REPORT TO

CECIL KOUTSOS

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

5 COMMONWEALTH PARADE, MANLY, NSW

Date: 8 July 2020

Ref: 23373SD2rpt Rev1

JKGeotechnics

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DOCUMENT REVISION RECORD

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 5 Commonwealth Parade, Manly, NSW. The location of the site is shown in Figure 1. This report was commissioned by Anna Soryal of Momentum Projects by “Letter of Acceptance” dated 3 July 2019. The commission was on the basis of our proposal (Ref. P49791PD) dated 27 June 2019.

We have been provided with updated Architectural drawings prepared by Platform Architects (Project Ref: CPM, drawing Nos. A1.00 to A1.06, A2.01 to A2.04, A3.01 to A3.04, A5.01 and A5.02, Revision S4.55), and a site survey plan prepared by Bee & Lethbridge Pty Ltd (Ref No. 12020, dated 11 September 2018).

We have also been provided with structural drawings for the existing contiguous pile wall along the northern site boundary prepared by R.J. Pearce & Associates Pty Ltd (Drawing Nos. 9945/01, dated November 1999).

Based on the above information, we understand that the proposed development will include demolition of the existing building and structures on site and construction of a four storey residential building over a single basement level. A centrally located lift, and an on-site detention (OSD) tank adjacent to the north-western corner of the basement are also proposed. Excavation to a maximum depth of about 7.6m will be required to achieve the basement design finished floor level at RL 10.59m, excavation to about 5.3m depth will be required for the OSD tank and localised deeper excavation to a maximum depth of about 1.5m below the basement level has been assumed for the proposed lift over run pit.

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

In 2009, JK Geotechnics completed a geotechnical investigation at the site for a proposed residential development, construction of which never eventuated. The results of our previous investigation have been used as a basis for comments and recommendations on excavation conditions, shoring system type and design parameters, footing design, soil aggression, on-grade floor slabs and drainage.

We note that JK Geotechnics (trading as Jeffery and Katauskas) carried out a geotechnical investigation on the neighbouring site to the north (No. 1 to 3 Commonwealth Parade). The borehole logs from this investigation are presented in Appendix B.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 11 and 14 September 2009 and comprised the hand auger drilling of six boreholes (BH1 to BH6) to refusal depths between 0.3m and 1.6m below the existing grade. BH3 and BH6 were then extended into the bedrock using rotary diamond core drilling techniques using our portable Melvelle rig, to final depths of 6.00m and 9.66m respectively. Six Dynamic Cone Penetration (DCP) tests (DCP1 to DCP6) were carried out to refusal depths between 0.7m and 1.65m. The test locations, as indicated on attached Figure 2, were set out using taped measurements from existing surface features. The surface RLs at the investigation locations were estimated by interpolation between spot heights shown on the provided survey plan and are therefore approximate. The survey datum is the Australian Height Datum (AHD). Figure 2 is based on the survey plan.

The degree of compaction, relative density and strength of the subsoils were assessed by interpretation of the DCP test results and hand penetrometer readings. We note that refusal of the DCP equipment often indicates the depth to the underlying bedrock, however, due to the equipment's limitations, it may also refuse on obstructions within fill, tree roots, ironstone gravel bands, other 'hard' layers within the soil profile, and not necessarily on bedrock. The strength of the underlying bedrock which was cored, was assessed by examination of the recovered rock core, and subsequent correlation with laboratory Point Load Strength Index testing. Groundwater observations were made during and on completion of hand augering, and during and shortly after completion of core drilling.

A slotted standpipe was installed into BH6 to allow longer term groundwater monitoring by others. For further details on the investigation procedure adopted, reference should be made to the attached Report Explanation Notes.

The recovered core bedrock was returned to Soil Test Services (STS) for photographing and Point Load Strength Index ($I_{S(50)}$) testing. Using established correlations, the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the $I_{S(50)}$ results. The Point Load Strength Index test results are summarised in the attached STS Table A and are also shown graphically on the borehole logs. The recovered core was also photographed, and copies of the photographs are presented with the cored borehole logs.

A selected soil sample was submitted to a second NATA registered laboratory (Envirolab Services Pty Ltd) for soil pH, chloride and sulphate content testing. Those results are summarised in Appendix A. Contamination screen testing of the site soils was not within the agreed scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located over the lower reaches of an east facing hillside, is roughly rectangular in plan shape being between 9m and 15m wide (north to south) by between 37m and 39m deep (east to west), and slopes by about 7° to 8° from The Crescent along the western boundary to Commonwealth Parade along the eastern boundary.

At the time of the fieldwork, the central and eastern portion of the site was occupied by a three-storey sandstone block and brick residential building, whilst a brick garage was located over the south-west corner. Based on a cursory inspection, the building appeared in relatively good external structural condition. However, the garage was in poor condition with several cracks observed. The rear yard to the west was concrete paved and included several brick retaining walls approximately 1m high. Concrete steps and pathway led down either side of the building to the front eastern yard which was generally grassed surfaced, and was supported above the Commonwealth Parade footpath by a brick wall up to about 1.5m high.

A seven-storey brick unit building was located approximately 1.2m beyond the southern site boundary, and a two and three storey rendered building was located about 1m beyond the northern site boundary. The basement to the northern building appeared to abut the common site boundary. The neighbouring buildings appeared in good condition when viewed from within the subject site. Ground levels across the southern site boundary appeared similar, except along the eastern and western ends, where the neighbouring property was up to 2m higher than the adjoining ground on the subject site, and was retained by a brick boundary wall.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is located in an area which is underlain by Hawkesbury Sandstone. The investigation has disclosed a generalised subsurface profile comprising surficial fill and residual sandy clay/clayey sand, over sandstone bedrock at relatively shallow depths. Reference should be made to the attached borehole logs and DCP test results for detailed subsurface conditions at specific locations. A graphical borehole summary is presented in Figure 2 and a summary of the subsurface conditions as encountered is presented below:

Pavement

A concrete surface 70mm thick was encountered at BH2, BH3 and BH5. A concrete surface 140mm thick was encountered at BH6.

Fill

Fill comprising silty sand, sand and gravelly sand generally with sandstone, ironstone and igneous gravel, and concrete fragments, was encountered to depths between 0.3m (BH2) to 1.0m (BH6). Based on the DCP results, the fill appeared poorly compacted.

Residual Soils
Residual sandy clay, clayey sand and silty clay were encountered below the fill in BH1, BH4 and BH6. The silty clay was of high plasticity and stiff to very stiff strength, the sandy clay was of low plasticity and stiff strength, and the sandy soils were generally loose.

Weathered Sandstone Bedrock
Based on the DCP results and the cored boreholes, weathered sandstone bedrock was encountered at depths between 0.7m (BH2) and 1.72m (BH6). The cored borehole in BH3 indicated that on first contact, the sandstone was of extremely low strength and improved to very low to low, medium and high with depth. Medium strength sandstone was encountered at a depth of 2.9m. A 0.63m thick core loss zone was encountered at a depth of 1.9m and a 0.25m thick core loss zone was encountered at a depth of 4.65m. Other defects within the sandstone rock mass included weathered seams and bed partings. Within BH6, the sandstone was generally of low and medium strength, improving to high at a depth of 9.3m. A 2.47m thick core loss zone was encountered at a depth of 3.25m and a 0.86m thick core loss zone was encountered at a depth of 6.3m. Several less significant core loss zones were also encountered. Other defects encountered within the rock mass included bed partings, extremely weathered seams, clay seams, and variably inclined joints. Given the thickness of the core loss zone, it is highly probable that BH6 intersected a sub-vertical joint. This would also explain difficulties which were experienced during the drilling process.

Groundwater
Groundwater seepage was encountered at a depth of 0.6m whilst drilling BH1. No further groundwater was encountered during auger drilling. As water is added to the borehole during core drilling, further groundwater observations were limited to estimation of a percentage recycled return which was generally 0% in BH3 and 100% in BH6 indicating a relatively permeable rock mass in places. As the neighbouring basement is drained it is expected that any groundwater level is drawn down to the basement level.

3.3 Laboratory Test Results

The point load strength test results (STS Table A) correlated well with the field logging assessments of rock strength, confirming the variable rock strength ranging from low to high, with an estimated Unconfined Compressive Strengths (UCS) ranging between, less than 1MPa and 32MPa, but more typically between 4MPa and 16MPa. We note however that due to the fractured nature of the core, it is likely the strength results are biased toward the more intact bands of rock which are usually of higher strength.

The soil aggression tests presented in the Envirolab Services Certificate of Analysis show a neutral to slightly acidic soil (pH 6.0), low sulfate content (less than 25mg/kg), very low chloride content (less than 100mg/kg). These conditions relate to a 'Mild' and 'Non-Aggressive' exposure classifications for concrete in contact with the soil.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

We consider the following to be the primary geotechnical issues for the proposed development:

- Excavation for the proposed development will extend to the northern and southern site boundaries, we therefore consider the principal geotechnical issue is the requirement to maintain stability of the neighbouring structures and surface levels adjacent to the excavation.
- The presence of the existing contiguous pile wall along the northern site boundary which is potentially founded slightly above the proposed bulk excavation level within the subject site. Excavation will need to be carefully staged and assessed by structural and geotechnical engineers.
- The depth of the current investigation only extends below the proposed BEL in BH3 at the eastern end of the proposed basement. Further geotechnical investigation of the site will be required to obtain a more accurate representation of the subsurface profile, particularly the sandstone bedrock quality, as the basis for the detailed design.

Based on the provided structural drawings for the northern neighbouring property (No. 1-3 Commonwealth Parade), we understand that during construction a contiguous pile wall was installed along the common boundary and forms the southern wall of the neighbouring basement car park. Assuming the wall was constructed as designed, the wall was founded approximately 0.8m below the finished basement level at RL 10.5m, which is about 0.1m higher than the proposed basement BEL at RL10.4m, and was laterally supported by temporary rock anchors which extend into the subject site. There is some uncertainty regarding the extent of the contiguous pile wall.

4.2 Excavation Conditions

4.2.1 Excavation

The following preliminary recommendations should be read in conjunction with the 'Excavation Work – Code of Practice' by Safe Work Australia (July 2012).

Prior to the commencement of rock excavation, a dilapidation report should be prepared for the adjoining buildings to the north and south. In addition, Council may also require that dilapidation survey reports be completed on their assets lining the street frontages, i.e. the pedestrian stepped walkway (and retaining wall), the roadway surface, kerbs and gutters. These reports should identify any existing cracks or other defects, including their location, length and width, together with photographs of the cracks. The reports should be signed by the owners of the neighbouring buildings to confirm they present a fair record of the existing conditions so they can be used as a benchmark on which to assess any claims for excavation induced damage.

We understand the excavation for the proposed basements will extend to depths between about 3m and 7.6m below the existing surface levels. This excavation will extend through the fill and residual soils, and be predominantly within sandstone bedrock. While much of the upper portion of the bedrock will be of banded strength, substantial parts of the excavation will likely be in sandstone of medium or higher strength. The following recommendations are based upon our general experience, and the borehole logs and point load strength index test results sheets should be made available to the proposed excavation contractor so they can make their own assessment and confirm their equipment and proposed techniques are suitable for the strength of the rock to be excavated. Great care will be required when anchors from the adjacent site are exposed as, although they should be destressed, they may still be engaged in the wall and must be cut or carefully withdrawn.

The soils and extremely low and very low strength bedrock should be able to be excavated using a large hydraulic excavator, possible with some ripping. The excavation of low and higher strength sandstone will require the use of hard rock excavation techniques such as rock breaker attachments to large tracked excavators. Alternatively, the excavation could be undertaken by grid sawing using large excavator mounted rock saws, and ripping the sawn blocks from the excavation. Rock breaker equipment would also be required for breaking up of blocks and boulders, trimming rock excavation faces, and for detailed rock excavations (such as for footings or buried services).

Excavation using rock breakers has the potential to cause damaging vibrations, and so full-time vibration monitoring must be undertaken during such excavation. The vibration monitoring should be completed with instruments that measure vibrations in three dimensions and calculates their vector sum. The monitoring equipment must also have audible and visible alarms to warn when potentially damaging vibrations occur. If these threshold vibration levels are exceeded, it will be necessary to cease work and assess why the limits were exceeded, and implement measures to prevent ongoing exceedances which may require the adoption of alternative plant or excavation techniques. Tolerable vibration levels for residential structures are provided in the attached “Vibration Emission Design Goals”.

4.2.2 Groundwater Seepage

We expect that groundwater seepage will occur at the soil-rock interface and through joints and bedding planes within the rock mass, particularly following periods of heavy or prolonged rainfall. However, such seepage, if encountered, would be controllable using conventional sump pumping techniques.

We recommend that groundwater seepage into the excavation be monitored by site personnel and the results (approximate volumes, source, location, etc) be presented to the geotechnical and hydraulic engineers, so that any unexpected conditions can be timeously addressed. Further, a toe drain should be formed at the base of all cut rock faces in order to collect groundwater seepage and direct it to a sump for pumped disposal to the stormwater system.

4.3 Excavation Support and Retention

Prior to bulk excavation commencing, we recommend that a number of test pits be excavated along the northern and southern site boundaries to expose the existing boundary wall footings and their founding materials. The test pits should then be jointly inspected by the geotechnical and structural engineers to detail any underpinning which may be required.

Along the eastern and western sides of the excavation, temporary batters should not exceed 1 Vertical (V) in 1.5 Horizontal (H) in the sandy soil and 1 Vertical (V) in 1 Horizontal (H) in the clayey soil and weathered rock of very low strength or less. All surcharge loads such as from plant or material should be kept well clear of the crests of these batters.

The drawings indicate that the building at No 7 is founded well above the proposed basement level and therefore it is essential that the excavation and shoring works do not destabilise the building. The future investigation will enable the rock quality along this boundary to be carefully evaluated and any necessary stabilisation works designed and implemented.

We expect that good quality sandstone of low or higher strength for the proposed lift over run pit may be cut vertically. However, localised stabilisation measures may be necessary if adverse defects (such as inclined joints or bedding) or weak seams are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. We therefore recommend that these rock faces be inspected progressively by a geotechnical engineer at vertical intervals not exceeding 1.5m to identify adverse defects and propose appropriate stabilisation measures. At this stage the limited investigation data is indicating poor rock quality and provision for stabilisation works should be included in the project budget.

The proposed basement extends to the northern and southern site boundaries, and there will be insufficient space for the above temporary batters. Therefore it will be necessary to install a shoring system prior to excavation commencing, except where such shoring already exists on the adjacent site.

Along the northern boundary the existing contiguous pile wall will need to be assessed by the structural engineer to determine whether it will remain stable when the ground it is supporting is removed. The wall is probably tied at the top to the neighbouring ground floor slab. It will be important to determine the depth at which the contiguous piles are founded. If the neighbouring piles are founded above their basement level, this will result in the contiguous pile wall being supported on a thin 'plinth' of sandstone which would be considered to be unstable. If this was the case the piles would need to be underpinned down to below the proposed bulk excavation level. Further input will be required on this issue during construction by the geotechnical and structural engineers. Under no circumstances is any excavation to extend below the toe of the contiguous piles without prior geotechnical approval.

The assessment should include an attempt to source 'as built' drawings for the neighbouring basement. In-lieu of as built drawings a number of test pits should be excavated along the northern site boundary, during demolition, in an attempt to expose the extent of the neighbouring shoring wall. These test pits should be inspected by the structural engineer. Inspection of the adjacent basement is also required.

4.4 Shoring Design

The provided drawings indicate the southern side of the excavation and the northern sides of the excavation, where the existing contiguous wall in not present on the adjacent site, will be supported by solider pile walls with shotcrete infill panels. We consider the proposed shoring system to be suitable for this site and should extend the full-depth of the proposed basement. Where the footings for the southern boundary retaining wall are founded within the soil profile, to be confirmed by inspection of the test pits outlined in Section 4.3 above, it may be necessary to included additional intermediate piles between the solider piles to support the upper soil profile. The intermediate piles should be founded a nominal depth into the underlying sandstone bedrock and the top of the piles should be tied into the capping beam. The shoring piles will be installed through sandstone which in places has an Unconfined Compressive Strength (UCS) exceeding 20MPa. Drilling this strength of sandstone will require moderate to large piling rigs. The piling contractor should be provided with copies of the borehole logs and point load strength index test results so they can confirm their equipment will be suitable to be able to penetrate this rock.

We presume the shoring system will be laterally supported in the short term by a combination of temporary anchors and toe socket. During the excavation, reinforced shotcrete panels should be sprayed progressively with the excavation to support the soil and weathered rock between the piles, such that there is no more than 1.8m vertical face of material exposed below the shotcrete at any time. It will be necessary to install strip drains behind each panel of shotcrete to dissipate the pore pressures behind the shotcrete.

The shoring should be designed for a rectangular lateral earth pressure distribution of 6H kPa, where H is the depth of excavation in metres. However, the lower portion of the excavation which extends into low or higher strength bedrock, a uniform lateral pressure of 10 kPa should be adopted for the low or higher strength rock. Appropriate surcharge loads are additional to these pressures, and may be calculated using an at-rest lateral earth pressure coefficient of 0.5.

Where soldier piles are spaced more than about 1.8m apart, the reinforced shotcrete between the soldier piles should be designed as withstanding lateral pressure from the soil and weathered rock, and distributing the loads back to the piles. The reinforced shotcrete between the piles should be designed for a lateral pressure of 15kPa.

Where soldier piles have a spacing of at least 3 pile diameters, the toe socket founded within rock of at least low strength may be provisionally designed for an allowable lateral pressure of 350kPa for sockets of at least 1.5m below the proposed excavation level; this pressure accounts for the 3-dimensional effects of soldier piles. Where soldier piles with less than 3 pile diameter spacing are adopted, a reduced allowable lateral pressure of 200kPa should be adopted. The upper 0.5m of toe socket below any excavation (including footing or service trench excavations) should not be taken into account in the design of the toe sockets to account for possible over-excavation.

Where temporary anchors are used to support the excavation, it will be necessary to obtain permission from the neighbours of the adjoining properties prior to the installation of the anchors. Anchors should be designed with minimum free lengths and bond lengths of 4m and 3m respectively, and the bond zone should be entirely behind a line drawn upward at 1V in 1H from the toe of the excavation. Anchors bonded into the sandstone of low or higher strength may be designed for an allowable bond strength of 200kPa. All anchors should be proof loaded to at least 130% of their working load in the presence of a geotechnical or structural engineer independent of the contractor to confirm the anchors are holding their design load. Lift-off tests should be completed on all anchors following lock-off, and at least 25% of anchors should be subjected to further lift-off testing four days after lock-off. If these are showing any significant loss of load in any of the anchors, all anchors should then be subject to lift-off testing to ascertain whether further lateral support is required. The anchors should be preferably be let as a design and construct tender to allow innovation in the anchoring, and so the contractor is directly responsible for the performance of the anchors.

We assume the final support of the shoring system will be provided by bracing it from the floor slabs of the proposed structure.

4.5 Footing Design

The site presently classifies as Class ‘P’ in accordance with AS2870 in view of the relatively deep fill which was encountered. Where the fill is removed or reduced in thickness, a Class ‘S’ classification would be applicable.

The proposed bulk excavation will expose bedrock. For uniformity of support, we recommend that the entire new building be supported on bedrock. Pad or strip footings founded in sandstone bedrock of at least low strength may be designed for an allowable bearing pressure of 1,000kPa. Piles founded in sandstone bedrock below bulk excavation level may be designed for an allowable end bearing pressure of 1,000kPa. Individual piles or soldier piles may, in addition, be designed for an allowable shaft adhesion value of 100kPa for rock sockets below bulk excavation level. Due to the numerous ‘No Core’ zones in the boreholes probably indicating extremely weathered bands or decomposed shale bands, we do not recommend founding footings at the crest of a cut face as the cut face may not be stable. Further investigation may reveal better conditions but it would be unwise to assume so from the information available to date.

We note that only one of the boreholes (BH3) in the current investigation extends below the proposed BEL, we consider that higher allowable bearing pressures up to 3,500kPa may be achievable following drilling of additional cored boreholes which extend to depths of about 3m below the proposed BEL (a depth comparable to the zone of influence of the basement pad/strip footings and piles)

All footings should be inspected by a geotechnical engineer prior to pouring to confirm that adequate founding has been achieved.

4.6 Basement Floor Slab

The proposed basement floor slab will directly overlie sandstone bedrock and therefore underfloor drainage must be provided. The underfloor drainage should comprise a strong, durable, single sized washed aggregate (such as 'blue metal' gravel) and should connect with the wall drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

Construction joints in the concrete on-grade floor slab should incorporate dowelled or keyed joints. The proposed access ramp is likewise expected to directly overlie bedrock.

4.7 Soil Aggression

As stated in Section 3.3 above, the soil aggression tests presented in the Envirolab Services Certificate of Analysis show a neutral to slightly acidic soil with very low sulfate and chloride contents. These conditions relate to a 'Mild' and 'Non-Aggressive' exposure classifications for concrete in contact with the soil.

4.8 Further Geotechnical Input

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

- Additional geotechnical investigation including additional cored boreholes extending below the proposed bulk excavation levels.
- Dilapidation surveys of neighbouring buildings to the north and south.
- Inspection of test pits exposing boundary wall footings and founding materials.
- Investigation of the adjacent pile wall.
- Geotechnical footing inspections.
- Proof-testing of anchors, if appropriate.
- Monitoring of groundwater into bulk excavation.

5 GENERAL COMMENTS

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

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. 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

Ref No: 23373Z

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
3	1.79-1.82	0.01	<1
	2.88-2.91	0.9	18
	3.21-3.25	0.8	16
	3.77-3.80	0.8	16
	4.36-4.40	0.2	4
	5.04-5.07	1.2	24
	5.78-5.81	1.6	32
6	1.78-1.80	0.5	10
	2.31-2.34	0.2	4
	2.83-2.85	0.5	10
	3.30-3.33	0.4	8
	5.97-6.00	0.1	2
	6.22-6.25	0.05	<1
	7.32-7.36	0.2	4
	7.76-7.79	0.5	10
	8.22-8.25	0.8	16
	8.78-8.80	0.5	10
	9.30-9.33	1.3	26

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 $I_{S(50)}$

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 13.2m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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
					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, with roots and fine to medium grained sandstone gravel.	M			GRASS COVER APPEARS POORLY COMPACTED
▶								CL/SC	SANDY CLAY/CLAYEY SAND: low plasticity, fine to medium grained, dark brown, with a trace of root fibres.	W/ MC>PL	St	-	RESIDUAL
						1		CL	SANDY CLAY: low to medium plasticity, orange brown.				HAND AUGER REFUSAL
									END OF BOREHOLE AT 0.9m				
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
2

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 15.8m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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					REFER TO DCP TEST RESULTS	0			CONCRETE: 70mm.t				
								-	FILL: Gravelly sand, fine to coarse grained, dark brown, with clay pipe and brick fragments.	M	-	-	APPEARS POORLY COMPACTED
			■						END OF BOREHOLE AT 0.3m				HAND AUGER REFUSAL
						1							
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
3
1/2

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 14.8m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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 AUGERING					REFER TO DCP TEST RESULTS	0			CONCRETE: 70mm.t				
								-	FILL: Gravelly sand, fine to coarse grained, dark brown, concrete and sandstone gravel.	M	-	-	APPEARS POORLY COMPACTED
										W			
						1							
									REFER TO CORED BOREHOLE LOG				50mm PVC STANDPIPE INSTALLED TO 6m DEPTH, SLOTTED BETWEEN 1.5m AND 6.0m DEPTH. GATIC COVER CONCRETED AT SURFACE
						2							
						3							
						4							
						5							
						6							
						7							


JEFFERY & KATAUSKAS PTY LTD
JOB NO: 23373Z
BH: 3
START CORING AT 1.47m
1
1.47m
CORE LOSS
2
CORELOSS 0.63m
3
4
CORELOSS 0.25m
5
END OF BH AT 6.00m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **3**

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z | **Core Size:** TT56 | **R.L. Surface:** ≈ 14.8m
Date: 14-9-09 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: MELVELLE | **Bearing:** - | **Logged/Checked by:** J.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
	1	START CORING AT 1.47m			
NO RET-URN		SANDSTONE: fine to medium grained, grey and orange brown.	XW	EL	
	2	CORE LOSS 0.63m			
	3	SANDSTONE: fine to medium grained, grey and orange brown, cross bedded at 20°.	DW	VL-L / M / L	- Be, 0°, P, S, IS - XWS, 220mm.t - Be, 20°, P, S, IS - Be, 20°, P, S - XWS, 20mm.t
	4				
		CORE LOSS 0.25m			
	5	SANDSTONE: fine to coarse grained, dark brown and grey, cross bedded at 20°	DW	H	- Be, 20°, P, S, IS
	6	END OF BOREHOLE AT 6.00m			
	7				
	8				

Column headings also include: Barrel Lift, Graphic Log, POINT LOAD STRENGTH INDEX $I_S(50)$ (EL VL L M H VH EH), DEFECT SPACING (mm) (500 300 100 50 30 10), General.

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 13.2m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, with roots.	M			GRASS COVER
									FILL: Clayey sand, fine to medium grained, dark brown.				APPEARS POORLY COMPACTED
								CL	SANDY CLAY: low to medium plasticity, orange brown.	MC>PL	St	-	RESIDUAL
						1			END OF BOREHOLE AT 0.9m				HAND AUGER REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 16.1m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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						0			CONCRETE: 70mm.t				
								-	FILL: Gravelly sand, fine to medium grained, light brown and dark brown, sandstone gravel.	M	-	-	APPEARS POORLY COMPACTED
									FILL: Sand, fine to medium grained, brown and dark brown, with a trace of fine to coarse grained sandstone and ironstone gravel.				HAND AUGER REFUSAL
						1			END OF BOREHOLE AT 0.7m				
						2							
						3							
						4							
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
6
1/3

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Method:** HAND AUGER **R.L. Surface:** ≈ 17.8m
Date: 11-9-09 **Datum:** AHD
Logged/Checked by: M.T./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					REFER TO DCP TEST RESULTS	0			CONCRETE: 140mm.t				
								-	FILL: Gravelly sand, fine to coarse grained, light brown and brown, igneous gravel.	M	-	-	APPEARS POORLY COMPACTED
									FILL: Clayey sand/sandy clay, fine to medium grained, medium plasticity, dark grey and grey, with fine to coarse grained igneous gravel.	M/MC>PL			
						1		CH	SILTY CLAY: high plasticity, orange brown.	MC>PL	St-VSt	220 170	RESIDUAL
								CL	SANDY CLAY: low to medium plasticity, orange brown.				
									REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							


JEFFERY & KATAUSKAS PTY LTD
JOB NO. 23373Z BH6 START CORING AT 1.58m
1.58m
CORE LOSS 0.14m
2
CORE LOSS 0.2m
CORE LOSS 0.32m
3
4
CORE LOSS 2.47m
5
6
COSS LOSS 0.86m
7
8
9
END BH6 AT 9.66m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
6

Client: CECIL KOUTSOS

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z | **Core Size:** TT56 | **R.L. Surface:** ≈ 17.8m

Date: 11-9-09 | **Inclination:** VERTICAL | **Datum:** AHD

Drill Type: MELVELLE | **Bearing:** - | **Logged/Checked by:** M.T./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
		1		START CORING AT 1.58m					
FULL RETURN				CORE LOSS 0.14m					
		2		SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW	M	•		- Be, 5°, P, R, Un - Be, 5°, P, R, Un
				CORE LOSS 0.2m					- XWS, 5°, 5mm.t
				SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW	L	•		- CS, 0°, 6mm.t
				CORE LOSS 0.32m					
		3		SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW	M	• •		- Be, 5°, P, S, Un
		4, 5		CORE LOSS 2.47m					
		6		SANDSTONE: fine to medium grained, light grey, orange brown and red.	XW	EL	• •		
				CORE LOSS 0.86m					
		7		SANDSTONE: fine to medium grained, light grey, orange brown and red.	DW XW	L EL	•		- Be, 5°, P, S, IS
		8		as above, but light grey stained orange brown and dark brown.	DW-SW	M	•		- Be, 5°, P, S, IS - Be, 5°, P, S, IS - Be, 5°, P, S, IS - J, 8°, P, R, IS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
6
3/3

Client: CECIL KOUTSOS
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 5 COMMONWEALTH PARADE, MANLY, NSW

Job No. 23373Z **Core Size:** TT56 **R.L. Surface:** ≈ 17.8m
Date: 11-9-09 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: MELVELLE **Bearing:** - **Logged/Checked by:** M.T./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
FULL RET-URN		9		SANDSTONE: fine to medium grained, light grey stained orange brown and dark brown.	DW-SW	M			- Be, 0°, P, R, Un - Be, 10°, R - J, 80°, P, R, IS	
						H			- Be, 0°, P, S, IS	
		10		END OF BOREHOLE AT 9.66m						
		11								
		12								
		13								
		14								

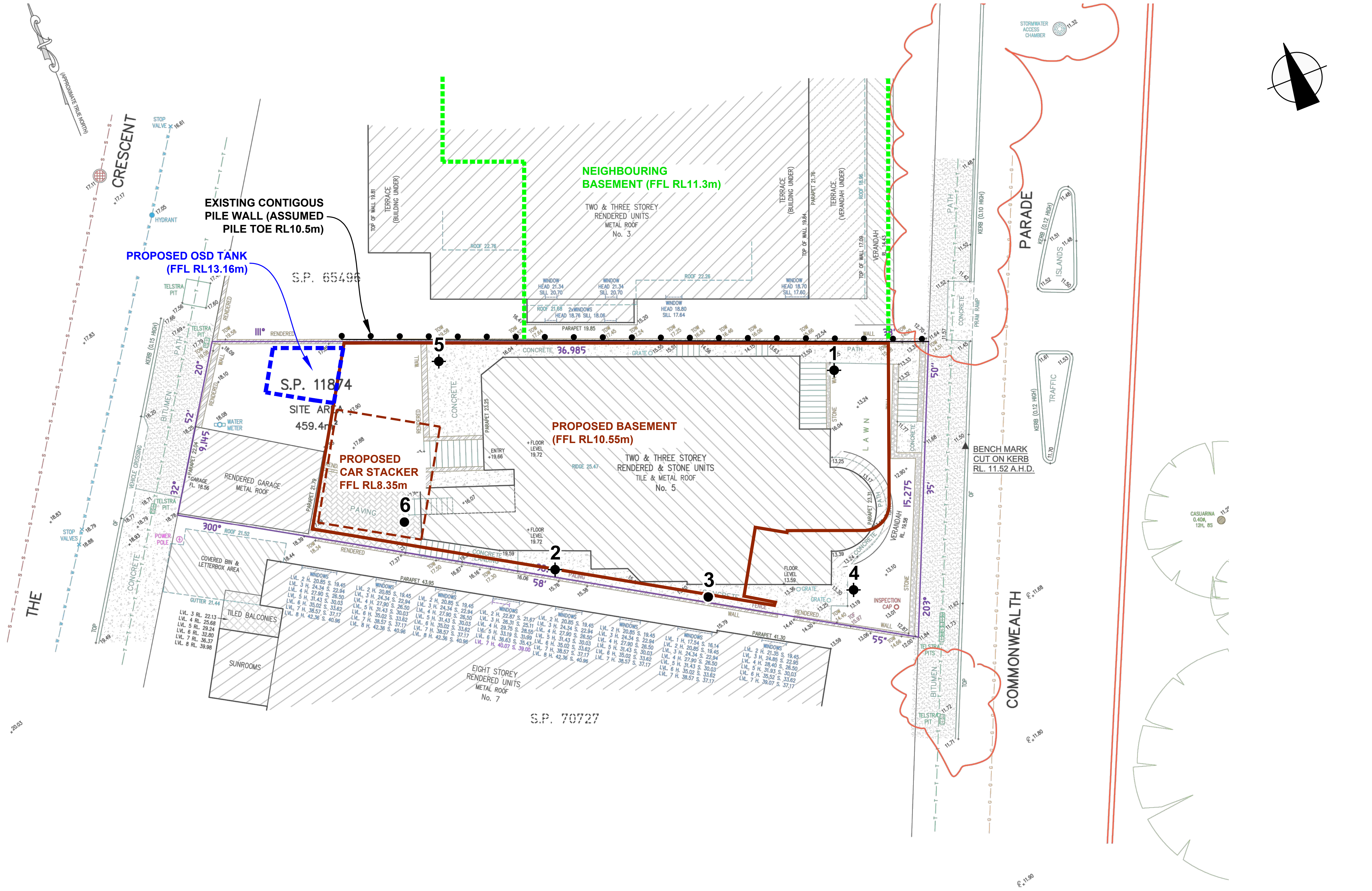
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 01 JUL 2019.

This plan should be read in conjunction with the JK Geotechnics report.

Title:	SITE LOCATION PLAN
Location:	5 COMMONWEALTH PARADE MANLY, NSW
Report No: 23373PD2rpt	Figure No: 1

JKGeotechnics

PLOT DATE: 15/07/2019 1:07:49 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\23000'S\23373PD2 MANLY\CAD\23373PD2RPT.DWG



LEGEND

- BOREHOLE
- BOREHOLE AND DCP TEST

SCALE 0 2 4 6 8 10 METRES 1:200 @A3

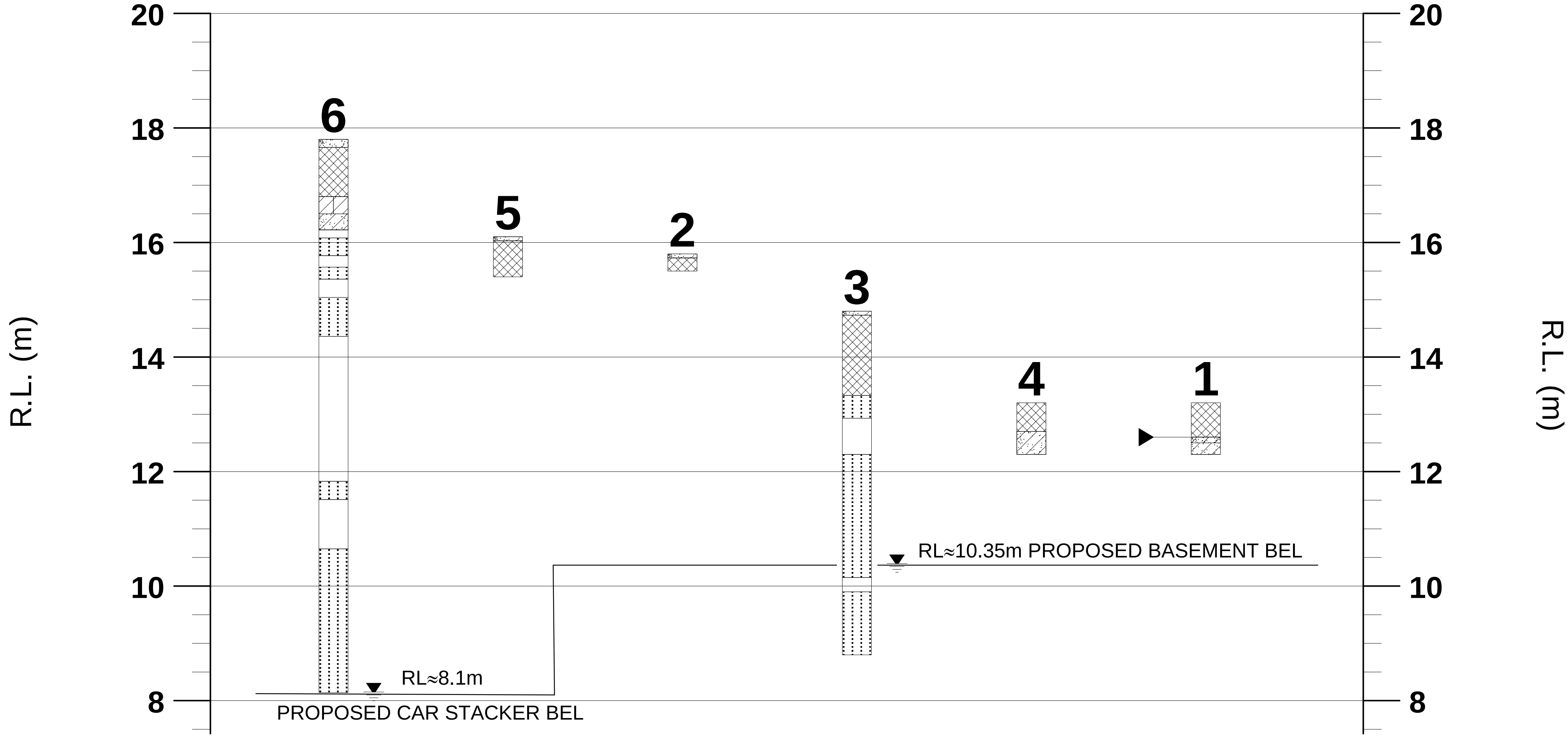
This plan should be read in conjunction with the JK Geotechnics report.

Title:	TEST LOCATION PLAN
Location:	5 COMMONWEALTH PARADE MANLY, NSW
Report No:	23373PD2rpt
Figure No:	2

JKGeotechnics

PLOT DATE: 15/07/2019 2:14:39 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\23000's\23373PD2 MANLY\CAD\23373PD2RPT.DWG

GRAPHICAL BOREHOLE SUMMARY



Core Loss/ Empty

Sandstone

Concrete

Fill

Silty Clay

Sandy Clay

Sandy Clay/ Clayey Sand

Groundwater seepage level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 23373Z

Figure No.: 3

APPENDIX A

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 33811

Client:
Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: Belinda Sinclair

Sample log in details:

Your Reference:	**23373Z, Manly**
No. of samples:	1 Soil
Date samples received:	30/09/09
Date completed instructions received:	30/09/09

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:	1/10/09
Date of Preliminary Report:	Not Issued
Issue Date:	1/10/09

NATA accreditation number 2901. This document shall not be reproduced except in full.
This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025.
Tests not covered by NATA are denoted with *.

Results Approved By:

Jacinta Hurst
Operations Manager

Miscellaneous Inorg - soil		
Our Reference:	UNITS	33811-1
Your Reference	------------	BH3
Depth	------------	0.7-0.8
Date Sampled		11/09/2009
Type of sample		Soil
Date prepared	-	1/10/2009
Date analysed	-	1/10/2009
pH 1:5 soil:water	pH Units	6.0
Sulphate, SO4 1:5 soil:water	mg/kg	<25
Chloride 1:5 soil:water	mg/kg	<100

Method ID	Methodology Summary
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
LAB.9	Sulphate determined turbidimetrically.
LAB.11	Chloride determined by argentometric titration.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			1/10/2009	[NT]	[NT]	LCS-1	1/10/2009
Date analysed	-			1/10/2009	[NT]	[NT]	LCS-1	1/10/2009
pH 1:5 soil:water	pH Units		LAB.1	[NT]	[NT]	[NT]	LCS-1	100%
Sulphate, SO4 1:5 soil:water	mg/kg	25	LAB.9	<25	[NT]	[NT]	LCS-1	92%
Chloride 1:5 soil:water	mg/kg	100	LAB.11	<100	[NT]	[NT]	LCS-1	100%

Report Comments:

Asbestos was analysed by Approved Identifier: Not applicable for this job
INS: Insufficient sample for this test NT: Not tested PQL: Practical Quantitation Limit <: Less than >: Greater than
RPD: Relative Percent Difference NA: Test not required LCS: Laboratory Control Sample NR: Not requested

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 sample batch were within laboratory acceptance criteria.
Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable. Surrogates: 60-140% is acceptable for general organics and 10-140% for SVOC and speciated phenols.

APPENDIX B

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1 1/1

BOREHOLE LOG

Client:

Project: P' PMENT

Location: 1-3 COMMONWEALTH PARADE, MANLY, NSW.

Job No. 14889WZ
Method: SPIRAL AUGER INTERTECH 350
R.L. Surface: ~15.1m
Date: 17-1-00
Datum: ASSUMED
Logged/Checked by: M.K./

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 & AFTER 5 MINS			0			FILL: Silty sand, fine to medium grained, dark brown, with brick fragments.	M	(L)		
		N > 9 9/150mm R								SPT REFUSAL ON BRICK FRAGMENTS
			1					MD		
		N = 14 3,5,9								
			2		–	SANDSTONE: fine to medium grained, yellow brown.	XW	EL	–	–
		N > 12 19,12/ 70mm R	3							
						END OF BOREHOLE AT 3.22m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **2** 1/1

Client:
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1-3 COMMONWEALTH PARADE, MANLY, NSW.

Job No. 14889WZ **Method:** SPIRAL AUGER INTERTECH 350 **R.L. Surface:** ~14.2m
Date: 17-1-00 **Datum:** ASSUMED
Logged/Checked by: M.K./

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 & AFTER 5 MINS		N = 5 3,3,2	0			FILL: Silty sand, fine to medium grained, dark brown, with trace of sandstone gravel and wood fragments.	M	L		APPEARS POORLY COMPACTED
			1		CL-CH	SANDY CLAY: medium to high plasticity, brown red.	MC>PL			RESIDUAL
		N = 16 4,8,8	2		-	SANDSTONE: fine to medium grained, brown grey.	XW	EL		-
							DW	L		
			3			END OF BOREHOLE AT 3.0m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

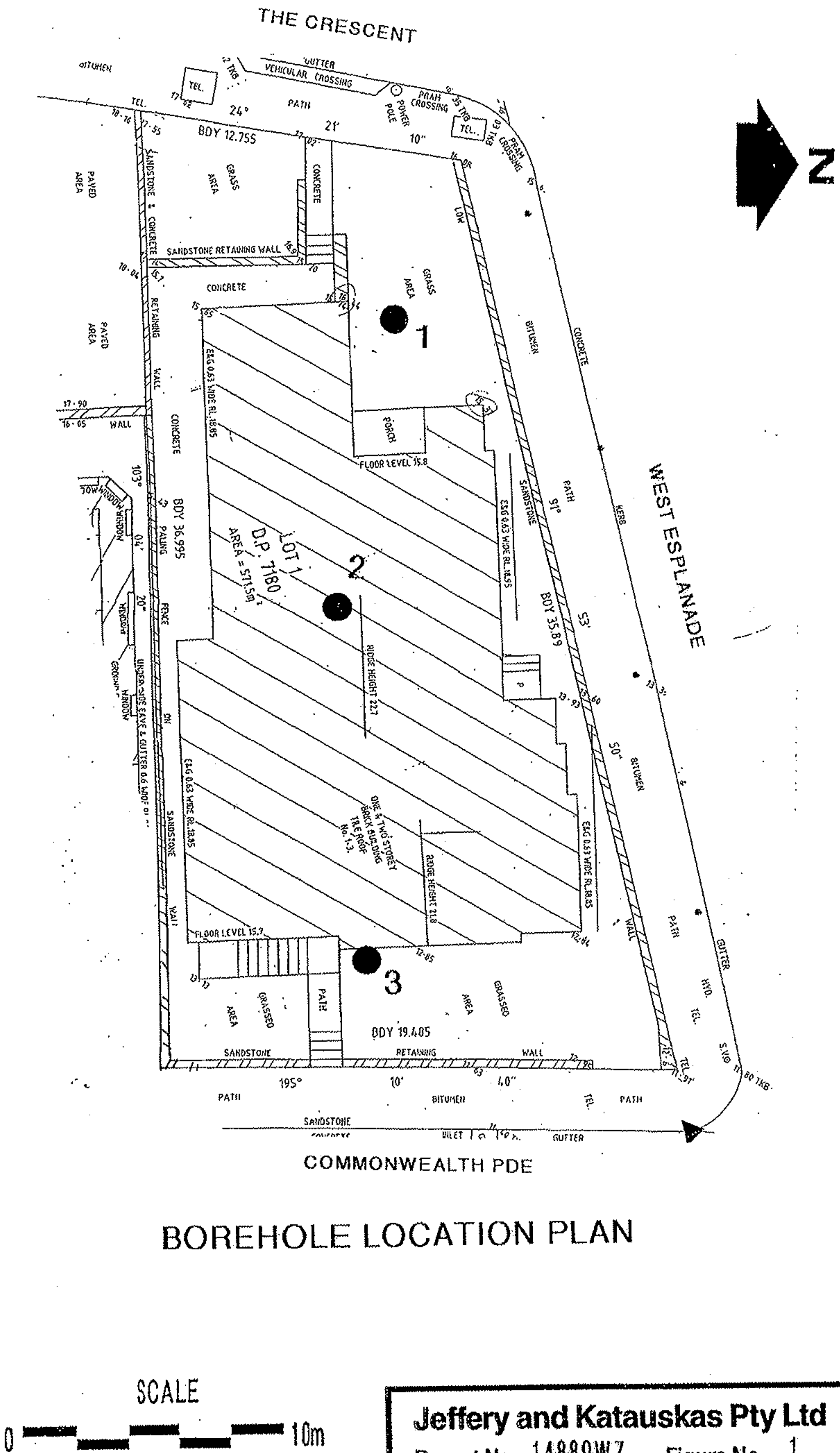
BOREHOLE LOG

Borehole No. **3** 1/1

Client: [illegible]
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1-3 COMMONWEALTH PARADE, MANLY, NSW.

Job No. 14889WZ **Method:** SPIRAL AUGER INTERTECH 350 **R.L. Surface:** ~12.8m
Date: 17-1-00 **Datum:** ASSUMED
Logged/Checked by: M.K./

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 & AFTER 5 MINS	DS	N = 2 1,1,1	0			FILL: Silty sand, fine to medium grained, dark brown, with sandstone and brick fragments and with a trace of clay.	M	VL		APPEARS POORLY COMPACTED
	DS	N > 10 4,10, 10/70mm			–	SANDSTONE: fine to medium grained, brown yellow grey.	XW	EL	–	–
	DS						DW	L-M		
			2.8			END OF BOREHOLE AT 2.8m				'TC' BIT REFUSAL

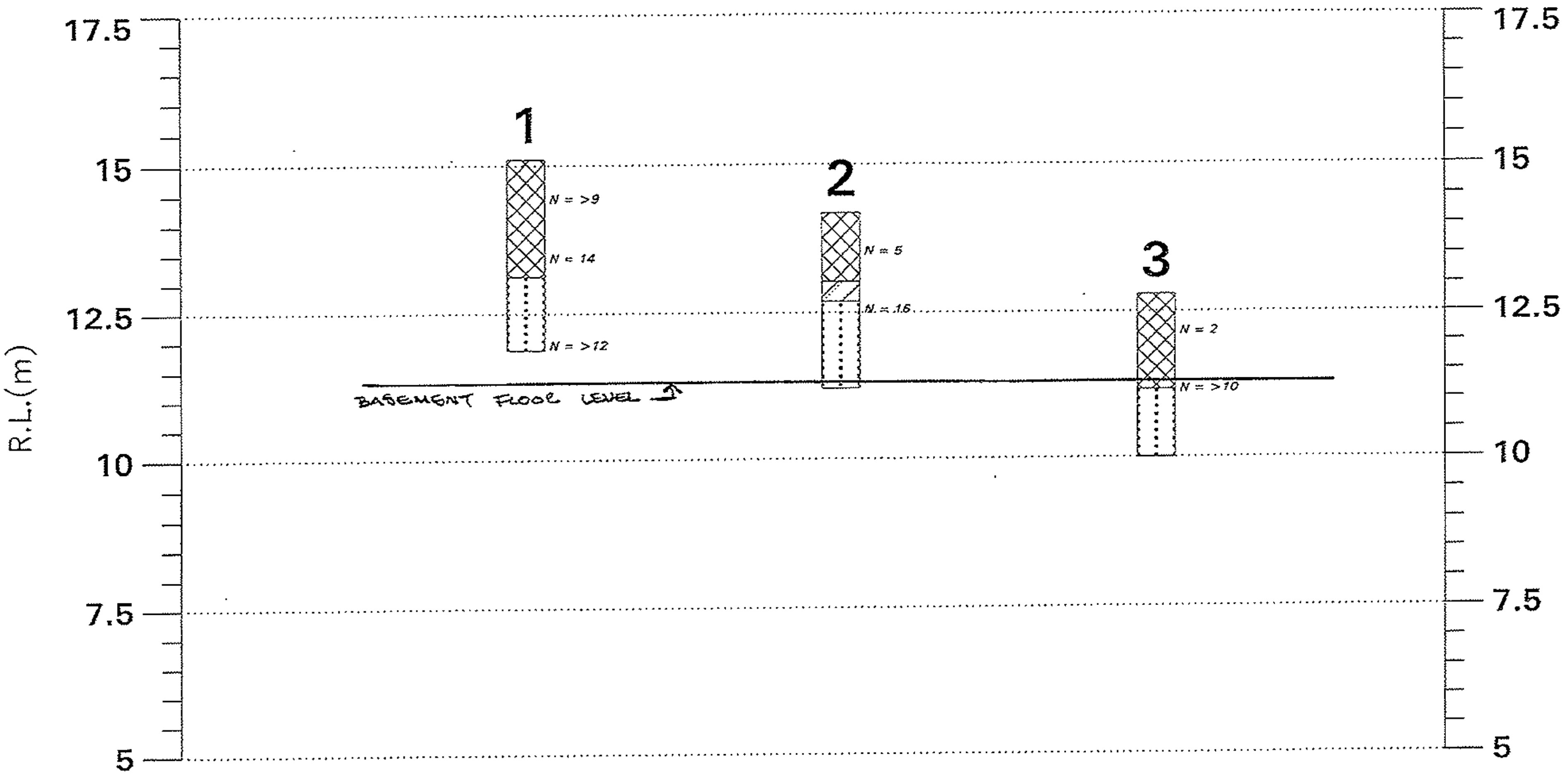


BOREHOLE LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No. 14889WZ Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



Strata symbols

- FILL
- SANDSTONE
- SANDY CLAY

ABBREVIATIONS

- N — SPT "N" VALUE
- Nc — SOLID CONE BLOW COUNTS PER 150mm

Scale:	1 : 100 (vert) ; NTS (horiz)
Jeffery and Katauskas Pty Ltd	
Job No.: 14889WZ	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 structure.

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 1726:2017 *'Geotechnical Site Investigations'*. 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 soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

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 (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	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 fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

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 shrink-swell behaviour, 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 methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock 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 a large 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. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, 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 limited 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 cuttings. 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 assessed 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. 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, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, 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 NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom 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 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

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 63.5kg 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.

A modification to the SPT 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.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm 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 a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm 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. The CPT does not provide soil sample recovery.

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. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT 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.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

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 terms and symbols used in preparation of the logs are defined in the following pages.

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 reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised 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 assess 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 Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. 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.

Reasonable 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.
- Details of the development that the Company could not reasonably be expected to anticipate.

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

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

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 and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity *Cu* > 4 and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

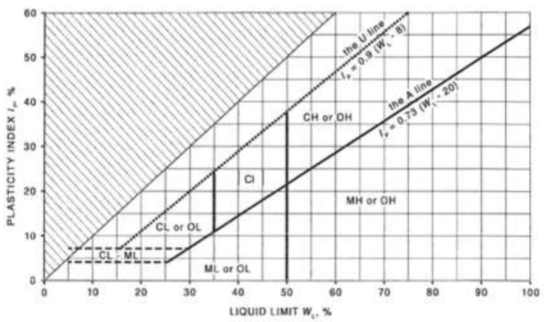
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	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 analysis.
	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. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	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° 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 (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained 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 ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		

Symbol		Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
VL	VERY LOOSE	≤ 15	0 – 4
L	LOOSE	> 15 and ≤ 35	4 – 10
MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
D	DENSE	> 65 and ≤ 85	30 – 50
VD	VERY DENSE	> 85	> 50
( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres