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REPORT

TO

GEOFORM DESIGN ARCHITECTS

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MULTI-STOREY COMMERICAL DEVEOLPMENT

AT

84-86 KIORA ROAD, MIRANDA, NSW

15 April 2011

Ref: 23955SrptREV1

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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1 INTRODUCTION

A geotechnical investigation for a proposed multi-storey commercial development at 84-86 Kiora Road, Miranda, NSW was carried out in May 2010 and the results presented in our report (ref 23955Srpt) dated 25 May 2010. The investigation was commissioned by Mr. Gemvaro Russo of Conserv (No 462) Pty Ltd, C/- Geoform Design Architects on 7 April 2010, and was carried out in accordance with our proposal (Ref: P32166S) dated 12 March 2010.

This revision to the report (Rev1) was commissioned by Mr Rusty Moran of Moran Corporation Pty Ltd, on behalf of Conserv 9 (No 462), and was carried out in accordance with our proposal (Ref:P23955Sprop) dated 17 February 2011. This revised report addresses the proposed addition of two extra levels of basement car parking.

Based on the architectural drawings (2.01, 2.02, 3.01, 3.02, 3.03, 4.01, 4.02 and 4.03, Issue No.3, dated 11 April 2011) prepared by Geoform Design Architects, we understand the proposed development is to be a seven storey building of concrete and steel framed construction with reinforced concrete floors and roof, and with four basement levels below. The basement levels will be accessed via a turntable and car lift. The existing buildings will be demolished and the lower basement finished floor reduced level (RL) is to be at RL26.15m. Hence, excavations to achieve the design subgrade level are expected to extend to a maximum depth of about 12.4m.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on footings, suspended and on grade floors, retention systems, excavation methodology and other geotechnical issues associated with the proposed works.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 4th and 5th May 2010 and comprised the drilling of four boreholes (BH1 to BH4) to maximum depth of 11.35m using our track mounted JK250 and truck mounted JK350 drilling rigs. BH1 and BH3 were auger drilled to the termination depths of 9.0m using spiral auger techniques and a Tungsten Carbide 'TC' bit. BH2 and BH4 were initially drilled to depths of 5.82m and 6.17m respectively, using spiral auger techniques and a 'TC' bit; the two boreholes were then continued by diamond core drilling using NMLC equipment to termination depths of 10.89m and 11.35m, respectively.

Prior to commencement of the fieldwork, the investigation locations were electromagnetically scanned to check for the presence of buried services by a specialist sub-contractor.

The borehole locations shown on the attached Figure 1 were set out by taped measurements from apparent site boundaries and existing surface features. The surface reduced levels shown on the attached borehole logs were interpolated from spot levels shown on the supplied unreferenced survey plan. The survey datum is Australian Height Datum.

The strength of the subsurface soils in the boreholes was assessed from Standard Penetration Test (SPT) 'N' values, which were augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube sampler. The strength of the weathered shale and sandstone within the augered portion of the boreholes was assessed from observation of the drilling resistance when using the TC' bit and examination of the recovered cuttings. Note that strengths assessed in this way are indicative and variances of one strength order should not be unexpected. The strength of the weathered rock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were made in the boreholes during drilling and on completion of auger drilling. We note that water is used as part of the coring process, and therefore the water level at completion of coring has not been recorded, as it is likely to be artificially high. No longer term groundwater monitoring was carried out. For more details of the investigation procedures, reference should be made to the attached Report Explanation Notes.

The fieldwork was carried out under the direction of our geotechnical engineer, who was present full-time on site to set out the borehole locations, nominate the insitu testing and sampling and log the encountered subsurface profile. The borehole logs (which include field test results, Point Load Strength Index test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected samples were also tested in a NATA registered laboratory to determine soil pH, chloride and sulphate contents. The results are summarised in the attached Table A. The recovered rock core was returned to Soil Test Services (STS), a NATA registered laboratory, where it was photographed and Point Load Strength Index tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are attached as Table B. The core photographs are included opposite the relevant cored borehole logs.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated in hilly terrain, sloping down to the south-west, about 3° to 4°. The subject site has a western frontage onto the asphaltic concrete (AC) surfaced Kiora Road, with Urunga Parade and Urunga Lane bounding the site to the north and east, respectively.

At the time of field work, there were two storey brick shops on the street frontage and the remainder of the site was asphalt paved. The ground floor level of the existing buildings was cut in about 1m below the level of the rear carpark.

Based on a cursory inspection the existing shop buildings appeared to be in reasonably good external condition whilst the paved area was in poor condition with a number of potholes present.

There was a two storey rendered brick building with a gravelly sand paved rear yard on the southern side of the site.

3.2 Subsurface Conditions

The 1:100,000 geological map of Wollongong indicates that the site is underlain by Hawkesbury Sandstone.

In general the subsurface conditions encountered in the boreholes comprised a shallow soil profile overlying a deeply weathered shale and sandstone sequence. For a more detailed description of the subsurface profile encountered in the boreholes, reference should be made to the attached borehole logs.

Pavement/Fill

Asphalt paved surfaces up to be 70mm thick were encountered at all borehole locations. Underlying the asphalt was a gravelly sand fill (roadbase) to depths of about 0.4m. Silty clay fill was encountered beneath the roadbase to depths of about 1.2m. The moisture content of the silty clay was assessed as greater than the plastic limit whilst the roadbase was dry.

Residual Silty Clay

Residual silty clay of medium plasticity and mainly hard strength was encountered below the fill and extended to depths of 2.1m to 2.3m. The moisture content of the silty clays was assessed to be generally less than the plastic limit.

Weathered Bedrock

Weathered shale bedrock was encountered beneath the residual silty clay at all borehole locations. On first contact the shale was assessed to be extremely weathered and of extremely low to low strength, except in BH1 where a low strength shale band was first encountered to depth of 3.0m.

Distinctly weathered sandstone bedrock was encountered between 4.1m and 6.2m and was assessed as low strength. Below depths of 7.0m (BH1), 7.73m (BH2), 7.4m (BH3) and 7.24m (BH4), the sandstone improved from distinctly to slightly weathered and from low to medium strength.

Within the cored sections of the boreholes (BH2 and BH4) a number of sub-horizontal clay seams, extremely weathered seams and crushed zones ranging between about 5mm and 50mm thickness were encountered, particularly in BH4 between the depths of 10.21m and 11.07m.

A core loss zone (about 0.6m thick) was encountered at about 6.4m depth in BH4 and may be interpreted as representing a clay seam, extremely weathered seam or crushed zone.

Groundwater

Groundwater seepage was only encountered during auger drilling in BH1 at depth of 8m. Groundwater seepage was not recorded on completion of auger drilling in BH2, BH3 and BH4. We note that BH1 was left open for approximately five hours after completion of auger drilling and a standing water level was recorded at 6.1m depth. In the cored boreholes (BH2 and BH4), we note that water is introduced during core drilling which obscures groundwater measurements and groundwater levels may not have stabilised during the relatively short period between borehole completion and measurement of water levels.

3.3 Laboratory Test Results

The soil pH test results for the samples indicated that the residual silty clay is acidic with pH values of 4.8 and 6.0 respectively. The same two samples had sulphate contents of 40mg/kg and 29mg/kg and chloride contents of 18mg/kg and 9.6mg/kg respectively. When assessed in accordance with the criteria for concrete and steel piling exposure classifications given in Tables 6.4.2(C) of AS2159-2009 'Piling-Design and Installation', the sulphate and chloride values are non aggressive towards buried concrete and steel structures. However, the pH values of this soil are classified as mildly aggressive towards buried concrete structures. The point load strength index tests also generally confirmed the low to high strength of the weathered sandstone bedrock which is consistent with our field logging.

4 COMMENTS AND RECOMMENDATIONS

4.1 General Issues

We note that the lowest basement level of RL26.15m is beyond the maximum depth of our boreholes and as such, we recommend the drilling of four additional boreholes to say, RL23m (approximately 15m depth) to assess the composition and quality of the bedrock beneath the lowest basement level. We stress that due to the lack of information of the founding strata beneath RL26.5m, any comments given on excavation, retention and footings founded below our borehole depths are assumptions and as such should be treated with caution.

We note that the basement excavation will be up to 12.4m at the eastern end of the site. Kiora Road, Urunga Parade and Urunga Lane bound the site to the west, north and east, respectively. Neighbouring retail buildings and paved surfaces lie beyond the southern site boundary. During demolition and excavation, there is the potential to damage the neighbouring building and to induce differential movement of the existing roads. Demolition and excavation will need to be completed carefully with shoring and/or underpinning of the adjoining building and the road surfaces. This work will need to be completed using suitably experienced (and insured) contractors.

We recommend that prior to demolition a structural engineer assess the condition and stability of the neighbouring building to the south-west of the site and prepare a dilapidation report. The structural engineer will then be in the best position to detail any temporary works that may be required. We recommend that during the demolition, test pits be excavated adjacent to the neighbouring footings. In this regard we note that the footings will probably need to be supported by the proposed basement retention system and some prior underpinning could possibly be beneficial. Furthermore, the neighbouring footings may step into the site and may have to be trimmed prior to the drilling of shoring piles.

4.2 Demolition and Excavation

Demolition of concrete footings, floor slabs and paved surfaces may well require the use of rock breaker attachments to tracked excavators. However, demolition will need to be carried out with care so as not to damage or de-stabilise the neighbouring building to the south-west of the site. Vibration monitoring should be carried out at the commencement of use of rock hammers as described below.

Excavation recommendations provided below should be complemented by reference to the Code of Practice *'Excavation Work'*, Cat No 312 (31 March 2000), by WorkCover NSW, and by reference to AS3798-2007 '*Guidelines on Earthworks for Commercial and Residential Developments*'.

The excavations will extend through the soil profile and penetrate the weathered shale and sandstone bedrock. Excavations through the soil profile and the extremely weathered shale may be readily completed using bucket attachments to the tracked excavators. The more competent (low and medium strength) sandstone and/or more continuous ironstone bands we expect to be excavated using ripping tynes fitted to heavy tracked excavators and/or hydraulic rock breakers which may also be needed for trimming excavation faces, footings, service trenches, lift pits etc. From about 7m to 8m depth it appears likely that there will be mainly medium to high strength sandstone which will provide "hard rock" conditions, necessitating use of rock breakers or a combination of saw-cutting and ripping.

To assist in controlling vibrations and over-break of the shale and sandstone, initial saw cuts through the bedrock may be provided using rock saw attachments fitted to the excavator. However, this is only likely to be required in the sandstone at the base of the excavation as the shale bedrock is expected to be of a low strength order. Final trimming of excavation faces in better quality sandstone between soldier piles may also be completed using a grinder attachment rather than a rock breaker in order to assist in limiting vibrations.

If hydraulic rock breakers are to be used, then electronic vibration monitoring will be required on the building at 88-90 Kiora Road. If light breakers (say, 600kg or less) are used, it will probably be possible to demonstrate that they can be used without exceeding the threshold values given in the attached *'Vibration Emission Design Goals'*. If heavier breakers are proposed, then full time electronic monitoring with alarm systems will be required.

Based on the seepage recorded during and on completion of auger drilling, we do not expect that substantial groundwater flows into the excavation will occur. However, the fieldwork was carried out in a period of relatively dry weather and somewhat greater flows may be encountered after heavy or prolonged rainfall, particularly where defects daylight into the excavation and where highly fractured sections of shale and sandstone are exposed in the excavation face. The likelihood of groundwater flows along defects increases with depth. Seepage is also likely to occur along the bedrock surface, particularly during periods of heavy or prolonged rainfall. We therefore recommend that the initial stages of the excavation are carefully monitored, and if substantial flows are encountered, appropriate drainage measures may be detailed at the time. At this stage we expect that any seepage that does occur within the excavation will be controlled using conventional sump and pump techniques.

4.3 Shoring and Retention

An engineer designed in-situ retention system such as an anchored soldier pile wall with shotcrete infill panels or an anchored contiguous pile wall will be required to support the soil profile and the shale bedrock. The piles should extend below bulk excavation level as indications are that the sandstone is not of sufficient quality to be able to support the piles over a vertical face. Further drilling investigations may be able to demonstrate the sandstone is mainly of high quality (high strength with few defects), but the conditions shown in BH4 do not appear favourable. The use of more closely spaced soldier piles is preferred adjacent to neighbouring building so as to reduce the movements in the building.

Bored piles will be suited to this site. However, relatively large capacity piling rigs will be required as the piles are required to extend below bulk excavation level and therefore penetrate significant thicknesses of medium to high strength sandstone. We recommend further advice be sought from piling contractors who should confirm suitable access and who should also be provided with a copy of this report. Working platforms may be required for heavy piling rigs. Note that rigs require minimum clearances from the building which extends up to the southern boundary.

In the long term, any low to medium strength sandstone revealed between soldier piles will also degrade and spall and should be protected by the shotcrete infill panels. Additional rock bolts to support potentially unstable wedges over the width of the infill panels may be required in the short term. Geotechnical inspections will be required to determine the presence of any such potentially unstable wedges.

There is also the possibility that a continuous steeply dipping defect plane may daylight towards the toe of the cut face. We therefore recommend that the bond zone of all anchors extend beyond a theoretical failure plane sloping up from the toe of the excavation at 45°. It is imperative that close inspection of the toe of the cut faces be completed by an experienced geotechnical engineer to check for the presence of such a defect plane. If encountered, additional support, such as additional, and possibly longer, anchors may need to be provided in the short term and building prop loads will increase in the long term.

We recommend that geotechnical inspections be carried out at 1.5m depth intervals and that work be scheduled to allow excavations to commence over portions of the site whilst possible stabilisation measures are being installed in other areas.

The following characteristic earth pressure coefficients and subsoil parameters should be adopted for the design of the full depth retention system:

- Where minor movements may be tolerated (ie. those portions of the southern site boundary where no movement sensitive structures or services are located within a horizontal distance of at least H from the line of the excavation, where 'H' is the retained height in metres of soil, shale and sandstone up to and including low strength), we recommend design using a trapezoidal lateral earth pressure distribution be based on 5H kPa, for temporary works and 6H kPa for permanent structures.
- Where structures lie within a horizontal distance of 'H' from the line of the excavation, we recommend the magnitude of the lateral earth pressure distribution be increased to 8H kPa, to reduce deflections to a practical minimum.
- The rock of more than low strength should be assumed to apply a load of 10kPa on the shoring piles.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads, traffic, etc) should be included in the design using an 'at rest' earth pressure coefficient, K_o, of 0.55.
- The bulk unit weight of the soil and rock units can be taken as follows:

a)	Fill and clay soil:	20kN/m^3.
b)	Shale, extremely-distinctly weathered, extremely low to very low strength:	22kN/m^3.
c)	Shale and sandstone – distinctly to slightly weathered, low to medium to high:	25kN/m^3.

- The retaining walls should be designed as drained and measures taken to provide permanent and effective drainage of the ground behind the walls. Strip drains protected with a non-woven geotextile fabric should be used behind the shotcrete infill panels of soldier pile walls or inserted between gaps in contiguous piles. Alternatively, for the contiguous pile walls, weepholes comprising 20mm diameter PVC pipes grouted into holes or gaps between adjacent piles at 1.2m centres (horizontal and vertical), may be used. The embedded end of the pipes must, however, be wrapped with a non-woven geotextile fabric (such as Bidim A34) to act as a filter against subsoil erosion.

- Anchors or rock bolts should be bonded at least 3m into shale and sandstone bedrock of at least low strength where an allowable bond stress of 200kPa may be adopted for design. Where the anchors are bonded into medium or high strength sandstone bedrock an allowable bond stress of 400kPa may be adopted for design. The anchor bond length should commence beyond a line projected up from the base of the excavation at an angle of 45°.
- All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. The anchors and rock bolts will extend beyond the site boundaries and therefore permission from the neighbours will be required prior to installation. We have assumed that permanent lateral support of the perimeter piles will be provided by the new structure. If not, permanent anchors and/or rock bolts will be required which should be designed for corrosion resistance and for long term durability.

4.4 Footings

Based on the results of the boreholes, it is likely that medium to high strength sandstone will be exposed at bulk excavation level. However, we note that there may be areas of extremely low to very low strength sandstone and shale and numerous seams below founding level, if the conditions revealed in BH4 extend to greater depth. For uniformity of support we recommend that all footings for the proposed structures should be founded within the sandstone. We consider that pad or strip footings are generally suitable for this site.

Assuming a nominal 300mm socket into the sandstone of at least medium strength, footings (and any shoring piles supporting structural loads) may be designed on the basis of a maximum ABP of 3,500kPa. Some allowance should be made for zones of poor quality rock that have to be over-excavated. In addition, we recommend that, in addition to further investigations which should comprise at least four cored boreholes, all footings be inspected by a geotechnical engineer to confirm our assumptions regarding bedrock quality. Somewhat higher bearing pressures may be feasible, however this would need to be confirmed by the further investigation, probably supplemented by intensive spoon testing to identify the number and thickness of seams in the zone of influence of the footings.

All footings should be excavated, cleaned, inspected, tested and poured with minimal delay, (preferably on the same day as excavation and/or drilling). Water should be prevented from ponding in the base of footing excavations, as this may lead to a softening of the base, particularly in areas of more weathered shale. If a delay in pouring is expected, then we recommend a blinding layer of concrete be placed in the base of pad footings excavations.

In accordance with Table 2 of the 'Engineering Classification of Shales and Sandstones in the Sydney Region', as revised by Pells *et al* 1998 'typical' settlements for footings founded on bedrock with the above ABP's would be less than 1% of the minimum footing width.

4.5 Further Investigations and Inspections

We recommend that the following investigation and inspections be completed as outlined in the preceding sections of this report:

- Drilling of four boreholes to depths of say, 15m to assess the composition and quality of bedrock beneath the lowest basement level.
- Dilapidation surveys.
- Witnessing of test pits to reveal existing footings.
- Witnessing/inspection of shoring wall piles.
- Geotechnical inspection of excavation faces.
- Monitoring of groundwater seepages within the excavation.
- Witnessing installation and proof testing of anchors.
- Geotechnical inspection and possible spoon testing of footing bases.

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 Jeffery and Katauskas Pty Ltd accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

A waste classification 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. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. 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.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

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Principal

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CERTIFICATE OF ANALYSIS 40902

Client:
Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: Belinda Sinclair

Sample log in details:
Your Reference: **23955S, Miranda**
No. of samples: 2 Soils
Date samples received: 12/05/10
Date completed instructions received: 12/05/10

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:
Date results requested by: 19/05/10
Date of Preliminary Report: Not Issued
Issue Date: 19/05/10
NATA accreditation number 2901. This document shall not be reproduced except in full.
This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025.
Tests not covered by NATA are denoted with *.

Results Approved By:

Jacinta Hurst
Laboratory Manager

TABLE A

Miscellaneous Inorg - soil			
Our Reference:	UNITS	40902-1	40902-2
Your Reference	-------------	BH1	BH2
Depth	------------	0.5-0.95	0.5-0.95
Date Sampled		4/05/2010	4/05/2010
Type of sample		Soil	Soil
Date prepared	-	18/5/2010	18/5/2010
Date analysed	-	18/5/2010	18/5/2010
pH 1:5 soil:water	pH Units	4.8	6.0
Sulphate, SO4 1:5 soil:water	mg/kg	40	29
Chloride, Cl 1:5 soil:water	mg/kg	18	9.6

Method ID	Methodology Summary
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
LAB.81	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 21st ED, 4110-B.

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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 23955S
TABLE B Page 1 of 1

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
BH2	5.90-5.93	0.5	10
	6.13-6.17	1.2	24
	6.90-6.93	2.0	40
	7.25-7.27	1.5	30
	7.73-7.76	0.2	4
	8.34-8.38	1.1	22
	8.83-8.87	1.3	26
	9.31-9.34	0.9	18
	9.96-10.00	1.3	26
	10.44-10.47	1.2	24
BH4	7.30-7.33	0.9	18
	7.87-7.90	0.7	14
	8.27-8.30	0.5	10
	8.79-8.82	1.1	22
	9.12-9.16	1.0	20
	9.72-9.75	1.1	22
	10.42-10.45	0.2	4
	10.96-10.98	0.2	4
	11.24-11.26	1.3	26

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: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

Borehole No.
1
1/2

BOREHOLE LOG

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Method: SPIRAL AUGER
JK350
Logged/Checked by: M.L.T./
R.L. Surface: ≈ 38.1m
Datum: AHD

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		-	ASPHALTIC CONCRETE: 70mm.t	D	-	-	APPEARS POORLY COMPACTED
						FILL: Gravelly sand, fine to medium grained, brown, fine to medium grained igneous gravel.	MC>PL			
		N = 7 3,3,4			CL	FILL: Silty clay, low plasticity, dark brown, with fine to medium grained shale and igneous gravel.	MC>PL	St-VSt	180 220 200	RESIDUAL
			1			SILTY CLAY: medium plasticity, grey, orange brown mottled red, with fine to medium grained ironstone gravel.	MC<PL	H		
		N = 32 11,17,15	2			as above, but light grey, orange brown and red.			550 >600	
					-	SHALE: light grey, brown and dark brown.	DW	L	-	LOW 'TC' BIT RESISTANCE
			3 4			as above, but light grey, with iron indurated bands.	XW	EL-VL		VERY LOW RESISTANCE
			5			as above, but brown.	DW	L		LOW RESISTANCE
▼ AFTER 5 HRS			6		-	INTERBEDDED SANDSTONE AND SHALE: fine to medium grained, orange brown, dark brown and grey.				LOW RESISTANCE WITH VERY LOW BANDS
			7			SANDSTONE: fine to medium grained, light grey.	DW-SW			LOW RESISTANCE WITH MODERATE BANDS

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
2/2

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Method: SPIRAL AUGER
JK350
Logged/Checked by: M.L.T./
R.L. Surface: ≈ 38.1m
Datum: AHD

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
				■					SANDSTONE: fine to medium grained, light grey.	SW	L-M		LOW TO MODERATE RESISTANCE
													MODERATE RESISTANCE
											L		LOW RESISTANCE WITH MODERATE AND VERY LOW BANDS
▶						8							
				■									
						9			END OF BOREHOLE AT 9.0m				
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No.
2
1/2

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 38.4m
Date: 4-5-10
Datum: AHD
Logged/Checked by: M.L.T./

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		-	ASPHALTIC CONCRETE: 40mm.t	D	-	-	APPEARS POORLY COMPACTED
									FILL: Gravelly sand, fine to medium grained, brown, fine to medium grained igneous gravel.				
					N = 7 3,4,3				FILL: Silty clay, low plasticity, dark grey, with fine grained shale gravel.	MC>PL			
						1							
					N = 35 8,13,22			CL	SILTY CLAY: medium plasticity, light grey, orange brown and red, with ironstone gravel.	MC<PL	H	- >600 >600	RESIDUAL
						2		-	SHALE: light grey, with iron indurated band.	XW-DW	VL-L	-	VERY LOW TO LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						3							
										XW	VL		VERY LOW RESISTANCE WITH MODERATE BANDS
						4							
						5			as above, but dark brown and light grey.	DW	L		LOW RESISTANCE
						6			REFER TO CORED BOREHOLE LOG				
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

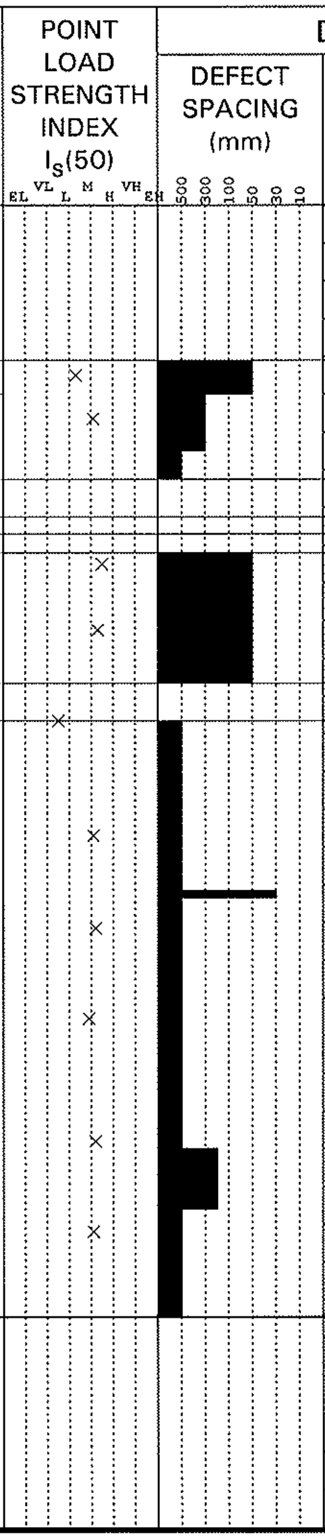
CORED BOREHOLE LOG

Borehole No.
2
2/2

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Drill Type: JK350
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 38.4m
Datum: AHD
Logged/Checked by: M.L.T./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		5		START CORING AT 5.82m				
		6		SANDSTONE: fine to medium grained, brown and light grey.	DW	L-M M-H	- Be, 0°, P, R - Be, 0°, P, R, IS - Be, 5°, P, R - J, 85°, Un, R, IS	
				CORE LOSS 0.2m				
				SANDSTONE: fine grained, light grey and grey.	DW	L		
		7		CORE LOSS 0.1m				
				SANDSTONE: fine to medium grained, orange brown and grey.	DW SW	M-H	- Be, 0°, P, IS - Be, 0°, P, S - Cr, 0°, 20mm.t - Cr, 0°, 25mm.t - Cr, 0°, 10mm.t - Cr, 0°, 30mm.t	
				as above, but light grey and grey, bedded at 0-5°.				
				CORE LOSS 0.2m				
		8 9 10		SANDSTONE: fine to medium grained, light grey and grey, bedded at 0-5°.	SW	M-H	- J, 30°, Un, R - XWS, 0°, 10mm.t - Be, 5°, P, R, IS - J, 45°, P, R - CS, 0°, 30mm.t - XWS, 0°, 10mm.t	
		11		END OF BOREHOLE AT 10.89m				



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BOREHOLE LOG

Borehole No.
3
1/2

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Method: SPIRAL AUGER JK350
Logged/Checked by: M.L.T./
R.L. Surface: ≈ 38.0m
Datum: AHD

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0	-	ASPHALTIC CONCRETE: 30mm.t	D	-	-	
								FILL: Gravelly sand, fine to medium grained, brown, fine to medium grained igneous gravel.	M			APPEARS POORLY COMPACTED
				■	N = 7 3,3,4			FILL: Sand, fine to coarse grained, brown.	MC>PL			
						1		FILL: Silty clay, low plasticity, grey and dark grey, with a trace of root fibres and ironstone and shale gravel.				
				■	N = 27 11,12,15		CL	SILTY CLAY: medium plasticity, light grey, orange brown mottled red, with fine to medium grained ironstone gravel.	MC<PL	H	- 400 420 450	RESIDUAL
				■		2	-	SHALE: light grey, with iron indurated bands.	XW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
				■		3 4 5						
				■			-	SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
										L		LOW TO MODERATE RESISTANCE WITH VERY LOW BANDS
						6		as above, but with clay bands.				
						7						

BOREHOLE LOG

Borehole No.
3
2/2

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Method: SPIRAL AUGER
JK350
Logged/Checked by: M.L.T./
R.L. Surface: ≈ 38.0m
Datum: AHD

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
									SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW	VL-L		VERY LOW RESISTANCE
						8			SANDSTONE: fine to medium grained, light grey.	SW	M		MODERATE RESISTANCE
						9			END OF BOREHOLE AT 9.0m				
						10							
						11							
						12							
						13							
						14							

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Borehole No.
4
1/2

BOREHOLE LOG

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

Job No. 23955S
Date: 4-5-10
Method: SPIRAL AUGER JK350
Logged/Checked by: M.L.T./
R.L. Surface: ≈ 37.9m
Datum: AHD

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						0		-	APSHALTIC CONCRETE: 20mm.t	D	-	-	APPEARS POORLY COMPACTED
									FILL: Silty sand, fine to medium grained, with fine to medium grained igneous gravel.				
				■	N = 6 2,3,3				FILL: Silty clay, low plasticity, brown and dark brown, with a trace of fine grained shale gravel and root fibres.	MC>PL			
				■	N = 20 7,9,11	1		CL	SILTY CLAY: medium plasticity, light grey, orange brown and red, with fine to medium grained ironstone gravel.	MC<PL	H	- >600 >600	RESIDUAL
				■		2		-	SHALE: light grey, with iron indurated bands.	XW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
				■		3 4		-	SANDSTONE: fine to medium grained, light grey, with clay bands.	DW	L		LOW TO MODERATE 'TC' BIT RESISTANCE WITH VERY LOW BANDS
						5					VL		VERY LOW RESISTANCE
				■		6					L-M		LOW TO MODERATE RESISTANCE
						7			REFER TO CORED BOREHOLE LOG				

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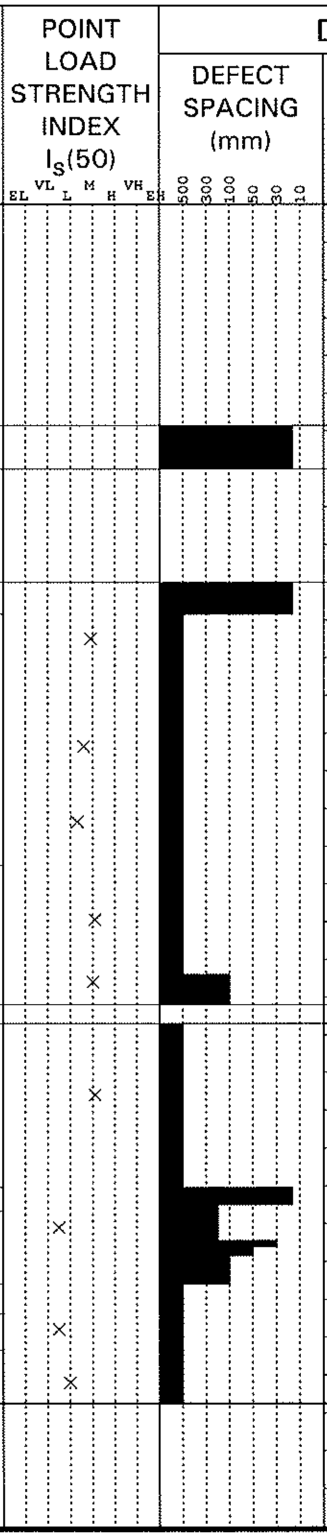
Borehole No.
4
2/2

CORED BOREHOLE LOG

Client: GEOFORM DESIGN ARCHITECTS
Project: PROPOSED MULTI-PURPOSE COMMERCIAL DEVELOPMENT
Location: 84-86 KIORA ROAD, MIRANDA, NSW

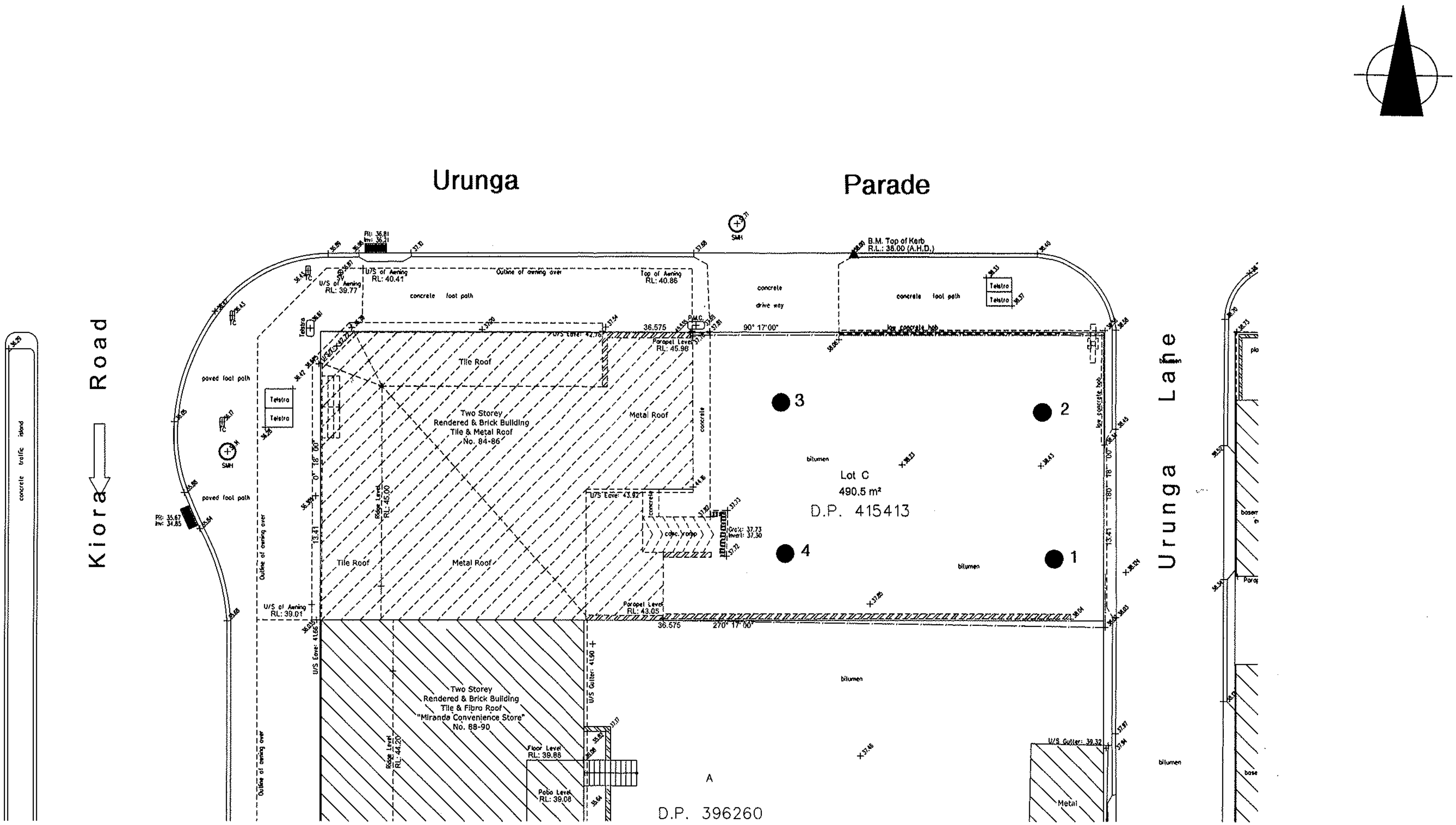
Job No. 23955S
Core Size: NMLC
R.L. Surface: ≈ 38.4m
Date: 4-5-10
Inclination: VERTICAL
Datum: AHD
Drill Type: JK350
Bearing: -
Logged/Checked by: M.L.T./RS

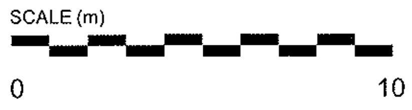


Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
5				
6	START CORING AT 6.17m			
	SANDSTONE: fine to medium grained, orange brown and red.	DW	VL-L	- Cr, 0°, 100mm.t - XWS, 0°, 100mm.t
	CORE LOSS 0.6m			
7	SANDSTONE: fine to medium grained, orange brown, grey and red brown.	XW DW	EL M	
	as above, but light grey, with carbonaceous laminated at 0-5°.	SW		- Be, 5°, P, R
8				- Be, 0°, P, R
			M-H	
9		DW		- Be, 0-5°, P, R - Cr, 0°, 90mm.t
	CORE LOSS 0.1m			
	SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	FR	M	
10	SHALE: dark grey.	XW DW	EL VL	- Be, 0°, P - Cr, 0°, 50mm.t - CS, 0°, 20mm.t - CS, 0°, 10mm.t - CS, 0°, 30mm.t - CS, 0°, 30mm.t - CS, 0°, 10mm.t
	SANDSTONE: fine to coarse grained, light grey and grey.	SW	M-H	
11	SHALE: dark grey.	DW	VL	
	SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	FR	M-H	- CS, 0°, 15mm.t
	END OF BOREHOLE AT 11.35m			


JEFFERY & KATAUSKAS PTY LTD
JOB NO. 23955S BH4 START CORING AT 6.17m
6
6.17
CORE LOSS 0.60m
7
8
9
CL 0.1m
10
11
END OF BH4 11.35m


JEFFERY & KATAUSKAS PTY LTD
JOB NO.23955S BH 2 START CORING AT 5.82m
6
CORE LOSS 0.2m
CORE LOSS 0.1m
7
CORE LOSS 0.2m
8
9
10
END BH2 AT 10.89m





BOREHOLE LOCATION PLAN

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 23955S **Figure No. 1**

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

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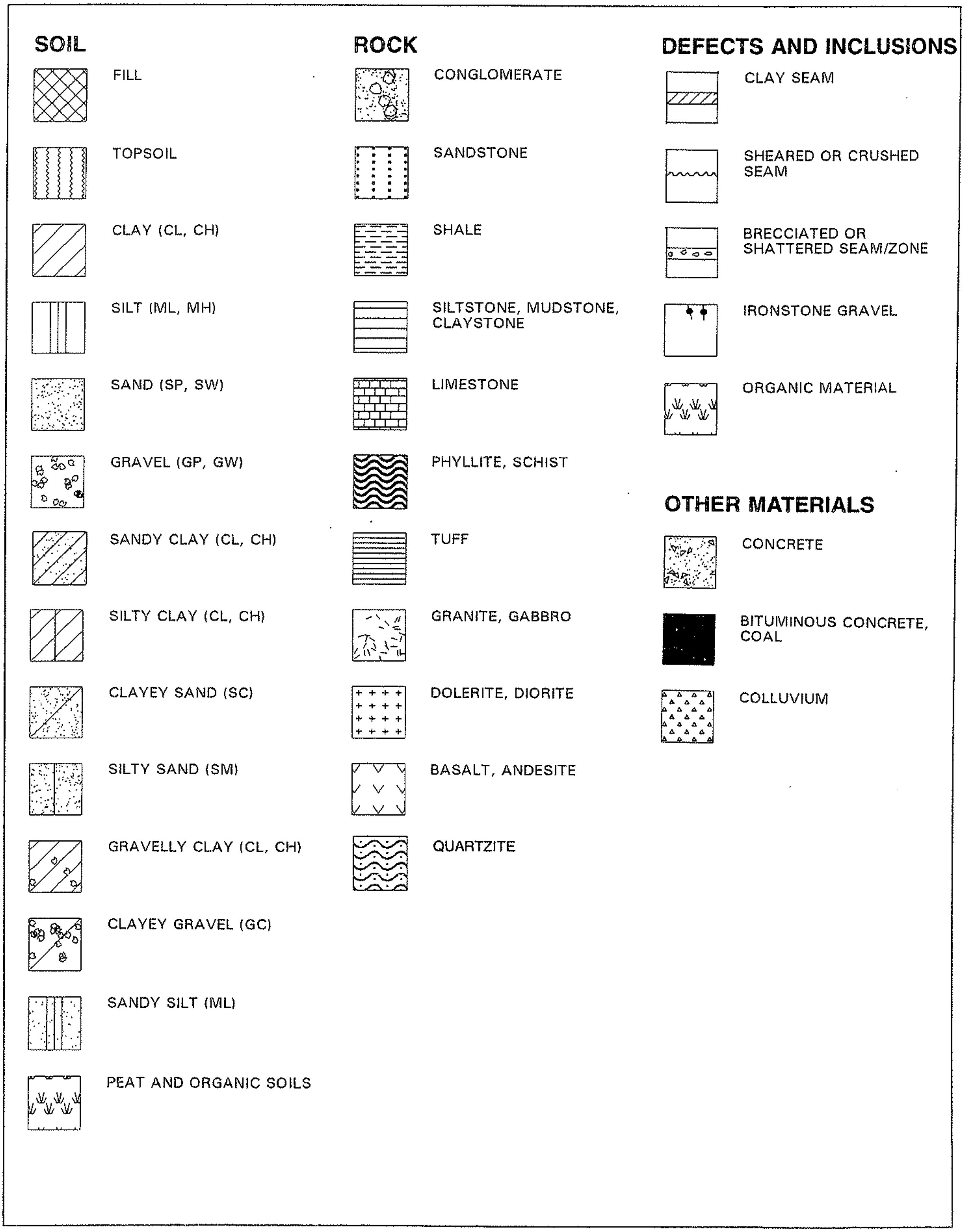
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



SOIL

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

ROCK

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

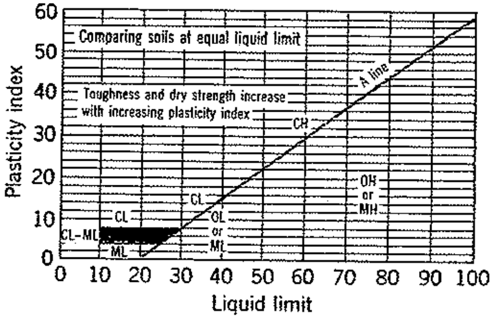
DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

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

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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LOG SYMBOLS

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

Ref: Standard Sheets/Log Symbols
November 2007

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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

Ref: Standard Sheets/Log Symbols
November 2007