REPORT

TO

UNIVERSITY OF TECHNOLOGY SYDNEY

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MULTI PURPOSE SPORTS HALL

AT

UTS BROADWAY CAMPUS, THOMAS ST, ULTIMO

21 July 2009

Ref: 23119SPrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed multi-purpose sports hall within the Broadway campus of the University of Technology Sydney. The proposed hall lies within the Alumni Green area, which is on the inside of the 'L' shaped Building 4 and extends to the basement of Building 1.

The proposed hall will have a floor level of about RL 3.4 metres (m) AHD (Australian Height Datum) which will require excavation to between 9.5m and 10.5m below the existing surface levels. This excavation will extend about 6m below the lowest level of Building 4, and will terminate several metres above the lowest level of Building 1. Following the construction of the sports hall, there will be about 2m thickness of soil and soft landscaping placed above the hall to reinstate the Alumni Green.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for our comments and recommendations on excavation, shoring, underpinning, footing design and hydrogeological considerations.

An investigation for extensions to the adjacent Building 4 was undertaken by Coffey Geosciences Pty Ltd in August 2003 (Reference S21586/1). Boreholes 2 and 3 from that investigation have been included in Appendix A of this report and the information used to prepare our recommendations.

2 INVESTIGATION PROCEDURE

At the commencement of the fieldwork, the borehole locations were electromagnetically scanned for buried services by an external service location contractor. Ground penetrating radar was then used to undertake a scan of the perimeter of the proposed sports hall to check for possible major service lines passing into the development area. No major services were encountered.

The fieldwork for the investigation comprised the drilling of:

- Borehole 101 to a depth of 2.8m using spiral auger techniques, which was then extended to a depth of 23.7m using NMLC diamond coring techniques. This borehole was drilled vertically.
- Borehole 102 to a depth of 3.6m using spiral auger techniques, which was then extended to a depth of 14.0m using NMLC diamond coring techniques. This borehole was drilled at an inclination of 35° to the horizontal, and was inclined to pass below the northern wing of Building 4.
- Borehole 103 which was augered drilled to a depth of 2.8m and then diamond cored to 14.6m depth. This borehole was also drilled vertically.
- Borehole 104 which was auger drilled to a depth of 0.5m. There were two attempts to drill the borehole with each achieving refusal to further penetration at a depth of 0.5m.

These boreholes were drilled using truck and track mounted drilling rigs. The borehole locations, as shown on the attached Figure 1, were determined by taped measurements from surface features shown on the plan.

The core recovered from the boreholes was placed in steel boxes and delivered to a NATA registered laboratory where it was colour photographed and Point Load Strength Index tests completed.

The fieldwork was completed in the full time presence of a geotechnical engineer, who set out the boreholes, nominated the testing and sampling locations, and prepared logs of the strata encountered in the boreholes. The borehole logs are attached, together with a glossary of the terms and symbols used in the logs.

The strength of the soils was assessed by correlation from the Standard Penetration Test (SPT) 'N' values and hand penetrometer test results on cohesive samples recovered from the SPT sampler. The strength of the rock was initially assessed from examination of the rock core, and was later confirmed by correlation with the results of Point Load Strength Index tests completed on the recovered core. The results of the Point Load Strength Index tests are summarised in Table a and are shown on the cored borehole logs.

Following the completion of the drilling, the site was visited by an Associate who accessed the basement space of Building 1 to map the conditions exposed in the northern face of the basement, to undertake a quick inspection of the conditions exposed in the eastern face of that basement, and also to inspect the sides of the vehicle tunnel which provides a link between the basement and Thomas Street to the north. This tunnel corresponds with the western extent of the proposed sports hall. The mapping of the northern face of the Building 1 basement is shown in Figure 2.

For further details of the investigation techniques adopted, reference should be made to the attached Report Explanation Notes.

An environmental screening of the soils and groundwater has been undertaken by Environmental Investigation Services (EIS). Reference should be made to the EIS report for details of any contamination of the soil and groundwater.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located at and near the broad crest of a hill in a region of very gently sloping topography. The local high point is approximately at the southern side of the site, with the land sloping generally to the north at about 2° to 3°.

The site currently forms part of the Alumni Green, and contains lawn with pathways and low height retaining walls. Adjacent to the southern wall of the north wing of

Building 4, the ground level is about 2m lower than the adjacent alumni green, with a sandstone faced retaining wall supporting the Green.

The site is bounded to the south by the basement of Building 1, and to the east and north by Building 4. The vehicle access tunnel to the Basement of Building 1 passes along the western side of the site.

3.2 Subsurface Conditions

The boreholes from this and previous investigations, and the rock exposures in the vehicle access tunnel and the Building 1 basement show the site to be underlain by fill and residual silty clay over sandstone bedrock. Some of the characteristic features of the strata encountered are described below. Fur further details of the strata encountered at each location, reference should be made to the attached borehole logs.

Fill

The fill encountered in the boreholes extended to depths of the order of 1m, but locally deeper in BH102. The fill comprised silty clayey sand and gravely sandy clay. The fill was moist (or had a moisture content of its plastic limit) and was moderately compacted.

Residual Silty Clay

Residual silty clay of high plasticity was encountered from about 1m depth in BH101 and BH103. In BH101, the clays were of very stiff strength with moisture contents in excess of the plastic limit, while in BH103, the moisture contents were below the plastic limit, with the soil being of hard strength and interbedded with sandstone bands.

Sandstone Bedrock

Sandstone bedrock was encountered from depths of about 2.0m in the boreholes, with the upper metre of sandstone often being of extremely low to low strength with clay bands. Below about 3.0m depth, the sandstone was distinctly weathered and slightly weathered and of medium and high strength. There were occasional near horizontal clay seams within the cored portions of the boreholes, as well as joints with inclinations of 35° to 70° to the horizontal. The inclined borehole below the northern wing of Building 4 did not encounter any joints which would appear to be extensive in nature.

The sandstone bedrock exposed in the Building 1 basement and the vehicle access tunnel was very similar to that encountered in the boreholes. The sandstone was generally of high strength with widely spaced near horizontal bedding parts and occasional steeply inclined joints which are mostly strata bound (that is they do not normally extend beyond the bedding partings). The soil in the upper portion of the cut for Building 1 was supported by a concrete retaining wall which had a height of about 1.5m.

Groundwater

Groundwater was not encountered within the augered portion of the boreholes. A PVC standpipe was installed in BH101, though there was an obstruction in the cored portion of the borehole which prevented the PVC extending past 15m depth. Five days after the installation of the piezometer, a water level was measured at a depth of 14.7m. As this was very close to the base of the borehole, it could not be determined whether this was a true groundwater level. Based upon the conditions exposed in the adjacent Building 1 basement, we expect that this a collection of minor seepage rather than a true groundwater level.

There was a concentrated flow of water flowing over the eastern portion of the northern face of Building 1, and a sample of this was delivered to an analytical laboratory for testing. This sample showed the presence of Fluoride, suggesting the water is from a leaking town water supply.

Ground Penetrating Radar

The ground penetrating radar did not encounter any anomalies which would be consistent with a large buried service entering the site. There was one return from the radar which corresponded with where the vehicle access tunnel passed along the western side of the site.

Laboratory Test Results

The Point Load Strength Index test results provided in Table A correlate well with the field logging assessments of rock strength.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation Conditions

The excavation for the proposed multi-purpose hall will extend through the soils and into the sandstone bedrock. The soils should be readily excavated using buckets on conventional excavators, and should be temporarily battered at no steeper than 1 Vertical (V) in 1 Horizontal (H).

The sandstone bedrock to low strength should be excavated using ripping tynes on large, say 30 tonne excavators, and should also be battered no steeper than 1V in 1H. The sandstone of medium and high strength contains relatively few defects, and will require the use of rock breaker attachments to large excavators. Alternatively, the excavation could be undertaken using lower percussive techniques such as using large excavator mounted rock saws to saw the sandstone into blocks and then use ripping tynes or hammers to loosen these blocks. The latter method of excavation may mean that the sandstone could be utilised as a quarried product for beneficial purposes.

The excavation of sandstone usually produces vibrations, with the higher vibration usually occurring when using rock breaker attachments to excavators. Where vibration issues are critical, consideration should be given to the use of rock saw or grinder attachments to excavators for the excavation to reduce possible transmitted vibrations.

There were several joints encountered within the sandstone core and observed in the cut faces for the Building 1 basement. When combined with near horizontal bedding partings, these can result in potentially unstable blocks or wedges of rock in the cut faces. Therefore, the cut faces should be progressively inspected by a geotechnical engineer such that any potentially unstable areas can be removed or stabilised. These inspections should be conducted following each 2m lift of excavation. Any stabilisation measures detailed during these inspections should be completed prior to recommencing excavation in that area.

Within the Sydney region, there are occasional dykes which generally run in west-northwest to east-southeast direction. Dykes can have a significant effect on basement excavations as they often act as a preferential flow path for water resulting in high concentrated inflows, as well as stability issues and possible founding problems where footings are coincident with the dyke. During the inspection of the Building 1 basement and the vehicle access tunnel, there was no evidence of a dyke passing through the site; there is still a possibility that there could be a dyke passing close to the north-eastern corner of the site. The possible (though remote) chance of there being a dyke in the north-eastern corner of the site could be discounted following the excavation of a trial trench from the north-eastern corner of the site in a south-westerly direction, with the trench extending down onto the bedrock and being inspected by a geotechnical engineer.

There are also joint swarms within Sydney which generally run in a north-northeast to south-southwest direction. Based upon mapping of past encounters with these joint swarms, it appears however that the Martin Place joint swarm would pass to the east of the site, and the GPO Fault zone to the west of the site.

In Sydney, there is a relatively high in-situ horizontal stress field. When excavations extend down into the sandstone bedrock, these horizontal stresses are relieved, resulting in movements of the excavated faces into the excavation. These movements are often of the order of 1mm of lateral movements for each metre depth of excavation into the sandstone. Due to the stresses involved, it is not feasible to try to restrain the face from these movements. When considering the effect of the proposed excavation on the existing Building 4, these movements must be taken into account. Therefore, at an early stage in the planning, a structural engineer should be asked to comment on the whether such movements are likely to cause unacceptable movements to Building 4. If these movements are considered critical, then further detailed investigation comprising in-situ rock stress testing then detailed finite element modelling could be undertaken to better assess the potential movements; we note that this level of detailed investigation and analysis are rarely undertaken within Sydney except in relation to rail infrastructure.

When the excavation has reached the existing founding level of Building 4, a detailed geotechnical inspection, including the excavation of test pits adjacent to each footing, should be undertaken to assess the potential presence of adversely oriented defects. If such defects are encountered, it will be necessary to either undertake stabilisation of the sandstone below the Building 4 footings, or to underpin the footings. It is more likely that stabilisation would be the option adopted due to the difficulty in underpinning heavily loadings footings.

Dust issues can arise during the excavation of sandstone bedrock. The usual method for the control of dust involves a fine water spray in the vicinity of the excavation. We also understand that there is a system where plastic can be draped over a frame over the work area, however we have no further details on its availability.

During the excavation, there could be some inflows of groundwater, particularly after periods of heavy or prolonged rainfall. These seepages are more likely to be localised through bedding partings and joints. Based upon the conditions encountered in the boreholes, and the observations of the conditions in the basement of Building 1, we expect that any seepage would be readily controlled using conventional sump and pump techniques (provided the leak associated with the town water flowing over the Building 1 basement cut face has been repaired).

4.2 Retaining Wall Design

It appears that the preferred method of construction at the University of Technology Sydney is to have free standing basement walls, and then to construct the new building within the excavation such that the long term stability of the cut batters is not provided by the building. This would require the use of temporary batters within the soil and more weathered bedrock to enable retaining walls such as cantilevered concrete walls to be founded on the sandstone of at least medium strength. Following construction, the retaining walls would then be backfilled with either engineered fill or with clean and durable single size aggregate.

This building may be different to the rest of those already on site, as following the construction, soil and landscaping will be placed above the completed structure. The method of construction in this case could be to batter the soil, construct the internal structure and then extend the roof slab of the hall to the top of the vertical cut in the sandstone bedrock with a downturn from the roof slab to the top of the vertical cut in the sandstone bedrock. Inclined starter bars could be grouted into the sandstone bedrock which would be of at least medium strength, to assist with carrying the lateral loads in the roof slab. Consideration could be given to having a small upturn on the roof slab over the top of the downturn to the sandstone bedrock; this could be used to contain the edge of a drainage blanket over the roof structure and limit the amount of water spilling from the roof slab onto the sandstone bedrock adjacent to the downturn.

Along the southern side of the site, the excavation will join the excavation for the Building 1 basement, and so in this area, the soil above the sports hall would be supported by a retaining wall turning up from the southern edge of the hall roof slab. As the retaining wall will be laterally restrained at the base, it should also be designed for an at rest earth pressure coefficient.

Where the retaining walls would be free standing, they should be designed for an active lateral earth pressure coefficient (Ka) of 0.35, assuming the backfill to the walls will be near level. Where they will be restrained such as in the case of a downturn from the roof slab, they should be designed using an at-rest earth pressure coefficient (Ko) of 0.55.

The retaining wall design should also incorporate appropriate surcharge loads and hydrostatic pressures. The design of the retaining walls should incorporate drainage to effectively and permanently reduce the pore water pressures.

4.3 Footing Design

The proposed floor level of the multi-purpose sports hall is 3.4m AHD, which will result in footings for the proposed hall being at a level of about 2.5m AHD. Based upon the results of the boreholes and the exposures mapped on the northern face of the Building 1 basement, the footings for the proposed sports hall will be founded within high strength sandstone with very few defects. This sandstone appears to be of at least Class II quality in accordance with 'Design Loadings for Foundations on Shale and Sandstone in the Sydney Region' by Pells et al 1978.

The design pressures for footings founded within this sandstone will be dependent upon the amount of proving to be undertaken during construction. An allowable bearing pressure of 6000kPa may be adopted provided that all footings are visually inspected by a geotechnical engineer, and spoon tests are completed within at least half of all footings.

For a lesser degree of proving such as visually inspecting all footing excavations, and completing spoon tests in at least one in five footings, an allowable bearing pressure of 3500kPa may be adopted.

We expect that the above bearing pressures would be suitable for the design of the proposed sports hall based upon the conditions encountered in the boreholes drilled to date, an allowable bearing pressure of 10,000kPa is likely to be feasible but would require the completion of a cored boreholes in at least half of all footings, with spoon tests completed in all of the remaining footings.

Part of the Building 1 basement extends below the proposed founding level of the multi-purpose sports hall, such that the perimeter footings could be founded above a vertical cut in the sandstone bedrock. However, the step between the two basement levels will only be of the order of 1m, and so it would be feasible to deepen the footings along that side of the site to ensure they found below the levels of the Building 1 basement.

4.4 Hydrogeological Considerations

As mentioned above, there appears to be a leaking service which results in water flowing over the eastern end of the northern cut face of Building 1. Apart from the expense and loss of resource, this water could eventually cause some damage and erosion of the rock face. Therefore, the source of this water should be determined and the service repaired.

Based upon the information obtained from the boreholes and from the adjacent Building 1 excavation, it appears that the groundwater level will be at least several metres below the proposed bulk excavation level. We note this is a very low groundwater level, corresponding with a reduced level of about 0.1m AHD, and it possible that the groundwater level is locally depressed by the drainage effects of the basement of Building 1. The seepage is therefore likely to be limited to minor seepage through defects such as joints and bedding partings, and from the soil rock interface. We expect that this water would be readily controlled using conventional sump and pump techniques.

The current design involves having about 2m of soil and soft landscaping above the roof of the sports hall. It will be necessary to pay particular attention to the detail of the waterproofing and drainage in this area. We presume that a watertight seal will be constructed above the roof of the sports hall. Immediately above this seal should be a drainage blanket of uniform size, hard, durable and free draining crushed rock. This should connect to a collection pit from which water can be pumped and either disposed of or reused for watering purposes. To aid the collection of the water and reduce the potential for ponding on the roof slab, consideration should be given to having a slight fall on the upper surface of the roof slab to promote runoff through the drainage blanket.

It is likely that there will be some seepage into the excavation, particularly at the soil/rock interface and from defects within the rock mass. We expect that the amount of seepage would be controllable using conventional sump and pump techniques.

4.5 Aggression of Groundwater on Buried Structures

The piezometer installed in BH101 silted up between its installation and attempting to sample the groundwater several days later. At this stage, we have not been able to collect a sample of the groundwater for analysis. We understand that EIS are in the process of clearing the silt from the standpipe to allow sampling of the groundwater. When we have been able to test a sample of the groundwater, we will provide an addendum to this report on the aggressiveness of the groundwater.

However, we note that the water and silt level in the piezometer was a depth of 14.0m, corresponding with a reduced level of about 0.1m AHD which is about 3m below the bulk excavation level for the sports hall, and so it is unlikely the structure would be in prolonged contact with the groundwater.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, the geotechnical inspection of pad footing excavations, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and 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.

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

The subsurface conditions between and beyond 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.

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.

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per

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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 23119SP
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TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	2.91-2.94	0.1	2
	3.29-3.32	1.1	22
	3.73-3.76	0.8	16
	4.32-4.35	0.4	8
	4.80-4.83	1.2	24
	5.26-5.30	1.2	24
	5.80-5.83	1.9	38
	6.26-6.30	1.0	20
	6.79-6.84	1.0	20
	7.34-7.36	1.1	22
	7.80-7.83	1.2	24
	8.24-8.27	1.1	22
	8.71-8.74	1.5	30
	9.24-9.28	1.5	30
	9.79-9.83	1.3	26
	10.26-10.29	0.8	16
	10.76-10.80	1.0	20
	11.28-11.32	1.2	24
	11.75-11.78	0.9	18
	12.25-12.29	1.0	20
	12.79-12.83	1.2	24
	13.30-13.33	1.4	28
	13.79-13.83	2.5	50
	14.30-14.33	1.1	22
	14.83-14.86	1.6	32

NOTES: See Page 4 of 4

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TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	15.31-15.34	1.5	30
	15.78-15.82	1.0	20
	16.29-16.32	1.3	26
	16.73-16.78	1.1	22
	17.21-17.24	1.8	36
	17.74-17.77	1.9	38
	18.26-18.29	1.3	26
	18.70-18.73	1.5	30
	19.21-19.23	1.7	34
	19.70-19.74	1.2	24
	20.25-20.29	1.7	34
	20.79-20.82	1.0	20
	21.29-21.32	1.6	32
	21.79-21.82	1.4	28
	22.30-22.33	1.8	36
	22.79-22.82	1.3	26
	23.30-23.33	1.9	38
102	3.80-3.84	0.7	14
	4.25-4.29	0.2	4
	6.18-6.21	0.2	4
	6.69-6.72	0.3	6
	7.28-7.32	0.1	2
	7.72-7.75	1.3	26
	8.22-8.25	1.0	20
	8.77-8.81	1.3	26

NOTES: See Page 4 of 4

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TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
102	9.21-9.26	0.6	12
	9.78-9.81	0.4	8
	10.57-10.60	1.0	20
	10.89-10.92	0.9	18
	11.30-11.35	1.2	24
	11.75-11.79	0.6	12
	12.24-12.27	1.2	24
	12.74-12.77	1.1	22
	13.22-13.25	0.9	18
	13.85-13.88	0.8	16
103	3.21-3.23	0.9	18
	3.70-3.73	1.8	36
	4.20-4.23	1.1	22
	4.70-4.73	1.0	20
	5.19-5.22	0.8	16
	5.70-5.73	1.8	36
	6.19-6.23	1.9	38
	6.78-6.81	0.1	2
	7.21-7.24	1.5	30
	7.74-7.78	0.3	6
	8.30-8.33	1.4	28
	8.75-8.78	1.1	22
	9.24-9.27	1.4	28
	9.73-9.76	1.2	24
	10.25-10.28	1.0	20

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TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
103	10.75-10.79	1.3	26
	11.29-11.33	0.9	18
	11.80-11.83	1.5	30
	12.28-12.31	1.0	20
	12.81-12.84	1.0	20
	13.28-13.32	0.9	18
	13.80-13.83	1.4	28
	14.37-14.40	1.6	32

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 LOG

Borehole No.
101
1/5

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 27-6-09
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 14.1m
Datum: AHD
Logged/Checked by: J.C./PW

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 OF AUGERING	■ ■			■ ■	N = 8 6,4,4	0 1	▨		FILL: Silty clayey sand, fine to medium grained, dark brown, with fine to medium grained sandstone gravel and a trace of roots.	M			GRASS COVER APPEARS MODERATELY COMPACTED
				■	N = 6 3,3,3	2	▨	CH	SILTY CLAY: high plasticity, red brown mottled orange brown and light grey, with fine to medium grained H strength ironstone gravel.	MC>PL	VSt	- 230 320 320	RESIDUAL
				■			⁞	-	SANDSTONE: fine to medium grained, light grey, red brown and orange brown.	DW	L	-	LOW 'TC' BIT RESISTANCE
						3 4 5 6 7			REFER TO CORED BOREHOLE LOG				

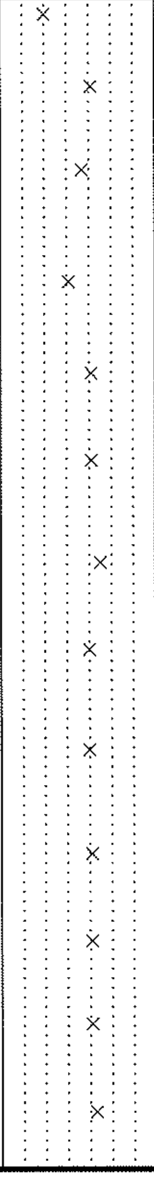

JEFFERY & KATAUSKAS PTY LTD
JOB NO 23119SP BH101 START CORING AT 2.83m
2
3
4
5

CORED BOREHOLE LOG

Borehole No. 101
2/5

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 27-6-09
Drill Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 14.1m
Datum: AHD
Logged/Checked by: J.C./RW

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
		2		START CORING AT 2.83m						
		3		SANDSTONE: fine to medium grained, light grey, orange brown and red brown, with orange brown laminae, bedded at 0-5°.	DW	M-H				
		4							- CS, 5mm.t, P, S	
						H				
		5								
FULL RET-URN										
		6		SANDSTONE: fine to coarse grained, grey, with dark grey laminae, bedded at 10-15°	SW				- Be, 5°, P, R, IS	
		7								
		8							- Cr, 20mm.t	
		9								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L 995
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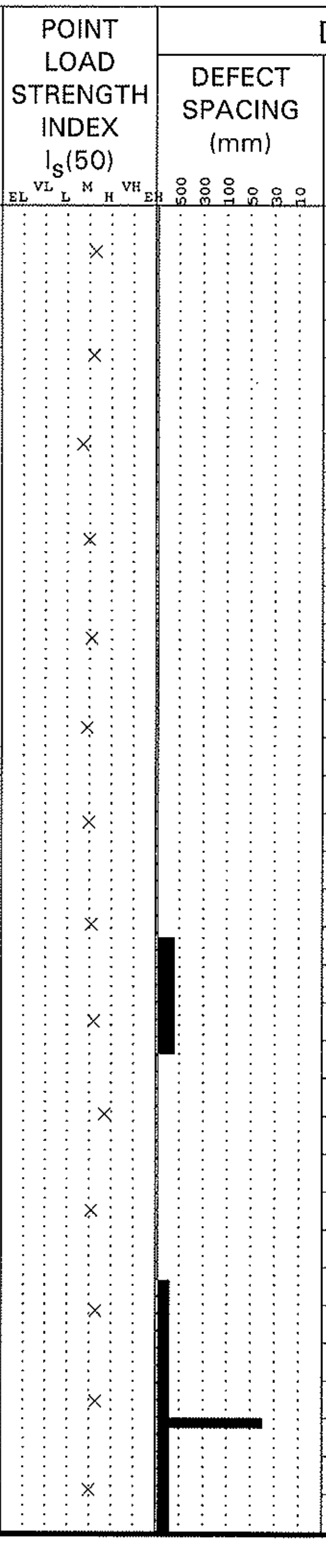
CORED BOREHOLE LOG

Borehole No.
101
3/5

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 14.1m
Date: 27-6-09
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.C./PW



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
FULL RET-URN				SANDSTONE: fine to coarse grained, grey, with dark grey laminae, bedded at 10-15°	SW	H		
				SANDSTONE: fine grained, grey, with dark grey laminae, bedded at 0°.		M-H		
		10						
		11						
		12				H		
							- J, 65°, P, S	
		13						
							- J, 70°, P, S	
		14						
				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 2-15°.			- J, 55°, P, S, CLAY INFILL	
		15						
							- J, 65°, P, S	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16
17
18
19
20

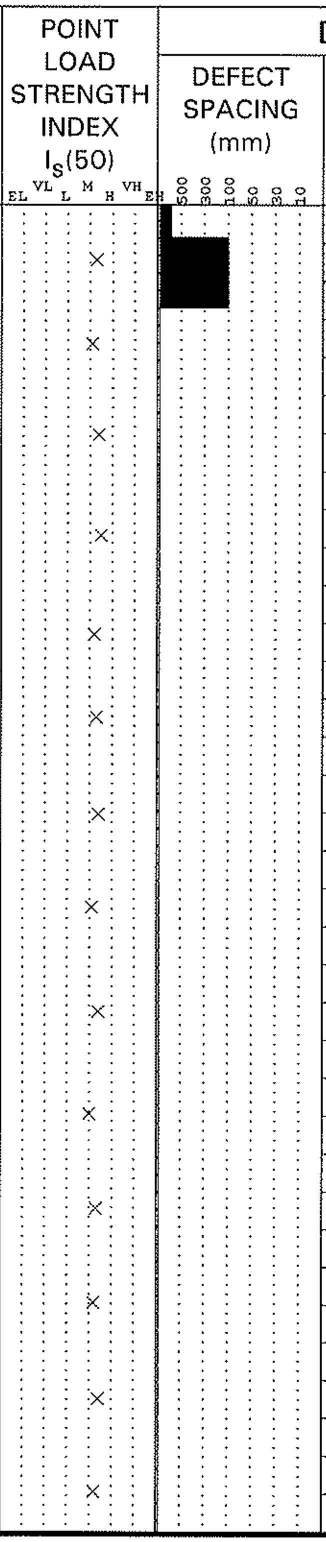
CORED BOREHOLE LOG

Borehole No.
101
4/5

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 14.1m
Date: 27-6-09
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.C./PW

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
				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 2-15°.	SW	H	- J, 60°, P, S	
							- CS, 5mm.t, P, S	
							- CS, 5mm.t, P, S	
		17						
				as above, but bedded at 10-15°.				
		18						
		19						
FULL RET-URN								
		20						
		21		as above, but bedded at 5-15°.				
							- CS, 10mm.t, P, S	
		22						

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21
22
23
END OF BOREHOLE AT 23.70m

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CORED BOREHOLE LOG

Borehole No.
101
5/5

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 14.1m
Date: 27-6-09
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.C./PW

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
FULL RET-URN				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 5-15°.	SW	H	×			
				END OF BOREHOLE AT 23.7m					50mm DIA. PVC STANDPIPE INSTALLED TO 15m, SLOTTED BETWEEN TO 9m AND 15m, COMPLETED WITH GATIC COVER	
		24								
		25								
		26								
		27								
		28								
		29								

BOREHOLE LOG

Borehole No.
102
1/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 27-6-09
Method: SPIRAL AUGER
JK350
R.L. Surface: ≈ 12.6m
Datum: AHD
Logged/Checked by: J.C./PW

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING	■			■		0	▨		FILL: Gravelly silty clay, low plasticity, dark brown, with fine to medium grained sandstone gravel and a trace of roots.	MC>PL			GRASS COVER WASHBORED MATERIAL DESCRIPTION FROM FLUSH CUTTING RETURN ONLY. DESCRIPTION IS THERFORE APPROXIMATE ONLY AND COULD BE IN ERROR.
						1	▨						
						2	▨						
						3	▨						
									REFER TO CORED BOREHOLE LOG				
						4							
						5							
						6							
						7							


JEFFERY & KATAUSKAS PTY LTD
JOB NO 23119SP BH102 START CORING AT 3.62m
3
4
CORE LOSS 0.40m
5
6
7
8
9
10
11

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CORED BOREHOLE LOG

Borehole No.
102
2/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 12.5m
Date: 27-6-09
Inclination: 55°
Datum: AHD
Drill Type: JK350
Bearing: 335°
Logged/Checked by: J.C./PW

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
		3		START CORING AT 3.62m						
60% RET-URN		4		SANDSTONE: fine to coarse grained, light grey, orange brown and red brown.	DW	L-M	× ×			
				CORE LOSS 0.40m						
50% RET-URN		5		INTERBEDDED SANDY CLAY AND SANDSTONE: high plasticity, fine grained, light grey and red brown, with fine to medium grained ironstone gravel.	RS/ XW	EL		▨		
		6		SANDSTONE: fine to coarse grained, red brown, orange brown and light grey, with orange brown and light grey laminae, bedded at 0-10°.	DW	L-M	× ×			
		7				H	× × × ×	■	- J, 15°, P, S - XWS, 110mm.t - J, 45°, P, S	
		9		SANDSTONE: fine to coarse grained, orange brown, red brown and light grey, with orange brown and light grey laminae, bedded at 0-10°.		M	× ×			

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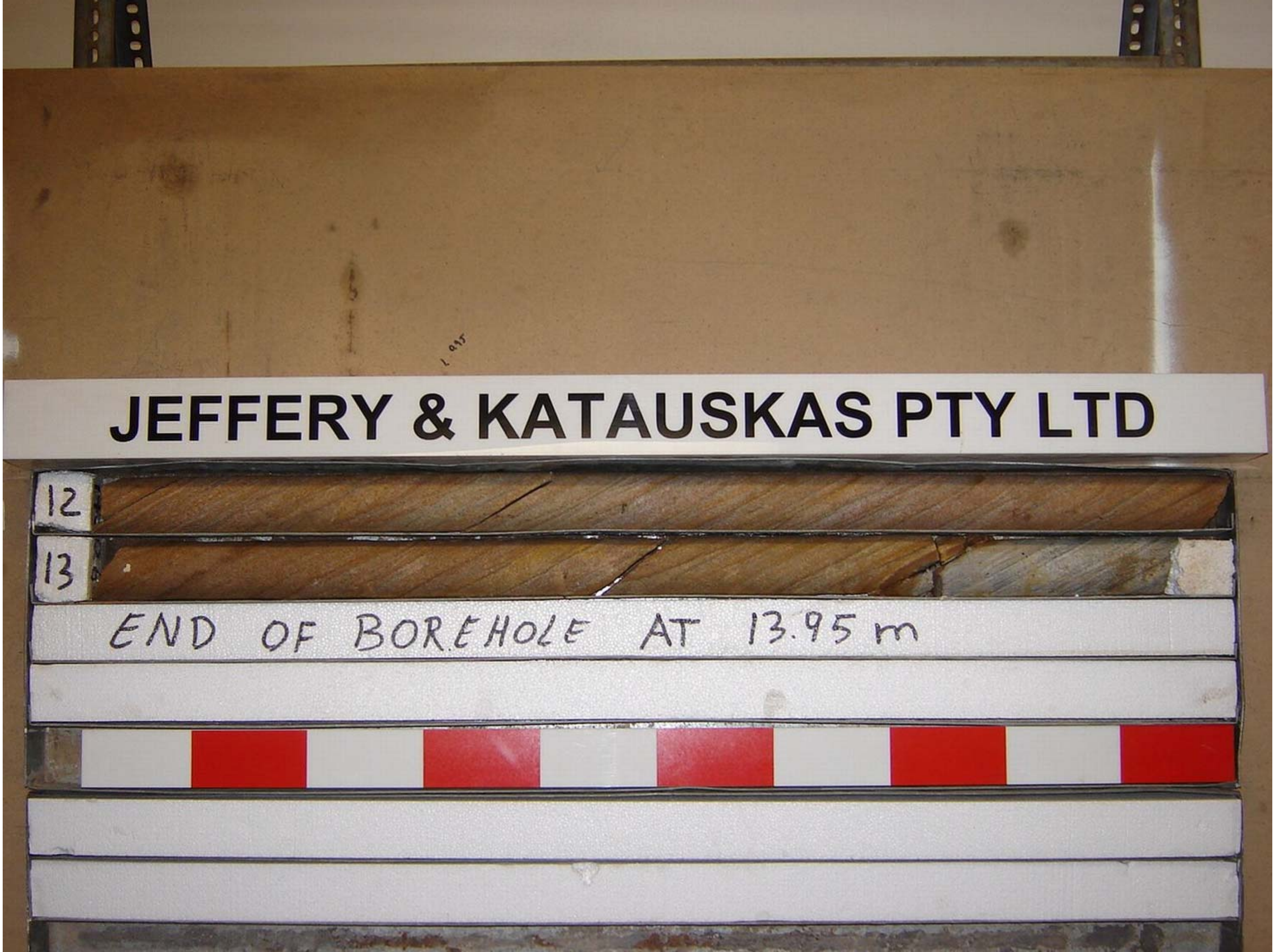
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12
13
END OF BOREHOLE AT 13.95m

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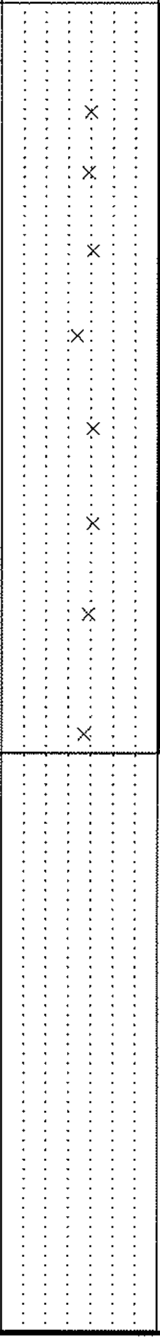
CORED BOREHOLE LOG

Borehole No. 102
3/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 12.5m
Date: 27-6-09
Inclination: 55°
Datum: AHD
Drill Type: JK350
Bearing: 335°
Logged/Checked by: J.C./PW

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
60% RETURN		11 12 13		SANDSTONE: fine to coarse grained, orange brown, red brown and light grey, with orange brown and light grey laminae, bedded at 0-10°.	DW	M M-H			- Be, 0°, P, S - J, 55°, P, S, CLAY INFILL	
				as above, but grey, with dark grey laminae, bedded at 60°.	SW					
		14 15 16		END OF BOREHOLE AT 13.95m						



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BOREHOLE LOG

Borehole No.
103
1/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 27-6-09
Method: SPIRAL AUGER JK300
Logged/Checked by: J.C./PW
R.L. Surface: ≈ 13.5m
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
DRY ON COMPLETION OF AUGERING	■			■		0			FILL: Gravelly silty clay, low plasticity, dark brown, with fine to medium grained sandstone gravel and a trace of roots.	MC>PL			GRASS COVER
	■			■	N = 12 11,5,7				FILL: Silty sand, fine to medium grained, red brown and dark grey, with ash and a trace of clay and concrete fragments.	M			APPEARS MODERATELY COMPACTED
				■		1		-	INTERBEDDED CLAY AND SANDSTONE: high plasticity, light grey and red brown.	MC<PL	H	-	RESIDUAL
	■			■	N > 11 8,11/ 30mm							490 460 430	
				■		2		-	SANDSTONE: fine to medium grained, red brown, light grey and orange brown.	DW	M	-	MODERATE 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						3							
						4							
						5							
						6							
						7							

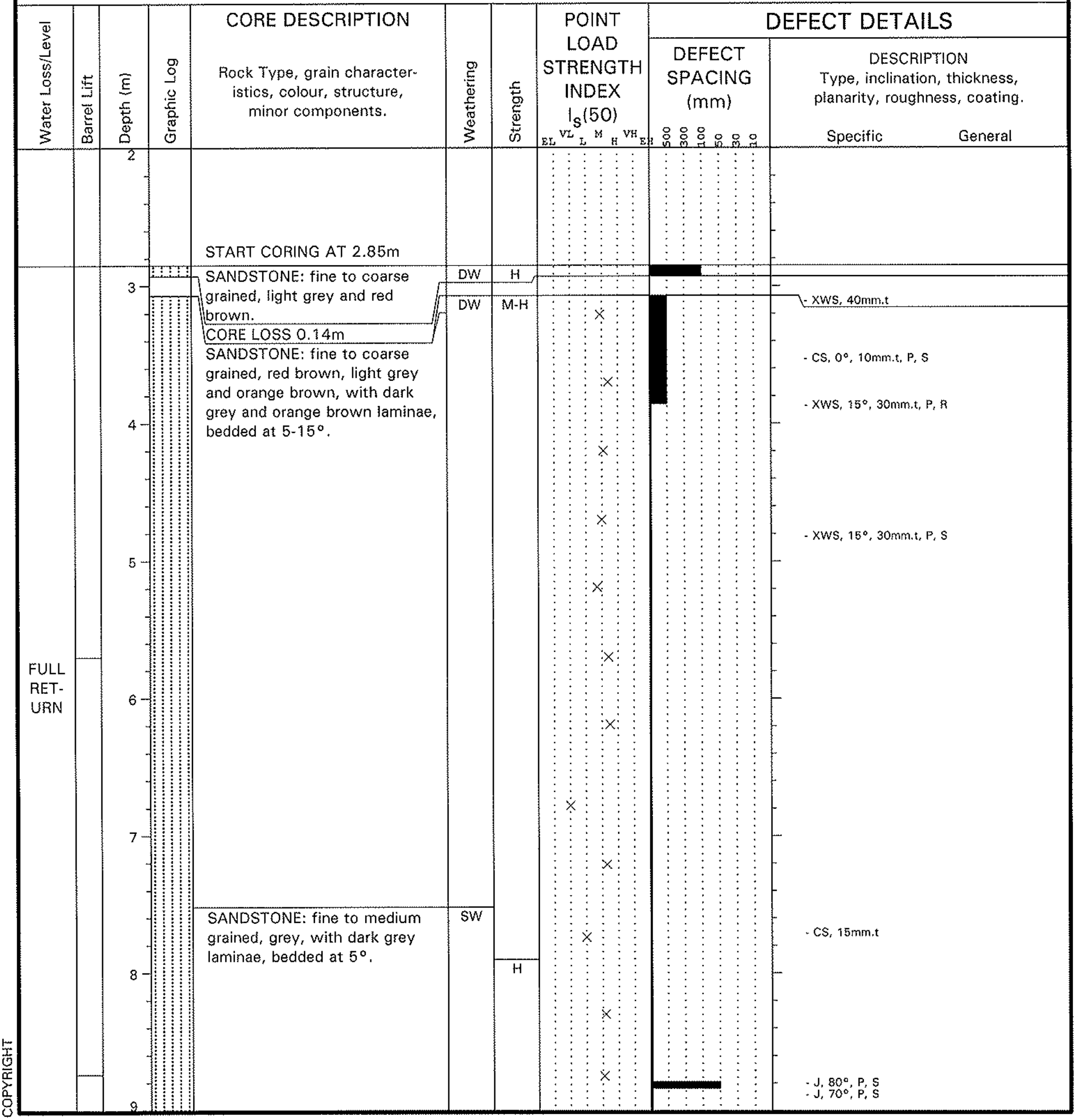

JEFFERY & KATAUSKAS PTY LTD
JOB NO 23119SP BH103 START CORING AT 2.85m
2
CORE
3
LOSS 0.14m
4
5

CORED BOREHOLE LOG

Borehole No.
103
2/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 28-6-09
Drill Type: JK350
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 13.5m
Datum: AHD
Logged/Checked by: J.C./PW



Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
	2.85	START CORING AT 2.85m				
		SANDSTONE: fine to coarse grained, light grey and red brown.	DW	H		
		CORE LOSS 0.14m				
		SANDSTONE: fine to coarse grained, red brown, light grey and orange brown, with dark grey and orange brown laminae, bedded at 5-15°.	DW	M-H	- XWS, 40mm.t	
					- CS, 0°, 10mm.t, P, S	
					- XWS, 15°, 30mm.t, P, R	
					- XWS, 15°, 30mm.t, P, S	
FULL RETURN						
		SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 5°.	SW		- CS, 15mm.t	
				H		
					- J, 80°, P, S - J, 70°, P, S	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JEFFERY & KATAUSKAS PTY LTD
11
12
13
14
END OF BOREHOLE AT 14.63m

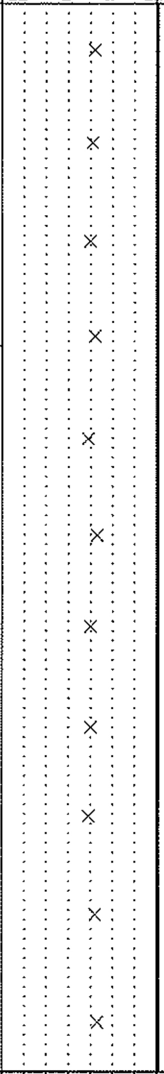
CORED BOREHOLE LOG

Borehole No.
103
3/3

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Core Size: NMLC
R.L. Surface: ≈ 13.5m
Date: 28-6-09
Inclination: VERTICAL
Datum: AHD
Drill Type: JK350
Bearing: -
Logged/Checked by: J.C./PW

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
				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 5-10°.	SW	H				
		10								
									- XWS, 10mm.t	
						M-H				
		11								
FULL RETURN										
		12							- J, 60°, P, S	
		13								
		14								
				END OF BOREHOLE AT 14.63m						
		15								



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BOREHOLE LOG

Borehole No.
104
1/1

Client: UNIVERSITY OF TECHNOLOGY SYDNEY
Project: PROPOSED MULTI-PURPOSE SPORTS HALL
Location: UTS BROADWAY, ULTIMO, NSW

Job No. 23119SP
Date: 29-6-09
Method: SPIRAL AUGER
JK300
R.L. Surface: ≈ 12.4m
Datum: AHD
Logged/Checked by: J.C./PW

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION	■			■		0	▨		FILL: Gravelly silty clay, low plasticity, dark grey, with fine to meidum grained sandstone gravel and a trace of sand and roots.	MC>PL			GRASS COVER APPEARS POORLY TO MODERATELY COMPACTED
									END OF BOREHOLE AT 0.5m				TWO LOCATIONS ATTEMPTED WITH REFUSAL ON OBSTRUCTION AT 0.5m DEPTH
						1							
						2							
						3							
						4							
						5							
						6							
						7							

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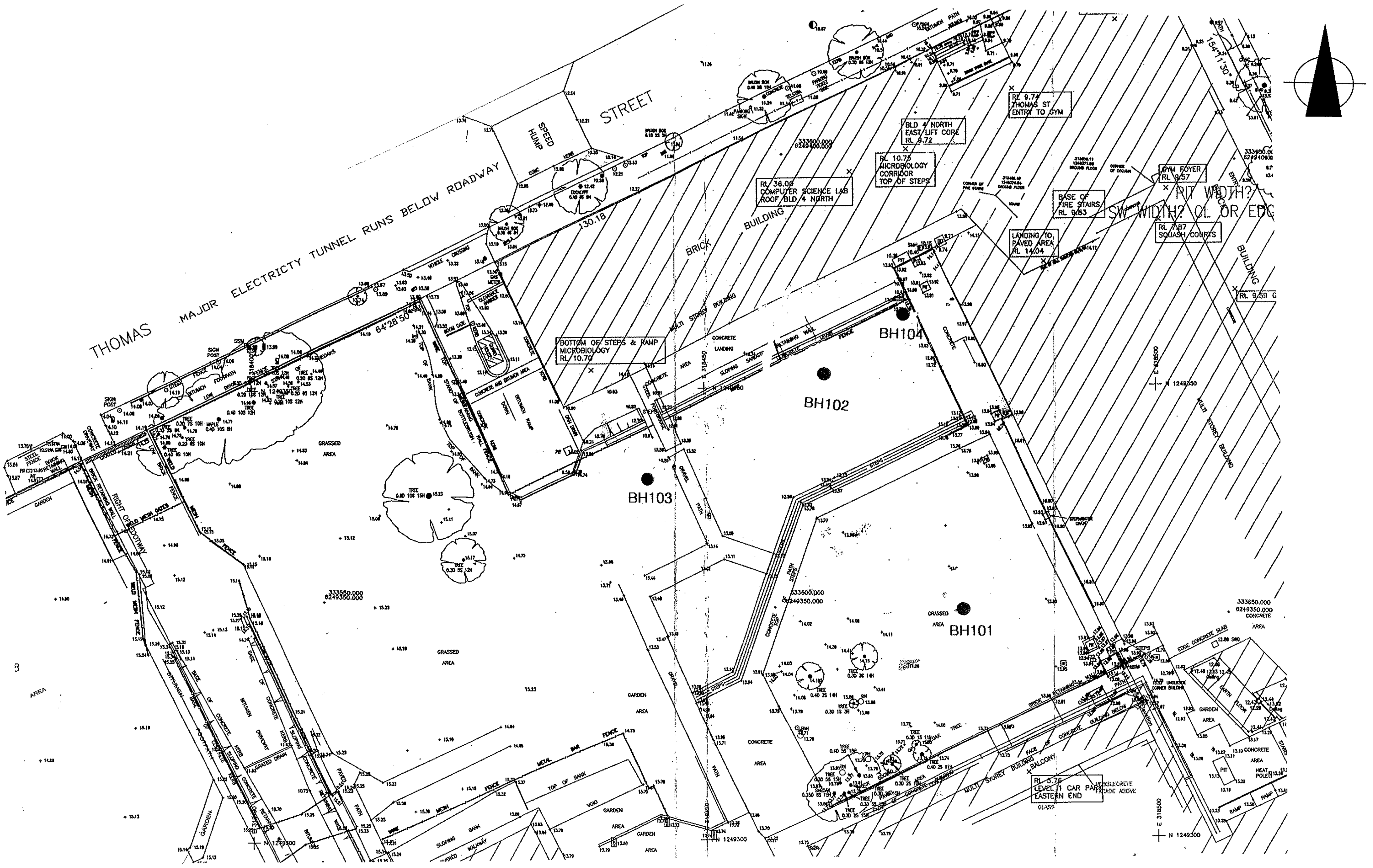

LEGEND

● BOREHOLE

SCALE (m)

0 20

INVESTIGATION LOCATION PLAN

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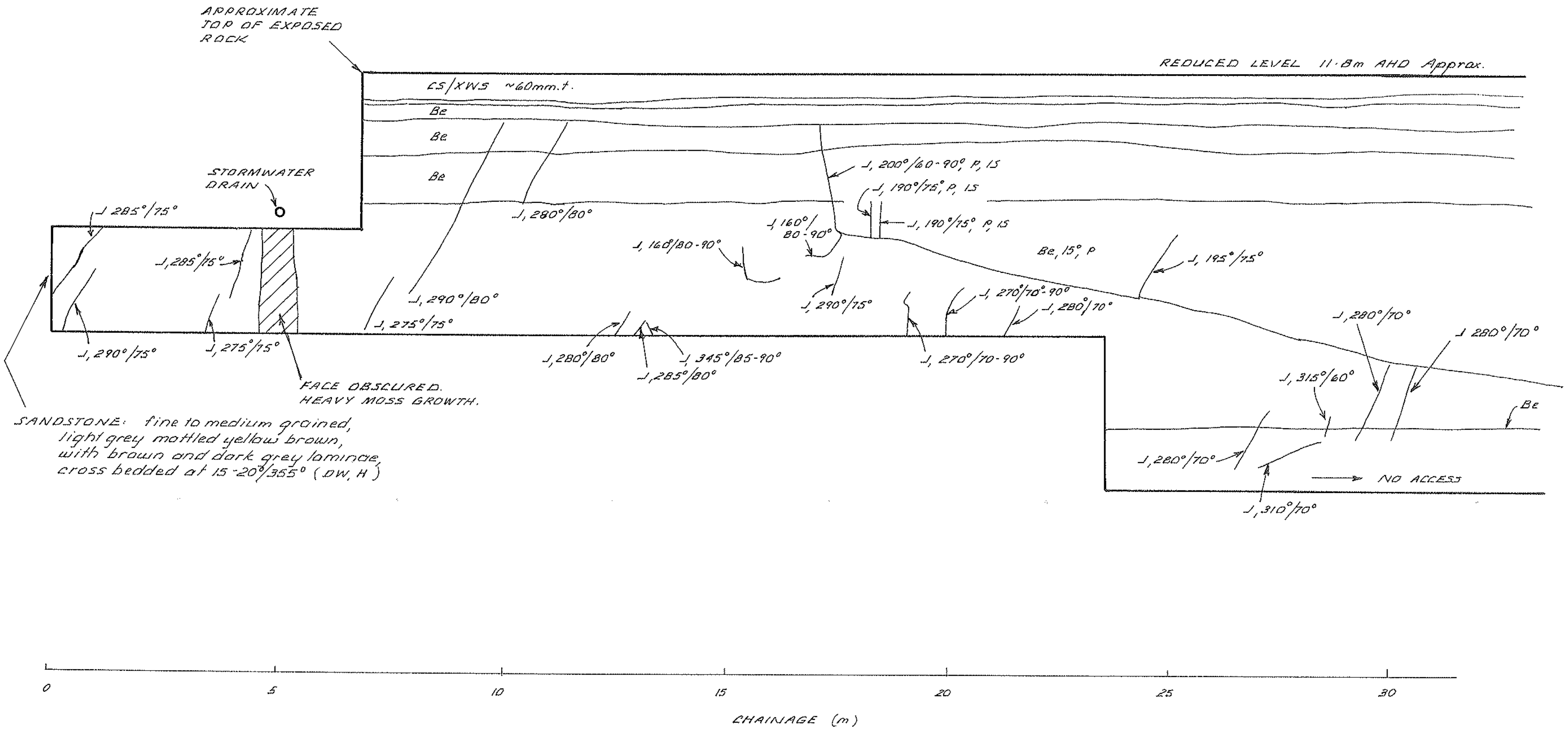
Report No.23119SP **Figure No. 1**



GEOTECHNICAL MAPPING – NORTH FACE OF BUILDING 1 BASEMENT

SCALE (M)
0 5

Jeffery and Katauskas Pty Ltd
Report No. 23119SP Figure No. 2

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ABN 17 003 550 801

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

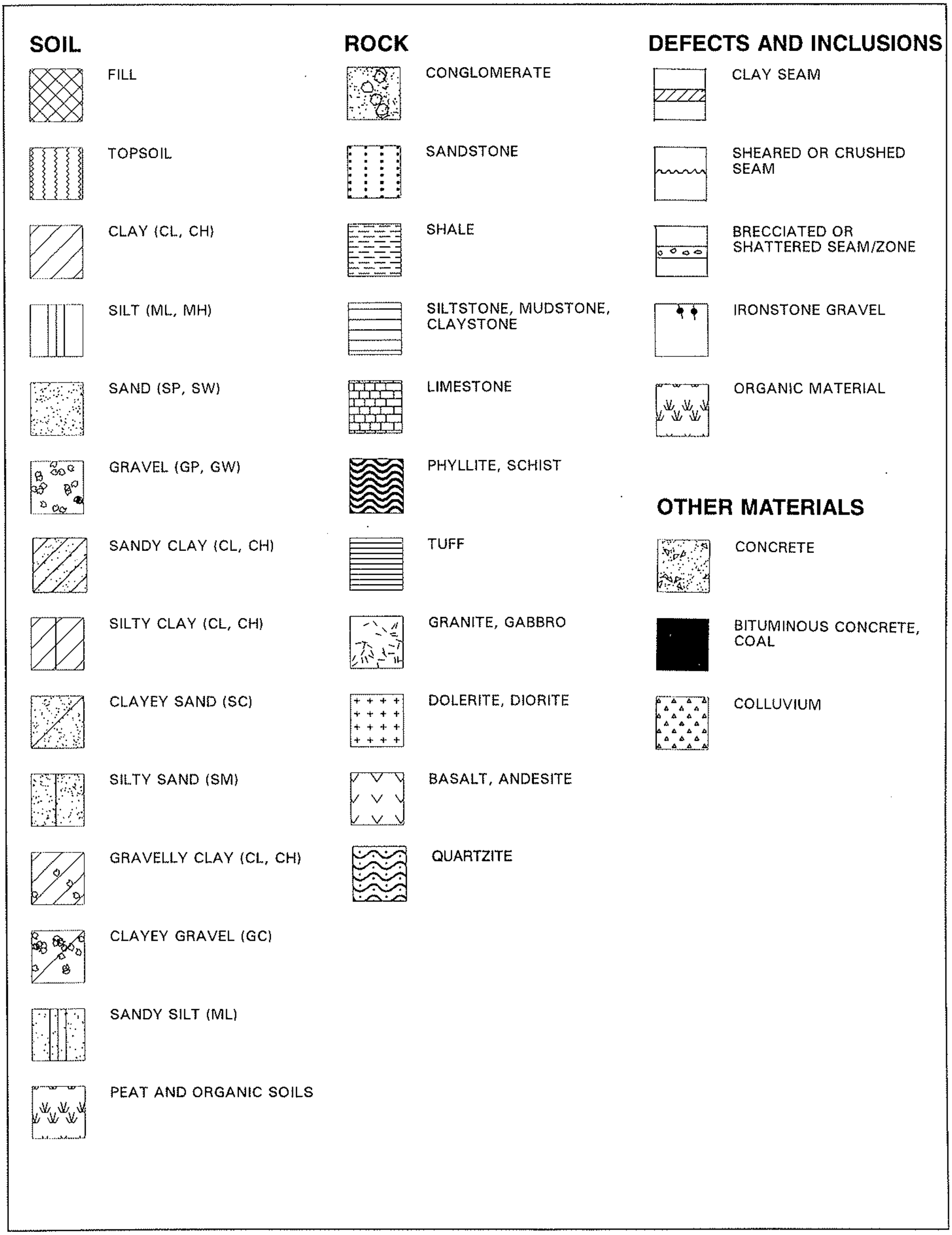
Requirements could range from:

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

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

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



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

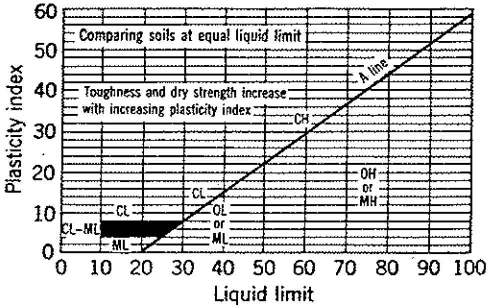
DEFECTS AND INCLUSIONS

- 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	
Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	Identification Procedures on Fraction Smaller than 380 µm Sieve Size											
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
	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	─⊤─	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