REPORT

TO
MOUNTIES GROUP

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MIXED USE DEVELOPMENT

AT
CONCORD RSL, NULLAWARRA AVENUE, CONCORD, NSW

23 May 2018
Ref: 31227Srpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5001
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as JK Geotechnics ABN 17 003 550 801

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Report prepared by:

Michael Serra
Geotechnical Engineer

Report reviewed by:

Paul Stubbs
Principal Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

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STS TABLE A: POINT LOAD STRENGTH INDEX TEST REPORT

ENVIROLAB SERVICES REPORT NO: 190015-A

BOREHOLE LOGS 1 TO 9 INCLUSIVE (WITH CORE PHOTOGRAPHS)

FIGURE 1: SITE LOCATION PLAN

FIGURE 2: BOREHOLE LOCATION PLAN

FIGURE 3: GRAPHICAL BOREHOLE SUMMARY – SECTION A-A

FIGURE 4: GRAPHICAL BOREHOLE SUMMARY – SECTION B-B

FIGURE 5: GRAPHICAL BOREHOLE SUMMARY – SECTION C-C

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REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed mixed use development at Concord RSL, Nullawarra Avenue, Concord, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Greg Pickering of Mounties Group trading as Mt Pritchard and District Community Club Limited by signed 'Acceptance of Proposal' form dated 8 May 2018 and was carried out in accordance with Option 1 of our proposal (P45101S) dated 5 June 2017.

As part of our investigation we have been provided with a built form options document prepared by GMU Urban Design and Architecture Pty Ltd (Project: UDR – Concord West – Concord RSL, Job No. 17046, dated 23 March 2018). Based on this document we understand the project is in the conceptual stage with three options currently under consideration.

Option 1 proposes the construction of three buildings over a common basement. Building A will comprise a 2 storey club building plus 1 level residential element above. Building B will comprise a 7 to 8 storey tower, while Building C will comprise a 5 storey tower. No surface levels have been given for the proposed basement, however we anticipate excavation to depths of about 3m will be required.

Option 2 proposes the construction of two buildings with no basement level. Building A will comprise a 2 storey club building and Building B will comprise two 6 storey towers over two podium levels. No excavation will be required.

Option 3 proposes the construction of three tower buildings over two common podium levels with no basement level. Building A and Building B will comprise two 6 storey towers over the podium levels, and Building C will comprise a 1 and 2 level building over the two podium levels. No excavation will be required.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations excavation, subgrade preparation, engineered fill, batters and retaining walls, groundwater, footings and slabs on-grade.

This geotechnical investigation was carried out in conjunction with a Preliminary Stage 1 Environmental Site Assessment by our specialist division, Environmental Investigation Services

(EIS). Reference should be made to the separate report by EIS for the results of the waste classification.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out between 16 and 20 April 2018 and comprised the drilling of nine boreholes (BH1 to BH9) using our truck mounted JK500 drilling rig. BH1 to BH4 were initially auger drilled to depths ranging from 4.26m to 8.83m below the existing ground surface and were then continued by diamond coring techniques using an NMLC core barrel with water flush to total depths ranging from 10.30 to 14.78m. BH5 to BH9 were auger drilled only to refusal depths ranging from 4.6m to 8.4m.

The borehole locations, as shown on Figure 2, were set out using taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between the spot levels and contours shown on the supplied survey plan by Linker Surveying (Ref. 171112, Issue 1 dated 14/12/2017). The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength of the natural soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on cohesive samples returned by the SPT split tube sampler. Within the augered portions of the boreholes, the strength of the underlying sandstone bedrock was assessed by observation of the drilling resistance of a Tungsten Carbide (TC) bit attached to the augers, together with examination of the recovered rock chips and subsequent correlation with laboratory moisture content test results. The strength of the recovered rock core was assessed with reference to Point Load Strength Index (I_{s50}) test results. The results of the point load strength index tests are summarised on the cored borehole logs and on the attached STS Table A.

Groundwater observations were made during and on completion of auger drilling. The use of water for coring limited further measurements of groundwater levels. No longer term groundwater monitoring has been carried out.

Our geotechnical engineer, Mr Bryan Zheng, set out the borehole locations, nominated sampling and testing and prepared logs of the subsurface strata encountered. The borehole logs are attached to this report, together with colour photographs of the rock core and a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine point load strength index values, soil pH, sulphate contents, chloride contents and resistivity. The results of the laboratory testing are summarised in the attached STS Tables A and Envirolab Report No. 190015-A. Samples were also collected from the boreholes for testing as part of the environmental site assessment by EIS.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in relatively flat alluvial topography, about 230m south-east of Yaralla Bay. The site itself as a slope of about 1° towards the south.

The site is currently occupied by the existing Concord RSL, a 2 level brick and concrete building surrounded by asphaltic concrete (AC) driveway and carparking. Adjoining the north-east of the RSL building are two grassed bowling greens which are retained along their southern corner, above the AC pavement, by a brick retaining wall up to about 1m in height. The RSL building appears in good external condition, while the AC carparks appear in fair condition with various sections of crocodile cracking, rutting and patchwork. Around the perimeter of the site are several large trees.

The site is surrounded by Arthur Walker Reserve, Ron Routley Oval and Majors Bay Reserve to the north-west, east and south/south-east, respectively; all of which are occupied by grassed playing fields. Arthur Walker Reserve is about 2.5m above the site and by a grassed batter slope of between 5° and 8° separates the two levels. Near the boundary between Majors Bay Reserve and the subject site there is a concrete rainwater tank, a steel framed and clad shed and a single level brick amenities block. Both Ron Routley Oval and Majors Bay Reserve have similar surface levels to the subject site.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by man-made fill and estuarine soils overlying Hawkesbury Sandstone of the Wianamatta Group. The investigation has revealed a fill layer of varying thickness, overlying alluvial and residual clay then weathered sandstone bedrock. Reference should be made to the attached borehole logs for more details of the encountered subsurface conditions. A graphical borehole summary is presented in Figures 3 to 5 and a summary of the subsurface conditions, as encountered, is presented below:

Pavements and Fill

Asphaltic concrete (AC) pavement was encountered in all boreholes between 50mm to 100mm thickness.

Fill was encountered in all boreholes beneath the pavements to depths between 1.2m (BH5) and 4.0m (BH2). The fill was predominantly gravelly sand, with the exception of BH7 where sand was present. The fill contained variable amounts of igneous gravel, slag, ash and trace amounts of building rubble including brick fragments, glass and plastic. The fill was generally assessed to be poorly compacted, based on the SPT 'N' values.

Natural Silty Clay

Beneath the fill, alluvial then residual silty clay was encountered in all boreholes. The alluvial clay was encountered to depths between 2m to 4.5m and was assessed to be of high plasticity and of soft to firm strength. Residual clay was encountered beneath the alluvial profile and was assessed to be of high plasticity and firm to very stiff strength.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered beneath the residual clay in all boreholes at depths ranging from 3.6m to 7.5m. In BH1, BH3, BH5, BH6 and BH8 the sandstone was initially distinctly weathered and assessed to be of very low to low strength. In BH2, BH4, BH7 and BH9, the sandstone was initially extremely weathered, improving to distinctly weathered and very low to low strength from depths between 4.5m and 8.2m. The sandstone improved to low to medium strength from depths between 3.7m and 9m, with auger refusal occurring in BH5 to BH9 at depths between 4.6m and 8.4m.

Within the cored sections of BH1 to BH4, generally, fine to medium grained sandstone bedrock was observed with sections of banded siltstone. Generally the sandstone was assessed to be of medium to high strength while in BH4 the sandstone was initially of very low strength, improving to medium then high strength from depths of 6.0m and 9.2m, respectively. The recovered rock cores contained significant defects including a number of extremely weathered and clay seams, bedding partings and inclined joints which reduced with depth.

Based on Pells *et al* (1998), the following sandstone classifications have been determined.

BH	Borehole Surface Level RL (m AHD)	Depth and Level the Top of Each Rock Class					
		Class V*		Class IV*		Class III or better	
		Depth	~RL	Depth	~RL	Depth	~RL
1	3.2m	3.8m	-0.6m	4.8m	-1.6m	6.2m	-3.0m
2	3.4m	7.5m	-4.1m	9.0m	-5.6m	9.3m	-5.9m
3	2.5m	4.0m	-1.5m	4.3m	-1.8m	7.0m	-4.5m
4	2.4m	4.1m	-1.7m	5.7m	-3.3m	6.2m	-3.8m
5	2.7m	3.6m	-0.9m	4.3m	-1.7m	-	-
6	2.4m	-	-	3.7m	-1.3m	-	-
7	3.5m	4.8m	-1.3m	6.0m	-2.5m	-	-
8	2.2m	-	-	5.8m	-3.6m	-	-
9	2.2m	6.0m	-3.8m	6.5m	-4.3m	-	-

* Estimated from auger drilling.

Groundwater

Groundwater seepage was encountered during auger drilling in all boreholes at depths between 0.8m (≈RL1.4m) in BH9 and 2.5m (≈RL1.0m) in BH7. We note that the groundwater may not have stabilised within the limited observation period. No longer term groundwater monitoring was undertaken.

3.3 Laboratory Test Results

The point load strength index test results on the recovered rock core showed reasonably good correlation with our field assessments of rock strength. The Unconfined Compressive Strength (UCS) of the sandstone, estimated from the point load strength index test results, ranged from 2MPa to 46MPa.

The soil pH values ranged from 7.5 to 8.8, while the chloride contents ranged between 25mg/kg and 2,100mg/kg and sulphate contents ranged between 79mg/kg and 730mg/kg. Resistivity ranged from 570ohm.cm to 6,400ohm.cm. Based on these results the soil would be classified as “Moderate” exposure classification for concrete and ‘Severe’ for steel piles in accordance with Table 6.4.2 (C) and Table 6.5.2 (C) of AS2159-2009 ‘Piling – Design and Installation’.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Where Option 1 is adopted, we anticipate excavation to depths of about 3m below the existing surface levels will be required. The excavation will extend through the fill with the deeper portion extending into the natural soils and below the water table. The soils should be readily excavated using conventional earthmoving equipment such as tracked excavators.

As the excavation would extend through sandy fill below the water table it is likely that the soils will 'flow' during excavation, and so their excavation would require the installation of a cut-off wall to below the proposed bulk excavation level, as well as dewatering from within the cut-off to allow effective excavation. We advise that consideration should be given to Option 2 and 3, where no basement excavation will be required to avoid some of the complications and costs associated with construction below the water table.

4.2 Groundwater Dewatering and Management

Where Option 1 is adopted, the proposed basement will extend about 2m or so below the current water table. However, due to potential contamination concerns, our groundwater monitoring was very limited. Therefore, detailed groundwater monitoring and seepage analysis will need to be undertaken once more detail of the basement level is known.

Dewatering will be required to allow excavation and construction below the groundwater table. As a general guide it is necessary to keep groundwater levels at least 1m below BEL to allow construction to be completed.

Detailed design of the dewatering system will be necessary, involving finite element seepage analysis to determine the required depth of the cut-off wall to limit drawdown and reduce the quantity of groundwater to be discharged during excavation and construction works. We expect that the shoring system will need to extend into the sandstone bedrock which was encountered at depths between 4.5m and 8.2m.

A dewatering licence will be required for this site, plus approval for groundwater disposal. Assessment and approval of the dewatering volumes in the temporary case will be required by DPI Water. In the long term the basement will need to be designed as a tanked structure to resist hydrostatic uplift pressures. At this stage, for preliminary design purposes, we consider that the basement tanked structure should be designed for a hydrostatic uplift pressure of about 12kPa to

30kPa (i.e. groundwater rising to surface). Consideration could also be given to the provision of hydrostatic pressure relief outlets within the tanked structure if the self-weight of the towers, or their distribution, are insufficient or to reduce the thickness of the floor slab. Pressure relief outlets would result in increased flows for the basement pump-out system to deal with during periods where the water table is higher than normal.

During excavation and dewatering, groundwater levels outside the basement excavation must be monitored to check that no significant drawdown of the groundwater level is occurring. The groundwater pumping volumes must also be recorded to check consistency with the groundwater seepage analysis and to ensure compliance with the dewatering licence. Groundwater quality must be tested and approved by Council for discharge to surface water bodies

Where no basement excavation is required (Option 2 or Option 3) then dewatering would not be necessary.

4.3 Excavation Support

The proposed basement excavation will need to be shored using an engineered full depth retention system, which must be installed prior to excavation commencing. As the fill is sandy and extends below the water table, the shoring should comprise a secant pile wall. If there are any gaps between the piles it would be possible for the water and sand to flow between the piles resulting in voids beyond the boundary which would lead to subsidence and damage around the shoring and hence construction of a secant wall would have to be to a high standard. We note that a secant pile wall would likely be quite expensive. We could provide further comments on the design and construction of such a system once more details are known.

Cantilevered walls up to 3m high may be designed based on the following geotechnical parameters;

- Where wall movements are of little concern, adoption of a triangular lateral earth pressure distribution using an 'active' earth pressure coefficient K_a of 0.35 for the sandy soils may be adopted.
- Where walls are restrained from lateral movement, such as by other structural elements in front of the wall, or where ground movements are to be reduced, an 'at rest' earth pressure coefficient K_o of 0.6 should be used.
- A bulk unit weight of $20kN/m^3$ for the soils.
- All surcharge loads, such as traffic loads, adjacent structures etc should be allowed for in the design.

- Full hydrostatic pressures should be assumed to apply unless measures are taken to provide complete and permanent drainage of the ground behind the walls. We recommend that behind wall drainage be incorporated in the design of the walls.

Where the toes of the piles socketed within the bedrock are used for lateral restraint, the design may be based upon an allowable lateral bearing pressure of 300kPa for rock of at least medium strength, providing the upper 0.3m of socket into rock below any adjacent excavation (including detailed excavation for the construction of footings or buried services) is ignored in the design. Whilst the soils will provide some passive restraint the socket of the piles into rock will prevent the full movement needed to generate full passive restraint. We recommend design be by finite element methods which enable the combined effects to be accurately modelled.

The required embedment of the shoring to limit groundwater inflows to the basement should also be checked as this may result in a deeper embedment than would be required then just for lateral restraint. The shoring design should also include an analysis of the potential movements of the shoring.

Due to the high water table the loads on the shoring will be moderately high and ground anchors may well be required to provide lateral restraint.

The shoring could be analysed using a soil structure interaction program such as “Wallap” to analyse pressures and deflections as well as bending moments in the wall; we can assist with such analysis if commissioned to do so. Models using spring stiffness should not be used for such analyses.

It may be feasible to adopt temporary batters, however this would require an extensive dewatering system around the perimeter of the batters to reduce the potential ‘flow’ of the sandy soils into the excavation and detailed groundwater monitoring and analysis would need to be carried out to determine the validity of this option.

We recommend that once details of the proposed development are known this office be approached for further advice.

4.4 Footings

We do not consider the soils on the site to be suitable to support the footings for any of the proposed development options, and recommend that the structural loads be transferred to the underlying sandstone bedrock by way of piled footings. The piles will need to be installed through the soil below the water table, and so conventional bored piles will not be possible. Therefore we recommend CFA auger grout injected, driven (steel or precast concrete) or g-pile piling techniques be adopted. The advantage of a driven or g-pile piling system is that the quantity of drilling spoil is very low compared to a CFA piling system.

The following table provides our recommended geotechnical parameters for design of footings bearing on the bedrock.

Ultimate (Limit State) and Serviceability Parameters for the Bedrock					
Rock Class	**Ultimate End Bearing Pressure (MPa)**	**Serviceability End Bearing Pressure (MPa)**	**Ultimate Shaft Adhesion (kPa)**	**Serviceability Shaft Adhesion (kPa)**	**Elastic Modulus (MPa)**
V	3	1.0	150	100	50
IV	6	2.0	500	200	300
III	30	3.5	1200	350	800

NOTE – Rock Classification in accordance with Foundations on Sandstone and Shale in the Sydney Region, Pells, Mostyn and Walker, Australian Geomechanics, Dec 1998.

The serviceability end bearing pressure values given above are based on a serviceability criteria of deflections at the pile toe of less than or equal to 1% of the pile diameter. Footings on rock can also be designed using ‘Limit State Design’ principles. For limit state design, ultimate bearing capacities could be adopted provided that higher settlements can be tolerated. It should be noted that such ultimate bearing pressures must be used in conjunction with an appropriate geotechnical strength reduction factor (Φ_g).

If the designer wishes to adopt the Limit State Design methods of the piling code, AS2159-1995, we recommend that a geotechnical strength reduction factor, Φ_g, of 0.5 be applied.

The allowable shaft adhesion value shown on the above table is for compressive loads. Under uplift loads the value should be halved. Pile sockets must be appropriately roughened. The minimum embedment of footings into the rock class should be around 0.3m.

Where driven piles are adopted, specialist piling contractors are typically engaged on a design and construct basis. As a minimum we recommend that trial piles be installed and dynamic (CAPWAP) and integrity testing be conducted to confirm the performance of the piles, in addition to the pile test programme. We also recommend that quantitative vibration monitoring be carried out on the neighbouring structures by an experienced vibration consultant or geotechnical engineer to check that vibrations are within acceptable limits.

We note that high strength sandstone bedrock was encountered at depth, and the potential piling contractors must ensure that pilling rigs capable of drilling into this material be used, particularly if deep rock sockets are required.

It is recommended that the initial piles be installed as close as practical to our borehole locations to calibrate the equipment (and operator) to the subsurface conditions by direction comparison of the installation performance and readings to the borehole results. These initial readings can then be used to assist with installation of piles away from the borehole locations to assess that the appropriate foundation material has been reached. The rock being drilled cannot be inspected, and therefore the piling contractor should be required to certify their piles are suitable for the adopted bearing pressures.

4.5 <u>On-Grade Floor Slabs and Pavements</u>

An important issue for slabs on-ground is the presence of very thick, poorly compacted 'uncontrolled' fill. Therefore the slab on-ground may be subjected to differential settlement under load which may lead to structural distress of the floor slab.

Where Option 2 or Option 3 are adopted, slab-on-ground construction of the floor slabs is considered feasible provided the loads are light and the slab is designed to tolerate subgrade movements, not just potential settlement, but also consolidation, although we consider the potential consolidation movements to be minimal due to the age of the fill. All this is on the condition that the subgrade is prepared in accordance with the recommendations given in Section 4.6 below. The on-ground floor slabs should be independent of the piled footing system. Some cracking and maintenance in the longer term should not be unexpected.

Consideration should be given to suspending the ground floor slabs on the pile footings, which would nullify the problem of settlement and also not require any particular subgrade preparation.

Concrete pavements should be supported on a subbase layer of 100mm of good quality fine crushed rock such as RTA QA Specification 3051 unbound base (e.g. DGB20) which is compacted to at least 100% of SMDD. This will provide more uniform subgrade support and would reduce ‘pumping’ of fines at joints.

4.6 Subgrade Preparation and Engineered Fill

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 ‘Guidelines on Earthworks for Commercial and Residential Developments’ and be carried out in accordance with the recommendations of the Environmental Site Assessment by EIS.

Following excavation to design subgrade levels if there is not basement (Option 2 or Option 3), we expect that the exposed subgrade will comprise a sandy fill material. The exposed soil subgrade should be proof rolled with a minimum twelve tonne dead weight vibratory smooth drum roller. The sandy surface soils may shear and heave under proof rolling with such a roller, however we consider that due to the presence of uncontrolled fill over the site, it is important to proof roll with at least a moderately sized roller. Therefore to allow proof rolling and near surface compaction of the sandy subgrade, a layer of at least 100mm thickness of crushed sandstone or crushed concrete may need to be placed across the stripped surface and then the surface rolled again using at least eight passes of the twelve tonne minimum deadweight vibratory smooth drum roller. The final passes of the rolling should be completed in the presence of a geotechnical engineer or experienced earthworks technician. The purpose of the proof rolling is to identify any soft or heaving areas and to improve the near surface compaction of the sandy soils.

Any soft or heaving areas encountered during proof rolling should be locally excavated to a sound base and replaced with engineered fill. Excavation in these areas should be kept to a minimum and if significant heaving or soft areas occur it may be necessary to use thick bridging layers with geogrid reinforcement.

From a geotechnical perspective, the existing sandy fill material on-site would be suitable for re-use as engineered fill. However suitability of the fill for re-use will be governed by the Environmental Site Assessment prepared by EIS. Further advice should be sought from EIS on methods and procedures that would need to be undertaken to make such fill material suitable for re-use. An alternative would be to import a well graded granular material such as crushed or ripped sandstone, on the condition the material is “clean”, containing no organics or other deleterious materials, with a soaked CBR of at least 10% and free of particle sizes greater than 75mm.

Engineered fill should be placed in layers not exceeding 250mm loose thickness and compacted to at least 98% of Standard Maximum Dry Density (SMDD) or 80% Density Index. Testing should be completed on each layer of engineered fill at a frequency of at least one test per layer per 500m^2, at least three tests per layer, or three tests per site visit, whichever requires the most tests. The testing should be carried out in accordance with Level 1 standards as defined in AS3798-2007.

4.7 Basement Floor Slabs

Where Option 1 is adopted, the basement will need to be designed as water proofed to resist hydrostatic uplift forces (i.e. a tanked basement). Design of the tanked basement should take into account the possibility of rises in the groundwater level due to flooding and therefore the design should assume a groundwater level at surface and the need for an overflow system must also be addressed.

The subgrade below the basement slab will need to be prepared prior to construction of the slab, as per our recommendations in Section 4.6 above.

Where plant and equipment (such as piling rigs) are trafficking the subgrade, a working platform of good quality granular material will be required to support the loads. Depending on the size of plant being used, a working platform in excess of 0.5m thickness would not be unexpected. Once the plant loads are known, we would be pleased to carry out a working platform thickness design. The working platform would form a good subgrade on which to construct the basement slabs.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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

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.

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31227S
Project:	Proposed Mixed Use Development	**Report:**	A
Location:	Nullawarra Avenue, Concord, NSW	**Report Date:**	20/04/2018

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	4.90 - 4.94	1.4	28
	5.34 - 5.37	1.0	20
	5.75 - 5.79	1.0	20
	6.30 - 6.34	0.9	18
	6.70 - 6.74	1.0	20
	7.15 - 7.18	1.7	34
	7.34 - 7.37	0.8	16
	7.71 - 7.75	1.8	36
	8.15 - 8.19	1.9	38
	8.66 - 8.70	2.3	46
	9.14 - 9.18	1.4	28
	9.69 - 9.73	1.3	26
	10.08 - 10.12	1.7	34
	10.75 -10.79	2.1	42
2	9.27 - 9.31	0.3	6
	9.96 - 9.99	0.6	12
	10.24 - 10.27	0.9	18
	10.80 - 10.84	2.0	40
	11.13 - 11.17	1.9	38
	11.70 - 11.73	1.0	20
	12.19 - 12.23	2.2	44
	12.70 - 12.73	2.0	40
	13.15 - 13.19	1.2	24
	13.80 - 13.83	1.5	30
	14.11 - 14.15	1.7	34

NOTES: See Page 3 of 3

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31227S
Project:	Proposed Mixed Use Development	**Report:**	A
Location:	Nullawarra Avenue, Concord, NSW	**Report Date:**	20/04/2018
		Page 2 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	14.70 - 14.74	1.8	36
3	4.40 - 4.43	0.5	10
	4.52 - 4.56	0.8	16
	4.76 - 4.79	0.4	8
	5.10 - 5.14	0.5	10
	5.74 - 5.78	0.9	18
	6.22 - 6.26	0.8	16
	6.46 - 6.50	1.3	26
	6.68 - 6.72	0.2	4
	6.83 - 6.86	0.2	4
	7.11 - 7.14	1.2	24
	7.60 - 7.70	1.6	32
	8.18 - 8.22	1.6	32
	8.75 - 8.79	1.7	34
	9.22 - 9.25	2.0	40
	9.76 - 9.80	1.7	34
	10.24 - 10.28	2.3	46
4	5.11 - 5.15	0.08	2
	5.41 - 5.44	0.2	4
	5.76 - 5.79	0.2	4
	6.26 - 6.30	0.6	12
	6.72 - 6.77	0.4	8
	7.22 - 7.26	0.3	6
	7.43 - 7.47	1.0	20
	7.67 - 7.71	0.7	14

NOTES: See Page 3 of 3

115 Wicks Road
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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31227S
Project:	Proposed Mixed Use Development	**Report:**	A
Location:	Nullawarra Avenue, Concord, NSW	**Report Date:**	20/04/2018
		Page 3 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	8.06 - 8.09	0.5	10
	8.55 - 8.58	0.7	14
	9.21 - 9.24	1.3	26
	9.76 - 9.79	1.6	32
	10.06 - 10.10	1.5	30
	10.58 - 10.61	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: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

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

CERTIFICATE OF ANALYSIS 190015-A

Client Details	
Client	JK Geotechnics
Attention	Michael Serra
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details	
Your Reference	**E31227K, Concord**
Number of Samples	Testing on 3 Soils
Date samples received	20/04/2018
Date completed instructions received	20/04/2018

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.

Report Details	
Date results requested by	09/05/2018
Date of Issue	09/05/2018
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***	

Asbestos Approved By
Analysed by Asbestos Approved Identifier: Lucy Zhu
Authorised by Asbestos Approved Signatory: Lucy Zhu

Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil				
Our Reference	UNITS	190015-A-10	190015-A-24	190015-A-50
Your Reference		BH2	BH4	BH8
Depth		1.5-1.95	1.5-1.95	1.5-1.95
Date Sampled		16/04/2018	17/04/2018	20/04/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	09/05/2018	09/05/2018	09/05/2018
Date analysed	-	09/05/2018	09/05/2018	09/05/2018
pH 1:5 soil:water	pH Units	8.8	7.5	7.5
Chloride, Cl 1:5 soil:water	mg/kg	25	650	2,100
Sulphate, SO4 1:5 soil:water	mg/kg	80	79	730
Resistivity in soil*	ohm m	64	14	5.7

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			09/05/2018	[NT]	[NT]	[NT]	[NT]	09/05/2018	[NT]
Date analysed	-			09/05/2018	[NT]	[NT]	[NT]	[NT]	09/05/2018	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	85	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	88	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

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.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

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

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

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

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

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

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 1

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~3.2 m
Date: 16/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			ASPHALTIC CONCRETE: 100mm.t				
	N = 8 4,4,4	-	FILL: Gravelly sand, fine to medium, dark brown, fine to medium sub angular and angular igneous gravel, with ash and slag.	M			APPEARS POORLY COMPACTED
	N = 4 3,1,3		as above, but with high plasticity, grey clay, trace fine to medium sub angular and angular gravel.				
ON COMPLETION OF AUGERING		CH	Silty CLAY: high plasticity, dark brown, with tree fibres, trace fine to medium sub angular gravel.	w~LL	(VS)		ALLUVIAL
	N = 8 2,3,5		Silty CLAY: high plasticity, light grey mottled red brown, trace fine to medium sub angular ironstone gravel.	w>PL	(F)		RESIDUAL
		-	SANDSTONE: fine to medium grained, light brown.	DW	VL - L		LOW 'TC' BIT RESISTANCE
					L - M		HAWKESBURY SANDSTONE MODERATE RESISTANCE
			REFER TO CORED BOREHOLE LOG				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
1

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S	**Core Size:** NMLC	**R.L. Surface:** ~3.2 m
Date: 16/4/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK500	**Bearing:** N/A	**Logged/Checked By:** B.Z./M.S.

Water Loss\Level: 100% RETURN
Formation: Hawkesbury Sandstone

RL (m AHD): -1, -2, -3, -4, -5, -6, -7
Depth (m): 5, 6, 7, 8, 9, 10

Point Load Strength Index $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20

CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength
START CORING AT 4.83m		
SANDSTONE: fine to medium grained, yellow brown and light brown, with iron staining.	MW	H
	SW	M
as above, but light grey, trace of iron staining.		H
SILTSTONE: dark grey.		M
SANDSTONE: fine to medium grained, light grey, with bands of siltstone.		H
SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-5°.	FR	
END OF BOREHOLE AT 10.83 m		

DEFECT DETAILS — DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness)

Specific / General

~~(4.83m) CS, 0°, 30 mm.t~~
(4.97m) CS, 0°, 50 mm.t
(5.05m) CS, 0°, 30 mm.t
(5.20m) J, 70°, Fe Sn
(5.26m) J, 70°, Fe Sn
(5.39m) Be, 0°, Fe Sn
(5.47m) Be, 0°, Fe Sn
(5.58m) XWS, 0°, 20 mm.t
(5.72m) XWS, 0°, 20 mm.t
(5.83m) CS, 0°, 10 mm.t
(5.90m) Be, 0°, Fe Sn
(6.05m) CS, 0°, 60 mm.t
(6.62m) XWS, 0°, 10 mm.t
(6.81m) CS, 0°, 30 mm.t
(6.91m) Be, 0°, Fe Sn
(7.55m) CS, 0°, 10 mm.t
(7.95m) XWS, 0°, 20 mm.t
(8.24m) XWS, 0°, 10 mm.t
(8.28m) XWS, 0°, 10 mm.t
(8.43m) XWS, 0°, 20 mm.t
(8.48m) XWS, 0°, 20 mm.t
(8.83m) XWS, 0°, 30 mm.t
(9.13m) XWS, 0°, 10 mm.t


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4m
31227S CONCORD, 16/04/18, BH1 Coring starts at 4.83m
5m
6m
7m
BH1
8m
Box 2/2
9m
10m
10.83m
CORING END

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~3.4 m
Date: 16/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		0		ASPHALTIC CONCRETE: 100mm.t				
	N = 7 3,3,4	0.1	-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium sub angular and angular igneous gravel, trace medium to high plasticity clay, with ash, slag, trace of glass fragments.	M			APPEARS POORLY COMPACTED HYDROCARBON ODOUR
ON COMPLETION OF AUGERING	N = 1 2,1,0							
	N = 2 1,2,0	~2.0			W			
		4.0	CH	Silty CLAY: high plasticity, grey and dark grey, with tree fibres and sub angular gravel.	w>LL	(VS)		ALLUVIAL ORGANIC ODOUR
	N = 6 1,2,4	4.5		Silty CLAY: high plasticity, grey mottled red brow, trace of fine to medium brown sub angular ironstone gravel.	w>PL	St	100 150 200	RESIDUAL
	N = 11 2,4,7	6.0				VSt	220 240 300	

RL (m AHD) scale: 3, 2, 1, 0, -1, -2, -3

SAMPLES: ES, U50, DB, DS

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~3.4 m
Date: 16/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						-4			CH	Silty CLAY: high plasticity, grey mottled red brow, trace of fine to medium brown sub angular ironstone gravel. *(continued)*	w>PL	VSt		RESIDUAL
				■	N = 30 5,12,18		8		-	Extremely Weathered sandstone: Clayey SAND, fine to medium grained, light grey and light brown, with ironstone bands.	XW	D		HAWKESBURY SANDSTONE
						-5				SANDSTONE: fine to medium grained, light brown, with ironstone and XW sandstone bands.	DW	VL		LOW 'TC' BIT RESISTANCE
						-6 -7 -8 -9 -10	9 10 11 12 13			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S | **Core Size:** NMLC | **R.L. Surface:** ~3.4 m
Date: 16/4/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK500 | **Bearing:** N/A | **Logged/Checked By:** B.Z./M.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness (Specific / General)	Formation
		-5			START CORING AT 8.83m				
					NO CORE 0.17m			(9.00m) J, 80°, Un, R, Fe Vn	
		-6	9		SANDSTONE: fine to medium grained, light grey, with orange brown bands and iron staining.	MW	M	(9.18m) XWS, 0°, 20 mm.t (9.25m) XWS, 0°, 20 mm.t (9.53m) XWS, 0°, 20 mm.t (9.55m) J, 70°, Un, R, Fe Sn (9.77m) XWS, 0°, 20 mm.t (10.10m) CS, 0°, 30 mm.t (10.16m) CS, 0°, 20 mm.t	Hawkesbury Sandstone
		-7	10		as above, but light grey.	SW	H	(10.90m) J, 90°, Un, R, Fe Sn (11.06m) XWS, 10°, 30 mm.t	
100% RETURN		-8, -9, -10, -11	11, 12, 13, 14		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°	FR		(12.88m) XWS, 0°, 30 mm.t (13.25m) CS, 0°, 20 mm.t (13.60m) XWS, 0°, 10 mm.t	
					END OF BOREHOLE AT 14.78 m				

POINT LOAD STRENGTH INDEX $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
SPACING (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
8m
31227S CONCORD BH2 16.04.18 Coring Starts at 8.83m
←8.83 9.0→
CORE LOSS
9m
10m
11m
BH2
12m
Box 2/2
13m
14m
←14.78m Coring End

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.5 m
Date: 17/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				0			ASPHALTIC CONCRETE: 50mm.t.				
	ES	N = 16 12,7,9	2			-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained sub angular igneous gravel, with ash and slag.	M			APPEARS MODERATELY COMPACTED
▶	ES	N=0 1,0,0	1	1.5		CH	Silty CLAY: high plasticity, dark brown, with trace of fine to medium grained sub angular ironstone gravel and root fibres.	w>LL	VS		ALLUVIAL ORGANIC ODOUR
			0				Silty CLAY: high plasticity, light grey and light brown, trace fine to medium sub angular ironstone gravel.	w>PL	(St)		RESIDUAL
							as above, but with ironstone bands.				
			-1				as above, but trace of ironstone gravel.				
	ES, DS			4		-	SANDSTONE: fine to medium grained, light brown and brown, with ironstone bands.	DW	VL - L		HAWKESBURY SANDSTONE
			-2				REFER TO CORED BOREHOLE LOG				LOW TO MODERATE RESISTANCE

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

CORED BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S	**Core Size:** NMLC	**R.L. Surface:** ~2.5 m
Date: 17/4/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK500	**Bearing:** N/A	**Logged/Checked By:** B.Z./M.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	Formation
				START CORING AT 4.26m			
100% RETURN			4.26–6.50	SANDSTONE: fine to medium grained, light brown, orange brown and light grey, with iron staining, bedded at 0-10°, with high strength bands.	MW	M	Hawkesbury Sandstone
			6.50–7.00	as above, but trace iron staining, light grey, with shale bands.	MW	L	Hawkesbury Sandstone
			7.00–8.20	SANDSTONE: fine to medium grained, light grey, with dark grey siltstone seams and lenses, bedded at 0°.	SW (to ~7.6), FR	H	Hawkesbury Sandstone
			8.20–10.30	as above, but with dark grey laminae.	FR	H	Hawkesbury Sandstone
				END OF BOREHOLE AT 10.30 m			

RL (m AHD): -2, -3, -4, -5, -6, -7, -8
Depth (m): 5, 6, 7, 8, 9, 10

POINT LOAD STRENGTH INDEX $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH

SPACING (mm): 600, 200, 60, 20

DEFECT DETAILS — DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness

Specific:
- ~~(4.26m) XWS, 0°, 90 mm.t~~
- (4.37m) Be, 0°, Fe Sn
- (4.43m) Be, 0°, Fe Sn
- (4.63m) XWS, 5°, 5 mm.t
- (4.67m) XWS, 5°, 5 mm.t
- (4.71m) XWS, 10°, 5 mm.t
- (4.86m) XWS, 5°, 10 mm.t
- (4.91m) J, 20°, P, R, Fe Sn
- (4.98m) Be, 0°, Fe Sn
- (5.02m) Be, 0°, Un, R, Fe Sn
- (5.13m) J, 20°, P, Cn, Healed
- (5.28m) XWS, 0°, 30 mm.t
- (5.46m) J, 80°, St, R, Cn
- (5.60m) J, 45°, Un, R, Fe Sn, 50 mm.t
- (6.20m) Be, 0°, Fe Sn
- (6.55m) XWS, 0°, 70 mm.t
- (6.92m) XWS, 0°, 40 mm.t
- (7.20m) Be, 0°, Cn, Siltstone, FILLED
- (7.29m) Be, 0°, Cn, Siltstone, FILLED
- (7.55m) XWS, 5°, 5 mm.t
- (7.86m) CS, 0°, 5 mm.t
- (8.00m) XWS, 0°, 20 mm.t
- (8.44m) XWS, 0°, 30 mm.t
- (9.17m) XWS, 0°, 20 mm.t
- (9.40m) Be, 0°, Clay FILLED
- (9.80m) XWS, 0°, 10 mm.t
- (10.00m) CS, 0°, 10 mm.t
- (10.10m) XWS, 0°, 5 mm.t

General

JK 9.01.2 LIB.GLB Log JK CORED BOREHOLE - MASTER 31227S CONCORD.GPJ <<DrawingFile>> 22/05/2018 12:00 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.00.5 2018-01-11

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
31227S CONCORD 17/04/18 BH3 Coring Starts at 4.26 m
4m
4.26 →
5m
6m
7m
BH3
8m
Box 2/2
9m
10m
← 10.30 m CORING END

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
1 / 2

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.4 m
Date: 17/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	0		ASPHALTIC CONCRETE: 50mm.t				
		-	FILL: Gravelly sand, fine to medium grained, dark brown, with fine to medium grained, sub angular and angular igneous gravel, trace of ash and slag.	M			POORLY COMPACTED
N = 4 3,3,1			as above, but with clay, medium to high plasticity, ash and slag.				
N=0 1,0,0		CH	Silty CLAY: high plasticity, dark brown, trace of fine to medium sub angular and angular ironstone gravel and root fibres.	w>LL	(VS)		ALLUVIAL ORGANIC ODOUR
N = 9 3,4,5	3		Silty CLAY: high plasticity, light grey mottled red brown and light brown, trace of fine to medium grained sub angular ironstone gravel.	w>PL	St - VSt	150 200 250	RESIDUAL
		-	Extremely Weathered sandstone: Clayey SAND, fine to medium grained, light brown and light grey, with ironstone bands.	XW	D		HAWKESBURY SANDSTONE
N=SPT 4/ 50mm REFUSAL			SANDSTONE: fine to medium grained, light brown and light grey, with XW sandstone bands.	DW	VL		LOW 'TC' BIT RESISTANCE
			REFER TO CORED BOREHOLE LOG				

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CORED BOREHOLE LOG

Borehole No.
4
2 / 2

Client:	MOUNTIES GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S	**Core Size:** NMLC	**R.L. Surface:** ~2.4 m
Date: 17/4/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK500	**Bearing:** N/A	**Logged/Checked By:** B.Z./M.S.

Water Loss\Level	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Formation
		START CORING AT 4.73m			
100% RETURN		NO CORE 0.11m			
		Silty CLAY: high plasticity, light grey.	RS		
	5	SANDSTONE: fine to medium grained, light grey and red brown, with iron stain, with XW sandstone bands, bedded at 0-20°.	HW	VL	Hawkesbury Sandstone
				L	
			XW	Hd	
			HW	L	
	6	SANDSTONE: fine to medium grained, orange brown and light brown, mottled red brown, bedded at 0-20°.	MW	M	
	8	as above, but light grey and grey trace of iron staining.	SW		
		as above, but light brown, with orange brown bands.			
	9	SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.	FR	H	
		END OF BOREHOLE AT 10.69 m			

RL (m AHD): -2, -3, -4, -5, -6, -7, -8
Depth (m): 5, 6, 7, 8, 9, 10

POINT LOAD STRENGTH INDEX $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH

SPACING (mm): 600, 200, 60, 20

DEFECT DETAILS – DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness)

Specific:

- (4.86m) XWS, 0°, 30 mm.t
- (5.03m) XWS, 0°, 70 mm.t
- (5.22m) XWS, 0°
- (5.32m) J, 90°, Un, R, Cn
- (5.75m) Be, 0°, P, R, Cn
- (5.90m) XWS, 0°, 40 mm.t
- (5.96m) XWS, 0°, 10 mm.t
- (6.05m) J, 35°, Ir, R, Fe Sn
- (6.18m) J, 30°, P, R, Fe Sn
- (6.56m) CS, 10°, 30 mm.t
- (6.61m) XWS, 10°, 20 mm.t
- (7.09m) XWS, 10°, 30 mm.t
- (7.20m) XWS, 10°, 50 mm.t
- (7.32m) XWS, 0°, 10 mm.t
- (7.73m) J, 50°, P, R, Fe Vn
- (7.86m) Be, 10°, P, Fe Sn
- (7.87m) XWS, 0°, 20 mm.t
- (8.05m) XWS, 5°, 20 mm.t
- (8.82m) XWS, 0°, 40 mm.t
- (8.86m) J, 90°, Ir, R, Fe Sn
- (8.90m) J, 65°, Un, R, Fe Sn
- (8.97m) J, 90°, Un, R, Fe Vn
- (9.09m) XWS, 0°, 80 mm.t
- (9.62m) J, 20°, Un, Cn, Healed
- (9.70m) J, 70°, Ir, Cn, Healed
- (9.83m) XWS, 0°, 5 mm.t
- (9.92m) Be, 0°, P, S, Clay FILLED
- (9.97m) XWS, 0°, 40 mm.t
- (10.30m) XWS, 5°, 30 mm.t

General

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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4m
31227S CONCORD 17/04/18 BH4 Coring
Starts at 4.73m
←4.73 4.84→
CORE LOSS
5m
6m
7m
8m
BH4
9m
Box 2/2
10m
←10.69m Coring
End

BOREHOLE LOG

Borehole No.
5
1 / 1

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S
Method: SPIRAL AUGER
R.L. Surface: ~2.7 m
Date: 18/4/18
Datum: AHD
Plant Type: JK500
Logged/Checked By: B.Z./M.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
							ASPHALTIC CONCRETE: 50mm.t.				
		N = 3 2,2,1	2			-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium sub angular and angular igneous gravel, with ash and slag.	M			APPEARS POORLY COMPACTED
				1			as above, but with high plasticity, brown and light grey, clay, with gravelly sand fill material.	W			
		N = 6 1,2,4	1	2		CH	Silty CLAY: high plasticity, brown and pale grey mottled red brown, with fine to medium grained sub angular and angular ironstone gravel.	w>PL	(S) St		RESIDUAL
		N = 6 3,4,2	0	3							
			-1	4			SANDSTONE: fine to medium grained, light brown and brown, with iron staining.	DW	VL - L		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE
									M		MODERATE TO HIGH RESISTANCE
			-2	5			END OF BOREHOLE AT 4.60 m				'TC' BIT REFUSAL
			-3	6							
			-4								

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Borehole No.
6
1 / 1

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.4 m
Date: 18/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	0	-	ASPHALTIC CONCRETE: 50mm.t				
			FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained, sub angular and angular igneous gravel, with slag and ash.	M			APPEARS POORLY COMPACTED
N = 5 7,2,3			as above, but with metal fibres, rubber fragments.				
			as above, but with cloth fibres.				
N = 2 3,1,1		CH	SILTY CLAY: high plasticity, dark brown, with trace fibres, with fine to medium brown sub angular gravel.	w~LL	VS		ALLUVIAL
			SILTY CLAY: high plasticity, light brown and light grey mottled red brown, with fine to medium grained sub angular and angular ironstone gravel.	w>PL	(S)		RESIDUAL
N = 12 3,4,8			as above, but light grey mottled light brown.		St		
			SANDSTONE: fine to medium grained, light grey and light brown mottled red brown, with XW sandstone bands and ironstone bands.	DW	L - M		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT RESISTANCE
	5.40		END OF BOREHOLE AT 5.40 m				'TC' BIT REFUSAL

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Borehole No.
7

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~3.5 m
Date: 18/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
					ASPHALTIC CONCRETE: 100mm.t				
	N = 9 8,5,4	3		-	FILL: Sand, fine to medium grained, dark brown, with fine to medium grained, sub angular and angular igneous gravel, with ash.	M			APPEARS MODERATELY COMPACTED
					as above, but light brown.				
			1		as above, but dark brown.				
	N = 3 1,2,1	2	2		FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained, sub angular and angular igneous gravel and brick fragments, trace medium plasticity, clay.				APPEARS POORLY COMPACTED
					as above, but with ash, slag and metal fragments.				
▶		1		CH	Silty CLAY: high plasticity, dark brown, with trace of root fibres, and fine to medium sub angular gravel.	w~LL	(VS)		ALLUVIAL
	N = 2 1,1,1	0	3		Silty CLAY: high plasticity, dark brown, trace fine to medium grained sub angular and angular gravel and root fibres.	w>PL	F	50 60 100	
	N > 10 4,6,4/ 80mm REFUSAL	-1	4		as above, but light grey.		(VSt)		RESIDUAL
		-2	5	-	Extremely Weathered sandstone: Clayey SAND, fine to medium grained, light grey, trace ironstone bands, with very low strength sandstone bands.	XW	D		HAWKESBURY SANDSTONE
			6		SANDSTONE: fine to medium grained, light grey mottled light brown and red brown, with ironstone bands, and XW bands.	DW	M		MODERATE 'TC' BIT RESISTANCE
		-3			END OF BOREHOLE AT 6.50 m				'TC' BIT REFUSAL

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

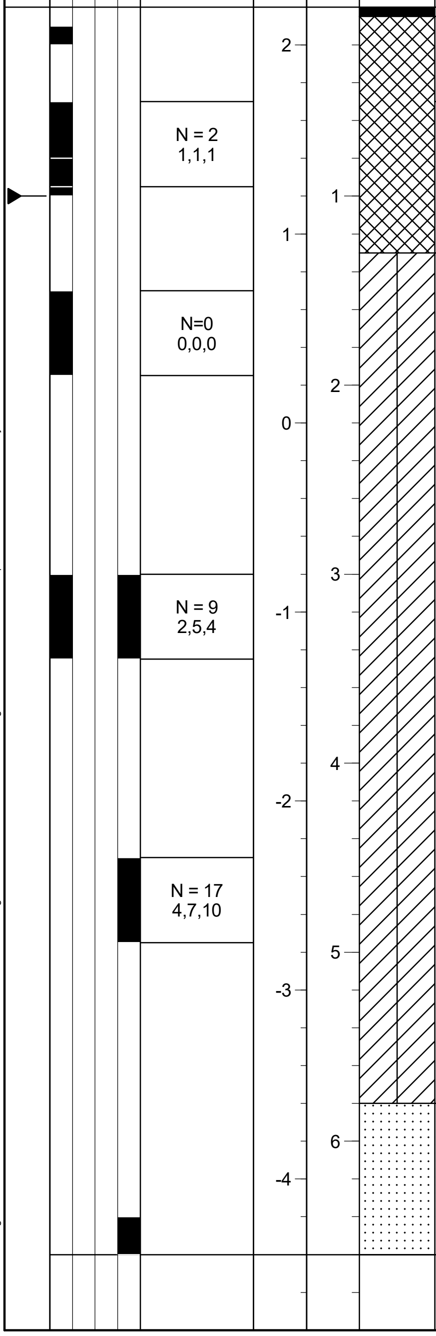
Borehole No.
8
1 / 1

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.2 m
Date: 20/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		ASPHALTIC CONCRETE: 50mm.t.				
	-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained, sub angular and angular igneous gravel.	M			APPEARS POORLY COMPACTED
N = 2 1,1,1		as above, but with slag and ash, trace of brick and glass fragments.	W			
N=0 0,0,0	CH	Silty CLAY: high plasticity, dark brown, with fine to medium grained sub angular and angular ironstone gravel, with trace of root fibres.	w~LL	VS	20 30 20	ALLUVIAL ORGANIC ODOUR
N = 9 2,5,4		Silty CLAY: high plasticity, light brown and grey, mottled red brown, with fine to medium grained sub angular ironstone gravel.	w>PL	St - VSt	150 200 230	RESIDUAL
N = 17 4,7,10		as above but, light grey.		VSt	230 240 300	
		SANDSTONE: fine to medium grained, brown and light brown mottled red brown, with XW sandstone bands and iron staining.	DW	L		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT RESISTANCE
		END OF BOREHOLE AT 6.60 m				'TC' BIT REFUSAL



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Borehole No.
9

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.2 m
Date: 20/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	-	ASPHALTIC CONCRETE: 50mm.t	M			APPEARS POORLY COMPACTED
		FILL: Gravelly sand, fine to medium grained, dark green brown, fine to coarse grained sub angular and angular igneous gravel.				
N = 6 3,3,3		as above, but dark brown, with ash and slag.	W			
N = 5 3,3,2	CH	Silty CLAY: high plasticity, dark brown, with fine to medium grained sub angular and angular ironstone gravel, trace of glass, shell fragments, and root fibers.	w~LL	VS - S		ALLUVIAL
N = 2 0,0,2			w>PL	S	30 40 50	SPT SUNK 300mm UNDER OWN WEIGHT
		Silty CLAY: high plasticity, light grey and light brown, fine to medium grained, sub angular ironstone gravel.	w>PL	F - St	80 90 120	RESIDUAL
				St	120 130 150	
		as above, but light grey.		VSt	180 200 220	
N > 14 5,6,8/ 70mm REFUSAL		Extremely weathered sandstone: Clayey SAND, fine to medium grained, light grey and light brown, with ironstone bands.	XW	VSt - Hd	280 300 320	HAWKESBURY SANDSTONE
		SANDSTONE: fine to medium grained, light grey and light brown mottled red brown, with XW bands.	DW	L		LOW TO MODERATE 'TC' BIT RESISTANCE

RL (m AHD): 2, 1, 0, -1, -2, -3, -4
Depth (m): 1, 2, 3, 4, 5, 6

Groundwater Record; SAMPLES: ES, U50, DB, DS; Graphic Log

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31227S CONCORD.GPJ <<DrawingFile>> 22/05/2018 12:01 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.00.5 2018-01-11

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
9
2 / 2

BOREHOLE LOG

Client: MOUNTIES GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: NULLAWARRA AVENUE, CONCORD, NSW

Job No.: 31227S **Method:** SPIRAL AUGER **R.L. Surface:** ~2.2 m
Date: 20/4/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./M.S.

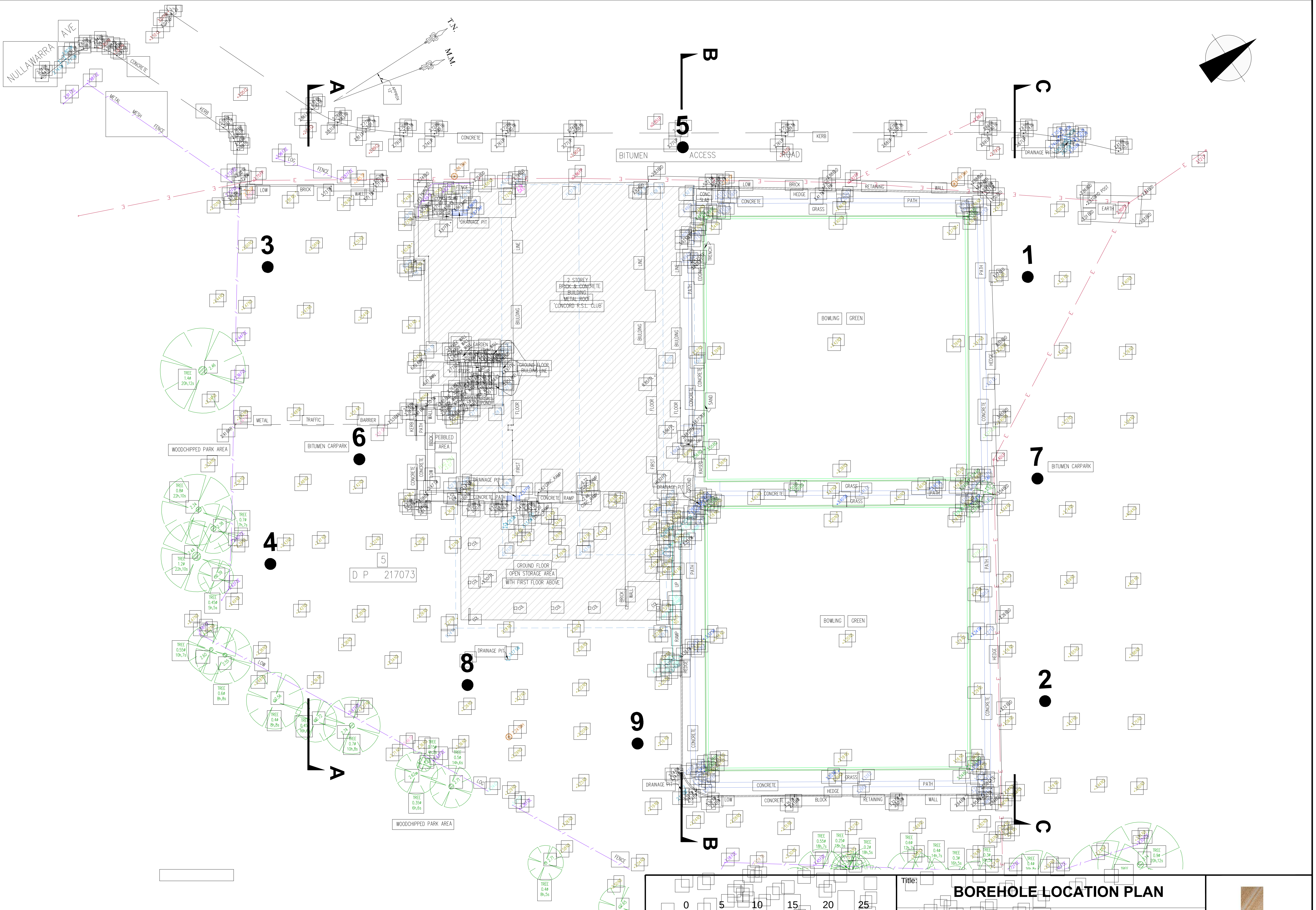
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						-5				SANDSTONE: fine to medium grained, light grey and light brown mottled red brown, with XW bands. *(continued)*	DW	L		LOW TO MODERATE 'TC' BIT RESISTANCE
						-6	8					M		MODERATE RESISTANCE
						-7	9			END OF BOREHOLE AT 8.40 m				'TC' BIT REFUSAL
						-8	10							
						-9	11							
						-10	12							
						-11	13							

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	SITE LOCATION PLAN
Location:	CONCORD RSL & COMMUNITY CLUB NIRRANDA STREET, CONCORD, NSW
Report No: 31227S	Figure No: 1
	JK Geotechnics

PLOT DATE: 24/04/2018 11:00:42 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31227S CONCORD\CAD\31227S.DWG



0 5 10 15 20 25

SCALE 1:500 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: CONCORD RSL & COMMUNITY CLUB
NIRRANDA STREET, CONCORD, NSW

Report No: 31227S

Figure No: 2

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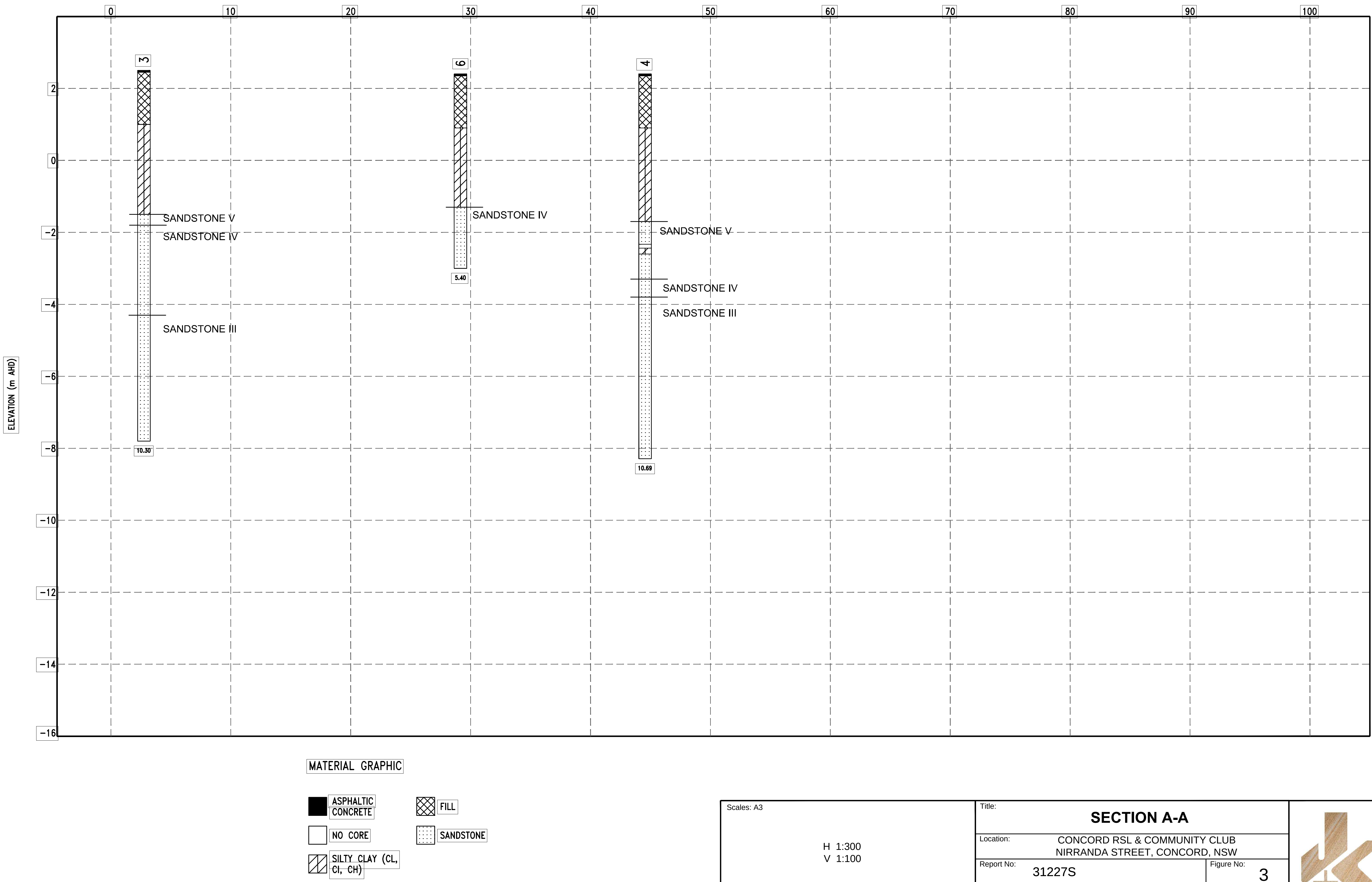
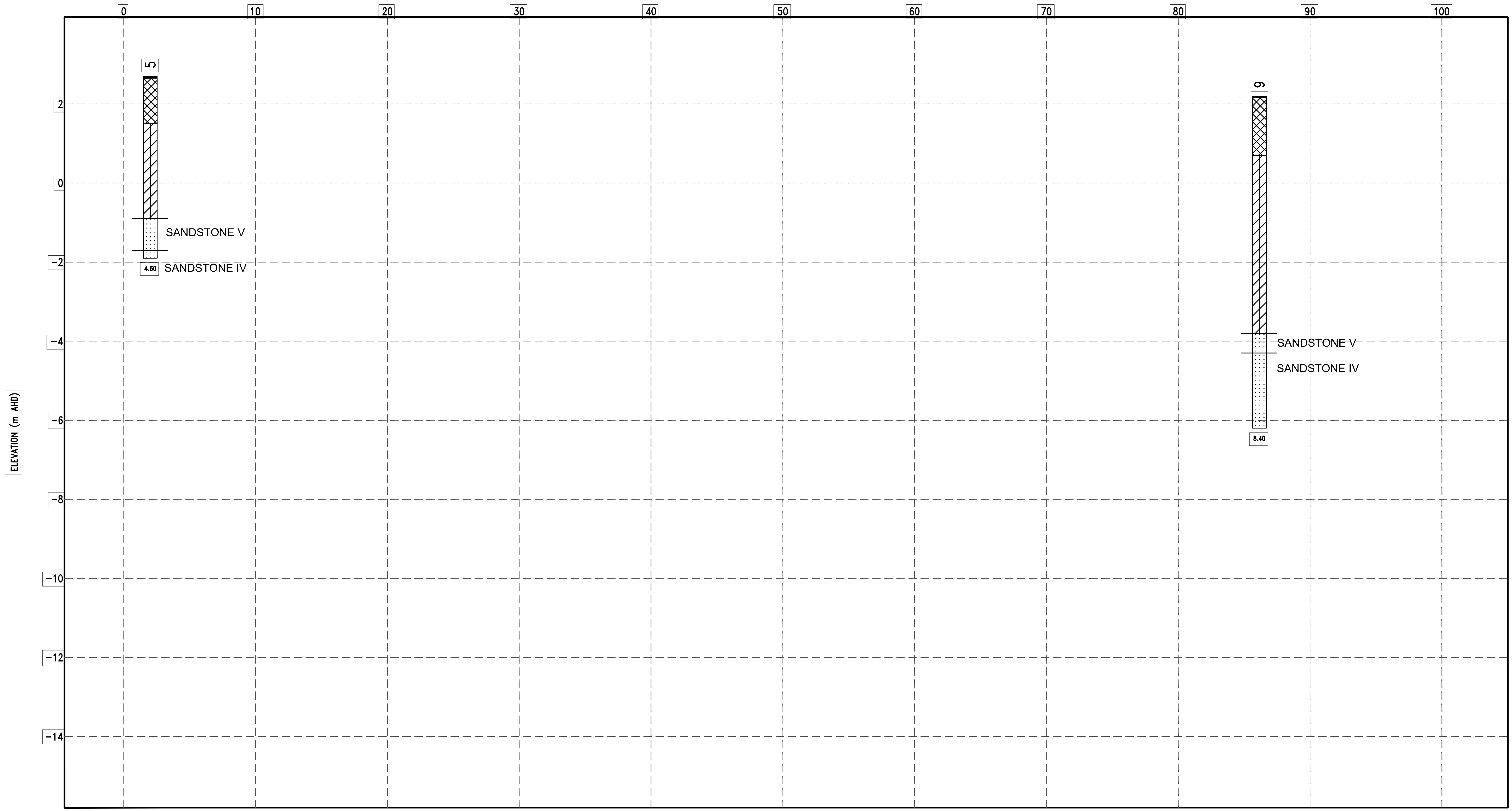
PLOT DATE: 18/05/2018 12:29:24 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31227S CONCORD\CAD\31227S.DWG



Scales: A3	Title: **SECTION A-A**	
H 1:300 V 1:100	Location: CONCORD RSL & COMMUNITY CLUB NIRRANDA STREET, CONCORD, NSW	
	Report No: 31227S	Figure No: 3
This plan should be read in conjunction with the JK Geotechnics report.	JK **Geotechnics**	

PLOT DATE: 22/05/2018 4:37:08 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31227S CONCORD\CAD\31227S.DWG



MATERIAL GRAPHIC

- ASPHALTIC CONCRETE
- SILTY CLAY (CL, CI, CH)
- FILL
- SANDSTONE

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title: **SECTION B-B**

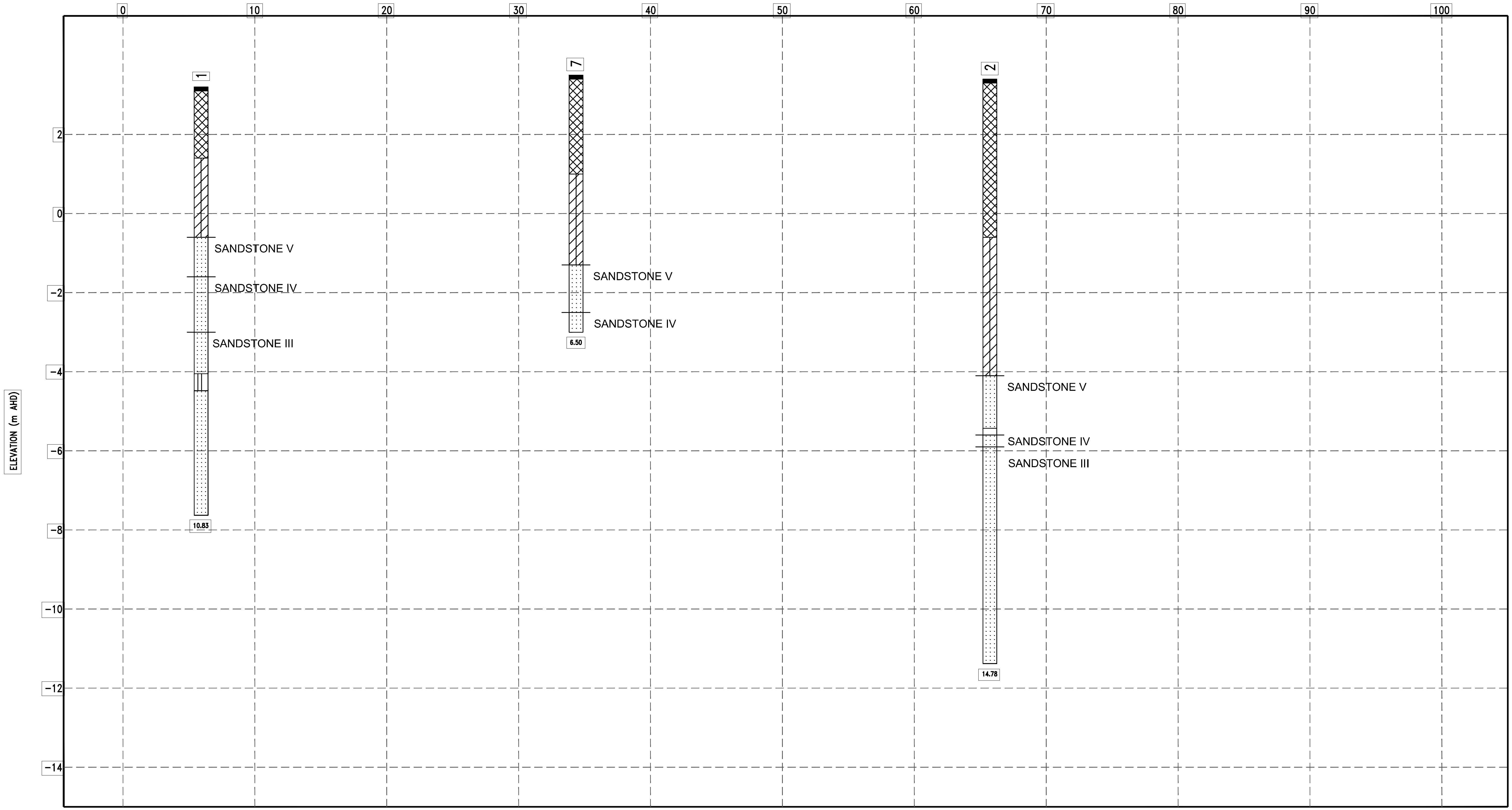
Location: CONCORD RSL & COMMUNITY CLUB
NIRRANDA STREET, CONCORD, NSW

Report No: 31227S

Figure No: 4

JK **Geotechnics**

PLOT DATE: 22/05/2018 4:52:05 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31227S CONCORD\CAD\31227S.DWG



MATERIAL GRAPHIC

- ASPHALTIC CONCRETE
- NO CORE
- SILTY CLAY (CL, CI, CH)
- FILL
- SANDSTONE
- SILTSTONE

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title: **SECTION C-C**

Location: CONCORD RSL & COMMUNITY CLUB
NIRRANDA STREET, CONCORD, NSW

Report No: 31227S

Figure No: 5

JK **Geotechnics**

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	ROCK
FILL	CONGLOMERATE
TOPSOIL	SANDSTONE
CLAY (CL, CI, CH)	SHALE/MUDSTONE
SILT (ML, MH)	SILTSTONE
SAND (SP, SW)	CLAYSTONE
GRAVEL (GP, GW)	COAL
SANDY CLAY (CL, CI, CH)	LAMINITE
SILTY CLAY (CL, CI, CH)	LIMESTONE
CLAYEY SAND (SC)	PHYLLITE, SCHIST
SILTY SAND (SM)	TUFF
GRAVELLY CLAY (CL, CI, CH)	GRANITE, GABBRO
CLAYEY GRAVEL (GC)	DOLERITE, DIORITE
SANDY SILT (ML, MH)	BASALT, ANDESITE
PEAT AND HIGHLY ORGANIC SOILS (Pt)	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:

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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:

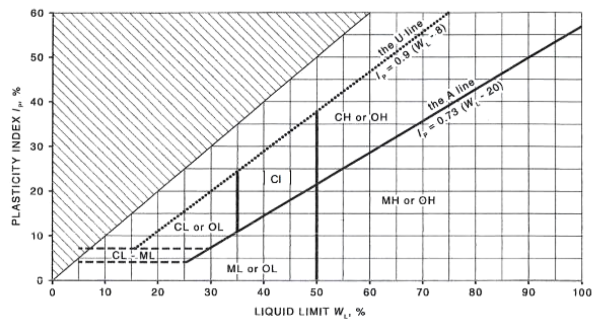
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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 U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. 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 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE — ≤ 15 — 0 – 4
	L	LOOSE — > 15 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.
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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength: Point Load Strength Index $Is_{(50)}$ (MPa)	Guide to Strength: 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