REPORT

TO
FIOSON PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED REDEVELOPMENT OF
COMPASS CENTRE

AT
THE APPIAN WAY, BANKSTOWN, NSW

2 September 2015
Ref: 28650Zrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 2 September 2015
Report No: 28650Zrpt
Revision No: 0

Report prepared by:

A ZENON
Principal Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed redevelopment of Compass Centre within the Bankstown CBD. The site encompasses a city block bounded to the north, east, south and west by The Mall, The Appian Way, North Terrace and Fetherstone Street, but excludes the existing multi-storey Fetherstone Apartments at 3-7 Fetherstone Street. The investigation was commissioned by Fioson Pty Ltd by signed ‘Acceptance of Proposal’ form dated 29 July 2015. The commission was on the basis of our proposal (Ref P40813Z Bankstown) dated 24 July 2015.

We understand from the JAPM geotechnical brief, that a mixed use development is proposed consisting of:

- Four separate towers ranging between four and 16 storeys.
- Two basement level carparks and a half level above ground parking level. The proposed basements will extend to the site boundaries and we have assumed a maximum excavation depth of about 6m.
- A podium level will be provided which will be accessible by residents and will include amenities (such as communal outdoor spaces, a pool and gym).

We have assumed that typical structural loads for this type of development apply.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, excavation support, retaining walls, footings and on-grade floor slabs.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 17 and 18 August 2015 and comprised the auger drilling of three boreholes (BH1 to BH3) to depths of 5.29m, 5.15m and 5.4m, respectively, using our truck mounted JK350 drilling rig. The boreholes were then extended by diamond coring techniques with water flush, to final depths of 10.84m, 10.52m and 10.68m, respectively. The borehole locations, as indicated on attached Figure 1, were set out using taped measurements from existing surface features, and were electromagnetically scanned for buried services prior to drilling commencing. The surface reduced levels (RLs) at the borehole locations were estimated by interpolation between spot heights shown on the provided survey plans (Ref 150425, Sheets 1/7 to 7/7, dated 12.05.15) prepared by Linker. The survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsurface soil and rock strata were assessed by logging the materials recovered during drilling. The strength of the subsoils was assessed from the Standard Penetration Test (SPT) ‘N’ number augmented by hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. The strength of the augered portion of the bedrock was assessed by observation of drilling resistance when using a tungsten carbide (TC) bit. The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with Point Load Strength Index testing. Groundwater observations were made during augering, on completion of augering, and on completion of core drilling individual boreholes. A slotted PVC standpipe was installed in BH2. The standpipe construction is described on the relevant borehole log. On 28 August 2015, about 10 days following completion of drilling, we returned to site and remeasured the groundwater level in the standpipe. Longer term groundwater monitoring was not carried out. For further details on the investigation procedure adopted, reference should be made to the attached Report Explanation Notes.

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

The recovered rock core was returned to our yard where it was photographed and select sections of core were submitted to Soil Test Services Pty Ltd (STS), a NATA registered laboratory, for Point Load Strength Index testing. The test results are summarised in attached STS Table A, and have been plotted on the borehole logs. The core photographs are presented opposite the relevant borehole logs. The Unconfined Compressive Strengths (UCSs), as estimated from the Point Load

Strength Index tests, are also summarised in STS Table A. Selected soil samples were also submitted Envirolab, a NATA registered laboratory, for soil pH and sulfate/chloride content testing. The test results are presented in Envirolab's 'Certificate of Analysis' (133024) which is included in Appendix A. Contamination screen testing of the site soils was outside the agreed scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located over a gently sloping north-east facing hillside. The site comprises the city block bounded to the north, east, south and west by The Mall, The Appian Way, North Terrace and Fetherstone Street, but excludes the Fetherstone Apartments at 3-7 Fetherstone Street.

At the time of the investigation, several single and double storey buildings lined the southern and eastern portions of the site. A three storey library building was located over the north-west, and a multi-storey brick building was located over the mid-west. An asphaltic concrete (AC) carpark was located over the north-east and was connected to Fetherstone Street by a laneway along the southern side of the library building.

The multi-storey residential building, Fetherstone Apartments, was located between the access laneway and the multi-storey building over the mid-west. This building which is not within the development site, appeared to include basement levels, but the depth and extent of the basements could not be determined.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith indicates that the site is underlain by Bringelly Shales. The investigation has revealed a generalised subsurface profile below the AC, comprising surficial fill over residual silty clay with weathered shale bedrock at relatively shallow depth. For detailed subsurface conditions at specific locations, reference should be made to the attached borehole logs. A graphical borehole summary is presented in Figure 2 and a summary of the subsurface conditions as encountered, is presented below:

- AC pavements 20mm (BH1 and BH3) and 70mm (BH2) thick were encountered at the borehole surfaces. The AC was underlain by a sandy gravel base to 0.4m (BH1 and BH2) and 0.7m (BH3) depth.

- Fill comprising silty clay of high plasticity was encountered below the pavements and extended to depths between 0.6m (BH2) and 1.2m (BH3).
- Residual silty clay of high plasticity was encountered below the fill. The silty clay was of variable strength between stiff and hard.
- Weathered shale bedrock was encountered at depths of 4.4m in all boreholes. On first contact, the shale was of very low to low strength and improved with depth to low to medium strength and better.
- Groundwater was not encountered whilst auger drilling the boreholes. On completion of core drilling BH2, the groundwater was measured at a depth of 1.35m. We note that water is added to the boreholes to facilitate coring, and this masks any natural groundwater seepage. However, the estimated full recycled flush water return indicates a relatively impermeable rock mass. On our return to site approximately 10 days following completion of drilling, the groundwater level in BH2 was measured at a depth of 2.49m.

3.3 Laboratory Test Results

The laboratory Point Load Strength Index test results correlated reasonably well with our field assessed rock strengths. The UCS of the rock core, as estimated from the Point Load Strength Index test results, varied between 6MPa and 28MPa.

3.4 Rock Classifications

The following classifications for the shale bedrock, in accordance with Pells *et al* (1998), apply:

Location	**Depth to Surface of Rock Class**			
	Class V (m)	**Class IV (m)**	**Class III (m)**	**Class II (m)**
BH1	4.4 – 5.1	5.1 – 6.6	–	6.6 – 10.84
BH2	4.4 – 4.8	–	–	4.8 – 10.52
BH3	4.4 – 5.0	8.4 – 10.7	–	5.0 – 8.4

4 COMMENTS AND RECOMMENDATIONS

The geotechnical comments and recommendations which follow are based on a limited number of boreholes, with poor site coverage. Also, the development details (eg. basement RLs) had not been finalised. We therefore recommend that further geotechnical investigations be carried out following demolition. The further geotechnical investigation must be designed to address the proposed development details. This report should then be reviewed and revised as necessary, based on the further investigation results and the final development proposal.

4.1 Geotechnical Issues and General Overview

1 The site is underlain by soil and variable shale bedrock.

2 As the proposed basement excavation will extend to the site boundaries, shoring will be required and should be installed prior to excavation commencing. Suitable shoring systems include soldier pile walls with shotcrete infill panels installed to below bulk excavation level. The shoring must be progressively anchored as excavation proceeds. Further investigations of the basement details at 3-7 Fetherstone Street are required prior to finalising shoring details at this location.

3 The proposed bulk excavation to estimated depths of about 6m will extend through the soil profile and Class V shale, and locally into the underlying Class IV and Class II shale bedrock. Hard rock excavations conditions must be expected within the Class II shale and should be accompanied by vibration monitoring as a precaution to avoid vibration damage to surrounding buildings and structures.

4 Class IV or better quality shale bedrock will be encountered at bulk excavation level, assumed to be at 6m depth, and therefore, conventional pad footings may be employed.

5 The bulk excavation to about 6m depth may extend below the measured groundwater level. We note, however, that there is uncertainty regarding whether the measured level was groundwater representing a body of water, the introduced flush water return or groundwater seepage from discreet joints under artesian pressure. We therefore recommend that pump-out tests be carried out to estimate likely groundwater infiltration rates.

The above issues are discussed in further detail in the sections which follow.

4.2 Excavation Conditions

4.2.1 Excavation Methods

Based on the investigation results, the proposed bulk excavation to an estimated depth of about 6m will encounter the soil and Class V bedrock, and will extend locally into Class IV and Class II shale. The soil in Class V/IV shale bedrock should be readily excavatable using conventional earthworks equipment (such as medium to large excavators). Some of the Class V/IV shale may require localised ripping if stronger ironstone or shale bands are encountered. Hard rock excavation conditions must be anticipated within the Class II and better quality shale bedrock. We expect that this class of bedrock would be most effectively excavated using hydraulic excavators fitted with hydraulic impact rock hammers. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations (such as for footings or buried services).

Particular care is required during bulk excavation to avoid over-breaks that could result in encroachment beyond the site boundaries.

4.2.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on the site as there will likely be direct transmission of ground vibrations to adjoining buildings, structures and services. The proposed bulk excavation will extend to No 3-7 Fetherstone Street. Prior to excavation commencing, we recommend that detailed dilapidation surveys be undertaken on this building, and the owners asked to confirm that the reports present a fair record of existing conditions. Council may also require dilapidation reports of their assets. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage as a result of the works. The excavation procedures and the dilapidation reports should be carefully reviewed prior to excavation commencing so that appropriate equipment is used.

We recommend that continuous vibration monitoring be carried out when using rock hammers on the site. Subject to review of the dilapidation reports, we recommend that vibrations, measured as Peak Particle Velocity (PPV), on the neighbouring building be limited no higher than 15mm/sec. If higher vibrations are measured, then it would be necessary to use lighter equipment or to use alternative low vibration excavation techniques. Lower vibration excavation techniques include providing a vertical saw cut slot along the perimeter of the excavation and maintaining the base of the slot at a lower level than the adjoining rock excavation at all times. Also the use of grid sawing in conjunction with hammering/ripping, present alternative low vibration techniques. When using the rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate one hammer at a time and in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

4.2.3 Seepage

We note that although the groundwater level was measured at a depth of 2.49m approximately 10 days following completion of drilling, groundwater was not encountered on completion of auger drilling, which extended to a maximum depth of 5.4m. There is thus uncertainty as to whether the measured groundwater is a natural body of water, flush water which is receding very slowly, or groundwater seepage from discreet joints within the shale under artesian pressures. Based on our experience in the area, groundwater, if present, is likely to be discreet seepage from joints or other defects within the shale bedrock. Further, the seepage infiltration rate into the basement is likely to be very slow due to the relatively impermeable rock mass. Based on the above, we recommend that a drained basement should be adopted.

However, in order to confirm groundwater conditions, we recommend that we return to site and carry out pump-out tests within the standpipe. The standpipe will be purged (possibly several times), and then the rate of groundwater recovery will be measured. Using established formulae, a mass permeability of the rock mass will be determined and then used to estimate the likely groundwater inflow into the basement. The proposed drained basement can then be confirmed.

Irrespective, we recommend that a toe drain be formed at the base of all cut rock faces to collect groundwater seepage and direct it to a sump for pumped discharge to the stormwater system. Groundwater flows into the bulk excavation must be monitored by the site foreman and geotechnical engineer and the results reviewed, so that the actual inflow rate can be confirmed and any unexpected conditions can be timeously addressed.

4.3 Excavation Support

As the excavation will extend to the site boundaries, a shoring system will be required and should be installed prior to excavation commencing. Given the subsurface profile encountered, a suitable shoring system includes a full depth soldier pile wall with shotcrete infill panels. Suitable pile types include conventional bored piles which are progressively anchored as excavation proceeds.

Bulk excavation within the rock profile must be inspected by a geotechnical engineer on completion to identify adverse defects and propose any additional stabilisation measures which may be required. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc.

As stated in Section 4.1 above, the basement details of 3-7 Fetherstone Street must be confirmed prior to finalising shoring details at the location. The basement details required include basement total depth, lateral extent and shoring/perimeter wall footing type/depth.

4.4 Retaining Walls

The major consideration in the selection of earth pressure in the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressures coefficients and subsoil parameters may be adopted for the static design of temporary or permanent retention systems:

- For free-standing cantilever walls which support areas where movement is of little concern, adopt a triangular lateral earth pressure distribution and an ‘active’ earth pressure coefficient, K_a, of 0.3, for the soil profile and a Class V/IV shale bedrock, assuming a horizontal backfill surface.
- For cantilever walls, the tops of which will be propped by the proposed floor slabs, adopt a triangular lateral earth pressure distribution and an ‘at rest’ earth pressure coefficient, K_o, of 0.6, for the soil profile and a Class V/IV shale bedrock, assuming a horizontal retained surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and Class V/IV shale bedrock.
- For anchored or internally propped walls which support areas which are highly sensitive to lateral movement (eg. adjacent to neighbouring buildings or movement sensitive buried services), a trapezoidal earth pressure distribution of 8H kPa should be adopted for the soil

profile and Class V/IV shale bedrock, where ‘H’ is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.

- For anchored or internally proposed walls where minor movements can be tolerated (eg. along the street frontages provided there are no movement sensitive buried services), we recommend the use of a trapezoidal earth pressure distribution of 6H kPa for the soil profile and Class V/IV shale bedrock, where ‘H’ is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.
- Shotcrete infill panels may be designed using trapezoidal earth pressure distributions of 6H kPa or 4H kPa, respectively.
- Any surcharge affecting the walls (eg. traffic loading, nearby buildings, construction loads, etc) should be allowed in the design using the appropriate earth pressure coefficient from above.
- The retaining walls should be designed as drained and measures taken to induce complete and permanent drainage of the ground behind the walls. Strip drains behind the shotcrete infill panels and which incorporate a non-woven geofabric (eg. Bidim A34) to act as a filter against subsoil erosion, would be suitable.
- For piles embedded into underlying bedrock below bulk excavation, an allowable lateral toe resistance of 300kPa may be adopted, the upper 0.3m depth of the sockets should not be taken into account to allow for tolerances and disturbance effects during excavation.
- Rock anchors will run below adjoining properties and the permission of the respective owners should be obtained before installation. Anchors bonded into at least Class III or better shale may be tentatively designed for an allowable bond stress of 300kPa. The anchors should have a minimum bond length of 3m which is bonded beyond a 45° line which extends up from the base of the excavation. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installation. For the new basement, we assume that permanent lateral support of retaining walls will be provided by the new structure. If not, permanent anchors will be required which should be designed for corrosion resistance and for long term durability.

However, if computer based analyses (such as WALLAP or PLAXIS) are to be carried out, we recommend the following parameters be adopted:

Material	**Depth (m)**	**Cohesion (c′ kPa)**	**Internal Friction (ϕ′°)**	**Bulk Unit Weight (kN/m^3)**	**Poisson's Ratio**	**Elastic Modulus (MPa)**
Fill/Clay	0 – 4.4	–	28	18	0.3	30
Class V/IV Shale	4.4 – 6.0	10	32	20	0.3	100
Class III Shale and better depth	6.0+	50	35	24	0.24	1,000

4.5 Footings

Based on the investigation results, Class II or better shale bedrock will be encountered at, and just below, the estimated bulk excavation level at about 6m depth. We note, however, that the Class II shale in BH3 was underlain by Class IV shale from 8.4m depth.

Soldier piles founded in bedrock below bulk excavation level, should be designed for an allowable end bearing pressure of 1MPa. In addition, an allowable shaft adhesion of 100kPa should be applied for rock sockets beyond 0.5m below bulk excavation level.

Pad footings founded in Class II shale bedrock at bulk excavation level may be designed for an allowable bearing pressure of 4MPa, subject to inspection by a geotechnical engineer.

The above pressures are based on serviceability considerations. Should the designer wish to adopt limited state design methods, then ultimate bearing pressures of 3MPa and 30MPa should be adopted, respectively. A geotechnical strength reduction factor will need to be used with the above, and should be determined for the project specifics of the proposed development. Typically the geotechnical strength reduction factor would be around 0.5.

Based on the Envirolab test results, concrete piles should be designed for a ‘non-aggressive’ exposure classification in accordance with AS2159–2009.

4.6 Lower Basement On-Grade Floor Slab

For the slab-on-grade, an underfloor drainage layer must be provided. The underfloor drainage should comprise a strong, durable, single sized washed aggregate (eg. ‘blue metal’ gravel). The underfloor drainage should connect with the wall drains, where appropriate, and lead groundwater seepage to a sump(s) for pumped disposal. Joints in the concrete on-grade floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.7 Earthquake Design Parameters

Based on the investigation results, a Class C_e – Shallow Soil Site, applies in accordance with AS1170.4–2007. The hazard factor of 0.08 is applicable for Sydney.

4.8 Further Geotechnical Input

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

- Further geotechnical investigations following demolition.
- Review of this report and revise as necessary in light of the results of the further geotechnical investigation and the final development details.
- Dilapidation surveys of neighbouring building and Council assets, if required.
- Confirmation of basement depth and extent at 3-7 Featherstone Street.
- Quantitative vibration monitoring during rock excavation.
- Progressive geotechnical inspections of cut rock faces.
- Proof-testing of anchors.
- Monitoring of groundwater seepage into bulk excavation with geotechnical review.
- Geotechnical footing inspections.

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	**Ref No:**	28650Z
Project:	Proposed Redevelopment of Compass Centre	**Report:**	A
		Report Date:	24/08/2015
Location:	The Appian Way, Bankstown, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	5.51-5.54	0.3	6
	6.75-6.77	0.8	16
	7.50-7.53	1.1	22
	8.51-8.54	0.7	14
	9.71-9.74	1.4	28
	10.38-10.41	0.8	16
2	5.29-5.32	0.6	12
	6.36-6.40	0.6	12
	7.65-7.68	0.4	8
	9.37-9.40	0.4	8
	10.38-10.41	1.2	24
3	5.89-5.91	0.6	12
	6.79-6.82	0.9	18
	7.65-7.68	1.4	28
	8.58-8.61	0.3	6
	9.82-9.84	0.3	6

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: 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 $I_{S\,(50)}$

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/2

Client: FIOSON PTY LTD
Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE
Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Date: 17-8-15
Method: SPIRAL AUGER JK350
Logged/Checked by: T.P./A.Z.
R.L. Surface: ≈ 22.9m
Datum: AHD

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING		0		ASPHALTIC CONCRETE: 20mm.t	D			
				FILL: Sandy gravel, fine to medium grained igneous, blue grey, fine to coarse grained sand.	M			
	N = 8 5,3,5			FILL: Silty clay, high plasticity, dark brown, trace of fine grained igneous gravel and ash.	MC>PL			
		1	CH	SILTY CLAY: high plasticity, brown mottled light grey, race of fine grained ironstone gravel.	MC>PL	St	140 150 190	POSSIBLY FILL
				SILTY CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.		VSt		
	N = 9 2,4,5			as above, but light grey mottled orange brown.		St	210 220 210	
		2	CL	SILTY CLAY: low to medium plasticity, orange brown and light grey, trace of fine grained sand.	MC≈PL	VSt	180 180 180	
				SILY CLAY: low to medium plasticity, orange brown and light grey.			380 250 320	
	N = 17 15,8,9	3 4		as above, but with XW shale seams, EL strength.	MC<PL	H		EXTREMELY LOW 'TC' BIT RESISTANCE VERY LOW RESISTANCE WITH LOW BANDS
	N = SPT 6/50mm REFUSAL	5	-	SHALE: grey, with M-H strength iron indurated bands.	DW	VL-L		
				as above, but without iron indurated bands.	SW	L		LOW RESISTANCE WITH MODERATE BANDS
		6 7		REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
1
2/2

Client: FIOSON PTY LTD

Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE

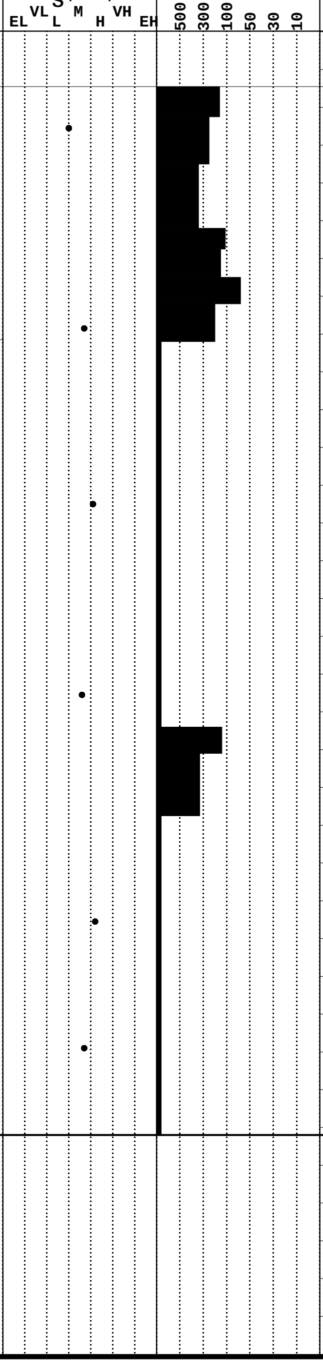
Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Core Size: NMLC
R.L. Surface: ≈ 22.9m

Date: 17-8-15
Inclination: VERTICAL
Datum: AHD

Drill Type: JK350
Bearing: -
Logged/Checked by: T.P./A.Z.

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
		5		START CORING AT 5.29m						
FULL RET-URN		6		SHALE: grey, with dark grey laminae, bedded at 0-5°, and fine grained sandstone bands.	SW	L-M			- Be, 5°, 15mm.t, P, R - J, 90°, Un, R - J, 90°, Un, R - XWS, 180mm.t - J, 75°, P, R - Cr, 50mm.t - J, 90°, Un, R - Cr, 0°, 20mm.t, CLAY INFILL	
		7 8 9 10				M-H			- Be, 5°, 10mm.t - Cr, 0°, 20mm.t - Be, 0°, 10mm.t - Cr, 0°, 15mm.t - J, 90°, Un, R	
		11		END OF BOREHOLE AT 10.84m						



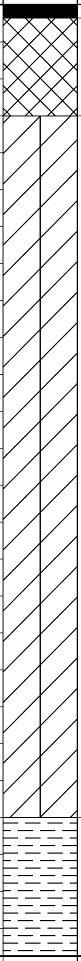

JK Geotechnics
JOB # 28650Z BH1 START CORING AT 5.29m
5
6
7
8
9
10
END OF BH AT 10.84m

BOREHOLE LOG

Borehole No.
2
1/2

Client: FIOSON PTY LTD
Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE
Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Date: 17-8-15
Method: SPIRAL AUGER
JK350
R.L. Surface: ≈ 22.1m
Datum: AHD
Logged/Checked by: T.P./A.Z.



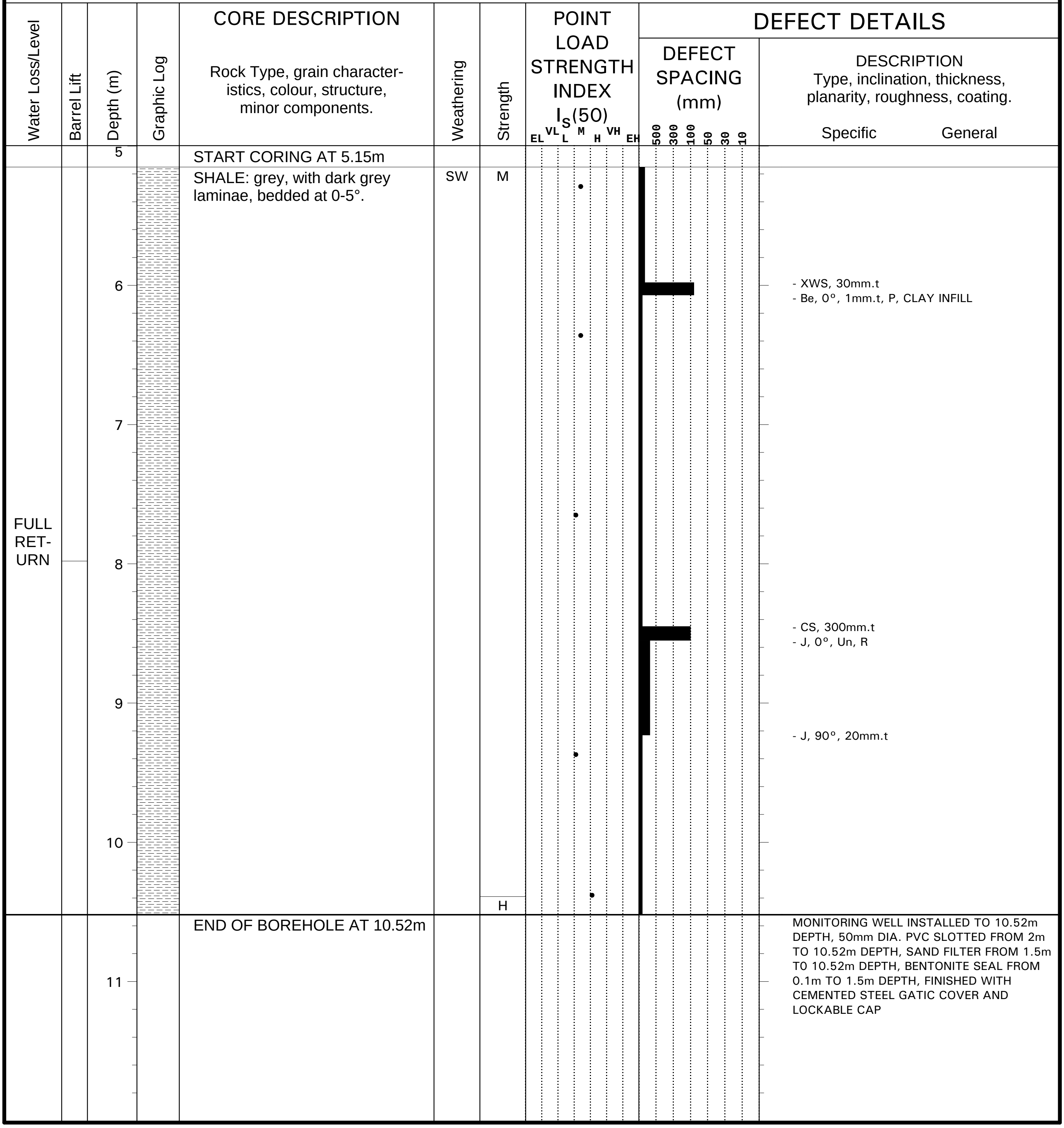
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING	■					0		ASPHALTIC CONCRETE: 70mm.t	D			
	■			■			-	FILL: Sandy gravel, fine to medium grained igneous, blue grey, fine to coarse grained sand.	M MC>PL			
	■			■	N = 8 2,3,5	1	CL-CH	FILL: Silty clay, low to medium plasticity, dark brown, dark grey, with fine to medium grained sand, trace of fine grained igneous gravel.	MC>PL	F-St	100 130 90	
								SILTY CLAY: medium to high plasticity, brown mottled light grey, trace of root fibres.				
ON COMPLETION OF CORING				■	N = 16 6,8,8	2		as above, but without root fibres, trace of fine grained sand and fine grained ironstone gravel.		VSt	300 260 180	
ON 28/8/15												
				■	N > 27 12,20, 7/20mm REFUSAL	3		SILTY CLAY: medium to high plasticity, red brown mottled light grey and orange brown, trace of fine grained sand and fine to medium grained ironstone gravel.			230 310 240	EXTREMELY LOW TO VERY LOW 'TC' BIT RESISTANCE
								as above, but with iron indurated seam.				
						4		SILTY CLAY: medium to high plasticity, red brown mottled light grey and orange brown, trace of fine grained sand and fine to medium grained ironstone gravel.				
				■	SPT 21/80mm REFUSAL		-	SHALE: grey.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						5			SW	M		LOW RESISTANCE
						6 7		REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
2
2/2

Client: FIOSON PTY LTD
Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE
Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Core Size: NMLC
R.L. Surface: ≈ 22.1m
Date: 18-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK350
Bearing: -
Logged/Checked by: T.P./A.Z.



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
		5		START CORING AT 5.15m						
FULL RET-URN				SHALE: grey, with dark grey laminae, bedded at 0-5°.	SW	M				
		6							- XWS, 30mm.t - Be, 0°, 1mm.t, P, CLAY INFILL	
		7								
		8							- CS, 300mm.t - J, 0°, Un, R	
		9							- J, 90°, 20mm.t	
		10				H				
		11		END OF BOREHOLE AT 10.52m						MONITORING WELL INSTALLED TO 10.52m DEPTH, 50mm DIA. PVC SLOTTED FROM 2m TO 10.52m DEPTH, SAND FILTER FROM 1.5m TO 10.52m DEPTH, BENTONITE SEAL FROM 0.1m TO 1.5m DEPTH, FINISHED WITH CEMENTED STEEL GATIC COVER AND LOCKABLE CAP


JK Geotechnics
JOB# 28650Z BH2 START CORING AT 5.15m
5
6
7
8
9
10
END OF BH AT 10.52m

BOREHOLE LOG

Borehole No.
3
1/2

Client: FIOSON PTY LTD

Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE

Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Date: 18-8-15

Method: SPIRAL AUGER
JK350

R.L. Surface: ≈ 22.4m
Datum: AHD

Logged/Checked by: T.P./A.Z.

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: 20mm.t	D			
	■								FILL: Sandy gravel, fine to medium grained igneous, blue grey, fine to coarse grained sand.	M			
				■	N = 6 5,3,3								
	■					1			FILL: Silty clay, medium to high plasticity, dark grey and brown, trace of fine grained sand and fine grained igneous gravel.	MC>PL			
								CL-CH	SILTY CLAY: medium to high plasticity, orange brown mottled light grey, trace of root fibres.	MC>PL	VSt		
	■			■	N = 12 4,6,6				as above, but trace of fine grained sand.			270 280 360	
						2							
				■					SILTY CLAY: medium to high plasticity, red brown mottled light grey and orange brown, trace of fine grained sand and fine to medium grained ironstone gravel.				
				■	N = 47 22,22,25	3			as above, but with iron indurated seam.			250 220 380	
						4			SILTY CLAY: medium to high plasticity, red brown mottled light grey and orange brown, trace of fine grained sand and fine to medium grained ironstone gravel.				
				■	N = SPT 18/80mm REFUSAL			-	SHALE: grey.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						5				SW	M		LOW TO MODERATE RESISTANCE
						6			REFER TO CORED BOREHOLE LOG				
						7							

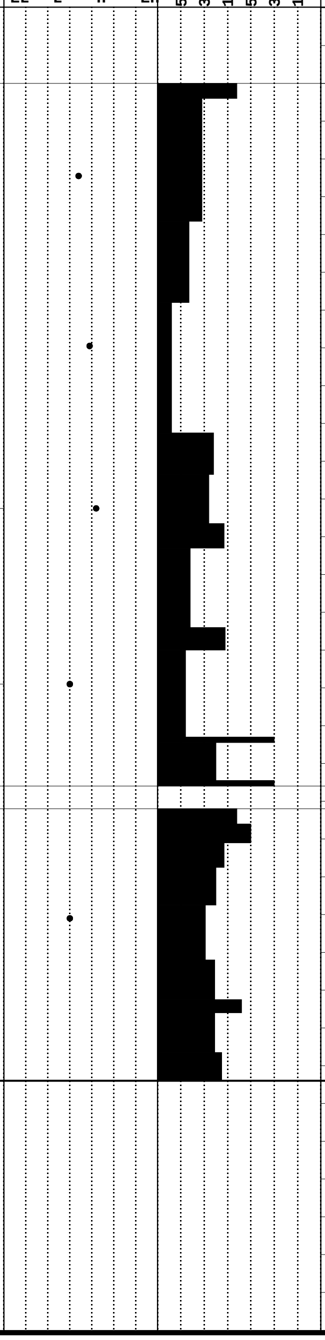
CORED BOREHOLE LOG

Borehole No.
3
2/2

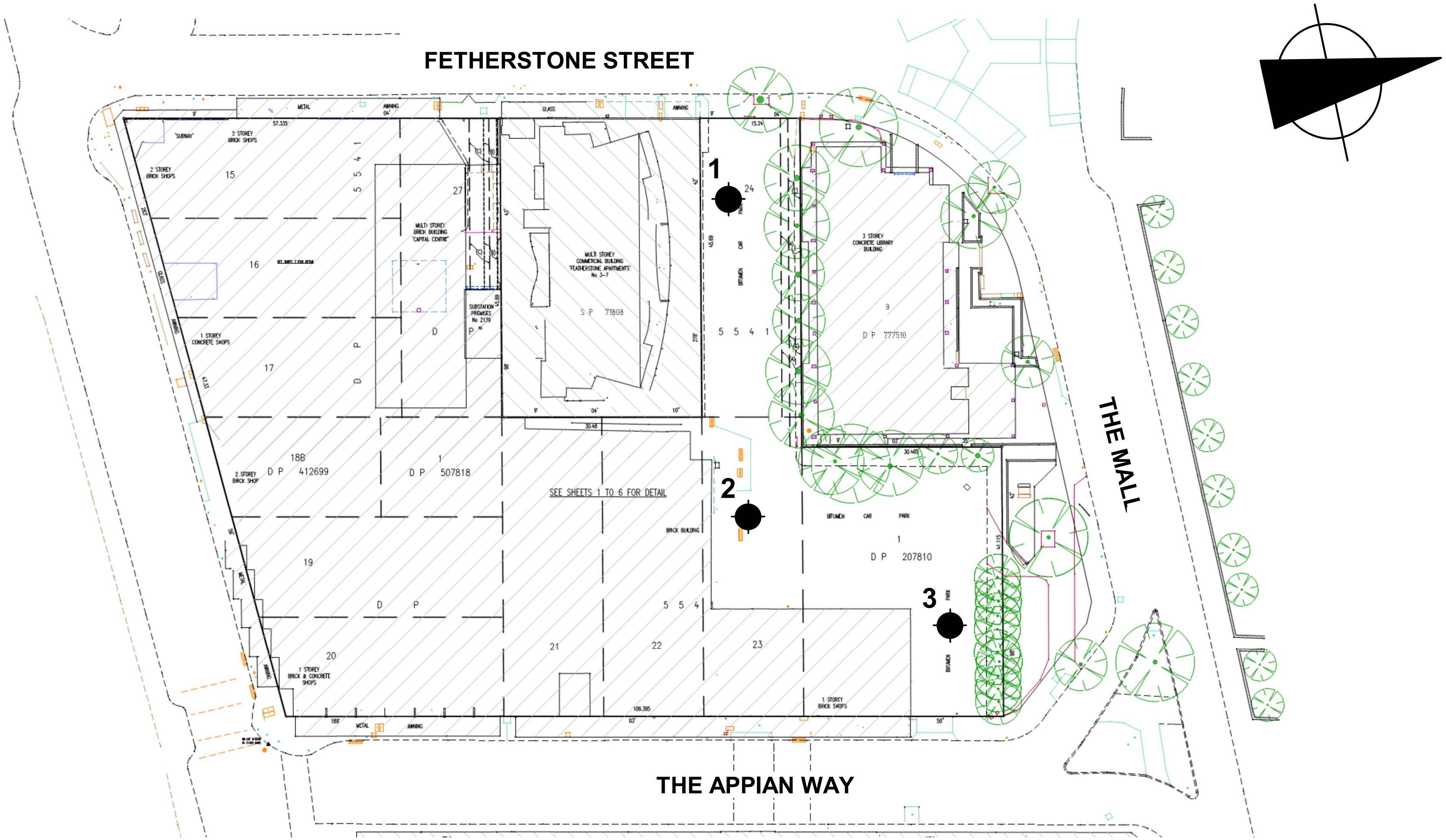
Client: FIOSON PTY LTD
Project: PROPOSED REDEVOPMENT OF COMPASS CENTRE
Location: THE APPIAN WAY, BANKSTOWN, NSW

Job No. 28650Z
Core Size: NMLC
R.L. Surface: ≈ 22.4m
Date: 18-8-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK350
Bearing: -
Logged/Checked by: T.P./A.Z.

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
	5	START CORING AT 5.40m			
FULL RETURN	6 7	SHALE: grey and dark grey laminae, bedded at 0-5°.	SW	M	- Cr, 160mm.t - J, 90°, P, S - J, 90°, P ,R - J, 90°, P, R, CLAY COATED - J, 90°, Un, S - J, 90°, Un, R
	8			H	- XWS, 2°, 10mm.t - J, 90°, P, R, CALCITE COATED - J, 90°, P, R, CALCITE COATED - J, 5°, P, R
	9			L-M	- J, 90°, Un, R, CALCITE COATED - J, 50°, - XWS, 2°, 10mm.t
		CORE LOSS 0.12m			
	10	SHALE: grey and dark grey laminae, bedded at 0-5°.	SW	L-M	- J, 60°, Un, S - J, 50°, P, HEALED - J, 40°, P, R - J, 50°, Un, HEALED - XWS, 35°, 90mm.t - J, 90°, P, HEALED - Cr, 2°, 45mm.t - J, 90°, Un, R, CLAY COATED - J, 90°, P, HEALED, CARBONACEOUS COATING
	11	END OF BOREHOLE AT 10.68m			




JK Geotechnics
JOB # 28650Z BH3 START CORING AT 5.40m
5
6
7
8
9
CORE LOSS 0.12m
10
END OF BH AT 10.68m



LEGEND

● BOREHOLE

Approximate Scale (m):

0 50

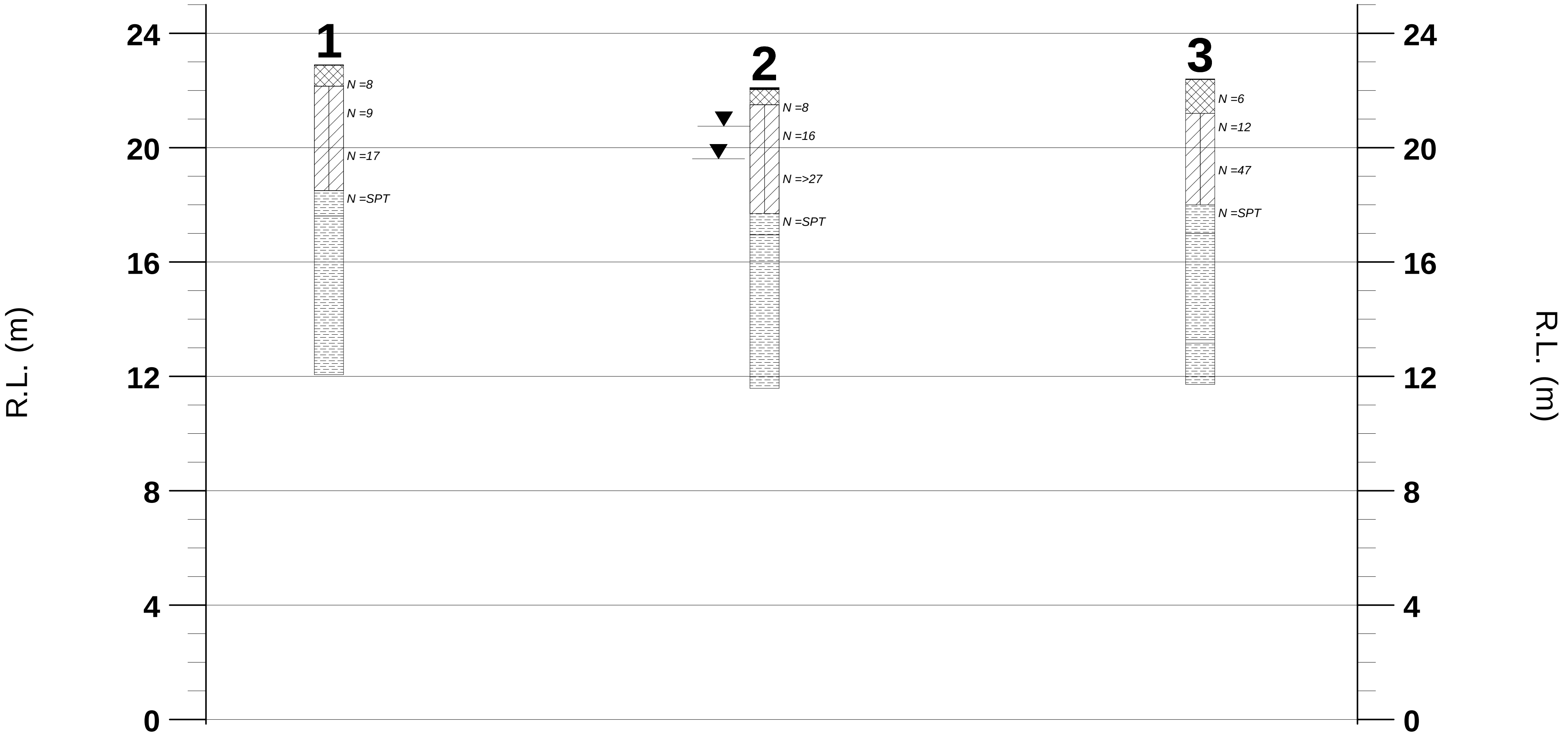
JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Title:

BOREHOLE LOCATION PLAN

Report Number:	Figure Number:
28650Z	1

GRAPHICAL BOREHOLE SUMMARY



■ **Asphaltic/ Bituminous Paving or Coal**

▩ **Fill**

▨ **Silty Clay**

▤ **Shale**

☐ **Core Loss/ Empty**

▼ **Observed water level**

N **SPT "N" VALUE**

Nc **SOLID CONE BLOW COUNTS PER 150mm**

NOTE: REFER TO BOREHOLE LOGS

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

JK Geotechnics

Job No.: 28650Z

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 (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 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.

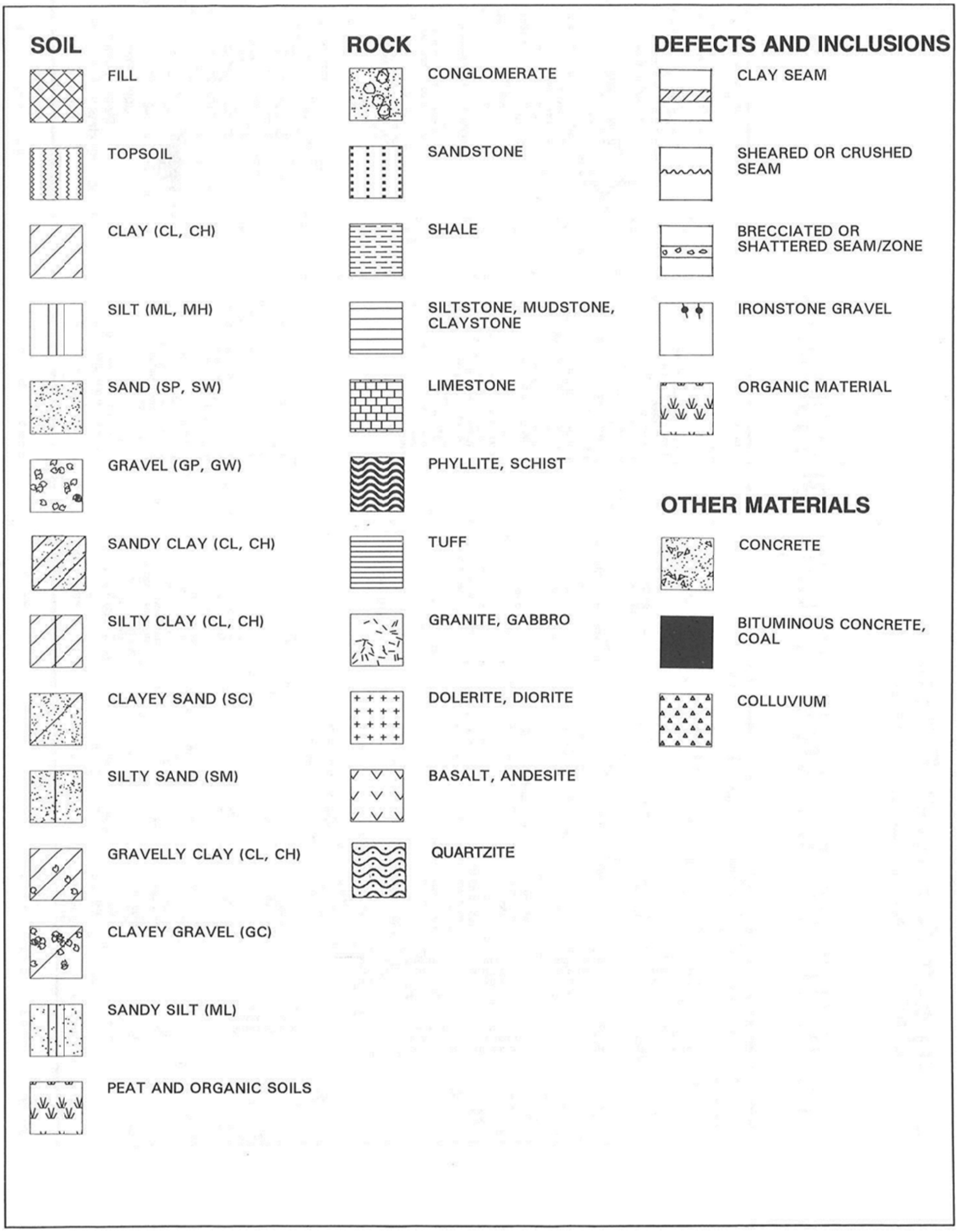
SITE INSPECTION

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

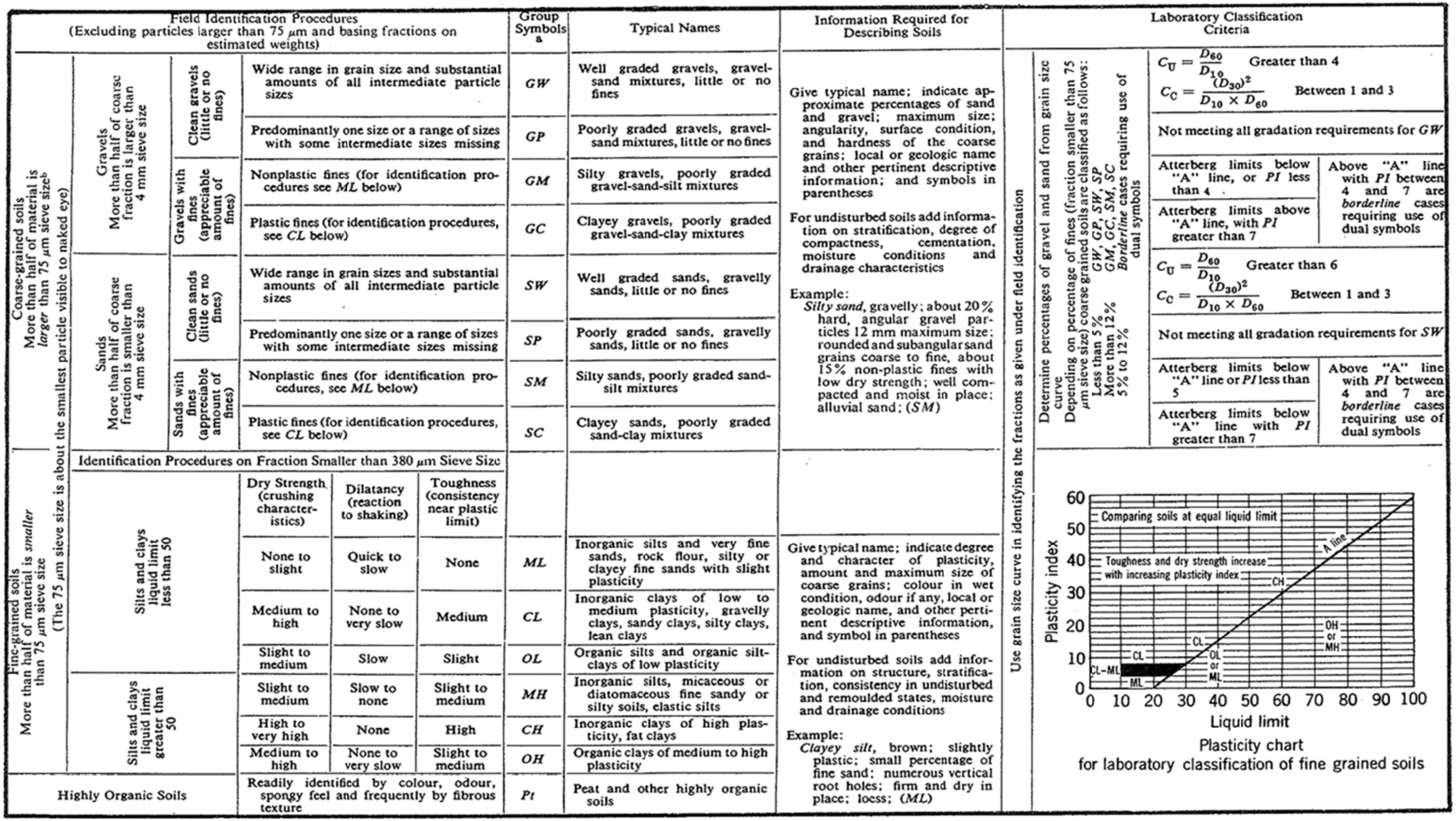
Requirements could range from:

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



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			
(The 75 μm sieve size is about the smallest particle visible to naked eye)	Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b]	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										Plasticity chart for laboratory classification of fine grained soils	
				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*)				
				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

Envirolab Services
‘Certificate of Analysis’
(133024)

ENVIROLAB
SERVICES

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 133024

Client:

JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Tristan Piat

Sample log in details:

Your Reference:	**28650Z, Bankstown**	
No. of samples:	1 soil	
Date samples received / completed instructions received	20/08/15	/ 20/08/15

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:	27/08/15	/ 25/08/15
Date of Preliminary Report:	Not Issued	

NATA accreditation number 2901. This document shall not be reproduced except in full.
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: Your Reference Depth Date Sampled Type of sample	UNITS ------------- ------------	133024-1 1 0.75-0.95 17/08/2015 Soil
Date prepared	-	21/08/2015
Date analysed	-	21/08/2015
pH 1:5 soil:water	pH Units	6.9
Sulphate, SO4 1:5 soil:water	mg/kg	300
Chloride, Cl 1:5 soil:water	mg/kg	130

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 II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			21/08/2015	[NT]	[NT]	LCS-1	21/08/2015
Date analysed	-			21/08/2015	[NT]	[NT]	LCS-1	21/08/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	102%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	106%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	103%

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.