



ADDENDUM REPORT
TO
NSW HEALTH INFRASTRUCTURE
ON
GEOTECHNICAL INVESTIGATION
FOR
PROPOSED CAR PARK
AT
WOLLONGONG HOSPITAL, 3 to 15 DUDLEY STREET
WOLLONGONG, NSW

30 April 2013
Ref: 26129Srptrev1Addendum



AS/NZS ISO 9001
Certified

Davis Langdon Certification Services

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5001
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as
JK Geotechnics ABN 17 003 550 801

Date: 30 April 2013
Report No: 26129SrptAddendum
Revision No: 1

Report prepared by:



Ian Squibbs
Engineering Geologist

Report reviewed by:



Paul Stubbs
Principal Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

© Document Copyright of JK Geotechnics.

This Report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JK) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JK and its Client and is therefore subject to:

- a) JK's proposal in respect of the work covered by the Report;
- b) the limitations defined in the Client's brief to JK;
- c) the terms of contract between JK and the Client, including terms limiting the liability of JK.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JK which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JK does so entirely at their own risk and to the fullest extent permitted by law, JK accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.



TABLE OF CONTENTS

1	INTRODUCTION	1
2	INVESTIGATION PROCEDURE	1
3	RESULTS OF INVESTIGATION	2
3.1	Site Description	2
3.2	Subsurface Conditions	3
3.3	Laboratory Test Results	4
4	COMMENTS AND RECOMMENDATIONS	5
4.1	Footing Design	5
4.2	Retention	5
4.3	Summary of Additional Geotechnical Work Recommended	8
5	GENERAL COMMENTS	9

STS TABLE A: POINT LOAD STRENGTH INDEX TEST REPORT
BOREHOLE LOGS BH101 TO BH103 AND CORE PHOTOGRAPHS INCLUSIVE
FIGURE 1: BOREHOLE LOCATION PLAN
FIGURE 2: GRAPHICAL BOREHOLE SUMMARY
FIGURE 3: PLAN SHOWING TOP OF ROCK CONTOURS
FIGURE 3A PLAN SHOWING TOP OF ROCK SUITABLE FOR 3500kPa
REPORT EXPLANATION NOTES



1 INTRODUCTION

This report presents the results of additional geotechnical investigations for a proposed 5 level car park on land between New Dapto Road, Urunga Parade and Dudley Street. The work was commissioned by Mei Soh of TSA Management Pty Ltd on behalf of the NSW Health Infrastructure, and was carried out in accordance with our proposal dated 7 February 2013 (Ref 26129Spropadd). This report forms an addendum to our previous report (Ref: 26129Srpt dated 3 December 2012).

The purpose of the investigation was to obtain additional geotechnical information on subsurface conditions due to a change in design of the proposed car park, so as to provide a better coverage and to supplement the existing boreholes as a basis for comments and recommendations on excavation, retention, footing design and groundwater conditions. We note that the previous plan for a split level basement has been superseded and the whole basement now has a proposed floor level of RL30.275 AHD, which is the lower of the two levels proposed previously.

2 INVESTIGATION PROCEDURE

The investigation was carried out between 2 and 4 April 2013, and comprised the drilling of three boreholes (BH101 to BH103) using our truck mounted JK350 drilling rig. The boreholes were initially auger drilled to depths ranging from 2.96m to 7.24m and were then continued by diamond coring techniques using a NMLC core barrel with water flush to final depths of between 10.09m and 16.34m.

The borehole locations, as shown on Figure 1, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated interpolation between spot levels shown on the supplied survey plan by Allen, Price & Associates (Ref: 25461-31, dated 15/8/11). The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength and relative density of the subsurface soils were assessed with reference to Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples returned by the SPT split tube sampler. The strength of the upper weathered bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with the examination of the recovered rock cuttings and hence are very approximate. The strength of the cored bedrock



was assessed by examination of the recovered rock cores, together with the correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests. The results of the point load strength index test are summarised on the cored borehole logs.

Groundwater observations were made during and on completion of drilling. However, the use of water for core drilling would have affected the groundwater readings taken on completion of drilling. No longer term monitoring of groundwater levels was carried out.

Our engineering geologist, Mr Ian Squibbs, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, and photographs of the rock core, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on the upper reaches of a north facing hillside and is situated to the west of the main campus of Wollongong Hospital. The site is bounded to the south by Urunga Parade, to the east by New Dapto Road and to the west by Dudley Street

At the time of the current investigation, the site development was the same as at the time of the previous investigation, but the eastern site boundary now extends to the New Dapto Road boundary. New Dapto Road had a slope of approximately 2° to 3° to the northeast.

BH101 and BH102 were carried out in the properties of No 106 and No 108 New Dapto road respectively. No 106 New Dapto Road consisted of a one and two storey brick property with a driveway sloping down 3° to 4° to the west from New Dapto Road, to a levelled area. No 108 New Dapto Road consisted of a one and two storey brick building with a driveway entering off New Dapto Road and sloping down 5° to 7° to the west, and extending to the rear of the property to a levelled concrete car park.

Both properties had levelled front lawns with brick and concrete retaining walls up to 1.0m in height. The retaining wall running along the driveway of No 108 had a stepped crack extending 1m to the end of the wall. Both properties appeared in a reasonable condition when given a cursory inspection.



BH103 was drilled close to the western boundary, in front of the two storey brick engineering building that was cut back into the site from Dudley Street, with the maximum cut being approximately 6m towards the southern corner of the building. The cut was supported by a reinforced concrete retaining wall in seemingly good condition.

3.2 Subsurface Conditions

Reference to the 1:100,000 Wollongong-Port Hacking Geological Sheet indicates the site to be underlain by Budgong Sandstone which is a volcanic sandstone unit of the Shoalhaven Group.

In summary, the boreholes revealed conditions consistent with those found in the previous investigation and consisted of a varying amount of fill over silty clay over sandstone bedrock. Further comments on the subsurface conditions encountered in the recent boreholes are provided below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions.

Asphalt Pavement and Fill

Fill was encountered in BH101 and BH102 down to a maximum depth of 1.1m in BH102, and generally consisted of silty clay with brick, ash and coal fragments. In BH103 a 50mm thick asphalt pavement overlay a cemented gravelly sand subbase.

Residual Soils

The thickness of residual soil varies from up to 2.5m in BH101 along the eastern boundary and extending to 4m in BH103 along the western boundary. Silty clay was encountered beneath the fill, and was assessed to be of high plasticity in BH101 and BH102. In BH103 the silty clay was assessed to be medium plasticity and this was underlain by sandy clay of low to medium plasticity.

Sandstone

Sandstone was encountered in the boreholes from depths of 2.1m in BH102 down to 4.0m in BH103. Generally the sandstone was initially assessed to be of very low and low strength and distinctly weathered improving to medium and high strength with depth. In BH103 the distinctly weathered sandstone of very low and low strength extended to depths of up to 7m before improving to medium and high strength.



The boreholes contained numerous defects in the top 3m of core that comprised extremely weathered seams up to 10mm thick, bedding partings and joints inclined up to 90°.

An updated preliminary engineering classification of the sandstone bedrock (in accordance with Pells et al. 1998) has been carried out to include the additional boreholes and is tabulated below. We note that the engineering classification has not taken into account specific footing sizes, pile types, pile diameters and founding levels, and is therefore only indicative.

BH	Approximate Surface RL (mAHD)	Depth(m)/Top of RL(mAHD)				
		Class V	Class IV	Class III	Class II	Class I
101	41.5	-	2.5/39.0	3.0/38.5	-	6.2/35.3-
102	44.5	2.1/42.4	2.8/41.7	-	-	6.6/37.9
103	31.5	4.0/27.5	5.8/25.7	-	8.9/22.6	-
1	44.0	-	0.5/43.5	6.5/37.5	6.8/37.2	-
2	42.5	-	3.2/39.3	7.0/35.5	7.2/35.3	-
3	42.0	-	1.0/41.0	-	-	-
4	39.8	1.4/38.4	-	-	11.7/28.1	13.0/26.8-
5	33.0	4.8/28.2-	6.8/26.2	9.0/24.0	10.0/23.0	-
6	37.0	-	1.2/35.8	-	3.0/34.0	-
7	35.4	-	2.4/33.0	-	-	3.8/31.6

Groundwater

All of the boreholes were dry on completion of augering with the exception of BH103 which encountered groundwater at a depth of approximately 4.5m rising to 3m on completion of the augering. It should be noted that the hole was drilled during extreme rainfall so water could have infiltrated from the surface during this period.

Groundwater was measured within the boreholes on completion of coring at depths ranging from 4.4m down to 6.9m. As water is used in the coring process, these groundwater levels have most likely been affected by the introduced drill flush water. No longer term groundwater monitoring was undertaken as part of this investigation.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on recovered rock cores correlated well with our field assessment of bedrock strength. The Unconfined Compressive Strength (UCS) of the sandstone, estimated from the point load strength index test results, generally ranged from 14MPa to 60MPa, with some lower results of 8MPa and some higher results up to 100MPa.



4 COMMENTS AND RECOMMENDATIONS

Comments and recommendations regarding excavation conditions, retaining walls, shoring design, groundwater and car park floor slab design are covered in report 26129Srpt dated December 2012. The following information is supplementary to the previous advice.

4.1 Footing Design

The main footing design is covered in report 26129Srpt, this section only covers the areas covered by the additional boreholes. Following bulk excavations to an RL30.275m, sandstone will be exposed over the majority of the proposed building area. Therefore, the proposed building should be supported on footings founded entirely within the sandstone to provide uniform support and reduce the risk of differential settlements. Pad or strip footings would be appropriate for the majority of the site, but piles will be required in the area of BH5 and BH103 to extend through the residual clay into the underlying sandstone bedrock.

At bulk excavation level footings founded on the Class III sandstone of at least medium strength may be designed for an allowable bearing pressure (ABP) of 3,500kPa, in which case all the footing excavations should be inspected by a geotechnical engineer to confirm that adequate sandstone has been encountered (provided that additional boreholes are completed prior to construction to confirm rock quality at depth). If allowable bearing pressures are limited to 1,500kPa on Class IV sandstone then only the inspection of selected footings spread throughout the basement would be required unless the bulk excavation reveals unusual geological conditions.

In the area of BH103 and BH5, bored piles extending to at least 4.0m and 4.8m below existing ground level (begl) can be designed for an ABP of 1,000kPa, which can be increased to an ABP of 3,500kPa if taken down to 9.0m begl. Piles in this area are likely to extend below the water table, so the use of pumps and/or tremie techniques may be required. Note that bands of very high strength sandstone would make socketed piles very difficult to construct.

4.2 Retention

The main retention design is covered in report 26129Srpt, this section only covers amendments made to shoring walls and vertical excavations. It is understood that where possible a vertical excavation in the rock will be maintained. This option only appears feasible for shoring along the New Dapto Road boundary and any shoring in the eastern half of the northern boundary, in the areas of BH101, BH102 and BH6. The shoring piles should extend down into the underlying sandstone of at least medium strength (Class 3). Below is a table indicating what depth the toe of



the shoring piles should extend based on boreholes drilled nearby. The piles should terminate close to a floor level and should contain temporary rock bolts close to the toe of the pile where piles do not a rock socket below bulk excavation level. In the long term structural propping of the pile toes will have to replace the temporary anchors. In addition, rock below the pile toes would need to be inspected by a geotechnical engineer once exposed to check if the piles are founded within appropriate material and assess if underpinning of the piles is required. Drilling of rock sockets in the medium to high strength or stronger rock will be relatively difficult requiring the use of heavy drilling rigs equipped with rock augers and possibly coring buckets. Interpolation of areas where piles can terminate above bulk excavation level can also take into account the plan showing the surface contours for Class 3 rock presented as Figure 3A but this plan should be used with caution as it assume a gradational change from one data point to another and this may not occur in practice as geological boundaries may be quite different.

Borehole No:	Depth to Class III Sandstone (m/RL)	Closest Floor Level	Recommended Minimum Shoring Pile Depth RL (mAHD)
BH101	3.0/38.5	Level 3b	36.35^
BH102	6.6/37.9	Level 3b	36.35^
BH3	-	-	29.275*
BH4	11.7/28.1	Level 1b	29.275*
BH5	10.0/23.0	Level 1b	29.275*
BH6	3.0/34.0	Level 2b	33.65^
BH7	3.8/31.6	Level 1b	29.275*

*Extended nominally 1m below Level 1b basement level. Socket length depends on lateral resistance required.

^ Piles must be extended to a suitable depth below the nominal floor surface level.

Care must be taken during rock excavation not to over-excavate the rock foundation supporting the toe of the shoring system. Where piles are founded above bulk excavation level it is highly desirable for there to be an offset of at least 0.2m to 0.3m to the cut face to the piles to give some “construction tolerance” against overbreak.

Any shoring piles that terminate above vertical rock cuts should not be designed to support loads from the main structure, any such structural loads should be transferred internally onto separate structural columns. It should be noted that if rock quality below the toe of shoring piles founded above bulk excavation level is inadequate, whether due to the rock being of lower strength than expected or due to defects in the rock mass, permanent rock bolts may not be feasible, particularly if they extend beyond property boundaries, in which case it will probably be necessary



to underpin the shoring piles once the temporary anchors are destressed and the shoring is propped by the structure. There is a major variation in rock quality between BH3 and BH102 in the south-eastern corner of the site and thus the point at which the shoring piles could potentially change from founding below bulk level to founding just below Level 3b is indeterminate.

The Geology of the Wollongong 1:100,00 Sheet by L. Sherwen and G.G Holmes (1982) states that Budgong Sandstone Unit shows moderately rapid rates of deterioration after exposure in man-made cuts. This is not surprising as the sandstone is a lithic/lithic felspathic rock rather than the quartz rich more durable sandstones which are more familiar from the Sydney area. It is understood that a shrinkage reinforced shotcrete facing will be applied to the excavated rock face, this will help prevent weathering of the sandstone and give a better flush finish and minimise water penetration into the car park from the rock face wall. The shotcrete will need to be pinned to the piles and the rock faces to maintain contact and strip drains or weepholes (on a grid at say 1.5m centres) will be required to relieve water seepage pressures.

The suitability of the sandstone to stand unsupported must be confirmed by regular geotechnical inspections of the cut faces to check for inclined joints or weak seams that may require treating. The geotechnical inspections should be carried out at depth intervals of no more than 1.5m to allow for any such additional support to be installed progressively during excavation.

Anchors and rock bolts should have their bond within sandstone of at least low strength and may be designed based on a bond stress of 250kPa for sandstone of low strength or 400kPa for sandstone of medium or higher strength. Anchors should have a minimum free length of 3m and a bond length of at least 3m formed behind a 45° line drawn upwards from the base of retention system. of the retention system. Anchors should be proof loaded to at least 1.3 times their working loads before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to check that the anchors are holding their load. Detailed rock bolt designs and specifications should be prepared when it is clear whether bolts will be permanent or temporary and the working life of the bolts is decided for permanent bolts.

Permission will need to be obtained from the RMS and Council with regards to anchoring below the New Dapto Road and Dudley Street. Such permission can take some time to obtain and we recommend that permission be sought early in the process to avoid project delays.



The subsurface profile across the site varies in that there is an abrupt transition from soil to good quality rock in some areas whilst there is a deep weathered profile of very poor quality rock at other locations. For simplicity of design and construction it would be appropriate to assume the 6HkPa earth pressure recommended in our main report for shoring design apply to all material above the Class 3 rock. This may be a little conservative where there is some better quality Class 4 rock above the Class 3 material, but it is very hard to define that interface from the limited borehole information and it would be necessary to have frequent geotechnical inspections at the site during excavation to determine whether there was a significant zone of Class 4 material for which a revised shoring design based on an earth pressure of (say) 4H kPa would be applicable. We consider that having to make such judgement calls during construction is not really practical as the upper anchors would already be installed when the materials are revealed and it would be too late to make any saving on them anyway. The good quality sandstone of at least medium strength (Class 3) can be taken to be self-supporting and no 'K' values need to be taken into consideration (see above table for depths). However, the retention system may be designed by assuming a nominal uniform lateral pressure of 10kPa to account for small-scale wedges of rock which could be isolated by inclined joints and horizontal bedding planes within the shale and low strength or weaker sandstone. The quality of the sandstone profile should be confirmed during geotechnical inspections during construction.

Applicable hydrostatic pressures should be added to the lateral earth pressures, unless specific measures are taken to introduce complete and permanent drainage of the ground behind the walls. Any surcharge affecting the walls (e.g. footings, retaining walls and their backfill, the ground slope behind the wall, etc.) should also be considered in the design.

4.3 Summary of Additional Geotechnical Work Recommended

- Geotechnical inspection of excavation to check for adversely inclined defects and geological anomalies and possibly to adjudge material quality to adjust shoring design.
- Geotechnical inspection of founding material for shoring pile wall.
- Monitoring ground anchor installation.
- Assessment of rock faces to determine whether works are required to stabilise in short or long term.
- Geotechnical inspections/testing of founding material for footings (additional cored boreholes are required if allowable bearing pressures over 1500kPa are proposed).
- Assessment of drainage requirements after bulk excavation.



5 GENERAL COMMENTS

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

The long term successful performance of floor slabs and pavements 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 judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be



undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

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

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	Ref No:	26129S1
Project:	Proposed New Car Park	Report:	A
Location:	Wollongong Hospital, 3-15 Dudley Street, Wollongong, NSW	Report Date:	8/04/2013
		Page 1 of 3	

BOREHOLE NUMBER	DEPTH	$I_s (50)$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
	m	MPa	(MPa)
101	3.24-3.29	2.5	50
	3.85-3.88	1.6	32
	4.16-4.18	0.9	18
	4.77-4.81	3.9	78
	5.17-5.20	1.0	20
	5.81-5.84	5.0	100
	6.16-6.19	1.2	24
	6.70-6.74	2.3	46
	7.14-7.18	2.0	40
	7.79-7.83	2.2	44
	8.28-8.31	2.7	54
	8.84-8.88	2.5	50
	9.15-9.18	2.4	48
	9.82-9.86	2.1	42
	10.20-10.23	2.2	44
	10.84-10.87	3.1	62
	11.12-11.16	3.8	76
	11.80-11.84	2.3	46
	12.16-12.19	2.0	40
	12.75-12.79	1.8	36
	13.10-13.14	1.5	30
	13.76-13.79	2.6	52
	14.31-14.35	2.1	42
	14.83-14.86	2.2	44
	15.16-15.20	1.4	28

NOTES: See Page 3 of 3



SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	Ref No:	26129S1
Project:	Proposed New Car Park	Report:	A
Location:	Wollongong Hospital, 3-15 Dudley Street, Wollongong, NSW	Report Date:	8/04/2013
		Page 2 of 3	

BOREHOLE NUMBER	DEPTH	$I_s (50)$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
	m	MPa	(MPa)
102	4.32-4.35	1.4	28
	4.83-4.87	4.4	88
	5.14-5.17	0.4	8
	5.84-5.87	4.0	80
	6.85-6.89	1.9	38
	7.20-7.24	2.5	50
	7.77-7.81	2.4	48
	8.17-8.21	1.5	30
	8.80-8.83	2.1	42
	9.12-9.16	2.1	42
	9.75-9.79	1.9	38
	10.19-10.22	1.2	24
	10.73-10.76	2.4	48
	11.13-11.17	2.6	52
	11.73-11.75	1.5	30
	12.15-12.19	1.2	24
	12.86-12.90	1.1	22
	13.20-13.25	1.9	38
	13.80-13.85	2.6	52
	14.25-14.29	2.0	40
	14.89-14.93	2.2	44
	15.17-15.21	3.3	66
	15.89-15.92	2.3	46
	16.17-16.21	2.9	58

NOTES: See Page 3 of 3



SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	Ref No:	26129S1
Project:	Proposed New Car Park	Report:	A
Location:	Wollongong Hospital, 3-15 Dudley Street, Wollongong, NSW	Report Date:	8/04/2013
		Page 3 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{s(50)}$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
		MPa	(MPa)
103	7.41-7.45	1.2	24
	7.88-7.91	0.8	16
	8.28-8.31	0.7	14
	8.66-8.68	1.5	30
	9.18-9.22	1.8	36
	9.85-9.89	2.7	54
	10.05-10.09	1.7	34

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
1/3

Client: NSW HEALTH INFRASTRUCTURE
Project: PROPOSED NEW CAR PARK
Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW
Job No. 26129S
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 41.5m
Date: 2-4-13
Datum: AHD
Logged/Checked by: I.S./P.S.

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									
DRY ON COMPLETION OF AUGERING					0			FILL: Gravelly sand, fine to medium grained, dark grey, fine to medium grained coal and ash fragments.	M			GRASS COVER APPEARS POORLY COMPACTED
				N = 4 3,2,2			FILL: Silty clay, low plasticity, dark brown, with sand, trace of brick and ash.	MC>PL				
					1		CH	SILTY CLAY: high plasticity, orange brown mottled red brown and light grey.	MC≈PL	VSt		RESIDUAL
				N = 19 10,6,13	270 280 280							
					2							
					3		-	SANDSTONE: fine to medium grained, grey brown and orange brown.	SW	L		LOW 'TC' BIT RESISTANCE
										M-H		MODERATE RESISTANCE
								REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
					4							
					5							
					6							
					7							

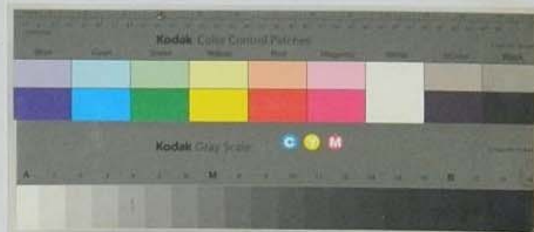
JK Geotechnics



CLIENT: NSW Health Infrastructure

PROJECT: Proposed Car Park

LOCATION: Wollongong Hospital, Wollongong, NSW



SCALE (CM)

JOB NO: 261295

BH101

START CORING AT 2.95m



END OF BH AT 15.97m





Borehole No.
101

2/3

CORED BOREHOLE LOG

Client: NSW HEALTH INFRASTRUCTURE
Project: PROPOSED NEW CAR PARK
Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW

Job No. 26129S Core Size: NMLC R.L. Surface: ≈ 41.5m
Date: 2-4-13 Inclination: VERTICAL Datum: AHD
Drill Type: JK350 Bearing: - Logged/Checked by: I.S./P.S.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION	Weathering	Strength	POINT LOAD STRENGTH INDEX	DEFECT DETAILS	
								DEFECT SPACING (mm)	DESCRIPTION
		2		START CORING AT 2.96m					
		3		SANDSTONE: fine grained, grey, with dark grey inclusions and occasional orange brown bands. Some very high strength bands.	SW	M-H			
		4							
		5							
		6		SANDSTONE: fine to medium grained, light grey.	FR	H			
		7							
		8							
		9							



BOREHOLE LOG

Borehole No.
102
1/3

Client: NSW HEALTH INFRASTRUCTURE														
Project: PROPOSED NEW CAR PARK														
Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW														
Job No. 26129S Method: SPIRAL AUGER JK350 R.L. Surface: ≈ 44.5m														
Date: 2-4-13 Datum: AHD														
Logged/Checked by: I.S./P.S.														
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					N = 4 1,2,2	0			FILL: Silty clay, low plasticity, dark brown, with medium to coarse grained brick and ash gravel.	MC>PL			GRASS COVER APPEARS POORLY COMPACTED	
						1		CH	SILTY CLAY: high plasticity, red brown mottled orange brown and light grey.	MC~PL	Vst			
					N = 11 3,6,5								340 310 350	RESIDUAL
						2		-	SANDSTONE: fine grained, orange brown.	DW	VL			
						3					L-M		VERY LOW 'TC' BIT RESISTANCE	
					4			SANDSTONE: fine grained, light grey.	SW	M		MODERATE RESISTANCE		
						5			REFER TO CORED BOREHOLE LOG					
						6								
						7								

JK Geotechnics



CLIENT: NSW Health Infrastructure

PROJECT: Proposed Car Park

LOCATION: Wollongong Hospital, Wollongong, NSW



SCALE (CM)

JOB NO: 261295 BH102 START CORING AT 4.22m

4

5

6

CORE LOSS 0.39m

7

8

9

10

11

12

13

14

15

16

END OF BH AT 16.34m

CORED BOREHOLE LOG

Client: NSW HEALTH INFRASTRUCTURE

Project: PROPOSED NEW CAR PARK

Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW

Job No. 26129S

Core Size: NMLC

R.L. Surface: $\approx 44.5\text{m}$

Date: 2-4-13

Inclination: VERTICAL

Datum: AHD

Drill Type: JK350

Bearing: -

Logged/Checked by: I.S./P.S.

[illegible]



Borehole No.
102

3/3

CORED BOREHOLE LOG

Client: NSW HEALTH INFRASTRUCTURE																		
Project: PROPOSED NEW CAR PARK																		
Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW																		
Job No. 26129S					Core Size: NMLC					R.L. Surface: ≈ 44.5m								
Date: 2-4-13					Inclination: VERTICAL					Datum: AHD								
Drill Type: JK350					Bearing: -					Logged/Checked by: I.S./P.S.								
Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION	Weathering	Strength	POINT LOAD STRENGTH INDEX						DEFECT DETAILS					
							I _s (50)						DEFECT SPACING (mm)	DESCRIPTION				
							EL	VL	L	M	H	VH		EH	Type, inclination, thickness, planarity, roughness, coating.		Specific	General
FULL RET-URN		12		SANDSTONE: fine to medium grained, light grey.	FR	H												- Be, 0°, R
				SANDSTONE: medium grained, light grey, with occasional black inclusions.														
		13																
		14		SANDSTONE: fine to medium grained, light grey, with black inclusions. Some very high strength bands.														
		15																
		16																
		17		END OF BOREHOLE AT 16.34m														



BOREHOLE LOG

Borehole No.
103
1/3

Client: NSW HEALTH INFRASTRUCTURE
Project: PROPOSED NEW CAR PARK
Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW
Job No. 26129S
Date: 4-4-13
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 31.5m
Datum: AHD
Logged/Checked by: I.S./P.S.

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									
ON COMPLETION OF AUGERING ON COMPLETION OF CORING						0		-	ASPHALTIC CONCRETE: 50mm.t				
					N = 12 5,6,6			CL	FILL: Gravelly sand, medium to coarse grained igneous gravel.	MC≈PL	VSt	250 250 200	RESIDUAL
						1			SILTY CLAY: medium plasticity, light grey and orange brown, with sand and medium to coarse grained sandstone gravel.				
					N = 24 13,11,13				SANDY CLAY: low to medium plasticity, orange brown and light grey, with medium to coarse grained sandstone gravel.			300 210 270	
						2			SANDY CLAY: medium plasticity, light grey.	MC>PL	St		
					N = SPT 17/100mm REFUSAL				as above, but with iron indurated sandstone bands.				VERY LOW 'TC' BIT RESISTANCE BANDS
						3							
						4		-	SANDSTONE: fine to medium grained, brown.	DW	VL		VERY LOW RESISTANCE
						5							
					N = SPT 5/50mm REFUSAL						VL-L		LOW REISTANCE
					6								
					7								

COPYRIGHT

JK Geotechnics



CLIENT: NSW Health Infrastructure

PROJECT: Proposed Car Park

LOCATION: Wollongong Hospital, Wollongong, NSW



SCALE (CM)

JOB NO: 261295 BH103 START CORING AT 7.24m

7

8

9

10

END OF TBH AT 10.09m

Location: WOLLONGONG HOSPITAL, 3-15 DUDLEY STREET, WOLLONGONG, NSW

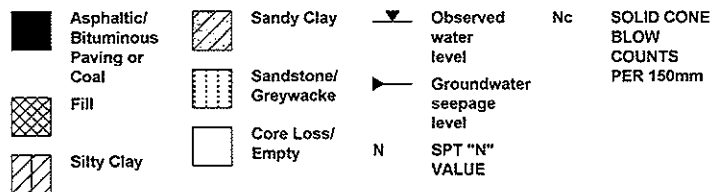
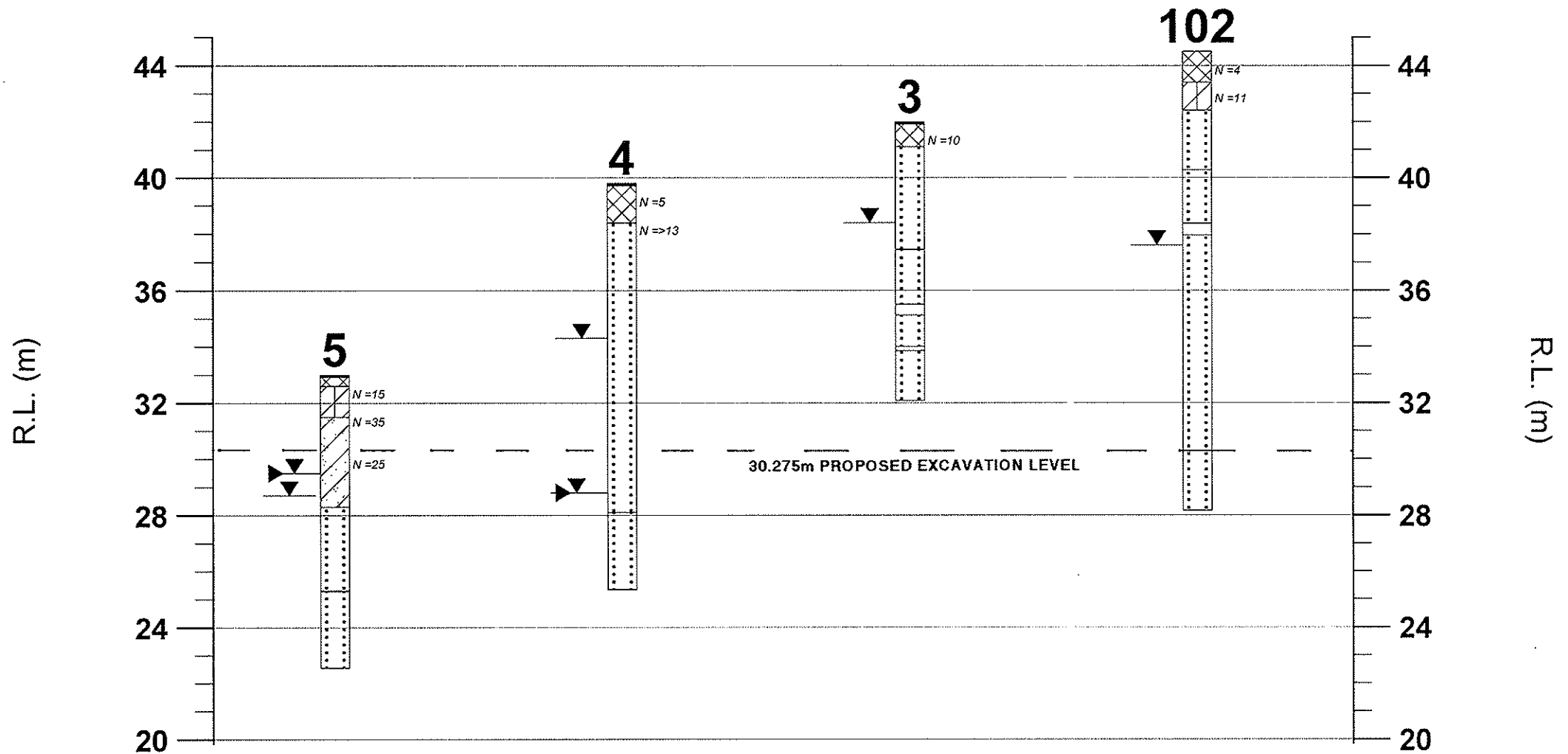
R.L. Surface: $\approx 31.5\text{m}$

Datum: AHD

Logged/Checked by: I.S./P.S.

[illegible]

GRAPHICAL BOREHOLE SUMMARY



NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

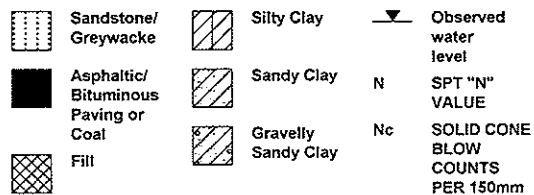
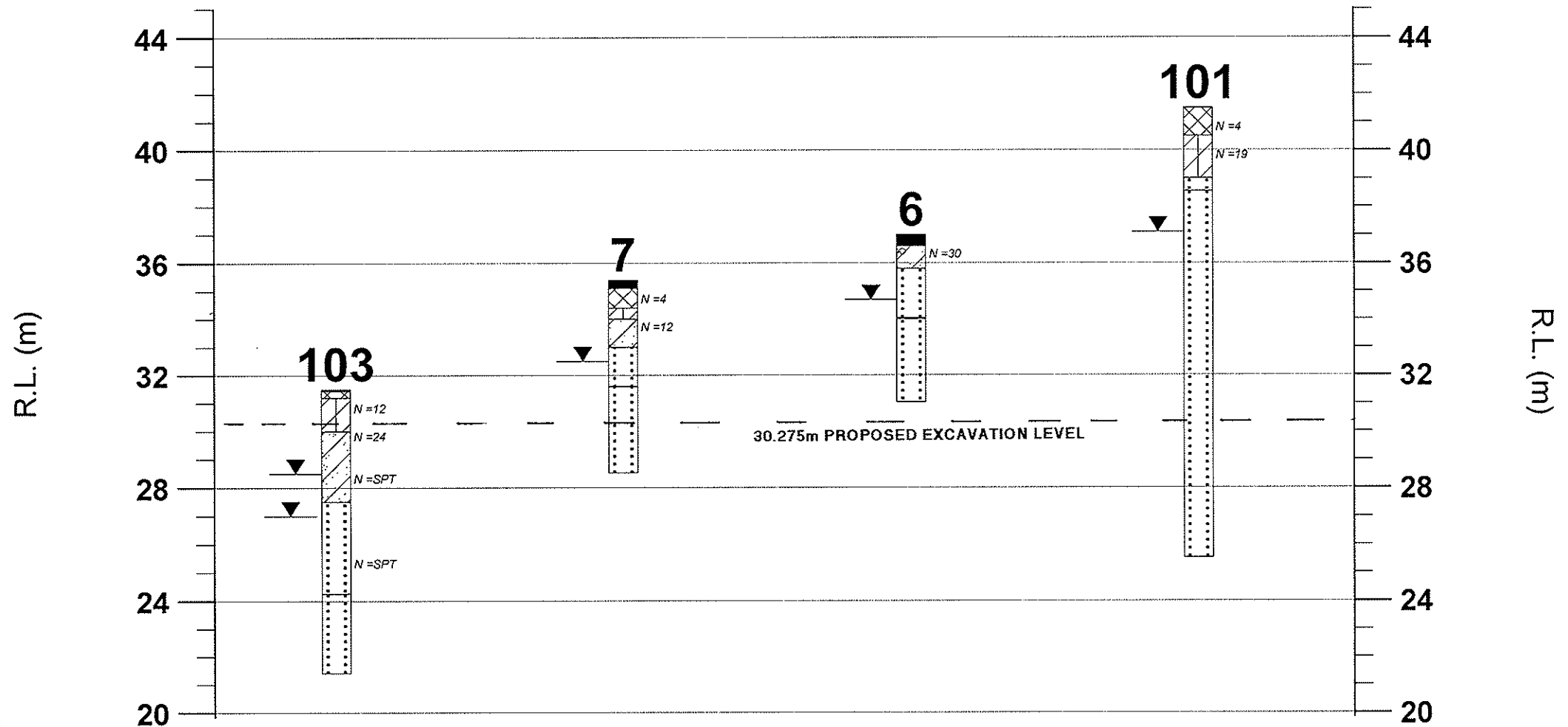
JK Geotechnics

Job No.: 26129S

Figure No.: 2



GRAPHICAL BOREHOLE SUMMARY



NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 26129S

Figure No.: 2



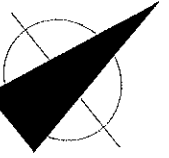
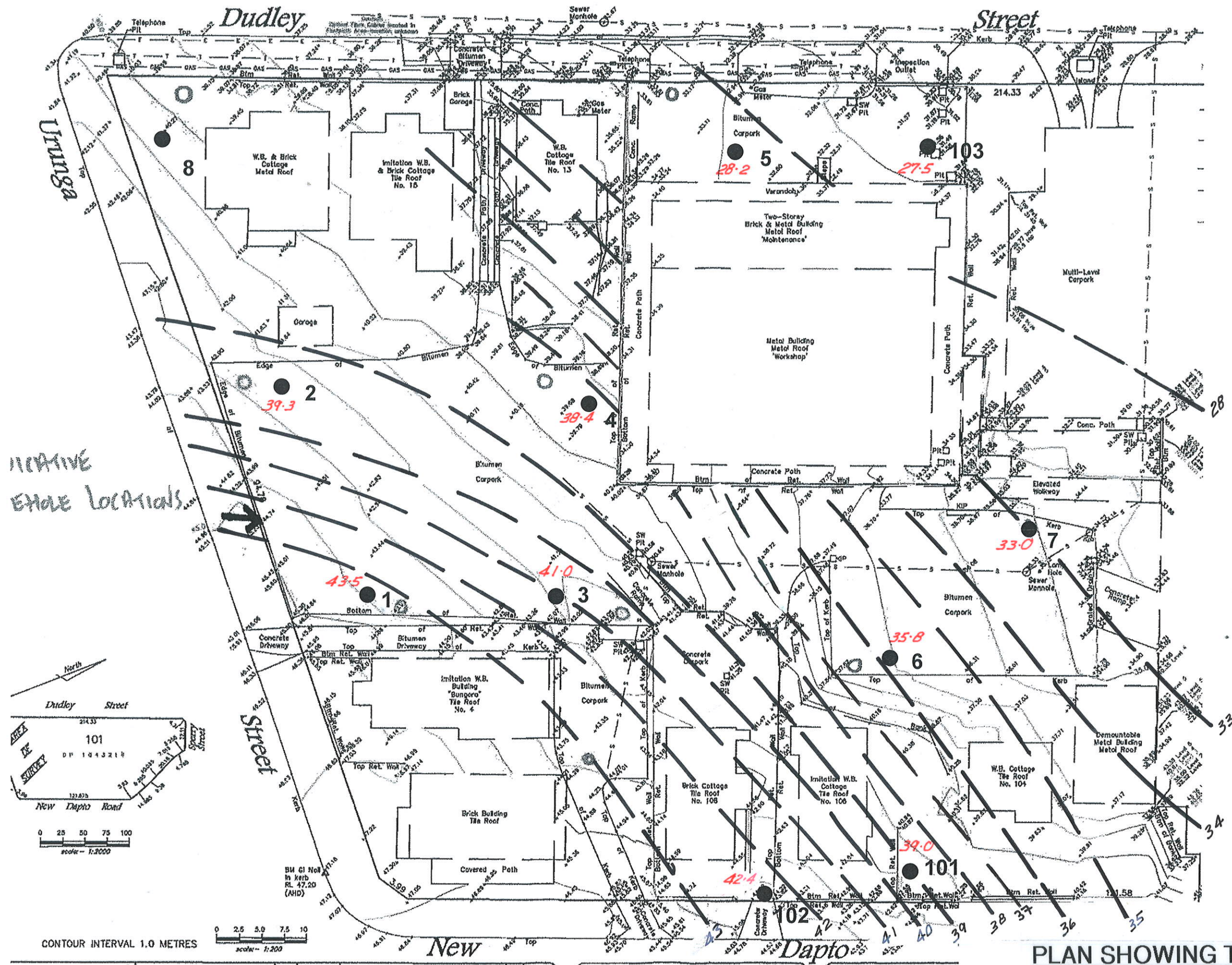


Figure No. 1





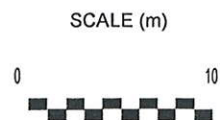
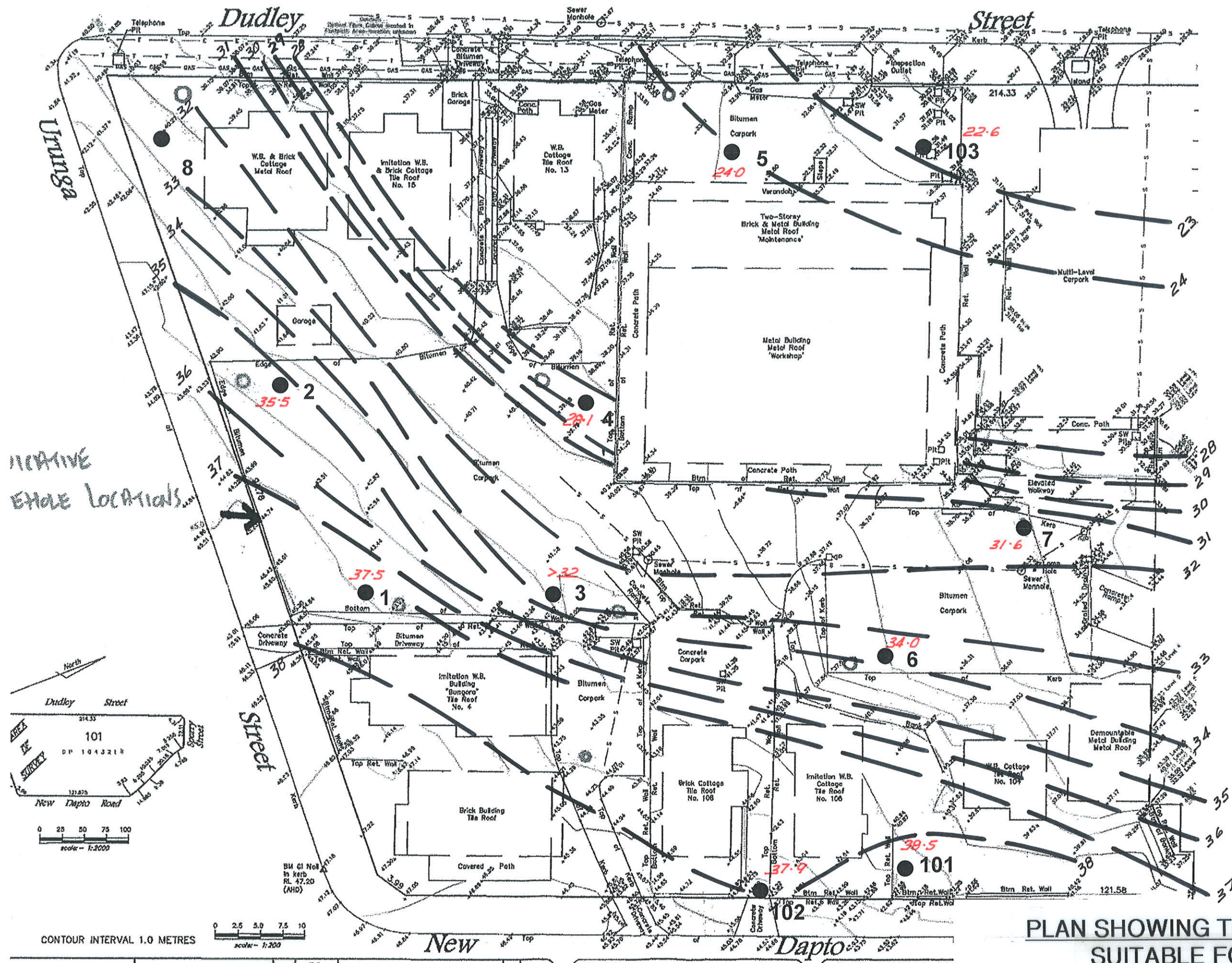
PLAN SHOWING TOP OF ROCK

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 26129S

Figure No. 3





LEGEND

37.5

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 26129S

Figure No. 3A





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 or where only a limited investigation has been completed or 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.



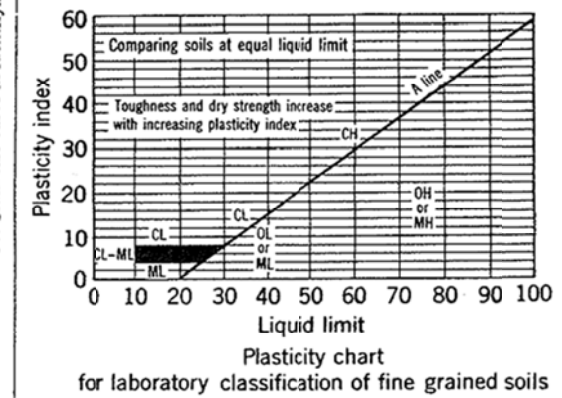
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL		ROCK		DEFECTS AND INCLUSIONS	
	FILL		CONGLOMERATE		CLAY SEAM
	TOPSOIL		SANDSTONE		SHEARED OR CRUSHED SEAM
	CLAY (CL, CH)		SHALE		BRECCIATED OR SHATTERED SEAM/ZONE
	SILT (ML, MH)		SILTSTONE, MUDSTONE, CLAYSTONE		IRONSTONE GRAVEL
	SAND (SP, SW)		LIMESTONE		ORGANIC MATERIAL
	GRAVEL (GP, GW)		PHYLLITE, SCHIST		
	SANDY CLAY (CL, CH)		TUFF		CONCRETE
	SILTY CLAY (CL, CH)		GRANITE, GABBRO		BITUMINOUS CONCRETE, COAL
	CLAYEY SAND (SC)		DOLERITE, DIORITE		COLLUVIUM
	SILTY SAND (SM)		BASALT, ANDESITE		
	GRAVELLY CLAY (CL, CH)		QUARTZITE		
	CLAYEY GRAVEL (GC)				
	SANDY SILT (ML)				
	PEAT AND ORGANIC SOILS				



Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)					Group Symbols	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)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for GW Atterberg limits below "A" line, or PI less than 4 Atterberg limits above "A" line, with PI greater than 7	
			Predominantly one size or a range of sizes with some intermediate sizes missing		GP	Poorly graded gravels, gravel-sand mixtures, little or no fines			
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see ML below)		GM	Silty gravels, poorly graded gravel-sand-silt mixtures			
	Plastic fines (for identification procedures, see CL below)		GC	Clayey gravels, poorly graded gravel-sand-clay mixtures					
	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
		Predominantly one size or a range of sizes with some intermediate sizes missing		SP	Poorly graded sands, gravelly sands, little or no fines				
Sands with fines (appreciable amount of fines)		Nonplastic fines (for identification procedures, see ML below)		SM	Silty sands, poorly graded sand-silt mixtures				
	Plastic fines (for identification procedures, see CL below)		SC	Clayey sands, poorly graded sand-clay mixtures					
	Identification Procedures on Fraction Smaller than 380 μm Sieve Size								
Fine-grained soils More than half of material is smaller than 75 μm sieve size (The 75 μm sieve size is about the smallest particle visible to naked eye)	Silt and clays liquid limit less than 50	Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	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)		
						CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity						
		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts						
	Silt and clays liquid limit greater than 50	Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	CH	Inorganic clays of high plasticity, fat clays			
						OH			Organic clays of medium to high plasticity
		Highly Organic Soils	Pt	Peat and other highly organic 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 5% GM, GC, SM, SC
 Borderline cases requiring use of dual symbols



Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. 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.



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 = <table><tr><td>5</td></tr><tr><td>7</td></tr><tr><td>3R</td></tr></table>	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.									
	5													
	7													
3R														
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 MC≈PL MC<PL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit.												
(Cohesionless Soils)	D M W	DRY – Runs freely through fingers. MOIST – Does not run freely but no free water visible on soil surface. WET – Free water visible on soil surface.												
Strength (Consistency) Cohesive Soils	VS S F St VSt H ()	VERY SOFT – Unconfined compressive strength less than 25kPa SOFT – Unconfined compressive strength 25-50kPa FIRM – Unconfined compressive strength 50-100kPa STIFF – Unconfined compressive strength 100-200kPa VERY STIFF – Unconfined compressive strength 200-400kPa 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)	VL L MD D VD ()	<table><tr><th>Density Index (I_p) Range (%)</th><th>SPT 'N' Value Range (Blows/300mm)</th></tr><tr><td>Very Loose <15</td><td>0-4</td></tr><tr><td>Loose 15-35</td><td>4-10</td></tr><tr><td>Medium Dense 35-65</td><td>10-30</td></tr><tr><td>Dense 65-85</td><td>30-50</td></tr><tr><td>Very Dense >85</td><td>>50</td></tr></table> Bracketed symbol indicates estimated density based on ease of drilling or other tests.	Density Index (I _p) Range (%)	SPT 'N' Value Range (Blows/300mm)	Very Loose <15	0-4	Loose 15-35	4-10	Medium Dense 35-65	10-30	Dense 65-85	30-50	Very Dense >85	>50
Density Index (I _p) Range (%)	SPT 'N' Value Range (Blows/300mm)													
Very Loose <15	0-4													
Loose 15-35	4-10													
Medium Dense 35-65	10-30													
Dense 65-85	30-50													
Very Dense >85	>50													
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.												



LOG SYMBOLS continued

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	0.03	Easily remoulded by hand to a material with soil properties.
Very Low:	VL	0.1	May be crumbled in the hand. Sandstone is "sugary" and friable.
Low:	L	0.3	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.
Medium Strength:	M	1	A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
High:	H	3	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.
Very High:	VH	10	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.
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	