REPORT

TO
DARCSOL PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MIXED-USE DEVELOPMENT

AT
2-12 TENNYSON ROAD, GLADESVILLE, NSW

29 October 2012
Ref: 26029SPrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 29 October 2012
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Revision No: 0

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For and on behalf of
JK GEOTECHNICS
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed mixed use development at 2-12 Tennyson Road, Gladesville, NSW. The investigation was commissioned by Darcsol Pty LTD by return of a signed 'Acceptance of Proposal' form dated 12 September 2012. The investigation was completed in accordance with our proposal P35601Prev1 dated 3 September 2012.

The site is the location of a previous quarry pit, with previous excavations extending to depths of the order of 15m below the surrounding ground levels. While the final development details were not known at the time of reporting, we understand the proposed development will comprise two basement car parking levels with a lower floor level of approximately 30.6-31.2m AHD which will require excavation to depths of about 3m below the floor of the old quarry. There will then be commercial and residential levels constructed in towers above this basement. It is likely that the development will incorporate trimming the existing quarry faces back to the boundary, with long term stabilisation measures required.

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

Preliminary information was forwarded to Grimshaw Architects by email dated 18 October 2012, and this report confirms and amplifies the preliminary information.

Jeffery and Katauskas Pty Ltd have previously completed reports at this site in 1987 (Reference 4967JS) to inspect the faces following a rock fall, and in 1990 (Reference 7979XS) where comprehensive mapping of the quarry faces was undertaken. The sections of the faces created at that time are attached in Appendix A, and the locations of the sections are provided in Figure 1.

An assessment of the potential contamination of the site soils was undertaken by Environmental Investigation Services (EIS) in conjunction with this report, and the results are provided in their report Reference E26029KPrpt.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation comprised the auger drilling of 7 boreholes to depths between 0.73m and 4.24m below the existing ground level using our truck mounted JK500 and track mounted JK305 drilling rigs and spiral auger techniques. The boreholes were then extended to depths between 6.85m and 10.19m using NMLC diamond coring techniques and water flush.

The borehole locations, as shown on the attached Figure 1, were set out by taped measurements from the existing buildings. The supplied survey plan by Stutchbury Jaques Pty Ltd (Ref: 5969/05, dated 19/10/05) forms the basis for Figure 1. Surface reduced levels (RL's) were interpolated between spot heights and contour lines on this survey plan and should therefore be considered approximate only.

The fieldwork was completed in the full time presence of a geotechnical engineer who set out the boreholes, nominated the sampling and testing locations and prepared the borehole logs. The borehole logs are attached to this report together with a glossary of terms and symbols used on the logs. The strength of the soil was assessed based on the results of Standard Penetration Test (SPT) 'N' values augmented by hand penetrometer readings on cohesive samples recovered in the SPT split spoon sampler. The strength of the rock in the augered portions of the boreholes was assessed by observation of the resistance to augering with a tungsten carbide drilling bit, and from examination of the recovered rock chips; the assessment of rock strength in such a way is subjective and variations of one strength order should not be unexpected. The strength the bedrock in the cored portions of the boreholes was assessed by inspection of the recovered core and correlation with the results of point load strength index tests completed on the core in the laboratory.

A Senior Engineering Geologist also visited the site to map the jointing observed in the quarry faces. The results of the mapping are provided in Section 3.1.

Selected samples were returned to a NATA registered laboratory, Soil Test Services Pty Ltd (STS), for moisture Atterberg limit, linear shrinkage, standard compaction and soaked CBR tests. The results of these tests are summarised in the attached Tables A and B. Additional samples were delivered to Envirolab Services Pty Ltd, a NATA registered analytical laboratory, for testing for soil pH, sulphate content and chloride content; the results of these tests are provided in the attached Envirolab Services Certificates of Analyses.

The core of the bedrock was also returned to STS where it was colour photographed, and where point load strength index tests were completed. Copies of the photographs are provided with the borehole logs, while the strength test results are summarised in Table C.

For further detail on the investigation procedures used reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located close to the top of a south facing hillside which slopes towards the Parramatta River. The area about the site generally slopes down to the south-west at about 5°.

The site itself is a former quarry which has been cut into the hillside. Shale bedrock is exposed in rock faces up to approximately 15m high along the northern and eastern edges of the site. The condition of these rock faces is described in more detail below. The western excavation face is concealed by an existing office building and vehicle access ramps. The lower southern face comprises a vegetated batter slope of 20° to 25°.

At the time of the fieldwork, the majority of the site was relatively level and was occupied by a single storey metal clad warehouse structure and associated asphaltic concrete driveway and hardstand areas. The north-western end of the site sloped down at between approximately 4° and 10° from the Tennyson Road site boundary. A two storey rectangular, cement rendered brick office building was located on this sloping portion of the site. Both the warehouse and office buildings appeared in good external condition based on a cursory inspection from the exterior. The asphaltic concrete driveway and hardstand areas appeared in poor condition with numerous repaired and delaminated areas observed.

Tennyson Road passes along the north-western site boundary and slopes down to the south-west at about 5°. There are existing hardstand areas with commercial building beyond to the north, east and south of the site. There are also numerous medium to tall trees around the toe and crest of the quarry faces.

Quarry Face Description

From assessment of the shale cuts surrounding the site both from our previous works in 1990 and from our current inspection, there are six major joint sets and three faults as detailed below.

Description	Dip	Dip Direction
Joint Set A	80° - 90°	290° - 300° / 110° - 120°
Joint Set B	80° - 85°	230° - 250°
Joint Set C	40° - 70°	025° - 040°
Joint Set D	85° - 90°	160°
Joint Set E	65°	140° - 160°
Joint Set G	35° - 50°	100° - 120°
Fault Plane F1	40° - 45 °	210° - 220°
Fault Plane F2	45°	285°
Fault Plane F3 (similar to JS C)	35° - 45°	020° - 030°

During the recent inspection of the face, we were unable to detect the previously mapped Joint Set G, as the two areas where it was picked up originally were very overgrown. However, as shown in the original Section 4 (7676XS, Fig 3), this is most likely at a similar dip/gradient to the scree slope locally.

The shale in the cut faces appeared to be predominantly of medium strength, and the faces ranged in height between 4m and 16m. During the recent inspection, there had been further degradation/spalling of the face since 1990 with a near continual scree pile of shale at the base of the cliff now being present. Less spalling was noted along the north-western portion of cliff, however seepage through bedding/jointing was noted in this area.

From the car park, up to the base of the cliff are old scree deposits or fill from clearing the lower portion of the site prior to construction. These slopes ranged in gradient between 25° - 50°, and up to 10 to 12m wide in places.

The ‘large slide debris’ noted in the 1987 report appeared to have been removed from site.

Approximately 50m section of the north-eastern shale cut/cliff face has been formed by a continuous joint belonging to Joint Set B, with the exposed portion of the joint being up to about 15m high. This joint was intersected by many joints belonging to Joint Set A along the majority of the face. At the south-eastern end of this exposed portion of joint, the cut face curves and the rock in front of the joint has separated from joint by about 100mm to 120mm.

Overhangs of the cliff face were up to 3m, and typically at the top of the cliff faces near the soil/rock interface. With undercutting of the cliff face quite prevalent to the eastern and north-western faces where undercuts were mostly 1m to 1.5m deep.

At north-western corner of the quarry there was a large detached block 4m × 6m about mid-height of the face, with trees growing behind the block.

3.2 Subsurface Conditions

In general terms, the investigation has disclosed a relatively thin layer of residual soil at the crest of the cut faces, with fractured shale forming the majority of the previous quarry cuts. The conditions encountered in the boreholes drilled in the floor of the quarry comprise concrete and asphaltic concrete pavements over a thin layer of fill which overlies high and very high strength sandstone bedrock; the bedrock also contains minor bands of interbedded sandstone and shale. The exception was in BH7 which was drilled in an elevated vehicle access area where deeper fill was encountered.

Some of the characteristic features of the strata encountered are described below. For further details of the conditions encountered at each location, reference should be made to the borehole logs. A graphical summary of the strata encountered in the boreholes is presented in Figure 2.

Pavements

The boreholes were drilled through pavements comprising up to 100mm of concrete, or 40mm to 60mm of asphaltic concrete. No crushed igneous rock roadbase was encountered in these pavements. The exception was BH3 where there was no pavement.

Fill

The pavements were overlying fill comprising sandy gravel (comprising crushed shale gravel) and gravelly silty clay of generally medium and high plasticity. The fill extended to depths ranging between 0.3m and 1.1m, with the exception of BH7 which was drilled in an elevated area and contained fill to a depth of 3.5m. The fill assessed as being moderately to well compacted.

Bedrock

The shale bedrock in the excavation faces is assessed as being moderately weathered and of medium strength, though the faces themselves are relatively fractured. The bedrock below the floor of the quarry comprises fine and medium grained sandstone, generally slightly weathered or fresh, with strengths ranging from high to very high, with sometimes a thin capping of low to medium strength rock. The sandstone contains bands which are interbedded with shale. There were very few defects encountered in the cored boreholes, and these were mostly near horizontal bedding partings and crushed seams; there were also several joints with inclinations of between 45° and subvertical, and these were within the upper 3m of the boreholes.

Laboratory Test Results

The moisture content test results correlated well with the field logging assessments of rock strength, and the Atterberg limits tests showed the samples tested to be of low to medium plasticity, correlating with a low to moderate for shrink-swell movements with changes in moisture content. The soaked CBR values were 3% and 8%, indicating the subgrade is fair.

The samples of the weathered shale tested were found to be slightly alkaline with pH ranging from 8.1 to 9.2, and very low sulphate and chloride contents with all results less than 150mg/kg.

The point load strength tests completed on the recovered core showed the sandstone and shale to have strengths generally of high and very high strength, with correlated UCS ranging between 12MPa and greater than 112MPa, with an average value of 50MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Project Overview

The majority of this project will be relatively standard in terms of the geotechnical issues. The excavation for the floor of the quarry will require the use of large equipment and maybe require sawing due to the strength of the rock likely to be encountered, and some stabilisation of new rock cuts such as by rock bolts will be required to stabilise wedges formed by the inclined joints encountered in the upper portions of the boreholes. We expect conventional pad and strip footings will be suitable due to the very high quality of the rock encountered.

By far the most difficult part of this project will be the safe excavation and support of the shale faces where it is proposed to excavate the site back to the boundaries. These activities will require careful consideration during the design process, and possibly significant geotechnical review during the construction phase.

4.2 Excavation of Quarry Faces

The excavation of the existing quarry faces back to the boundary will extend through the upper residual soils, and through shale bedrock which is likely to be of medium strength. This is likely to require moderate ripping with tracked excavators of say 30 tonne size. However gaining access to the excavation area will be difficult and will require the placement of a large fill platform against the existing cut faces.

It will also be difficult to undertake this excavation in a safe manner as it will be necessary to protect the existing properties, infrastructure and buildings beyond the site boundary. The shale faces currently expose relatively major continuous joints which are inclined, and if these daylight in the final excavation face, they could collapse into the excavation unless properly restrained. Some concepts for appropriate restraint are provided below, though it is likely that there will be additional information required by the structural engineers when the general concept of the shoring has been decided.

Anchored Shotcrete Facing Approach

We expect the preferred way to undertake this excavation and shoring would be to progressively excavate and install an anchored reinforced shotcrete facing to the batters. The shotcrete facing could then be used both in the short term and long term, provided the reinforcement is designed to suit both cases. In the short term, the shotcrete would need to be designed to span between the anchor locations, while in the long term, it would need to span between the floor slabs and/or shear walls in the adjacent building.

We expect that it would be advantageous to use large excavator mounted rock grinders to grind the shale face following the bulk excavation of each lift to provide a shotcrete facing of relatively uniform thickness and geometry.

The soils and weathered shale to low strength should be excavated in vertical lifts not exceeding 1.5m, with anchors in the upper 2.5m (but in any case to the bottom of any low strength shale) not being spaced more than 1.5m vertical height apart. Where the excavation is in shale of medium strength or greater, it would be suitable to undertake the excavation in 2.5m vertical lifts with anchors at no more than 2.5m vertical centres.

The temporary anchors would extend through to the outside face of the shotcrete to allow them to be de-stressed following the bracing of the walls by the structures. These anchors should have a bond zone entirely behind a line drawn upward at 1V in 1H from the toe of the proposed excavation, and the bond length may provisionally be designed for an allowable bond of 150kPa in shale of at least low strength and 250kPa in shale of at least medium strength. All anchors should then be proof loaded to at least 130% of their working load, and 50% of anchors subjected to lift off tests approximately three days after lock-off to confirm the anchors are holding their load.

If any of these anchors show a load loss of more than 10% from their lock-off load, then all anchors should be lifted-off. It will be necessary to obtain permission from the owners of the adjacent properties prior to the installation of anchors beyond the boundary. It is normal practice for anchors to be a design and construct package so that the risk of anchor failure is balanced against the cost of drilling and replacing any anchors that fail load tests.

Soldier Pile Wall

An alternative excavation support system would be to install soldier piles around the site perimeter prior to the commencement of excavation. These would then be restrained with multi-level temporary anchors installed progressively with the excavation. Following approximately each 1.0m of vertical excavation in the residual soils and shale of extremely and very low strength, and each 2.5m of vertical excavation in the shale of low strength or stronger, reinforced shotcrete panels should be sprayed between the soldier piles to prevent localised collapse resulting from small scale joints and wedges.

The installation of the piles themselves would be problematic as they will either require the use of a piling rig in the adjacent properties which is unlikely to be possible, or the installation of a high working platform to provide safe access to the work site. The piling rig would also need to be large to be able to reach the required founding depth, and drill through the shale which is expected to be of medium or higher strength. It is also likely there would be some degree of wander in such long piles, and so these may cut into the proposed basement area. The likelihood of wander may be able to be reduced by using large diameter down-hole hammers for the piling, though we note their use is not common at this point in time.

Design Pressures

If no other investigation work is done, the shoring should be designed for a semi-trapezoidal lateral earth pressure with a maximum magnitude of 7H kPa (where H is the depth of excavation in metres) applying over the lower 75% of the face, tapering to zero at the crest. .

These pressures are relatively conservative, and it is likely that they could be reduced following further detailed investigation of the perimeter conditions to prove that any jointing is relatively step (about 75º from the horizontal) rather than being closer to 55º which causes the highest shoring loads. It would then be likely to be able to reduce the pressures to say the same pressure distribution but with a maximum pressure of about 5H. Such investigation would likely be completed following removal of vegetation and cleaning debris from the face, and involve inspecting the faces from a boom lift, combined with drilling some inclined cored boreholes from the toe of the existing cut to look for defects which dip at around 55º which may not be evident in the face. The investigation could not be undertaken with a fixed scope as the program would need to be amended based upon the results obtained during the fieldwork.

The above pressures are based upon the rock mass stabilisation only, and appropriate surcharge loads and hydrostatic pressures should also be taken into account in the design.

Drainage

Irrespective of the retention system used, drainage must be installed behind the facing to allow the permanent dissipation of the pore water pressures.

4.3 Excavation Below Quarry Floor

It appears that it will be necessary to excavate to a depth of about 3m below the floor of the quarry to achieve the required basement levels. This excavation will be through the pavements and fill, and into the sandstone bedrock of up to very high strength.

The removal of the concrete pavements will require the use of rock hammer attachments to hydraulic excavators for effective excavation. The fill below would then be readily excavated using buckets on tracked excavators.

Some of the upper rock in the boreholes has strength ranging from extremely low to low to medium strength. Any sandstone to low strength and shale to medium is likely to be rippable using ripping tynes on large (say 30 tonne or larger) tracked excavators.

Higher strength rock will require the use of rock breaker attachments to the excavators, and even then productivity may be very low due to the high and very high strength of the rock. It may be necessary to saw cut the sandstone and break blocks of rock from the excavation. The use of an impact ripper on a heavy (D10 or D11 sized) tractor should also be considered.

There is the potential for inclined defects to form wedges of rock in the excavation faces which could be unstable. Therefore the excavation faces should be inspected following each 1.5m vertical lift of excavation to assess the presence of any such features, If these features are present, it is likely that stabilisation will be required, such as by the installation of rock bolts.

Provided the sandstone can be appropriately crushed, it is likely to produce an ideal engineered fill material.

If there is any rock of less than low strength exposed at the perimeter of the excavation, this should be supported by a reinforced shotcrete panel, which could be laterally restrained by rock bolts in the short term, and by bracing from the structure in the long term.

4.4 Footing Design

It is likely that sandstone and shale of high and very high strength will be encountered at the bulk excavation levels, and so pad and strip footings would be considered to be feasible. Footings founded with an embedment of at least 0.5m below the surrounding ground level in the shale or sandstone of high or very high strength may be designed for an allowable bearing pressure of 3,500kPa based upon visual inspection of each footing excavation by a geotechnical engineer. Undertaking spoon testing in one in three footing excavations and visually inspecting the remainder would allow the adoption of an allowable bearing pressure of 6,000kPa.

It may in fat be feasible to use even higher bearing pressures, possibly to 10,000 kPa, though this will require significant additional diamond coring, and we do not expect that such pressures would be advantageous for the proposed development.

4.5 Subgrade Preparation

It is expected that the subgrade to the proposed lower basement floor slabs will comprise high and very high strength sandstone and shale bedrock. Therefore, unless there is significant excavation below the proposed floor slabs such as for the installation of services etc, detailed preparation of the subgrade is unlikely to be required. We recommend the placement of a crushed rock separation layer between the rock and the concrete floor slabs to prevent curling of slabs associated with concrete shrinkage where the underside of the slabs has a rough contact with the rock and to permit drainage.

Where such excavation of the rock occurs, or if pavements are deleted such that the existing soils form the subgrade, the pavements should be provisionally designed for a soaked CBR value of 3.0%. This may well be a conservative, and should be confirmed by geotechnical inspection and possibly additional testing when final development details are known.

When details of the proposed development have been finalised, further advice should be obtained with regard to subgrade preparation, as it may be necessary to excavate some of the fill soils from site and rework it as engineered fill placed and compacted in layers.

4.6 Hydrogeological Considerations

Seepage is currently occurring at least through the north-western portion of the quarry face. Also, the total excavation depth will be relatively deep, and it will be necessary to incorporate drainage into the proposed retaining walls and below the lower basement floor slab. This could comprise vertical lengths of strip drain installed behind retaining wall elements, combined with a subsoil drain around the perimeter of the lower basement level, and also either a gravel drainage blanket or a grid of subsoil drains below the proposed lower basement floor slabs.

4.7 Soil Aggression

The weathered shale is alkaline, and has very low sulphate and chloride contents. These conditions must be taken into account in the design of metal or reinforced concrete elements in contact with the shale.

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, geotechnical inspection of the footing excavations prior to pouring, 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 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.

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.

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.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville

Ref No: 26029SP
Report: A
Report Date: 10/10/2012
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	0.60-0.78	1.5				
2	0.30-0.50	14.6				
2	0.60-0.73	2.6				
3	0.00-0.20	7.9	33	16	17	7.5
3	0.50-0.65	13.5				
4	0.50-0.90	14.6				
4	1.10-1.28	2.7				
5	0.60-0.80	1.7				
6	0.85-0.95	9.0				
6	1.35-1.55	4.1				
7	0.70-0.95	13.9				
7	1.30-1.50	15.8				
7	1.70-1.95	19.7				
7	3.00-3.45	10.4	28	19	9	5.0
7	4.00-4.24	6.3				

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 24/09/2012

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville

Ref No: 26029SP
Report: B
Report Date: 10/10/2012
Page 1 of 1

BOREHOLE NUMBER	3	4
DEPTH (m)	0.00 - 0.50	0.10 - 0.90
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.89 STD	1.93 STD
Optimum Moisture Content (%)	12.3	12.6
Moulded Dry Density (t/m^3)	1.86	1.89
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	102	102
Moisture Contents		
Insitu (%)	11.8	13.5
Moulded (%)	12.5	12.8
After soaking and		
After Test, Top 30mm(%)	19.7	16.0
Remaining Depth (%)	15.8	13.7
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	2.5	0.0
C.B.R. value: @2.5mm penetration		8
@5.0mm penetration	3.0	

NOTES:

- Refer to appropriate notes for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample: 24/09/2012

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Authorised Signature / Date
(A. Tatikonda) 10/10/12

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	26029SP
Project:	Proposed Development	**Report:**	C
Location:	2-12 Tennyson Road, Gladesville	**Report Date:**	
		Page 1 of 4	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	0.81-0.85	2.4	48
	1.38-1.41	2.6	52
	2.10-2.14	2.2	44
	2.90-2.93	3.9	78
	3.51-3.54	2.4	48
	4.21-4.25	3.4	>68
	4.89-4.92	3.8	76
	5.53-5.58	0.9	18
	6.18-6.21	1.4	28
	6.78-6.82	1.0	20
2	1.05-1.08	3.4	68
	1.73-1.75	3.7	74
	2.39-2.42	4.2	84
	3.05-3.07	3.4	68
	3.77-3.80	1.7	34
	4.40-4.42	2.4	48
	5.04-5.08	2.0	40
	5.70-5.74	1.1	22
	5.56-5.60	0.9	18
3	0.81-0.84	0.6	12
	0.84-0.87	0.6	12
	1.47-1.51	3.5	>70
	1.96-1.99	3.3	66
	2.58-2.61	3.5	70
	3.20-3.22	4.7	94

NOTES: See Page 4 of 4

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville

Ref No: 26029SP
Report No: C
Report Date:
Page 2 of 4

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
3	3.86-3.89	2.9	58
	4.54-4.57	1.8	36
	5.09-5.12	1.7	34
	5.81-5.84	2.2	44
	6.59-6.62	0.9	18
4	1.47-1.51	1.0	20
	2.17-2.21	3.7	>74
	2.84-2.87	3.0	60
	3.51-3.55	4.1	>82
	4.18-4.20	4.2	84
	4.92-4.95	5.4	108
	5.50-5.53	2.7	54
	6.15-6.19	3.3	66
	6.81-6.84	0.9	18
	7.35-7.40	1.1	22
	8.04-8.07	1.3	26
	8.70-8.73	1.5	30
	8.73-8.76	0.6	12
5	0.87-0.91	1.5	30
	1.55-1.58	1.3	26
	2.19-2.23	0.9	18
	2.86-2.89	1.5	30
	3.52-3.56	3.7	74
	4.24-4.27	3.2	64
	4.96-4.99	4.6	>92

NOTES: See Page 4 of 4

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville

Ref No: 26029SP
Report No: C
Report Date:
Page 3 of 4

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
5	5.62-5.65	5.1	>102
	6.31-6.33	5.0	100
	7.02-7.06	2.2	44
	7.71-7.74	1.2	24
	8.40-8.44	1.4	28
	9.03-9.06	1.2	24
6	2.13-2.16	4.5	90
	2.83-2.86	4.2	84
	3.50-3.53	4.2	84
	4.15-4.17	1.9	38
	4.80-4.83	5.6	>112
	5.45-5.48	3.5	70
	6.07-6.09	1.2	24
	6.69-6.73	1.4	28
	7.43-7.46	0.9	18
	8.07-8.10	1.3	26
	8.74-8.78	1.6	32
	9.52-9.55	1.1	22
7	4.31-4.34	4.1	82
	4.97-5.00	2.4	48
	5.64-5.68	2.8	56
	6.29-6.32	2.1	42
	6.97-7.00	2.5	50
	7.68-7.72	3.0	60
	8.32-8.34	1.9	38

NOTES: See Page 4 of 4

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville

Ref No: 26029SP
Report No: C
Report Date:
Page 4 of 4

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
7	9.00-9.03	1.8	36
	9.78-9.82	1.1	22

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 $I_{S\,(50)}$

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

CERTIFICATE OF ANALYSIS 79191-A

Client:

Jeffery & Katauskas Pty Ltd
PO Box 976
North Ryde BC
NSW 1670

Attention: Rob Cater

Sample log in details:

Your Reference:	**E26029KP, Gladesville**
No. of samples:	Additional testing on 2 soils
Date samples received / completed instructions received	21/09/2012 / 25/09/12

Analysis Details:

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

Report Details:

Date results requested by: / Issue Date:	3/10/12 / 3/10/12
Date of Preliminary Report:	Not issued

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Results Approved By:

Nick Sarlamis
Inorganics Supervisor

ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil			
Our Reference:	UNITS	79191-A-4	79191-A-11
Your Reference	-------------	BH2	BH5
Depth	------------	0.6-0.73	0.4-0.6
Date Sampled		19/09/2012	19/09/2012
Type of sample		Soil	Soil
Date prepared	-	27/09/2012	27/09/2012
Date analysed	-	27/09/2012	27/09/2012
pH 1:5 soil:water	pH Units	8.9	9.2
Chloride, Cl 1:5 soil:water	mg/kg	50	31
Sulphate, SO4 1:5 soil:water	mg/kg	87	94

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			27/09/2012	[NT]	[NT]	LCS-1	27/09/2012
Date analysed	-			27/09/2012	[NT]	[NT]	LCS-1	27/09/2012
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	102%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	116%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	119%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batched of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.

Aileen Hie

From: Brendan Page [bpage@jkgroup.net.au]
Sent: Tuesday, 25 September 2012 1:56 PM
To: Aileen Hie
Cc: 'Robert Cater'; 'Peter Wright'
Subject: Additional Analysis 79306 and 79191

Hi Aileen,

Could you please schedule the following additional analysis on the samples in Envirolabs custody (please issue as separate 'A' reports):

79306-5 (BH7 4-4.25): pH, sulfate and chloride
79191-4 (BH2 0.6-0.73): pH, sulfate and chloride
79191-11 (BH5 0.4-0.6): pH, sulfate and chloride

Note sure if its possible, however, could you arrange for this **additional analysis to be invoiced under JK project reference 26029SP** and attention it to Rob Cater.

The invoice for the remaining analysis can come to me.

Give me a buzz if there are any issues.

79191 A
std T/A
due 3/10

Regards,

Brendan Page
Senior Environmental Scientist

EIS Environmental Investigation Services
CONSULTING ENVIRONMENTAL ENGINEERS AND SCIENTISTS
Tel: 02 9888 5000 PO Box 976 115 Wicks Road
Fax: 02 9888 5001 North Ryde BC NSW 1670 Macquarie Park NSW 2113
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Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 79306-A

Client:

Jeffery & Katauskas Pty Ltd
PO Box 976
North Ryde BC
NSW 1670

Attention: Rob Cater

Sample log in details:

Your Reference:	**E26029KP, Gladesville**
No. of samples:	Additional testing on 1 soil
Date samples received / completed instructions received	24/09/2012 / 25/09/2012

Analysis Details:

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

Report Details:

Date results requested by: / Issue Date:	3/10/12 / 3/10/12
Date of Preliminary Report:	Not issued

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Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Nick Sarlamis
Inorganics Supervisor

ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil		
Our Reference:	UNITS	79306-A-5
Your Reference	-------------	BH7
Depth	------------	4-4.25
Date Sampled		24/09/2012
Type of sample		Soil
Date prepared	-	27/09/2012
Date analysed	-	27/09/2012
pH 1:5 soil:water	pH Units	8.1
Chloride, Cl 1:5 soil:water	mg/kg	93
Sulphate, SO4 1:5 soil:water	mg/kg	140

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

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Date prepared	-			27/09/2012	[NT]	[NT]	LCS-1	27/09/2012
Date analysed	-			27/09/2012	[NT]	[NT]	LCS-1	27/09/2012
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	102%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	116%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	120%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

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Aileen Hie

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Sent: Tuesday, 25 September 2012 1:56 PM
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Cc: 'Robert Cater'; 'Peter Wright'
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Note sure if its possible, however, could you arrange for this **additional analysis to be invoiced under JK project reference 26029SP** and attention it to Rob Cater.

The invoice for the remaining analysis can come to me.

Give me a buzz if there are any issues.

79306 A
std T/A
due 3/10

Regards,

Brendan Page
Senior Environmental Scientist

Environmental Investigation Services
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Fax: 02 9888 5001 North Ryde BC NSW 1670 Macquarie Park NSW 2113
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www.jkgeotechnics.com.au

JK **Geotechnics**
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/2

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

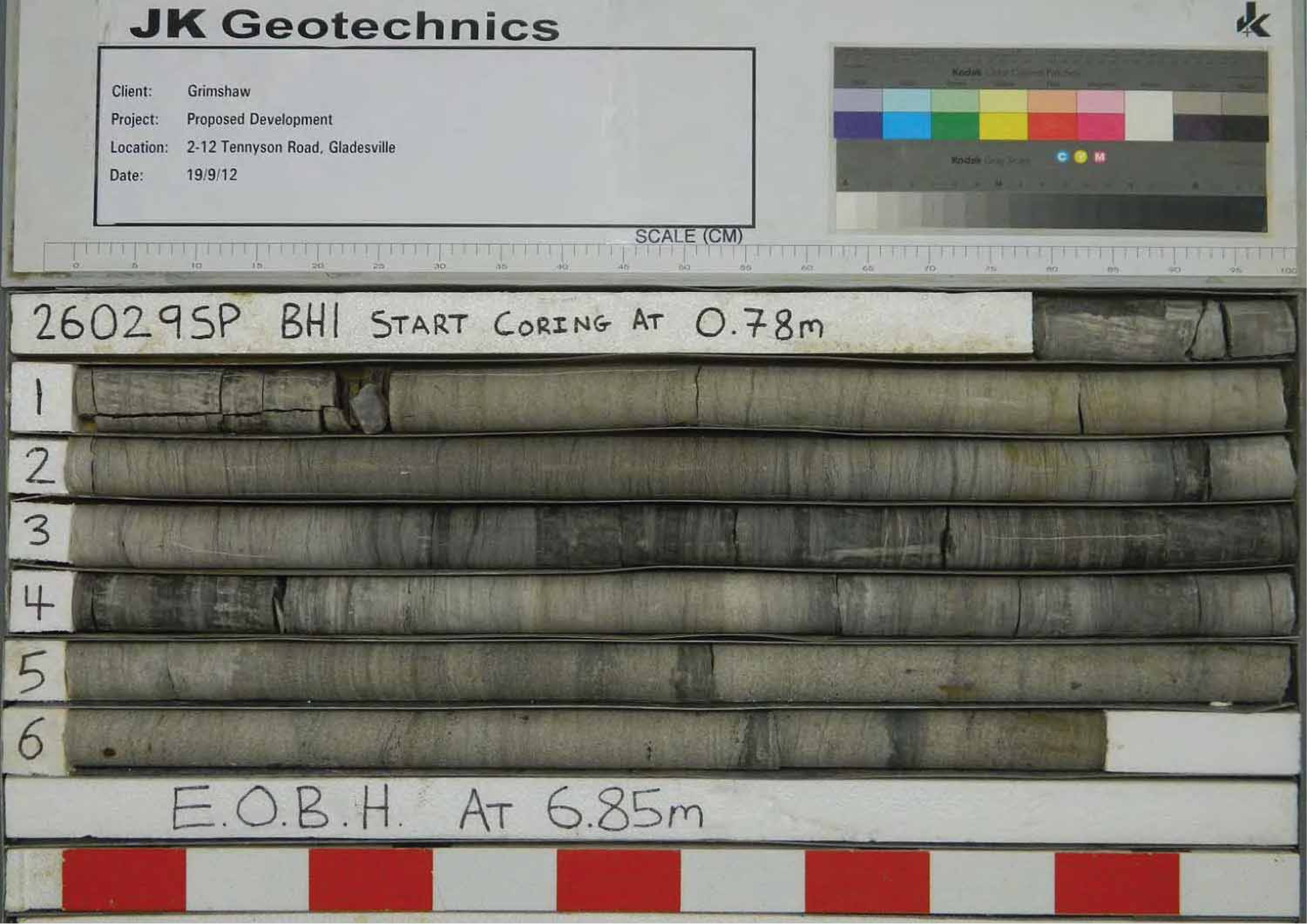
Job No. 26029SP
Date: 19-9-12

Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 33.4m
Datum: AHD

Logged/Checked by: R.V.C./P.W.

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	▨		CONCRETE: 100mm.t				10mm DIA. REINFORCEMENT
	■			■			☒	-	FILL: Sandy gravel, fine to coarse grained crushed shale, fine to medium grained sand, trace of silt.	W			
	■			■			▤	-	SHALE: dark grey.	DW	L-M		BANDED LOW TO MODERATE 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						1							
						2							
						3							
						4							
						5							
						6							
						7							



JK Geotechnics

Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 19/9/12

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
1
2/2

Client: Darcsol Ltd Pty

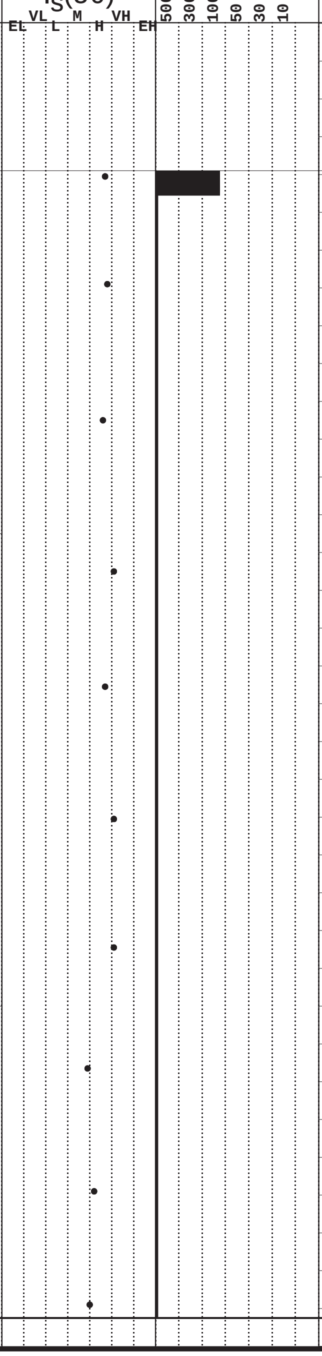
Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Date: 19-9-12
Drill Type: JK500

Core Size: NMLC
Inclination: VERTICAL
Bearing: -

R.L. Surface: ≈ 33.4m
Datum: AHD
Logged/Checked by: R.V.C./P.W.



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 $I_S(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0		START CORING AT 0.78m						
FULL RET-URN		1		SHALE: dark grey.	FR	H			- J, 85°, P, R, IS, 345mm.t	
		2		SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.						
		3				H-VH				
		4		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-10°.						
		5		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.					- Be, 0°, 4mm.t	
		6				H				
				SANDSTONE: medium grained, light grey, bedded at 0-10°.						
		7		END OF BOREHOLE AT 6.85m						

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BOREHOLE LOG

Borehole No.
2
1/2

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP

Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 33.1m

Date: 19-9-12

Datum: AHD

Logged/Checked by: R.V.C./P.W.

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						0	■		ASPHALTIC CONCRETE: 60mm.t				
	■						▨	-	FILL: Sandy gravel, fine to coarse grained crushed shale gravel and fine to medium grained sand, trace of silt and clay.	M			
				■			▨		FILL: Gravelly silty clay, low to medium plasticity, dark grey, fine to coarse grained shale gravel, trace of fine grained sand.	MC>PL			
	■			■	SPT 4/100mm REFUSAL		▤	-	SHALE: dark grey.	DW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						1			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 19/9/12

26029SP BH2 START CORING AT 0.73m

E.O.B.H. AT 6.87m

CORED BOREHOLE LOG

Borehole No.
2
2/2

Client: Darcsol Ltd Pty

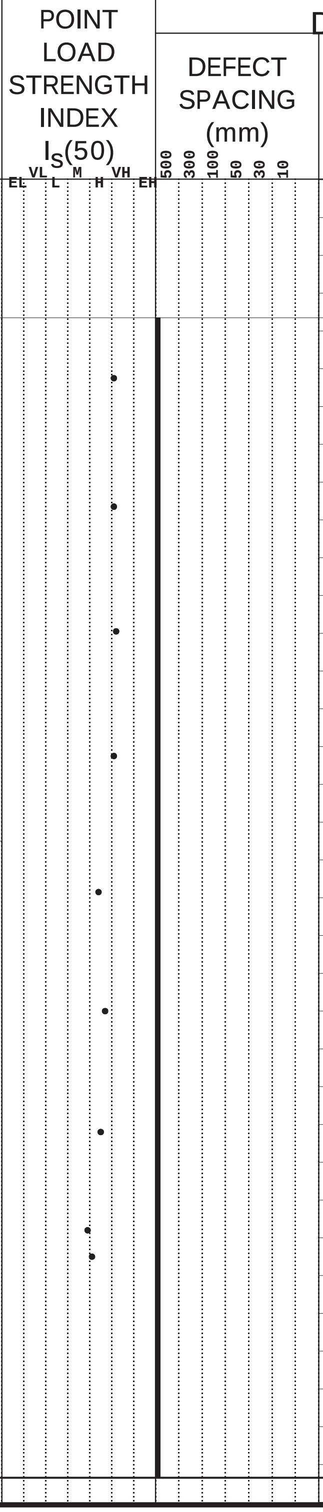
Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Core Size: NMLC
R.L. Surface: ≈ 33.1m

Date: 19-9-12
Inclination: VERTICAL
Datum: AHD

Drill Type: JK500
Bearing: -
Logged/Checked by: R.V.C./P.W.



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 $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0		START CORING AT 0.73m						
				SHALE: dark grey. SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.	FR	VH			- Cr, 25mm.t, IS	
		1								
		2								
		3		INTERBEDDED SANDSTONE: light grey, and SHALE: dark grey, bedded at 0°.						
						H				
FULL RET-URN				SANDSTONE: fine to medium grained, light grey, with occasional dark grey laminae, bedded at 0-10°.						
		4								
				as above, but cross bedded at 20°.						
		5								
				SANDSTONE: medium grained, light grey, bedded at 0-10°.						
		6								
				END OF BOREHOLE AT 6.87m						
		7								

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/2

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP

Date: 19-9-12

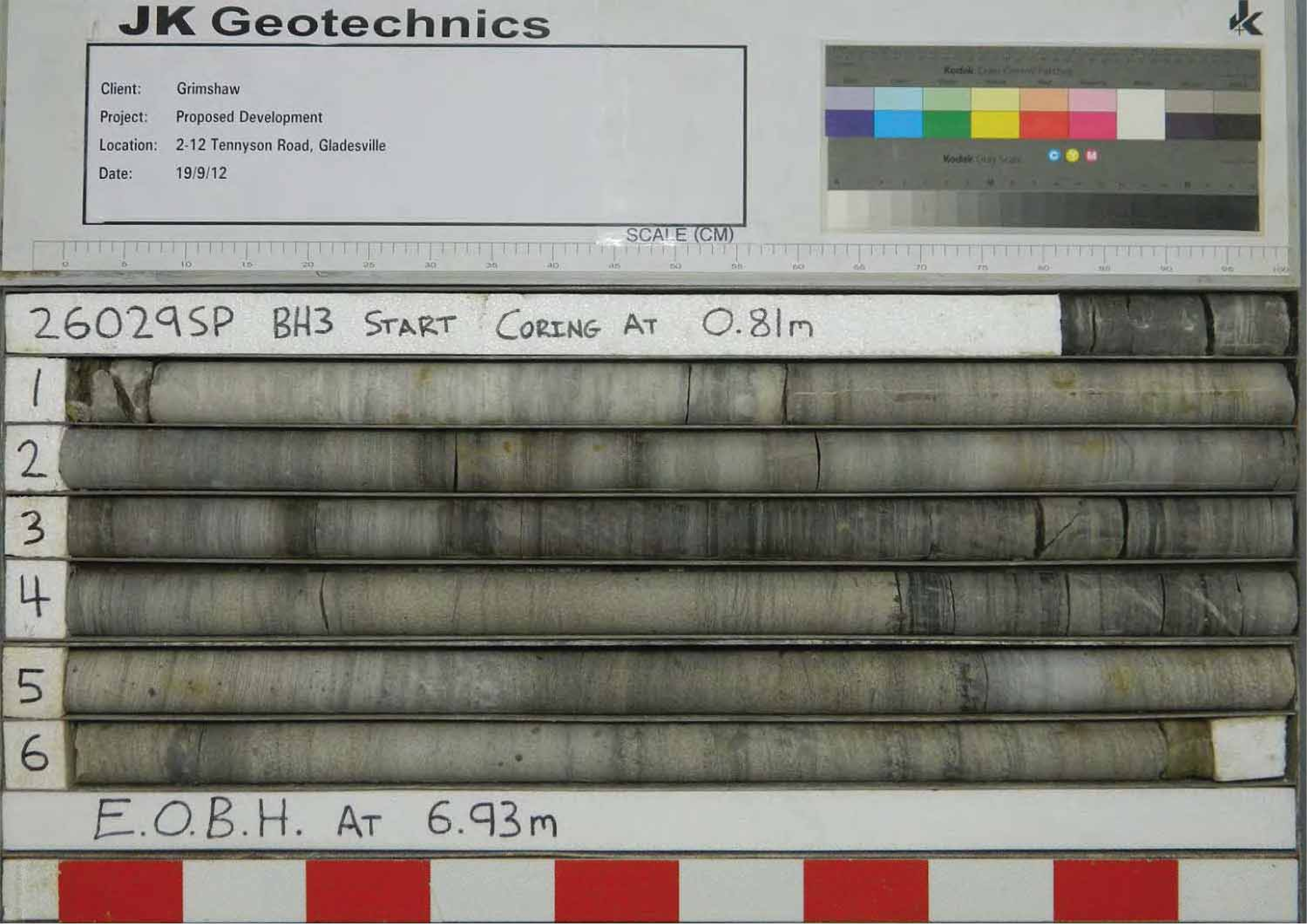
Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 32.7m

Datum: AHD

Logged/Checked by: R.V.C./P.W.

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 > 13 3,13/50mm REFUSAL	0			FILL: Gravelly silty clay, low to medium plasticity, brown and grey, fine to coarse grained shale and ironstone gravel.	MC<PL	(H)	400	
								-	SHALE: dark grey.	DW	M	520 460	
						1 2 3 4 5 6 7			REFER TO CORED BOREHOLE LOG				



JK Geotechnics

Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 19/9/12

CORED BOREHOLE LOG

Borehole No.
3
2/2

Client: Darcsol Ltd PtyProject PRO POSED DEVELOPMENT

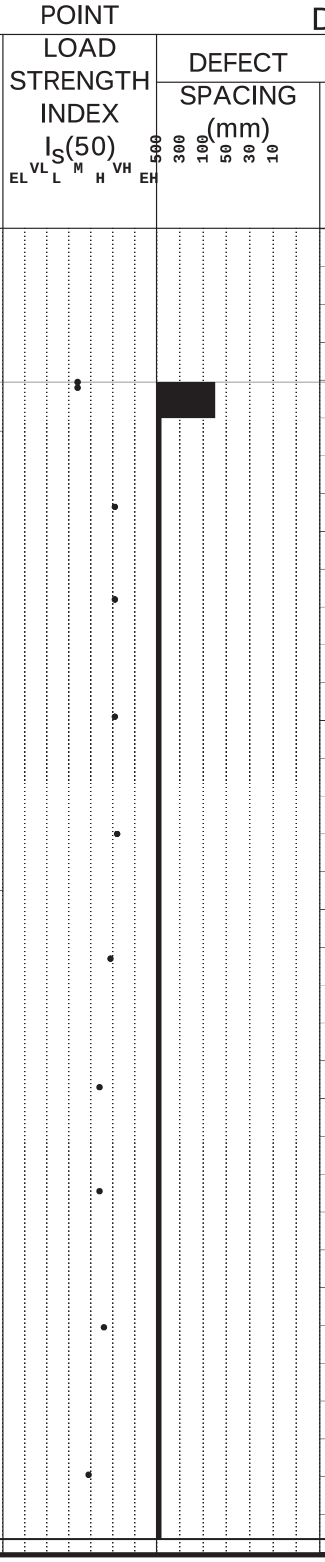
Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP **Core Size:** NMLC **R.L. Surface:** ≈ 32.7m

Date: 19-9-12 **Inclination:** VERTICAL **Datum:** AHD

Drill Type: JK500 **Bearing:** - **Logged/Checked by:** R.V.C./P.W.

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
		0		START CORING AT 0.81m				
		1		SHALE: dark grey, bedded at 0°.	DW	M	- Cr, 34mm.t, IS	
		2		SANDSTONE: fine grained, light grey, bedded at 0-5°.	FR	VH		
FULL RETURN		3 4		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-5°.		H	- Cr, 4mm.t	
				SANDSTONE: fine to medium grained, light grey.				
		5		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-5°.				
		6 7		SANDSTONE: fine to medium grained, light grey, bedded at 5-15°. END OF BOREHOLE AT 6.93m				



JK **Geotechnics**
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP

Date: 20-9-12

Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 32.7m

Datum: AHD

Logged/Checked by: R.V.C./P.W.

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			ASPHALTIC CONCRETE: 60mm.t				
		N = 11 3,4,7			-	FILL: Gravelly silty clay, medium plasticity, dark grey, grey and orange brown, fine to coarse grained crushed shale gravel.	MC>PL		240 210 220	APPEARS MODERATELY COMPACTED
			1		-	SHALE: dark grey.	DW	L-M		LOW 'TC' BIT RESISTANCE MODERATE TO HIGH RESISTANCE
						REFER TO CORED BOREHOLE LOG				
			2							
			3							
			4							
			5							
			6							
			7							

Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 20/9/12

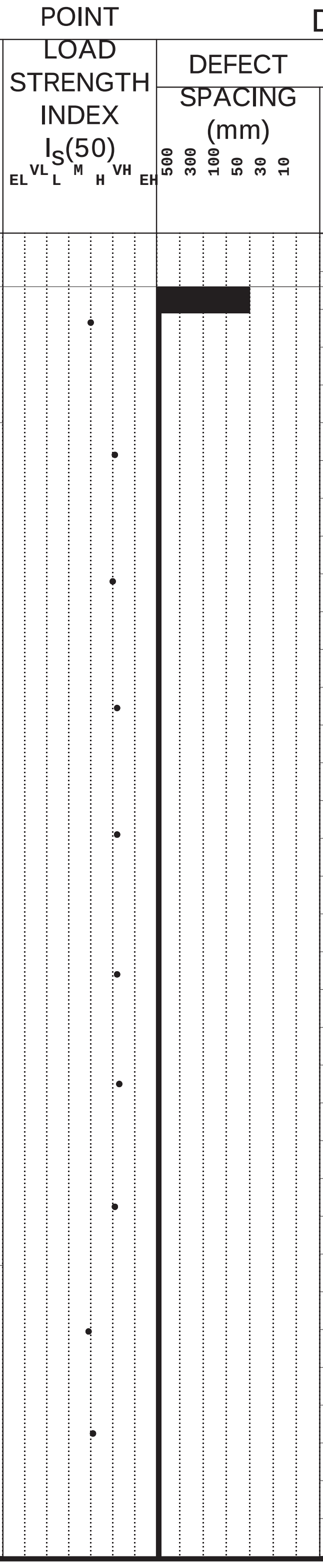
CORED BOREHOLE LOG

Borehole No.
4
2/3

Client: Darcsol Ltd Pty **Project:** PROPOSED DEVELOPMENT
Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP **Core Size:** NMLC **R.L. Surface:** ≈ 32.7m
Date: 20-9-12 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** R.V.C./P.W.

Water Loss/Level	Barrel Lift	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
		1	START CORING AT 1.28m				
FULL RET-URN			SHALE: dark grey.	FR	H	- HIGHLY FRACTURED ZONE, 136mm.t - Cr, 30mm.t DEFECTS NOT INDIVIDUALLY DESCRIBED ARE Be, 0°, P, S	
		2	SANDSTONE: fine grained, light grey, with dark grey laminae.		VH		
		3					
		4	INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-5°.			- Cr, 17mm.t	
		5	SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-5°.				
		6	SANDSTONE: medium grained, light grey, with dark grey laminae, bedded at 0-10°.		H	- Be, 4°, 3mm.t, P, S	
		7	as above, but cross bedded at 15-20°.				
		8					

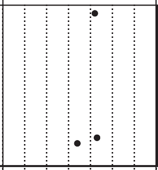


CORED BOREHOLE LOG

Borehole No.
4
3/3

Client: Darcsol Ltd Pty
Project: PROPOSED DEVELOPMENT
Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Date: 20-9-12
Drill Type: JK500
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 32.7m
Datum: AHD
Logged/Checked by: R.V.C./P.W.

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) 600 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				⁝⁝⁝	SANDSTONE: medium grained, light grey, with dark grey laminae, bedded at 0-10°.	FR	H				
		9 10 11 12 13 14			END OF BOREHOLE AT 8.85m						

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BOREHOLE LOG

Borehole No.
5
1/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

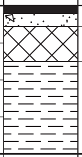
Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

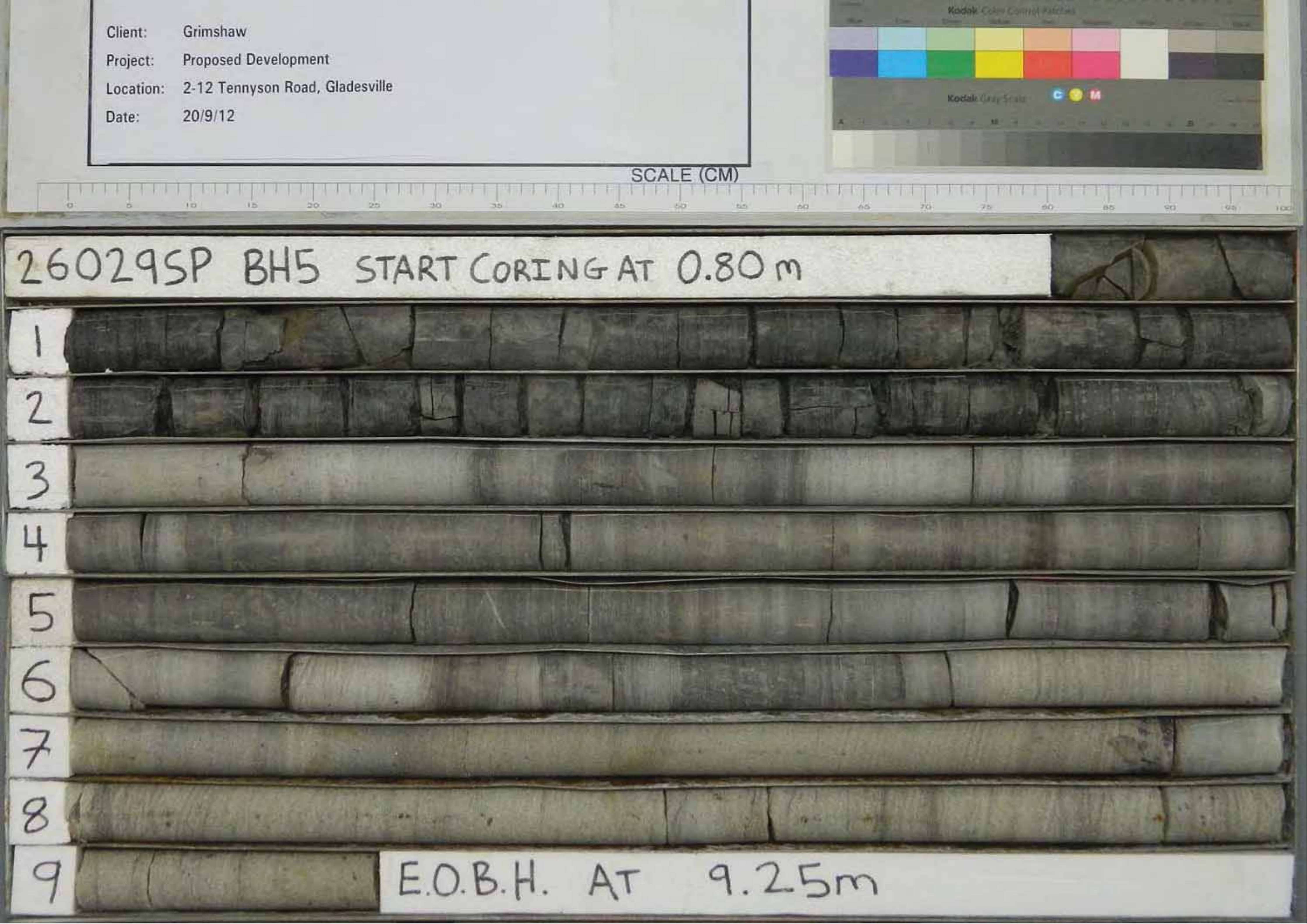
Job No. 26029SP
Date: 20-9-12

Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 32.7m
Datum: AHD

Logged/Checked by: R.V.C./P.W.

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		-	ASPHALTIC CONCRETE: 40mm.t over CONCRETE: 70mm.t				6mm DIA. REINFORCEMENT
								-	FILL: Clayey gravel, fine to coarse grained crushed shale.				
									SHALE: dark grey.	SW	H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						1			REFER TO CORED BOREHOLE LOG				
						2							
						3							
						4							
						5							
						6							
						7							



Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 20/9/12

J K Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
5
2/3

Client: Darcsol Ltd Pty

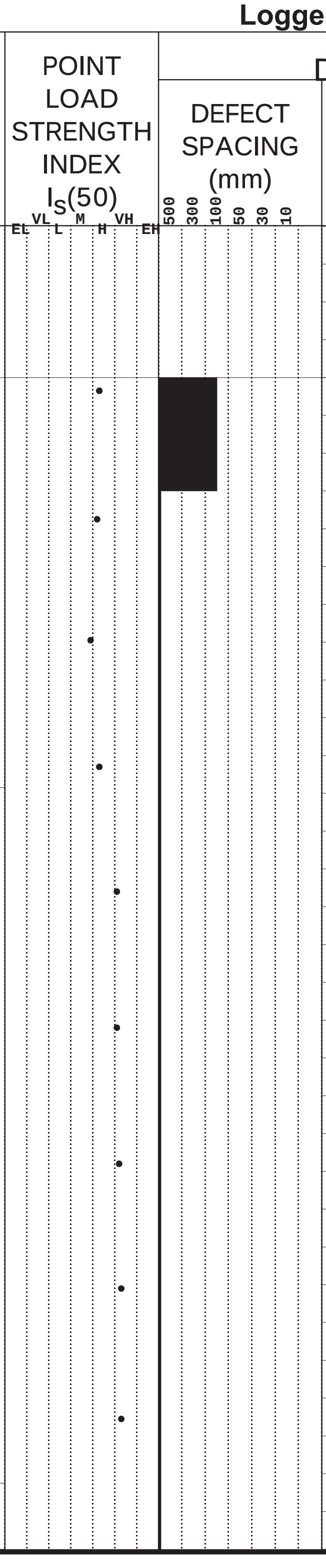
Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Core Size: NMLC
R.L. Surface: ≈ 32.7m

Date: 20-9-12
Inclination: VERTICAL
Datum: AHD

Drill Type: JK500
Bearing: -
Logged/Checked by: R.V.C./P.W.



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
		0		START CORING AT 0.80m				
		1		SHALE: dark grey, bedded at 0°.	FR	H	- J, 60°, P, S, IS - J, 45°, P, S, IS - J, 70°, P, S, IS - J, 80°, P, S, IS	
		2		as above, but with numerous crushed seams between 35mm and 160mm spacing.				
				SHALE: dark grey.				
		3		SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.		VH		
		4						
80% RET-URN		5		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-5°.				
		6		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-5°.				
						H		
		7						

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CORED BOREHOLE LOG

Borehole No.
5
3/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Core Size: NMLC
R.L. Surface: ≈ 32.7m

Date: 20-9-12
Inclination: VERTICAL
Datum: AHD

Drill Type: JK500
Bearing: -
Logged/Checked by: R.V.C./P.W.

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, light grey, with dark grey laminae, bedded at 0-5°	FR	H	●			
		8		SANDSTONE: medium grained, light grey, with dark grey laminae, bedded at 0-10°.			●			
		9					●			
				END OF BOREHOLE AT 9.25m						
		10								
		11								
		12								
		13								
		14								

BOREHOLE LOG

Borehole No.
6
1/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP

Date: 20-9-12

Method: SPIRAL AUGER
JK500

R.L. Surface: ≈ 32.9m

Datum: AHD

Logged/Checked by: R.V.C./P.W.

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						0			ASPHALTIC CONCRETE: 60mm.t				
								-	FILL: Silty sandy gravel, fine to coarse grained crushed shale, dark grey, fine grained sand.	M			
					N = 18 4,7,11				FILL: Gravelly silty clay, medium plasticity, orange brown, grey and dark grey, fine to coarse grained shale and ironstone gravel.	MC<PL		440 550 600	HP READINGS AFFECTED BY GRAVEL
						1		-	SHALE: dark grey.	DW	L-M		LOW 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						2							
						3							
						4							
						5							
						6							
						7							

Client: Grimshaw

Project: Proposed Development

Location: 2-12 Tennyson Road, Gladesville

Date: 20/9/12

CORED BOREHOLE LOG

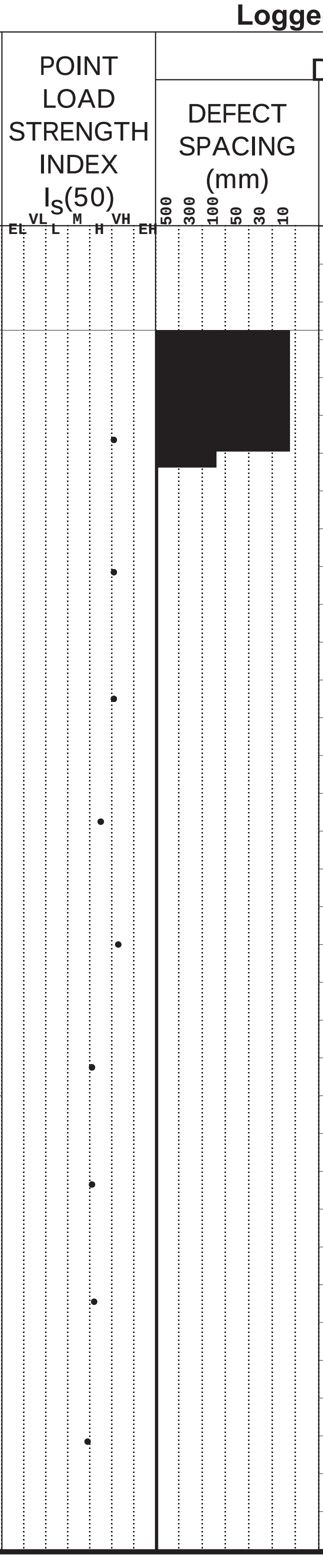
Borehole No.
6
2/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Core Size: NMLC
R.L. Surface: ≈ 32.9m
Date: 20-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK500
Bearing: -
Logged/Checked by: R.V.C./P.W.



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
		1		START CORING AT 1.55m			
		2		SHALE: dark grey and orange brown, highly weathered and fractured.	XW-DW	EL-VL	FRAGMENTED TO 2.15m DEPTH
				SANDSTONE: fine grained, light grey, with orange brown staining.	DW	VH	- J, 70-90°, UN, R, IS
		3		as above, but light grey, with dark grey laminae.	FR		
		4		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-10°.			
		5		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°. Shale band, 0.2m.t.			
		6				H	
		7		SANDSTONE: medium grained, light grey, with dark grey laminae, bedded at 10-15°.			
		8					

CORED BOREHOLE LOG

Borehole No.
6
3/3

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP

Core Size: NMLC

R.L. Surface: ≈ 32.9m

Date: 20-9-12

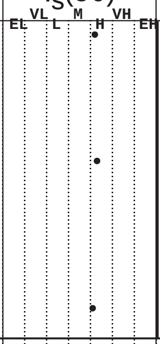
Inclination: VERTICAL

Datum: AHD

Drill Type: JK500

Bearing: -

Logged/Checked by: R.V.C./P.W.



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 $I_S(50)$	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
				SANDSTONE: medium grained, light grey, with dark grey laminae, bedded at 10-15°.	FR	H			
		9							
				END OF BOREHOLE AT 9.68m					
		10							
		11							
		12							
		13							
		14							

BOREHOLE LOG

Borehole No.
7
1/2

Client: Darcsol Ltd Pty

Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP
Date: 24-9-12

Method: SPIRAL AUGER
JK305

R.L. Surface: ≈ 35.1m
Datum: AHD

Logged/Checked by: R.V.C./P.W.

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 AUGER-ING			0			ASPHALTIC CONCRETE: 60mm.t				
						FILL: Gravelly silty sand, fine to medium grained, grey brown, fine to medium grained shale and sandstone gravel.	M MC<PL			APPEARS MODERATELY TO WELL COMPACTED
		N = 16 7,8,8				FILL: Silty clay, medium to high plasticity, grey and orange brown, with fine to coarse grained shale and ironstone gravel.	MC>PL		280 350 280	
			1			as above, but dark brown, grey and red brown.				
		N = 16 6,7,9	2						240 400 310	
							MC<PL			
		N = 26 7,14,12	3						390 290 280	
			4		-	SHALE: dark grey.	DW	L-M		BANDED LOW 'TC' BIT RESISTANCE
			5 6 7			REFER TO CORED BOREHOLE LOG				

JK Geotechnics

Client: Grimshaw
Project: Proposed Development
Location: 2-12 Tennyson Road, Gladesville
Date: 24/9/2012

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CORED BOREHOLE LOG

Borehole No.
7
2/2

Client: Darcsol Ltd Pty

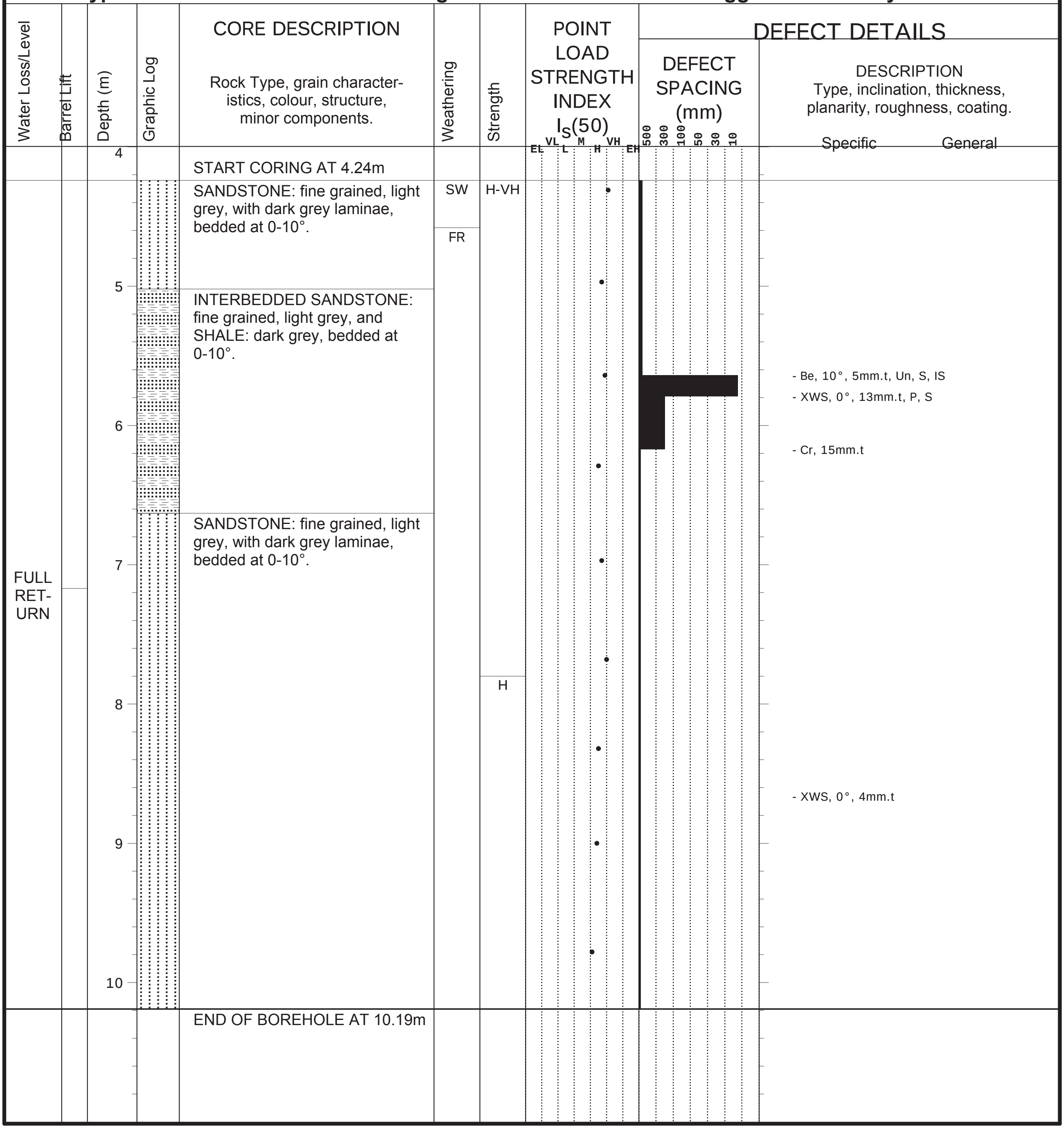
Project: PROPOSED DEVELOPMENT

Location: 2-12 TENNYSON ROAD, GLADESVILLE, NSW

Job No. 26029SP | **Core Size:** NMLC | **R.L. Surface:** ≈ 35.1m

Date: 20-9-12 | **Inclination:** VERTICAL | **Datum:** AHD

Drill Type: JK350 | **Bearing:** - | **Logged/Checked by:** R.V.C./P.W.



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
	4.24	START CORING AT 4.24m			
FULL RET-URN	4.24	SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.	SW FR	H-VH	
	5.0	INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-10°.			- Be, 10°, 5mm.t, Un, S, IS - XWS, 0°, 13mm.t, P, S - Cr, 15mm.t
	6.6	SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.		H	- XWS, 0°, 4mm.t
	10.19	END OF BOREHOLE AT 10.19m			

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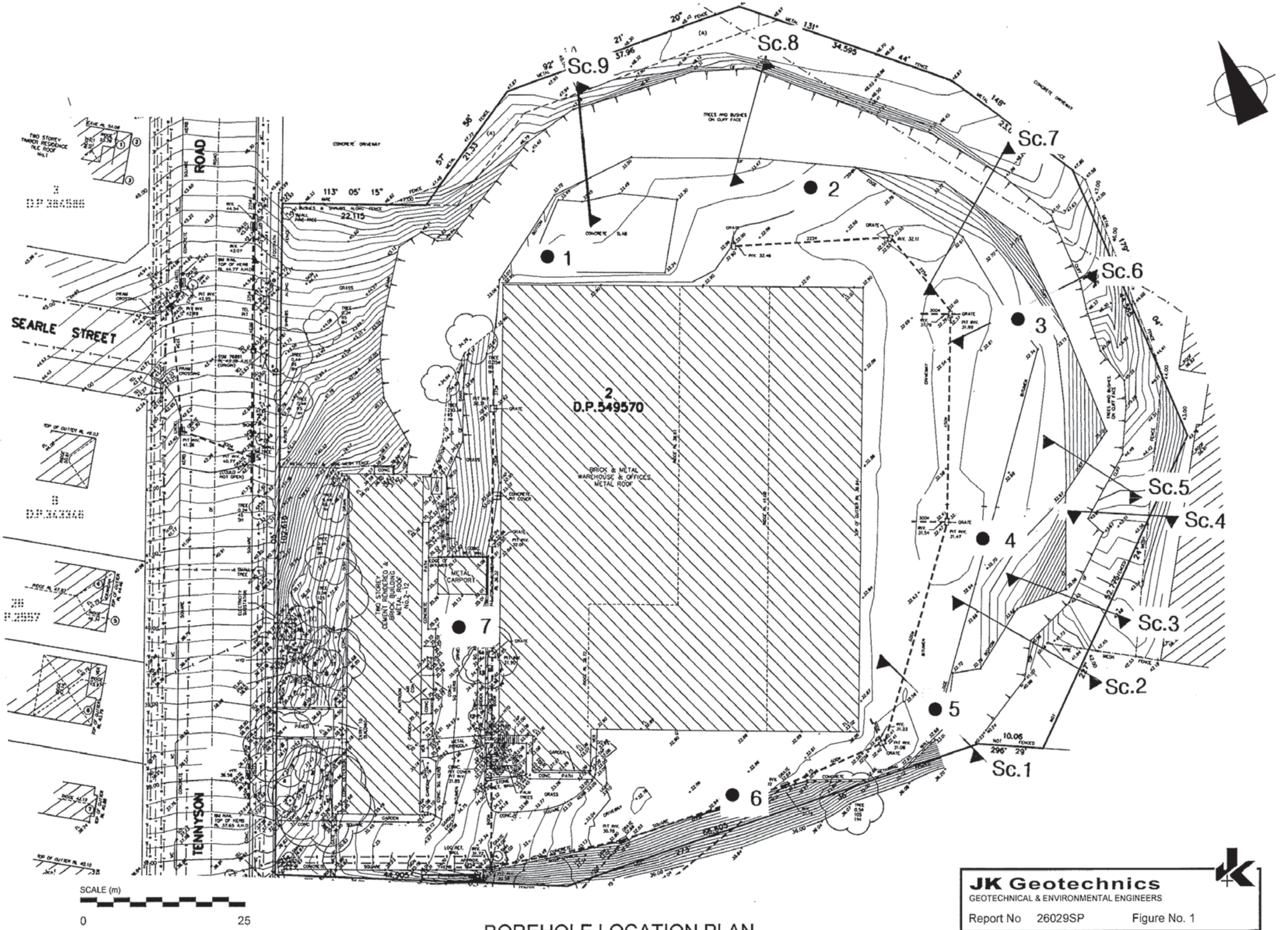

BOREHOLE LOCATION PLAN

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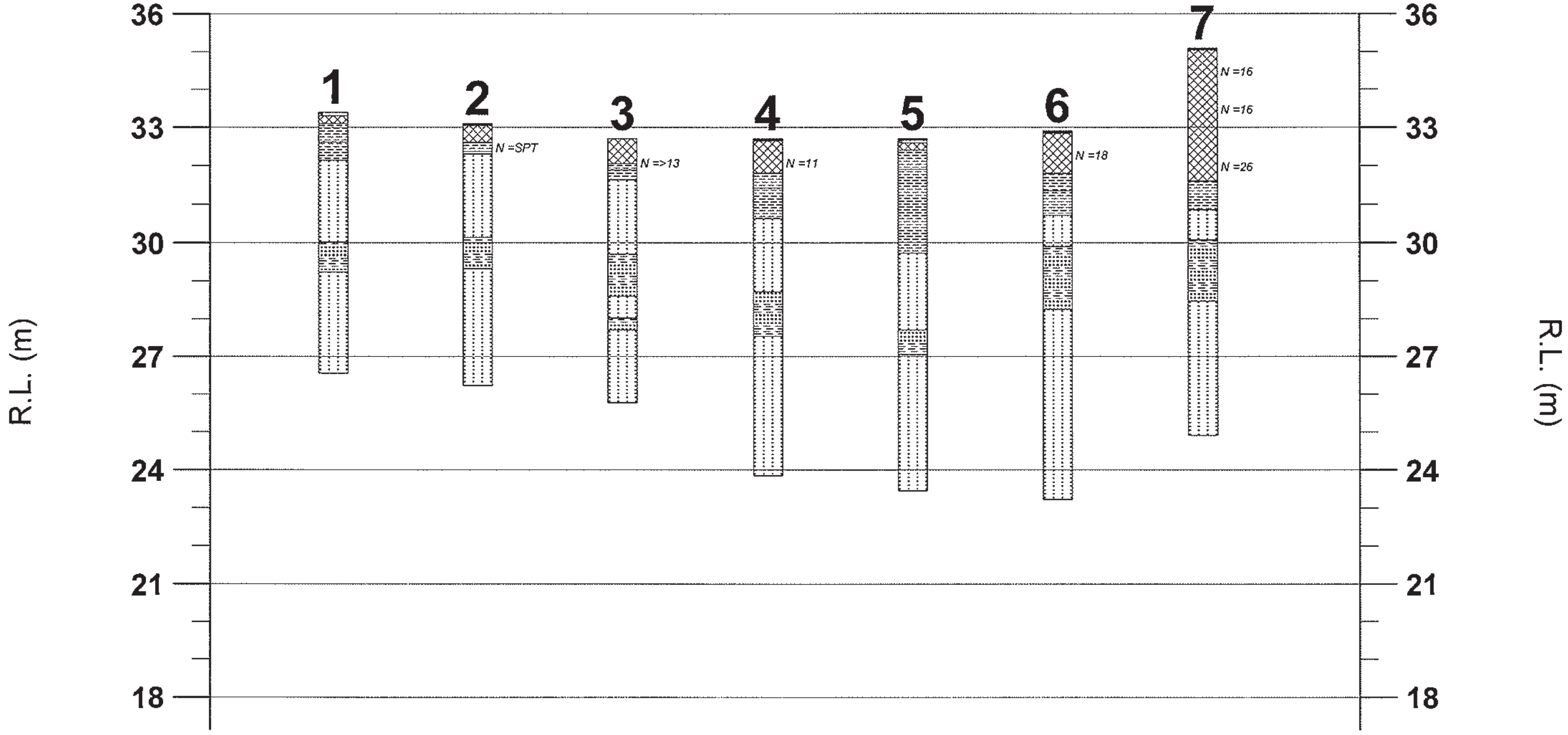
Report No 26029SP

Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



▤	Shale	▩	Concrete	N	SPT "N" VALUE
▥	Sandstone/ Greywacke	▦	Fill	Nc	SOLID CONE BLOW COUNTS PER 150mm
▩	Interbedded Sandstone and shale	■	Asphaltic/ Bituminous Paving or Coal		

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 150 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 26029SP

Figure No.: 2

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed 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.

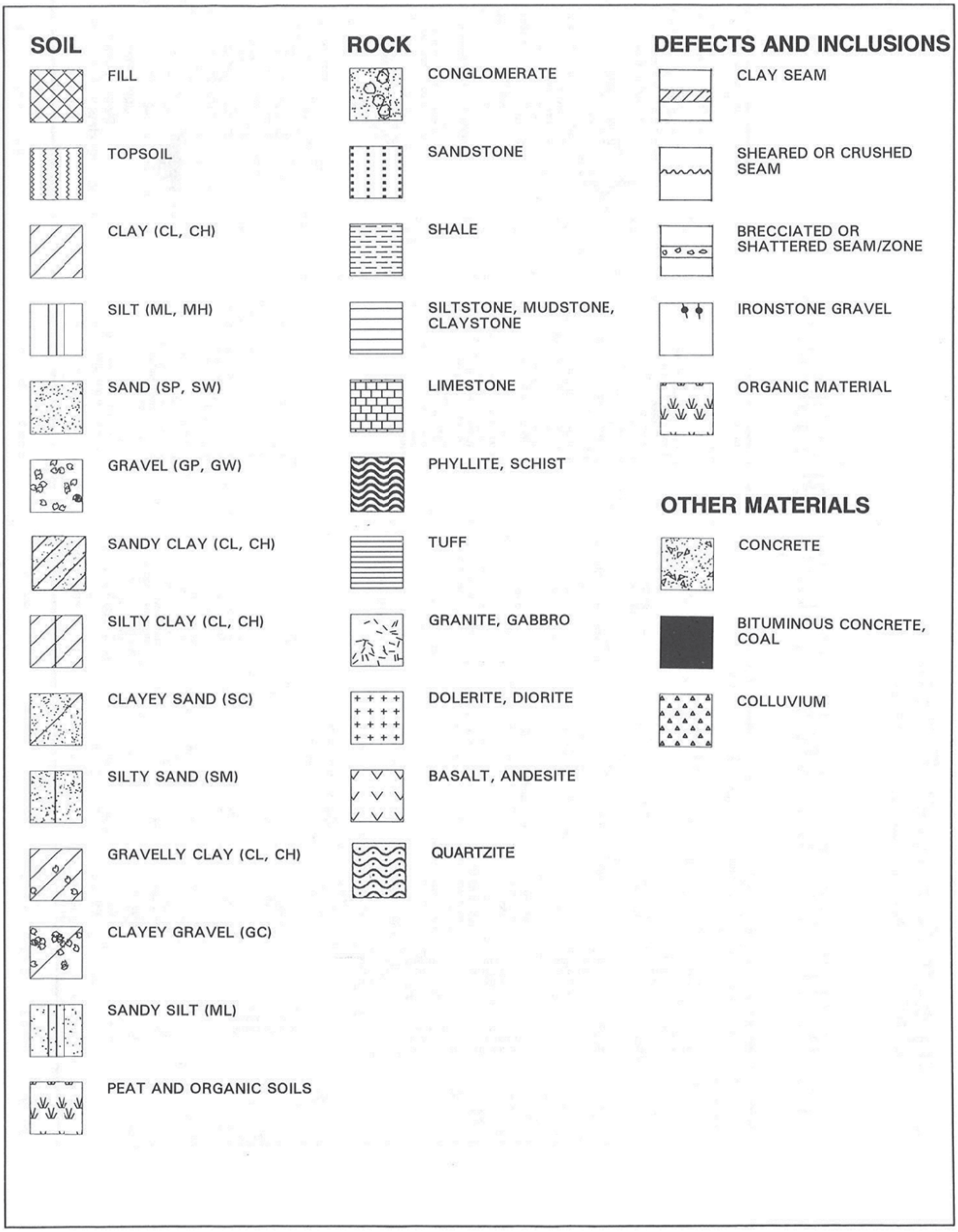
Requirements could range from:

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

- 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

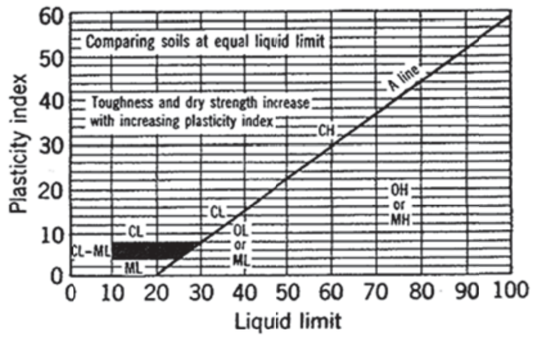
- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

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

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 (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.
	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.

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

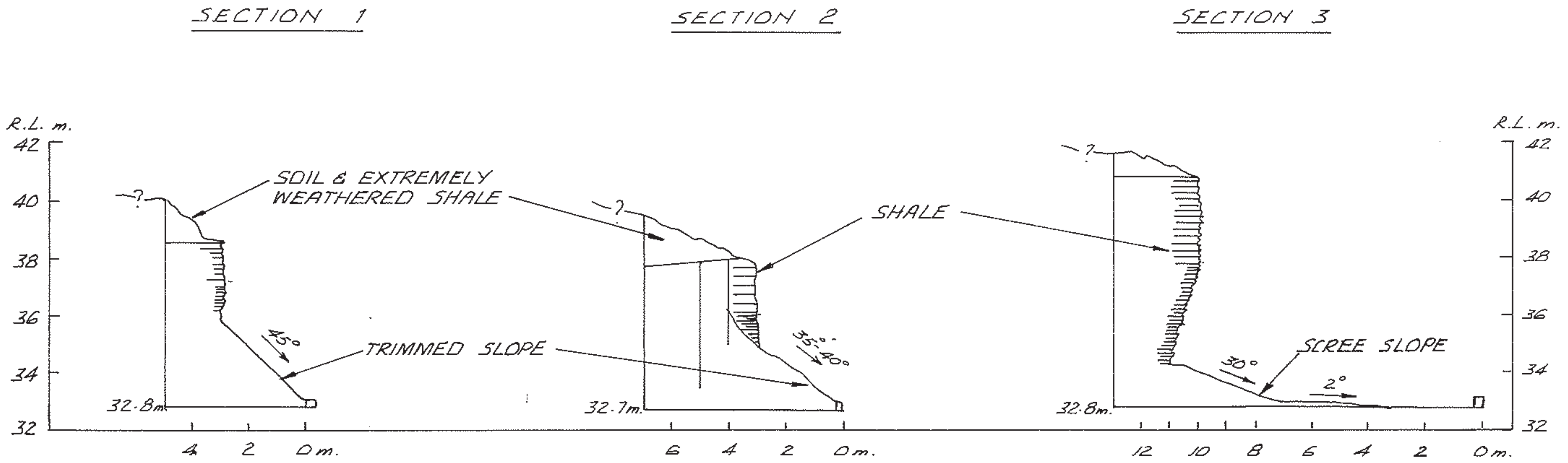
Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		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	

APPENDIX A



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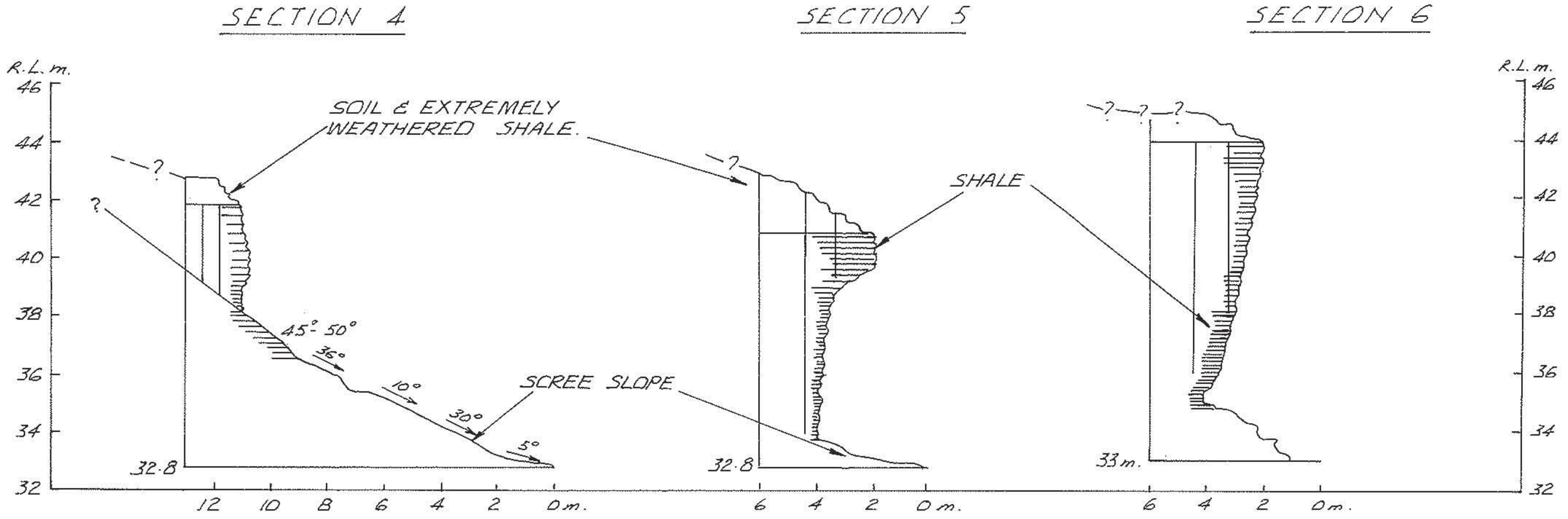
Report No. 7676XS Figure No. 2



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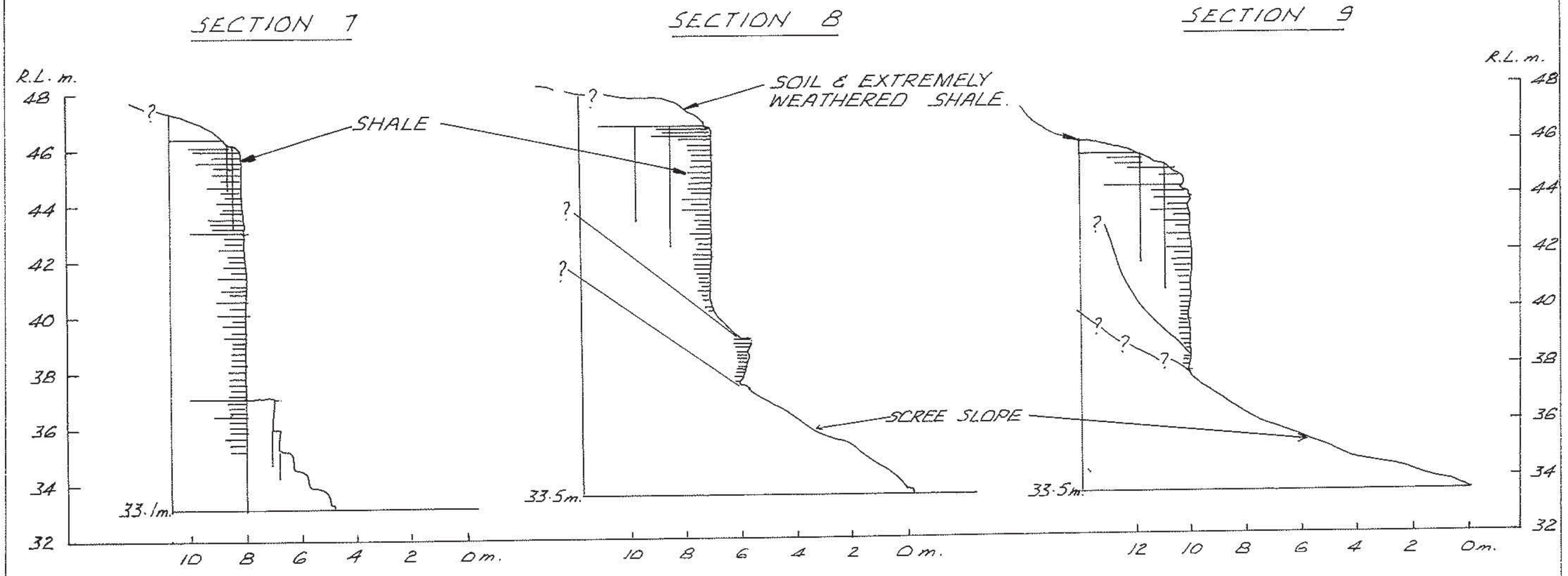
Report No. 7676XS Figure No. 3



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Report No. 7676XS Figure No. 4