SPECIALIST ADVICE TO

BLUECHP

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

15-17 LUPIN AVENUE AND 82 BELMORE STREET, FAIRFIELD, NSW

Date: 19 May 2023
Ref: 35918PNrpt

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DOCUMENT REVISION RECORD

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ATTACHMENTS

1 INTRODUCTION

This specialist advice report presents the results of a geotechnical investigation for a proposed residential development at 15-17 Lupin Avenue & 82 Belmore Street, Fairfield East, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by email from Gareth Bird of Bluechp dated 10 March 2023, on the basis of our proposal P58278PNrev1 dated 9 March 2023.

From the provided architectural drawings prepared by Loucas Architects (Project No. Pn-21020, Drawing Nos. A-800 to A-1500, Rev. B, dated 20 April 2023), we understand the proposed development will comprise a six-storey residential apartment building over two levels of basement carparking. The floor of the lowest basement level will vary between RL14.05m and RL15.25m, and excavation to a maximum depth of about 7.5m below existing levels is expected to be required. The basement will abut the southern boundary and will be set back between about 5m from the remaining boundaries. As no structural loads have been provided, typical loads for this type of development have been assumed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on geotechnical aspects of the proposed development such as excavation conditions, hydrogeological condition, shoring options and design parameters, footing design, and on-grade floor slabs.

This report provides specialist advice for use by the structural and civil designers in preparing their designs and no part of this report is considered a regulated design in accordance with the Design and Building Practitioners Act 2020.

2 INVESTIGATION PROCEDURE

Two boreholes, BH2 and BH4, were drilled to depths of 5.60m and 4.55m, respectively, using spiral augering techniques with our truck mounted JK309 drill rig. Both boreholes were subsequently extended to final depths of 10.0m (BH2) and 7.75m (BH4) using rotary diamond coring techniques with water flush. At two additional locations where access was not available for the drilling rig, boreholes were drilled using hand auger techniques to refusal at depths of 1.5m (BH1) and 1.3m (BH3). The compaction of the fill and strength of the residual soils were assessed from the Standard Penetration Test (SPT) 'N' values and Dynamic Cone Penetrometer (DCP) blow counts, and correlation with the results of hand penetrometer tests completed on recovered disturbed soil samples.

The strength of the augered bedrock was assessed from observation of the drilling resistance of a tungsten carbide (TC) drill bit attached to the augers, tactile examination of recovered rock chips, and correlation with the results of subsequent laboratory moisture content tests. The assessment of rock strength in this manner is approximate and variation by about one order of strength should not be unexpected. The strength of the cored bedrock was assessed from tactile examination of the recovered rock core and the results of subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests, the results of which are presented on the attached Table B and are plotted on the cored borehole logs.

Groundwater observations were made in the boreholes during and on the completion of drilling each borehole. Groundwater monitoring wells were installed in BH2 and BH4 to depths of 10.0m and 7.21m, respectively, with their response zone wholly in the bedrock profile, and a cast-iron ‘Gatic’ cover was concreted flush with the ground surface to protect the top of each groundwater monitoring well. The installation details are presented on the relevant borehole log. Long-term groundwater level monitoring in each well was outside the scope of the geotechnical investigation.

The borehole locations, as shown on the attached Borehole Location Plan (Figure 2) were set out by tape measurements from existing surface features. Figure 2 is based on available Nearmap (aerial) imagery. As no topographical survey was provided, the existing surface levels at the borehole locations were not estimated.

Our geotechnical engineer, Quang Minh Vu, was on site full-time during the fieldwork and set out the borehole locations, nominated the sampling and testing, and prepared the attached borehole logs and DCP test result sheet. For details of the investigation techniques adopted and a glossary of logging terms and symbols used, reference should be made to the attached Report Explanation Notes.

Selected rock chip samples were returned to the Soil Test Services (STS) NATA registered laboratory for moisture content testing. The results of these tests are presented in the attached STS Table A. Additional samples were sent to the Envirolab Services NATA registered analytical laboratory for pH, sulfate content, chloride content and resistivity; with the results presented in the attached Envirolab ‘Certificate of Analysis’ 320869.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in hilly terrain, characterised by maximum surface gradients of less than 5°, and the site itself grades to the north at about 1°. Belmore Street and Lupin Avenue bound the site to the north and west, respectively.

At the time of the fieldwork, three brick and clad frame houses, of one and two stories, along with sheds and granny flats, occupied the majority of the site, with landscaping surrounding the buildings.

The neighbouring property to the east of the site contained a single-storey brick house and separate garage, with the garage abutting the site boundary.

The neighbouring property to the south of the site contained a single storey clad frame house, which was set back about 1m from the site boundary.

Ground surface levels across the boundaries were similar.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates the site to be underlain by Bringelly Shale of the Wianamatta Group.

The boreholes encountered a generalised subsurface profile comprising shallow fill overlying residual clayey soils, then siltstone bedrock from moderate depth. Groundwater was not encountered during the investigation. For details of the conditions encountered at each location, reference should be made to the attached borehole logs and DCP test result sheet. A summary of the encountered conditions is presented below.

Pavements and Fill

Reinforced concrete 130mm thick was penetrated from the surface in BH2.

Fill comprising predominantly clayey soils was encountered from beneath the pavement (BH2) or from surface in the remaining boreholes, to a maximum depth of 0.5m, and was assessed as being poorly compacted where tested.

Residual Silty Clay

Residual silty clay of medium to high or high plasticity was encountered beneath the fill in all four boreholes. The clays were initially of stiff to very stiff strength, and improved to hard strength with depth.

Bedrock

Bedrock, predominantly comprising siltstone but with some sandstone bands, was encountered in BH2 and BH4 from a depth of 2.5m. Whilst the DCP equipment does not provide sample return, the similar refusal depth of the DCP tests (2.2m or 2.6m) is inferred to be on, or close to bedrock.

In BH2 and BH4, the bedrock was predominantly extremely weathered and of hard (soil) strength to about 6m or 7m depth, below which the bedrock was of highly weathered and very low to low strength. Further the bedrock below about 9m depth in BH2 was of low to medium strength. The 'no core' zone in BH2 likely represents extremely weathered bedrock which was washed away during coring.

Groundwater

All boreholes were 'dry' during and on completion of auger drilling. As water is injected into the borehole during coring, meaningful water level measurements could not be taken on completion of coring in each borehole. No long term groundwater monitoring was completed.

3.3 Laboratory Test Results

The results of the moisture content and Point Load Strength Index ($I_{S(50)}$) tests correlated well with our field assessment of the bedrock strength. The UCS of the bedrock, as assessed using the commonly adopted correlation of UCS = 20 x $I_{S(50)}$ was generally between 1MPa and 8MPa, with one result of 14MPa from the base of BH2.

The results of the Envirolab testing indicated the soils to be moderately acidic to neutral, with moderate chloride and sulfate content, and low (unfavourable) resistivity.

4 COMMENTS AND RECOMMENDATIONS

4.1 Dilapidation Surveys

Prior to any demolition and excavation commencing, we recommend that detailed dilapidation reports be prepared for the adjoining properties to the east and south of the site. The dilapidation surveys should comprise detailed inspections both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of existing conditions. Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works. We note that Council may also require that any damage to their adjoining assets be reported prior to any works commencing on site.

4.2 Excavation and Vibration

4.2.1 Excavation Conditions

Excavation for the proposed development is expected to extend to a maximum depth of about 7.5m below existing surface levels, with locally deeper excavations for lift pits and services. Excavation to such depths will extend through the soil profile, and be mostly within the siltstone bedrock profile

Following installation of appropriate shoring, excavation of the soils, extremely weathered bedrock, and very low strength bedrock is expected to be readily achievable using conventional techniques such as the buckets of large hydraulic excavators, with the assistance of ripping hooks where required.

Excavation through the siltstone bedrock of low and greater strength will be expected to be slower, and we recommend grid sawing and hammering with smaller excavators and/or ripping using a large excavator (at least 30 tonne in size).

4.2.2 Potential Vibration Risks

We recommend that considerable caution be taken during rock excavation on the site as there will likely be direct transmission of ground vibrations to the neighbouring buildings to the east and south.

The dilapidation reports and the excavation procedures should be carefully reviewed prior to the commencement of excavation, so that appropriate equipment is used.

Excavation using hydraulic rock hammers (if adopted) should commence away from likely critical areas (i.e. commence in the north-western corner of the site). We recommend that continuous vibration monitoring be carried out during all demolition and excavation works. Vibrations, measured as Peak Particle Velocity (PPV), must be limited to no higher than 5mm/sec for the nearby buildings, subject to confirmation by the project structural engineer and/or a specialist vibration consultant that these vibration levels can be tolerated by those structures. This vibration limit must also be reviewed following completion of the dilapidation reports on the nearby buildings. If higher vibrations are recorded, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative techniques, e.g. grid sawing in conjunction with ripping.

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

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Use rock hammers in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil on the hammer.

We recommend use of excavation contractors with experience in such work with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

4.2.3 Seepage

The boreholes were all ‘dry’ during and on completion of auger drilling.

Given the expected generally low permeability of the soil and bedrock profiles, we expect that construction of a drained basement would be feasible and appropriate. Groundwater seepage into the basement excavation would be expected to reduce as the excavation progresses, and any small amounts of water stored in void spaces and fissures in the surrounding profile is drained of water. Locally higher inflows could be expected to occur through open joints or bedding planes during and following heavy rainfall events. Such a process would not be expected to cause any adverse effects on any surrounding structures or improvements.

Long term groundwater flows would be expected to be of limited volume and would be able to be controlled by draining them to a sump for periodic pumped disposal to the stormwater system.

Groundwater seepage into all excavations should be monitored by the site foreman and geotechnical engineer as excavation progresses to confirm that seepage volumes are within the range anticipated.

Overall, it is considered that the construction of the proposed development will not be adversely affected by groundwater provided engineer designed drainage systems are constructed. Similarly, it is not expected that the development will have an adverse effect on the surrounding structures or improvements, or on regional groundwater flows.

4.3 Shoring

4.3.1 Shoring Options

Where space permits, temporary batter slopes within the soil and bedrock profiles of 1 Vertical (V) to 1 Horizontal (H) are recommended in the short term, provided that no surcharge loads, including construction loads and existing footing loads, are placed at the top of the batters. If batter slopes exceed 3m height, a horizontal bench not less than 1m wide must be adopted mid height of the batter. However, based on the depth and geometry of the proposed excavation, such batter slopes are not expected to be feasible. Therefore, we consider that the excavation will need to be supported by a full depth engineer designed shoring system installed prior to excavation commencing.

The shoring system(s) may be incorporated into the permanent basement retention system. The effect of ground movements on any structures and services that lie within the influence zone of the excavation must also be taken into account. The influence zone of the excavation may be defined as a horizontal distance of 2H (where 'H' is the depth of the excavation in metres) behind the wall. Suitable retention systems, given the subsoil conditions encountered, would include a soldier pile wall with shotcrete infill panels, or a contiguous pile wall. Conventional bored piles are considered suitable for use on this site.

To reduce excavation induced movements along site boundaries, the shoring system(s) must be anchored or braced as the excavation progresses. Approval from neighbouring land owners would be required prior to the installation of anchors into their property.

The piles should be installed to sufficient depth below bulk excavation level, including below footing and service excavations, to provide adequate toe restraint.

Drilling of rock sockets into the low to medium strength bedrock below bulk excavation level may be difficult, requiring the use of medium to large piling rigs equipped with rock augers. Some groundwater inflow may occur into bored piles and we expect that such inflow will be controllable by conventional pumping methods.

Given the expected depth of the shoring piles, pouring using tremie methods is recommended. Constructing conventional bored piles where seepage is occurring can be time consuming and difficult, and consideration should be given to drilling a trial pile in the middle of the site. If the drilling and cleaning process is found to be problematic, continuous flight auger (CFA) piles could be used to overcome this issue.

4.3.2 General Shoring Design Parameters

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

For anchored or propped soldier pile or contiguous pile walls where minor movements can be tolerated, e.g. landscaped areas or similar, we recommend the use of a trapezoidal earth pressure distribution of 6HkPa for the soil profile and the bedrock of up to low strength, where 'H' is the retained height in metres. These pressures should be assumed to be uniform of the central 50% of the support system, tapering to zero at the toe and crest of the excavation.\. Where movements are to be limited, e.g. where neighbouring structures or movement sensitive services are located within 2H of the wall, a trapezoidal earth pressure distribution of 8HkPa should be adopted. The shotcrete infill panels can be designed for 4H and 6H respectively.

Any surcharge affecting the walls (e.g. traffic loading, sloping backfill surfaces, construction loads, adjacent high level footings, etc.) should be allowed for in the design using an 'at-rest' earth pressure coefficient, K_0, of 0.5. A bulk unit weight of $21kN/m^3$ should be adopted for the soil and bedrock.

The retaining walls should be designed as 'drained' and measured taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

Lateral toe restraint of the walls may be achieved by embedding the piles into the bedrock below bulk excavation level. An allowable lateral resistance of 200kPa can be adopted for bedrock of consistent very low or greater strength below bulk excavation level. For piles embedded into bedrock below bulk excavation level, a minimum embedment depth of 1.0m is recommended. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, tanks, buried services, etc. must be taken into account in the wall design.

Anchors bonded into extremely weathered bedrock may be designed on the basis of a maximum allowable bond stress of 100kPa. We note that the bedrock will be susceptible to softening when in contact with water, we recommend all anchor drilling be completed suing air methods only. If water ingress into the anchor holes occurs, the resulting softened 'skin' will likely result in a significant reduction in the bond stress which can be achieved. All anchors should be proof loaded to at least 1.3 times their working load and then locked off/prestressed to 85% of their working load. Proof loading should be carried out in the presence of an engineer independent of the anchor contractor. Anchors must be bonded behind a line drawn up at 45° from the base of the excavation, with all anchors having minimum free and bond lengths of 3m each. Lift off tests should be carried out on at least 10% of all anchors to confirm that they are maintaining their load. It is normal practice for anchor design and construction to be a separate subcontract so that disputes do not arise if any anchors fail to achieve their test load.

Alternatively, the retaining walls could be designed using computer based Finite Element Analysis methods (e.g. Plaxis), which could result in cost savings compared to a design based on the above simplified earth pressure assumptions. Analysis software treating the soil as 'equivalent springs' should not be used for this design. Plaxis type analysis methods can model the actual excavation stages, including progressive anchoring/shoring, and outputs include structural actions in the piles, anchor/prop loads, and wall movements. The analysis should be completed by an engineer with a good understanding of soil-structure interaction behaviour, including an understanding of when soil-wall friction should and should not be used etc. We can complete such analysis if commissioned to do so.

4.4 Footings

On completion of excavation, bedrock of at least very low strength is expected to be uniformly exposed.

For footings founded in siltstone bedrock of consistent very low or greater strength, an allowable bearing pressure of 1,000kPa can be adopted based on a serviceability criteria of limiting the settlement to not more that 1% of the least footing dimension. Perimeter shoring piles founded below bulk excavation level could also be designed based on an allowable bearing pressure of 1,000kPa.

The following inspection regime is recommended for load bearing piles and footings for this project:
- The socket below bulk excavation level for all piles should be drilled in the presence of a geotechnical engineer; and
- All footing excavations should be visually inspected by a geotechnical engineer.

Special consideration will need to be given to any footings which are located close to the crest of excavation cuts within the basement (e.g. near lift pits). Any footings above a 45° line rising from the base of any excavations will require special consideration, and a thorough inspection of the nearby rock face will be required.

4.5 On-Grade Floor Slab

The basement on-grade floor slab is expected to directly overlie bedrock, and no particular subgrade preparation will be required. Slab-on-grade construction is therefore considered appropriate. Underfloor drainage must however be provided. The underfloor drainage should connect with the wall drains, where appropriate, and direct groundwater seepage to a sump(s) for pumped disposal to a stormwater system following obtaining authority approval. Joints in the on-grade floor slabs should incorporate dowels or keys, and a joint must be adopted at the interface between a soil and bedrock subgrade.

4.6 Soil Aggression

Based on the soil chemistry test results, a 'Mild' exposure classification for concrete piles is applicable in accordance with Table 6.4.2 (C) in AS2159-2009.

4.7 Earthquake Design

In accordance with Table 4.1 of AS1170.4-2007, the site subsoil class is 'Class B_e – Bedrock'.

4.8 Further Geotechnical Input

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

- Finite element analysis of basement retaining walls (if required);
- Quantitative vibration monitoring during rock excavation;
- Geotechnical pile inspections;
- Proof-testing of anchors;
- Geotechnical inspection of bedrock cut faces near footings;
- Seepage monitoring during excavation; and
- Geotechnical footing inspections and spoon testing.

5 SALINITY

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

6 GENERAL COMMENTS

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

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

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

If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A

MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Residential Development
Location: 15-17 Lupin Avenue & 82 Belmore Street, Fairfield, NSW

Report No.: 35918PN - A
Report Date: 2/05/2023
Page 1 of 1

AS 1289	TEST METHOD	2.1.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
2	4.00 - 4.20	8.2
2	5.00 - 5.20	9.5
4	2.50 - 3.00	6.8
4	4.00 - 4.50	5.7

Notes:

- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 28/04/2023.
- Sampled and supplied by client. Samples tested as received.

NATA
NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
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the items tested or sampled.

02/05/2023
Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Bluechp	**Ref No:**	35918PN
Project:	Proposed Residentila Development	**Report:**	B
Location:	15-17 Lupin Avenue & 82 Belmore Street, FAIRFIELD, NSW	**Report Date:**	6/04/23

Page 1 of 1

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	7.31 - 7.35	0.2	4	A
	7.82 - 7.86	0.2	4	A
	8.02 - 8.06	0.03	1	A
	8.73 - 8.75	0.2	4	A
	9.37 - 9.40	0.3	6	A
	9.89 - 9.92	0.4	8	A
4	4.55 - 4.59	0.7	14	A
	6.10 - 6.13	0.04	1	A
	6.71 - 6.74	0.04	1	A
	7.22 - 7.24	0.06	1	A
	7.56 - 7.60	0.02	<1	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
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 based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' 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 320869

Client Details

Client	JK Geotechnics
Attention	Quang Minh Vu
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35918PN, Fairfield**
Number of Samples	4 Soil
Date samples received	14/04/2023
Date completed instructions received	14/04/2023

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	21/04/2023
Date of Issue	21/04/2023

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Jenny He, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference	UNITS	320869-1	320869-2	320869-3	320869-4
Your Reference		BH2	BH2	BH4	BH4
Depth		1.5-1.95	2.6-2.8	0.5-0.95	1.5-1.95
Date Sampled		05/04/2023	05/04/2023	05/04/2023	05/04/2023
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	20/04/2023	20/04/2023	20/04/2023	20/04/2023
Date analysed	-	20/04/2023	20/04/2023	20/04/2023	20/04/2023
pH 1:5 soil:water	pH Units	6.2	7.2	4.8	7.0
Chloride, Cl 1:5 soil:water	mg/kg	200	240	200	390
Sulphate, SO4 1:5 soil:water	mg/kg	300	210	400	190
Resistivity in soil*	ohm m	38	37	33	37

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 (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

Client Reference: 35918PN, Fairfield

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	320869-2
Date prepared	-			20/04/2023	1	20/04/2023	20/04/2023		20/04/2023	20/04/2023
Date analysed	-			20/04/2023	1	20/04/2023	20/04/2023		20/04/2023	20/04/2023
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	6.2	6.1	2	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	200	190	5	105	#
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	300	290	3	96	#
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	38	38	0	[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.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

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: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-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.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

MISC_INORG_DRY:# Percent recovery is not applicable due to the high concentration of the analyte/s in the sample/s. However an acceptable recovery was obtained for the LCS.

MISC_INORG_DRY:pH:Samples were out of the recommended holding time for this analysis.

BOREHOLE LOG

Borehole No.
1
1 / 1

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 5/4/23 **Datum:** AHD
Plant Type: **Logged/Checked By:** Q.V./N.E.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS SHEET				FILL: Silty clay, medium plasticity, dark grey, dark brown and yellow brown, with fine to medium grained igneous gravel, trace of fine grained sand, and root fibres.	w>PL			GRASS COVER APPEARS POORLY COMPACTED
						1		CI-CH	Silty CLAY: medium to high plasticity, light grey, red brown and yellow brown, with fine to medium grained ironstone gravel, trace of root fibres.	w>PL	F - St St - VSt	150 120 130 170 170 190 250 250 270	RESIDUAL HP ON REMOULDED SAMPLE HP ON REMOULDED SAMPLE HP ON REMOULDED SAMPLE HAND AUGER REFUSAL ON HARD CLAY
						2 3 4 5 6			END OF BOREHOLE AT 1.50 m				

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BOREHOLE LOG

Borehole No.
2
1 / 2

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN **Method:** SPIRAL AUGER **R.L. Surface:** N/A
Date: 5/4/23 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING									CONCRETE: 130mm.t				7mm DIA. REINFORCEMENT, 100mm TOP COVER
								-	FILL: Silty clay, medium plasticity, dark brown and brown, with fine to medium grained sand and fine to medium grained igneous gravel, trace of root fibres.	w>PL			
					N = 5 2,2,3	1		CI-CH	Silty CLAY: medium to high plasticity, light grey, red brown and yellow brown, with fine to medium grained ironstone gravel, trace of root fibres.	w>PL	St		RESIDUAL
					N = 20 8,9,11	2			as above, but yellow brown, dark grey and light grey.	w~PL	Hd		
						3		-	Extremely Weathered siltstone: silty CLAY, low plasticity, dark brown, yellow brown and red brown, with fine to medium grained ironstone gravel and very low strength bands.	XW	Hd		BRINGELLY SHALE
													VERY LOW 'TC' BIT RESISTANCE
						4							VERY LOW TO LOW RESISTANCE
						5							LOW RESISTANCE
						6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 10.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.0m TO 10.0m. CASING 0.05m TO 6.0m. 2mm SAND FILTER PACK 6.0m TO 10.0m. BENTONITE SEAL 5.5m TO 6.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN
Core Size: NMLC
R.L. Surface: N/A
Date: 5/4/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK309
Bearing: N/A
Logged/Checked By: Q.V./N.E.S.

Water Loss\Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
				START CORING AT 5.60m						
90% RETURN		6		Extremely Weathered siltstone: silty CLAY, medium plasticity, dark brown, yellow brown and dark grey, with occasional very low strength siltstone bands and ironstone bands.	XW	Hd				Bringelly Shale
		7 8		SILTSTONE: dark grey, red brown and dark brown, with occasional extremely weathered bands, bedded at 0-10°.	HW	VL - L	0.20 0.20 0.030		(7.13m) J, 90°, Ir, R, Fe Sn (7.50m) XWS, 0°, 150 mm.t (7.72m) XWS, 0°, 20 mm.t (7.99m) XWS, 0°, 20 mm.t (8.11m) J, 70°, Ir, S, Fe Vn	
				Extremely Weathered siltstone: silty CLAY, medium plasticity, dark brown, yellow brown and dark grey, with occasional very low strength bands.	XW	Hd		600 200 60 20		
		9		SILTSTONE: dark grey and grey, bedded at 0-5°.	HW	VL - L	0.20		(8.55m) J, 90°, Ir, S, Fe Sn (8.67m) Be, 0°, P, S, Clay FILLED, 1 mm.t (8.82m) Be, 0°, P, S, Cn (8.90m) XWS, 0°, 17 mm.t (8.99m) XWS, 0°, 40 mm.t	
				NO CORE 0.15m						
		10		SILTSTONE: dark grey and grey, bedded at 0-5°.	HW	L - M	0.30 0.40		(9.23m) XWS, 0°, 150 mm.t (9.83m) Be, 0°, P, S, Clay FILLED, 2 mm.t	Bringelly Shale
		11		END OF BOREHOLE AT 10.00 m				600 200 60 20		

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Job No: 35918PN
Borehole No: BH2
Depth: 5.60m to 10.00m

35918PN BH2 START CORING AT 5.60 m

NO CORE: 0.15m

END OF BOREHOLE @ 10.0m

BOREHOLE LOG

Borehole No.
3
1 / 1

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 5/4/23 **Datum:** AHD
Plant Type: **Logged/Checked By:** Q.V./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				■	REFER TO DCP TEST RESULTS SHEET		▩		FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained igneous gravel, and clay nodules, trace of root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
				■			▩		FILL: Silty clay, medium plasticity, dark brown and brown, with fine to medium grained sand and fine to medium grained igneous gravel, trace of root fibres.	w>PL			
				■ ■		1	▨	CH	Silty CLAY: high plasticity, light grey and red brown, with fine to medium grained ironstone gravel, trace of root fibres.	w>PL	St - VSt VSt	200 220 250 350 380 380	RESIDUAL
						2 3 4 5 6			END OF BOREHOLE AT 1.30 m				HAND AUGER REFUSAL ON HARD CLAY

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BOREHOLE LOG

Borehole No.
4
1 / 2

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN
Method: SPIRAL AUGER
R.L. Surface: N/A
Date: 5/4/23
Datum: AHD
Plant Type: JK309
Logged/Checked By: Q.V./N.E.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING							▩		FILL: Silty clay, low plasticity, dark brown and dark grey, with fine to medium grained sand and fine to medium grained igneous and ironstone gravel, trace of root fibres.	w>PL			GRASS COVER
				■	N = 7 3,3,4	1	▨	CH	Silty CLAY: high plasticity, light grey, red brown and yellow brown, with fine to medium grained ironstone gravel, trace of root fibres.	w>PL	St - VSt	380 420 350	RESIDUAL
				■	N = 14 4,6,8	2	▨		as above, but light grey and yellow brown, with fine to medium grained ironstone gravel, trace of root fibres.	w<PL	Hd	>600 >600	
				■		3	░	-	SANDSTONE: fine to medium grained, light grey and light brown, with occasional ironstone bands.	HW	VL - L		BRINGELLY SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
				■		4	▥		Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey and grey, with very low and low strength bands.	XW	Hd		LOW RESISTANCE
						5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 7.21m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.0m TO 7.21m. CASING 0.05m TO 3.0m. 2mm SAND FILTER PACK 3.0m TO 7.21m. BENTONITE SEAL 2.5m TO 3.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
4
2 / 2

Client: BLUECHP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No.: 35918PN
Core Size: NMLC
R.L. Surface: N/A
Date: 5/4/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK309
Bearing: N/A
Logged/Checked By: Q.V./N.E.S.

Water Loss\Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	Formation
				START CORING AT 4.55m					
80% RETURN				SILTSTONE: dark grey and grey, bedded at 0-5°.	HW	L - M	0.70		Bringelly Shale
		5		Extremely Weathered siltstone: silty CLAY, medium plasticity, dark grey, red brown and brown, with occasional very low strength siltstone bands.	XW	Hd			
		6 7		SILTSTONE: dark grey and red brown, bedded at 0-10°.	HW	VL	0.040 0.040 0.060	(6.17m) XWS, 0°, 40 mm.t (6.60m) XWS, 0°, 40 mm.t (6.85m) Be, 0°, P, S, Clay FILLED, 1 mm.t	
				SILTSTONE: dark grey and grey, bedded at 0-5°.	MW			(7.47m) Jx 2, 40°, Ir, R, Cn (7.67m) J, 90°, Ir, S, Cn	
		8 9 10		END OF BOREHOLE AT 7.75 m					

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 35918PN
Borehole No: BH4
Depth: 4.55m to 7.75m

DYNAMIC CONE PENETRATION TEST RESULTS

Client: BLUECHIP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 LUPIN DRIVE & 82 BELMORE STREET, FAIRFIELD, NSW

Job No. 35918PN
Date: 5-4-23
Tested By: Q.V.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	**1**	**3**					
Surface RL	N/A	N/A					
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK	SUNK					
100 - 200	2	2					
200 - 300	1	4					
300 - 400	2	4					
400 - 500	1	2					
500 - 600	1	3					
600 - 700	2	3					
700 - 800	2	2					
800 - 900	4	2					
900 - 1000	3	3					
1000 - 1100	3	2					
1100 - 1200	4	4					
1200 - 1300	4	4					
1300 - 1400	4	5					
1400 - 1500	5	7					
1500 - 1600	13	8					
1600 - 1700	10	9					
1700 - 1800	18	8					
1800 - 1900	21	10					
1900 - 2000	16	17					
2000 - 2100	15	15					
2100 - 2200	13	14					
2200 - 2300	18	6/30mm					
2300 - 2400	21	REFUSAL					
2400 - 2500	25						
2500 - 2600	34						
2600 - 2700	END						
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET,
FAIRFIELD, NSW

Report No: 35918PN

Figure No: 1

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PLOT DATE: 11/04/2023 2:12:38 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\35000'S\35918PN FAIRFIELD\CAD\35918PN.DWG

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LEGEND

● BOREHOLE

⊕ BOREHOLE AND DCP TEST

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title: **INVESTIGATION LOCATION PLAN**

Location: 15-17 LUPIN AVENUE & 82 BELMORE STREET,
FAIRFIELD, NSW

Report No: 35918PN

Figure No: 2

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PLOT DATE: 11/04/2023 2:19:25 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\35000'S\35918PN FAIRFIELD\CAD\35918PN.DWG

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VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

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

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

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

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

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

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

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

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

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

- ◌ CONGLOMERATE
- ░ SANDSTONE
- ▤ SHALE/MUDSTONE
- ▥ SILTSTONE
- ▤ CLAYSTONE
- ■ COAL
- ▥ LAMINITE
- ▦ LIMESTONE
- ≋ PHYLLITE, SCHIST
- ▤ TUFF
- ▒ GRANITE, GABBRO
- ✛ DOLERITE, DIORITE
- ⩗ BASALT, ANDESITE
- ≈ QUARTZITE

OTHER MATERIALS

- ▦ BRICKS OR PAVERS
- ▒ CONCRETE
- ■ ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory **Classification** Criteria

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

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

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:

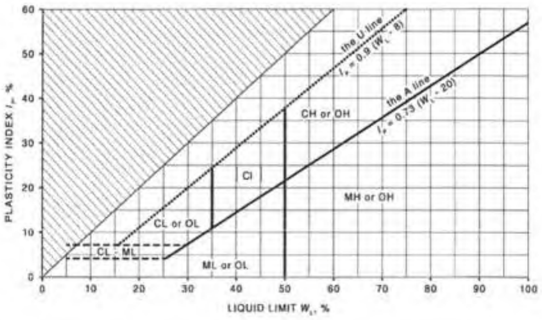
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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	Sample taken over depth indicated, for environmental analysis.		
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.		
	DB	Bulk disturbed sample taken over depth indicated.		
	DS	Small disturbed bag sample taken over depth indicated.		
	ASB	Soil sample taken over depth indicated, for asbestos analysis.		
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.		
	SAL	Soil sample taken over depth indicated, for salinity analysis.		
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	VNS = 25	Vane shear reading in kPa of undrained shear strength.		
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).		
Moisture Condition (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained Soils)	D	DRY – runs freely through fingers.		
	M	MOIST – does not run freely but no free water visible on soil surface.		
	W	WET – free water visible on soil surface.		
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
Density Index/ Relative Density (Cohesionless Soils)			**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

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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