REPORT

TO
JANPEC PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MIXED USED DEVELOPMENT

AT
171-189 PARRAMATTA ROAD, GRANVILLE

24 September 2014
Ref: 24711PYrpt

AS/NZS ISO 9001
Certified
Davis Langdon Certification Services

JK Geotechnics
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Date: 24 September 2014
Report No: 24711PYrpt
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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 a proposed mixed use development at 171-189 Parramatta Road, Granville, NSW. The investigation was commissioned by Mr Arthur Maroon of Janpec Pty Ltd and was completed in accordance with our proposal (Ref: P38423SYemail, dated 11 March 2014).

At this stage the details of the proposed development have not yet been finalised although we understand that it is proposed that development of the site will comprise buildings varying from 5 to 37 storeys constructed over between two and four basement carparking levels. It is also understood that excavation will extend close to the site boundaries and will result in cuts to depths of between 6m and 14m.

The purpose of the investigation was to use information from our previous investigation of the site in 2004as a basis for providing a report complete with our comments and recommendations on excavation, shoring, dewatering and footing design for the current development proposal.

An environmental site screening was completed by Environmental Investigation Services (EIS), a division of Jeffery and Katauskas Pty Ltd. The results of this investigation are provided in their report E27710KGrpt dated 18 September 2014.

2 INVESTIGATION PROCEDURE

The previous investigation comprised the drilling of twelve boreholes with a track mounted JK250 rig and a truck mounted JK550 drilling rig. Seven of these boreholes were drilled for environmental sampling to a depth of 1.5m. One Additional borehole was drilled to 0.6m depth with a hand auger, also for environmental sampling.

The remaining five boreholes (BH1, BH6, BH10, BH11 and BH13) were drilled primarily for geotechnical purposes and were initially advanced using spiral auger drilling techniques. Once better quality shale bedrock was encountered, diamond coring of the shale bedrock was completed with between 2.6m and 3.1m of core recovered from each location. These geotechnical boreholes were drilled to depths of between 11.5m and 13.1m below the existing site levels.

The fieldwork was completed in the full-time presence of a geotechnical engineer who nominated the sampling and testing locations and compiled the borehole logs. The borehole locations, as shown on the attached Figure 1, were set out by taped measurements from the apparent site boundaries. The borehole logs are also attached, together with a glossary of the terms and symbols used in the logs.

The soil strengths were assessed from the recorded SPT ‘N’ values and hand penetrometer tests completed on clayey samples recovered from the SPT sampler. In the environmental sampling boreholes, the soil strengths were assessed from tactile examination of the samples.

The strength of the shale was initially assessed by examination of the recovered rock cuttings and core. The recovered rock chips and core was returned to our NATA registered laboratory for moisture content and Point Load Strength Index tests. The results of the moisture content and Point Load Strength Index tests are summarised in the attached Tables A and B. The core was also photographed in the laboratory and copies of the photos are provided with the borehole logs.

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

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in a relatively flat region with surface slopes less than about 2°. The site contains two showrooms of one and two storeys, and these appear to be in good condition. There is an asphaltic concrete car park in good condition to the east of these buildings.

Residential buildings are located in the north-eastern portion of the site and there is a concrete car park in poor condition in the central north part of the site.

The remainder of the site comprises an open yard for the storage of scaffold equipment. There are stockpiles of rubble and timber on this portion of the site.

The site is bounded to the north and south by Victoria Street and Parramatta Road respectively. There is a two storey brick building to the east of the site which appears in good condition, and this extends to the common boundary.

Duke Street is located to the west of the site, and the Main Western Railway Line is on the far side of the road. The railway is located at the crest of an embankment that has a height of about 3m to 4m above the site level, with the embankment comprising a 25° batter that slopes down from the tracks to the top of a 2m high concrete retaining wall; the retaining wall appeared to be in good condition when viewed from the site.

Adjacent to the south-west corner of the site is a fenced Sydney Water compound. It appears there is an in-ground pump station within that property.

3.2 Subsurface Conditions

The boreholes disclosed a subsurface profile comprising a relatively thin fill layer over silty clays which in turn overlie shale bedrock. The more pertinent features of the materials encountered are described below. For details of the materials encountered at each borehole, reference should be made to the borehole logs. A graphical summary of the strata encountered is presented in Figure 2.

Fill

Concrete pavements were encountered in BH6, BH10 and BH11 and had thickness ranging from 100mm to 140mm. In BH11 a second slab was encountered with about 0.2m of sand fill present between the slabs.

The fill was generally comprised either silty or gravelly clays and contained varying proportions of ash, slag and concrete fragments. The fill extended to depths ranging from 0.2m to 0.7m.

Silty Clay

Natural silty clay was encountered below the fill in all boreholes. The silty clay ranged from stiff and very stiff strength (with moisture content above its plastic limit) to hard (with moisture content below its plastic limit). In general, the lower strength clays were toward the eastern end of the site. The silty clays were of medium and high plasticity.

Shale Bedrock

Shale bedrock was encountered at depths ranging from 6.8m to 9.2m below the existing surface levels. The shale was initially of extremely low strength, improving to medium strength within 0.4m to 1.6m below the surface of the shale. Occasional bands of shale were of borderline medium and high strength. Defects within the shale generally comprised bedding partings, the occasional crushed seam and joints dipping from about 45° to 60°.

Groundwater

Slight groundwater seepage was noted in the deeper boreholes, except BH6, at depths from 6.5m to 8.6m below the existing site levels. The boreholes were dry on the completion of augering, though the introduction of flush water from the coring precluded further useful measurements of groundwater levels. A PVC standpipe was installed in a borehole augered to 7.5m depth adjacent to BH6. The water level after about 24 hours was at a depth of 5.1m below the ground surface level.

3.3 Laboratory Test Results

The laboratory test results correlated reasonably well with the field logging assessments of rock strength while the point load strength index tests indicated that the unconfined compressive strength (UCS) of the shale varied from 8MPa to 24MPa.

4 COMMENTS AND RECOMMENDATIONS

At the time of preparation of this report, the final details of the development were not known. We have therefore provided generalised recommendations for the construction of buildings varying from five to 37 storeys with between two and four levels of basement car parking. Some further information may be required at a later dated when the details of the proposed development are confirmed.

4.1 Excavation

We understand that the proposed basement will extend to the site boundaries, and could range in depth from 6m to about 14m. Such excavation will extend through the soils and for the deeper excavation, into the underlying shale bedrock.

The soil and shale of extremely low strength should be readily excavated using conventional hydraulic excavators, while shale to low strength should be rippable for 30 tonne excavators fitted with ripping tynes. Hydraulic rock breaker attachments or large bulldozers, such as a D11, with a ripping tyne will be required for removal of shale of low strength or better and will represent “hard rock” excavation techniques with UCS values for the shale expected to range up to about 24MPa. Considering the size of the site, a bulldozer with a ripping tyne may be the most cost effective way of excavating the shale bedrock.

"Hard rock" excavation techniques may consist of percussive or non-percussive techniques. Percussive techniques comprise the use of rock hammers while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting etc. Where percussive excavation techniques are adopted there is the risk that transmitted vibrations may damage nearby movement sensitive structures such as the adjoining house, retaining walls and boundary walls. Consequently, we recommend that the following measures be taken:

- Prior to the commencement of construction dilapidation reports should be completed on the adjoining properties to both the north and east of the site. The purpose of dilapidation reports is to provide a baseline condition survey of the structures. In this way a diligently prepared comprehensive dilapidation report can help protect the builder and owner from spurious claims relating to pre-existing damage and the owners of the adjoining structures have a baseline report on the condition of their structures prior to the commencement of construction should their structures suffer from construction related damage.
- During percussive excavation quantitative vibration monitoring must be completed. This monitoring may be either continuous or periodic, depending on the level of assurance required and will provide feedback to the excavation contractor on the suitability of the excavation equipment adopted. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation. Where non-percussive excavation techniques are adopted no vibration monitoring is required,
- Percussive excavation should be completed so that the excavation is progressively enlarged by breaking small wedges out of the face,
- Rock hammers should only be operated in short bursts to prevent amplification of vibrations.
- Where transmitted vibrations exceed prescribed limits, excavation techniques must be altered to reduce transmitted vibrations to within acceptable limits. This may mean that the size of percussive equipment used may need to be reduced, or non-percussive techniques adopted. Whether reducing the size of the percussive equipment is effective in controlling transmitted vibrations must be confirmed by further quantitative vibration monitoring.

Alternatively, non-percussive excavation techniques may comprise the use of rock saws, ripping tynes, rotary grinders etc. Where non-percussive excavation techniques are adopted dilapidation reports should still be considered but are not essential from a vibration perspective and quantitative vibration monitoring is not required.

The prescribed vibration limits that should be adopted on this site where percussive excavation techniques are adopted are set out in the Design Vibration Emission Goals attached to the rear of this report.

It is anticipated that groundwater inflows will typically occur at the interface between the soils and bedrock and through defects within the bedrock such as joints and bedding partings. We anticipate that control of groundwater flows by means of sump and pump should be quite feasible. Groundwater inflows are likely to be variable and greatest during and following rainfall events.

4.2 Retention

As it is proposed to excavate close to the common boundaries, prior to the commencement of excavation a shoring system must be installed to support the soils and shale bedrock of low strength or less. Shale bedrock of medium strength or greater should be able to be cut vertically and left unsupported provided no adverse defects are present.

Due to the proposed depth of excavation, it is anticipated that an anchored soldier pile wall with reinforced shotcrete infill panels will be used to support the soils and poorer quality bedrock. Where there are structures within a distance of twice the retained height of excavation from the perimeter of the excavation, the shoring should be designed for a trapezoidal earth pressure distribution with a maximum magnitude of lateral earth pressure of 8H kPa, where H is the depth of supported excavation in metres. A magnitude of earth pressure of 6H kPa may be adopted where there are no settlement sensitive structures or services within the zone of influence of the excavation. The design should adopt these maximum earth pressures over the central 60% of the depth of excavation, tapering to zero at the crest and toe of the excavation. Appropriate surcharge loads and hydrostatic pressures are additional to the above.

Where anchors extend beyond the boundary it will be necessary to obtain permission from the owners of the adjacent properties prior to installing the anchors. Anchors in shale of at least extremely low and medium strength may be designed for allowable bond values of 60kPa and 200kPa respectively. Anchors should have minimum free length and bond lengths of 4m and 3m respectively. Where neighbours do not agree to the installation of anchors below their properties the shoring system will require propping internally; alternatively it may be possible to apply to the court to force permission for anchor installation.

Where the shale bedrock is of medium strength or better it may be cut vertically and left unsupported provided it contains no adverse defects. However, it is likely that the shale will contain joints that, if adversely orientated will potentially affect the stability of the unsupported cuts. Consequently, where the retention system does not extend to the below Bulk Excavation Level (BEL) excavation will need to be completed in a carefully staged manner such that inspections of the cut face may be competed by a geotechnical engineer prior to excavation removing support to the cuts. In this regard excavation should extend no closer to the proposed cutlines than the height of the proposed cut for that stage of excavation (ie if the total height of unsupported cut is 1.5m then all excavation should be maintained a minimum horizontal distance of 3m from the proposed cutline).

Once excavation has been completed to the offsets from the proposed cutlines, slots can then be excavated perpendicular and extending to the proposed cutlines. These slots will allow the geotechnical engineer to observe the bedrock present at the proposed cut line for the presence of adverse defects whilst unexcavated rock is still in place providing support to the face should adverse defects be present which may allow failure of the cut. This process will be an iterative process and following the inspection of the slots further slots may be required to expose more of the face, remedial stabilisation works may be required or, where no adverse defects are present the excavation may be extended uniformly to the proposed cutline and the next stage of excavation commenced where excavation requires deepening. All unsupported cuts must be inspected by a geotechnical engineer every 1.5m of vertical cut.

We note this is a time consuming process and there are increased risks of instability compared to installing a full depth soldier pile wall, and for this reason, our experience is that it usually preferable to install a full depth shoring system prior to the commencement of the works.

Where adverse defects are present remedial measures will be required. Remedial measures typically comprise the installation of bolts and reinforced shotcrete. If continuous slickensided joints are present in the shale, major stabilisation such as the installation of high capacity anchors may be required. Whilst stabilisation using rock bolts will provide temporary support of the cut, it is anticipated that long term support will be provided by the structure of the building. Where clay or poorer quality shale seams are encountered in the medium strong bedrock that has been cut vertically and left unsupported, these seams should be treated by grubbing out and dry packing. Where this is the case seams should be grubbed out to the depth equal to their width (but not less than 20mm) and dry packed with a non-shrink grout. Weepholes should be left every 1m. Where clay or poorer quality shale seams have a thickness of greater than about 0.3m bolts and reinforced shotcrete should be used to support the seams.

From a geotechnical perspective groundwater seepage is anticipated to be adequately controlled by perimeter and internal drains with a sump and pump. In this regard we consider that the upper soils and poor quality bedrock may be supported with a soldier pile wall with the better quality underlying bedrock cut vertically and left unsupported. Notwithstanding this, it is possible that local authorities may consider that long term groundwater seepage is unacceptable and that a tanked basement will be required.

4.3 Footing Design

Where three or four basement levels are adopted it is expected that shale will uniformly be exposed over the base of the excavation and that pad and strip footings will be adopted. Where there will be two basement levels, with excavation to possibly 7m below the existing ground levels, shale bedrock is likely to be close to the basement excavation level, and so pad and strip footings founded on the shale would be feasible over most of the site. There will however be some areas, such as near BH1 in the north-western part of the site, where the pad footings would need to be relatively deep. It would be possible to have a mixture of pad footings where the shale is shallow and piled footings elsewhere.

Where piles or pad footings are founded with a nominal embedment of 0.3m into shale of very low strength, they may be designed for an allowable bearing pressure of 700kPa, while the allowable bearing pressure could be increased to 1000kPa for nominal sockets into low strength shale.

The better quality, medium strength, dark grey shale would provide a much better foundation and footings founded on this may be designed for an allowable bearing pressure of 3500kPa. Sockets into the medium strength shale may be designed for an allowable shaft adhesion of 350kPa provided the sockets are clean and rough. For these load bearing sockets, it will be necessary to use a sidewall grooving tool following the initial drilling. The use of such equipment is not standard practice in Sydney and so the use of grooving tools should be nominated on the structural drawings should load bearing sockets be adopted.

There was some slight seepage into the boreholes noted during the augering. There did not appear to be sufficient seepage to prevent cleaning of the pile base, though possibly enough that it will be necessary to pour concrete with a tremmie. As the seepage volumes are likely to be dependent on weather conditions prior to and during the works, we recommend that a trial pier be drilled just prior to the commencement of piling. Piles must be poured as soon as possible after drilling, and at the very latest on the completion of each days drilling. Having the sockets full of water will soften the shale, requiring redrilling of the sockets.

Where bearing pressures equal to or less than 1000kPa are adopted, all footings should be inspected by a geotechnical engineer to confirm that the design bearing pressure is achieved. For bearing pressures of 3500kPa, in addition to visual inspection by a geotechnical engineer one third of all footings from across the site should be spoon tested. All footings should be free from all loose and softened materials prior to pouring. Where water ponds in the base of the footings and these footings have been formed in shale bedrock of low strength or poorer the bedrock will soften and no longer be suitable for the design allowable bearing pressure. Where this occurs, the footing should first be pumped dry and then re-excavated to remove all loose and softened materials.

Higher bearing pressures, possibly to 5000kPa or 6000kPa may be feasible, though these would require the drilling of additional cored boreholes and full-time geotechnical inspection of all piled footings.

4.4 Basement Slab Design and Subsoil Drainage

As the basement floor slab will carry traffic loads, the design should incorporate a subbase layer of at least 100mm thickness of DGB20 or similar quality fine crushed rock. This layer should be compacted to at least 98% of Standard Maximum Dry Density (SMDD).

There is the possibility of groundwater seepage below the basement slab, especially for the deeper excavation and following wet weather. We therefore recommend that a network of subsoil drains be installed below the basement slab, with connection to a pumped sump for disposal of collected water. The subsoil drains should be installed around the perimeter of the basement, as well as on a grid or herringbone pattern through the basement area.

4.5 Further Work/Inspections

Prior to and during construction we recommend that the following further works/inspections be completed:

- Where three (depending on the proposed depth of excavation) or four basements are adopted further boreholes will be required to confirm the quality the shale bedrock for the design of footings,

- Dilapidation reports on the properties to the north and east of the site to provide a baseline record of the condition of these properties prior to the commencement of construction,
- During percussive excavation vibration monitoring will be required. This vibration monitoring may be either continuous or periodic, depending on the level of assurance required and will provide feedback to the excavation contractor on the suitability of the excavation equipment adopted,
- Where vertical cuts will be formed through shale bedrock of medium strength or better and left unsupported, inspection of the excavated slots and cut faces every 1.5m of vertical cut,
- Inspection of all footings prior to pouring to confirm that the design bearing pressure is achieved. Where bearing pressures of 3,500kPa or greater are adopted further testing/boreholes will also be required.

5 SALINITY

The site is located in an area where soil and groundwater salinity may occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. The local council has guidelines relating to salinity issues which should be checked for relevance to this project.

6 GENERAL COMMENTS

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

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

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

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

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

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.

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SOIL TEST SERVICES

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TABLE A
SUMMARY OF MOISTURE CONTENT TEST RESULTS

AS 1289 TEST METHOD 2.1.1

BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %
6	8.80-9.00	4.2
13	7.00-7.50	8.6

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SOIL TEST SERVICES

Ref No: 24711SY
Table B: Page 1 of 2

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	10.18-10.22	0.6	12
	10.87-10.91	0.7	14
	11.17-11.21	0.8	16
	11.80-11.84	0.6	12
	12.16-12.20	0.4	8
	12.74-12.77	0.5	10
	13.00-13.06	0.5	10
6	9.35-9.38	1.0	20
	9.86-9.90	0.6	12
	10.32-10.35	0.5	10
	10.78-10.81	0.7	14
	11.16-11.20	0.8	16
	11.75-11.79	0.5	10
	12.18-12.21	0.7	14
10	9.08-9.12	0.7	14
	9.75-9.78	0.7	14
	10.18-10.22	0.5	10
	10.77-10.81	0.7	14
	11.16-11.19	0.6	12
	11.77-11.81	0.4	8
11	8.88-8.91	0.5	10
	9.16-9.20	1.2	24
	9.89-9.91	0.6	12
	10.24-10.27	0.9	18
	10.86-10.89	0.9	18
	11.26-11.29	0.6	12

NOTES: **SEE PAGE 2**

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Table B: Page 2 of 2

TABLE B
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
13	9.24-9.27	0.7	14
	9.87-9.89	1.0	20
	10.25-10.28	0.6	12
	10.78-10.81	0.7	14
	11.16-11.19	0.6	12
	11.90-11.93	0.5	10

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = $20\ I_{S(50)}$

BOREHOLE LOG

Borehole No.
1
1/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER
JK550
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	■					0	▩		FILL: Gravelly clay, medium plasticity, brown, with concrete, brick and fibro fragments, timber and fine to coarse grained sand.	MC>PL			
	■			■	N = 18 5,8,10		▧	CL	SILTY CLAY: medium plasticity, grey mottled orange brown and red brown.	MC<PL	H	- >600 >600	
						1	▧		SILTY CLAY: low to medium plasticity, grey mottled orange brown and red brown.	MC>PL	VSt		
				■	N = 14 6,6,8	2	▧					250 420 300	
							▧	CH	SILTY CLAY: high plasticity, grey mottled red brown and orange brown.				
▼ ON COMPLETION OF CORING				■	N = 13 4,6,7	3	▧					360 250 360	
						4	▧	CL	SILTY CLAY: medium plasticity, grey mottled red brown and orange brown.				
				■	N = 18 5,8,10	5	▧					320 310 250	
							▧				St- VSt		
				■	N = 6 2,3,3	6	▧					100 120 250	
▶						7	▧		as above, but with fine to coarse grained ironstone gravel bands.				

BOREHOLE LOG

Borehole No.
1
2/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER JK550
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

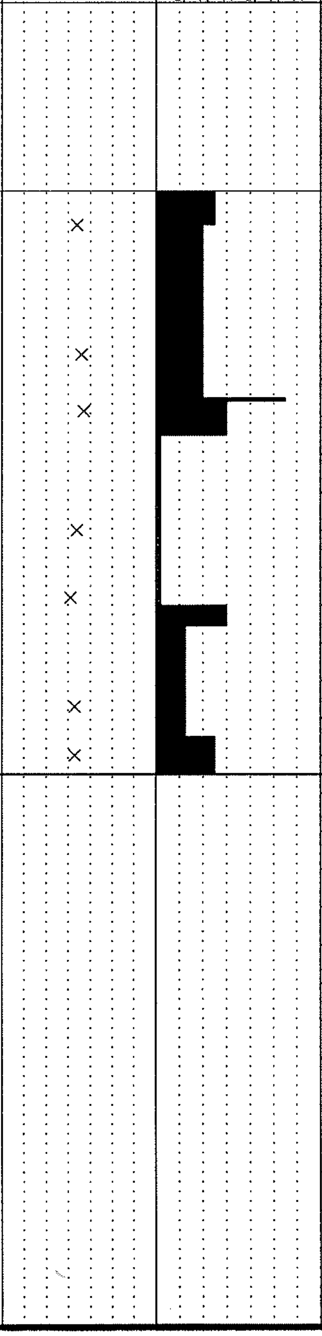
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■	N = 22 8,10,12	8			SILTY CLAY: medium plasticity, grey mottled red brown and orange brown, with fine to coarse grained ironstone gravel bands.	MC>PL	St-VSt		
						9		-	SHALE: grey mottled orange brown.	XW	EL	-	
									SHALE: grey brown.	DW	L-M		LOW 'TC' BIT RESISTANCE
						10			SHALE: dark grey.	SW	M		LOW TO MODERATE RESISTANCE
						11 12 13 14			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
1
3/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP	**Core Size:** NMLC	**R.L. Surface:**
Date: 28-7-04	**Inclination:** VERTICAL	**Datum:**
Drill Type: JK550	**Bearing:** -	**Logged/Checked by:** A.H./PW



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		9		START CORING AT 10.0m						
FULL RET-URN		10 11 12 13		SHALE: dark grey, with thin light grey bands, bedded at 0°.	SW	M			- Be, 0°, Un, R - Be, 0°, Un, R - J, 20°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Cr, 0°, 3mm.t - Cr, 0°, 5mm.t - J, 20°, Un, R - J, 80°, Un, R	
		14 15		END OF BOREHOLE AT 13.10m						

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
1/1

BOREHOLE LOG

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Method: HAND AUGER
R.L. Surface:
Date: 28-7-04
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION	■					0	▩		FILL: Sandy gravelly clay, low to medium plasticity, grey brown, with timber, fine to medium grained igneous gravel, slag and charcoal.	MC>PL			
	■						▨	CL-CH	SILTY CLAY: medium to high plasticity, grey mottled orange brown and red brown.	MC<PL	(H)	-	
						1			END OF BOREHOLE AT 0.6m				
						2							
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
3
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER
JK550
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

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						0	▩		FILL: Silty clay, medium plasticity, brown, with fine to medium grained igneous gravel and rootlets.	MC<PL			GRASS COVER
	■						▨	CL-CH	SILTY CLAY: medium to high plasticity, orange brown mottled grey.	MC>PL	(H)	-	
						1	▨		as above, but grey mottled orange brown and red brown.				
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER
JK550
Logged/Checked by: A.H./PW
R.L. Surface:
Datum:

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	■					0			FILL: Silty clay, medium plasticity, grey brown mottled orange brown, with fine to coarse grained igneous and quartz gravel.	MC<PL			
	■							CL-CH	SILTY CLAY: medium to high plasticity, mottled orange brown and grey.	MC<PL	(H)	-	
						1		CH	SILTY CLAY: high plasticity, grey mottled orange brown.				
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
5
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER
JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION	■					0	▩		FILL: Silty clay, low plasticity, grey brown, with fine to coarse grained shale and igneous gravel, timber and a trace of charcoal.	MC>PL			
	■					1	▨	CH	SILTY CLAY: high plasticity, grey mottled orange brown.	MC>PL	St-VSt	180 200 190	HP TESTING CARRIED OUT ON RECOVERED AUGER SAMPLES
						2 3 4 5 6 7			END OF BOREHOLE AT 1.5m				

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BOREHOLE LOG

Borehole No.
6
1/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 27-7-04
Method: SPIRAL AUGER JK250
Logged/Checked by: A.H./PW
R.L. Surface:
Datum:

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			CONCRETE: 100mm.t				NO APPARENT REINFORCEMENT
	■							-	FILL: Gravelly clay, medium plasticity, dark grey, fine to coarse grained igneous gravel, with slag and concrete fragments.	MC>PL	-	-	
	■			■	N = 7 3,3,4			CH	SILTY CLAY: high plasticity, grey mottled red brown and orange brown.	MC>PL	St	150 160 170	
						1					VSt		
				■	N = 12 5,6,6							250 300 200	
						2							
											St		
▼ ON COMPLETION OF CORING													
				■	N = 10 3,5,5	3							
						4							
				■	N = 14 4,5,9						VSt	150 200 210	
						5							
									as above, but with fine to coarse grained ironstone gravel bands.				
				■	N > 18 11,18/ 150mm REFUSAL	6					H	450 450	
								-	SHALE: grey mottled orange brown.	XW	EL		
										DW	L		VERY LOW 'TC' BIT
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
6
2/3

BOREHOLE LOG

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 27-7-04
Method: SPIRAL AUGER JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
							▤		SHALE: grey mottled orange brown.	DW	L		RESISTANCE
							▤				M		LOW RESISTANCE
						8	▤						
				■			▤				M-H		MODERATE RESISTANCE
						9	▤						
									REFER TO CORED BOREHOLE LOG				
						10							
						11							
						12							
						13							
						14							


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JOB No. 18756SP BH6 START AT 9.19m
9
10
11
12
END AT 12.27m

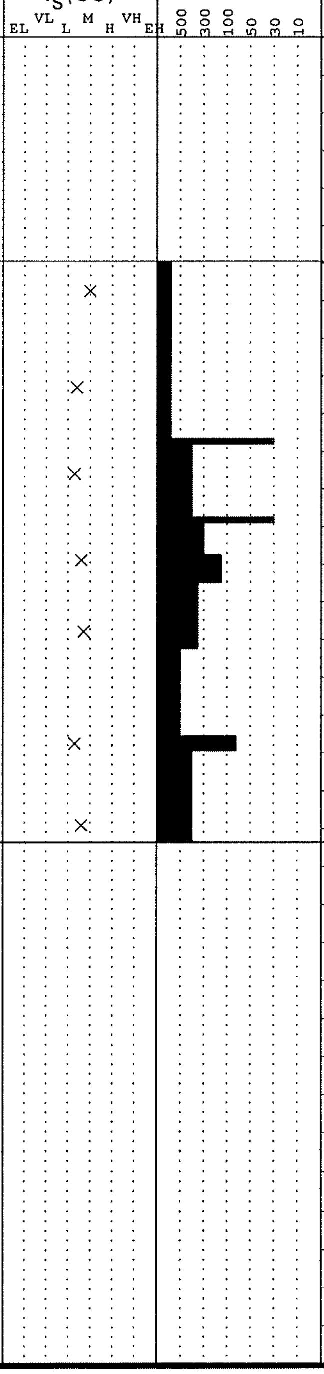
CORED BOREHOLE LOG

Borehole No.
6
3/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Core Size: NMLC
R.L. Surface:
Date: 27-7-04
Inclination: VERTICAL
Datum:
Drill Type: JK550
Bearing: -
Logged/Checked by: A.H./PW

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8								
		9		START CORING AT 9.19m						
FULL RET-URN		10 11 12		SHALE: dark grey, with thin light grey bands, bedded at 0°.	SW	M			- Cr, 0°, 3mm.t - J, 30°, Un, R - J, 60°, Un, R - Cr, 0°, 3mm.t - Cr, 0°, 5mm.t - Be, 0°, Un, R - J, 20°, Un, R - Cr, 60mm.t - Be, 0°, Un, R - Be, 0°, Un, R	
		13 14		END OF BOREHOLE AT 12.27m						



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BOREHOLE LOG

Borehole No.
7
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER
JK250
Logged/Checked by: A.H./PW
R.L. Surface:
Datum:

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	■					0	▩		FILL: Silty clay, medium plasticity, brown, with fine to coarse grained sandstone gravel, brick and concrete fragments and rootlets.	MC<PL			WEED COVER
	■						▨	CL	SILTY CLAY: medium plasticity, orange brown.	MC<PL	(H)	-	
						1	▨	CL-CH	SILTY CLAY: medium to high plasticity, grey mottled red brown.				
						2 3 4 5 6 7			END OF BOREHOLE AT 1.5m				

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BOREHOLE LOG

Borehole No.
8
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER
JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Sandy clay, low plasticity, dark grey, with fibro, glass, fine to coarse grained gravel and charcoal fragments and rootlets.	MC<PL			WEED COVER
								CL	SILTY CLAY: medium plasticity, orange brown and grey.	MC<PL	(H)	-	
						1		CL-CH	SILTY CLAY: medium to high plasticity, grey mottled red brown and orange brown.				
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
9
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER
JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION	■					0	▩		FILL: Silty clay, medium plasticity, brown, with rootlets, and with a trace of fine to coarse grained igneous gravel.	MC<PL			
	■						▨	CL-CH	SILTY CLAY: medium to high plasticity, orange brown mottled grey.	MC<PL	(H)	-	
						1	▨		as above, but grey mottled red brown and orange brown.				
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
10
1/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 27-7-04
Method: SPIRAL AUGER JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			CONCRETE: 140mm.t				7mm DIAMTER REINFORCEMENT AT 100mm TOP COVER
	■							-	FILL: Silty clay, high plasticity, brown, with fine to coarse grained igneous gravel and concrete fragments.	MC>PL	-	-	
								CH	SILTY CLAY: high plasticity, grey mottled orange brown and red brown.	MC>PL	St	-	
	■			■	N = 5 1,2,3							150 160 140	
						1					VSt		
				■	N = 15 5,6,9							300 350 350	
						2							
▼ ON COMPLETION OF CORING													
				■	N = 19 6,8,11	3						200 200 320	
						4							
				■	N = 18 5,7,11							350 380 300	
						5							
				■	N = 13 4,6,7	6						350 300 250	
						7							

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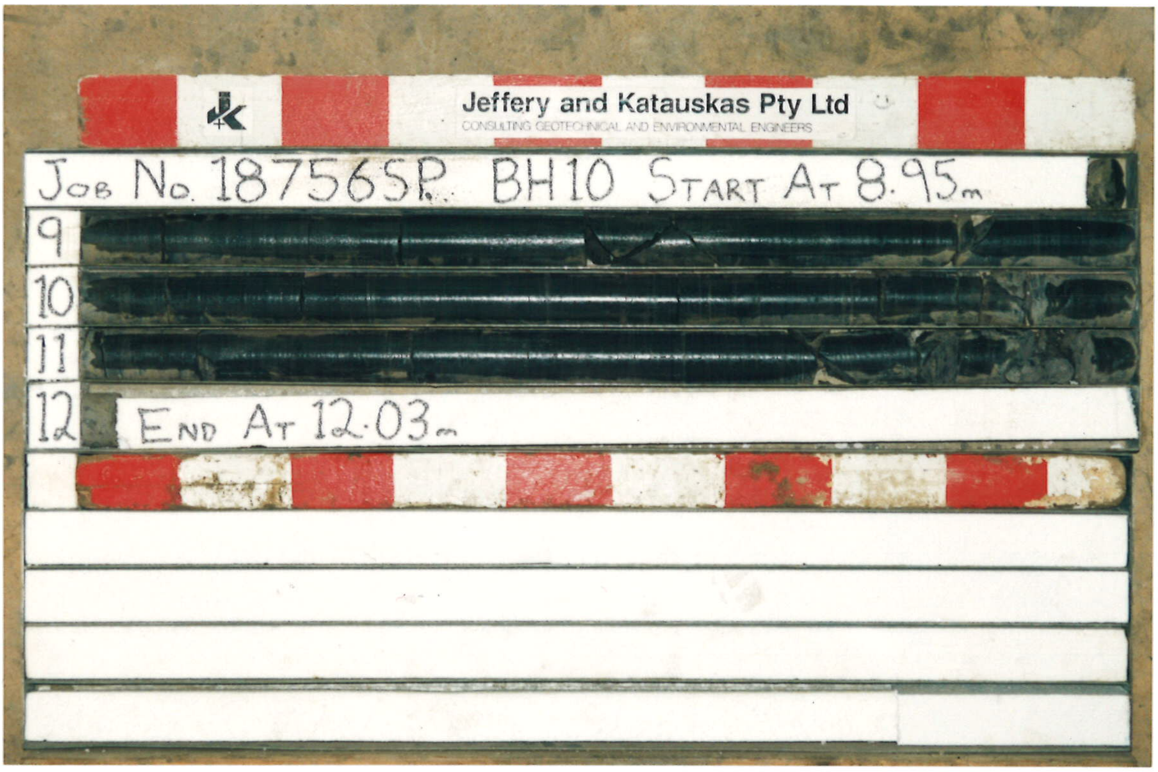
BOREHOLE LOG

Borehole No.
10
2/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 27-7-04
Method: SPIRAL AUGER JK250
Logged/Checked by: A.H./PW
R.L. Surface:
Datum:

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								CH	SILTY CLAY: high plasticity, grey mottled orange brown and red brown.	MC>PL	VSt		
▶				■	N > 18 8,18/ 150mm REFUSAL			-	SHALE: grey mottled orange brown.	XW	EL	-	
						8			SHALE: grey brown.	DW	VL-L		VERY LOW 'TC' BIT RESISTANCE
											L-M		
						9			SHALE: dark grey, with high strength iron indurated bands.	SW			MODERATE RESISTANCE WITH HIGH BANDS
									REFER TO CORED BOREHOLE LOG				
						10							
						11							
						12							
						13							
						14							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JOB No. 18756SP. BH10 START AT 8.95m
9
10
11
12
END AT 12.03m

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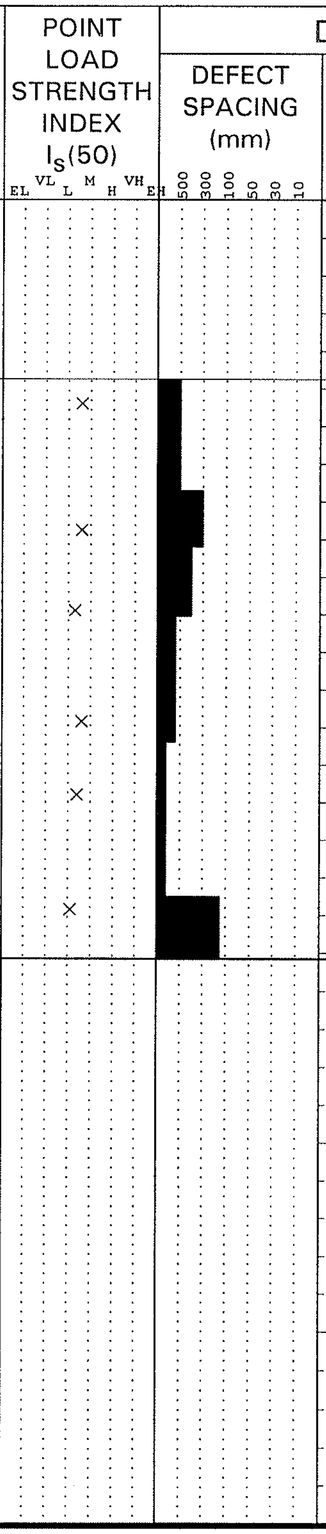
CORED BOREHOLE LOG

Borehole No. 10
3/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Core Size: NMLC
R.L. Surface:
Date: 27-7-04
Inclination: VERTICAL
Datum:
Drill Type: JK550
Bearing: -
Logged/Checked by: A.H./PW

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8		START CORING AT 8.95m				
FULL RET-URN		9		SHALE: dark grey, with thin light grey bands, bedded at 0°.	SW	M	- J, 60°, Un, R - J, 50°, Un, R	
		10					- Cr, 0°, 1mm.t - J, 40°, Un, R	
		11					- J, 45°, Un, R - J, 50°, Un, R - Cr, 40mm.t	
		12		END OF BOREHOLE AT 12.03m				
		13						
		14						



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Borehole No.
11
1/3

BOREHOLE LOG

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	■					0		CONCRETE: 100mm.t				7mm DIAMETER REINFORCEMENT AT 100mm TOP COVER
	■						-	FILL: Sand, fine to medium grained, orange brown.	M	-	-	
	■						-	FILL: Sand, fine to medium grained, orange brown, with fine to coarse grained gravel and concrete fragments.	-	-	-	
							-	CONCRETE: 150mm.t	MC>PL	St	-	
							CH	FILL: Silty clay, medium to high plasticity, grey brown, with fine to coarse grained gravel.	MC>PL			
	■			■	N = 10 1,2,8	1.5		SILTY CLAY: high plasticity, grey mottled red brown.		VSt	100 350 450	
ON COMPLETION OF CORING						2.6						
				■	N = 10 3,4,6	3					220 250 260	
				■	N = 24 3,10,14	4.5					280 300 250	
						6.5	-	SHALE: grey brown, with low strength bands, iron indurated bands and clay bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
						7						

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Borehole No.
11
2/3

BOREHOLE LOG

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER
JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
									SHALE: grey brown, with low strength bands, iron indurated bands and clay bands.	XW	EL		
▶										DW	L-M		
						8			SHALE: dark grey.		M		MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						9							
						10							
						11							
						12							
						13							
						14							


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JOB NO. 18756SP BH11 START AT 8.45m
8
9
10
11
END AT 11.5m

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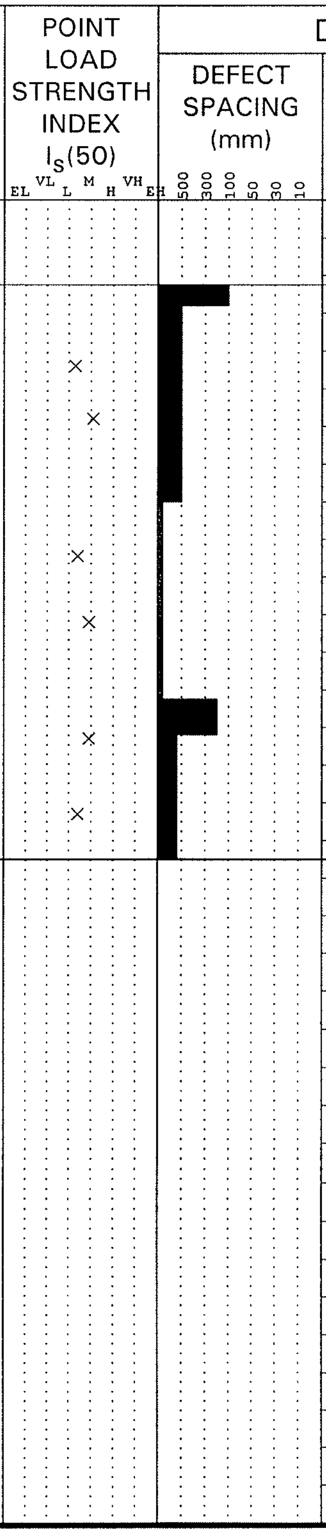
CORED BOREHOLE LOG

Borehole No.
11
3/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Drill Type: JK550
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8		START CORING AT 8.45m				
FULL RET-URN				SHALE: dark grey, with thin light grey bands, bedded at 0°.	SW	M	- Cr, 60mm.t	
		9					- XWS, 0°, 3mm.t	
							- J, 75°, Un, R	
		10						
							- J, 45°, Un, R	
							- J, 85°, Un, R	
		11						
				END OF BOREHOLE AT 11.5m				
		12						
		13						
		14						

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
12
1/1

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Method: SPIRAL AUGER JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

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						0			ASPHALTIC CONCRETE: 40mm.t	M	-	-	ROADBASE
					N = 4 2,2,2				FILL: Gravelly sand, fine to medium grained, dark grey, fine to coarse grained igneous gravel with a trace of clay.	MC>PL			APPEARS POORLY COMPACTED
								CH	FILL: Silty clay, medium plasticity, grey, with a trace of fine to coarse grained sansdtone gravel.	MC>PL	(VSt)	-	
						1			SILTY CLAY: high plasticity, grey mottled orange brown.				
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
13
1/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER JK250
R.L. Surface:
Datum:
Logged/Checked by: A.H./PW

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
	■					0			ASPHALTIC CONCRETE: 40mm.t	M	-	-	ROADBASE
	■								FILL: Gravelly sand, fine to coarse grained, brown, fine to coarse grained igneous gravel.				
	■			■	N = 3 1,1,2			CH	FILL: Gravelly sand, fine to coarse grained, brown, fine to coarse grained igneous gravel, with brick fragments and clay.	MC>PL	St	100 110 110	
						1			SILTY CLAY: high plasticity, orange brown.				
									SILTY CLAY: high plasticity, grey mottled red brown.		VSt		
				■	N = 15 3,6,9							320 390 360	
						2							
ON COMPLETION OF CORING													
				■	N = 14 7,7,7	3						320 400 350	
						4			as above, but with fine to coarse grained ironstone gravel bands.	MC<PL	H		
				■	N = 30 8,10,20							410 >600	
						5							
				■	N > 22 12,22/ 150mm REFUSAL	6		-	SHALE: grey mottled red brown, with iron indurated bands.	XW	EL	>600 >600	
						7							

BOREHOLE LOG

Borehole No.
13
2/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 26-7-04
Method: SPIRAL AUGER
JK250
Logged/Checked by: A.H./PW
R.L. Surface:
Datum:

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■		8			SHALE: grey brown.	DW	L		VERY LOW 'TC' BIT RESISTANCE
▶						9			SHALE; dark grey.	SW	M		LOW RESISTANCE WITH MODERATE BANDS HIGH RESISTANCE
						10 11 12 13 14			REFER TO CORED BOREHOLE LOG				


Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB No. 18756SP BH13 START AT 9.15m
9
10
11
12
END AT 12.1m

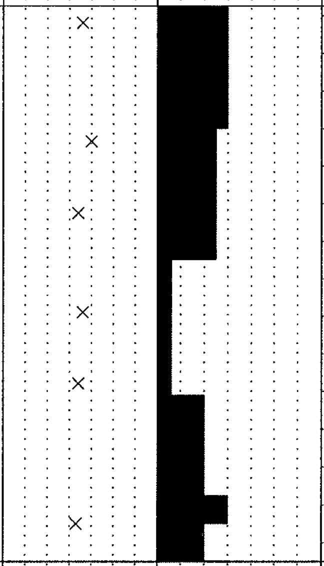
Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

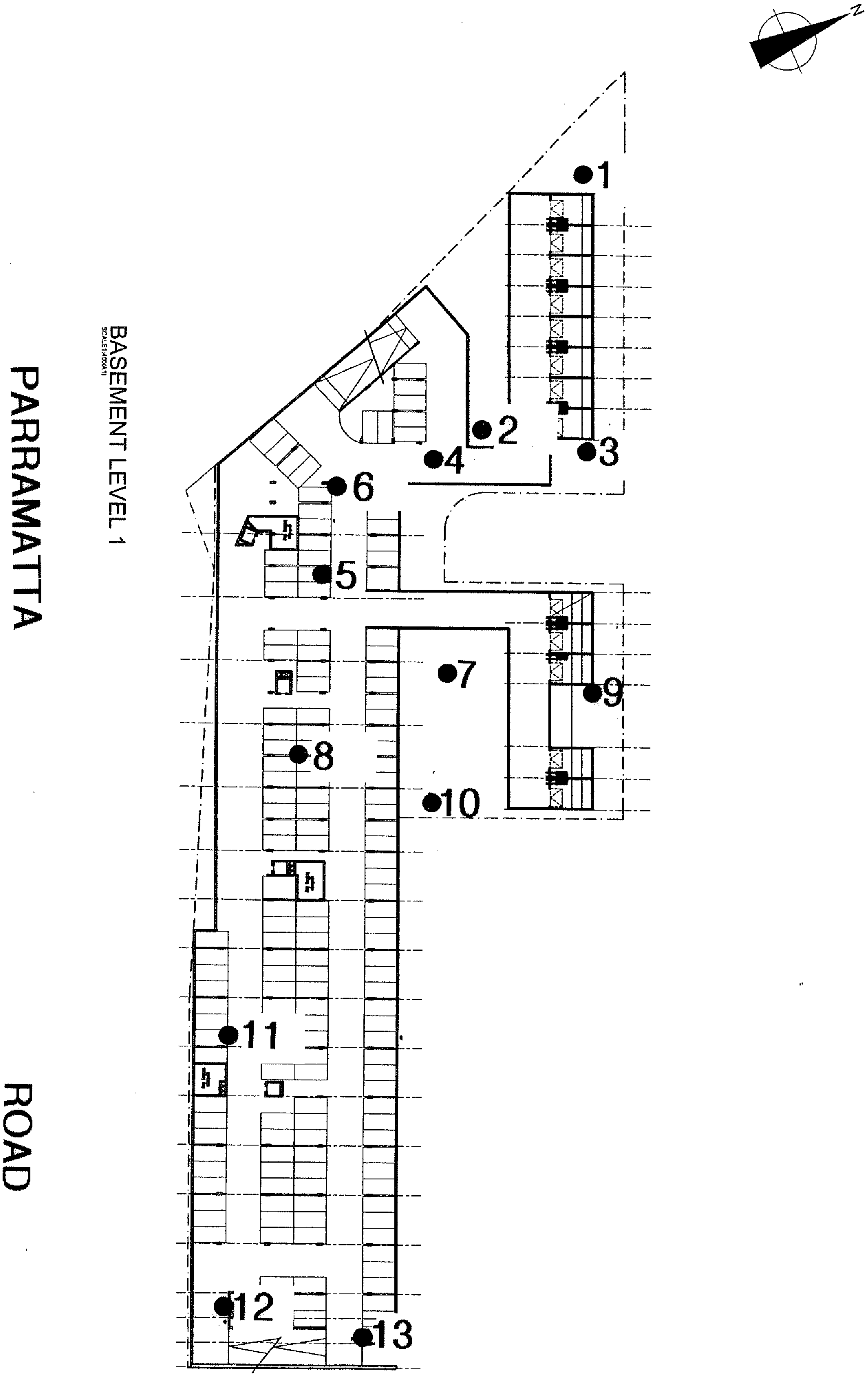
CORED BOREHOLE LOG

Borehole No. 13
3/3

Client: BERACI PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 171-189 PARRAMATTA ROAD, GRANVILLE, NSW

Job No. 18756SP
Date: 28-7-04
Drill Type: JK550
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface:
Datum:
Logged/Checked by: A.H./Pw

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8 9		START CORING AT 9.15m						
FULL RET-URN		10 11 12		SHALE: dark grey with thin light grey bands, bedded at 0°.	SW	M			- Be, 0°, Un, R - 2 x Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - 2 x Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Be, 0°, Un, R - Cr, 10mm.t	
		13 14		END OF BOREHOLE AT 12.1m						



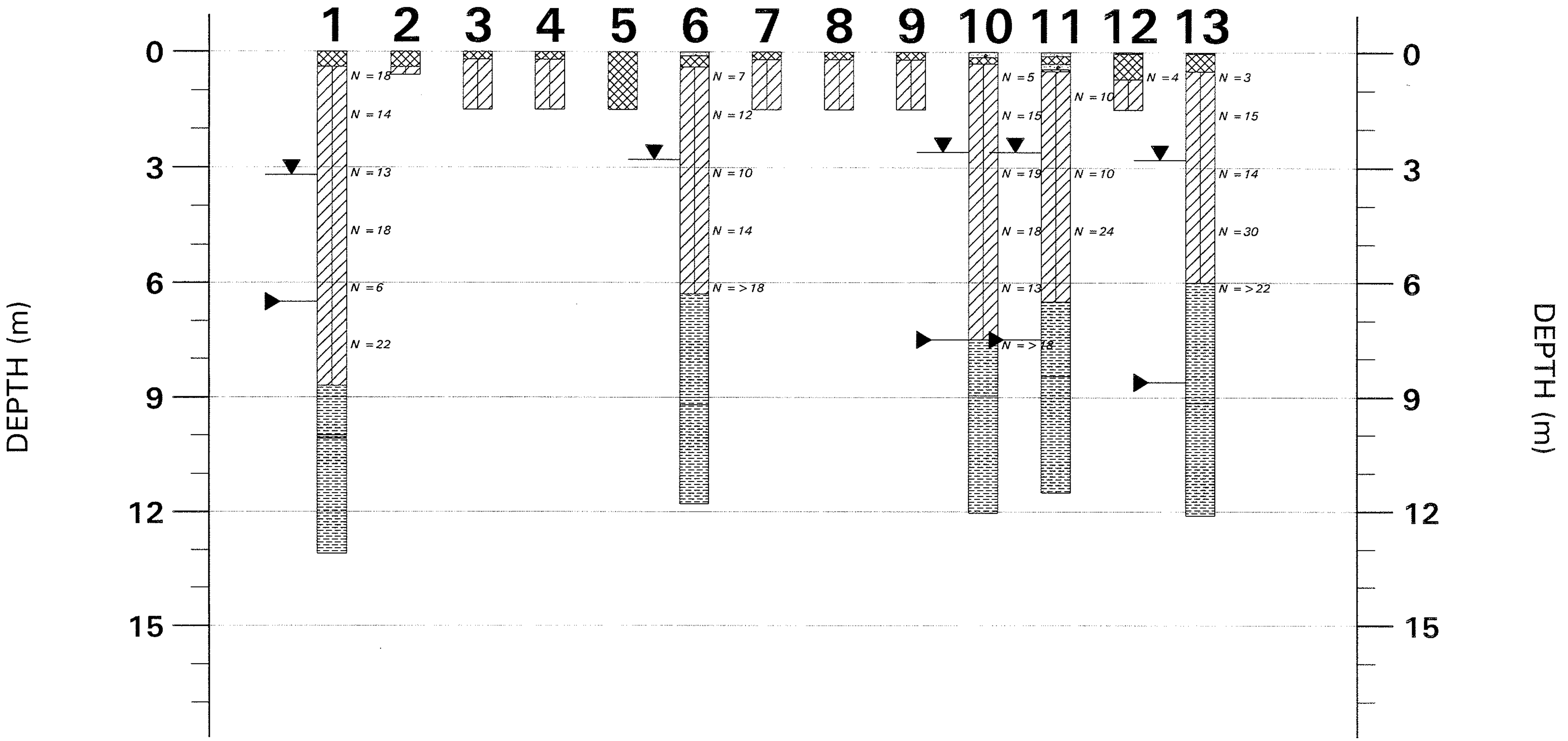
SCALE (M)
0
50

BOREHOLE LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No. 18756SP Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



▩ Fill

▨ Silty Clay

▤ Shale

░ Concrete

■ Asphaltic/ Bituminous Paving or Coal

▼ Observed water level

▶— Groundwater seepage level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

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

Jeffery and Katauskas Pty Ltd

Job No.: 18756SP

Figure No.: 2

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

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.

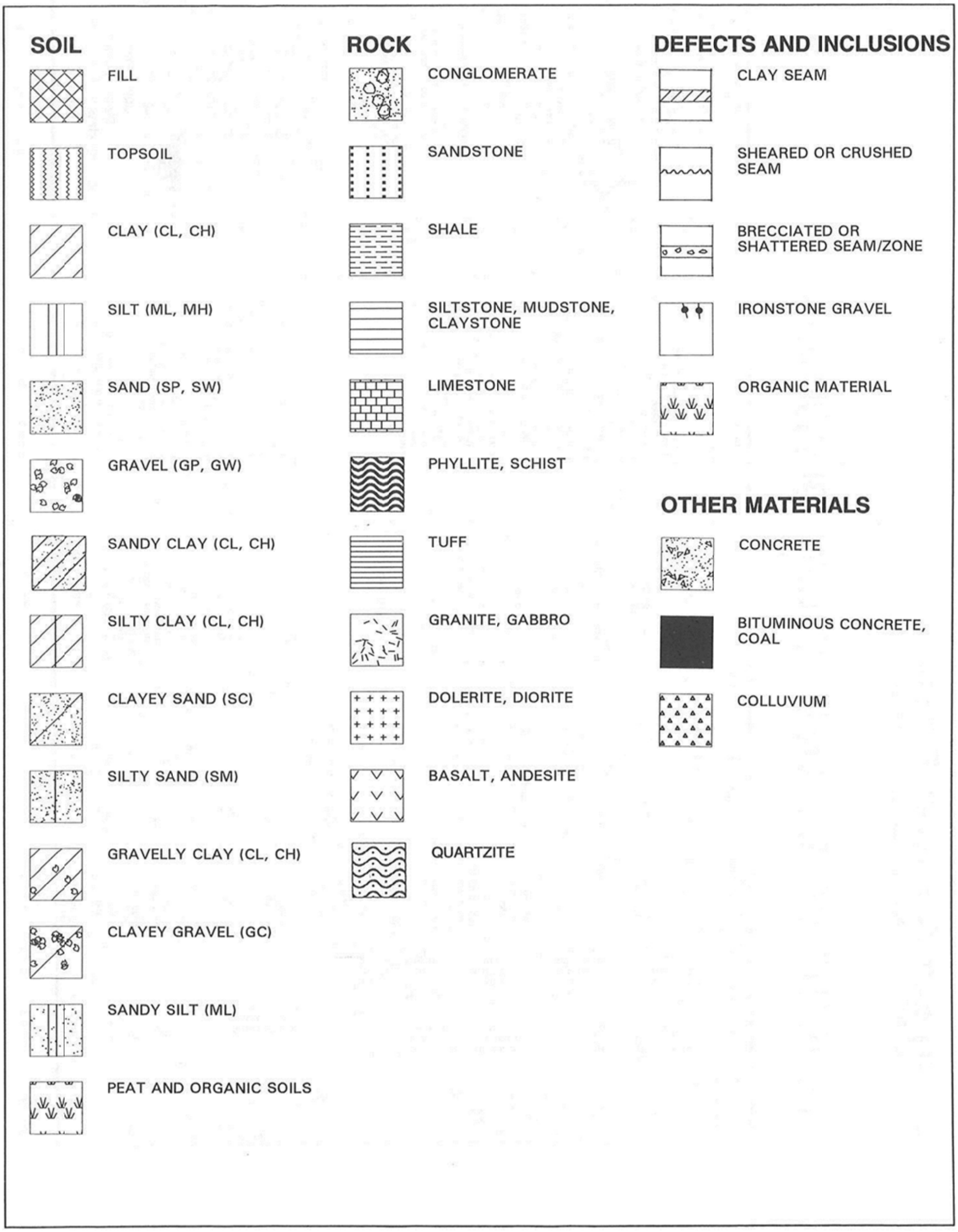
Requirements could range from:

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

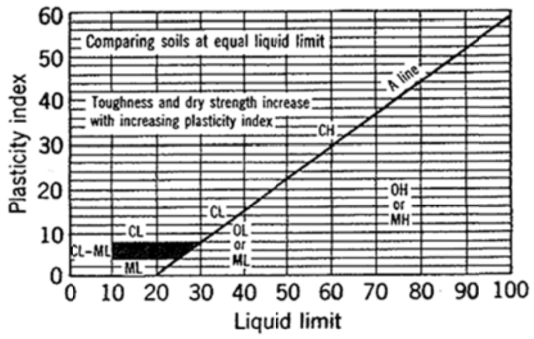
DEFECTS AND INCLUSIONS

- 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 sizeb (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	