Appendix 18

Geotech Report

JK Geotechnic

REPORT TO

CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

22-34 BERRY ROAD, 21-31 HOLDSWORTH AVENUE AND 42-46 RIVER ROAD, ST LEONARDS, NSW

Date: 28 January 2022
Ref: 33629Brpt

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Daniel Bliss
Principal | Geotechnical Engineer
NSW Fair Trading PRE No. PRE0000088

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

DOCUMENT REVISION RECORD

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development known as St Leonards South – East Quarter, located at 22-34 Berry Road, 21-31 Holdsworth Avenue and 42-46 River Road, St Leonards, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Greaton Development, on behalf of CIFI St Leonards Development Management Pty Ltd, and was carried out in accordance with our proposal dated 14 October 2021, Ref: P52861Brev2.

The proposed development is only at concept stage and detailed architectural drawings have not been provided. However, from preliminary drawings by Koichi Takada Architects, dated December 2021, the existing houses will be demolished and a multi-level residential development constructed. The development will comprise four residential towers with eight or nine above ground levels over four basement levels, with the ground and lower ground floor levels at the northern end of the site also below ground. The lowest basement is proposed at RL49.6m, which will require excavation to depths ranging from about 3m at the southern River Road end to about 20m at the northern end. The basement will be offset about 4m from the northern, eastern and western boundaries and about 10m from the southern boundary and will not occupy the central portion of the site, where deep planting will be present within a central courtyard.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, where possible prior to demolition, and to use this as a basis for providing preliminary comments and recommendations on geotechnical issues for the proposed development to assist with the DA stage of the project.

This geotechnical investigation was carried out in conjunction with an environmental preliminary (Stage 1) site investigation by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E33629BTrpt, for the results of the environmental investigation.

2 INVESTIGATION PROCEDURE

This investigation was carried out prior to demolition of the existing houses and so access for drilling was limited to the driveways of the properties. Boreholes BH1 to BH6 were drilled where access was possible for our track mounted JK205 drilling rig. The boreholes were auger drilled to depths ranging from 1.6m to 6m below the existing ground surface.

The borehole locations, as shown on Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the boreholes logs, were estimated by interpolation between spot levels and contours shown on the supplied survey plans by LTS (Ref: 42520 008DT, Sheets 1 to 3, dated 21/10/20) and SDG (Ref. 7025, Issue A, dated 14/4/16). The datum of the levels is the Australian Height Datum (AHD).

The strength of the residual silty clay was assessed from the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples recovered in the SPT split-spoon sampler.

The strength of the weathered rock was assessed by observation of resistance to penetration of a Tungsten Carbide (TC) bit, together with examination of recovered cuttings and correlations with subsequent laboratory moisture content test results. Rock strengths assessed in this manner are approximate only and variations of one strength order should not be unexpected.

Groundwater observations were made during and on completion of drilling of the boreholes. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer set out the borehole locations, nominated the sampling and testing locations and logged the subsurface conditions encountered. The borehole logs are attached, together with 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 moisture contents, Atterberg limits, linear shrinkages, pH values, chloride contents, sulphate contents and resistivity values. The results of the laboratory testing are summarised in the attached STS Table A and Envirolab Certificate of Analysis 285979. Samples were also taken from the boreholes for testing as part of the environmental investigation by JKE.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on the side of a hill that generally slopes down to the south at a maximum of about 6° and then steps down over a steep escarpment towards its southern end onto River Road. The site is bound to the east and west by Berry Road and Holdsworth Avenue, respectively, which are linked by pedestrian walkways to River Road, which bounds the site to the south. Sandstone outcrop are present within the steeper parts of the site at the southern end adjacent to River Road.

The site comprises 16 residential allotments occupied by one or two storey houses, which are generally in good to fair external condition based on a cursory inspection from the surrounding streets. Some minor cracking is present in some houses. Access to several of the properties is along driveways, which are in fair to poor condition, with cracking noted in several driveways. The remainder of the properties are grass covered, with scattered small to medium sized trees. The ground surface within the properties steps down the hillside to the south, with the changes in level supported by retaining walls generally of masonry construction. Retaining walls are also located along the frontages of some properties, particularly along Holdsworth Avenue and River Road. The retaining walls have maximum heights of about 1.5m and are in variable condition from poor condition to good condition.

To the north of the site are two residential allotments, similar to those within the subject site. These properties contain two storey houses, located about 1.5m from the common boundary. The ground surface within the adjoining properties continues to rise to the north with the hillside, with the adjoining properties being about 0.3m to 1m higher than the northern end of the subject site.

3.2 Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is mapped to be underlain by Hawkesbury Sandstone, but is close to the boundary with the overlying Ashfield Shale to the north.

In summary, the boreholes encountered fill covering residual silty clay that graded into weathered sandstone at relatively shallow depths. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered at each borehole location.

Fill

In BH4, concrete of 200mm thickness was initially encountered. Fill was encountered in all boreholes to depths ranging from 0.25m to 0.5m. The fill comprised clayey gravel, clayey sand, silty sandy clay, silty sand and silty clay with varying proportions of brick fragments, sandstone gravel, ironstone gravel and root fibres.

Natural Soils

The majority of the soils encountered comprised residual silty clay that was assessed to be of medium to high plasticity. The strength of the silty clay was generally of very stiff to hard strength, but some of the upper clays were of stiff strength. In BH4, silty clayey sand and clayey sand was encountered to a depth of 2.3m and was assessed to be of loose and then medium dense relative density. In BH6, what appeared to be a sandstone boulder was encountered within the natural soil profile between depths of 1.1m and 2m.

Weathered Sandstone

Weathered sandstone was encountered at depths ranging from 0.35m to 5m. On first contact the sandstone was variable, ranging from extremely weathered in BH3 and BH5, to medium strength in BH4 and BH6. Refusal of the TC bit attached to the augers occurred within sandstone assessed to be of medium or high strength in BH1, BH2, BH4 and BH6, at depths of 1.6m, 4.7m, 4.7m, and 5.4m, respectively.

Groundwater

In BH4 and BH6, groundwater seepage was encountered during auger drilling at depths of 4.0m and 4.5m, respectively. Groundwater was measured on completion of BH4 at the depth of 3.4m, but BH6 was dry on completion. No groundwater was encountered within the remaining boreholes during or on completion of drilling.

3.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the silty clay tested is of medium or high plasticity and is assessed to have a moderate to high potential for shrink/swell movements with changes in moisture content.

The moisture content test results on samples of the weathered sandstone showed a reasonably good correlation with our field assessment of rock strength.

The soil pH values were 7.4 for a sample of the fill indicating alkaline conditions and 4.8 and 5.0 for samples of the sandstone and natural soils indicating acidic conditions. The sulphate contents ranged from 31mg/kg to 42mg/kg, the chloride contents were <10mg/kg and the resistivity ranged from 130ohm.m to 360ohm.m. Based on these results, the soils would be classified as 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'. For steel piles, the soils would be classified as 'non-aggressive' in accordance with Table 6.5.2(C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Profile and Additional Geotechnical Investigation

This geotechnical investigation was only preliminary involving auger drilled boreholes where access was possible for drilling. Therefore, the boreholes were drilled to limiting depths and most refused at relatively shallow depths. However, we consider that sufficient geotechnical information has been obtained for assessment of the DA and to allow planning and preliminary design.

The boreholes encountered surface fill covering mainly residual soils grading into sandstone bedrock. The sandstone bedrock is at variable depths, with sandstone exposed at the southern end of the site and encountered as deep as 5m in the current boreholes. We note that the deepest sandstone was encountered in BH6 close to where sandstone is exposed, indicating that the surface of the sandstone varies considerable within short distances. We expect that the sandstone surface would comprise a series of relatively level areas separated by buried sandstone cliff lines.

The subsurface profile must be confirmed and determined to greater depths following demolition of the existing houses. The houses will need to be demolished so that full access to the site is possible for a drilling rig to drill deeper boreholes to below the base of the proposed excavation. To allow the boreholes to reach such depths coring of the sandstone will be required, which will also allow optimisation of bearing pressures for the design of footings.

As part of the detailed geotechnical investigation, groundwater monitoring wells should be installed within the boreholes. Approval for the development may be referred to WaterNSW and as such the investigation should follow the guidelines given in "Minimum Requirements for Building Site Groundwater Investigations and Reporting" DPIE, January 2021. These guideline required the installation of at least three monitoring wells, and monitoring of the groundwater levels for at least three months, but preferably six months. In addition, at least one of the boreholes will need to be drilled well below the base of the proposed excavations to confirm the subsurface profile. For the geotechnical investigation, we recommend drilling of cored boreholes to at least 3m below the base of the proposed excavation, but these guidelines required at least one boreholes to be drilled deeper. The guidelines are written for excavations with a uniform depth so it is unclear what the required borehole depth is for the variable excavation depth at this site of 3m to 20m. For a 3m deep excavation at least one borehole needs to be drilled to a depth of 10m, but for a 20m deep excavation the borehole depth would be about 35m to 40m. We recommend that if possible WaterNSW be contacted early in the planning process to determine their investigation requirements. We note that from a geotechnical perspective completion of investigations following these guidelines is not considered necessary given the geotechnical conditions at the site, but these may be imposed by WaterNSW.

The preliminary comments and recommendations contained herein should be reviewed and amplified as part of the more detailed geotechnical investigation carried out following demolition.

4.2 Excavation

Prior to the start of excavation, if not completed prior to demolition, dilapidation surveys should be carried out on the adjoining properties to the north and the south-east (No. 41 River Road). Dilapidation surveys may also be required on the surrounding roads and other council assets. The dilapidation surveys should comprise detailed inspections of the adjoining properties, both external and internal, with all defects rigorously described, i.e. defect type, defect location, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The dilapidation reports can then be used as a baseline for assessing damage that may be caused by the excavation works and will help to guard against opportunistic claims for damage that was present prior to the start of the work.

Excavation to the required depths of about 3m to 20m is expected to encounter surface fill, which may be deeper behind the existing retaining walls than encountered within the existing boreholes, residual soils and sandstone bedrock.

Excavation of the soils, and any extremely weathered sandstone, will be possible using conventional excavation equipment, such as the buckets of hydraulic excavators.

Excavation of the sandstone of low strength or higher strength will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. Sandstone of high or even very high strength is expected to be encountered and this would represent 'hard rock' excavation conditions and reduced productivity and high equipment wear should be expected.

Hydraulic rock hammers must be used with care due to the risk of damage to adjoining structures from the vibrations generated by the hammers. Excavations using rock hammers should commence away from the adjoining houses to the north and south-east and the vibrations transmitted to these houses quantitively monitored at all times during rock hammer use. Monitors should be solidly fixed to the adjoining houses, with the monitors attached to flashing warning lights, or other suitable warning devices, so the operator is aware when acceptable limits have been reached and excavation work can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive, it would be necessary to change to alternative lower vibration emitting equipment, such as ripping hooks, rotary grinders or rock saws. A rock saw could be used to cut a slot along the excavation perimeter before using a rock hammer to break out the rock from between the saw cuts in order to limit the transmitted vibrations. However, the effectiveness of this would need to be confirmed from the results of vibration monitoring.

4.3 Groundwater

Groundwater seepage was encountered in BH4 and BH6 at about the surface of the sandstone and so would represent seepage flowing across the sandstone and down the hill towards the south. We would expect that the standing groundwater table would be at depth within the sandstone, with a gradient down towards the south, but the depth cannot be determined from this investigation.

As detailed in Section 4.1, determination of groundwater levels may be required by WaterNSW as part of the detailed geotechnical investigation. However, the wells installed may collect seepage flowing down the hillside and determination of the standing groundwater level may be difficult.

Seepage should be expected into the excavation and will tend to occur along the soil/rock interface and through joints and bedding partings within the sandstone, particularly during and following rainfall. During construction such seepage should be able to be controlled using gravity drainage and conventional sump and pump techniques.

In the long term, drainage should be provided behind the basement walls and at the base of any rock cuts to control any seepage that does occur. The completed excavation should be inspected by the hydraulic consultant to confirm that the designed drainage system is adequate for the actual seepage flows.

If development is referred to WaterNSW it is likely that they will require design of a tanked basement where groundwater (either seepage flow or standing groundwater) is encountered above the base of the excavation. Given the subsurface profile of mainly sandstone bedrock adoption of a tanked basement is not considered necessary from a geotechnical perspective, but may be imposed by WaterNSW. Similarly, WaterNSW may require a detailed geotechnical investigation of the site and modelling of seepage in accordance with “Minimum Requirements for Building Site Groundwater Investigations and Reporting” DPIE, January 2021. From a geotechnical perspective these guidelines are not particularly relevant for this site given the geological profile, but WaterNSW may require them to be meet for approval of the development.

4.4 Retention

Retention of the soils and poor quality soils will be required, but good quality sandstone of low strength or higher strength with few defects would be able to be excavated unsupported. However, the depth of the sandstone is variable and the retention required may also vary. Where good quality sandstone is at shallow depths the use of temporary batters within the soils and vertical cut faces within the good quality sandstone may be feasible. However, where the depth of good quality sandstone does not allow the use of temporary batters retention system would need to be installed prior to the start of excavation.

If such variable retention systems are adopted it may be prudent to drill several boreholes along the perimeter of the excavation to assess the depth of the sandstone and where temporary batters would be possible. However, it may be more practical to construct a retention system prior to excavation for the entire perimeter of the basement to avoid difficulties at the connection between the batters and the retention system.

Where temporary batters are adopted they should be no steeper than 1 Vertical in 1 Horizontal (1V:1H) and no more than 3m in height. Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters.

Vertical excavations within the sandstone must be regularly inspected by a geotechnical engineer to check for any weak seams or inclined joints that require additional support. Such inspections should be carried out at depth intervals of no more than 1.5m and any additional support recommended by the geotechnical engineer installed prior to further excavation. The additional support may comprise rock bolts to support potentially unstable blocks, shotcrete and mesh to support weak seams or dental treatment of thin weak seams.

Suitable retention system would comprise soldier pile retaining walls with shotcrete infill panels, but a closer spacing of the piles, or even contiguous piles, may be required where sandy soils are present, such as near BH4. If soldier pile walls are adopted it would be advisable to excavate test pits where the more sandy soils are present to assess if they can stand unsupported sufficiently to allow placement of shotcrete. If the soils cannot stand unsupported then contiguous pile walls may need to be adopted.

Lateral support of the retaining walls will be required in the form of external anchors or internal props. This lateral support will also be required at the toe of the walls where they are terminated within the sandstone bedrock above the base of the excavation. Where anchors extend below adjoining properties permission will need to be obtained from the owners of the adjoining properties before installing anchors below their properties. Such permission can take some time to obtain and should be sought early to allow time for negotiation.

Anchors should have their bond formed within the sandstone bedrock and may be provisionally designed for an allowable bond stress of 250kPa for sandstone of at least low strength. Higher bond stresses are likely within the sandstone and should be assessed following the drilling of cored boreholes. All anchors should be proof loaded to at least 1.3 times their working load before locking off at about 80% of their working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to conform that the anchors are holding their load. Usually anchors are commissioned on a design and construct basis so that failure of anchors to hold their test load does not then become a contractual issue.

Preliminary design of anchored or propped walls may be based on a trapezoidal pressure distribution of magnitude 6H kPa (where H is the retained height in metres) where some resulting ground movements are tolerable and no structures are located within a horizontal distance of 2H of the wall. Where movements are to be kept low, a trapezoidal pressure distribution of 8H kPa should be used.

All surcharge loads should be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the wall.

4.5 Footings

Since sandstone will be encountered within the excavation all footings should be founded within the sandstone to provide uniform support and reduce the risk of differential settlements. Pad or strip footings are likely to be appropriate for the majority of the building as we expect sandstone will be present at the bulk excavation level.

Where any above ground portion of the building extends outside of the basement footprint it should be supported on piles founded within the sandstone bedrock. Such piles should be founded below the zone of influence of the basement excavation unless the basement retaining walls have been designed to support the surcharge loads. The zone of influence may be taken as a line drawn up at 1V:1H from the base of the excavation. Care must be taken with such piles that they are not founded on the edge of buried cliff lines within the sandstone profile. A series of closely spaced boreholes may be required where piles are required to profile the rock surface and assess where the cliff faces may be present.

Final bearing pressures appropriate for the design of the footings will need to be assessed as part of the detailed geotechnical investigation and can be optimised from the results of the cored boreholes. Allowable bearing pressures would commence at 1000kPa, but if the cored boreholes prove sandstone of medium to high strength with few defects allowable bearing pressures of 3000kPa to 5000kPa or more may be appropriate.

The footing excavations should be inspected by a geotechnical engineer to confirm that the appropriate quality sandstone has been encountered. The extent of the inspections and additional proving will depend on the bearing pressure adopted and where high bearing pressures are used spoon testing or additional cored boreholes may be required to check the defects within the rock.

5 GENERAL COMMENTS

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

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

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

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
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A

MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Report No.:**	33629B - A
Project:	Proposed Residential Development - East Quarter St Leonards	**Report Date:**	11/01/2022
Location:	22-34 Berry Road, 21-31 Holdsworth Avenue & 42-46 River Road, St Leonards, NSW	**Page 1 of 1**	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	1.20 - 1.50	4.1	-	-	-	-
2	3.50 - 4.00	4.6	-	-	-	-
3	0.50 - 0.95	20.7	52	21	31	11.5
3	3.00 - 3.50	6.2	-	-	-	-
4	4.00 - 4.50	6.3	-	-	-	-
5	0.50 - 0.95	24.9	50	19	31	11.0
5	4.00 - 4.50	7.2	-	-	-	-
6	5.00 - 5.40	4.4	-	-	-	-

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 21/12/2021.
- Sampled and supplied by client. Samples tested as received.

NATA
NATA Accredited Laboratory Number:1327

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In full without approval of the laboratory. Results relate only to
the items tested or sampled.

11/01/2022
Authorised Signature / Date
(D. Treweek)

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

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 285979

Client Details

Client	JK Geotechnics
Attention	Arthur Kourtesis
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33629B, St Leonards**
Number of Samples	3 Soil
Date samples received	22/12/2021
Date completed instructions received	22/12/2021

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	06/01/2022
Date of Issue	24/12/2021

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		285979-1	285979-2	285979-3
Your Reference	UNITS	BH1	BH3	BH4
Depth		0.05-0.2	1.5-1.75	1.5-1.95
Date Sampled		16/12/2021	16/12/2021	16/12/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	23/12/2021	23/12/2021	23/12/2021
Date analysed	-	23/12/2021	23/12/2021	23/12/2021
pH 1:5 soil:water	pH Units	7.4	4.8	5.0
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	32	31	42
Resistivity in soil*	ohm m	130	360	300

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.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			23/12/2021	[NT]	[NT]	[NT]	[NT]	23/12/2021	[NT]
Date analysed	-			23/12/2021	[NT]	[NT]	[NT]	[NT]	23/12/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	87	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	90	[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.

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.

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.

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BOREHOLE LOG

Borehole No. 1

1/1

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 59.0m

Date: 16/12/2021 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Clayey gravel, fine to medium grained, dark grey brown, trace of fine to medium grained sand.	M			PEBBLE COVER SCREEN: 9.08kg 0-0.05m
					-	FILL: Clayey sand, fine to medium grained, dark grey brown, with fine to medium grained sandstone gravel.				NO FCF
						SANDSTONE: fine to medium grained, light grey brown, with extremely weathered bands.	DW	VL		LOW 'TC' BIT RESISTANCE
			1					M		MODERATE TO HIGH RESISTANCE WITH LOW BANDS
										MODERATE TO HIGH RESISTANCE
						SANDSTONE: fine to medium grained, light grey and red brown.		H		HIGH RESISTANCE
						END OF BOREHOLE AT 1.6m				'TC' BIT REFUSAL
			2							
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
2
1/1

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 64.0m
Date: 16/12/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty sandy clay, medium plasticity, dark grey brown, fine to medium grained sand, trace of brick fragments and root fibres.	w>PL			GRASS COVER SCREEN: 10.27kg 0-0.1m NO FCF
	N = 5 1,2,3	1	CI-CH	FILL: Silty sandy clay, low plasticity, grey brown, fine to medium grained sand, trace of fine to medium grained ironstone gravel.	w>PL	St-VSt	200 190 180	RESIDUAL
				Silty CLAY: medium to high plasticity, light grey brown and orange brown, with fine to medium grained sand.				
	N = 31 10,14,17		CL-CI	Sandy CLAY: low to medium plasticity, light grey mottled brown, with fine to medium ironstone bands.		(VSt-Hd)		
		2		as above, but with extremely weathered bands.				
	N = SPT 6/50mm REFUSAL	3						
			-	SANDSTONE: fine to medium grained, light grey, with high strength iron indurated bands and extremely weathered bands.	DW	M		MODERATE 'TC' BIT RESISTANCE
		4		SANDSTONE: fine to medium grained, light grey and grey.	SW	M-H		MODERATE RESISTANCE WITH HIGH BANDS
		5		END OF BOREHOLE AT 4.7m				'TC' BIT REFUSAL
		6						
		7						

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BOREHOLE LOG

Borehole No.
3

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 65.8m
Date: 16/12/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Clayey sand, fine to medium grained, dark grey, with silt, trace of root fibres.	M			GRASS COVER SCREEN: 8.5kg 0-0.1m NO FCF
	N = 7 2,3,4		CH	Silty CLAY: high plasticity, light brown, with fine to medium grained sand, trace of root fibres.	w>PL	VSt-Hd	480 370 530	RESIDUAL
	N > 12 20,12/ 100mm REFUSAL	1	-	Extremely Weathered sandstone: silty sandy CLAY, medium plasticity, light grey and light brown, with high strength iron indurated bands.	XW	Hd		VERY LOW 'TC' BIT RESISTANCE
		2		SANDSTONE: fine to medium grained, light grey and light brown, with high strength iron indurated bands.	DW	L-M		MODERATE RESISTANCE
		3		SANDSTONE: fine to medium grained, light grey.				
		4						
		5				M		MODERATE TO HIGH RESISTANCE
						H		
		6		END OF BOREHOLE AT 6.0m				
		7						

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BOREHOLE LOG

Borehole No. 4

1/1

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 65.0m

Date: 16/12/2021 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			CONCRETE: 200mm.t				6mm DIA. REINFORCEMENT, 105mm TOP COVER
					-	FILL: Silty clay, medium plasticity, dark grey brown, with fine to medium grained sand, trace of fine to medium grained sandstone gravel.	w>PL			INSUFFICIENT RETURN FOR BULK SCREEN
		N = 9 3,4,5			SC	Silty clayey SAND: fine to medium grained, grey brown.	M	L		RESIDUAL
			1			as above, but light grey brown, trace of fine to medium grained ironstone gravel.				
		N = 18 5,7,11				Clayey SAND: fine to medium grained, light grey and orange brown, trace of ash.		MD		
ON COMPLET-ION			2 / 3		CI-CH	Silty CLAY: medium to high plasticity, light grey, with fine to medium grained ironstone gravel, fine to medium grained sand, and extremely weathered sandstone bands.	w<PL	(Hd)		
			4		-	SANDSTONE: fine to medium grained, light grey and light brown, with iron indurated bands.	DW	M		MODERATE 'TC' BIT RESISTANCE HIGH RESISTANCE 'TC' BIT REFUSAL
			5 / 6 / 7			END OF BOREHOLE AT 4.7m				

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BOREHOLE LOG

Borehole No.
5
1/1

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 68.2m
Date: 16/12/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty sandy clay, low plasticity, dark grey brown, trace of brick fragments and root fibres.	w>PL			GRASS COVER SCREEN: 11.05kg 0-0.1m NO FCF
	N = 4 1,2,2		CI	Silty CLAY: medium plasticity, light grey brown and orange brown, with fine to medium grained sand, trace of fine to medium grained ironstone gravel.	w>PL	St-VSt	100 125 300	RESIDUAL
		1	CL-CI	Sandy CLAY: low to medium plasticity, light grey, orange brown and red brown, fine to medium grained sand, trace of silt.				
	N = 24 4,9,15		CI-CH	Silty CLAY: medium to high plasticity, light grey, with fine grained sand and extremely weathered sandstone bands.	w<PL	Hd	550 600 >600	
		2	-	Extremely Weathered sandstone: Sandy silty CLAY, medium plasticity, light grey, fine to medium grained sand.	XW	Hd		VERY LOW 'TC' BIT RESISTANCE
		3			DW	L		LOW TO MODERATE RESISTANCE
		4		SANDSTONE: fine to medium grained, light grey and light brown, with iron indurated bands.				
		5						
		6		END OF BOREHOLE AT 6.0m				
		7						

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BOREHOLE LOG

Borehole No.
6

Client: CIFI ST LEONARDS DEVELOPMENT MANAGEMENT PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22-34 BERRY RD, 21-31 HOLDSWORTH AVE & 42-46 RIVER RD, ST LEONARDS, NSW

Job No.: 33629B **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 51.8m
Date: 16/12/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.F./D.B.

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 COMPLET-ION			0			FILL: Silty sandy clay, low to medium plasticity, grey brown, fine to medium grained sand, trace of root fibres, and fine to medium grained sandstone gravel.	w≈PL M			GRASS COVER SCREEN: 9.4kg 0-0.1m NO FCF
		N = 9 3,3,6			CL-CI	FILL: Silty sand, fine to medium grained, light brown, with sandstone gravel.	w≈PL	(VSt-Hd)		COLLUVIAL
			1		-	Silty sandy CLAY: low to medium plasticity, grey mottled brown, fine to medium grained sand, trace of ash, root fibres and fine to medium ironstone gravel. SANDSTONE BOULDER: fine to medium grained, light grey brown.	DW	L		MODERATE 'TC' BIT RESISTANCE
			2		CI	Silty CLAY: medium plasticity, light grey, with fine to medium grained sand.	w≈PL	(VSt-Hd)		SOIL RESISTANCE RESIDUAL
		N = 11 3,4,7	3		SP	SAND: fine to medium grained light brown, trace of silt.	M	L		
					CL	Sandy CLAY: low plasticity, light grey mottled light brown, fine to medium grained sand, trace of silt.	w>PL	St-VSt	220 170 270	
			4		CI	Silty CLAY: medium plasticity, grey and orange brown, trace of fine to medium grained ironstone gravel.		Hd		
▶		N > 29 14,15/ 100mm REFUSAL							560 400 590	
			5		-	SANDSTONE: fine to medium grained, light grey brown.	SW	M-H		HIGH RESISTANCE
						END OF BOREHOLE AT 5.4m				'TC' BIT REFUSAL
			6							
			7							

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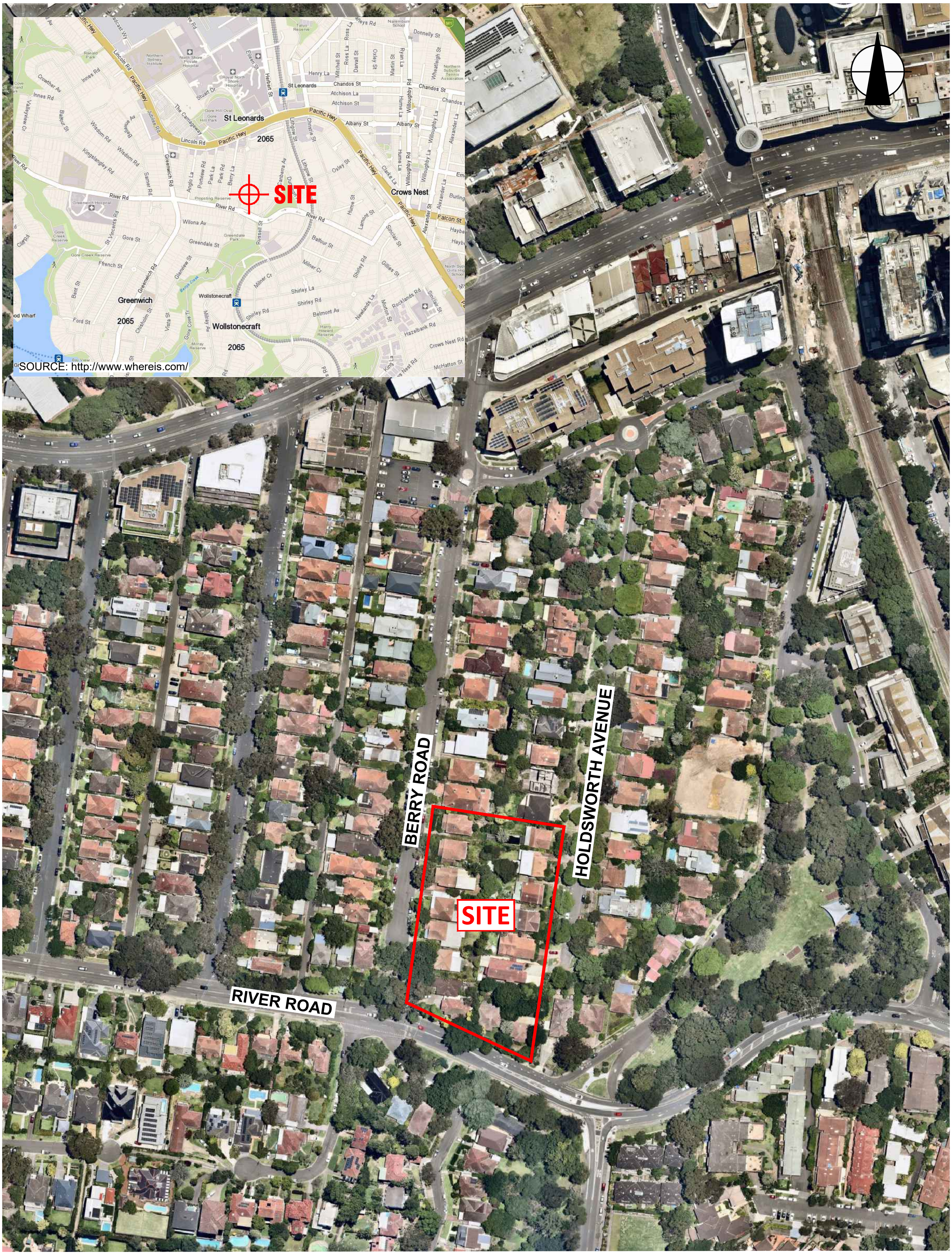

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title: **SITE LOCATION PLAN**

Location: 22-34 BERRY ROAD, 21-31 HOLDSWORTH AVENUE & 42-46 RIVER ROAD, ST LEONARDS, NSW

Report No: 33629B

Figure No: 1

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PLOT DATE: 19/01/2022 2:00:46 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33629B ST LEONARDS\CAD\33629B.DWG

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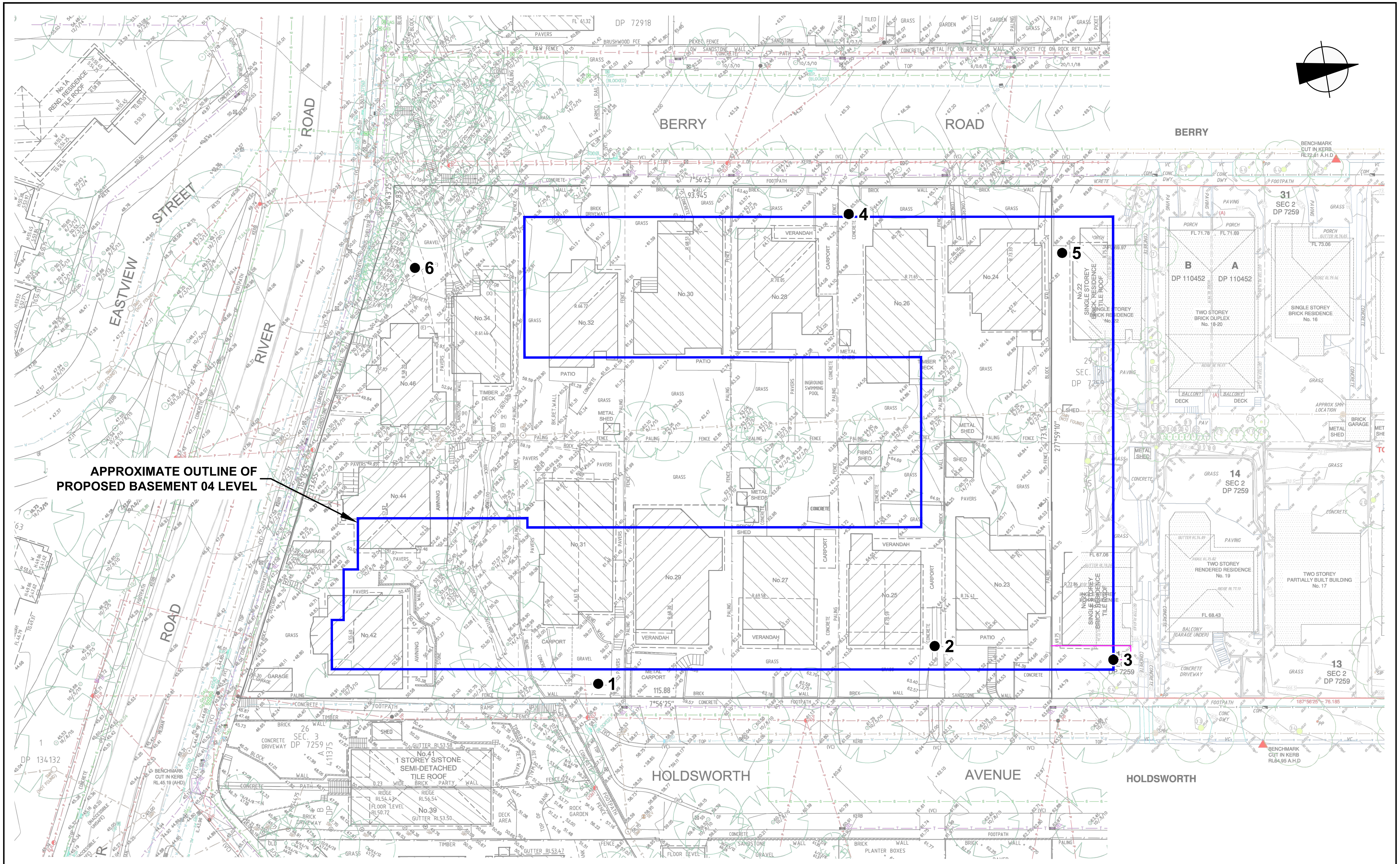

0 5 10 15 20 25

SCALE 1:500 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: 22-34 BERRY ROAD, 21-31 HOLDSWORTH AVENUE & 42-46 RIVER ROAD, ST LEONARDS, NSW

Report No: 33629B

Figure No: 2

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PLOT DATE: 19/01/2022 2:01:01 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33629B ST LEONARDS\CAD\33629B.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:

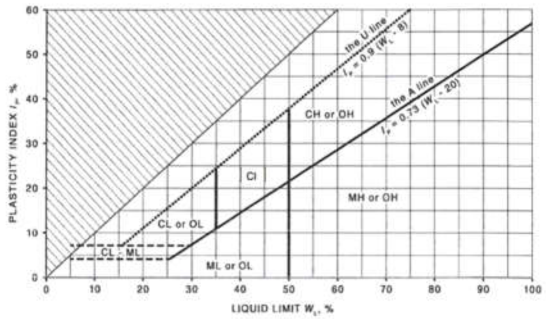
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**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

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**Point Load Strength Index $Is_{(50)}$ (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 $\leq$ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres