			GRASS COVER
	N = 16 6,7,9	CI	Silty CLAY: medium plasticity, brown, yellow brown and light grey.	w<PL	Hd		RESIDUAL
	N = 39 3,14,25		Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey brown and yellow brown, with very low to low strength siltstone bands.	XW	Hd		ASHFIELD SHALE
			SILTSTONE: grey, dark grey and brown, with medium strength siltstone bands.	DW	VL - L		LOW 'TC' BIT RESISTANCE WITH OCCASIONAL MODERATE BANDS
			SILTSTONE: grey and dark grey.		M		LOW TO MODERATE RESISTANCE

SAMPLES: ES, U50, DB, DS

RL (m AHD): 77, 76, 75, 74, 73, 72, 71

Depth (m): 1, 2, 3, 4, 5, 6

Graphic Log

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
2 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~77.1 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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
				■		70		▥		SILTSTONE: grey and dark grey. *(continued)*	DW	M		LOW TO MODERATE RESISTANCE
				■		69	8	▥		SILTSTONE: dark grey.	FR	H		HIGH RESISTANCE
						68	9			END OF BOREHOLE AT 9.00 m				
						67	10							
						66	11							
						65	12							
						64	13							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33484BJ MACQUARIEPARK.GPJ <<DrawingFile>> 20/10/2020 11:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 3

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~77.3 m
Date: 16/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

Groundwater Record	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		77			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and siltstone gravel, trace of root fibres and organic material.	w<PL			
	N = 18 5,7,11			CH	Silty CLAY: high plasticity, light grey, brown and orange brown, trace of organic material.	w<PL	Hd	>600 >600 >600	
			1	-	Extremely Weathered siltstone: silty CLAY, low to medium plasticity, light grey, yellow brown and orange brown, with iron indurated bands.	XW	Hd		ASHFIELD SHALE
		76			SILTSTONE:grey and brown.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
							L		LOW RESISTANCE
		75	2						
AFTER 6.5 HRS		74	3				L - M		LOW TO MODERATE RESISTANCE
ON 21/9/20		73	4						ON 21/9/20 WELL PUMPED OUT TO A DEPTH OF 7.5m
		72	5		as above, but grey, dark grey and orange brown.				
ON 1/10/20		71	6				M		MODERATE RESISTANCE

COPYRIGHT

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
2 / 3

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~77.3 m
Date: 16/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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										
									-	as above, but grey, dark grey and orange brown. *(continued)*	DW	M		
						70 69 68 67 66 65 64	8 9 10 11 12 13			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 10.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 10.23 m TO 4.2m. CASING 4.2m TO 0.1m. 2mm SAND FILTER PACK 10.23 m TO 2.2m. BENTONITE SEAL 2.2m TO 0.4m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

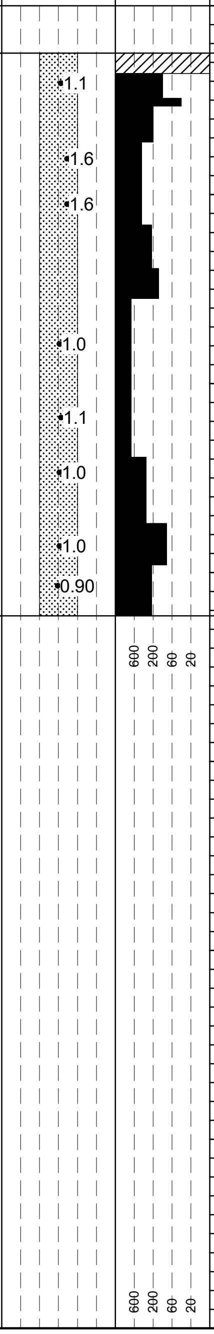
CORED BOREHOLE LOG

Borehole No.
5
3 / 3

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ	**Core Size:** NMLC	**R.L. Surface:** ~77.3 m
Date: 16/9/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.A./J.M.

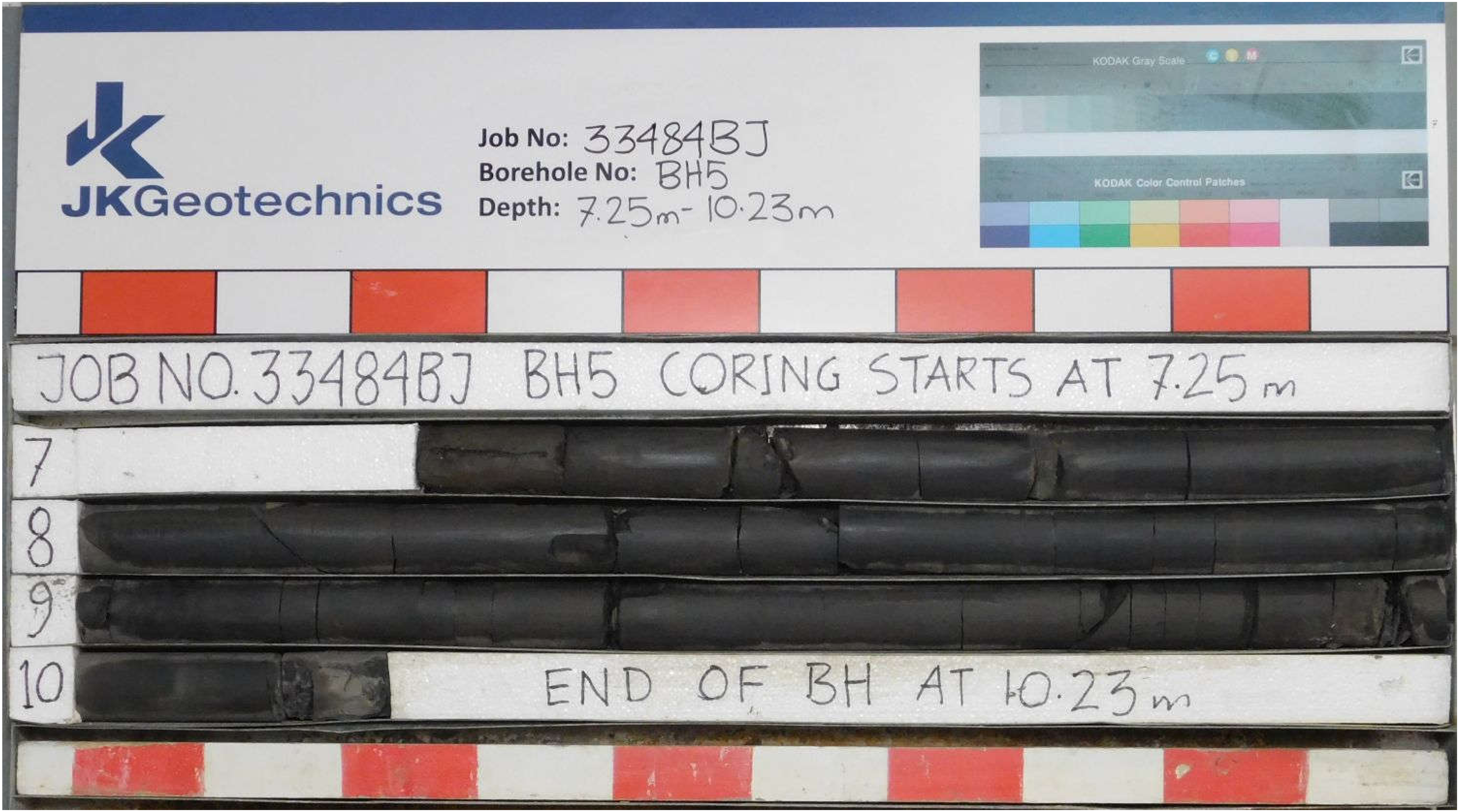
Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 7.25m						
100% RETURN		70			SILTSTONE: dark grey, indistinct rock fabric.	FR	M - H	1.1 1.6 1.6 1.0 1.1 1.0 1.0 0.90	~~(7.25m) CS, 0°, 110 mm.t~~ (7.49m) Be, 2°, Ir, S, Clay Ct (7.53m) J, 30°, Ir, S, Clay Ct (7.72m) Be, 10°, P, S, Cn (8.16m) J, 45°, P, S, Cn (8.39m) Be, 0°, Ir, S, Clay Ct (8.55m) J, 60°, Ir, S, Cn (9.39m) XWS, 0°, 12 mm.t (9.74m) J, 45°, P, S, Cn (9.84m) Be, 0°, P, S, Clay Ct (9.96m) XWS, 2°, 30 mm.t (10.19m) J, 80 - 90°, Ir, S, Cn		Ashfield Shale
		67			END OF BOREHOLE AT 10.23 m						



RL (m AHD): 70, 69, 68, 67, 66, 65, 64
Depth (m): 8, 9, 10, 11, 12, 13

COPYRIGHT

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


Job No: 33484BJ
Borehole No: BH5
Depth: 7.25m - 10.23m

JKGeotechnics

BOREHOLE LOG

Borehole No.
6
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~71.8 m
Date: 15/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON COMPLETION OF CORING					FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, fine to medium grained ironstone and siltstone gravel, trace of ash.	w<PL			GRASS COVER
	N = 20 5,9,11	71		CH	Silty CLAY: high plasticity, light grey, yellow brown and orange brown, trace of fine to medium grained ironstone and siltstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
DRY ON COMPLETION OF AUGERING	N=SPT 17/ 150mm REFUSAL		1	-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey and brown.	XW	Hd		ASHFIELD SHALE
AFTER 28 HRS ON 16/9/20		70 69	2 3		SILTSTONE: grey and brown, with medium strength ironstone bands.		VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
ON 1/10/20 ON 21/9/20		68	4				L - M		LOW TO MODERATE RESISTANCE 3.75m: ON 21/9/20 WELL PUMPED OUT TO A DEPTH OF 8.40m
		67 66 65	5 6		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.67m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 8.67m TO 2.6m. CASING 2.6m TO 0.1m. 2mm SAND FILTER PACK 8.67m TO 1.2m. BENTONITE SEAL 1.2m TO 0.5m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

SAMPLES: ES U50 DB DS

Graphic Log

CORED BOREHOLE LOG

Borehole No.
6
2 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ	**Core Size:** NMLC	**R.L. Surface:** ~71.8 m
Date: 15/9/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.A./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
					START CORING AT 4.20m						
80% RETURN					Extremely Weathered siltstone: silty CLAY, medium plasticity, brown and orange brown, with very low to low strength siltstone bands.	XW	Hd	0.080		(4.20-4.64m) SOIL RESISTANCE	Ashfield Shale
		67	5		SILTSTONE: dark grey and brown, with extremely weathered bands and very low to low strength siltstone bands, indistinct rock fabric.	HW	VL			(4.64-5.40m) NOTE: MANY FRACTURES IN ROCK	
					SANDSTONE: fine to medium grained, orange brown, indistinct rock fabric.	HW	M	0.40		(5.70m) XWS, 0°, 40 mm.t	Hawkesbury Sandstone
		66 65 64	6 7 8		SANDSTONE: fine to medium grained, light grey, with grey laminae, distinctly bedded at 0-5°.	FR	VH H - VH H	5.8 3.1 2.2 3.4 2.7 2.4 1.9	600 200 60 20	(5.95m) Cr, 0°, 20 mm.t (6.04m) Be, 0°, P, S, Cn (6.55m) Be, 3°, Ir, S, Fe Sn	
		63 62 61	9 10		END OF BOREHOLE AT 8.67 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33484BJ
Borehole No: BH6
Depth: 4.20m - 8.67m
KODAK Gray Scale
C Y M
KODAK Color Control Patches
JOB NO. 33484BJ BH6 CORING STARTS AT 4.20m
4
5
6
7
8
END OF BH AT 8.67m

BOREHOLE LOG

Borehole No.
7
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Method: SPIRAL AUGER
R.L. Surface: ~75.7 m
Date: 15/9/20
Datum: AHD
Plant Type: JK305
Logged/Checked By: B.A./J.M.

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
										FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and siltstone gravel, trace of ash and root fibres.	w<PL			GRASS COVER
					N = 9 4,5,4	75	1		CH	Silty CLAY: high plasticity, light grey and orange brown, trace of fine to medium grained ironstone and low strength siltstone gravel, trace of ash.	w<PL	VSt - Hd	440 310 290	RESIDUAL
					N = 27 11,10,17	74	2						350 380 390	
						73 72	3 4		-	SILTSTONE: grey and brown, with extremely weathered bands.	DW	VL - L		ASHFIELD SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						71	5					L		LOW RESISTANCE
ON COMPLETION OF AUGERING						70 69	6					L - M		MODERATE RESISTANCE WITH LOW BANDS

JKGeotechnics

BOREHOLE LOG

Borehole No.
7
2 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~75.7 m
Date: 15/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

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
				■		68	8	▥	-	SILTSTONE: grey and brown, with extremely weathered bands. *(continued)*	DW	L - M		
				■		67 66	9	▥		SILTSTONE: dark grey	FR	H		HIGH RESISTANCE
				■			10	░		SANDSTONE: fine to medium grained, light grey.				HAWKESBURY SANDSTONE
						65 64 63 62	11 12 13			END OF BOREHOLE AT 10.40 m				'TC' BIT REFUSAL

JKGeotechnics

BOREHOLE LOG

Borehole No.
8
1 / 2

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ **Method:** SPIRAL AUGER **R.L. Surface:** ~74.0 m
Date: 15/9/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.A./J.M.

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										FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and siltstone gravel, trace of slag, root fibres and ash.	w<PL			GRASS COVER APPEARS MODERATELY COMPACTED
					N = 8 4,3,5								260 320 150	
						73	1							
									CI	Silty CLAY: medium plasticity, light grey and yellow brown, trace of fine to medium grained siltstone gravel.	w<PL	VSt - Hd		RESIDUAL
					N = 34 4,13,21								440 600 600	
									-	Extremely Weathered siltstone: silty CLAY, medium plasticity, brown, yellow brown and grey.	XW	Hd		ASHFIELD SHALE
						72	2			SILTSTONE: grey and brown, with extremely weathered bands and medium strength ironstone and siltstone bands.	DW	VL - L		VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
▽ ON COMPLETION OF CORING														
						71	3							
						70	4							
						69	5							
										REFER TO CORED BOREHOLE LOG				
						68	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33484BJ MACQUARIEPARK.GPJ <<DrawingFile>> 20/10/2020 11:53 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

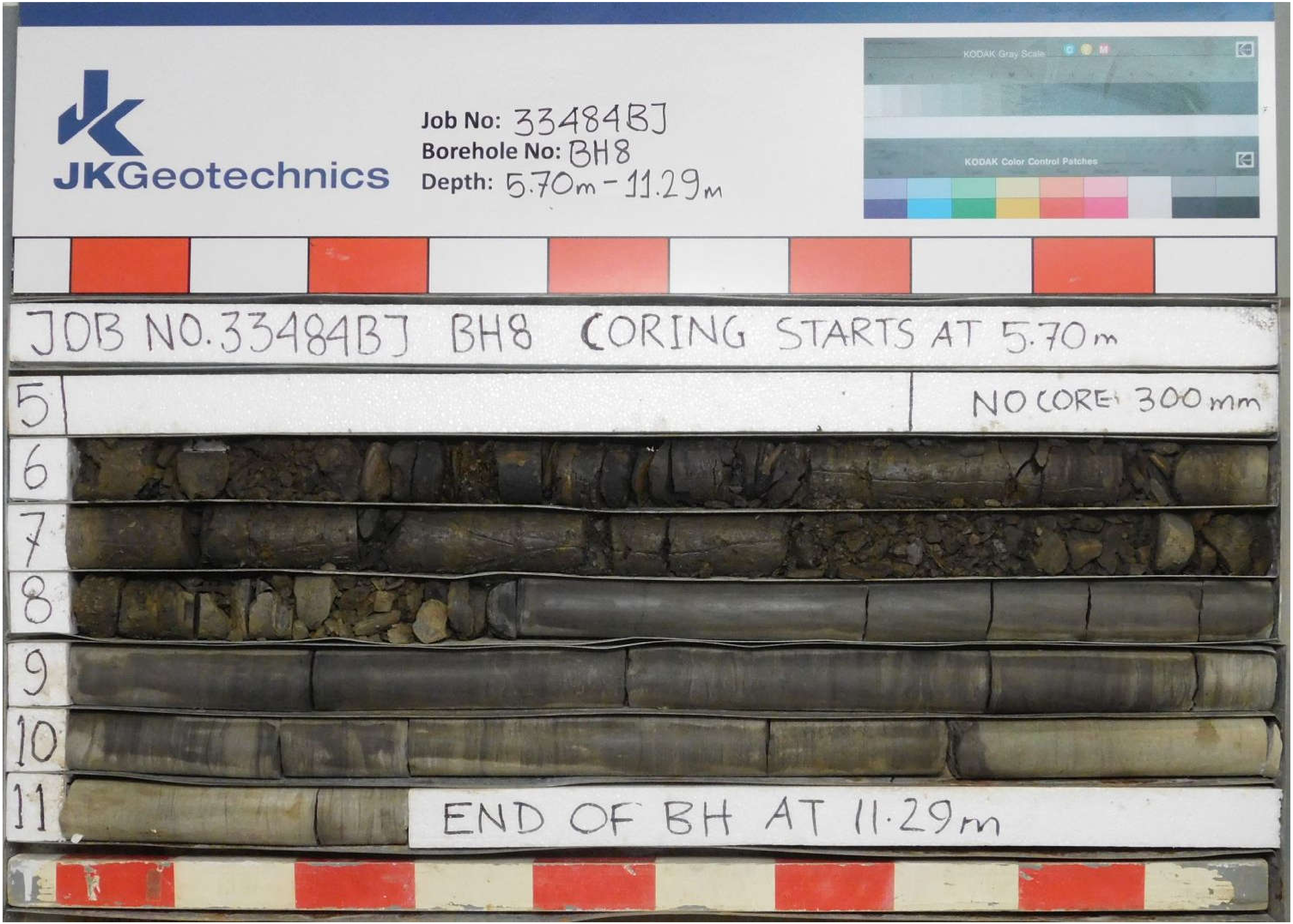
Borehole No.
8

Client: ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN
Project: PROPOSED ROYAL INSTITUTE FOR DEAF AND BLIND CHILDREN - CENTRE OF EXCELLENCE
Location: CULLODEN & GYMNASIUM ROADS, MACQUARIE PARK, NSW

Job No.: 33484BJ
Core Size: NMLC
R.L. Surface: ~74.0 m
Date: 15/9/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK305
Bearing: N/A
Logged/Checked By: B.A./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
					START CORING AT 5.70m						
80% RETURN					NO CORE 0.30m						
		68	6		SILTSTONE: grey and brown, indistinct rock fabric.	HW	L	0.30		(6.00-6.60m) MANY BEDDING PARTINGS, TYPICALLY 0°, P, S, Cn AND ROCK IS FRACTURED (6.60m) Be, 0°, P, S, Cn	Ashfield Shale
		67	7				M	0.60 0.50		(6.80m) Be, 0°, P, S, Cn (6.88m) Be, 0°, P, S, Cn (6.92m) J, 30°, P, Cn (7.10m) Be, 0°, P, Cn (7.25m) Be, 0°, P, Cn (7.28m) Jh, 30°, P, Cn	
					Extremely Weathered siltstone: silty CLAY, medium plasticity.grey and brown.	XW	Hd				
		66	8		SILTSTONE: grey and brown, indistinct rock fabric.	HW	L - M			(7.60-7.90m) ROCK IS FRACTURED (7.90m) Be, 0°, P, S, Cn (7.93m) J, 30°, P, S, Cn (8.00-8.35m) MANY BEDDING PARTINGS, TYPICALLY 0°, P, S, Cn	
		65	9		SILTSTONE: grey, indistinct rock fabric.	FR	H - VH	2.0 3.3 2.7 2.2	600 200 60 20		Ashfield Shale
		64	10		SILTSTONE: grey, with light grey laminae and fine grained sandstone laminae, distinctly bedded at 0-5°.			3.9 4.2 3.9			
		63	11		SANDSTONE: fine to medium grained, light grey, with grey laminae, distinctly bedded at 0-5°.			3.1 2.4			Hawkesbury Sandstone
					END OF BOREHOLE AT 11.29 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33484BJ
Borehole No: BH8
Depth: 5.70m - 11.29m

JOB NO. 33484BJ BH8 CORING STARTS AT 5.70m

5 NO CORE: 300mm
6
7
8
9
10
11 END OF BH AT 11·29m

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:
SITE LOCATION PLAN

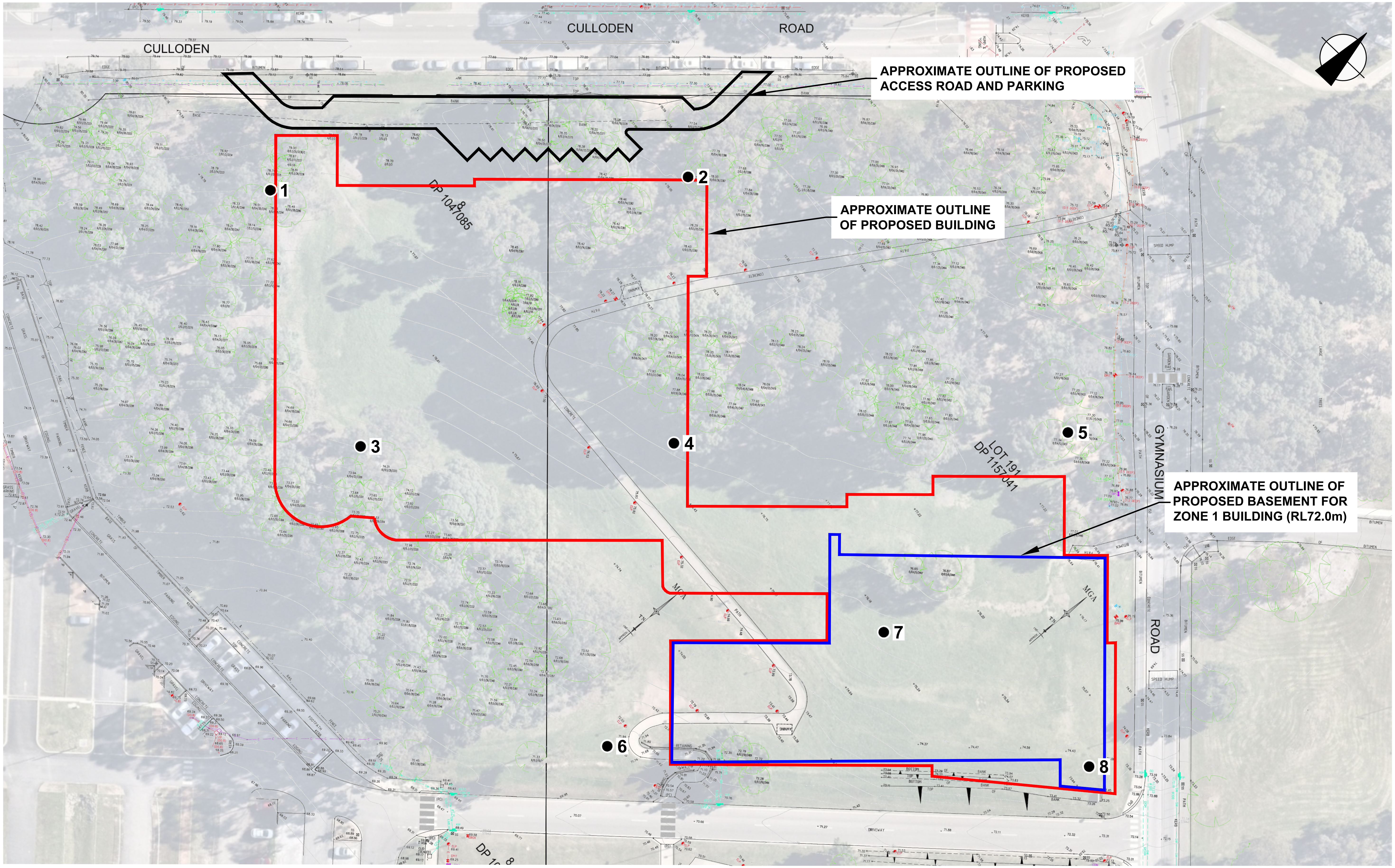
Location: CULLODEN AND GYMNASIUM ROAD,
MACQUARIE PARK, NSW

Report No: 33484BJ

Figure No: 1

JKGeotechnics

PLOT DATE: 17/09/2020 9:35:48 AM DWG FILE: Y:\33000'S\33484BJ MACQUARIE PARK\CAD\33484BJ.DWG



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 7 14 21 28 35

SCALE 1:700 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: CULLODEN AND GYMNASIUM ROAD, MACQUARIE PARK, NSW

Report No: 33484BJ

Figure No: 2

JKGeotechnics

PLOT DATE: 22/09/2020 3:31:51 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000's\33484BJ MACQUARIE PARK\CAD\33484BJ.DWG

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 1726:2017 *'Geotechnical Site Investigations'*. 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 soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

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 (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	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 fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

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 shrink-swell behaviour, 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 methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock 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 a large 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. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, 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 limited 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 cuttings. 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 assessed 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. 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, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, 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 NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom 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 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

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 63.5kg 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.

A modification to the SPT 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.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm 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 a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm 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. The CPT does not provide soil sample recovery.

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. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT 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.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface 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 terms and symbols used in preparation of the logs are defined in the following pages.

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 reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised 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 assess 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 Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. 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.

Reasonable 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.
- Details of the development that the Company could not reasonably be expected to anticipate.

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

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

SITE INSPECTION

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

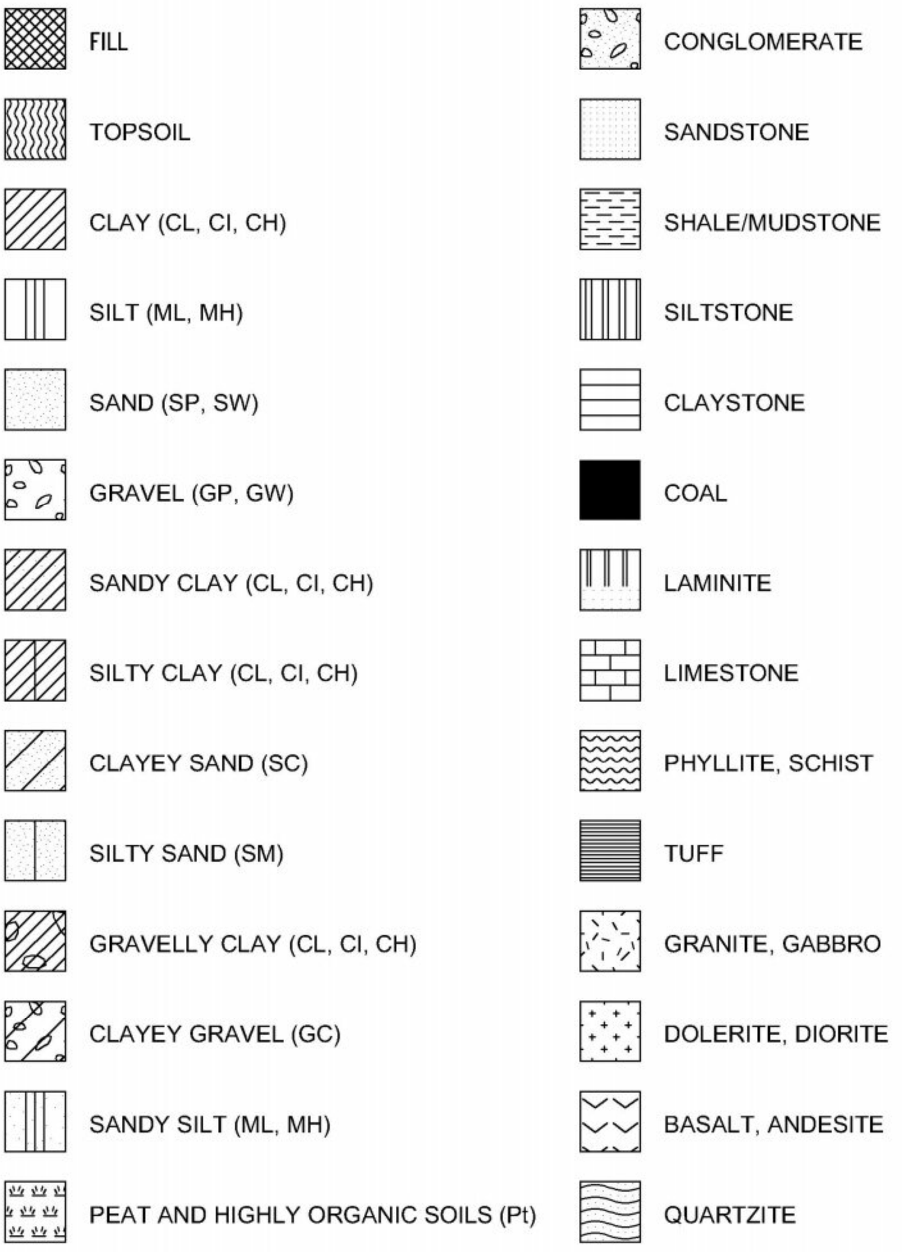
Requirements could range from:

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 and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS



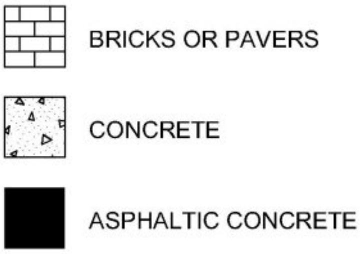
SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS



- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

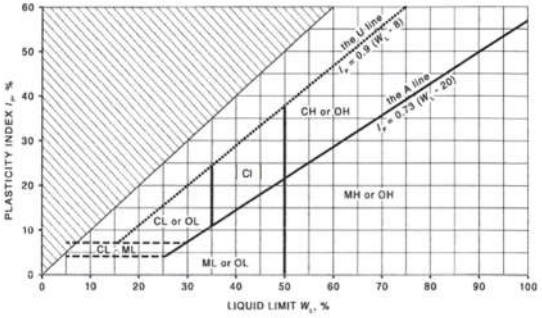
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.		
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.		
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	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 analysis.		
	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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	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° 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 (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained 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 ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
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 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

Log Column	Symbol	Definition
Remarks	‘V’ bit	Hardened steel ‘V’ shaped bit.
	‘TC’ bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description ‘slopewash’ is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index Is$_{(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

Abbreviations Used in Defect Description

Cored Borehole Log Column	Symbol Abbreviation	Description
Point Load Strength Index	• 0.6	Axial point load strength index test result (MPa)
	x 0.6	Diametral point load strength index test result (MPa)
Defect Details – Type	Be	Parting – bedding or cleavage
	CS	Clay seam
	Cr	Crushed/sheared seam or zone
	J	Joint
	Jh	Healed joint
	Ji	Incipient joint
	XWS	Extremely weathered seam
– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
– Shape	P	Planar
	C	Curved
	Un	Undulating
	St	Stepped
	Ir	Irregular
– Roughness	Vr	Very rough
	R	Rough
	S	Smooth
	Po	Polished
	Sl	Slickensided
– Infill Material	Ca	Calcite
	Cb	Carbonaceous
	Clay	Clay
	Fe	Iron
	Qz	Quartz
	Py	Pyrite
– Coatings	Cn	Clean
	Sn	Stained – no visible coating, surface is discoloured
	Vn	Veneer – visible, too thin to measure, may be patchy
	Ct	Coating ≤ 1mm thick
	Filled	Coating > 1mm thick
– Thickness	mm.t	Defect thickness measured in millimetres