REPORT TO

AVEO GROUP

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS (DINING AREA AND LIFT)

AT

BAYVIEW GARDENS RETIREMENT LIVING, 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Date: 16 October 2024

Ref: 37080Yrpt1

JKGeotechnics

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DOCUMENT REVISION RECORD

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations and additions (dining area and lift) at Bayview Gardens Retirement Village, 36-42 Cabbage Tree Road, Bayview, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Jessica Gleeson by signed 'Acceptance of Proposal' form dated 26 September 2024 and was carried out on the basis of our fee proposal, (Ref: P70180YF, dated 13 September 2024).

Based on the provided architectural drawings prepared by Bokor Architecture (Project No. 22043, Relevant Drawing Nos. DA-000, DA-011, DA-100, DA-101 to DA-103, DA-200, DA-201, DA-400 and DA-401, all Revision 2, dated 24 July 2024), we understand the proposed alterations and additions include the following:

- Demolition of various internal elements and the existing outdoor dining area.
- Construction of an outdoor dining area which will replace the existing tiled area and will comprise a new concrete slab and single storey awning. The proposed dining area will have a floor level at about RL20.4m and will therefore be constructed at, or close to, existing ground levels.
- Construction of a new lift adjacent to the northern side of the existing building. The depth of excavation at this stage is not known, although we anticipate about 1.5m of excavation will be required.
- New paved areas to the south of the existing structure which have a limited footprint and will be constructed at, or close to, existing ground levels.
- Interior alterations and additions.

Based on Council's Geotechnical Hazard Mapping the site of the proposed alterations and additions is located within the Council Geotechnical Hazard Zone H3, and therefore, a geotechnical slope stability assessment is not required.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions. Based on this we have provided comments and recommendations on excavation, underpinning and retention, footings and slabs on grade.

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

Due to the limited site access, the investigation was carried out using hand operated equipment. The fieldwork was carried out on 2 October 2024 and comprised the following:

- Three hand auger boreholes, BH1, BH3 and BH4, were drilled to refusal depths of 0.65m, 0.8m and 0.65m below existing surface levels, respectively. The purpose of the boreholes was to identify the materials present.
- Four Dynamic Cone Penetration tests, DCP1 to DCP4, were completed adjacent to the borehole locations (DCP1, DCP3 and DCP4) and at one additional location (DCP2). The DCP tests extended to refusal depths ranging from 0.65m to 1.35m below existing ground levels. The purpose of the DCP tests was to provide an assessment of the apparent compaction of the fill and the relative density of the natural soils and attempt to probe to the surface of the underlying bedrock. The refusal depth of the DCP tests may provide an indicative depth to rock, though we note that premature refusal can also occur on obstructions in fill, 'floaters' and other hard layers.

The test locations, as shown on the attached Figure 2, were set out by tape measurements from existing surface features. The approximate surface levels at the test locations have been interpolated from spot heights shown on the survey plan prepared by CMS Surveyors (Ref: 15880Edetail, Sheet 3, Issue 7, dated 7 June 2024). The height datum is Australian Height Datum.

Groundwater observations were made in the boreholes during and on completion of drilling. No longer term groundwater monitoring was completed.

The fieldwork was completed in the full-time presence of our senior geotechnical engineer, Mr Ben Sheppard, who set out the investigation locations, nominated the sampling and testing, prepared the borehole logs and recorded the DCP test results. The borehole logs and DCP test results are attached with this report, together with a set of explanatory notes which define the logging terms and symbols used and provide further details of the investigation techniques adopted and their limitations.

Selected soil samples were returned to NATA accredited laboratory (Envirolab Services Pty Ltd) for soil pH, chloride content, sulphate content and resistivity testing. The test results are summarised in the attached Envirolab Services Certificate of Analysis 363220.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located about mid-slope on a south and south-west sloping hillside, which slopes down from an east-west orientated spur to a low-lying alluvial gully associated with Pittwater Bay. The site is located within Bayview Gardens Retirement Village, which generally comprises several one to three storey structures, internal Asphaltic Concrete (AC) paved roads, pathways, gardens and lawns.

The site comprises areas to both the north and south of the existing two-storey brick building. To the north of the building where the new outdoor area and lift are proposed levels step down from west to east, with the proposed outdoor area located over the higher level and the lift at the lower level. The change in height is facilitated by a brick retaining wall which extends perpendicular to the structure, appears to be in good condition and ranges in height between about 3m adjacent to the structure to about 1m near the Annan Road frontage. The upper level contains an outdoor tile paved dining area with perimeter lighting and gardens and is generally level. Some low-height sandstone cobble retaining walls were present within this area. The lower level is also relatively level and contains an artificial turf area and pathways. A 0.6m high brick retaining wall is situated near the toe of the brick retaining wall adjacent to the artificial turf area, and generally appears to be in good condition. Some small to medium sized boulders were observed at the surface scattered across the site, although these appear to be placed for landscaping purposes.

On the southern side of the building, where new paving is proposed is an existing paved area that extends out to the existing asphaltic concrete access road and carpark. This area slopes gently down to the south-east.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by the Newport Formation of the 'Narrabeen Group', which comprises interbedded laminite, shale and quartz to lithic quartz sandstone. The upper portions of the ridgeline to the north of the site are mapped to comprise Hawksbury Sandstone.

The boreholes generally encountered fill overlying inferred weathered bedrock. The fill was highly variable and comprised silty sand, sandy silty clay, sandy clayey silt, sandy clay and clayey sand, and based on the DCP test results was assessed to be poorly compacted. Inclusions within the fill comprised sandstone and ironstone gravel and sandstone cobbles.

The DCP tests refused at depths of 0.65m, 1.35m, 0.97m and 0.7m below existing ground levels at DCP1, DCP2, DCP3 and DCP4, respectively. The refusal depths of the DCP tests have been inferred to have occurred on the surface of the underlying weathered bedrock. However, since these tests do not provide sample recovery this could not be confirmed and it is possible that refusal may have occurred on obstructions within the fill or other hard layers.

No groundwater was encountered within the boreholes during or on completion of drilling.

Reference should be made to the boreholes logs and DCP test results for detailed descriptions of the subsurface conditions encountered.

3.3 Laboratory Test Results

The results of the soil aggression testing are tabulated below:

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides CL (ppm)	Resistivity (ohm.cm)
BH1	0.4 – 0.6	Silty sandy Clay	6.7	10	<10	150,000
BH4	0.2 – 0.4	Clayey SAND	7.6	20	<10	91,000

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Prior to any excavation commencing we recommend that reference be made to the latest version of the WorkCover Authority of NSW's Code of Practice – Excavation Work.

Excavation for the proposed lift is anticipated to depths of about 1.5m, although this will need to be confirmed. Based on BH4/DCP4, excavation to these depths will likely encounter clayey sand and sandy clay fill and then weathered bedrock over the lower portion of the excavation.

Due to the limited site access, we do not expect that an excavator will be able to be used and all excavation may need to be carried out using hand tools, including hand-held jackhammers for removal of any bedrock. We consider that excavation using hand-held tools, such as hand-held jackhammers, are unlikely to produce damaging vibrations during bedrock excavation and therefore vibration monitoring is not required. However, if concerns are raised regarding potentially damaging vibrations caused by rock excavation, then some initial vibration monitoring may be carried out to assess whether vibration levels fall within acceptable limits.

If access is possible for a small excavator and excavation using a hydraulic rock hammer is proposed, continuous vibration monitoring will be required as there is a risk that transmitted vibrations may damage the existing structures on site. On the assumption that any vibration generated damage to the existing buildings adjacent to the works will be repaired as part of the works a dilapidation report is not required for the existing building adjacent to the lift. However, should this not be the case, a dilapidation report should also be completed for the existing building. The purpose of dilapidation reports is to provide a baseline condition survey of the structures. In this way the builder is protected from spurious claims relating to pre-existing damage. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation.

Where transmitted vibrations exceed prescribed limits, excavation techniques must be altered to reduced transmitted vibrations to within acceptable limits. This may mean that the size of percussive equipment used may need to be reduced, or non-percussive techniques adopted. Whether reducing the size of the percussive equipment is effective in controlling transmitted vibrations must be confirmed by quantitative vibration monitoring.

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

A waste classification will need to be assigned to any excavated material that is to be disposed of offsite. This needs to be completed prior to offsite disposal. We can provide the appropriate testing if required.

No groundwater was encountered in the boreholes during hand auger drilling. Notwithstanding this, we expect that some groundwater seepage may occur at the fill/natural soil and soil/bedrock interface and through defects present within the rock mass during or immediately following periods of wet weather. We consider that any seepage will be able to be controlled by gravity discharge and/or sump and pump techniques.

4.2 Existing Footings and Retention

The proposed lift excavation will extend adjacent to and possibly below footings of the existing structure. Therefore, it must be carried out with care to ensure that the existing footings are not undermined or rendered unstable.

The depth of embedment and materials on which the impacted footings are founded will need to be confirmed to assess if underpinning of the footings is required. Investigation of the footings could be carried out prior to construction by the excavation of test pits to expose the existing footing. Alternatively, the footing details could be confirmed in the early stages of construction.

Excavation should be initially carried out until the top of the existing footings are exposed. Test pits should then be excavated at locations advised by the geotechnical engineer to expose the base of the footings and the foundation material. Once exposed, the test pits should be inspected by the geotechnical engineer and structural engineer to determine whether underpinning of the footings is required. If the existing footings are founded below the base of the proposed excavations or are founded on good quality bedrock, underpinning of the footings would not be required. Where supported on good quality bedrock, any unsupported cut below the footing must be inspected by the geotechnical engineer, and stabilisation measured implemented as directed should they be required. However, if the footings are founded on soils or poor-quality sandstone above the base of the proposed excavation, they will need to be underpinned prior to bulk excavation. If underpinning is required, a detailed underpinning methodology must be developed by the structural engineer and reviewed by the geotechnical engineer prior to commencing such works. Any underpinning should be carried out by the excavation of discrete sections with each section fully underpinned prior to excavation of the adjacent sections. Regular geotechnical and structural inspections would be required during the underpinning works.

The remaining sides of the lift excavation may be temporarily battered with long term support provided by the lift pit walls. Temporary batters through the granular fill may be formed at no steeper than 1 Vertical(V):1.5 Horizontal(H) and at 1V:1H through the cohesive soils or poor-quality bedrock. Where temporary batters are formed, we recommend that a geotechnical engineer inspect the batters during excavation. Surcharge loads, including construction loads, vehicles, stockpiles, etc. must be kept well clear of the crest of temporary batters (at least 2H from the crest, where H is the vertical height of the batter slope in metres). All loose sandstone boulders should be removed from the crest of the batters.

4.3 Retention Design Parameters

The characteristic earth pressure coefficients and subsoil parameters provided below may be adopted for the design of the lift pit walls:

- We recommend that the lift pit walls be designed using a triangular lateral earth pressure distribution using at an 'at rest' coefficient of lateral earth pressure, K_o, of 0.6, assuming a horizontal backfill surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile.
- The above lateral pressures assume horizontal backfill surfaces and where inclined backfill is proposed, the pressures will need to be increased or the inclined backfill taken as a surcharge load.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design. Appropriate hydrostatic pressures should be allowed for in the design.

4.4 Footing Design

Weathered bedrock was inferred at shallow depth over the site and as such, all footings should be uniformly founded on bedrock. Footings founded on bedrock of at least very low strength may be designed based on an allowable bearing pressure (ABP) of 600kPa. Higher bearing pressures may be possible within the weathered bedrock; however, this would require proving of the bedrock with cored boreholes. A movement joint should be incorporated in the design between the new and existing buildings.

We recommend that prior to pouring concrete, all footings be inspected by a geotechnical engineer to confirm that appropriate bedrock has been encountered. All footings should be free from all loose or softened materials prior to pouring.

For the proposed dining area, all footings must be founded below the zone of influence of the brick retaining wall, taken as a line drawn up from the base of the wall at 1V:1 H. This is to ensure no additional surcharge loads are imposed onto the wall.

4.5 Slab On-Grade/Pavements

We understand that new floor slabs will be constructed for the proposed outdoor dining area and external paved area. Based on the boreholes, the area is underlain by fill overlying sandstone bedrock. We are unaware of any records of placement or compaction control of the fill and as such it must be considered 'uncontrolled'. Due to this and the poorly compacted nature of the fill, it is not considered desirable to support floor slabs on this fill due to the risk of differential settlements. Excavation and replacement of the fill with controlled, engineered fill is not considered practical given the limited size of the site and the risk of undermining adjacent footings. Therefore, our preferred option is to design the proposed floor slab as a fully suspended slab supported on footings founded on the bedrock.

Where it is decided not to fully suspend the new pavements it must be recognised that the pavements may be adversely impacted by differential settlement of the existing fill. However, given the limited depth of fill that will remain and the typically good condition of the existing tiled outdoor area and pavements, consideration could be given the casting the floor slab and external paved area on the fill, provided the risk of future settlement and damage to the pavements is accepted by the site owner. Any damage that does occur will be difficult to repair as removal and replacement of the slab is unlikely to be practical. The risk of settlement cannot be quantified due to the unknown condition of the fill as a whole, but the condition of the existing floor slab/pavements provides some guidance on their past performance (the condition of the existing floor slab/pavements can be confirmed during demolition). The risk may be able to be reduced by compaction of the upper fill following bulk excavation. However, this will have only a minor effect as only a static roller will be able to be used due to the close proximity of the adjoining buildings and the size of the roller and the restricted access. If a slab on-grade is considered, the following subgrade preparation should be carried out.

- Following demolition of the existing slab, strip any root affected soils and deleterious fill.
- Following stripping, proof roll the exposed subgrade with the heaviest roller that can access the site. Proof rolling should comprise at least 8 passes of a smooth drum roller. The purpose of the proof rolling is to detect any soft or heaving areas. The final pass of the proof rolling should be carried out without vibration and within the presence of a geotechnical engineer to detect any weak or unstable subgrade areas.
- Care must be taken when operating the roller close to existing buildings. It may be necessary to operate the roller without vibration close to existing buildings.
- Any unsuitable subgrade areas should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during the proof rolling inspection.

4.6 Soil Aggression

The above results indicate that the fill would have an exposure classification of ‘Non-aggressive’ when assessed in accordance with the criteria for concrete piling exposure classification given in Table 6.4.2 (C) and ‘Non-aggressive’ for steel structures in contact with the ground in accordance with Table 6.5.2 (C) of AS2159-2009 “Piling Design and Installation”.

4.7 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:

- Inspection of excavated test pits to confirm the dimensions and founding conditions of the existing footings.
- Inspection of temporary batters.
- Inspection of any vertical unsupported cuts formed through good quality sandstone.
- Inspection of footing excavations.
- Inspection of proof rolling where slabs on grade or pavements are adopted.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phases of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the 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.

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

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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CERTIFICATE OF ANALYSIS 363220

Client Details

Client	JK Geotechnics
Attention	Ben Sheppard
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**37080Y Bayview**
Number of Samples	2 Soil
Date samples received	03/10/2024
Date completed instructions received	03/10/2024

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	11/10/2024
Date of Issue	10/10/2024

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Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		363220-1	363220-2
Your Reference	UNITS	BH1	BH4
Depth		0.4-0.6	0.2-0.4
Date Sampled		02/10/2024	02/10/2024
Type of sample		Soil	Soil
Date prepared	-	03/10/2024	03/10/2024
Date analysed	-	08/10/2024	08/10/2024
pH 1:5 soil:water	pH Units	6.7	7.6
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	10	20
Resistivity in soil*	ohm m	150	91

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. 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	363220-2
Date prepared	-			03/10/2024	[NT]	[NT]	[NT]	[NT]	03/10/2024	03/10/2024
Date analysed	-			08/10/2024	[NT]	[NT]	[NT]	[NT]	08/10/2024	08/10/2024
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	105	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	109	[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.

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.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1/1
OUTDOOR DINING AREA

Client: AVEO GROUP
Project: PROPOSED ALTERAIONS AND ADDITIONS
Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.: 37080Y **Method:** HAND AUGER **R.L. Surface:** ≈ 20.4m
Date: 2/10/24 **Datum:** AHD
Plant Type: - **Logged/Checked by:** B.S./ W.T.

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					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, trace of sandstone cobbles, clay nodules and root fibres.	M			APPEARS POORLY COMPACTED
						0.5			FILL: Sandy silty clay, low to medium plasticity, dark brown, fine grained sand, trace of fine to medium grained sandstone gravel, and root fibres.				
									END OF BOREHOLE AT 0.65m				HAND AUGER REFUSAL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

JKGeotechnics

BOREHOLE LOG

Borehole No. 3

1/1

OUTDOOR DINING AREA

Client: AVEO GROUP

Project: PROPOSED ALTERAIONS AND ADDITIONS

Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.: 37080Y **Method:** HAND AUGER **R.L. Surface:** ≈ 20.3m

Date: 2/10/24 **Datum:** AHD

Plant Type: - **Logged/Checked by:** B.S./ W.T.

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	0			FILL: Sandy clayey silt topsoil, low plasticity, dark brown, fine grained sand, with root fibres, trace of organics.	w≈PL			APPEARS POORLY COMPACTED
						0.5			FILL: Sandy clay, low plasticity, brown and dark brown, fine to medium grained sand, trace of fine to medium grained sandstone and ironstone gravel.			140 150 150 100	HP ON CUTTINGS RECOVERED FROM HAND AUGER
													APPEARS MODERATELY COMPACTED
						1			END OF BOREHOLE AT 0.8m				HAND AUGER REFUSAL
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No.
4
1/1

OUTDOOR DINING AREA

Client: AVEO GROUP
Project: PROPOSED ALTERAIONS AND ADDITIONS
Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.: 37080Y **Method:** HAND AUGER **R.L. Surface:** ≈ 17.5m
Date: 2/10/24 **Datum:** AHD
Plant Type: - **Logged/Checked by:** B.S./W.T.

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	0			FILL: Clayey sand topsoil, fine to medium grained, dark brown, with root fibres, trace of fine grained sandstone gravel.	M			APPEARS POORLY COMPACTED
									FILL: Sandy clay, medium plasticity, dark brown, fine to medium grained sand, trace of fine to medium grained ironstone gravel.	w>PL			
						0.5							
									END OF BOREHOLE AT 0.65m				HAND AUGER REFUSAL 2 ATTEMPTS WERE MADE TO ADVANCE THE BOREHOLE, HOWEVER REFUSAL OCCURRED AT 0.65m
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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DYNAMIC CONE PENETRATION TEST RESULTS

OUTDOOR DINING AREA

Client:	AVEO GROUP
Project:	PROPOSED ALTERATIONS AND ADDITIONS
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.	37080Y	Hammer Weight & Drop: 9kg/510mm
Date:	2-10-24	Rod Diameter: 16mm
Tested By:	B.S.	Point Diameter: 20mm

Test Location	**1**	**2**	**3**	**4**			
Surface RL	≈20.4m	≈20.3m	≈20.3m	≈17.5m			
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	1	1	1	SUNK			
100 - 200	1	2	2	↓			
200 - 300	2	2	1	1			
300 - 400	2	2	2	1			
400 - 500	3	3	3	1			
500 - 600	2	3	4	6			
600 - 700	7/50mm	9	4	20/100mm			
700 - 800	REFUSAL	6	7	REFUSAL			
800 - 900		9	9				
900 - 1000		9	12/70mm				
1000 - 1100		9	REFUSAL				
1100 - 1200		10					
1200 - 1300		21					
1300 - 1400		15/50mm					
1400 - 1500		REFUSAL					
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
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
3. Datum of levels is AHD

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No: 37080YF1	Figure No: 1

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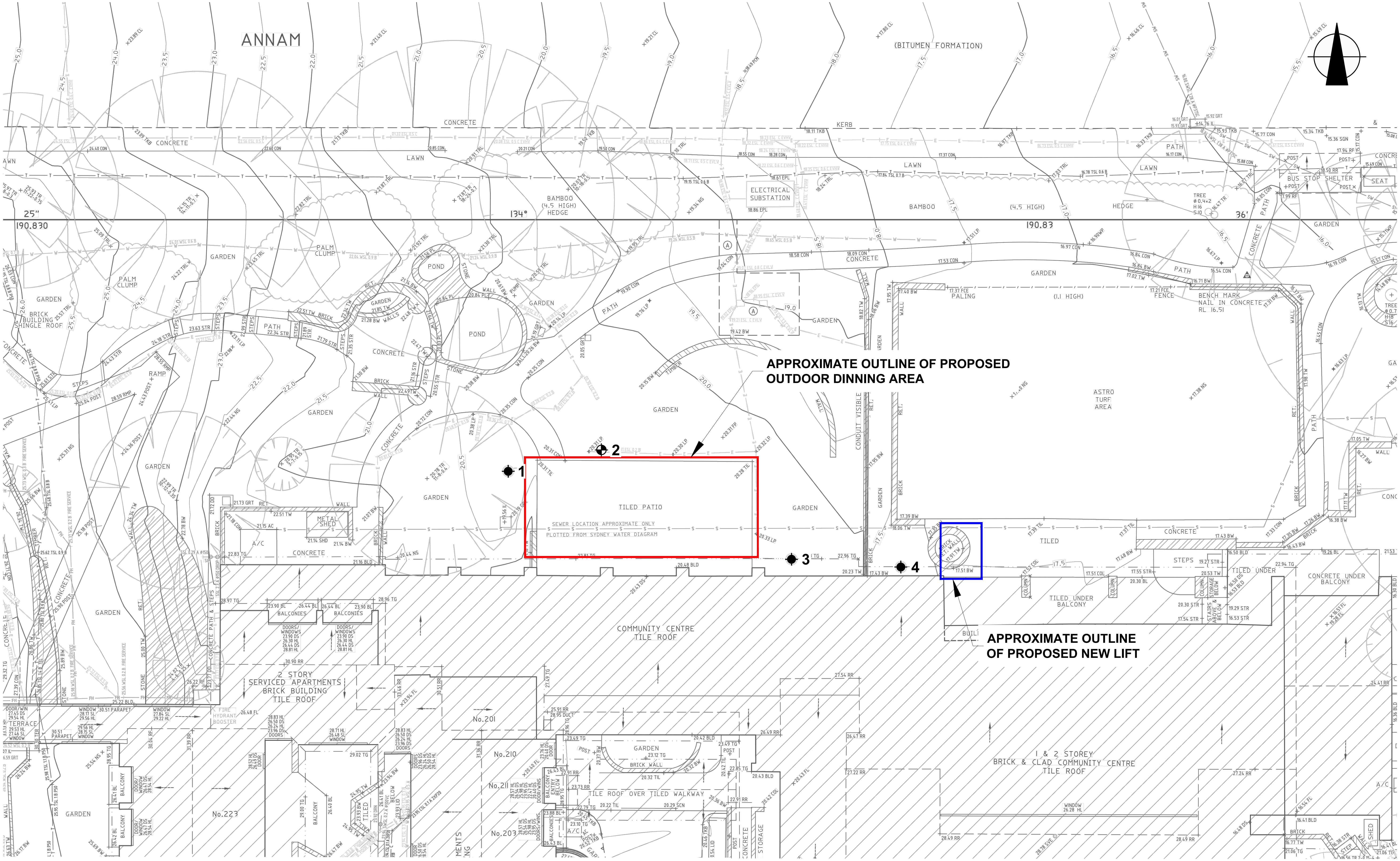
PLOT DATE: 4/10/2024 2:33:58 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF1.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST

0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

Title: **INVESTIGATION LOCATION PLAN**

Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Report No: 37080YF1

Figure No: 2

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PLOT DATE: 4/10/2024 2:34:08 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF1.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

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

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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:

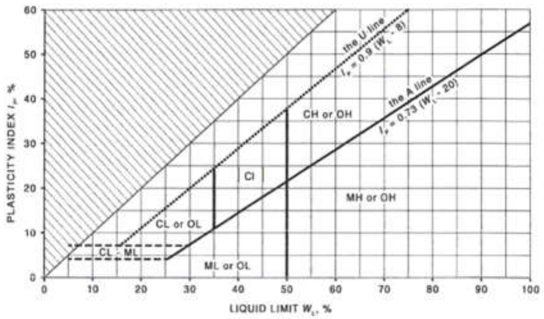
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



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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 $I_{s(50)}$ (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