REPORT TO

MACQUARIE TELECOM PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED EXTENSION TO DATA CENTRE

AT

17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Date: 21 October 2021
Ref: 31074Yrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for proposed extension to data centre at 17-23 Talavera Road, Macquarie Park, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Ms Patsy Meechan of Macquarie Telecom Pty Ltd by Purchase Order Number PO00022661, dated 14 September 2021 in accordance with our proposal (Ref. P54569Yrev dated 17 August 2021) and agreed amendments to Macquarie Telecom Supplier Terms & Conditions.

We understand that it is proposed to extend the existing data centre. In this regard the centre will be extended further to the west and north while the eastern loading bay will be upgraded. It is understood that the proposed development will be located at existing levels and any cut and fill will be minimal. The building will be heavily loaded and column loads are understood to be high. The ultimate bearing capacity of the deep fill present across the site will impact on the structural design of the building to resist earthquake loads. Where the ultimate capacity is less than 250kPa, deep beams will be required to tie the piles together. Due to the presence of inground services, if required, it will be difficult to install these beams.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at the test locations. Based on this we have provided comments and recommendations on site preparation, footing design, pavements and slabs on grade, soil aggression and earthquake parameters.

We have previously investigated the site for the extension to the data centre that is currently being constructed. The results of this investigation were presented in our geotechnical report (Ref: 31074SYrpt dated: 19 January 2018). In preparing this report we have included the relevant boreholes (BH3 to BH5) and laboratory test results.

2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 30 September and 1 October and comprised the following:

- The drilling of seven boreholes (BH6 to BH12) to depths ranging from 4.45m to 10.91m using a truck mounted JK400 drilling rig.
- All boreholes were drilled using spiral auger techniques with an attached tungsten carbide (TC) bit.
- Four of the boreholes (BH9 to BH12) were drilled to depths ranging from 4.45m to 6.45m to assess the degree of compaction of the fill while the remaining three boreholes (BH6 to BH8) were drilled to depths ranging between 9.87m and 10.91m to both assess the degree of compaction of the fill and assess the quality of the underlying sandstone bedrock.
- While spiral auger techniques were used to drill through the soils and upper weathered rock in BH6 to BH8, once better quality bedrock was encountered the boreholes were then core drilled using NMLC diamond coring techniques with water flush and core samples of the sandstone bedrock recovered.

Prior to the commencement of the fieldwork, a specialist sub-contractor reviewed available 'Dial Before You Dig' information and electro-magnetically scanned the borehole locations for buried services.

The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features shown on the survey plan prepared by HDR Pty Ltd (Project No: 10301489, Drawing No: HDR-AR-1301). The approximate surface reduced levels shown on the borehole logs were estimated by interpolation between spot levels shown on the survey plan prepared by Linker Surveying (Title No: 527/DP752035, Ref: 170621, Issue: 3, Issue Date: 21 July 2017) and consequently are approximate only. The datum of the levels is Australian Height Datum (AHD).

The strength of the subsurface soils was interpreted from the Standard Penetration Test (SPT) 'N' values and the results of hand penetrometer readings completed on the clayey samples obtained from the SPT sampler. The strength of the weathered bedrock in the augered portion of the boreholes was assessed by observation of the auger penetration resistance and examination of the recovered rock cuttings.

Where the bedrock was core drilled, the recovered rock core was returned to our NATA registered laboratory, Soil Test Services (STS), for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations, the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is_{50} results. The Point Load Strength Index test results and estimated UCS values are summarised in the attached Table C and are also plotted on the borehole logs. Colour photographs of the rock core are provided with the borehole logs. We have also included the results of Atterberg Limit tests, four-day soaked CBR tests and soil aggression tests from our earlier investigation which are presented in Tables A and B and Envirolab Services Certificate of Analysis 181199. For further details of the investigation techniques adopted, reference should be made to the attached Report Explanation Notes.

Groundwater observations were made in the boreholes and test pit during and on completion of drilling/excavation. No longer term monitoring of groundwater levels has been carried out.

Our geotechnical engineer, Mr Quang Minh Vu, was present full time during the fieldwork to set out the borehole locations, nominate the testing and sampling locations, and log the subsurface conditions encountered. The borehole logs are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

A contamination screen of site soils and groundwater was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site lies within a gently undulating topography. The proposed building extension is located on the north-western side of the existing Macquarie Telecom data centre and the area is generally level. At the time of the investigation the rear portion of the site was fenced off and was part of the construction site for the extension to the rear of the existing building. The proposed building area was predominantly covered by asphaltic concrete pavements with some site sheds and trees ranging up to about 15m located around the property boundary. The pavements appear to be in good condition.

Adjoining the site to the east is Macquarie Telecom's existing data centre which comprised a multi-level building and surrounding pavements. To the north-west, south-west and south-east of the site were multi-storey buildings and carparking structures; above ground structures were all set back at least 10m from the common boundary. The adjoining buildings appeared in good condition when viewed from the subject site.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Map of the Sydney region indicates that the site is underlain by Ashfield Shale but is located close to the geological boundary with Hawkesbury Sandstone. The investigation revealed that the site is underlain by a deep, predominantly clayey fill that in turn overlies natural silty clay and sandstone bedrock. For a more detailed description of the materials encountered at a particular location reference should be made to the attached borehole logs. The more pertinent details of the materials encountered are discussed below.

Pavements

Asphaltic concrete pavement was encountered at each borehole with thickness varying between 20mm and 40mm.

Fill

Underlying the pavements, a predominantly clayey fill was encountered that extended to depths varying from 2.9m to 6.4m. The fill was typically assessed to be moderately to well compacted, although in BH6 and BH10 the fill was assessed to be poorly compacted between 5.0m and 6.4m and 1.5m to 5.1m respectively. The silty clay fill varied from low to medium plasticity and contained varying proportions of sand and igneous and ironstone gravel. In BH12, a silty gravelly sand fill was encountered to a depth of 0.4m.

Silty Clays

Silty clays of medium or high plasticity were encountered below the fill and overlay the sandstone bedrock in BH4 to BH9. This clay layer ranged in thickness from 0.25m to 1.3m and was typically of stiff to hard strength.

Sandstone Bedrock

Sandstone bedrock was encountered at depths ranging from 4.1m to 7.2m. When first encountered the bedrock was typically extremely weathered and of hard (soil) or very low to low strength but quickly increased in strength to at least medium to high or high strength. Defects within the core typically comprised bedding partings and extremely weathered seams, although some jointing was also noted. In BH8 a "no core" zone was encountered from 4.7m to 5.26m. No core zones typically represent soil or poor quality bedrock that has been washed away during drilling. In BH6, extremely weathered siltstone bedrock of hard (soil) strength was encountered from 8.39m to 9.08m. From 9.08m to 10.08m the siltstone increased to high strength and contained fine grained sandstone laminations. High strength siltstone bedrock was also encountered in BH8 at a depth of 10.38m and was still present at a depth of 10.91m, at which depth the borehole was terminated. Siltstone bedrock was not encountered in any other boreholes.

Groundwater

With the exception of BH's 6, 7, 10, and 12, all other boreholes were dry on the completion of auger drilling. In BH6, 7, 10 and 12 groundwater seepage was measured at a depth of 6.5m, 5.7m, 4.6m and 5.8m. On the completion of drilling groundwater was measured at a depth of 5.2m in BH12. In BH1 to BH8 the boreholes were deepened using coring techniques which requires the flushing of the boreholes with water. Consequently, the measured levels in these boreholes on completion of drilling have not been recorded as these are unlikely to represent groundwater levels across the site. No longer term groundwater monitoring has been carried out.

Laboratory Test Results

The results of the Atterberg Limit and Linear Shrinkage tests indicated that the samples tested were of high plasticity and had a medium to high shrink-swell potential. The four day soaked CBR tests completed on the recovered samples returned values ranging from 1.5% to 10%. Considering the high plasticity of the fill samples tested we would expect CBR values to be in the order of about 1.5% to 3% rather than values of 5% and 10% obtained from the samples taken from BH's 1 and 3. Consequently, it is possible that the gravel content within the samples has affected the test results and we would not consider these results to be representative of suitable design values.

The point load strength index tests completed on the recovered rock core returned unconfined compressive strength (UCS) test results varying from 6MPa to 68MPa. Notwithstanding this range in values, UCS values typically fall in the range of 16 to 24MPa,

The results of the pH tests indicated that the samples tested ranged from 4.9 to 5.2 while the chloride and sulphate contents ranged from 21mg/kg to 230mg/kg and 25mg/kg to 36mg/kg respectively. The minimum resistivity of the soils was 5,700ohm.cm to 25,000ohm.cm.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

Based on the results of the boreholes the site is uniformly underlain by fill that extends to depths of 2.9m to 6.4m. This fill was assessed to be mostly moderately to well compacted, although poorly compacted fill was encountered in BH6 and BH10 from 5.0m to 6.4m and 1.0m to 5.1m respectively. BH6 and BH12 are located in close proximity to the stormwater drain and it is possible that this poorly compacted fill may be associated with the backfilling of the stormwater trench. Notwithstanding this, it generally appears that some compactive effort has been applied to the fill, although it is likely that there will be no records indicating that the fill has been placed as engineered fill and as a consequence it is considered to be uncontrolled fill.

The problem with uncontrolled fill is that the degree of compaction of the fill may be variable as is its thickness, which may result in variable settlement of the fill platform over time. As result, the long term performance of uncontrolled fill may not meet the long term expectations of the asset owner if it is used as a bearing stratum.

A number of different approaches may be adopted when considering the best design approach to adopt where uncontrolled fill is present. The best solution or approach for a particular site will be dependent on a number of criteria. These criteria are typically:

- The cost, both of undertaking the required measures to achieve an adequate degree of confidence that the fill will perform satisfactorily and of undertaking remedial measures should the structure supported on the fill fail to perform satisfactorily over its design life,
- The proposed use and required performance of the fill platform over the design life of the platform and any structures supported on it, and
- The degree of confidence that performance expectations will be met for both the platform and any structures supported on it over their design life.

A number of design approaches may be adopted. These are:

- Option 1 - Remove all uncontrolled fill and replace with engineered fill,
- Option 2 - Remove the upper portion of uncontrolled fill (say the upper 1m to 2m) and replace with engineered fill,
- Option 3 - Leave the existing fill in place, remove the existing pavements/topsoil and proof roll,
- Option 4 - Leave all uncontrolled fill in place and fully suspend all structures on the underlying sandstone bedrock and place a void former between the underside of the structure and the fill to accommodate all potential shrink/swell movements.

A deep stormwater drain runs diagonally below the proposed building. It is anticipated that within the zone of influence of this drain that the ground floor building slabs will be fully suspended and supported on the underlying sandstone bedrock. It is also anticipated that due to the relatively high floor loads that the remainder of the slab outside the zone of influence of the stormwater drain will also be fully suspended on the underlying bedrock. If this is not the case and it is proposed to construct slabs on grade consideration must be given to the costs, required performance and confidence that the slabs will perform satisfactorily over their design life.

To provide a high degree of confidence that the structures will perform as expected over their design life either Option 1 or 4 must be adopted. The costs associated with these options are anticipated to be high. Option 2 provides a greater degree of confidence than Option 3 that structures supported on the fill platform will perform satisfactorily although the presence of uncontrolled fill in the lower levels of the fill does introduce a greater level of uncertainty than in Options 1 or 4. Option 3 provides the lowest degree of confidence that the fill platform will perform satisfactorily over its design life as the only testing undertaken is limited to proof rolling of the surface of the fill platform.

While Options 2 and 3 provide lower levels of confidence than Options 1 and 4 do, past performance of the fill can also provide a good guide to how the fill platform will perform in the future. In this regard where the fill has been in place for a significant period of time (say greater than 10 years), will not be loaded to a greater extent than it already has been and has previously performed satisfactorily, then it could be inferred that it will continue to perform satisfactorily. In this case while it appears that the pavements have generally performed satisfactorily it also appears that there has been some loss of fall in at least one localised area and that design gradients to pits and drains are exaggerated. This may indicate that the original designers had some concerns regarding the long term performance of the fill and increased falls to allow for some variable settlement or consolidation of the fill. It also appears that proposed ground floor building loads will be greater than those the pavements are currently subjected to. Where this is the case past performance of the fill becomes less of an indicator that future performance will be satisfactory. Consequently, we recommend that all ground floor building slabs be fully suspended and supported on the underlying bedrock.

Where external pavements are to be constructed the current performance of the existing pavements provides a better indication of future performance as it is assumed that they will be subjected to similar traffic loadings. In this regards the existing pavements appear to generally have performed well. In addition, the design life of these pavements is much shorter than that of the ground floor slabs which helps reduce some of the risk. Similarly, the impact and cost of repairing these pavements, should they fail to perform satisfactorily, is likely to be much lower than for the ground floor slabs. Consequently, for the external pavements it would appear that the adoption of Option 3 provides a viable approach.

It should be noted that the proposed external pavements will extend over areas where existing pavements are present and also those areas currently landscaped. Consequently, to infer that the current performance of the existing pavements also provides confidence that the proposed pavements will perform satisfactorily over the landscaped areas assumes that all fill placed has been uniformly placed. To help mitigate these risks pavements should be designed with exaggerated falls so that should they settle differentially there is less chance that these falls will be lost and water will pond. In the end the asset owner must weigh the risks and costs and decide which approach they wish to take.

Should records indicate that the fill has been placed as engineered fill the subgrade should be treated in accordance with the recommendations provided below in *Subgrade Preparation*.

Subgrade Preparation

Where it is decided to adopt one of Options 1, 2 or 3 we recommend that the following subgrade preparation be completed:

- Strip all topsoil and root affected soils where present. These soils are not suitable for use as engineered fill but may be used for landscaping purposes.
- Strip as much fill as necessary, depending on the option preferred.
- Proof roll the subgrade with a minimum 8 passes using a smooth drum roller with a minimum static weight of 8 tonnes in the presence of an experienced geotechnical engineer or geotechnician. The purpose of proof rolling is to improve the near surface density of the subgrade and identify any soft or heaving areas.

- Any soft or heaving areas should be excavated down to a sound base and replaced with engineered fill.

Engineered Fill

Engineered fill should preferably comprise a granular material free from all organic or otherwise deleterious materials with a maximum particle size of no greater than 75mm. Engineered fill should be placed in loose layers of no greater than 200mm thickness and where granular materials are used should be compacted to a minimum of 98% standard maximum dry density (SMDD). Although not desirable, the clayey materials on site may be used as engineered fill provided they are compacted to between 98% and 102% of SMDD and within +/-2% of Standard Optimum Moisture Content (SOMC). 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.

Earthworks Control

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 500m^2 or three tests per visit, whichever requires the most tests. Where localised areas are backfilled, such as trenches or similar the frequency of testing should be increased to 1 test per 2 layers per 50m^2. Where the fill is to support building loads it should be placed under Level 1 control, as defined by AS3798. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.2 Footing Design

As there are no records indicating that the fill has been placed as engineered or controlled fill it should be assumed that it is uncontrolled. As a consequence the site classifies as a Class P site in accordance with AS2870-2011. The clay fill is moderately to highly reactive. It should be noted that the proposed development does not fall within the scope of AS2870 however it is useful to use the site classification to convey the ground issues affecting this site.

Due to the depth of fill present across the site and the anticipated column loads we recommend that all footings be uniformly founded on the underlying sandstone bedrock. Piles founded on sandstone bedrock of at least medium strength may be designed for an allowable bearing pressure (ABP) of 3.5MPa or 6MPa, as shown in the table below.

Depth and Reduced Level to Bedrock suitable for Allowable Bearing Pressures of 3.5MPa and 6MPa						
Allowable Bearing Pressure (MPa) *	BH3	BH4	BH5	BH6	BH7	BH8
3.5MPa	5.4m RL46.7m	6.96m RL45.24m	6.25m RL45.65m	9.38m RL42.02m	6.77m RL45.23m	6.4m RL45.6m
6MPa	5.4m RL46.7m	8.0m RL44.2m	7.4m RL44.5m	9.38m RL42.02m	6.77m RL45.23m	6.4m RL45.6m

* The above bearing pressures and depth to appropriate quality bedrock assumes that the zone of influence of the piles does not extend below the termination depth of the boreholes. In this regard the zone of influence of the piles is typically considered to extend a distance equal to twice the diameter of the pile below the toe of the pile.

All piers should have a nominal socket of 0.3m, which has been allowed for in the above table. Where piles extend below this nominal 0.3m, shaft adhesion values of 10% and 5% of the ABP may be adopted for compressive and tensile loads respectively.

It should be noted that there is an appreciable change in depth to bedrock suitable for an ABP of 3.5MPa and 6 MPa at BH6 when compared with the remainder of the site due to the presence of the extremely weathered siltstone band. We recommend that further investigation be completed to gain a better understanding of where this transition occurs. It should be noted that the diameter of the piles and design socket lengths should be known prior to the commencement of investigation as this will potentially affect drilling depths.

Due to the clayey nature of the soils and the apparent absence of groundwater it is expected that bored piers may be adopted for this site. Due to the strength of the bedrock, large powerful drill rigs with rock augers will be required. In addition, fill can be variable and some collapsing of the soils or shaft instability should not be unexpected. In this regard, piling contractors should be provided with this report and should be asked to provide advice on the suitability of bored piers for this site and the ability of their rigs to socket into the underlying sandstone bedrock.

Prior to pouring concrete all piers should be free from all loose and softened materials. Where water ponds in the base of the footings the piers should first be pumped dry and then re-excavated to removal all loose and softened materials. Alternatively, provided the base of the pier is free from all loose and softened materials tremmie methods may be used to pour the concrete where water is present in the base of the pier hole. All piers should be inspected by a geotechnical engineer prior to pouring to confirm that they are suitable for the design ABP.

Where it is proposed to fully suspend all slabs we recommend that void formers with a minimum thickness of 60mm be used below the slabs. This will allow any swelling or heave of the ground to be accommodated below the slab without jacking the slabs off the piles as insitu moisture contents trend to equilibrium.

4.3 Pavements and Slabs on Grade

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre. Flexible pavements may have a lower initial cost but maintenance costs are likely to be higher. These factors should be considered when making the final decision on pavement design. The subgrade below pavements should be carefully prepared in accordance with the recommendations given in Section 4.1 above.

We recommend that flexible pavements be designed using a CBR value of 1.5%. For concrete or rigid pavement design, the subgrade must be first improved to an equivalent modulus of subgrade reaction of at least 20kPa/mm (750mm plate). This may be completed by lime stabilisation, the use of a select fill layer or a lean mix concrete subbase layer. Where rigid pavements are adopted we recommend further advice be sought on the most efficient means of improving the existing subgrade.

Concrete pavements or slabs on grade should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements subject to traffic loadings should be supported on a sub-base layer of RTA Specification 3051 unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD. Where slabs on grade are adopted and are in contact with structural elements of the building that are supported on the underlying bedrock the slabs should be isolated from those elements to allow differential movement.

Subsoil drains should be provided on the uphill side of external pavements, with inverts not less than 0.2m below clay subgrade level. The drainage trench should be excavated with a longitudinal fall to appropriate discharge points so as to minimise the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

4.4 Soil Aggression

Reference to AS2159-2009 indicates that based on the results of the soil aggression testing that for buried concrete structures the site poses a mildly aggressive environment while for steel elements in contact with the soils it poses a non-aggressive environment.

4.5 Earthquake Parameters

The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia'.

- Hazard Factor (Z) = 0.08
- Site Subsoil Class = Class C_e.

The fill on site generally appears to be moderately or well compacted, although poorly compacted fill was encountered at BH6 and BH10 between depths of 5.0 and 6.4m and 1.0m and 5.1m respectively. Consequently, while testing completed generally indicates that the fill has been compacted it does appear that there may be some isolated zones where the degree of compaction is poor, although this appears likely to be associated with backfilling around services. Consequently, based on the number of tests completed, it appears likely that the fill will have an ultimate bearing capacity of at least 250kPa, although there is the possibility that some isolated pockets may have an ultimate bearing capacity of slightly less than this.

4.6 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:

- Where earthworks are completed proof rolling of the exposed subgrade and density testing of all engineered fill placed to confirm that the earthworks specification is complied with.
- Further advice on temporary/permanent batters and retention design where excavation is considered, particular if nearby structures will be in the zone of influence of the excavation.
- Additional boreholes where ABP's in excess of 3.5MPa are required.
- The inspection of all pier holes by a geotechnical engineer prior to pouring concrete to confirm that the design ABP's have been achieved.

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.

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.

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SOIL TEST SERVICES

ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Data Centre
Location: 17-23 Talavera Road, Macquarie Park, NSW

Ref No: 31074SY
Report: A
Report Date: 13/12/2017

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	0.50-0.95	26.0	61	21	40	14.5
2	2.70-3.15	17.6	57	23	34	14.5
5	5.70-5.80	21.3	58	24	34	15.5

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: 30/11/2017

NATA

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STS
SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Data Centre
Location: 17-23 Talavera Road, Macquarie Park, NSW

Ref No: 31074SY
Report: B
Report Date: 13/12/2017

BOREHOLE NUMBER	1	2	3	4
DEPTH (m)	0.50 - 1.00	0.50 - 1.50	0.50 - 1.50	0.50 - 1.50
Surcharge (kg)	4.5	4.5	4.5	4.5
Maximum Dry Density (t/m^3)	1.65 STD	1.70 STD	1.85 STD	1.77 STD
Optimum Moisture Content (%)	21.0	16.9	14.0	16.2
Moulded Dry Density (t/m^3)	1.63	1.67	1.81	1.75
Sample Density Ratio (%)	99	98	98	99
Sample Moisture Ratio (%)	99	99	105	102
Moisture Contents				
Insitu (%)	18.6	14.4	12.5	14.0
Moulded (%)	20.8	16.7	14.6	16.5
After soaking and After Test, Top 30mm(%)	24.7	27.1	17.8	23.8
Remaining Depth (%)	20.5	21.5	16.6	20.0
Material Retained on 19mm Sieve (%)	0	0	0	0
Swell (%)	1.0	3.0	0.0	2.0
C.B.R. value: @2.5mm penetration	5	1.5		2.0
@5.0mm penetration			10	

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample: 30/11/2017

NATA
NATA Accredited Laboratory
Number:1327

Approved Signatory / Date
13/12/17
(A. Tatikonda)

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PO Box 976
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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31074SY
Project:	Proposed Data Centre	**Report:**	C
Location:	17-23 Talavera Road, Macquarie Park, NSW	**Report Date:**	11/12/2017
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	6.92-6.95	0.8	16
	7.25-7.28	1.0	20
	7.60-7.63	0.8	16
	8.24-8.27	0.9	18
	8.71-8.75	1.1	22
	9.23-9.27	1.2	24
2	5.76-5.80	0.8	16
	6.51-6.54	0.6	12
	6.72-6.75	1.2	24
	7.08-7.11	1.0	20
	7.66-7.69	0.7	14
	8.17-8.21	1.0	20
3	5.87-5.89	0.7	14
	6.29-6.32	0.7	14
	6.79-6.82	1.0	20
	7.25-7.28	0.6	12
	7.76-7.79	0.9	18
	8.28-8.31	0.9	18
4	6.74-6.77	1.6	32
	7.21-7.25	0.8	16
	7.78-7.82	0.3	6
	8.39-8.42	1.4	28
	8.77-8.81	1.2	24
	9.26-9.30	0.7	14

NOTES: See Page 2 of 2

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TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31074SY
Project:	Proposed Data Centre	**Report:**	C
Location:	17-23 Talavera Road, Macquarie Park, NSW	**Report Date:**	11/12/2017
		Page 2 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
5	6.19-6.23	0.4	8
	6.65-6.67	0.4	8
	7.41-7.44	0.6	12
	7.89-7.92	1.1	22
	8.19-8.24	1.1	22
	8.36-8.40	0.8	16

NOTES:

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

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Macquarie Telecom Pty Ltd	**Ref No:**	31074SY
Project:	Proposed Extension to Data Centre	**Report:**	C
Location:	17-23 Talavera Road, Macquarie Park, NSW	**Report Date:**	12/10/21

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH6	7.94 - 7.97	1	20	A
	8.20 - 8.24	1.7	34	A
	8.79 - 8.81	0.4	8	A
	9.15 - 9.18	1.6	32	A
	9.60 - 9.63	1.4	28	A
	10.26 - 10.29	1.2	24	A
	10.78 - 10.81	0.9	18	A
BH7	6.91 - 6.95	1.4	28	A
	7.11 - 7.14	1.1	22	A
	7.65 - 7.68	1	20	A
	8.23 - 8.26	1.2	24	A
	8.78 - 8.81	1.2	24	A
	9.04 - 9.08	1.1	22	A
	9.71 - 9.74	1.1	22	A
BH8	5.46 - 5.49	1.6	32	A
	5.94 - 5.96	0.3	6	A
	6.25 - 6.28	0.9	18	A
	6.87 - 6.90	1	20	A
	7.11 - 7.14	1.1	22	A
	7.81 - 7.84	1.3	26	A
	8.13 - 8.16	1	20	A
	8.86 - 8.89	1.1	22	A
	9.24 - 9.27	1.1	22	A
	9.77 - 9.80	1.1	22	A
	10.09 - 10.12	1.5	30	A

NOTE: SEE PAGE 2

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Macquarie Telecom Pty Ltd	**Ref No:**	31074SY
Project:	Proposed Extension to Data Centre	**Report:**	C
Location:	17-23 Talavera Road, Macquarie Park, NSW	**Report Date:**	12/10/21

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH8	10.71 - 10.74	3.4	68	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 $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 181199

Client Details

Client	JK Geotechnics
Attention	K Singh
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31074SY**
Number of Samples	3 Soil
Date samples received	01/12/2017
Date completed instructions received	01/12/2017

Analysis Details

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

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

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

Report Details

Date results requested by	08/12/2017
Date of Issue	08/12/2017

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

David Springer, General Manager

Misc Inorg - Soil				
Our Reference		181199-1	181199-2	181199-3
Your Reference	UNITS	BH1	BH2	BH4
Depth		1.5-1.95	5.2-5.5	5.7-6.15
Date Sampled		28/11/2017	28/11/2017	28/11/2017
Type of sample		Soil	Soil	Soil
Date prepared	-	04/12/2017	04/12/2017	04/12/2017
Date analysed	-	04/12/2017	04/12/2017	04/12/2017
pH 1:5 soil:water	pH Units	4.9	5.0	5.2
Chloride, Cl 1:5 soil:water	mg/kg	21	230	38
Sulphate, SO4 1:5 soil:water	mg/kg	46	46	25
Resistivity in soil*	ohm m	220	57	250

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.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			04/12/2017	[NT]	[NT]	[NT]	[NT]	04/12/2017	[NT]
Date analysed	-			04/12/2017	[NT]	[NT]	[NT]	[NT]	04/12/2017	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	101	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	100	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[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.

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 3

BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** ~52.1 m
Date: 27/11/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./W.T.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		0		FILL: Silty sand, fine to medium grained, brown, with clay and roots and root fibres.	M			GRASS COVER
	N = 12 7,6,6	~0.2		FILL: Silty clay, high plasticity, grey and brown, with fine to coarse grained ironstone and shale gravel and fine to coarse grained sand.	MC<PL	(St - VSt)		TOO FRIABLE FOR HP TESTING APPEARS MODERATELY TO WELL COMPACTED
	N = 7 2,2,5							
	N = 9 3,4,5							
	N = 12 2,6,6							
		~5.1	-	SANDSTONE: fine to coarse grained, yellow brown.	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
				REFER TO CORED BOREHOLE LOG				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

CORED BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Core Size:** NMLC **R.L. Surface:** ~52.1 m
Date: 27/11/17 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** K.S./W.T.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		47			START CORING AT 5.70m					
		46	6		SANDSTONE: fine to coarse grained, grey.	SW - FR	M			$I_{S(50)}(A) = 0.7MPa$; (6.03m) FRAGMENTED ZONE, 0°, 55 mm.t; $I_{S(50)}(A) = 0.7MPa$; (6.64m) Be, 0°, P, S, IS; $I_{S(50)}(A) = 1MPa$
		45	7							$I_{S(50)}(A) = 0.6MPa$; $I_{S(50)}(A) = 0.9MPa$
		44	8							$I_{S(50)}(A) = 0.9MPa$; (8.54m) Be, 0°, P, S, IS
		43	9		END OF BOREHOLE AT 8.65 m					
		42	10							
		41	11							

Point load scale: EL -0.03, VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Defect spacing scale: 500, 300, 100, 50, 30, 10


JK Geotechnics
31074SY
BH3
START CORING AT 5.70m
5
6
7
8
EOH AT 8.65m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
1 / 2

BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** ~52.2 m
Date: 28/11/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./W.T.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING							ASPHALTIC CONCRETE: 50mm.t				
			52				FILL: Silty gravelly sand, fine to coarse grained, dark grey.	MC<PL			
		N = 15 7,7,8 N = 10 4,5,5 N = 15 7,8,7	51 50 49	1 2 3			FILL: Silty clay, high plasticity, orange brown and light grey, with fine to coarse grained ironstone and shale gravel and fine grained sand.				APPEARS WELL COMPACTED
		N = 11 3,5,6	48	4			FILL: Silty clay, low to medium plasticity, dark brown, with fine to coarse grained shale, igneous and ironstone gravel, and timber.	MC~PL			
			47	5		CH	SILTY CLAY: high plasticity, grey mottled red brown, with fine to coarse grained ironstone gravel.	MC<PL	(St - VSt)		RESIDUAL
		N = 13 4,6,7	46	6		CL	SANDY CLAY: medium plasticity, light grey.		VSt	260 260 200	
							SANDSTONE: fine to coarse grained, light grey.	DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE
							REFER TO CORED BOREHOLE LOG				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
2 / 2

CORED BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD

Project: PROPOSED DATA CENTRE

Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY	**Core Size:** NMLC	**R.L. Surface:** ~52.2 m
Date: 28/11/17	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** K.S./W.T.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		46			START CORING AT 6.66m						
			7		SANDSTONE: fine to coarse grained, grey and orange brown.	DW	M - H			(6.90m) J, 80 - 90°, Un, S	$I_{S(50)}(A)$ = 1.6MPa
		45								(7.19m) Be, 0°, P, S	$I_{S(50)}(A)$ = 0.8MPa
										(7.30m) XWS, 0°, 68 mm.t	
										(7.40m) Be, 10°, P, S	
										(7.48m) XWS, 5°, 6 mm.t	
										(7.60m) J, 90°, Un, S	
										(7.82m) Be, 5°, P, S	$I_{S(50)}(A)$ = 0.3MPa
										(7.88m) XWS, 0°, 16 mm.t	
			8							(7.91m) XWS, 0°, 26 mm.t	
		44								(8.24m) J, 70°, P, S, HEALED	
											$I_{S(50)}(A)$ = 1.4MPa
											$I_{S(50)}(A)$ = 1.2MPa
			9							(8.94m) Be, 0°, P, S	
		43								(9.10m) XWS, 0°, 8 mm.t	
										(9.20m) XWS, 0°, 14 mm.t	$I_{S(50)}(A)$ = 0.7MPa
										(9.46m) Be, 0°, P, S	
										(9.48m) XWS, 0°, 6 mm.t	
					END OF BOREHOLE AT 9.56 m						
			10								
		42									
			11								
		41									
			12								
		40									


JK Geotechnics
31074SY
BH4
START CORING AT 6.66m
6
7
8
9
EOH AT 9.56m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
5
1 / 2

BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** ~51.9 m
Date: 28/11/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.S./W.T.

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	ES			0			ASPHALTIC CONCRETE: 50mm.t				
							FILL: Silty gravelly sand, fine to coarse grained, dark grey, fine to coarse grained igneous gravel.	M			
	ES, DS	N = 17 7,9,8	51				FILL: Silty clay, high plasticity, brown, orange brown and grey, with fine to coarse grained sand and fine to coarse grained ironstone gravel.	MC<PL			APPEARS MODERATELY TO WELL COMPACTED
				1							
	ES, DS	N = 7 2,3,4	50	2							
ON COMPLETION OF CORING	ES, DS	N = 7 2,3,4	49	3							
			48	4			FILL: Silty clay, low to medium plasticity, dark brown, with fine to coarse grained sand and fine to coarse grained ironstone gravel.	MC~PL			APPEARS WELL COMPACTED GROUNDWATER MONITORING WELL INSTALLED TO 8.7m, CLASS 18 MACHINE SLOTTED 50mm PVC STANDPIPE 5.7m TO 8.7m, CASING 0m TO 5.7m, 2mm SAND FILTER PACK 4m TO 8.7m, BENTONITE SEAL FROM 1m TO 4m, BACKFILLED WITH SAND TO SURFACE AND COMPLETED WITH A CONCRETED GATIC COVER
	ES, DS	N = 14 3,6,8	47	5							
	ES, DS	N=SPT 5/ 100mm REFUSAL	46			CH	SILTY CLAY: high plasticity, brown mottled orange brown, with fine to coarse grained ironstone gravel.	MC~PL	St - VSt	110 170 210	
				6			REFER TO CORED BOREHOLE LOG				
			45								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Client:	MACQUARIE TELECOM PTY LTD
Project:	PROPOSED DATA CENTRE
Location:	17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY	**Core Size:** NMLC	**R.L. Surface:** ~51.9 m
Date: 28/11/17	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** K.S./W.T.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DETAILS DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
		46			START CORING AT 5.95m						
100% RETURN		45 – 43	6 – 8.90	Sandstone	SANDSTONE: fine to coarse grained, grey and orange brown.	DW	M - H			(6.00m) J, 90°, P, S, HEALED (6.13m) XWS, 0°, 10 mm.t (6.24m) J, 20°, P, S (6.37m) Be, 0°, P, S (6.38m) XWS, 0°, 42 mm.t (6.54m) XWS, 0°, 3 mm.t (6.75m) J, 20°, P, S (6.77m) Be, 0°, P, S (7.06m) XWS, 0°, 14 mm.t (7.12m) Be, 0°, P, S (7.21m) Be, 0°, P, S (7.32m) XWS, 0°, 26 mm.t (7.55m) Be, 0°, P, S (7.57m) Be, 0°, P, S	$I_{S(50)}(A) = 0.4MPa$ $I_{S(50)}(A) = 0.4MPa$ $I_{S(50)}(A) = 0.6MPa$ $I_{S(50)}(A) = 1.1MPa$ $I_{S(50)}(A) = 1.1MPa$ $I_{S(50)}(A) = 0.8MPa$
		43 – 40	9 – 11		END OF BOREHOLE AT 8.90 m						


JK Geotechnics
31074SY BH5 START CORING AT 5.95m
5
6
7
8
END OF BH5 AT 8.90m

JKGeotechnics

BOREHOLE LOG

Borehole No.
6

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 51.4 m
Date: 30/9/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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
				0		-	ASPHALTIC CONCRETE: 20mm.t	w<PL			APPEARS WELL COMPACTED
			51				FILL: Silty gravelly clay, low plasticity, brown, orange brown and grey, with fine to coarse grained sand, fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.				
		N = 21 6,10,11								>600 >600	
			50	1							
		N = 17 8,9,8								>600 >600	
			49	2							
							FILL: Silty clay, low to medium plasticity, brown and light grey, with fine to coarse grained sand, fine to coarse grained igneous and ironstone gravel.				
		N = 8 5,4,4	48	3							APPEARS MODERATELY COMPACTED SPT SAMPLE TOO FRIABLE FOR HP TESTING
			47	4			as above, but medium plasticity.	w~PL			
		N = 7 2,3,4								330 250 400	
ON COMPLETION			46	5			FILL: Silty clay, high plasticity, dark grey, with fine to medium grained ironstone gravel, trace of ash and root fibres.				APPEARS POORLY COMPACTED
		N > 2 0,1,1/ 50mm REFUSAL		6						70 80 50	
			45			CH	Silty CLAY: high plasticity, orange brown and light grey, with occasional ironstone bands and extremely weathered sandstone bands.	w>PL	(VSt)		RESIDUAL

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 31074SY1 MACQUARIEPARK.GPJ <<DrawingFile>> 21/10/2021 16:13 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

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BOREHOLE LOG

Borehole No.
6

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 51.4 m
Date: 30/9/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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
									CH	Silty CLAY: as above	w>PL	(VSt)		
					N=SPT 9/ 100mm REFUSAL	44			-	Extremely Weathered sandstone: silty CLAY, medium plasticity, yellow brown and light grey, with fine to medium grained sand.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
							8			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
						43								
							9							
						42								
							10							
						41								
							11							
						40								
							12							
						39								
							13							
						38								

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CORED BOREHOLE LOG

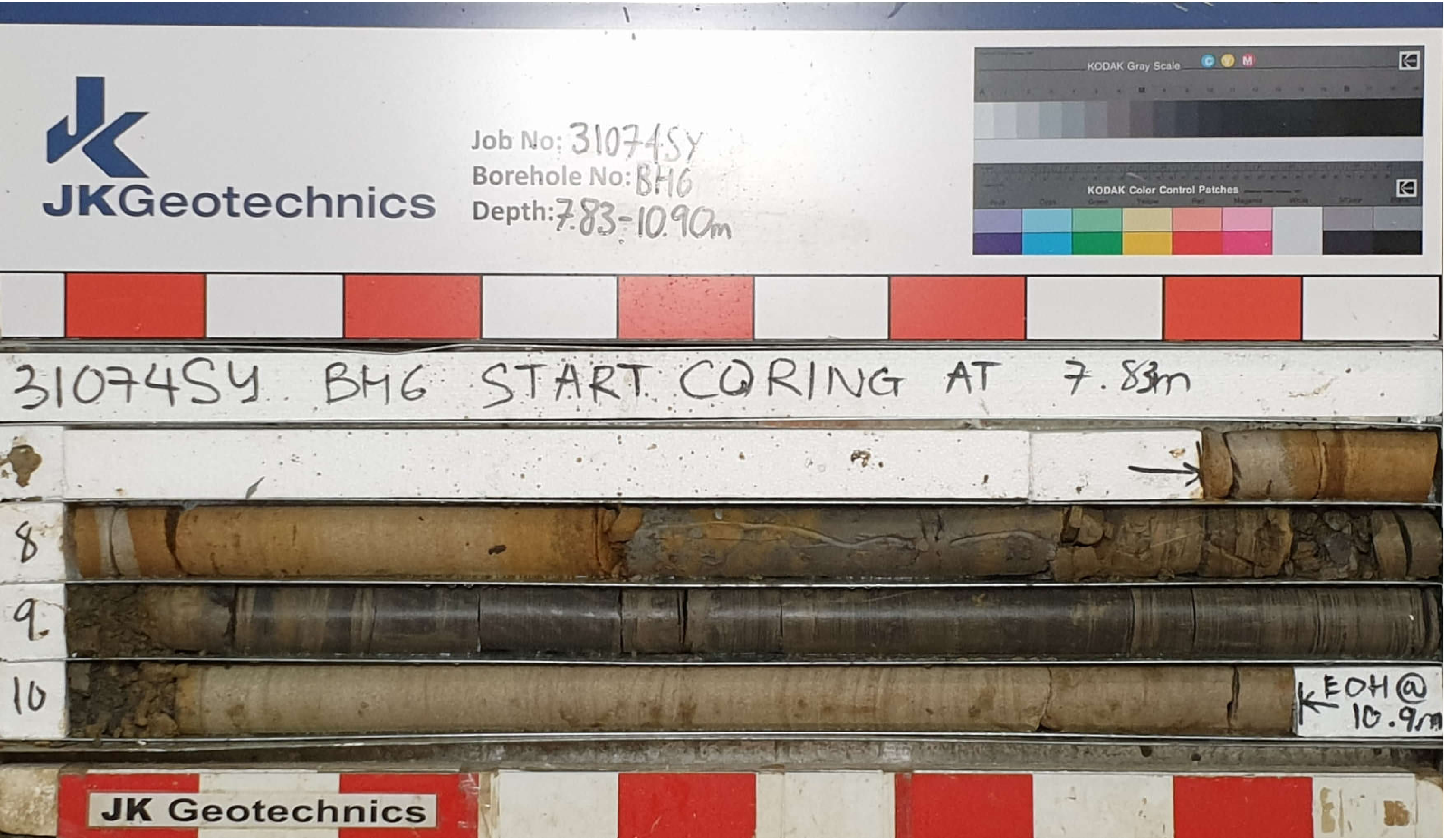
Borehole No.
6

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Core Size:** NMLC **R.L. Surface:** 51.4 m
Date: 30/9/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK400 **Bearing:** N/A **Logged/Checked By:** Q.V./

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
		44			START CORING AT 7.83m							
100% RETURN		43	8		SANDSTONE: fine to medium grained, orange brown and grey.	MW	H	1.0, 1.7		(8.06m) Be, 5°, P, S, Cn		Hawkesbury Sandstone
					Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey mottled brown, with occasional low strength sandstone bands.	XW	Hd	0.40		(8.39m) Be, 5°, P, S, Cn		
		42	9		SILTSTONE: dark grey, with grey and light grey fine grained sandstone laminae, bedded at 0-20°.	SW	H	1.6, 1.4		(9.13m) J, 45°, P, S, Cn (9.44m) XWS, 0°, 13 mm.t		
		41	10		SANDSTONE: fine to medium grained, light grey.			1.2, 0.90		(10.00m) Cr, 0°, 75 mm.t (10.71m) J, 40°, P, S, Cn		
		40	11		END OF BOREHOLE AT 10.90 m							
		39	12									
		38	13									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 31074SY
Borehole No: BH6
Depth: 7.83-10.90m
KODAK Gray Scale
KODAK Color Control Patches
31074SY BH6 START CORING AT 7.83m
8
9
10
EOH @ 10.9m
JK Geotechnics

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BOREHOLE LOG

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 52.0 m
Date: 1/10/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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
									-	ASPHALTIC CONCRETE: 30mm.t	w<PL			APPEARS MODERATELY COMPACTED
										FILL: Silty clay, low plasticity, dark brown and grey, with fine to medium grained igneous and ironstone gravel, trace of fine to medium grained sand, and root fibres.				
				■	N = 8 4,4,4								500 450 530	
				■	N = 5 3,2,3	51	1			as above, but low to medium plasticity, brown, red brown and grey.	w~PL		350 340 350	
				■	N = 5 2,3,2	50	2						400 350 420	
				■	N = 5 3,3,2	49	3			FILL: Silty clay, medium plasticity, brown, orange brown and light grey, with fine to medium grained igneous and ironstone gravel, trace of fine to medium grained sand.			280 250 250	
ON COMPLETION				■	N = 11 5,5,6	48	4			as above, but with coarse grained ironstone gravel.	w>PL		450 450 400	APPEARS MODERATELY COMPACTED
									CI-CH	Silty CLAY: medium to high plasticity, red brown mottled light grey, with fine to medium grained ironstone gravel.	w>PL	(VSt)		RESIDUAL
				■	N = 14 13,6,8	47	5		-	Extremely Weathered sandstone: sandy CLAY, medium plasticity, red brown, yellow brown and light grey, with iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
				■		46	6			SANDSTONE: fine to medium grained, light grey and yellow brown.	DW	M		MODERATE RESISTANCE
										REFER TO CORED BOREHOLE LOG				MODERATE TO HIGH RESISTANCE

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CORED BOREHOLE LOG

Borehole No.
7

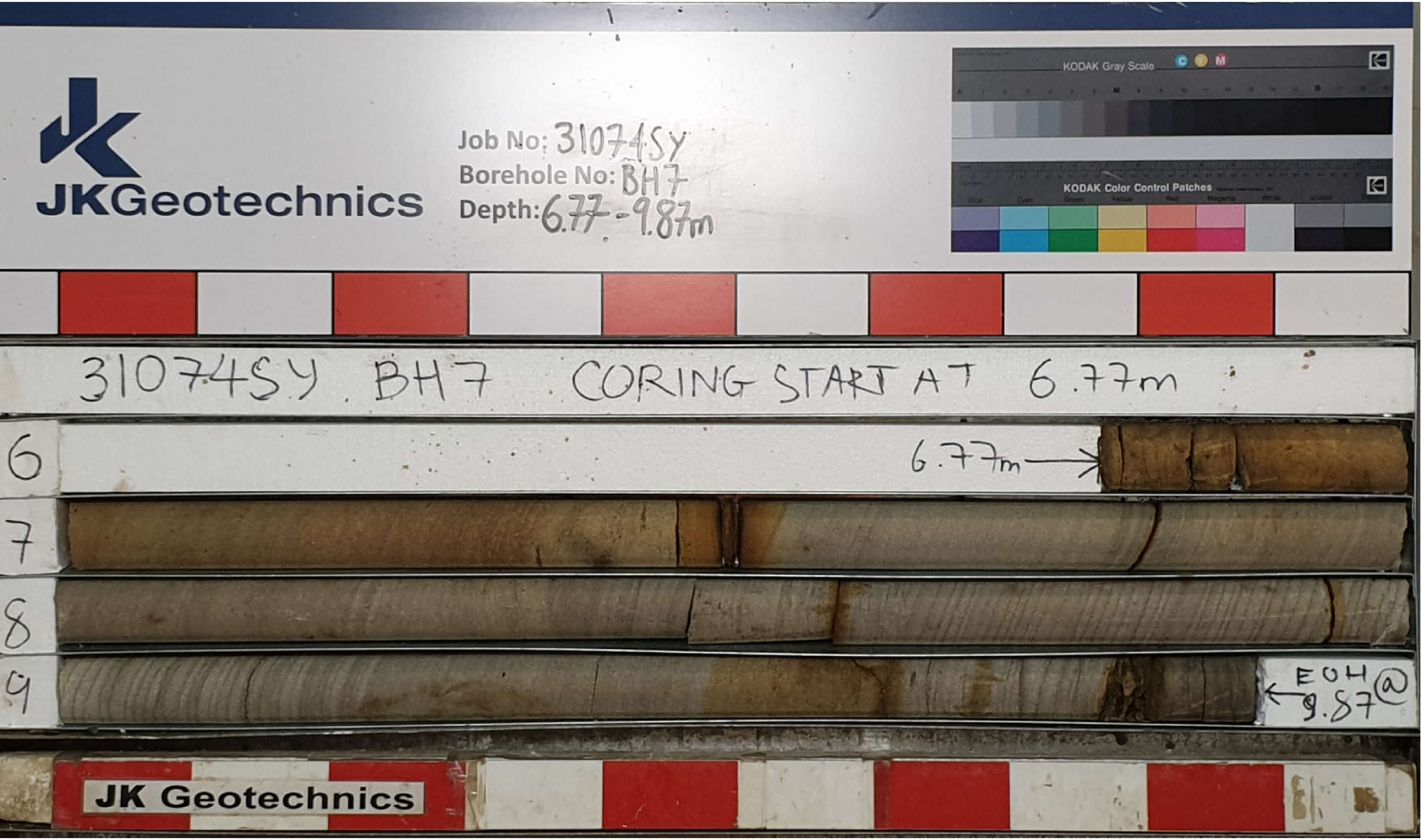
Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY | **Core Size:** NMLC | **R.L. Surface:** 52.0 m
Date: 1/10/21 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK400 | **Bearing:** N/A | **Logged/Checked By:** Q.V./

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 6.77m					
		45	7		SANDSTONE: fine to medium grained, brown and orange brown, with iron indurated bands.	MW	H	1.4 1.1	(6.86m) Be, 5°, P, S, Cn (6.88m) Be, 5°, P, S, Cn	Hawkesbury Sandstone
		44 43	8 9		SANDSTONE: fine to medium grained, light grey, bedded at 0-20°.	SW		1.0 1.2 1.2 1.1 1.1	(7.46m) Be, 0°, P, S, Cn (7.49m) Be, 0°, P, S, Cn (7.80m) J, 20°, P, S, Cn (8.57m) Be, 5°, P, S, Cn (8.93m) Be, 0°, P, S, Cn (9.60m) J, 50°, Un, S, Cn (9.77m) Cr, 0°, 30 mm.t	Hawkesbury Sandstone
		42 41 40	10 11 12		END OF BOREHOLE AT 9.87 m					

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 31074SY
Borehole No: BH7
Depth: 6.77-9.87m
KODAK Gray Scale
KODAK Color Control Patches
31074SY BH7 CORING START AT 6.77m
6
6.77m
7
8
9
EOH @ 9.87
JK Geotechnics

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BOREHOLE LOG

Borehole No.
8

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 52.0 m
Date: 1/10/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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
						-	ASPHALTIC CONCRETE: 40mm.t				
							FILL: Silty clay, low to medium plasticity, red brown, grey and light grey, with fine to coarse grained igneous and ironstone gravel, trace of fine to medium grained sand, and root fibres.	w<PL			APPEARS WELL COMPACTED
		N = 20 4,7,13								>600 >600	
			51	1			as above, but low plasticity.				
		N = 22 5,11,11								>600 >600	
			50	2			FILL: Silty clay, medium plasticity, red brown, grey and light grey, with fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.	w~PL			
		N = 14 3,7,7								>600 >600	
			49	3		CI	Silty CLAY: medium plasticity, red brown mottled light grey, with occasional iron indurated bands, trace of fine grained sand.	w~PL	Hd		RESIDUAL
		N = 23 8,12,11								>600 >600	
			48	4							
ON COMPLETION						-	Extremely Weathered sandstone: silty CLAY, medium plasticity, red brown mottled light grey, with occasional iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT RESISTANCE WITH VERY LOW BANDS
							REFER TO CORED BOREHOLE LOG				
			47	5							
			46	6							

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CORED BOREHOLE LOG

Borehole No.
8

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY	**Core Size:** NMLC	**R.L. Surface:** 52.0 m
Date: 1/10/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK400	**Bearing:** N/A	**Logged/Checked By:** Q.V./

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)	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 4.70m						
100% RETURN		47	5		NO CORE 0.56m						Hawkesbury Sandstone
					Extremely Weathered sandstone: silty CLAY, high plasticity, red brown, with fine to medium grained ironstone gravel.	XW	Hd			(5.44m) Be, 0°, P, S, Cn (5.50m) Be, 0°, P, S, Cn (5.59m) Be, 0°, P, S, Cn (5.64m) Be, 0°, P, S, Cn	
		46	6		SANDSTONE: fine to medium grained, red brown and grey, sub-horizontal, with iron indurated bands.	HW	L - M	1.6 0.30		(5.84m) Be, 0°, P, S, Cn (6.05m) Cr, 0°, 10 mm.t (6.08m) Cr, 0°, 30 mm.t (6.17m) Be, 0°, P, S, Cn	
		45	7		SANDSTONE: fine to medium grained, light grey, bedded at 20°.	SW	H	0.90 1.0 1.1		(7.49m) Be, 20°, P, S, Cn	
		44	8					1.3 1.0			
		43	9					1.1 1.1		(8.80m) Be, 0°, P, S, Clay Ct	
		42	10					1.1 1.5		(10.31m) Be, 0°, P, S, Cn (10.39m) Be, 0°, P, S, Cn	
					SILTSTONE: dark grey mottled grey and light grey, fine grained sandstone laminae, bedded at 0-10°.			3.4			
					END OF BOREHOLE AT 10.91 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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Job No: 31074SY
Borehole No: BH8
Depth: 4.70-10.91m
KODAK Gray Scale
KODAK Color Control Patches
31074 SY BH8 START CORING AT 4.7m.
4
4.7m → NO CORE: 0.56m
5
6
7
8
9
10
EOH @ 10.91m
JK Geotechnics

JKGeotechnics

BOREHOLE LOG

Borehole No.
9
1 / 1

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 52.0 m
Date: 1/10/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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						-	ASPHALTIC CONCRETE: 30mm.t				APPEARS WELL COMPACTED
							FILL: Silty clay, low to medium plasticity, brown, red brown and grey, with fine to medium grained igneous and ironstone gravel, trace of fine to coarse grained sand, and root fibres.	w<PL			
	DS	N = 12 6,5,7								>600 >600	
	DS	N = 17 5,7,10	51	1			as above, but brown and grey.			>600 >600	
	DS	N = 8 5,4,4	50	2						520 550 480	APPEARS MODERATELY COMPACTED
							FILL: Silty clay, high plasticity, dark grey and dark brown, with fine to medium grained igneous gravel and as, trace of root fibres.	w>PL			
	DS	N = 5 2,2,3	49	3						80 70 50	
						CI	Silty CLAY: medium plasticity, orange brown, red brown mottled light grey, with occasional iron indurated bands.	w>PL	VSt		RESIDUAL
	DS	N = 16 6,7,9	48	4						350 330 350	
							END OF BOREHOLE AT 4.45 m				
			47	5							
			46	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 31074SY1 MACQUARIEPARK.GPJ <<DrawingFile>> 21/10/2021 16:14 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

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BOREHOLE LOG

Borehole No.
10
1 / 1

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 52.0 m
Date: 30/9/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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
			52.0	0		-	ASPHALTIC CONCRETE: 30mm.t				
							FILL: Silty gravelly clay, low plasticity, brown and dark grey, with fine to coarse grained igneous and ironstone gravel, trace of fine to coarse grained sand, and root fibres.	w<PL	Hd		APPEARS WELL COMPACTED
		N = 21 15,11,10									
		N = 5 5,3,2	51	1							
											APPEARS POORLY COMPACTED
							FILL: Silty clay, medium plasticity, dark grey and orange brown, with fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.	w~PL			
		N = 4 2,2,2	50	2							
		N = 3 1,1,2	49	3			as above, but high plasticity, dark grey, light grey and orange brown.	w>PL			
		N = 3 1,2,1	48	4			FILL: Silty clay, high plasticity, dark grey, with fine grained igneous and ironstone gravel, trace of ash and root fibres.				
		N > 9 6,9/ 150mm REFUSAL	47	5		-	Extremely Weathered sandstone: silty CLAY, medium plasticity, orange brown.	XW			HAWKESBURY SANDSTONE
							END OF BOREHOLE AT 5.30 m				
			46	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 31074SY1 MACQUARIEPARK.GPJ <<DrawingFile>> 21/10/2021 16:14 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

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BOREHOLE LOG

Borehole No.
11

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 51.9 m
Date: 30/9/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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				0		-	ASPHALTIC CONCRETE: 20mm.t	w<PL			APPEARS WELL COMPACTED
		N = 19 6,8,11	51				FILL: Silty clay, low plasticity, brown, red brown, light grey, with fine to medium grained igneous and ironstone gravel, trace of fine grained sand, and root fibres.			>600 >600	
		N = 17 5,8,9		1			as above, but with fine to coarse grained igneous gravel.			>600 >600	
		N = 11 3,5,6	50	2			FILL: Silty clay, low plasticity, red brown mottled light grey and brown, with fine to medium grained igneous and ironstone gravel, trace of root fibres.			>600 >600	
		N = 7 4,4,3	49	3			as above, but medium plasticity.	w~PL		450 420 390	APPEARS MODERATELY COMPACTED
		N = 7 3,3,4	48	4			FILL: Silty clay, medium to high plasticity, red brown, brown and grey, with fine to medium grained igneous and ironstone gravel, trace of root fibres.			250 320 330	
		N = 5 2,2,3	47	5			FILL: Silty clay, high plasticity, dark grey and brown, with fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.			200 190 210	
						-	Extremely Weathered sandstone: silty SAND, fine to medium grained, red brown.	XW	Hd		HAWKESBURY SANDSTONE
			46				SANDSTONE: fine to medium grained, light grey.	DW	M		VERY LOW TO LOW 'TC' BIT RESISTANCE
				6			END OF BOREHOLE AT 6.00 m				MODERATE RESISTANCE
			45								

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BOREHOLE LOG

Borehole No.
12
1 / 1

Client: MACQUARIE TELECOM PTY LTD
Project: PROPOSED EXTENSION TO DATA CENTRE
Location: 17-23 TALAVERA ROAD, MACQUARIE PARK, NSW

Job No.: 31074SY **Method:** SPIRAL AUGER **R.L. Surface:** 51.6 m
Date: 30/9/21 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** Q.V.

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						-	ASPHALTIC CONCRETE: 20mm.t	M			
							FILL: Silty gravelly sand, fine to coarse grained, dark brown, with fine to medium grained igneous and ironstone gravel.				
	DS	N = 15 6,7,8	51				FILL: Silty clay, low plasticity, brown, grey and orange brown, with fine to medium grained igneous and ironstone gravel, trace of fine to medium grained sand.	w<PL		>600 >600	APPEARS WELL COMPACTED
	DS	N = 7 4,3,4	50	1						>600 >600	APPEARS MODERATELY COMPACTED
	DS	N = 4 2,1,3	49	2			as above, but medium plasticity.			250 350 300	
	DS	N = 7 3,4,3	48	3			FILL: Silty clay, medium plasticity, orange brown, red brown and grey, with fine to medium grained igneous and irons tone gravel, trace of fine to medium grained sand, and root fibres.			300 320 250	
	DS	N = 5 1,2,3	47	4							
ON COMPLETION	DS	N = 4 1,2,2	46	5			FILL: Silty clay, high plasticity, grey, with fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.	w>PL		70 50 80	APPEARS MODERATELY COMPACTED
	DS	N = 7 2,3,4		6		-	Extremely Weathered sandstone: clayey SAND, fine to medium grained, grey mottled yellow brown.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
			45				END OF BOREHOLE AT 6.45 m				

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 31074SY1 MACQUARIEPARK.GPJ <<DrawingFile>> 21/10/2021 16:14 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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title:

SITE LOCATION PLAN

Location:	17-23 TALAVERA ROAD, MACQUARIE PARK, NSW
Report No: 31074SY	Figure No: 1

JKGeotechnics

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

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0 10 20 30 40 50
SCALE 1:1000 @A3 METRES

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

Title:	BOREHOLE LOCATION PLAN
Location:	17-23 TALAVERA ROAD, MACQUARIE PARK, NSW
Report No: 31074SY	Figure No: 2

JKGeotechnics

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REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

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

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

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

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

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

SAMPLING

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

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

INVESTIGATION METHODS

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

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

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

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

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

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from “feel” and rate of penetration.

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

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

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, “Methods of Testing Soils for Engineering Purposes” – Test F3.1.

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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.

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

The information provided on the charts comprise:

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

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

Correlations between 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.

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

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed <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 a senior geotechnical engineer.

SITE INSPECTION

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

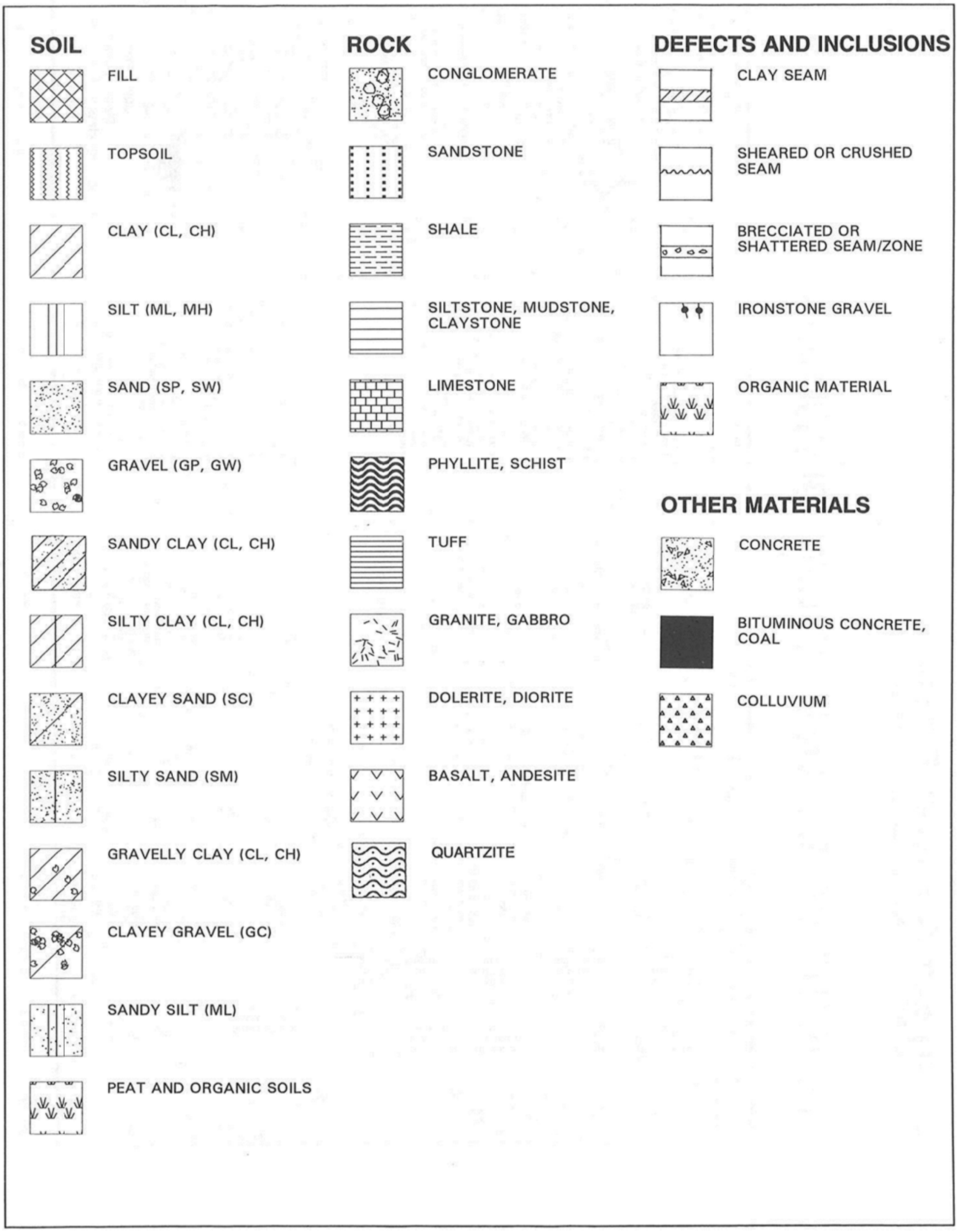
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

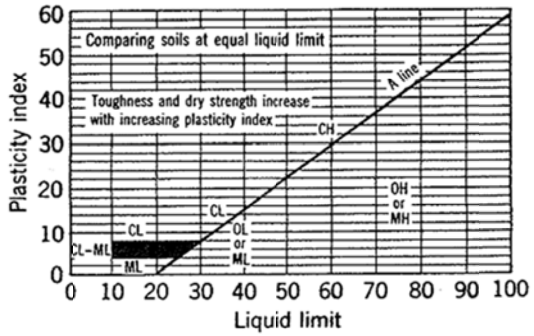


(The 75 μm sieve size is about the smallest particle visible to naked eye)

Major Divisions		Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] — Gravels: More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
		Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*	
	Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
		Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7	
Sands: More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
		Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*	
	Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
		Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7	

Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols

Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
	High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

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

LOG SYMBOLS

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

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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