REPORT TO

ROBSEA NOMINEES PTY LTD & BILGOLA BEACH PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

1-5 RAINBOW ROAD, MITTAGONG, NSW

Date: 9 September 2022

Ref: 35082BTrptRev1

JKGeotechnics

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
35082BTrpt	Final Report	13 July 2022
35082BTrptRev1	Updated Drawing References	9 September 2022

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 1-5 Rainbow Road, Mittagong, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Andrew Reeves of TBG Constructions Pty Ltd by signed 'Acceptance of Proposal' form dated 28 April 2022. The commission was on the basis of our fee proposal, Ref. P56432AT dated 20 April 2022.

We understand from the supplied architectural drawings prepared by Coble Stephens Architects (Project No. 610-21-561, Sheet Nos 01 to 19, Revision M, dated 2 September 2022), that following demolition of the existing structures within the site the proposed development will comprise construction of two interconnected three-storey unit blocks over a common single-level basement. Proposed floor levels for the basement range from RL616.385m to RL615.7m, which will require excavation to depths ranging from about 2m to 4.5m below existing surface levels. The proposed basement is generally to be set back more than 6m from the boundary, except along the eastern and western edge of the northern basement footprint where setbacks of 2.3m and 1.8m, respectively, are proposed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions as a basis for providing comments and recommendations on excavation, groundwater, retention, earthworks, footings and floor slabs.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was caried out on 8 and 14 June 2022, and comprised the drilling of six boreholes (BH1 to BH6) to refusal at depths ranging from 0.7m to 5m using our track mounted JK205 drill rig. The boreholes were advanced using spiral augers fitted with a Tungsten Carbide (TC) bit.

The borehole locations were set out by taped measurements from existing surface features and are shown on Figure 2. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between contours and spot heights shown on the survey plan by Richard Cox Surveyors Pty Ltd (Project No: 19103, dated July 2019). The datum of the levels is the Australian Height Datum (AHD).

The apparent compaction of fill and strength/relative density of the natural soils were assessed by the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples recovered by the SPT split tube sampler. The strength of the bedrock was assessed from observation of the drilling resistance using a TC drill bit, tactile examination of recovered rock chips, and correlation with the results of subsequent laboratory moisture content tests. The assessment of rock strength in this manner is approximate and variations of one order of strength should not be unexpected.

Groundwater observations were made during and on completion of drilling of each borehole. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Cho Sum Yip, set out the borehole locations, nominated the testing and sampling locations and logged the subsurface conditions encountered. The boreholes logs are attached, together with a set of Report Explanation Notes which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories. STS tested rock chip samples to determine moisture contents as shown in STS Table A. Envirolab Services tested soil samples to determine pH values, sulphate contents, chloride contents and resistivity values, as shown in their Certificate of Analysis 298000.

Sampling and testing of soil and groundwater samples for potential contamination was outside the scope of this geotechnical investigation.

3 RESULTS OF INVESTIGATION

3.1 Site History

The site is located within an area previously occupied by the Fitzroy Iron Works which was established in 1848 to mine the iron ore deposits. One of the chalybeate (iron-rich) springs which were responsible for the limonite ore deposits is understood to have been located on the property to the east of the subject site.

3.2 Site Description

The site is located within an east-west striking valley situated between Mt Gibraltar and Mt Alexandra to the south and north, respectively. The site is located on a spur of the northern foot slope of Mt Gibraltar formed by Iron Mines Creek and Chinamans Creek to the east and west, respectively. Surface levels across the spur generally slope down to the north at approximately 3°. Surface levels within the site generally appear to follow the contours of the natural hillslope. The site has a southern frontage with Rainbow Road, which is relatively level adjacent to the site.

The site comprises three rectangular lots with combined dimensions of approximately 64m (north-south) and 80m (east-west). At the time of the fieldwork, each lot was occupied by a single-storey detached residence with a carport/garage, with the structures generally located towards the centre of each lot. The residences at No. 1 and No. 3 Rainbow Road were of brick construction, whilst the residence at No. 5 was of weatherboard and fibro construction. The existing buildings generally appeared to be in good external condition. The buildings were generally surrounded by lawns and garden beds, with medium to large trees interspersed throughout the site. An exposure of iron indurated sandstone was observed in the north-eastern corner of No. 5 Rainbow Road.

North of the site were four residential properties (Nos. 180, 182, 184 and 186 Old Hume Highway). Each property contained a single-storey brick or weatherboard residences set back at least 20m from the common boundary. The residences appeared to be in good external condition based on cursory observation from within the subject site. Between the residences and the site were grassed rear yards with trees and some small metal sheds. Surface levels across the boundary appeared to follow the natural hillslope although the slope increases to approximately 5° to 7° from the common boundary.

To the west was a residential property (No. 7 Rainbow Road) occupied by a single-storey brick residence, set back about 2m from the common boundary and in good external condition when viewed from within the subject site. Surface levels across the boundary were similar to those within the subject site.

East of the site was a large lot that contains a single-storey brick building currently in use as the Wingecarribee Aboriginal Community Centre. The building was set back about 14m from the common boundary and was surrounded by grassed areas, with large trees interspersed within the property. Surface levels across the boundary were similar to those within the subject site and follow the natural hillslope.

3.3 Subsurface Conditions

The Mittagong-Bullio 1:50,000 Geological Series Sheet 8929-II indicates the site is mapped to be underlain by Hawkesbury Sandstone bedrock. The Geological Map of Mittagong by J.B. Jaquet (1899) indicates that the site is located in an area either underlain by or adjacent to an iron ore (limonite) deposit on the surface of the underlying Hawkesbury Sandstone.

In summary, the boreholes encountered fill covering residual silty clay and silty sand that graded into sandstone bedrock. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered at each borehole location.

Fill

Fill was encountered in BH1, BH2 and BH3 to depths of 0.4m. The fill comprised gravelly clayey sand or silty sand, with inclusions of ironstone, sandstone and igneous gravel.

Residual Soil

Residual soils generally comprising gravelly silty clay or gravelly silty sand. The clays were assessed to be of low or medium plasticity and ranged in strength from stiff to very stiff strength to very stiff to hard strength. The sands were assessed to be of loose relative density. The residual soils contained a high proportion of ironstone gravel.

Sandstone Bedrock

Weathered sandstone bedrock was encountered in all boreholes at depths ranging from 0.4m to 1.65m correlating with levels ranging from approximately RL619.5m to RL618.0m. The sandstone was generally assessed to be distinctly weathered and of at least low strength from initial contact, although in BH1 the upper 1.3m was assessed as extremely weathered. The sandstone, particularly in the eastern half of the site, was ferruginous (iron-rich) and of medium to high strength. We anticipate that given the proximity of the site to the chalybeate springs to the east that the depth of the ferruginous sandstone will generally increase towards the west, which correlates with visual observations of the rock chips recovered from the lower portion of BH1, BH5 and BH6. All boreholes refused on high strength bedrock at depths ranging from 0.7m to 5m.

Groundwater

In BH1 and BH5 groundwater seepage was encountered during drilling at a depth of 3.7m and 4.2m, respectively. In BH1, groundwater was measured at a depth of 2.2m (≅RL616.8m), 2 hours after completion, and in BH5 groundwater was measured at a depth of 3.5m (≅RL615.5m), 0.5 hour after completion of drilling. We note that due to the relatively short monitoring period groundwater levels may not have stabilised. No groundwater was encountered within the remaining boreholes during or on completion of drilling.

3.4 Laboratory Test Results

The moisture content test results correlated reasonably well with our field assessment of the rock strength.

The results of the soil aggression testing are tabulated below:

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides CL (ppm)	Resistivity (ohm.cm)
BH1	1.5-1.95	XW sandstone BEDROCK	6.4	24	<10	46,000
BH3	1.0-1.2	RESIDUAL Gravelly silty sand	5.8	<10	<10	120,000
BH4	0.5-0.95	RESIDUAL Gravelly silty sand	5.7	<10	<10	77,000
BH5	3.5-4.0	Sandstone BEDROCK	5.9	10	<10	68,000

The above results indicate that the residual soils and weathered rock would have an exposure classification of 'Non-aggressive' for both concrete and steel structural elements in accordance with Tables 6.4.2 (C) and Table 6.5.2 (C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Dilapidation Reports

Prior to commencement of demolition and construction, dilapidation reports should be prepared for any neighbouring properties that have the potential to be adversely impacted by any proposed development works. We consider it would be prudent to complete dilapidation surveys on the adjoining properties to the east and west of the site. Consideration could also be given to completing dilapidation surveys on the properties to the north as preparation of such reports can help to guard against opportunistic claims for damage that was present prior to the start of the works. Council or utility owners may also require dilapidation surveys of their assets.

Dilapidation reports should include detailed descriptions of all defects, including location, type, crack length, crack width, etc.. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The dilapidation report should be carefully reviewed prior to excavation commencing to ensure that appropriate equipment is used.

4.2 Excavation

The following recommendations should be read in conjunction with the latest version of *'Excavation Work – Code of Practice'* prepared by SafeWork NSW.

The proposed basement floor level will require excavation to depths ranging from about 2m to 4.5m below existing surface levels. Such excavation will encounter fill, residual soils and sandstone bedrock. Excavation of the soils and the upper extremely weathered or very low strength rock should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. However, the bedrock is typically of medium to heigh strength and we anticipate that excavation, particularly of the ferruginous sandstone, will be difficult and present 'hard rock' excavation conditions which will require assistance with rock excavation equipment, such as hydraulic impact hammers, ripping hooks, rotary grinder or rock saws.

During the use of hydraulic impact hammers, precautions must be made to reduce the risk of damage to adjoining structures caused by vibrations generated by the hammers. At the commencement of the use of hydraulic impact hammers we recommend that at least some quantitative vibration monitoring be carried out on all adjoining structures, or at the boundaries, by an experienced vibration consultant or geotechnical engineer to check that vibrations are within acceptable limits. If the vibrations measured during such trials are close to acceptable limits, continuous monitoring of transmitted vibrations to the adjoining buildings should be carried out. We expect that monitoring of the transmitted vibrations to at least the house to the west will be required due to its close proximity to the proposed excavations. Where this is carried out, the monitors should be solidly fixed to the adjoining buildings. The monitors should also be attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached so that excavation works can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

If during excavation using the hydraulic impact hammers, vibrations are found to be excessive or there is concern, then alternative lower vibration emitting equipment, such as rock saws, rock grinders or smaller hammers may need to be used. The use of a rotary grinder or rock sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures should also be used when excavating using rock hammers in order to limit vibrations:

- Maintain a sharp moil on the hammer.
- Orientate the hammer out of the face and enlarge the excavation by breaking small pieces off the face.
- Operate the hammer in short bursts to limit the amplification of the vibrations.
- Operate only one hammer at a time.

We recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. Excavation contractors should be provided with a copy of this geotechnical report, including the borehole logs, so that they can make their own assessment of suitable excavation equipment.

Material to be disposed of offsite will need to be suitably classified for waste disposal.

4.3 Groundwater

Given the location of the site on the footslopes of Mt Gibraltar and being located on a spur between creeks to the east and west, we anticipate that hydraulic gradients will dip down to the north in the area of the site. Groundwater seepage was encountered in BH1 and BH5 and as such we expect that seepage will occur at the soil/rock interface and through any joints and bedding planes within the bedrock exposed in the completed cut faces, particularly after periods of heavy or prolonged rainfall. Seepage during excavation is expected to be satisfactorily controlled using conventional sump and pump techniques or gravity drainage to the stormwater system.

We recommend that groundwater seepage into the excavation be monitored by site personnel and the results (quantity, location, source, etc.) be reported to the geotechnical and hydraulic engineers so that any unexpected conditions can be promptly addressed. In the long term, drainage should be provided behind all retaining or basement walls, and below the basement floor slabs. The completed excavation should be inspected by the hydraulic engineer to assess any changes required to the designed drainage system to accommodate the actual seepage flows.

4.4 Excavation Batters

The proposed basement will be generally set back at least 6m from the site boundaries and where this is the case, sufficient space will be available to form temporary batters through the residual soils and any extremely to highly weathered bedrock and then vertical excavation within sandstone bedrock of low strength or higher strength. Where the excavation encroaches to about 2m from the eastern and western boundaries the feasibility to form temporary batters will depend on the depth of soil/extremely weathered rock in these areas of the site. In this regard, we recommend that following demolition a series of test pits be excavated along these portions of the basement perimeter to assess the depth to rock. Where the depth to rock is shallow temporary batters may be formed and sub-vertical excavation completed subject to inspection by the geotechnical engineer. Where the depth of soil and extremely weathered rock does not permit the formation of temporary batters along these portions of the boundary then the soils will need to be retained or stabilised. Further advice can be provided following inspection of the test pits by a geotechnical engineer.

Temporary batters formed through the fill and residual soils/extremely weathered sandstone should be no steeper than 1 Vertical in 1.5 Horizontal (1V:1.5H). Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters, say at least twice the excavation depth from the crest of the batters.

Sandstone bedrock of low or higher strength may be cut vertically, subject to regular geotechnical inspections. Inspection of vertical cut faces should occur at each 1.5m depth interval until bulk excavation level is reached. The purpose of the cut face inspections is to determine whether potentially unstable wedges, clay seams and extremely weathered seams are present within the sandstone bedrock that may adversely affect the stability of the cut faces. Such features may require remedial measures including rock bolts, shotcrete and mesh, or dental treatment of thin seams, and this would be advised by the geotechnical engineer at the time of the inspection. Any additional support recommended by the geotechnical engineer must be installed prior to further excavation. Provision should be made in the contract documents for such inspections and stabilisation measures, though the boreholes have indicated very few fractures in the rock.

Permanent batters within the soils should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. All permanent batters should be covered with topsoil and planted with a deep-rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent slopes to also reduce erosion. Although good quality sandstone may be left unsupported in the long term, subject to inspection by a geotechnical engineer, it will deteriorate and fret with time and allowance must be made for ongoing maintenance to remove any debris from the base of the rock cuts so that it does not block drains and cause damp issues. Alternatively, the cut faces can be covered with shotcrete or retaining walls constructed in front of the faces and the resulting gaps backfilled with gravel.

4.5 Retaining Walls

Retaining walls will be required to support excavation batters in the long term and we assume that the walls will be constructed from the base of the excavation and waterproofed before backfilling. For the design of retaining walls, landscape walls or wall stabilisation measures the following earth pressure coefficients and subsoil parameters may be adopted:

- For design of any retaining walls that will be propped by the structure, we recommend the use of an 'at-rest' lateral earth pressure coefficient (K_0) of 0.55 for the retained soil profile, assuming a horizontal backfill surface.
- Where some minor movements of retaining walls may be tolerated (e.g. landscape walls), they may be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient (K_a) of 0.35 for the soil profile.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile.
- For all walls in front of vertically cut rock faces of low strength or above assume a uniform pressure of 5kPa.
- Any surcharge affecting the walls (e.g. construction traffic, pavement and ground floor slab loads, compaction stresses during backfilling, etc.) should be allowed in the design using the appropriate above earth pressure coefficients.
- The retaining walls should be designed as permanently drained. Subsurface drains should incorporate a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion.

- The passive lateral toe resistance for retaining walls founded in sandstone of medium strength may be taken to be 250kPa assuming horizontal ground in front of the wall and no excavations for footings or service trenches. The upper 0.2m below bulk excavation level should be ignored in the design to cater for excavation tolerances.
- Following retaining wall construction and backfilling, we recommend that a dish drain be provided immediately upslope of the walls to intercept surface water run-off. The discharge from such drains should be piped to the stormwater system.
- Due to the difficulty in achieving suitable compaction between batters and retaining walls we recommend that retaining walls should be backfilled with a single size durable gravel such as 20mm blue metal or similar product. Blue metal does not require compaction in layers but should be tamped or vibrated until consolidation no longer occurs. A 0.3m thick clay capping layer should be placed over the gravel and separated by a geofabric to limit penetration of surface water into the backfill.

4.6 Footings

Following bulk excavation, we anticipate that weathered bedrock will be exposed throughout the basement excavation and therefore all footings should be founded within the rock. In this regard, pad or strip footings should be feasible which may be designed for an allowable bearing pressure of 1,000kPa.

If any of the above ground portions of the building extend outside of the basement footprint, they should be supported on piles founded within the sandstone bedrock below the zone of influence of the basement retaining walls so that additional surcharge loads are not placed on the walls. The zone of influence should be taken as a line drawn up at 1V:1H from the base of the retaining walls.

We recommend that a geotechnical engineer inspects footing excavations following excavation and prior to placement of reinforcement. Prior to pouring concrete, all water and water-softened or loose material must be removed from the base of the footing excavations.

4.7 Slabs On-Grade and Pavements

Basement floor slabs will be underlain by a subgrade of weathered bedrock and in this regard no particular subgrade preparation will be required. Basement slabs should be supported on a minimum 100mm thick layer of DGB20 or other approved granular material to act as a separation layer between the rock and the slab.

Drainage measures comprising subsoil drains should be provided below all basement slabs. The subsoil drains should be directed to sumps with permanent fail-safe pumps for discharge.

Concrete pavements should be isolated from the structural columns to allow relative movement.

4.8 Earthquake Design 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 Ce

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between 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 TEST REPORT

Client:	JK Geotechnics	**Report No.:**	35082AT - A
Project:	Proposed Residential Development	**Report Date:**	16/06/2022
Location:	1-5 Rainbow Road, Mittagong, NSW	**Page 1 of 1**	

AS 1289	TEST METHOD	2.1.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
1	2.40 - 3.00	9.0
1	3.50 - 3.90	7.7
2	0.40 - 0.70	6.6
3	2.40 - 2.70	5.1
4	1.10 - 1.50	3.5
4	2.50 - 2.60	7.7
5	0.80 - 1.20	7.3
5	4.80 - 5.00	4.6
6	1.50 - 1.80	5.4
6	2.60 - 2.80	4.9

Notes:

- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/06/2022.
- Sampled and supplied by client. Samples tested as received.

16/06/2022
Authorised Signature / Date
(D. Treweek)

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
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CERTIFICATE OF ANALYSIS 298000

Client Details

Client	JK Geotechnics
Attention	Arthur Billingham
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35082AT, Mittagong**
Number of Samples	4 Soil
Date samples received	14/06/2022
Date completed instructions received	14/06/2022

Analysis Details

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

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

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

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

Report Details

Date results requested by	21/06/2022
Date of Issue	21/06/2022

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

Results Approved By
Nick Sarlamis, Assistant Operation Manager

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference		298000-1	298000-2	298000-3	298000-4
Your Reference	UNITS	BH1	BH3	BH4	BH5
Depth		1.5-1.95	1-1.2	0.5-0.95	3.5-4.0
Date Sampled		08/06/2022	08/06/2022	08/06/2022	14/06/2022
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	20/06/2022	20/06/2022	20/06/2022	20/06/2022
Date analysed	-	20/06/2022	20/06/2022	20/06/2022	20/06/2022
pH 1:5 soil:water	pH Units	6.4	5.8	5.7	5.9
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	24	<10	<10	10
Resistivity in soil*	ohm m	460	1,200	770	680

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			20/06/2022	[NT]	[NT]	[NT]	[NT]	20/06/2022	[NT]
Date analysed	-			20/06/2022	[NT]	[NT]	[NT]	[NT]	20/06/2022	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	95	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	89	[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.

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments

Sample #1-3:pH/EC: Samples were out of the recommended holding time for this analysis.

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BOREHOLE LOG

Borehole No. 1

1/1

Client: TBG CONSTRUCTION PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 619.0m

Date: 8/6/22 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty sand topsoil, fine to medium grained, dark brown, trace of roots.	w>PL			GRASS COVER
		N = 10 2,3,7			CI	Gravelly Silty CLAY: medium plasticity, red brown, fine to coarse grained ironstone gravel.	w>PL	(St VSt)		RESIDUAL
			1			Extremely Weathered sandstone: gravelly silty CLAY, low plasticity, red brown, medium to coarse grained sandstone and ironstone gravel.	XW	Hd		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE TOO FRIABLE FOR HP TESTING
2 HRS AFTER COMPLET-ION		N = 16 4,4,12	2							
						SANDSTONE: fine to medium grained, grey and red brown, with iron indurated bands.	DW	M-H		MODERATE RESISTANCE
			3							
			4			END OF BOREHOLE AT 4.0m				HIGH 'TC' BIT RESISTANCE 'TC' BIT REFUSAL
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. 2

1/1

Client: TBG CONSTRUCTION PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 618.4m

Date: 8/6/22 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Gravelly clayey sand, fine to medium grained, brown, with fine to medium grained ironstone gravel.	M			GRASS COVER
								-	SANDSTONE: fine to medium grained, brown, ferruginous.	DW	H		HAWKESBURY SANDSTONE
									END OF BOREHOLE AT 0.7m				HIGH 'TC' BIT RESISTANCE
						1							'TC' BIT REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
3
1/1

Client: TBG CONSTRUCTION PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 620.2m
Date: 8/6/22 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty sand, fine to coarse grained, dark grey, with fine grained igneous gravel.	M			GRASS COVER
		N = 5 2,3,2			SM	Gravelly Silty SAND: fine to medium grained, red brown and brown, medium to coarse grained ironstone gravel.	M	L		
		N > 2 2,2/0mm REFUSAL	1							
			2		-	SANDSTONE: fine to medium grained, brown, ferruginous.	DW	H		HAWKESBURY SANDSTONE MODERATE RESISTANCE HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
4
1/1

Client: TBG CONSTRUCTION PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 620.6m
Date: 8/6/22 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0		CI	Silty CLAY: medium plasticity, red brown.	w>PL	(VSt)		GRASS COVER RESIDUAL
		N = 6 3,3,3			SM	Gravelly Silty SAND: fine to medium grained, brown and red brown, medium to coarse grained ironstone and sandstone gravel.	M	L		
			1		-	SANDSTONE: fine to medium grained, light grey, with high strength iron indurated bands and clay bands.	DW	M		HAWKESBURY SANDSTONE MODERATE 'TC' BIT RESISTANCE
			2					H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.8m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **5**

1/1

Client: TBG CONSTRUCTION PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 619.0m

Date: 14/6/22 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		SM	Gravelly Silty SAND: fine to medium grained, grey, fine to medium grained gravel.	M	(L)		GRASS COVER
			1		-	SANDSTONE: fine to medium grained, red brown and brown, ferruginous.	DW	L-M		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BITRESISTANCE
			2							
0.5 HR AFTER COMPLET-ION			3			SANDSTONE: fine to medium grained, light grey.				
			4							
								H		HIGH RESISTANCE
			5			END OF BOREHOLE AT 5.0m				'TC' BIT REFUSAL
			6							
			7							

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BOREHOLE LOG

Borehole No. 6

1/1

Client: TBG CONSTRUCTION PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

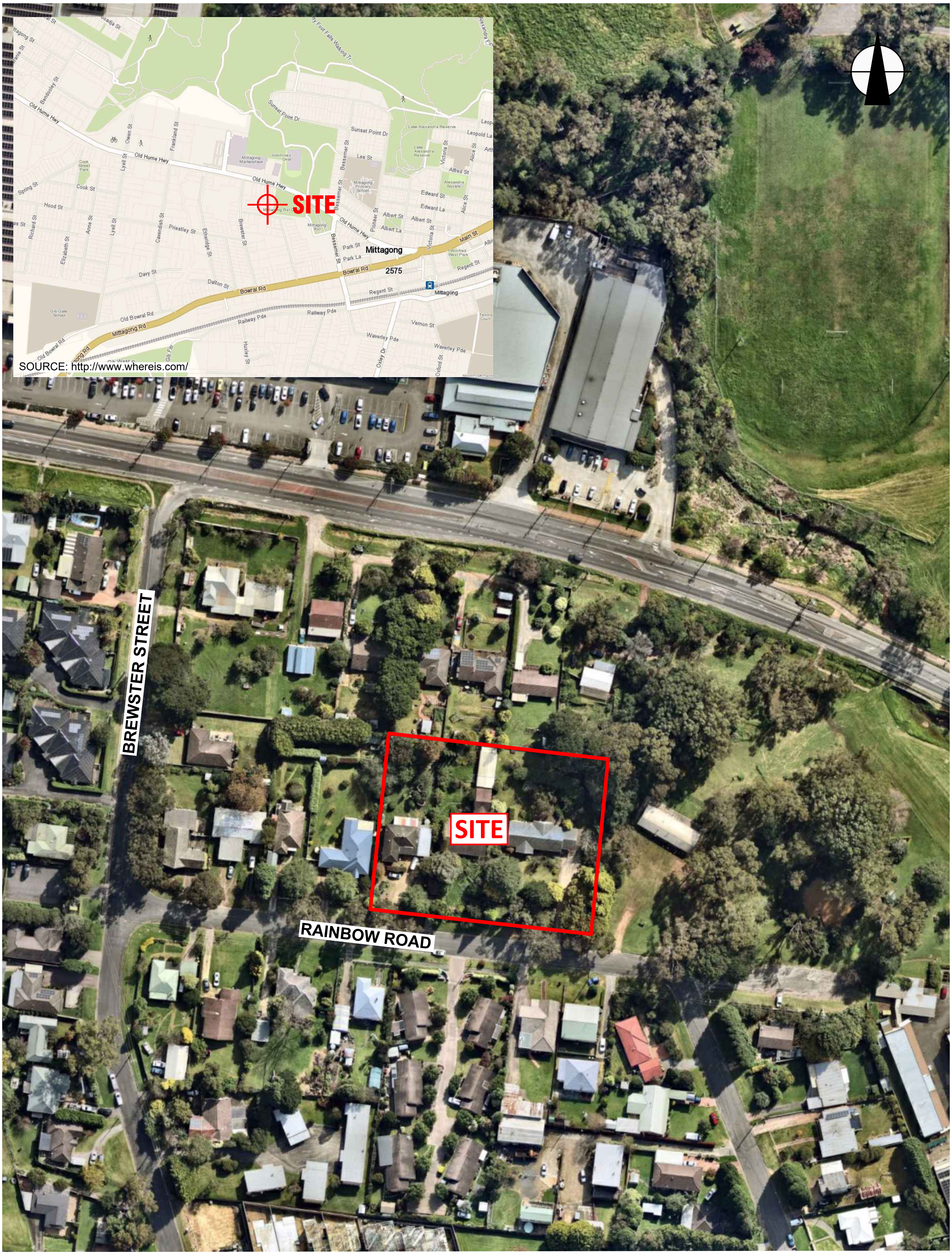
Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW

Job No.: 35082AT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 619.7m

Date: 14/6/22 **Datum:** AHD

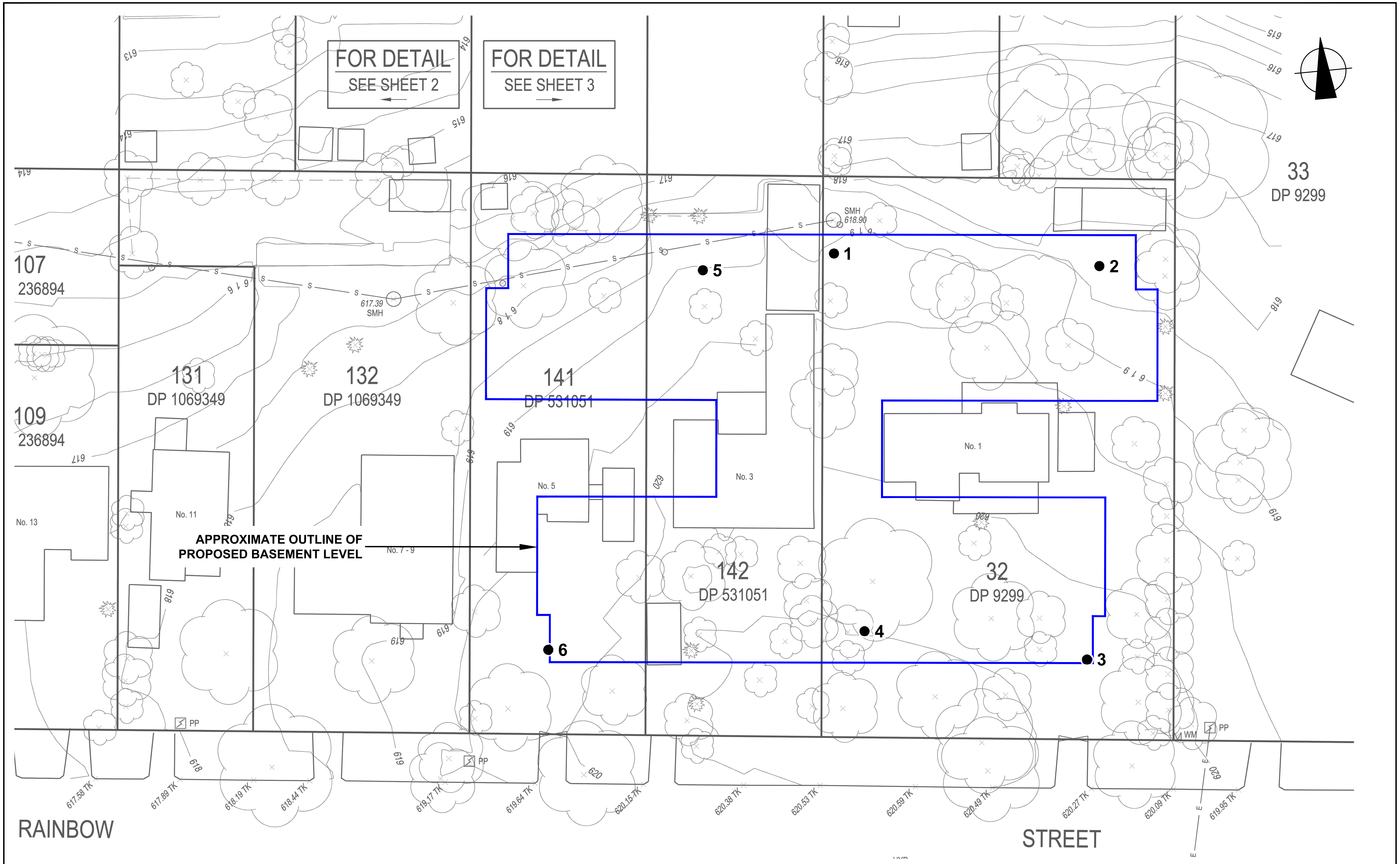
Plant Type: JK205 **Logged/Checked by:** C.S.Y./A.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	CL	Sandy CLAY: low plasticity, red brown, fine to medium grained sand, with fine to coarse grained ironstone gravel.	w≈PL	(VSt - Hd)		GRAVEL COVER
	N > 13 5,7, 6/100mm REFUSAL							TOO FRIABLE FOR HP TESTING
		1	-	SANDSTONE: fine to medium grained, red brown, ferruginous.	DW	L-M		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE
ON COMPLETION		2		SANDSTONE: fine to medium grained, light grey, with iron indurated bands.				
						H		HIGH RESISTANCE
		3		END OF BOREHOLE AT 3.0m				'TC' BIT REFUSAL
		4						
		5						
		6						
		7						



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM	Title: **SITE LOCATION PLAN**	
	Location: 1-5 RAINBOW ROAD, MITTAGONG, NSW	
	Report No: 35082AT	Figure No: 1
This plan should be read in conjunction with the JK Geotechnics report.	JKGeotechnics	

PLOT DATE: 17/06/2022 10:55:10 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35082AT MITTAGONG\CAD\35082AT.DWG



Title:	BOREHOLE LOCATION PLAN
Location:	1-5 RAINBOW ROAD, MITTAGONG, NSW
Report No:	35082AT
Figure No:	2

SCALE 0 4 8 12 16 20 METRES 1:400 @A3

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

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PLOT DATE: 8/07/2022 10:13:54 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35082AT MITTAGONG\CAD\35082AT.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

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

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

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

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

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

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

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

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

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

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

FILL

TOPSOIL

CLAY (CL, CI, CH)

SILT (ML, MH)

SAND (SP, SW)

GRAVEL (GP, GW)

SANDY CLAY (CL, CI, CH)

SILTY CLAY (CL, CI, CH)

CLAYEY SAND (SC)

SILTY SAND (SM)

GRAVELLY CLAY (CL, CI, CH)

CLAYEY GRAVEL (GC)

SANDY SILT (ML, MH)

PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

CONGLOMERATE

SANDSTONE

SHALE/MUDSTONE

SILTSTONE

CLAYSTONE

COAL

LAMINITE

LIMESTONE

PHYLLITE, SCHIST

TUFF

GRANITE, GABBRO

DOLERITE, DIORITE

BASALT, ANDESITE

QUARTZITE

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

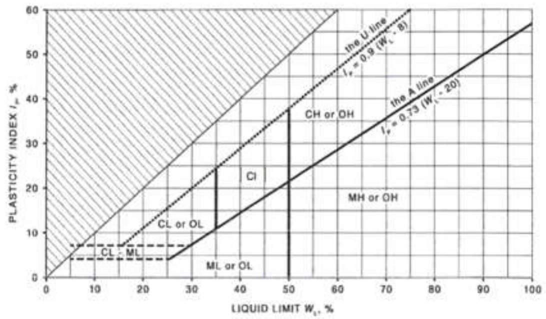
$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

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

NOTES:

1. For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
2. Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.
3. Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.
4. The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



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LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	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 analysis. 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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	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. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	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 Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
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 and ≤ 35 4 – 10 MEDIUM DENSE > 35 and ≤ 65 10 – 30 DENSE > 65 and ≤ 85 30 – 50 VERY DENSE > 85 > 50 Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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