REPORT

TO
FULTON TROTTER ARCHITECTS

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED NEW BUILDINGS

AT
SAMUEL GILBERT PUBLIC SCHOOL
RIDGECROP DRIVE, CASTLE HILL, NSW

19 June 2018
Ref: 31056Srptrev1

JK Geotechnics
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For and on behalf of
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ENVIROLAB SERVICES ‘CERTIFICATE OF ANALYSIS 181465

BOREHOLE LOGS 1 TO 8 INCLUSIVE

BOREHOLE LOGS 101 TO 103 INCLUSIVE

CORE PHOTOGRAPHS

DYNAMIC CONE PENETRATION TEST RESULTS No 6

FIGURE 1: SITE LOCATION PLAN

FIGURES 2A AND 2B: BOREHOLE LOCATION PLANS

FIGURE 3: GRAPHICAL BOREHOLE SUMMARY (BH1 TO BH8)

VIBRATION EMISSION DESIGN GOALS

REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents the results of a two-stage geotechnical investigation for the proposed alterations and additions at Samuel Gilbert Public School, Ridgecrop Drive, Castle Hill, NSW. A site location plan is presented as Figure 1. The investigations were commissioned by John Kennedy of Fulton Trotter by email dated 9 November 2017 and letter dated 21 May 2018. The commissions were on the basis of our proposals (Ref. P45897Srev1 dated 20 October 2017 and 31056Sprop dated 7 May 2018).

The purpose of the investigations was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions and support, retaining walls, footings and on-grade floor slabs and soil aggression. Details of the proposed development are not known at the time of preparing this report though in general terms it is proposed to create 15 new teaching spaces to replace 11 demountable buildings. New buildings are expected to be between one and three stories in height. The larger buildings will be situated in the southern part of the site adjoining Ridgecrop Drive.

We were also commissioned to carry out a Waste Classification Assessment. The results are presented in the report (E31057K dated December 2017) prepared by Environmental Investigation Services (EIS), the environmental consulting division of the JK Group. This geotechnical report must be read in conjunction with the EIS report.

2 INVESTIGATION PROCEDURE

The first stage of fieldwork for the investigation comprised the auger drilling of eight boreholes (BH1 to BH8). Seven of the boreholes were auger drilled to depths between 1.5m and 4.5m using our track mounted JK305 drilling rig. One borehole location was inaccessible to the rig and so a hand augered borehole and Dynamic Cone penetration (DCP) test were completed.

The second stage of fieldwork involved drilling three additional boreholes using our track mounted JK308 drilling rig. The boreholes were initiated by auger drilling until high resistance or refusal was encountered and then continued by diamond core drilling using NMLC equipment and water flush. Approximately 3m of core were recovered from each borehole and the drilling was terminated at depths between 4.2m and 5.9m.

The borehole locations, as indicated on attached Figure 2A and 2B, were set out using taped measurements from existing surface features and were electromagnetically scanned for buried services prior to drilling commencing. Ground levels at Boreholes BH101 to BH103 were interpolated from spot levels shown on the supplied survey plan prepared by LandPartners Ltd and hence are only approximate.

The nature and composition of the subsurface soil and rock horizons were assessed by logging the materials recovered during drilling. The relative strength of the subsoils was assessed from the Standard Penetration Test (SPT) ‘N’ numbers augmented by hand penetrometer readings on clay samples recovered in the split tube SPT sampler. The strength of the underlying bedrock in the augered boreholes was assessed by observation of drilling resistance when using a tungsten carbide (TC) bit, examination of the recovered rock chip samples, and subsequent correlation with laboratory moisture content testing. Rock strength in the cored boreholes was assessed on site and later confirmed with reference to point load strength index tests completed on the core samples. The hand augered borehole and DCP test both refused at shallow depth which is considered likely to correspond to the bedrock surface. Groundwater observations were made during and on completion of drilling individual boreholes. Long term groundwater monitoring was not carried out. For further details on the investigation procedures adopted, reference should be made to the attached Report Explanation Notes.

Our geotechnical engineer was present full time on site during the fieldwork and set out the borehole locations, directed electromagnetic scanning, nominated sampling and testing, and logged the subsurface profile. The borehole logs are presented with this report together with a glossary of logging terms and symbols used and colour photographs of the recovered rock core.

Rock chip samples were submitted to Soil Test Services Pty Ltd (STS) for moisture content testing. The test results are presented in attached STS Table A. The point load strength test results are presented in Table B and plotted on the borehole logs. Selected samples were submitted to Envirolab Services Pty Ltd for soil pH, resistivity, chloride and sulphate content testing; the test results are summarised in Section 3.3 below and the Envirolab Services ‘Certificate of Analysis’ which is attached.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated in hilly terrain generally sloping down to the south-east at 4-6°. The site slopes at a similar grade as the general terrain and was bound by Gilbert Road and Ridgecrop Drive to the east and south respectively.

At the time of the investigation the southern end of the site was occupied by one and two storey brick buildings with demountable classrooms on brick piers on their northern and eastern sides. An on grade asphaltic concrete (AC) car park and associated access roads located on the western side of the site were generally in good condition with isolated cracking in places. A concrete basketball court and small shelter structures were located at the north-western corner of the site. An approximately level grassed playing field was located at the northern end of the site. Bushland was located adjacent to the north, south and west of the buildings and playing field and extended beyond the site's northern and western boundaries. Medium to large trees were observed within the bushland and scattered throughout the site within planter beds.

At the time of the fieldwork, all structures mentioned above appeared to be in good condition based on a cursory inspection from within the subject site.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by the Hawkesbury Sandstone. The investigation has revealed a subsurface profile generally comprising a shallow cover of sandy soils overlying weathered sandstone. Groundwater was not encountered within the depths investigated. Reference should be made to the attached borehole logs for detailed subsurface conditions at specific locations. A graphical summary of the augered boreholes is presented in Figure 3 and a summary of the subsurface conditions, as encountered, is presented below:

Fill

Fill comprising silty sand with a trace of gravel was encountered only in BH 3 to 0.2m depth. In BH101 to BH103 the surface topsoil layer is logged as fill and was only 100mm in thickness.

Residual Soil

The natural soil cover is probably residual in origin but may also be slopewash. It comprises a fine to medium grained silty sand.

Sandstone Bedrock

Sandstone was encountered at depths between 0.1m and 0.5m at all borehole locations. In most locations there was an upper layer of extremely low to very low strength material but at depths between 0.7m and 1.1m there is a transition to low strength and from between 1.2m and 2.5m an increase to medium to high strength. Most augered boreholes were terminated at or close to Tungsten Carbide bit refusal which is consistent with the medium to high strength assessment.

Groundwater

No groundwater was encountered within the depth of investigation. Water levels recorded in the cored boreholes on completion were almost certainly due to the drill flush water which had not had time to drain away.

3.3 Laboratory Test Results

The moisture content test results were generally consistent with the field assessments of rock strength. The point load strength index tests showed rock strengths (UCS calculated empirically from point load results) ranging from 18MPa to 30 MPa.

The soil chemistry testing showed the following:

- pH values between 5.0 and 6.7 which is slightly acidic;
- Chloride contents between 20 and 89 mg/kg which are low;
- Sulphate contents between 10mg/kg and 44mg/kg which are low;
- Resistivity between 14000 ohm.cm and 44,000 ohm.cm which are quite high.

In light of the above the results indicate a mild aggression potential for buried concrete and non-aggressive condition for steel structures, in accordance with the guidelines in AS2159-2009.

Based on the borehole logs and laboratory test results, the bedrock classifications in accordance with the Pells *et* al (1998) system, in the table below, apply to the sandstone bedrock encountered.

Borehole	Surface RL (m AHD)	Depth (m)			
		Class V	Class IV	Class III	Class II/I
101	~98.1	0.1 - 1.0	1.0 – 2.5	2.5 – 3.0	3.0 – 5.9
102	~99.1	0.1 – 0.9	0.9 – 2.91	-	2.91 – 4.2
103	~97.2	0.1 – 0.9	0.9 – 1.2	-	1.2 – 4.3

Note that the rock class of the deepest layer assumes the rock quality below the termination depth of the borehole does not decrease which is not necessarily the case. The zone of influence of a footing is taken as 1.5 times its width/diameter.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings and Issues

The boreholes encountered a fairly consistent subsurface profile of very minor surficial fill underlain by residual sandy soils that overlie sandstone bedrock at shallow depths in the range of 0.1m to 0.5m.

Based on the above known subsurface conditions, the following are considered to be principal geotechnical issues for prospective development of typical one to three storey school buildings, associated structures and pavements. It must be noted that the recommendations are of a general nature as we have no details of the proposed developments.

1. The presence of fill of generally minor thicknesses is not generally sufficient to result in a site classification of Class P in accordance with AS2870-2011.
2. Sandstone bedrock is at such shallow depth that all footings should be supported uniformly upon it.
3. Any significant excavation to regrade the site will encounter sandstone bedrock and “hard rock” excavation equipment will be needed. This will also affect buried service installation.
4. The sandy soils will form a moderate subgrade for pavement construction but will require care in preparation including careful moisture control to enable good compaction.

4.2 Excavation Conditions

4.2.1 Excavation Methods

Based on the investigation results, even shallow excavation will encounter the sandstone bedrock.

The soil cover should be readily excavated by conventional earthworks equipment (eg. hydraulic excavator). Some of the underlying weathered sandstone of extremely and very low strength, if encountered, may also be excavated by large bucket excavator, possibly with some ripping. However, we expect excavation of medium strength and stronger sandstone would be most effectively excavated using hydraulic impact rock hammers. This equipment would also be required for breaking up of boulders or blocks, for trimming rock excavation side slopes and for detailed rock excavations such as for footings or buried services.

4.2.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to adjoining structures and buildings. Prior to excavation commencing, consideration should be given to preparing detailed dilapidation reports on the existing buildings so that careless work by the contractor can be clearly identified. Excavation procedures and the dilapidation reports should be carefully reviewed prior to excavation commencing so that appropriate equipment is used.

The excavation with hydraulic rock hammers, if used, should preferably commence away from likely critical areas using a moderately sized excavator fitted with a relatively low energy hydraulic hammer no larger than a Krupp 900 size or equivalent. We commend that continuous quantitative vibration monitoring be undertaken on the closest buildings during rock excavations with impact hammers. Subject to review of the dilapidation reports, we recommend that vibrations on the existing buildings, measured as Peak Particle Velocity (PPV), be limited to no higher than 5mm/sec. If it is confirmed that higher vibrations are measured, they should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable, depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, it would be necessary to change to a considerably smaller rock hammer or to use alternative excavation techniques. Alternative excavation techniques which will significantly reduce vibrations include a rotary grinder or the use of grid sawing in conjunction with ripping and/or hammering. The vibrations due to rock hammering or ripping may be further dampened by providing a vertical saw cut slot along the perimeter of the excavation and maintaining the base of the slot at a lower level than the adjoining rock excavation at all times. When using a rock saw or rotary grinder the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers are used.

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate hammer in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

4.3 Excavation Batters

Temporary batter slopes will likely be feasible for the perimeter of any proposed excavations. Temporary batters through the sandy soils should be no steeper than 2 Horizontal (H) to 1 Vertical (V) and all bedrock up to and including low strength should be battered at not steeper than 1 Vertical (V) in 1 Horizontal (H). At the toe of such batters and at the soil rock interface some seepage may occur following wet weather and it may be necessary to flatten batters or to provide some other local toe support, where there is a risk of instability.

Permanent batters should be no steeper than 2H:1V and protected by vegetation or hard surfacing. Flatter batters of at least 3H:IV are strongly recommended to allow maintenance of vegetation etc. It is very difficult to establish topsoil and vegetation on slopes as steep as 2H:1V.

Sandstone of medium or high strength may be cut vertically subject to geotechnical inspection to identify whether there are any adverse defects in the rock that would give rise to instability. Faces should be inspected at vertical intervals no greater than 1.5m and allowance made in the contract for stabilisation works such as rock bolts and shotcrete.

Surcharge loads, including construction loads, must be kept well clear of the crest of temporary batters (at least 2H from the crest, where H is the vertical height of the batter slope in metres).

Where temporary batters are formed to allow construction of retaining walls, consideration needs to be given to the type of backfill to be used against permanent basement and/or lower ground floor walls. Uncompacted backfill placed up against walls will result in large settlements which can have adverse effects on structures, paving or landscaping supported above. Compaction will be difficult in confined spaces with light equipment and therefore the backfill placed against the permanent retaining walls should preferably comprise a uniform durable granular material such as blue metal which is surrounded in a geotextile fabric.

4.4 Retaining Walls

Retaining walls may be required to support low height cuts and/or fills. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of such retention systems including cantilevered soldier pile shoring type walls.

- For free-standing cantilever walls located in areas where movement is of little concern (ie. where grassed or garden areas are being retained), adopt a triangular lateral earth pressure distribution and an “active” earth pressure coefficient (K_a) of 0.35 for the soil profile and weathered sandstone, assuming a horizontal backfill surface.

- For free-standing cantilever walls located in areas where movement is of concern (ie. where the walls are supporting proposed on-grade floor slabs and external pavements), adopt a triangular lateral earth pressure distribution and an “at rest” earth pressure coefficient (K_0) of 0.55 for the retained profile, assuming a horizontal backfill surface.
- For retaining walls incorporated into and supported by propping from the proposed new buildings, adopt the above K_0 value, assuming a horizontal backfill surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile and $24kN/m^3$ for weathered sandstone.
- Any surcharge affecting the walls (eg. 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 weathered sandstone may be designed for an allowable lateral resistance of 200kPa. The upper 0.3m 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 uphill of the walls to intercept surface water run-off. The discharge from such drains should be piped to the stormwater system.

4.5 Subgrade Preparation

The proposed development may require the removal of existing trees (including their root balls), stripping of grass, topsoil and root-affected soil, and any poorly compacted, deleterious or contaminated existing fill. Stripped topsoil and root affected soils should be stockpiled separately as they are not suitable for reuse as engineered fill. They may be reused at a later stage for landscaping purposes. The site can then be excavated down to design subgrade level using hydraulic excavators.

Following stripping and excavation, we recommend that the subgrade be inspected by a geotechnical engineer and any poor quality material removed. If the fill is to support structures then all existing fill should be removed. Some fill may be suitable for replacement as engineered fill, if it is appropriately moisture conditioned. The surface should then be proof rolled with at least eight passes of a static smooth drum roller (at least 8 tonnes deadweight). The final two passes of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

Subgrade heaving may occur if the subgrade becomes “saturated” or there are areas of poorly compacted fill. Heaving areas should be locally removed to a stable base and replaced with engineered fill, or further advice could be sought. Engineered fill should also be used where site levels need to be raised.

4.6 Engineered Fill

The excavated sandy soils and ripped sandstone may be reused as engineered fill. Material must be free of organic matter and free of particle sizes greater than 75mm prior to reuse as engineered fill. The select engineered fill should be compacted in maximum 200mm loose layers using a minimum 8 tonne static roller to a minimum density of 98% of SMDD. Clay soils and ripped shale should not be brought onto the site for use as engineered fill.

4.7 Footings

The proposed buildings should be uniformly supported on the underlying sandstone bedrock. The site classifies as Class A in its current condition.

Strip or pad footings bearing on the upper weathered sandstone (Class V) may be designed for an allowable bearing pressure (ABP) of 700kPa increasing to 1000kPa on the low strength sandstone (Class IV) which generally occurs within 1m of the rock surface and 3500kPa for the Class III or better sandstone. Unless there is significant excavation it would be necessary to use bored piers to found on the Class III or better sandstone; rigs would need to be able to penetrate medium strength sandstone to reach the consistently better quality rock.

All footings should be free of any loose or softened material and any water prior to concreting. At least a selection of footings should be inspected by a geotechnical engineer to confirm conditions are in accordance with expectations.

4.8 Ground Floor Slabs and Pavements

Slab-on-grade construction for the proposed buildings are considered feasible provided the subgrade is prepared as discussed in Section 4.5. The slabs-on-grade should be constructed independent of the building footings and walls (ie. designed as “floating” slabs) to permit relative movement.

Based on the laboratory test results we recommend that the design of pavements and concrete slabs be based on an estimated CBR of 5%. For pavements subject to heavy vehicle traffic there will need to a 300mm thick capping layer of material with a CBR of at least 10%. This can be a product such as crushed/ripped sandstone.

Building slabs-on-grade and light duty concrete pavements should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RTA Specification 3051 unbound base (eg. DGB20) or similar quality, and compacted to a minimum density of 100% of SMDD. The sub-base material will provide a more uniform slab support and would reduce “pumping” of subgrade “fines” at joints. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

Subsoil drains should be provided along the upslope perimeter of all proposed new external pavement areas, with invert levels at least 200mm below subgrade level. The drainage trench should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.9 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:

- Review design when details of specific buildings are known and provide additional advice as necessary
- Inspect subgrade after stripping
- Proof roll subgrade prior to filling
- Compaction testing of fill
- Inspect footing excavations

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.

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

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

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

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

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

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Samuel Gilbert Public School, Ridgecrop Drive, Castle Hill, NSW

Ref No: 31056S
Report: A
Report Date: 11/12/2017

AS 1289	TEST METHOD	2.1.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
1	0.80-1.50	10.1
1	2.20-2.60	2.3
2	2.20-2.60	2.9
3	1.10-1.50	5.0
3	1.50-2.70	3.4
4	2.20-2.70	3.4
8	1.00-1.50	7.2
8	2.30-3.00	4.3

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SOIL TEST SERVICES

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31056S
Project:	Proposed alterations and additions	**Report:**	B
Location:	Samuel Gilbert Public School - Ridgecrop Drive, Castle Hill, NSW	**Report Date:**	6/06/2018
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
BH101	2.56 - 2.59	1.0	20
	2.82 - 2.85	1.3	26
	3.21 - 3.25	1.2	24
	3.69 - 3.73	1.4	28
	4.31 - 4.35	0.9	18
	4.76 - 4.79	1.2	24
	5.21 - 5.24	1.5	30
	5.86 - 5.89	1.9	38
BH102	1.34 - 1.37	0.8	16
	1.86 - 1.89	1.1	22
	2.21 - 2.24	0.9	18
	2.57 - 2.61	0.8	16
	3.11 - 3.13	0.9	18
	3.4 - 3.44	0.8	16
	3.78 - 3.81	1.0	20
	4.11 - 4.13	0.9	18
	4.17 - 4.2	1.3	26
BH103	1.25 - 1.28	1.0	20
	1.86 - 1.89	1.0	20
	1.89 - 1.91	0.9	18
	2.24 - 2.27	1.4	28
	2.73 - 2.76	1.0	20
	3.27 - 3.3	1.6	32
	3.79 - 3.82	1.0	20
	4.22 - 4.25	1.2	24

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 IS (50)

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

CERTIFICATE OF ANALYSIS 181465

Client Details

Client	JK Geotechnics
Attention	A Frost
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31056S, Castle Hill**
Number of Samples	3 Soil
Date samples received	06/12/2017
Date completed instructions received	06/12/2017

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	13/12/2017
Date of Issue	11/12/2017

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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, Inorganics Supervisor

Authorised By

David Springer, General Manager

Misc Inorg - Soil				
Our Reference		181465-1	181465-2	181465-3
Your Reference	UNITS	BH2	BH4	BH6
Depth		0.5-0.65	0.5-0.65	0.2-0.4
Date Sampled		02/12/2017	02/12/2017	02/12/2017
Type of sample		Soil	Soil	Soil
Date prepared	-	07/12/2017	07/12/2017	07/12/2017
Date analysed	-	07/12/2017	07/12/2017	07/12/2017
pH 1:5 soil:water	pH Units	6.7	5.0	5.3
Chloride, Cl 1:5 soil:water	mg/kg	30	89	20
Sulphate, SO4 1:5 soil:water	mg/kg	44	27	<10
Resistivity in soil*	ohm m	210	140	440

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	-			07/12/2017	[NT]	[NT]	[NT]	[NT]	07/12/2017	[NT]
Date analysed	-			07/12/2017	[NT]	[NT]	[NT]	[NT]	07/12/2017	[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]	113	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	105	[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.

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

1/1

Client: FULTON TROTTER ARCHITECTS

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S | **Method:** SPIRAL AUGER JK305 | **R.L. Surface:** N/A

Date: 2-12-17 | **Datum:**

Logged/Checked by: A.F./P.S.

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		SM	SILTY SAND: fine to medium grained, orange brown, trace of fine to coarse grained ironstone and sandstone gravel.				RESIDUAL
		N = SPT 8/100mm REFUSAL			-	SANDSTONE: fine to medium grained, orange grey.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
			1				DW	L		LOW RESISTANCE
			2				SW	M-H		MODERATE TO HIGH RESISTANCE
						END OF BOREHOLE AT 2.6m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **2**
1/1

Client: FULTON TROTTER ARCHITECTS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S
Date: 2-12-17
Method: SPIRAL AUGER JK305
Logged/Checked by: A.F./P.S.
R.L. Surface: N/A
Datum:

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		SM	SILTY SAND: fine to medium grained, brown, trace of fine to coarse grained ironstone and sandstone gravel.	D			RESIDUAL
					-	SANDSTONE: fine to medium grained, orange grey.	XW	EL		
			1				DW	L		LOW 'TC' BIT RESISTANCE
			2				SW-FR	M-H		MODERATE TO HIGH RESISTANCE
						END OF BOREHOLE AT 2.6m				'TC' BIT REFUSAL
			3							
			4							
			5							
			6							
			7							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: FULTON TROTTER ARCHITECTS

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S

Method: SPIRAL AUGER JK305

R.L. Surface: N/A

Date: 2-12-17

Datum:

Logged/Checked by: A.F./P.S.

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, grey brown, trace of fine grained ironstone and igneous gravel.				
					SM	SILTY SAND: fine to medium grained, orange brown and light grey.				RESIDUAL
		N = 28 7,12,16			-	SANDSTONE: fine to medium grained, orange brown and light grey.	XW-DW	EL-VL	>600 >600 >600	
			1				DW	L		LOW 'TC' BIT RESISTANCE
										MODERATE RESISTANCE
			2				SW	M-H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 4
1/1

Client: FULTON TROTTER ARCHITECTS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S **Method:** SPIRAL AUGER JK305 **R.L. Surface:** N/A
Date: 2-12-17 **Datum:**
Logged/Checked by: A.F./P.S.

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		SM	SILTY SAND: fine to medium grained, orange brown.				
		N > 12 12/150mm REFUSAL			-	SANDSTONE: fine to medium grained, orange brown/grey.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
			1				DW	VL-L		LOW RESISTANCE
			2				SW	M		MODERATE RESISTANCE
								H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
5
1/1

Client: FULTON TROTTER ARCHITECTS

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S
Method: SPIRAL AUGER JK305
R.L. Surface: N/A

Date: 2-12-17
Datum:

Logged/Checked by: A.F./P.S.

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		SM	SILTY SAND: fine to medium grained, orange brown, trace of root fibres and fine to coarse grained ironstone and sandstone gravel.	D			
		N > 21 7,9,12/ 50mm REFUSAL			-	SANDSTONE: fine to medium grained, orange brown.	XW	EL	>600 >600 >600	VERY LOW 'TC' BIT RESISTANCE
			1				DW	L		LOW RESISTANCE
			2			as above, but orange brown and light grey.	SW	M-H		MODERATE TO HIGH RESISTANCE
						END OF BOREHOLE AT 2.6m				'TC' BIT REFUSAL
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
6
1/1

Client: FULTON TROTTER ARCHITECTS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 2-12-17 **Datum:**
Logged/Checked by: A.F./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION	■			■	REFER TO DCP TEST RESULTS	0		SM	SILTY SAND: fine to medium grained, yellow brown, trace of root root fibres and fine to coarse grained ironstone and sandstone gravel.	D	MD		
									END OF BOREHOLE AT 0.4m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BRDROCK
						1							
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
7
1/1

Client: FULTON TROTTER ARCHITECTS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S
Date: 2-12-17
Method: SPIRAL AUGER JK305
R.L. Surface: N/A
Datum:
Logged/Checked by: A.F./P.S.

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		SM	SILTY SAND: fine to medium grained, orange brown, trace of root root fibres and fine grained ironstone gravel.	D	MD		GRASS COVER
		N > 24 8,16/ 150mm REFUSAL			-	SANDSTONE: fine to medium grained, orange brown and light grey.	DW	VL		RESIDUAL VERY LOW 'TC' BIT RESISTANCE
			1					L		LOW RESISTANCE
							SW			MODERATE TO HIGH RESISTANCE 'TC' BIT REFUSAL
			2			END OF BOREHOLE AT 1.2m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
8
1/1

Client: FULTON TROTTER ARCHITECTS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW

Job No. 31056S **Method:** SPIRAL AUGER JK305 **R.L. Surface:** N/A
Date: 2-12-17 **Datum:**
Logged/Checked by: A.F./P.S.

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 COMPLET-ION						0		SM	SILTY SAND: fine to medium grained, orange brown, trace of root root fibres and fine grained ironstone and sandstone gravel.	D	MD		RESIDUAL
					N > 20 12,8/50mm REFUSAL			-	SANDSTONE: fine to medium grained, orange brown and grey.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
						1				DW	L-M		LOW TO MODERATE RESISTANCE
						2				SW	M-H		MODERATE TO HIGH RESISTANCE
						3			END OF BOREHOLE AT 3.0m				
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
101
1 / 2

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S **Method:** SPIRAL AUGER **R.L. Surface:** ~98.1 m
Date: 2/6/18 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.Z./P.S.

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	DS		98	0		-	FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sub angular and angular ironstone gravel. with timber fragments and root fibres.	D	(Hd)		APPEARS POORLY COMPACTED
	DS	N > 14 14,14/ 69mm REFUSAL					Extremely Weathered sandstone, Silty SAND, fine grained, brown, trace of fine to medium grained sub angular ironstone gravel, and moderately cemented sandstone gravel.	XW	VL		HAWKESBURY SANDSTONE
	DS		97	1			SANDSTONE: fine to medium grained, orange brown and yellow brown, with extremely weathered bands and iron indurated bands.	DW	L - M		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			96	2			as above, but light grey and orange brown.				
			95	3			REFER TO CORED BOREHOLE LOG				MODERATE TO HIGH RESISTANCE
			94	4							
			93	5							
			92	6							

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31056S CASTLE HILL.GPJ <<DrawingFile>> 18/06/2018 17:18 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101

CORED BOREHOLE LOG

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S	**Core Size:** NMLC	**R.L. Surface:** ~98.1 m
Date: 2/6/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** B.Z./P.S.

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 roughness and shape, defect coatings and seams, openness and thickness Specific / General	Formation
		96			START CORING AT 2.52m					'TC' BIT RESISTANCE	
100% RETURN		95	3		SANDSTONE: fine to medium grained, orang brown and light grey, with iron indurated bands.	SW	M	1.0 1.3		(2.78m) XWS, 10°, 20 mm.t (2.91m) XWS, 10°, 5 mm.t	Hawkesbury Sandstone
					as above, but very thinly bedded at 20-30°.					(3.07m) J, 80°, P, Vr, Cn	
▽					as above, but light grey and light brown.			1.2			
		94	4		as above, but light brown and red brown.			1.4 0.90		(4.30m) XWS, 5°, 3 mm.t (4.43m) Be, 0°, P, Vr, Fe Sn (4.46m) Be, 0°, P, Vr, Fe Sn (4.47m) Be, 0°, P, Vr, Fe Sn (4.49m) Be, 0°, P, Vr, Fe Sn	
					as above, but bedded at 10-20°.						
		93	5		as above, but light grey and dark grey.	FR	M - H	1.2			
					as above, but with carbonaceous laminae.			1.5	600 200 60 20		
					as above, but trace of carbonaceous laminae.			1.9			
		92	6		END OF BOREHOLE AT 5.89 m						
		91	7								
		90	8						600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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31056S
BH101
Coring Starts at: 2.52m
2
2.52 →
3
4
5
← 5.89
Coring End

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
102
1 / 2

BOREHOLE LOG

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S **Method:** SPIRAL AUGER **R.L. Surface:** ~99.1 m
Date: 2/6/18 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.Z./P.S.

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	DS		99	0		-	FILL: Silty sand, fine to medium grained, brown and dark brown, trace of fine to medium grained sub angular ironstone gravel, timber fragments and root fibres.	D			APPEARS POORLY COMPACTED
	DS	N = 21 6,10,11					Extremely Weathered sandstone: Silty SAND, fine to medium grained, brown and yellow brown, with fine to medium grained sub angular moderately cemented sand gravel, trace of fine to medium grained sub angular ironstone gravel.	XW	Hd		
			98	1			as above, but with roots.	DW	L - M		HAWKESBURY SANDSTONE
							SANDSTONE: fine to medium grained, light brown and yellow brown.				MODERATE TO HIGH 'TC' BIT RESISTANCE
							REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
			97	2							
			96	3							
			95	4							
			94	5							
			93	6							

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31056S CASTLE HILL.GPJ <<DrawingFile>> 18/06/2018 17:18 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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CORED BOREHOLE LOG

Borehole No. 102

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S | **Core Size:** NMLC | **R.L. Surface:** ~99.1 m
Date: 2/6/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** B.Z./P.S.

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: SPACING (mm)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific / General	Formation
		98			START CORING AT 1.19m						
100% RETURN					SANDSTONE: fine to medium grained, yellow brown and light grey, with iron indurated bands.	MW	M	0.80		(1.49m) XWS, 0°, 3 mm.t	Hawkesbury Sandstone
		97	2		as above, but purple brown, with fine to medium grained sub rounded quartz gravel.			1.1; 0.90			
					as above, but without quartz gravel, light grey and yellow brown, very thinly bedded at 10-20°.			0.80		(2.62m) J, 80°, P, Vr, Cn; (2.66m) XWS, 0°, 30 mm.t	
					NO CORE 0.10m						
		96	3		SANDSTONE: fine to medium grained, light grey and yellow brown, very thinly bedded at 10-20°.			0.90; 0.80		(3.24m) J, 70°, P, Vr, Cn	
		95	4			SW	M - H	1.0; 0.90; 1.3		(3.63m) J, 30°, P, Vr, Fe Sn; (3.72m) XWS, 15°, 3 mm.t; (3.75m) Be, 15°, P, Vr, Fe Vn; (3.84m) J, 75°, P, Vr, Fe Vn; (3.96m) Be, 15°, P, Vr, Fe Vn	
		94	5		END OF BOREHOLE AT 4.20 m						
		93	6								
		92	7								

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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31056S BH102 Coring Starts at: 1.19
1
1.19
2
3
4
4.2 Coring End

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
103

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S **Method:** SPIRAL AUGER **R.L. Surface:** ~97.2 m
Date: 2/6/18 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.Z./P.S.

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	DS		97			-	FILL: Silty sand, fine to medium grained, brown and orange brown, trace of fine to medium grained sub angular ironstone gravel, timber fragments and roots.	D			APPEARS POORLY COMPACTED
	DS	N = 18 8,8,10					Extremely Weathered sandstone: Silty SAND, fine to medium grained, brown and yellow brown, with fine to medium grained sub angular moderately cemented sand gravel, trace of fine to medium grained sub angular ironstone gravel, and roots,	XW	(Hd)		
			96	1			SANDSTONE: fine to medium grained, light brown and yellow brown, with fine to medium grained sub rounded quartz gravel.	DW	VL - L		HAWKESBURY SANDSTONE 'TC' BIT REFUSAL
							REFER TO CORED BOREHOLE LOG				
			95	2							
			94	3							
			93	4							
			92	5							
			91	6							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
103
2 / 2

Client: FULTON TROTTER ARCHITECTS
Project: SAMUEL GILBERT PUBLIC SCHOOL - RIDGECROP DRIVE, CASTLE HILL, NSW
Location: PROPOSED ALTERATIONS AND ADDITIONS

Job No.: 31056S	**Core Size:** NMLC	**R.L. Surface:** ~97.2 m
Date: 2/6/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** B.Z./P.S.

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 SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness (Specific / General)	Formation
					START CORING AT 1.18m						
100% RETURN		96			SANDSTONE: fine to medium grained, light grey and light brown, with fine to medium grained sub angular and sub rounded quartz gravel inclusion, very thinly bedded at 15-20°, with iron indurated bands.	SW	M - H	1.0			Hawkesbury Sandstone
								1.0			
			2					0.90		(2.00m) Be, 10°, P, Vr, Fe Vn	
										(2.06m) Be, 10°, P, Vr, Fe Vn	
		95						1.4			
								1.0			
			3								
		94						1.6			
								1.0			
			4								
		93						1.2			
					END OF BOREHOLE AT 4.30 m						
		92	5								
		91	6								
		90	7								

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
31056S
BH103
Coring Starts at: 1.18
1
1.18
2
3
4
43
Coring End

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	FULTON TROTTER ARCHITECTS		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	SAMUEL GILBERT PUBLIC SCHOOL, RIDGECROP DRIVE, CASTLE HILL, NSW		
Job No.	31056S	Hammer Weight & Drop: 9kg/510mm	
Date:	2-12-17	Rod Diameter: 16mm	
Tested By:	A.F.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location							
Depth (mm)	**6**						
0 - 100	5						
100 - 200	7						
200 - 300	14						
300 - 400	17						
400 - 500	25						
500 - 600	REFUSAL						
600 - 700							
700 - 800							
800 - 900							
900 - 1000							
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:

SITE LOCATION PLAN

Location: SAMUEL GILBERT SCHOOL
RIDGECROP DRIVE, CASTLE HILL, NSW

Report No: 31056S

Figure No: 1

JK Geotechnics

PLOT DATE: 6/06/2018 2:55:09 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31056S CASTLE HILL\CAD\31056S.DWG

LEGEND

● BOREHOLE

HAND AUGER AND DCP TEST

0 10 20 30 40 50

SCALE 1:1000 @A3 METRES

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

Title: **TEST LOCATION PLAN**

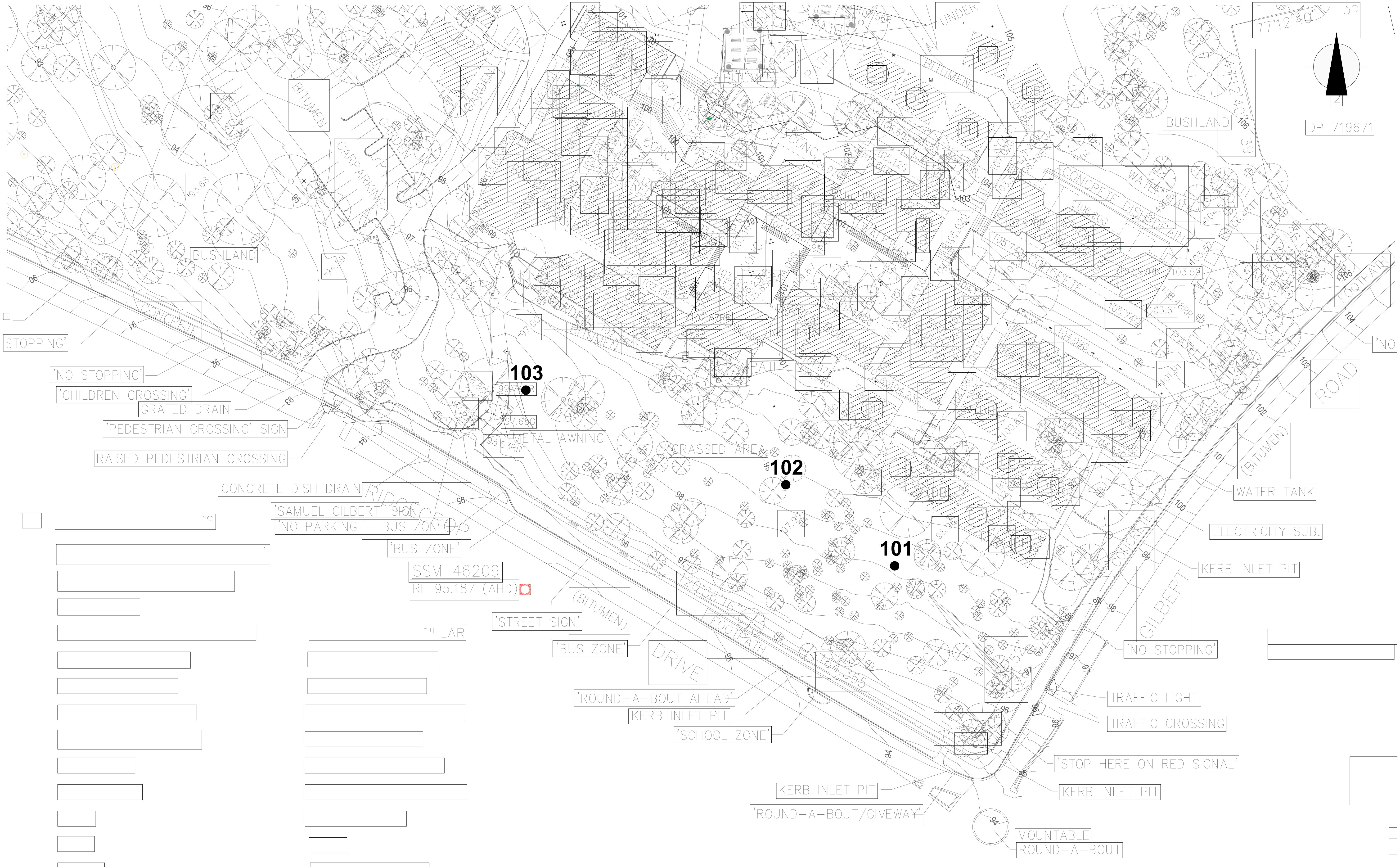
Location: SAMUEL GILBERT SCHOOL
RIDGECROP DRIVE, CASTLE HILL, NSW

Report No: 31056S

Figure No: 2A

JK **Geotechnics**

PLOT DATE: 18/06/2018 3:36:20 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31056S CASTLE HILL\CAD\31056S.DWG



SCALE 0 8 16 24 32 40 METRES 1:800 @A3

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

Title: **BOREHOLE LOCATION PLAN**

Location: SAMUEL GILBERT SCHOOL
RIDGECROP DRIVE, CASTLE HILL, NSW

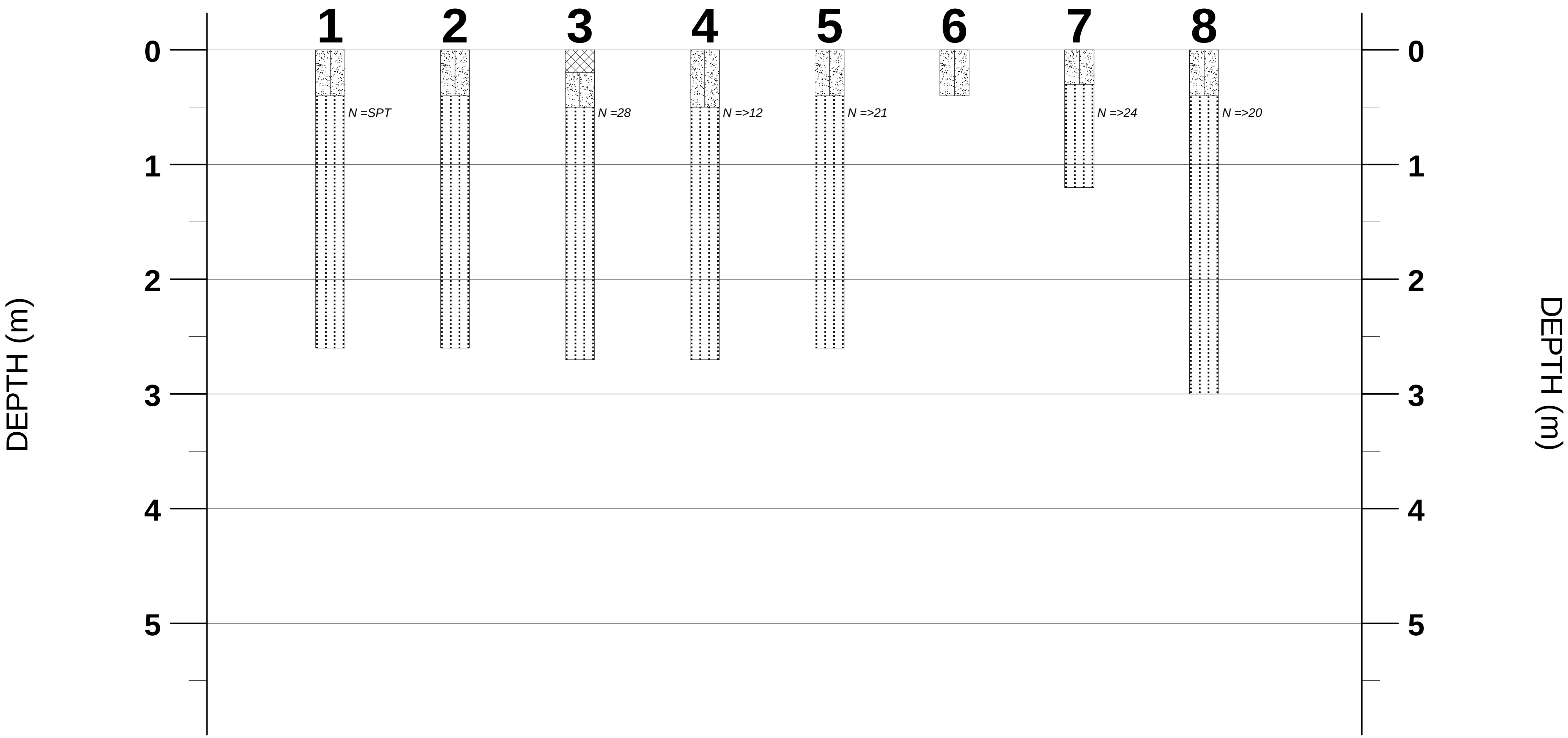
Report No: 31056S

Figure No: 2B

JK **Geotechnics**

PLOT DATE: 18/06/2018 3:32:45 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31056S CASTLE HILL\CAD\31056S.DWG

GRAPHICAL BOREHOLE SUMMARY



Silty Sand

Sandstone/ Greywacke

Fill

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 50 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 31056S

Figure No.: 3

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:

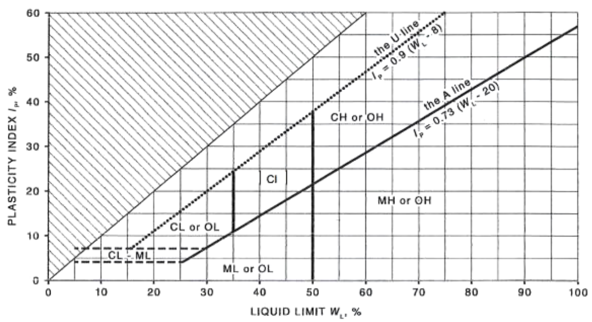
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**Point Load Strength Index $I_{s(50)}$ (MPa)**	**Field Assessment**
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	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.
Low Strength	L	2 to 6	0.1 to 0.3	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.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	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.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

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