REPORT

TO
GLADESVILLE RSL & COMMUNITY CLUB LIMITED
C/- PHILON PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RSL YOUTH CENTRE DEVELOPMENT

AT
6-8 WESTERN CRESCENT, GLADESVILLE, NSW

25 August 2015
Ref: 28618Srpt

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For and on behalf of
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed RSL youth centre redevelopment at 6 to 8 Western Crescent, Gladesville, NSW. The investigation was commissioned by Andrew Collier of Philon Pty Ltd on behalf of the client, Gladesville RSL & Community Club Limited, by signed Acceptance of Proposal form dated 25 July 2015. The investigation was carried out in accordance with our proposal, Ref: P40721Srev1, dated 25 June 2015.

We have been supplied with the following:

- Consultant Brief by Philon Pty Ltd dated June 2015.
- Architectural drawings prepared by Contempo Architects (Job No. P1119, SK-00 to SK-03 and SK-08 to SK-15 dated 23/04/15).
- Survey plan prepared by Linker Surveying (Ref: 150733, Sheets 1 to 3, Issue 1, dated 10/08/15).

We understand that it is proposed to demolish all existing improvements and construct a multi-storey building with five basement levels. The basements levels will result in an excavation of up to 15m depth below existing surface levels. Based on the supplied drawings it appears the basement footprint extends to all boundaries of the site. We haven't been provided with the structural loadings but we expect them to be in the high range for this type of structure.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention systems, footings, basement slabs and soil aggression.

A summary of the principal geotechnical issues, based on the findings of this investigation, is provided in Section 4.1.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 10 and 11 August 2015 and comprised two boreholes drilled with our track mounted JK300 and JK305 drilling rigs. The boreholes were drilled to 3.26m and 2.89m depth below existing surface levels in BH1 and BH2, respectively, using spiral auger techniques and a Tungsten Carbide (‘TC’) bit. These boreholes were then extended to 18.42m and 17.68m in BH1 and BH2, respectively, using an NMLC triple tube barrel fitted with a diamond coring bit and water flush.

Prior to commencement of the fieldwork the investigation locations were electromagnetically scanned by a specialist subcontractor so that all borehole locations could be located clear of buried services.

The strength of the subsurface soils was assessed from Standard Penetration Test (SPT) ‘N’ values augmented by hand penetrometer tests on the SPT split tube samples. The strength of the sandstone bedrock was assessed by observation of the auger penetration resistance using a tungsten carbide ‘TC’ drill bit, together with examination of the recovered rock cuttings. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Where bedrock was diamond cored, the recovered core was logged in the field and then returned to our NATA registered laboratory (Soil Test Services (STS)) for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations the Unconfined Compressive Strengths (UCS) of the bedrock was then calculated from the Is_{50} results and are presented in the attached Table A and plotted on the borehole logs. Copies of the colour photographs are provided with the borehole logs. Selected samples were tested by Envirolab Services, a NATA registered laboratory, to determine pH, sulphate and chloride contents of the soil and the results are in the attached Envirolab Report 132556.

Groundwater observations were made in the boreholes during and on completion of drilling, at the end of the field work and two weeks later on 24 August 2015 in the standpipe in BH1. We note that water is introduced into the borehole during coring and therefore the water levels measured at completion of coring may be artificially high as the water levels have not had time to stabilise. In BH1 a hand slotted PVC standpipe was installed to a depth of 18m and finished with a cast iron gatic cover to allow longer term groundwater monitoring to be completed.

The fieldwork was completed in the full-time presence of our geotechnical engineer who set out the borehole locations, nominated the testing and sampling, and prepared the attached borehole logs. The borehole locations are shown on the attached Figure 1, and these were set out by taped measurements from assumed site boundaries as shown on survey plans by Linker Surveying. The surface reduced levels (RLs) shown on the borehole logs were estimated by interpolation between spot levels shown on the supplied survey plan and are therefore only approximate and are based on the Australian Height Datum (AHD).

For more details of the investigation procedures and their limitations, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site lies in gently undulating topography with slopes up to 6° observed. The site itself appears to be mid-slope of a southward facing hill with slopes of approximately 5°. Site north is taken to be the boundary with Western Crescent.

The site contains two linked one to two storey brick buildings that abut parts of the site's northern, southern and eastern boundaries. There is a metal shed in the south-east corner. The brick buildings appear to be in good condition with no visible defects based upon a cursory external inspection. The site steps down from the northern boundary via concrete steps and brick retaining walls. The walls appear in moderate condition with vertical cracking up to 4mm wide observed. The site is retained in its south-west corner by a brick retaining wall up to 0.9m high above the street level. The wall appears in moderate condition with cracking up to 4mm wide observed. All external areas are paved with a mixture of asphaltic concrete (AC) and concrete pavements which generally appear in poor to moderate condition. The AC pavement contains extensive cracking and potholes up to 100mm wide. Cracking up to 3mm wide was observed in the concrete pavement.

The site has northern, western and southern street frontages onto Western Crescent, Ross Street and Coulter Street, respectively. The streets have AC pavements that generally appear in moderate condition based upon a cursory inspection from the roadside. The road defects predominantly comprise of crocodile cracking up to 10mm wide but typically no more than 4mm wide. Western Crescent and Coulter Street have a slight south-westerly slope of no more than 2° whilst Ross Street is steeper, sloping down towards the south at approximately 5°.

An AC paved Council car park abuts the site on its eastern boundary with the car park pavement generally appearing to be in moderate condition with cracking up to 3mm wide observed. The car park is up to 2m higher than the ground level within the subject site and it is supported by a brick retaining wall that appears in good condition with no visible defects based upon a cursory inspection.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by Ashfield Shale of the Wianamatta Group comprising black to dark grey shale and laminite. It should also be noted that the site lies approximately 180m east from the Hawkesbury Sandstone boundary. This subsurface profile does not take into account the soils derived from insitu weathering of the bedrock or earthworks (e.g. cutting and filling) that have previously been undertaken at the site.

The boreholes disclosed a subsurface profile generally comprising pavement and fill overlying natural silty clays of high plasticity and sandstone which was generally initially of extremely low to very low strength becoming medium to high strength at depths of 3m to 5m. Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations, a summary is presented below:

Pavement and Fill

Asphaltic concrete pavement of 10mm thickness was encountered at the surface in both boreholes.

Fill was encountered beneath the pavement and initially comprised a fine to medium grained gravelly sand/sandy gravel material that was moist. In BH2 the fill material contained trace amounts of clay. In BH1 beneath the sandy fill layer, a silty clayey fill assessed as being medium plasticity was encountered and appearing to be moderately compacted. The clay had a moisture condition greater than the plastic limit and contained varying amounts of fine to medium grained ironstone gravel and ash.

Natural Clays

Natural residual clays were encountered beneath the fill in both boreholes at 1.5m (RL~38.0m) in BH1 and 0.25m (RL~39.5m) in BH2. The natural clays were assessed as being high plasticity and of very stiff to hard strength based upon hand penetrometer readings. The clays contained trace amounts of fine to medium grained ironstone gravel.

Bedrock

Sandstone bedrock was encountered at 2.5m (RL~37.0m) and 1.1m (RL~38.6m) depth below existing surface levels in BH1 and BH2, respectively.

In BH1 the sandstone was initially distinctly weathered and of very low to low strength before quickly improving to slightly weathered medium to high strength sandstone at 3.0m (RL~36.5m). The medium to high strength sandstone extended to 4.89m (RL~34.6m) depth before improving into fresh, high strength sandstone. It should be noted that a 0.23m thick band of core loss was encountered at 8.51m depth which is usually indicative of an extremely weathered, extremely low strength sandstone or clay band.

The sandstone encountered in BH2 was initially extremely to distinctly weathered and of extremely low to very low strength sandstone. This poorer quality rock extended to 2.2m (RL~37.5m) depth before improving to distinctly weathered very low to low strength sandstone. The good quality fresh high strength sandstone was encountered at 4.81m (RL~34.9m) depth and extended to the borehole termination depth.

Groundwater

The boreholes did not encounter any groundwater seepage and were dry on completion of augering. The groundwater was measured up to 24 hours later at 2.8m (RL~36.7m) depth below existing surface levels in the standpipe installed in BH1. Two weeks later, the level in the standpipe was measured at 3.05m depth (RL~36.5m).

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on recovered rock core samples correlated well with our field assessment of bedrock strength. The estimated UCS’s ranged between 1MPa and 42MPa.

The soil pH test results were 5.4 and 5.2 in BH1 and BH2, respectively, indicating the samples tested to be slightly acidic. The soil sulphate and chloride tests results returned sulphate contents of 230mg/kg and 89mg/kg and chloride contents of 45mg/kg and less than 10mg/kg in BH1 and BH2, respectively.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings, Issues and Further Work

As discussed in more detail in Section 3.2, the boreholes penetrated a cover of pavement and fill overlying residual clays grading into weathered sandstone bedrock at depths of 2.5m and 1.1m in BH1 and BH2, respectively. The sandstone improved with medium to high strength sandstone encountered at 3.0m and 4.81m in BH1 and BH2, respectively. No groundwater seepage was encountered during drilling but the groundwater level was measured at 2.8m in BH1 up to 24 hours after borehole completion and at 3.05m depth up to two weeks later.

Based on the results of the two boreholes and our understanding of the proposed development (refer to Section 1.1), we have summarised the principal geotechnical findings, issues and recommendations to be considered in the planning, design and construction of the development:

1. Prior to demolition or excavation we recommend detailed dilapidation surveys be completed on any neighbouring structures that fall within in the zone of influence, taken as twice the excavation depth from the common boundary.
2. Excavation for the proposed basement will be through pavement, fill, residual clays and predominantly sandstone bedrock of medium to high strength. Excavation of the sandstone will require the use of “hard rock” excavation equipment for effective excavation, which may transmit vibrations through the rock mass that could affect adjoining infrastructure.
3. The retention systems may comprise a soldier pile wall with shotcrete infill panels socketed no less than 0.3m into medium to high strength sandstone and installed prior to the start of bulk excavations. Lateral support for the piles comprising temporary ground anchors will be required. The medium to high strength sandstone may then be excavated vertically provided regular geotechnical inspections are carried out.
4. At this stage, groundwater seepage into the excavation is unlikely to be a significant issue for the proposed basement as we would expect seepage rates to be manageable using conventional sump and pump or gravity methods due to the low permeability of the soils and bedrock.
5. Pad footings founded within the sandstone bedrock at the base of the excavation can be used to support the proposed building. Depending on the allowable bearing pressure chosen for the footings, geotechnical inspections of the footing excavations will have to be scheduled.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report.

4.2 Further Geotechnical Work

We recommend that following demolition of the existing building that at least two more cored borehole be drilled towards the eastern side of the site to confirm the subsurface conditions encountered in BH1 and BH2 extend uniformly across the site.

We summarise below the recommended additional geotechnical input that needs to be carried out:

- Dilapidation surveys for the neighbouring structures, especially as rock hammers are almost certain to be used.
- Monitoring of groundwater seepage into the excavation to confirm drainage requirements.
- At least initial Vibration Monitoring during bulk excavation.
- Progressive inspection of excavated cut faces to confirm if additional support or treatment is required.
- Witnessing installation and proof testing of anchors
- Footing inspections and testing

We also recommend a review of proposed earthworks and structural drawings and section details (once available) in order to confirm that these follow the guidelines of this report.

4.3 Excavation Conditions

All excavation recommendations should be complemented by reference to Safe Work Australia's 'Excavation Work Code of Practice', dated July 2014.

4.3.1 Dilapidation Surveys

Prior to the commencement of excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the north, east and south of the site for a distance from the common boundaries of at least twice the excavation depth. As the excavation is around 15m deep this zone of influence extends about 30m. As the nearest structures are about 20m distant, a reduction in the detail of the dilapidation reports could be considered.

The dilapidation surveys should include internal and external inspections of the buildings, where all defects including defect location, type, length and width are described and photographed.

The respective owners of the neighbouring buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.3.2 Excavation Methods

We expect an excavation depth of up to 15m which will encounter pavements, fill, natural clays and sandstone bedrock. Excavation of the soils and up to very low strength sandstone will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Excavation of the sandstone will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping hooks and rock saws. Sandstone of high strength will require excavation and will represent 'hard rock' excavation conditions. The excavator contractor should be made aware of this by being supplied with all geotechnical information; low productivity and increased equipment wear should be expected due to the rock strength.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there could be direct transmission of ground vibrations to nearby structures and buried services. We recommend that initial quantitative vibration monitoring be carried when out using hydraulic rock hammers to determine if the transmitted vibrations are within an acceptable limit for the nearby structures and services. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations. Where the transmitted vibrations are excessive, it would be necessary to change to alternative excavation methods, such as smaller rock hammers, rotary grinders, ripping hooks or rock saws.

The following procedures are recommended to reduce vibrations if rock hammers are used and transmitted vibrations are found to be excessive:

- Rock saw the perimeter faces. This will effectively reduce ground borne vibrations provided the base of the rock saw slot is maintained at a lower level than the adjacent excavation level at all times. Rock sawing would also improve the stability and aesthetics of the completed rock cut faces and is strongly recommended.
- Maintain rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate the rock hammer in short bursts only, to reduce amplification of vibrations.

- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances.

We recommend that a copy of this report be provided to the excavation contractor so that they can make their own assessment of excavation conditions.

4.3.3 Seepage

Groundwater was not encountered within the boreholes during auger drilling but groundwater levels were measured at about 3m (RL~36.5m) depth in BH1. This level is considered too high to be the groundwater table and is likely related to perched water or seepage. Nevertheless, allowance should be made for groundwater seepage into the excavation, which would tend to occur along the soil/rock interface and through joints within the sandstone, particularly during and following rainfall.

We expect that during construction the seepage would be able to be controlled using conventional sump and pump techniques. In the long term, drainage should be provided around the basement perimeter and below the lowest basement slab to direct seepage into sumps with automatic pumps to remove water from the basement. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage is sufficient for the actual seepage flows. If rock faces are exposed in the basement then access to clean out dish drains should not be a problem. We caution against placing dry walls in front of the rock face as removal of the debris that inevitably frets from the rock faces into drains becomes problematic.

4.4 Retention Systems

Excavations within the soils and up to low strength sandstone will not be self-supporting and retention systems will need to be installed prior to the start of excavation. A suitable retention system to support the cut would be a soldier pile wall with shotcrete infill panels. The pile wall can be founded with sufficient embedment below bulk excavation level to satisfy stability and founding considerations. However, as most of the excavation will be in good quality rock the sockets would be difficult and expensive to construct. The piles can instead be terminated no less than 0.3m in at least medium strength or stronger sandstone bedrock above bulk excavation level, though we recommend that permanent structural loads on the piles be minimised and transferred instead to below bulk excavation level. Toe restraint for the piles may be achieved by using an additional row of temporary anchors or toe bolts, which must be installed prior to excavating in front of the pile toe. A vertical face in the medium strength or stronger sandstone may be excavated below the toe of the piles, but must not undermine the pile toes. Sawing of the cut faces is considered essential below the pile toes.

The sandstone bedrock at this site, including the sandstone below the toe of the soldier piles can be cut vertically, but must be progressively inspected by a geotechnical engineer at no more than 1.5m depth increments to assess the need for temporary support (e.g. rock bolts, dowels, shotcrete etc.) of potentially unstable rock wedges or extremely weathered bands. Although the limited borehole information shows relatively few defects in the sandstone, we expect some stabilisation measures, particularly treatment of extremely weathered bands will be required. A provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures including rock bolts.

We also recommend that for the initial stages of excavation in front of the pile toes, the excavation extends to a depth of not more than 1.5m below the pile toes and not closer than 1.5m from the face of the pile, so that a geotechnical engineer can check for the possible presence of any adverse defects or weathered zones of rock which may destabilise the pile. An allowance should be made for temporary rock bolts below the pile toes to provide lateral restraint for the rock in these sensitive areas.

Retaining Wall Design Parameters

Propped or anchored soldier pile walls may be designed based on a trapezoidal earth pressure distribution of 6H kPa, where H is the retained height of soils and extremely low to very low strength sandstone. All surcharge loads, i.e. adjoining buildings, traffic, sloping backfill, etc., should be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the walls.

Anchors should have their bond formed within sandstone of at least medium strength and may be provisionally designed based on an allowable bond stress of 400kPa. The anchor bond should be formed outside a line drawn up at 45° from the bulk excavation level, with a minimum free length of 3m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load.

Passive toe resistance of the retention system below the base of the bulk excavation, where piles extend below the base of the excavation, may be estimated based on a maximum allowable lateral resistance of 400kPa for sandstone of medium or higher strength. The passive resistance should be ignored to at least 0.3m below the base of the excavation, including footing and service excavations. Where the retention piles are to accommodate building loads we recommend that these piles be founded below the basement the bulk excavation.

Due to the presence of medium and high strength bedrock, only high torque drilling rigs equipped with rock augers should be brought to this site. We strongly recommend that a full copy of this report be provided to the prospective piling contractors.

If temporary anchors are to run below neighbouring properties, then permission from the owners must be obtained prior to installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping. We assume that permanent support of the shoring system will be provided by bracing from the proposed building.

4.5 Footings

Based on the limited results of the investigation, it appears sandstone of high strength will be encountered at the bulk excavation level. Following completion of additional cored boreholes it appears pad footings founded within the sandstone would be appropriate and the sandstone of high strength would be suitable for the preliminary design bearing pressure of 3500kPa.

We recommend that all footing excavations be inspected by a geotechnical engineer, with spoon testing carried out within at least one third of the footing locations. The extent of this spoon testing should be confirmed once further cored boreholes are drilled as it may increase if significant weak seams are encountered in the boreholes.

Depending on the results of the additional cored boreholes, allowable bearing pressures of 6000kPa (or even more) may be used within the high strength sandstone. If such a bearing pressure was to be considered, there may need to be more boreholes and an increase in spoon testing would be required, likely to comprise spoon testing at all footing locations. Again, higher bearing pressures and the extent of construction inspection should be confirmed following the drilling of additional cored boreholes.

4.6 Basement Slab

Based on the investigation results, the proposed basement floor slab will directly overlie sandstone bedrock. We therefore recommend that underfloor drainage blanket be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as ‘blue metal’ gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

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

4.7 Soil Aggression

Based on the laboratory soil chemistry results and in accordance with Table 6.4.2(C) of AS2159-2009 (“Piling – Design and Installation”), the site has a ‘mild’ exposure classification for concrete piles.

5 GENERAL COMMENTS

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of 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
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed RSL Youth Centre
Location: 6-8 Western Crescent, Gladesville, NSW

Ref No: 28618S
Report: A
Report Date: 19/08/2015
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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	3.62-3.66	1.5	30
	3.89-3.93	1.0	20
	4.35-4.39	1.1	22
	4.86-4.89	1.0	20
	5.18-5.23	1.2	24
	5.74-5.78	1.5	30
	6.00-6.03	1.5	30
	6.64-6.67	1.1	22
	7.10-7.14	1.2	24
	7.59-7.62	1.4	28
	8.17-8.22	1.5	30
	8.87-8.91	1.6	32
	9.25-9.29	1.6	32
	9.75-9.79	1.8	36
	10.11-10.15	1.6	32
	10.64-10.68	2.1	42
	11.00-11.04	1.4	28
	11.67-11.71	1.3	26
	12.03-12.08	1.5	30
	12.65-12.68	1.2	24
	13.17-13.21	1.2	24
	13.78-13.82	1.4	28
	14.24-14.28	1.2	24
	14.65-14.69	1.2	24
	15.00-15.04	1.0	20

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	15.88-15.92	1.5	30
	16.30-16.34	1.6	32
	16.77-16.81	1.6	32
	17.48-17.53	1.2	24
	17.88-17.91	1.2	24
	18.22-18.25	1.0	20
2	3.09-3.12	0.06	1
	3.91-3.95	0.1	2
	4.27-4.31	0.3	6
	4.83-4.87	1.4	28
	5.15-5.20	1.7	34
	5.85-5.89	1.7	34
	6.22-6.26	1.8	36
	6.66-6.69	1.1	22
	7.00-7.05	1.5	30
	7.74-7.76	1.1	22
	8.30-8.33	1.7	34
	8.85-8.90	1.1	22
	9.39-9.42	1.4	28
	9.81-9.85	1.5	30
	10.19-10.23	1.7	34
	10.70-10.73	1.7	34
	11.22-11.27	1.8	36
	11.70-11.74	1.4	28
	12.28-12.32	1.5	30

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BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	12.85-12.88	1.4	28
	13.35-13.39	1.6	32
	13.96-14.00	1.6	32
	14.22-14.26	1.7	34
	14.74-14.78	1.8	36
	15.32-15.36	1.3	26
	15.96-15.99	1.4	28
	16.30-16.34	1.7	34
	16.76-16.80	1.5	30
	17.15-17.18	1.5	30
	17.59-17.63	1.5	30

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: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. 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   132556

Client:

JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Owen Fraser

Sample log in details:

Your Reference:	**28618S**	
No. of samples:	2 Soils	
Date samples received / completed instructions received	12/8/2015	/ 12/8/2015

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:	19/08/15	/ 17/08/15
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:

Jacinta Hurst
Laboratory Manager

NATA
ACCREDITED FOR
TECHNICAL
COMPETENCE

Misc Inorg - Soil			
Our Reference:	UNITS	132556-1	132556-2
Your Reference	-------------	BH1	BH2
Depth	------------	1.5-1.95	0.5-0.95
Date Sampled		10/08/2015	11/08/2015
Type of sample		Soil	Soil
Date prepared	-	13/08/2015	13/08/2015
Date analysed	-	14/08/2015	14/08/2015
pH 1:5 soil:water	pH Units	5.4	5.2
Chloride, Cl 1:5 soil:water	mg/kg	45	<10
Sulphate, SO4 1:5 soil:water	mg/kg	230	89

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			13/08/2015	[NT]	[NT]	LCS-1	13/08/2015
Date analysed	-			14/08/2015	[NT]	[NT]	LCS-1	14/08/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	100%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	120%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	112%

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
NA: Test not required
<: Less than
PQL: Practical Quantitation Limit
RPD: Relative Percent Difference
>: Greater than
NT: Not tested
NA: Test not required
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 batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

BOREHOLE LOG

Borehole No.
1
1/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Date: 10-8-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: O.F./P.S.
R.L. Surface: ≈ 39.5m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING	■					0		-	ASPHALTIC CONCRETE: 10mm.t	D			
									FILL: Gravelly sand, fine to medium grained, grey brown, fine to coarse grained igneous gravel.				
	■			■	N = 8 3,3,5	1			FILL: Silty clay, medium plasticity, grey brown, brown and red brown, trace of fine to medium grained ironstone gravel and ash.	MC>PL		70 120 180	POSSIBLY NATURAL APPEARS MODERATELY COMPACTED
	■			■	N = 12 4,6,6	2		CH	SILTY CLAY: high plasticity, red brown and light grey, trace of fine to medium grained ironstone gravel.	MC≈PL	VSt -H	350 390 400	RESIDUAL
AFTER 24 HRS ▼ ▼ ON 24-8-15				■ ■		3		-	SANDSTONE: fine to medium grained, grey brown and red brown, with XW seams, trace of iron indurated bands.	DW	VL-L		VERY LOW 'TC' BIT RESISTANCE
									SANDSTONE: fine to coarse grained, brown and red brown.	SW	M-H		MODERATE TO HIGH RESISTANCE
						4 5 6 7			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
1
2/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Date: 10-8-15
Drill Type: JK305
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 39.5m
Datum: AHD
Logged/Checked by: O.F./P.S.

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 DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		3		START CORING AT 3.26m						
				SANDSTONE: fine to medium grained, orange brown.	SW	M-H				
ON COMPLETION OF CORING		4		as above, but light grey and orange brown, trace of grey laminae, bedded at 0-5°.					- Be, 0°, P, R, IS	
		5		SANDSTONE: fine to medium grained, light grey and dark grey, trace of grey laminae, bedded at 0-10°.	FR	H				
		6							- CS, 0°, 10mm.t	
FULL RETURN		7								
		8							- J, 60°, P, R	
				CORE LOSS 0.23m						
		9		SANDSTONE: fine to coarse grained, light grey, trace of grey and dark grey laminae, bedded at 0-10°.	FR	H			- CS, 0°, 5mm.t	

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CORED BOREHOLE LOG

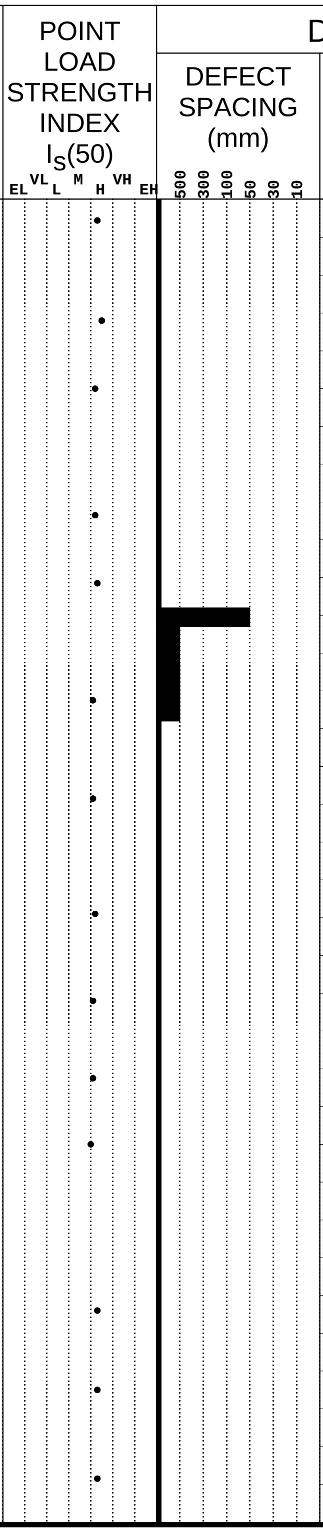
Borehole No.
1
3/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD

Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT

Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Core Size: NMLC
R.L. Surface: ≈ 39.5m
Date: 10-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: O.F./P.S.



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				SANDSTONE: fine to coarse grained, light grey, trace of grey and dark grey laminae, bedded at 0-10°.	FR	H		
							- Be, 10°, P, R	
		11						
		12		as above, but dark dark laminae more frequent.				
							- XWS, 5°, 2mm.t - XWS, 0°, 4mm.t - XWS, 0°, 2mm.t	
							- Be, 0°, P, R	
		13						
FULL RET- - URN								
		14					- CS, 10°, 2mm.t - CS, 0°, 1mm.t	
		15						
		16						

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CORED BOREHOLE LOG

Borehole No.
1
4/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Core Size: NMLC
R.L. Surface: ≈ 39.5m
Date: 10-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: O.F./P.S.

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 DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RET-URN		18		SANDSTONE: fine to coarse grained, light grey, numerous grey and dark grey laminae, bedded at 0-10°.	FR	H	• • •			
		19 20 21 22 23		END OF BOREHOLE AT 18.42m					PVC MONITORING WELL INSTALLED TO 18m	


JK Geotechnics
JOB No. 28618S
BH1
START CORING AT 3.26m
3
4
5
6
7
8
CORE LOSS
9
10
11
12
13
14
15
16
17
18
END OF BOREHOLE AT 18.42m

BOREHOLE LOG

Borehole No.
2
1/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Date: 11-8-15
Method: SPIRAL AUGER
JK300
Logged/Checked by: O.F./P.S.
R.L. Surface: ≈ 39.7m
Datum: AHD

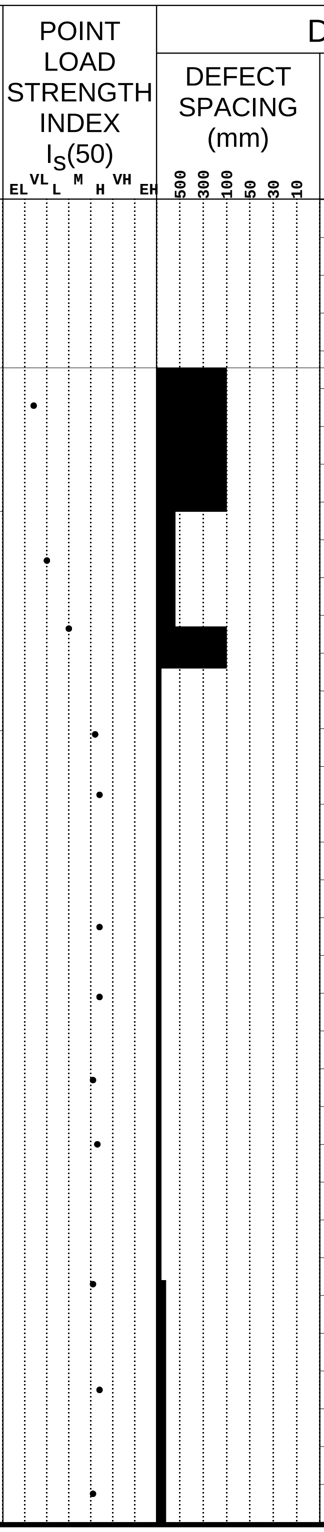
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGER-ING	■					0		-	ASPHALTIC CONCRETE: 10mm.t	M			
									FILL: Sandy gravel, fine to medium grained, grey brown, fine to coarse grained sand, trace of clay.				
	■			■	N = 16 3,4,12	1		CH	SILTY CLAY: high plasticity, red brown and light grey.	MC≈PL	VSt	320 350 350	RESIDUAL
						2		-	SANDSTONE: fine to medium grained, grey and red brown, trace of iron indurated bands.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
				■					as above, but grey and light grey.	DW	VL-L		MODERATE RESISTANCE
						3			REFER TO CORED BOREHOLE LOG				
						4							
						5							
						6							
						7							

CORED BOREHOLE LOG

Borehole No.
2
2/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Core Size: NMLC
R.L. Surface: ≈ 39.7m
Date: 11-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: O.F./P.S.



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
		2		START CORING AT 2.89m				
		3		SANDSTONE: fine to medium grained, light grey, trace of iron indurated bands.	DW	VL-L	- J, 90°, P, R - J, 80°, P, R	
▼ ON COMPLETION OF CORING				SANDSTONE: fine to medium grained, light grey.			- J, 40-90°, Un, R - J, 30-45°, Un, R - XWS, 0°, 60mm.t - Be, 10°, P, R	
		4		as above, but red brown and orange brown.	SW	L-M	- J, 90°, P, R - J, 60°, P, R - Be, 0-10°, Un, R, IS	
FULL RETURN		5 6 7		SANDSTONE: fine to medium grained, light grey, trace of grey laminae, bedded at 10°.	FR	H		
		8		as above, but dark grey laminae more frequent.			- Be, 0°, P, R	
		9		as above, but fine to coarse grained.			- CS, 10°, 2mm.t	

CORED BOREHOLE LOG

Borehole No.
2
3/4

Client: GLADESVILLE RSL & COMMUNITY CLUB LTD

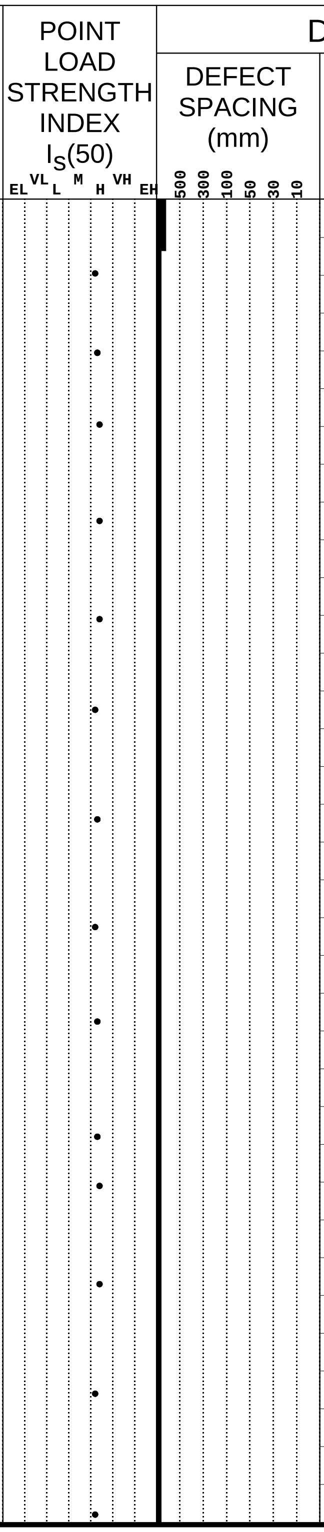
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT

Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Core Size: NMLC
R.L. Surface: ≈ 39.7m

Date: 11-8-15
Inclination: VERTICAL
Datum: AHD

Drill Type: JK300
Bearing: -
Logged/Checked by: O.F./P.S.



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RET-URN		10 11 12 13 14 15		SANDSTONE: fine to coarse grained, light grey, grey laminae, bedded at 10°.	FR	H	- J, 70°, P, R - J, 70°, P, R - CS, 5°, 1mm.t - CS, 0°, 5mm.t - CS, 0°, 4mm.t - Cr, 0°, 10mm.t	

CORED BOREHOLE LOG

Borehole No.
2
4/4

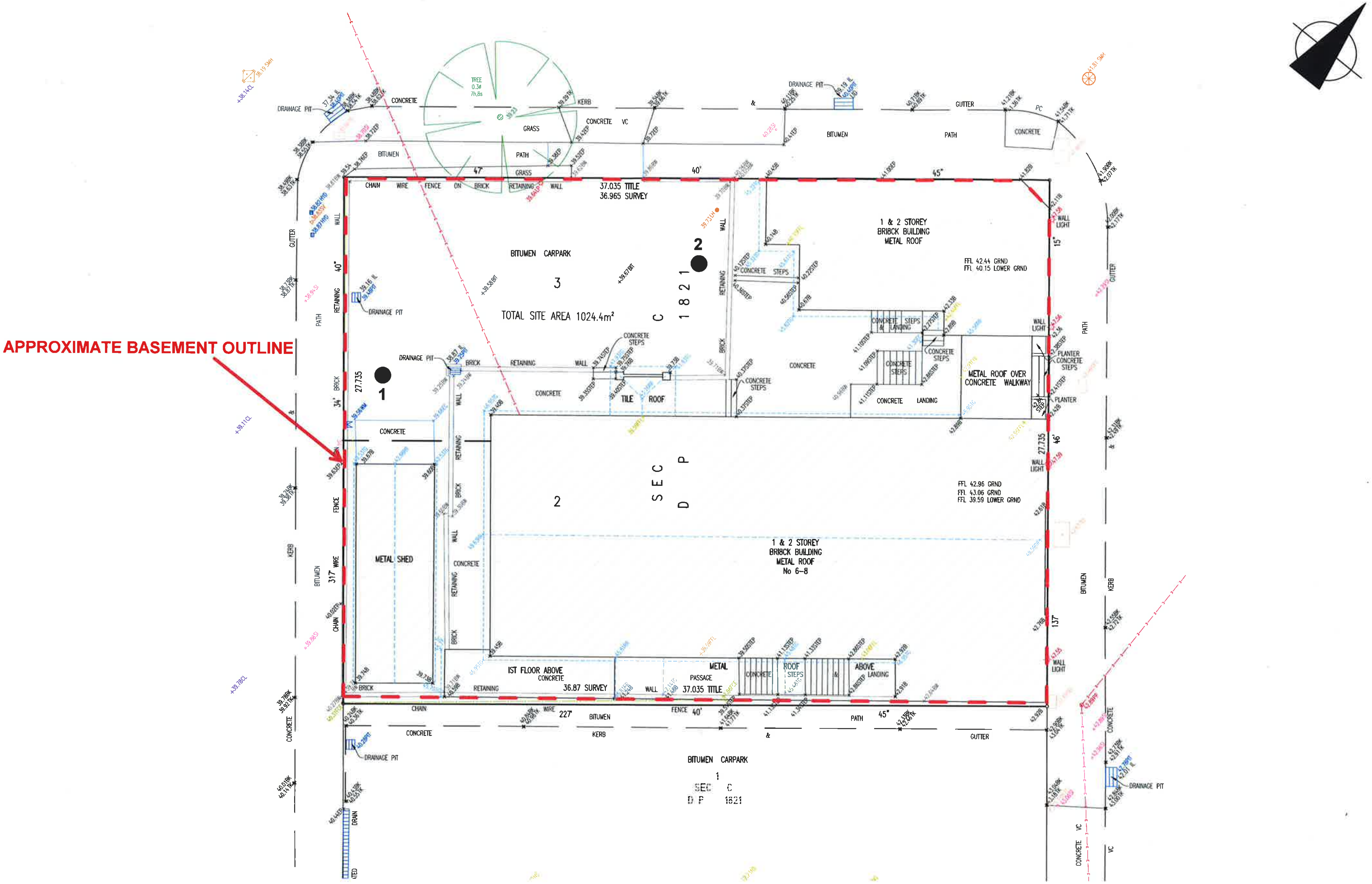
Client: GLADESVILLE RSL & COMMUNITY CLUB LTD
Project: PROPOSED RSL YOUTH CENTRE DEVELOPMENT
Location: 6-8 WESTERN CRESCENT, GLADESVILLE, NSW

Job No. 28618S
Core Size: NMLC
R.L. Surface: ≈ 39.7m
Date: 11-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: O.F./P.S.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_S(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RET-URN		17		SANDSTONE: fine to coarse grained, light grey, grey laminae, bedded at 10°. as above, but with grey laminae, bedded at 15°.	FR	H	● ● ● ●		- Be, 10°, P, R	
		18 19 20 21 22		END OF BOREHOLE AT 17.68m						

JOB NO. 28618S BH2 START CORING AT 2.89m

END OF BOREHOLE AT 17.68m



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Scale (m):
0 10

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GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Title:	Report Number:	Figure Number:
BOREHOLE LOCATION PLAN	28618S	1

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite ‘safe’, depending on the frequency content of the vibration and the actual condition of the structures.

It should also be noted that these levels are ‘safe limits’, up to which no damage due to vibration effects has been observed for the particular class of building. ‘Damage’ is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the ‘safe limits’, then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the ‘safe limits’ are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

NOTE: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

-

REPORT EXPLANATION NOTES

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 (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

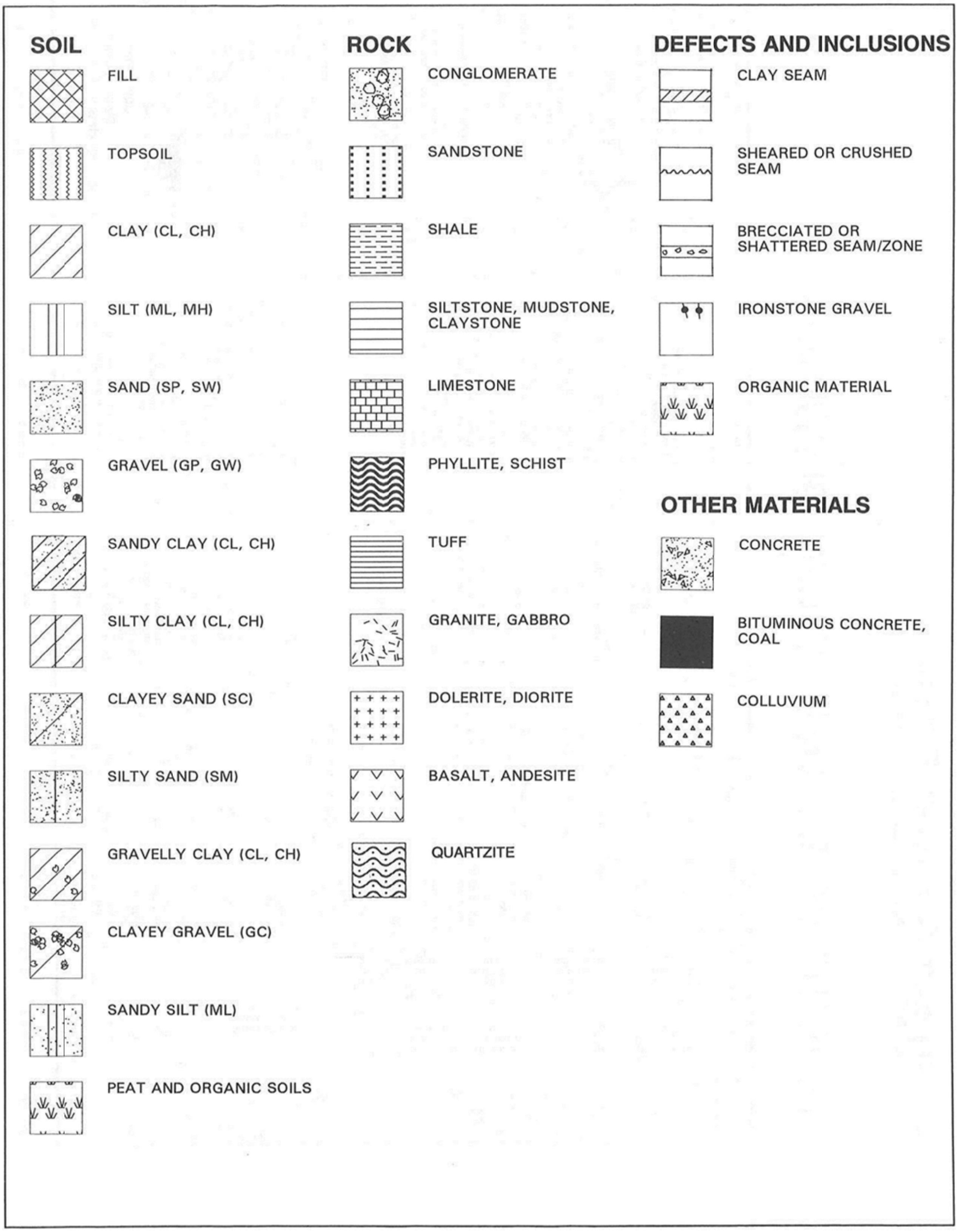
Requirements could range from:

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

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

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



SOIL

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

ROCK

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

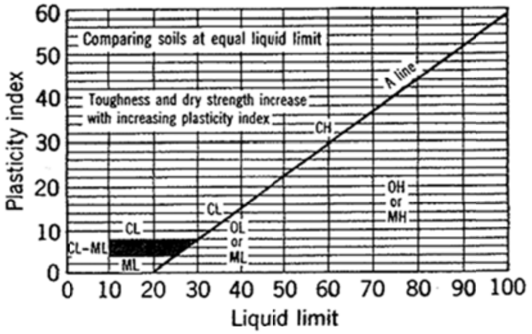
DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

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

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