REPORT

TO

SYDNEY ANGLICAN SCHOOLS CORPORATION

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED STAGE 1 ADDITIONS

TO

ST LUKE'S COLLEGE
TANGO AVENUE, DEE WHY, NSW

20 January 2009

Ref: 22631Zrpt

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Stage 1 building works at St Luke's Grammar School, Dee Why. The investigation was commissioned by Sydney Anglican School Corporation, by signed 'Acceptance of Proposal' form dated 17 December 2008. The commission was on the basis of our proposal (ref. P30384Zemail) dated 16 December 2008.

We understand from the Simpson Design Associates' geotechnical brief dated 15 December 2008, and from the provided architectural drawings (Drawing Nos A001, A100, A105 and A109) prepared by Tonkin Zulaikha Greer Architects, that the Stage 1 building works involve the construction of a new multi storey building comprising two levels of underground carparking, new classrooms at ground level and rooftop playing courts. A maximum excavation depth of about 6.3m will be required to achieve the finished lower basement floor reduced level (RL) at 57.5m. Column loads in the order of 2,500kN (working) have been anticipated by Simpson Design Associates.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at five nominated locations as a basis for comments and recommendations on excavation conditions, excavation techniques, excavation support, retaining walls, footings and on-grade floor slabs. We note that a contamination investigation was carried out in conjunction with the geotechnical investigation by our environmental division, Environmental Investigation Services (EIS). The contamination report (ref. E22631K) should be read in conjunction with this report.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation comprised the drilling of four boreholes (BH1 to BH4) and the completion of one Dynamic Cone Penetration (DCP) test (DCP5). Prior to the commencement of drilling or DCP testing, the investigation locations were electromagnetically scanned for buried services. The boreholes were auger drilled to depths between 1m and 3m using our track mounted JK250 drilling rig. Two of the boreholes were then extended to depths of 4.96m and 7.25m respectively, using core drilling techniques with recycled water flush. The DCP test was carried out in an area which was inaccessible to the rig and achieved refusal after 0.1m penetration. The test locations, as indicated on attached Figure 1, were set out using taped measurements from existing surface features and apparent site boundaries. The surface reduced levels (RLs) at the investigation locations, were estimated by interpolation between spot heights shown on the provided survey plan (ref. 12795K, dated 13/10/08) prepared by Paul Keen & Company. The survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsurface soils and rocks were assessed by logging the materials recovered during drilling. The relative density of the deeper fill was assessed from the Standard Penetration Test (SPT) 'N' number. The strength of the underlying augered bedrock was assessed by observation of the drilling resistance when using a tungsten carbide (TC) bit and examination of the recovered rock chip samples. The strength of the bedrock within the cored boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing. The DCP refusal provided an indication of the bedrock surface, though we note that refusal can also occur on buried obstructions, 'floaters' and other hard layers. Groundwater observations were made during augering, on completion of augering, and on completion of coring. Longer term groundwater monitoring was not carried out. For further details on the investigation procedure adopted, reference should be made to the attached Report Explanation Notes.

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

The recovered rock core was submitted to Soil Test Services (a NATA registered laboratory) where it was photographed and select sections of core subjected to Point Load Strength Index testing. The core photographs are presented opposite the borehole logs and the laboratory test results are plotted on the relevant borehole logs and summarised in attached Table A. The unconfined compressive strength (UCS) of the rock core, as estimated from the Point Load Strength Index testing, is also summarised on Table A.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in undulating topography over the north-east corner of the St Luke's College grounds. The school is bounded by Tango Avenue, along the east, Headland Road along the south, and slopes down to the north-west at 5° to 10°. The site itself is rectangular in plan shape, being about 45m wide (north to south), by 35m deep (east to west) and has an eastern frontage onto Tango Avenue.

At the time of the investigation, the eastern half of the site was occupied by a concrete paved open carpark area which had been excavated into the hillside along the south-west. The western half was occupied by a tennis/basketball court which had a single storey amenities/storage building below its northern end. A concrete ramp was located along the northern end of the site and provided access to the amenities/storage building.

Beyond the northern site boundary was bushland, to the south was the main three storey brick school building and to the west was a large grassed area. The school building appeared in good condition when viewed from within the subject site. A concrete paved area with shade-cloth covering was located between the school building and the site. The tennis/basketball court was supported above the large grassed area at its southern end by a brick wall which was founded on sandstone bedrock. Extensive sandstone bedrock was evident in this south-west corner of the grassed area. Sandstone bedrock was also evident over the cut face along Tango Avenue. The sandstone was assessed to be distinctly weathered and of low to medium strength, with cross bedding and bed partings evident. Groundwater seepage was visible from the base of the wall supporting the southern end of the tennis/basketball court and generally from bed partings within the exposed sandstone bedrock.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. The investigation has revealed a subsurface profile below the concrete surfacing comprising surficial fill over sandstone bedrock which extended to the borehole termination depths. Reference should be made to the attached borehole logs for detailed subsurface conditions at specific locations. A graphical borehole summary is presented in Figure 2 and a summary of the subsurface conditions, as encountered, is presented below:

- Concrete, 150mm and 120mm thick, was encountered at the surface of BH1 and BH2 respectively.
- Fill, comprising gravelly silty sand, with igneous and sandstone gravel, was encountered below the concrete in BH1 and BH2. Silty sand fill, with sandstone gravel and general rubble, was encountered from the surface in BH3 and BH4. The fill in BH4 was assessed to be poorly compacted.
- Sandstone bedrock was encountered at depths of 0.4m (BH1, BH2 and BH3) and 1.9m (BH4). Based on the DCP refusal, sandstone bedrock has been inferred at a depth of 0.1m in DCP5. The sandstone has been assessed to be distinctly weathered and of medium to high strength, except for BH4, where the upper 0.6m of sandstone was of low strength. Cored loss zones of 0.08m and 0.1m thick were encountered between depths of 2.45m and 2.8m in BH4. The remaining bedrock was generally of good quality, with few defects encountered.
- Groundwater was not encountered during or on completion of auger drilling the boreholes. The groundwater level which was measured at the completion of coring has probably been influenced by the introduction of water into the borehole to assist the coring process. The relatively high estimated recycled water return indicates a relatively impermeable rock mass.

3.3 Laboratory Test Results

The laboratory Point Load Strength Index test results correlate well with our field assessment of rock strength. The UCS of the rock, as estimated from the Point Load Strength Index tests, vary between 6MPa and 32MPa, with average values of about 23MPa in BH2 and 11MPa in BH4.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation Conditions

4.1.1 Excavation Methods

The results of our investigation indicate that the site area is characterised by a shallow fill profile with sandstone bedrock either at shallow depth or exposed at surface.

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

4.1.2 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 the adjoining school building which is located approximately 10m to the south. We recommend that continuous vibration monitoring be carried out during rock excavations. The vibrations on the school building should be limited to no higher than 5mm/sec. We note that this vibration limit is based on structural considerations with respect to potential vibration damage. A lower vibration limit would be appropriate to reduce discomfort to the building occupants.

The proposed excavation within bedrock should commence 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. Excavation with hydraulic rock hammers, if used, should commence over the north using a moderately sized excavator fitted with a relatively low energy hydraulic hammer no larger than a Krupp 580 or equivalent. If it is found that transmitted vibrations are excessive, then 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 grid sawing in conjunction with ripping and/or hammering. 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 oriented towards the face and enlarge excavation by breaking small wedges off the face.
- Operate one hammer at a time and 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.

4.1.3 Seepage

We would expect some groundwater seepage flows will occur over the rock surface, at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, during excavation, is expected to be satisfactorily controlled by conventional sump pumping.

We recommend that a toe drain be formed at the base of all rock cuttings to collect groundwater seepage and direct it to a sump for pumped disposal. The groundwater seepage should be monitored during bulk excavation so that unexpected conditions can be timeously addressed.

4.2 Excavation Support

Excavations in the soil profile may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1 H (Horizontal). On the basis of the provided architectural drawings and survey plan, it would appear that temporary batters can probably be accommodated within the site geometry. Where temporary batters cannot be accommodated, the shallow soil profile may be supported using sandbags or similar. Possible seepage at the soil-rock interface may cause localised instability at the toe of the soil batters and allowance should, in any event, be made for sand-bagging. Care is required during excavation not to undermine the existing Tango Avenue frontage nor the existing school buildings and structures to the south.

We expect that good quality sandstone of low or higher strength may be cut vertically. However, localised stabilisation measures may be necessary if adverse defects such as inclined joints or bedding are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay seams occurring in permanently exposed sandstone slopes may also require 'dental' treatment. We therefore recommend that the rock face be progressively inspected by a geotechnical engineer as excavation proceeds (ie at no more than 3m vertical intervals) to identify adverse defects and to propose appropriate stabilisation measures. We note that the details and extent of localised stabilisation measures can only be finalised following geotechnical inspection. However, based on the investigation results, localised stabilisation measures are expected to be minimal.

4.3 Retaining Walls

Retaining walls, if required, should be designed using the following characteristic earth pressure coefficients and subsoil parameters:

- The retaining walls should be uniformly founded in sandstone bedrock. For footing recommendations, refer to Section 4.4 below.
- Free-standing cantilever walls which support areas which are not sensitive to movement (eg where garden or open areas are being retained), should be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.35 for the soil profile, assuming a horizontal retained surface.
- Cantilever walls, the tops of which will be restrained by the permanent structure, or which support areas which are sensitive to movement, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, K_o, of 0.55 for the soil profile, assuming a horizontal retained surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile.
- All surcharge loads affecting the walls (eg traffic loads, construction loads, etc) should be taken into account into the wall design, using the appropriate earth pressure coefficient from above.
- The walls should be designed as drained, and measures taken to provide permanent and effective drainage of the ground behind the walls. Subsoil drains should incorporate a non-woven geotextile fabric, eg Bidim A34, to act as a filter against subsoil erosion.
- Lateral toe restraint may be achieved by keying the wall footing into sandstone bedrock below bulk excavation level. An allowable lateral stress of 200kPa may be used for key design subject to geotechnical inspection. Footings founded in sandstone adjacent to the crest of a cut or natural cliff face, should be checked for lateral restraint based on the concrete to rock friction, using a

friction angle of 35°. The use of rock dowels for lateral restraint is also feasible; such rock dowels should be grouted into holes which are drilled down and away from the rock face and provided with a cogged end for incorporation into the footing. An allowable bond stress of 200kPa may be used for dowel embedment design.

4.4 Footings

The proposed building should be uniformly supported on sandstone bedrock. Conventional strip or pad footings founded in sandstone bedrock of at least low strength may be designed for an allowable bearing pressure of 1,000kPa. Where there is doubt regarding the quality of sandstone exposed, further geotechnical advice should be sought. The allowable bearing pressure may be increased to 3,500kPa subject to inspection by a geotechnical engineer. Spoon testing of a select number of footing excavations may however also be required.

4.5 On-Grade Floor Slab

The proposed on-grade floor slab will directly overlie sandstone bedrock. Under-floor drainage must therefore be provided. The under-floor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The under-floor drainage should connect with the wall drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the concrete on-grade floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.6 Further Geotechnical Input

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

- Vibration monitoring during rock excavations.
- Geotechnical inspections of cut rock faces.
- Monitoring of groundwater seepage into bulk excavation.
- Geotechnical footing inspections, if appropriate.

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.

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 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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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	1.25-1.27	0.8	16
	1.78-1.81	0.9	18
	2.66-2.69	1.3	26
	3.10-3.12	1.1	22
	3.79-3.82	1.1	22
	4.48-4.53	1.6	32
4	2.86-2.90	0.4	8
	3.52-3.57	0.4	8
	4.48-4.52	0.8	16
	5.65-5.70	0.9	18
	6.53-6.57	0.5	10
	7.13-7.16	0.3	6

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 $I_{S(50)}$

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Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1/1

BOREHOLE LOG

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION
Project: PROPOSED ADDITIONS
Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z
Date: 22-12-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 61.6m
Datum: AHD
Logged/Checked by: J.P./ [signature]

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			CONCRETE: 150mm.t				
	ES, DS				-	FILL: Gravelly silty sand, fine to medium grained, grey, with igneous gravel.	M	-	-	
					-	SANDSTONE: fine to coarse grained, light yellow.	DW	M-H	-	MODERATE TO HIGH 'TC' BIT RESISTANCE
			1							HIGH RESISTANCE
	ES									
						END OF BOREHOLE AT 1.5m				PRACTICAL REFUSAL
			2							
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **2**
1/2

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION
Project: PROPOSED ADDITIONS
Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z
Date: 22-12-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 59.8m
Datum: AHD
Logged/Checked by: J.P./

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			CONCRETE: 120mm.t				NO OBSERVED REINFORCEMENT
	ES, DS				-	FILL: Gravelly silty sand, fine to coarse grained, light brown, with sandstone gravel.	M	-	-	
	DS	SPT 8/10mm REFUSAL			-	SANDSTONE: fine to coarse grained, light brown.	DW	M-H	-	MODERATE TO HIGH 'TC' BIT RESISTANCE
			1			REFER TO CORED BOREHOLE LOG				
			2							
			3							
			4							
			5							
			6							
			7							

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JOB NO: 22631Z
BH: 2
START CORING AT 1.0m
1
2
3
4
END OF BH 4.96
END OF BOREHOLE AT 4.96m

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CORED BOREHOLE LOG

Borehole No. 2

2/2

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION
Project: PROPOSED ADDITIONS
Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 59.8m
Date: 22-12-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** J.P./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0								
		1		START CORING AT 1.0m						
ON COMPLETION OF CORING				SANDSTONE: fine to coarse grained, grey, with iron stained.	DW	M	×			
AFTER 3 HRS							×			
		2				H				
100% RETURN							×		- Be, 0-5°, P, S, IS	
		3					×			
				as above, but with iron indurated bands.						
				SANDSTONE: fine to coarse grained, grey.	SW				- Be, 5°, P, S, IS	
		4					×			
							×			
		5		END OF BOREHOLE AT 4.96m						
		6								
		7								

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Borehole No.
3
1/1

BOREHOLE LOG

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION

Project: PROPOSED ADDITIONS

Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z **Method:** SPIRAL AUGER JK250 **R.L. Surface:** ≈ 60.4m

Date: 22-12-08 **Datum:** AHD

Logged/Checked by: J.P./

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/TOPSOIL: Silty sand, fine to medium grained ,brown, with a trace of sandstone and a trace of roots.	M			
					-	SANDSTONE: fine to coarse grained, grey and brown, with a trace of ironstone gravel.	DW	M	-	LOW TO MODERATE 'TC' BIT RESISTANCE
			1							
			2					H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 3.0m				PRACTICAL REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **4**
1/2

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION
Project: PROPOSED ADDITIONS
Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z
Date: 22-12-08
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 60.2m
Datum: AHD
Logged/Checked by: J.P./

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 OF AUGERING	ES	N = 2 3,1,1	0			FILL: Silty sand, fine to medium grained, brown, with sandstone gravel and rubble and a trace of roots.	M			GRASS COVER APPEARS POORLY COMPACTED
	ES	N > 12 4,4,8/ 70mm REFUSAL	1							
AFTER 1.5 HRS OF CORING			2		-	SANDSTONE: fine to coarse grained, light grey.	DW	L	-	
						REFER TO CORED BOREHOLE LOG				
			3							
			4							
			5							
			6							
			7							

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JOB NO: 22631Z
BH:4
START CORING AT 2.45m
2
2.45m
CORE LOSS 0.88
CORE LOSS 0.10
3
4
5
6
7
END OF BOREHOLE AT 7.25m

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CORED BOREHOLE LOG

Borehole No. 4

Client: SYDNEY ANGLICAN SCHOOLS ASSOCIATION

Project: PROPOSED ADDITIONS

Location: ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW

Job No. 22631Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 60.2m

Date: 22-12-08 | **Inclination:** VERTICAL | **Datum:** AHD

Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** J.P./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL VL L M H VH EH)	DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS – DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
		1								
		2								
				START CORING AT 2.45m						
				SANDSTONE: fine to medium grained, grey and light brown.	DW	M				
				CORE LOSS 0.08m	DW	M				
				SANDSTONE: fine to medium grained, grey and light brown.	DW	M	×			
		3		CORE LOSS 0.10m						
ON COMPLETION OF CORING				SANDSTONE: fine to medium grained, grey and light brown.			×			
		4		as above, but grey.						
							×		- Cr, 20mm.t	
									- Be, 30°, P, S, IS	
		5		SANDSTONE: fine to coarse grained, grey and light brown, bedded at 30°.					- Be, 30°, P, S	
				as above, but cross bedded at 30°.			×			
70% RETURN		6								
							×			
		7					×			
				END OF BOREHOLE AT 7.25m						
		8								

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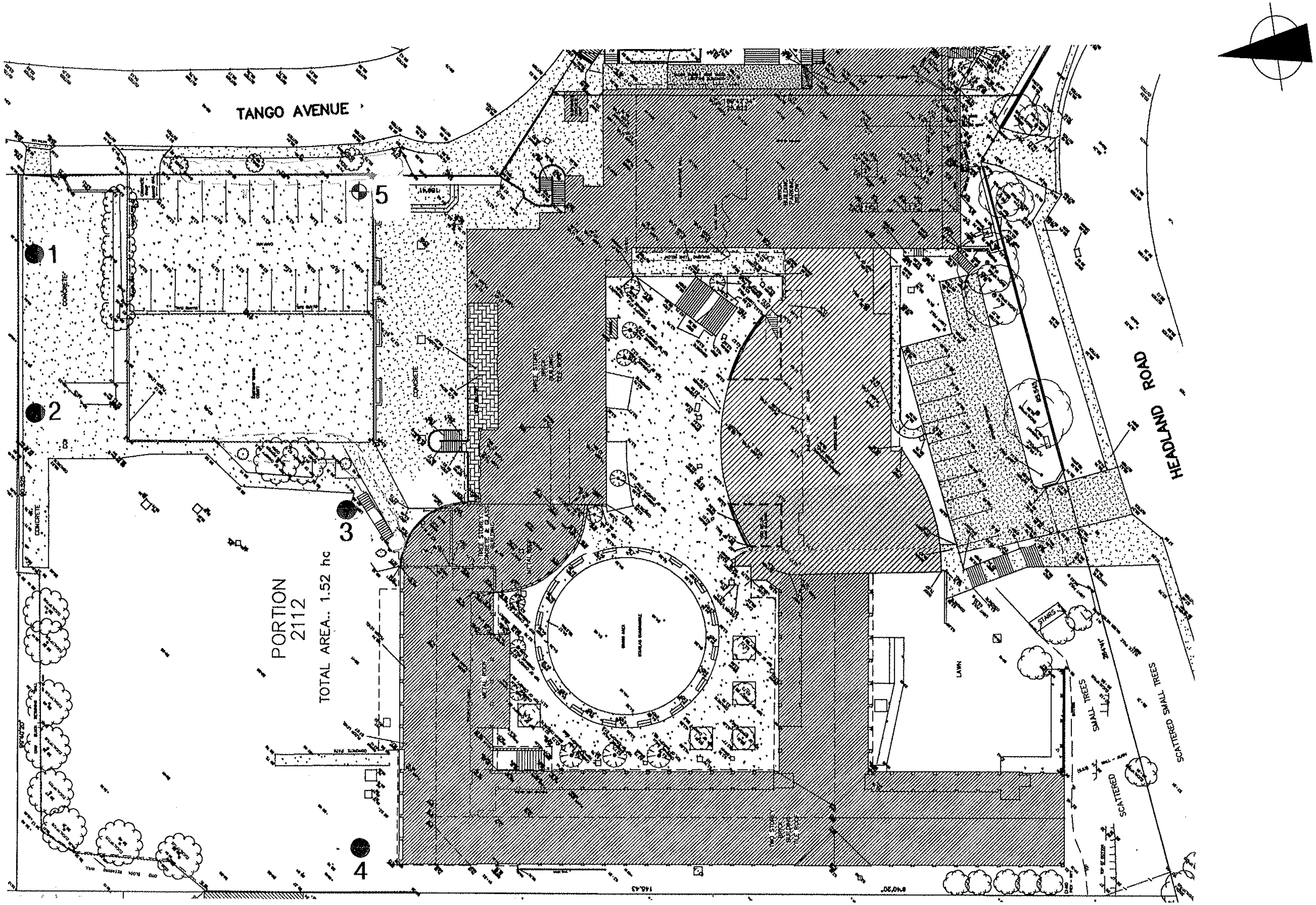
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	SYDNEY ANGLICAL SCHOOLS ASSOCIATION		
Project:	PROPOSED ADDITIONS		
Location:	ST LUKES COLLEGE, TANGO AVENUE, DEE WHY, NSW		
Job No.	22631Z	Hammer Weight & Drop: 9kg/510mm	
Date:	22-12-08	Rod Diameter: 16mm	
Tested By:	J.P.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration							
Test Location	RL ~63.6m						
Depth (mm)	**5**						
0 - 100	1						
100 - 200	20/10mm						
200 - 300	REFUSAL						
300 - 400							
400 - 500							
500 - 600							
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
3. Survey datum is AHD.

Ref: Scala3.xls April 99



LEGEND

BOREHOLE

DCP TEST

SCALE (m)

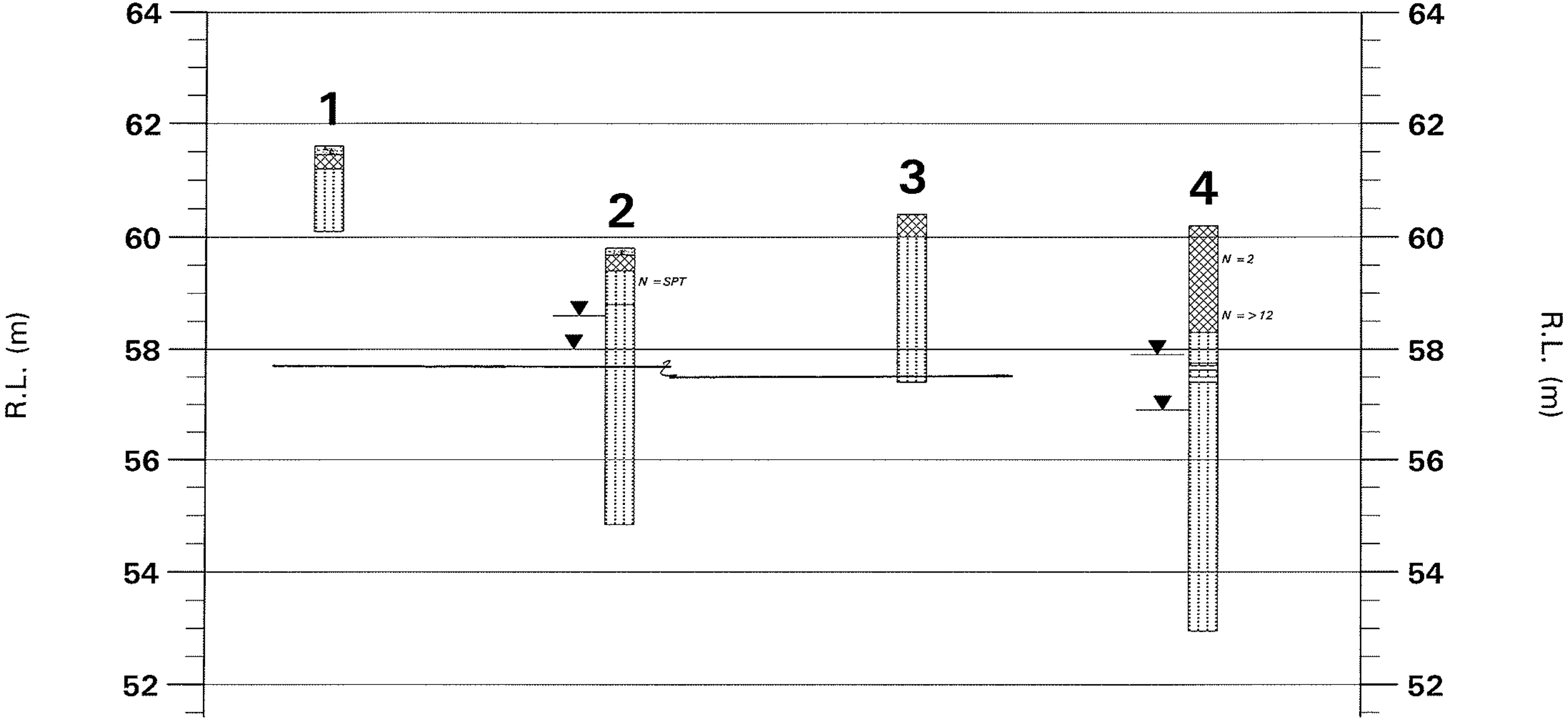
0 25

INVESTIGATION LOCATION PLAN

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 22631Z **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY



Concrete	Core Loss/ Empty	Nc	SOLID CONE BLOW COUNTS PER 150mm		
Fill	Observed water level				
Sandstone/ Greywacke	N	SPT "N" VALUE			

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 22631Z

Figure No.: 2

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

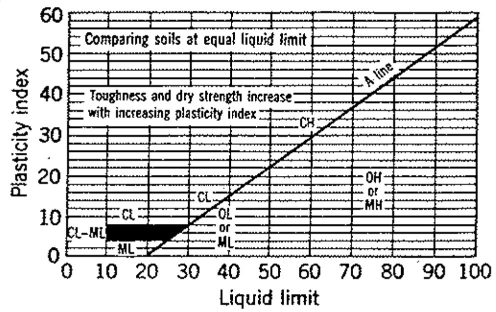
UNIFIED SOIL CLASSIFICATION TABLE

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	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20 % hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Laboratory criteria for coarse-grained soils: 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.

Use grain size curve in identifying the fractions as given under field identification

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



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (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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ABN 17 003 550 801

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		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

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

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	