REPORT

TO

WARREN AND MAHONEY ARCHITECTS LTD

ON

ADDITIONAL GEOTECHNICAL INVESTIGATION

FOR

PROPOSED REDEVELOPMENT

AT

MONA VALE SURF LIFE SAVING CLUB,
SURFVIEW ROAD, MONA VALE NSW

14 November 2018
Ref: 28092Rrpt2

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5003
www.jkgeotechnics.com.au

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

Date: 14 November 2018
Report No: 28092Rrpt2
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Report prepared by:

Paul Roberts

Paul Roberts
Principal Associate I Engineering Geologist

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

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1 INTRODUCTION

This report presents the results of an additional geotechnical investigation for the proposed redevelopment of Mona Vale Surf Life Saving Club (SLSC), Surfview Road, Mona Vale, NSW. A site location plan is presented as Figure 1.

We note that we prepared a preliminary geotechnical report on behalf of Pittwater Council (Ref. 28092ZRrpt) dated 13 February 2015, comprising two boreholes; one close to the existing building and one at an alternative location to be considered for the redevelopment. The boreholes for the recent additional investigation were drilled by our specialist division Environmental Investigation Services (EIS) for a preliminary acid sulfate soil assessment. Based on our email proposal dated 8 November 2018, Mr Andrew Walsh (Warren and Mahoney Architects Ltd [WMAL]) commissioned this updated report, which supersedes our previous report.

We have been provided with the following information:

- Survey plan (Plan No. 171678_A, dated 25 August 2018) prepared by Total Surveying Solutions.
- Architectural drawings (Drawing Numbers A.DA.02.004, A.DA.10.001, A.DA.10.002, A.DA.11.001, A.DA.20.001, A.DA.20.002 and A.DA.30.001 Rev. A, dated 15 October 2018) prepared by WMAL.
- Structural concept design report (Ref. No. 171328 Rev. A, dated 30 August 2018) prepared by Taylor Thomson Whitting (NSW) Pty Ltd (TTW).
- Pre DA meeting minutes (Application No. PLM2018/0177, dated 23 August 2018) prepared by Northern Beaches Council.
- A coastal engineering report (Ref. lrJ0183-Mona Vale SLSC-v3, dated 31 October 2018) prepared by Horton Coastal Engineering Pty Ltd (HCEPL).

Based on a review of the provided information, following demolition, a new two storey SLSC will be constructed essentially at the location of the existing SLSC building. Concrete paved footpaths and landscaping will be provided around the new building. The proposed ground floor level will be similar to the current surface levels.

The seaward margin of the site is essentially coincident with the '*Acceptable risk on conventional foundations*' for a 100 year design life. TTW indicated that conventional foundations included "*shallow strip footings, isolated pad footings and raft slabs.*" Based on the content of our previous report, and with regard to the depth to the medium dense sands, to avoid temporary shoring of pad footing excavations (which could be a maximum of 1.5m depth) TTW proposed an alternative footing solution using reinforced concrete piers. In addition, TTW have indicated a suspended ground floor slab (either post tensioned or pre-cast 'hollowcore' slab). HCEPL also stated that "*the building foundations can be designed without allowance for any undermining due to coastal erosion (that is, based on conventional structural and geotechnical considerations).*"

The purpose of our commission, was to review the boreholes completed for the preliminary acid sulfate soil assessment and our previous investigation to obtain geotechnical information on subsurface conditions as a basis for providing comments and recommendations on footing design, any other pertinent geotechnical issues or considerations and the scope of further geotechnical input.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigations was carried out on 30 January 2015 (BH1) and 25 October 2018 (BH101 and BH102) and comprised the auger drilling of the boreholes to 18m depth (BH1) and to 6.0m depth (BH101 and BH102) using our track mounted JK300 and JK305 drilling rigs.

Prior to the commencement of the fieldwork, a specialist sub-consultant electro-magnetically scanned the borehole locations for buried services.

The borehole locations, as shown on Figure 2, were set out by tape measurements from existing surface features. Figure 2 is based on aerial imagery sourced from '*Nearmap*'. The approximate surface reduced levels (RLs) shown on the borehole logs were interpreted between spot levels and contours indicated on the provided survey plan. The survey datum is the Australian Height Datum (AHD).

The relative density and strength of the natural sands and clays were assessed from the Standard Penetration Test (SPT) 'N' values, which were augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube.

Groundwater observations were made in the boreholes during and on completion of auger drilling. We note that water levels may not have stabilised in the short time period after drilling and may also be affected by tidal variations. No longer term groundwater monitoring was carried out.

For more details on the investigation procedures, reference should be made to the attached Report Explanation Notes.

The fieldwork was carried out under the full time direction of our geotechnical engineer (David Fisher) and our environmental scientist (Alistair Mitchell), who set out the borehole locations, directed the buried services scans, logged the encountered subsurface profile and nominated in-situ testing and sampling. The borehole logs (which also include field test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected soil samples were submitted to a NATA registered analytical laboratory (EnviroLab Services Pty Ltd) for soil pH, chloride and sulfate content and resistivity testing. The test results are presented in the attached 'Certificate of Analysis'.

A Preliminary Acid Sulfate Soil Assessment, Environmental Screen (Ref: E28092Plet, dated 5 November 2018) has been prepared by Environmental Investigation Services (EIS), our specialist environmental division, and should be read in conjunction with this report.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on a section of the gently sloping South Pacific Ocean foreshore immediately to the west (landward) of Mona Vale Beach and has a western frontage onto Surfview Road (which for descriptive purposes has been assumed to be orientated north-south).

The existing two storey brick SLSC building with surrounding concrete paved pathways was located over the southern end of the site. The asphaltic concrete (AC) paved Surfview Road lined the western side of the building and extend north and south beyond the existing building. An AC paved car park lined the eastern side of the northern portion of Surfview Road. A grass covered reserve area lined the western side of Surfview Road and a partially vegetated reserve area (including large trees) extended to the east of the car park to the sand dunes lining the landward side of the beach.

A brick amenities building was situated adjacent to the northern end of the car park.

Based on a cursory inspection from within the site, the existing SLSC building, toilet building and paved areas generally appeared to be in good condition. However, the concrete paved pathways adjacent to the SLSC were locally in poor condition.

3.2 Subsurface Conditions

Reference to the 1:100,000 geological map of Sydney indicates that the study area is underlain by Quaternary age alluvial and estuarine sands, silts and clays and marine sands overlying the interbedded shale, laminite and sandstone of the Triassic age Newport Formation. The investigation disclosed a subsurface profile comprising a limited thickness of fill overlying marine sands and clays. Weathered bedrock was not encountered over the depth of the investigations. Groundwater was noted at moderate depth. Reference should be made to the attached borehole logs for specific details at each location. A summary of the pertinent subsurface characteristics is presented below:

Fill

Fill comprising silty clay assessed to be of low to medium plasticity was encountered from surface level in BH101 and BH102 and extended to respective depths of 0.2m and 0.1m.

Marine Sands

Marine sands (becoming clayey with depth) were encountered in all the boreholes either from surface level (BH1) or beneath the fill (BH101 and BH102) and extend to 9.6m depth (BH1) and to the termination depth of BH101 and BH102 at 6.0m. On first contact, the relative density of the sands was loose (BH1) or medium dense (BH101 and BH102). In BH1, the sands improved to medium dense below 1.5m depth. The relative density of the sands was then consistently medium dense with depth, with the exception of BH102, where the sands were loose below 2.5m depth to the borehole termination depth. We note that BH2, drilled to the north of the site, encountered medium dense sands which extended to 5.0m depth.

Marine Clays

Marine silty clays were encountered below the sands in BH1 and extended to the borehole termination depth at 18.0m. The silty clays were assessed to be of high plasticity and, on first contact, firm to stiff strength. Below 12.0m depth the clays improved to very stiff strength. We note that in BH2, drilled to the north of the site, the silty clays were encountered at 5.0m depth and were very stiff from first contact.

Groundwater

Groundwater seepage was encountered whilst auger drilling the boreholes at depths of 3.2m (BH102), 4.6m (BH101) and 6.0m (BH1). On completion of the boreholes, standing water levels were recorded at depths of 5.4m (BH1 and BH102) and 5.6m (BH101). We note that groundwater levels may not have stabilised over the relatively short observation period. No longer term groundwater monitoring has been undertaken.

3.3 Laboratory Test Results

A summary of the laboratory chemical test results is provided in the table below.

Borehole Number	Sample Depth (m)	Description	pH Units	Sulphate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm.cm)
1	3.0 – 3.45	SAND	9.6	<10	<10	20,000
2	6.0 – 6.45	Silty CLAY	8.5	31	110	5,000

4 RISK ASSESSMENT

We note that this report has not been prepared in accordance with the Pittwater Council Risk Management Policy. The site is relatively flat and, as identified in the Pre DA Council minutes, "The subject site is shown as affected by coastal erosion and inundation hazards on Council's coastal hazard identification mapping. As such, the Coastline Risk Management Policy for Development in Pittwater (Appendix 6, P21 DCP) and associated 3.3 Coastline (Beach) Hazard Controls" applies. The HCEPL coastal engineering report address these issues.

With regard to geotechnical risk assessment issues, we note that the site is relatively flat lying and based on a review of Council's Geotechnical Hazard Mapping, the site is not located within a Council Geotechnical Hazard Zone. Furthermore, there are no excavations proposed as part of the development. We have therefore assumed the Council Geotechnical Risk Management Policy does not apply.

5 COMMENTS AND RECOMMENDATIONS

5.1 Footing Design

We note that the TTW report indicates either pad footings or piled footings would be suitable with the floor slab suspended between the footings. Based on the results of the current and previous investigations, between surface level and 1.5m depth the relative density of the sands ranges between loose and medium dense. In addition, below 2.5m depth in BH102, the sands were loose. As a result, pad or pile footings would need to be designed assuming loose sands.

On this basis, a pad footing (at least 1m width) founded within loose sands at a depth of 1.0m may be designed using a maximum allowable bearing pressure of 150kPa.

For piers installed into loose sands and founded at least 3.6m below surface level, a maximum allowable end bearing pressure of 500kPa may be adopted.

For pad or pile footings total settlements are likely to be of the order of 5m and will essentially occur 'instantaneously' as the load is applied to the sandy soils i.e. mainly during construction. In addition, we note that the bearing pressures and settlements in sandy soils are a function of footing geometry in addition to soil properties. Therefore, once the structural loads have been finalised, additional bearing capacity and settlement analyses should be carried out to assess the suitability of the footing options discussed above.

Excavations for pad or strip footings in the sands should be supported with formwork, as vertical cuts will be potentially unstable. Just prior to pouring, all pad and strip footings should have their bases compacted using a hand operated vibrating plate compactor, together with thorough moistening of the footing base.

DCP testing should be carried out by an experienced geotechnical engineer from the base of all pad footing excavations and/or at proposed pile footing location to confirm that the design foundation material has been reached, or is present, and to check initial assumptions about ground conditions and possible variations that may occur between the test locations. We recommend the trafficking of the pad footing excavations be kept to a minimum, preferably not at all, and that they protected by a blinding layer of concrete immediately after inspection.

Bored piles could be used and drilled using a piling rig or an auger attachment to an excavator. However, temporary liners would need to be used, particularly if seepage is encountered, and collapse could occur before they were installed.

Suitable alternatives would be steel screw piles, driven piles or grout injected (CFA) piles installed into the loose (or denser) sands at a similar depth as described above. Steel screw piles could be installed suing a tracked excavator whilst driven or CFA piles would require a piling rig.

We note that 'decompression' could occur if the CFA piles extend below the groundwater and the associated surface settlements could have a detrimental impact on buried services, paved car parks areas that are to remain or other infrastructure. A site trial in an appropriate area would need to be undertaken under the direction of a geotechnical engineer to assess potential 'decompression'. Alternatively, 'decompression' effects would be satisfactory controlled by using double rotary CFA piles, which includes a casing system to support the drill hole.

5.2 Construction Considerations

5.2.1 Working Platform

The piling rig will require a working platform designed by a geotechnical engineer. The design of the working platform will need to be based on the loadings and track dimensions supplied by the contractor for the specific equipment proposed and so a generic design cannot be completed ahead of time. Further, the assessment of the working platform will need to be based on the methodology outlined in BRE 2004 *'Working Platforms for Tracked Plant'*.

The working platform, if required, may need to be constructed using DGS40, DGS20 or DGB20 (or a similar durable granular material approved by the geotechnical engineer) compacted to at least 95% Modified Maximum Dry Density (MMDD) using minimum 8 tonne.

Following removal of vegetation, stripping of any root affected soils, the exposed sand subgrade will require proof rolling before placing the working platform. Proof rolling will improve the near surface compaction of the soils and assist in identifying any soft or unstable areas and should be completed in the following manner:

- Proof roll the existing soil subgrade with at least eight passes of a minimum 8 tonne deadweight smooth drum vibratory roller (used on the static [non-vibration] mode). The sand subgrade will need to be thoroughly moistened before commencing proof rolling.
- To assist with proof rolling, we recommend that a thin layer of road base (75mm size) be placed over the sand subgrade to improve near surface compaction and prevent shearing during rolling.
- Proof rolling should be carried out under the direction of an experienced geotechnical engineer to assist in the detection of soft or unstable areas not disclosed by this investigation.
- Any soft or unstable areas identified during proof rolling should be locally excavated down to a competent base and replaced with engineered fill comprising the working platform materials, described above.

Care will need to be exercised close to any existing nearby structures, any buried services and the sloping section of the platforms that extend into the pond, as ground borne vibrations caused by the proof rolling may cause damage/instability. Ground vibrations should be qualitatively monitored by the site supervisor and if there are any causes for concern during proof rolling, then further geotechnical advice should be sought and/or the non-vibration (static) mode of the roller used.

Density tests should be regularly carried out on the working platform materials to confirm the above density has been achieved. The frequency of density testing should be at least one test per layer per 2500m^2 or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. However, our preference would be for Level 1 control of fill placement and compaction, in accordance with AS3798-2007.

Further advice can be given once details of the proposed tracked equipment have been provided.

5.2.2 Potential Vibration and Ground Surface Movement Risks

There may be areas of poorly compacted fill and/or very loose sands not disclosed by the investigation. We therefore advise that sudden stop/start movements of any tracked equipment should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures within the site and beyond the site boundaries.

5.3 External Paved Areas

External paved areas (footpaths etc) that will not be supporting traffic loads will be constructed largely on the sandy subgrade. The preparation in such areas should comprise placing between 50mm and 100mm of fine crushed igneous rock (or roadbase), light watering of this material, and compaction using at least 10 passes of a 2 tonne smooth drum roller. This fine crushed rock will help provide uniform support at the footpath joints, thereby reducing the potential for stepping at the joints.

5.4 Soil Aggression

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability and AS3600-2009 “Concrete Structures” we note that the laboratory chemical test results have indicated that the following Exposure Classifications are applicable:

- An A1 Exposure Classification for high level concrete footings, based on Table 4.8.1 of AS3600,
- A ‘Mild’ Exposure Classification for concrete pile footings founded in sands as the piles may extend below groundwater level (refer BH102), based on Table 6.4.2(C) of AS2159, and
- A ‘Mild’ Exposure Classification applies for steel screw piles assuming they will be impacted by saline groundwater, based on Table 6.5.2(A) of AS2159.

However, we note that the groundwater has the potential to be saline and a more onerous exposure classification may apply. We therefore recommend that groundwater sampling and testing be undertaken to confirm that the above exposure classifications apply, particularly if pile footings are adopted.

5.5 Earthquake Design Parameters

Based on the results of the investigation, the following design parameters should be adopted for earthquake design in accordance with AS1170.4-2007 ("Structural Design Actions, Part 4: Earthquake Actions in Australia"):

- Hazard Factor (Z) = 0.09
- Site Subsoil Class = Class C_e

5.6 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Groundwater sampling to assess aggression and confirm exposure classifications.
- Working platform design.
- Additional analysis of proposed footing systems founded in the sandy soil profile.
- Inspection and testing of pad or strip footing bases, if appropriate.
- Witnessing installation of pile footings.

6 GENERAL COMMENTS

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

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

CERTIFICATE OF ANALYSIS 122817

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: D Fisher

Sample log in details:

Your Reference:	**28092ZR, Mona Vale**	
No. of samples:	2 Soils	
Date samples received / completed instructions received	02/02/15	/ 02/02/15

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	9/02/15	/ 9/02/15
Date of Preliminary Report:	Not Issued	

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

Results Approved By:

Jacinta Hurst
Laboratory Manager

Misc Inorg - Soil			
Our Reference:	UNITS	122817-1	122817-2
Your Reference	-------------	BH1	BH2
Depth	------------	3.0-3.45	6.0-6.45
Date Sampled		30/01/2015	30/01/2015
Type of sample		Soil	Soil
Date prepared	-	04/02/2015	04/02/2015
Date analysed	-	04/02/2015	04/02/2015
pH 1:5 soil:water	pH Units	9.6	8.5
Chloride, Cl 1:5 soil:water	mg/kg	<10	110
Sulphate, SO4 1:5 soil:water	mg/kg	<10	31
Resistivity in soil*	ohm m	200	50

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-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B.
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.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			04/02/2015	[NT]	[NT]	LCS-1	04/02/2015
Date analysed	-			04/02/2015	[NT]	[NT]	LCS-1	04/02/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	109%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	116%
Resistivity in soil*	ohm m	1	Inorg-002	<1.0	[NT]	[NT]	[NR]	[NR]

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC 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.

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BOREHOLE LOG

Borehole No.
1

Client: PITTWATER COUNCIL
Project: PROPOSED RE-DEVELOPMENT OF SLSC BUILDING
Location: SURFVIEW ROAD, MONA VALE, NSW

Job No. 28092ZR
Date: 30-1-15
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 7.4m
Datum: AHD
Logged/Checked by: D.A.F./ JR

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		SP	SAND: fine to medium grained, light brown.	M	L		MARINE
	DS	N = 4 3,2,2								
			1							
	DS	N = 12 5,6,6						MD		
			2							
	DS	N = 12 7,7,5	3							
			4			as above, but trace of fine to medium grained sandstone gravel.				
	DS	N = 30 9,13,17				SAND: fine to medium grained, dark grey, grey and light brown.				
ON COMPLETION ▼			5							
▶	DS	N = 14 4,5,9	6				W	MD		
			7							

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BOREHOLE LOG

Client: PITTWATER COUNCIL
Project: PROPOSED RE-DEVELOPMENT OF SLSC BUILDING
Location: SURFVIEW ROAD, MONA VALE, NSW

Job No. 28092ZR **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 7.4m
Date: 30-1-15 **Datum:** AHD
Logged/Checked by: D.A.F./PL

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
					SP	SAND: fine to medium grained, dark grey, with clay fines, and trace of silt fines.	W	MD		
	DS	N = 18 2,7,11	8			SAND: fine to medium grained, grey and brown, trace of clay and silt fines.				
	DS				SC	CLAYEY SAND: fine to medium grained, grey and brown.				
	DS	N = 12 5,5,7	9			as above, but trace of fine to coarse grained sandstone gravel.				
			10		CH	SILTY CLAY: high plasticity, grey and orange brown.	MC>PL	F-St		
	DS		11						100 90 110	HP TESTING ON REMOULDED SAMPLE
	DS	N = 19 4,7,12	12					VSt	350 400 400	
			13			as above, but grey and red brown.				
			14							

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BOREHOLE LOG

Borehole No.
1

Client: PITTWATER COUNCIL
Project: PROPOSED RE-DEVELOPMENT OF SLSC BUILDING
Location: SURFVIEW ROAD, MONA VALE, NSW

Job No. 28092ZR
Date: 30-1-15
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 7.4m
Datum: AHD
Logged/Checked by: D.A.F./PR

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
								CH	SILTY CLAY: high plasticity, grey and red brown.	MC>PL	VSt		
				■	N = 14 4,6,8	15						300 300 310	
						16							
						17							
						18			END OF BOREHOLE AT 18.0m				
						19							
						20							

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BOREHOLE LOG

Borehole No. **101**
1/1

Client: WARREN AND MAHONEY ARCHITECTS PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: MONA VALE SLSC, SURFVIEW ROAD, MONA VALE, NSW

Job No. 28092R
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 7.5m
Date: 25/10/18
Datum: AHD
Logged/Checked by: A.M./P.R.

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			FILL: Silty clay, low to medium plasticity, brown, trace of root fibres.	w<PL			
					SP	SAND: fine to medium grained, brown.	M	MD		MARINE
		N = 15 8,8,7								
			1							
		N = 10 6,4,6								
			2							
		N = 20 10,9,11	3							
			4							
▶		N = 18 8,9,9				SAND: fine to medium grained, yellow brown.	W			
			5							
▼ ON COMPLETION						SAND: fine to medium grained, grey.				
			6			END OF BOREHOLE AT 6.0m				
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
102
1/1

Client: WARREN AND MAHONEY ARCHITECTS PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: MONA VALE SLSC, SURFVIEW ROAD, MONA VALE, NSW

Job No. 28092R
Date: 25/10/18
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 7.1m
Datum: AHD
Logged/Checked by: A.M./P.R.

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			FILL: Silty clay, low to medium plasticity, brown, fine to medium grained sand, trace of ironstone gravel and root fibres.	w>PL			GRASS COVER
					SP	SAND: fine to medium grained, red brown, trace of ironstone gravel.	D	MD		MARINE
		N = 20 7,10,10				SAND: fine to medium grained, brown, trace of clay fines.	M			
			1							
		N = 13 5,6,7								
			2					L		
			3							
▶		N = 8 4,4,4					W			
			4							
		N = 6 4,4,2								
			5							
▼ ON COMPL-ETION						Clayey SAND: fine to medium grained, yellow brown.	W			
						Clayey SAND: fine to medium grained, dark grey mottled yellow brown.				
			6			END OF BOREHOLE AT 6.0m				
			7							

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

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

Title: **SITE LOCATION PLAN**

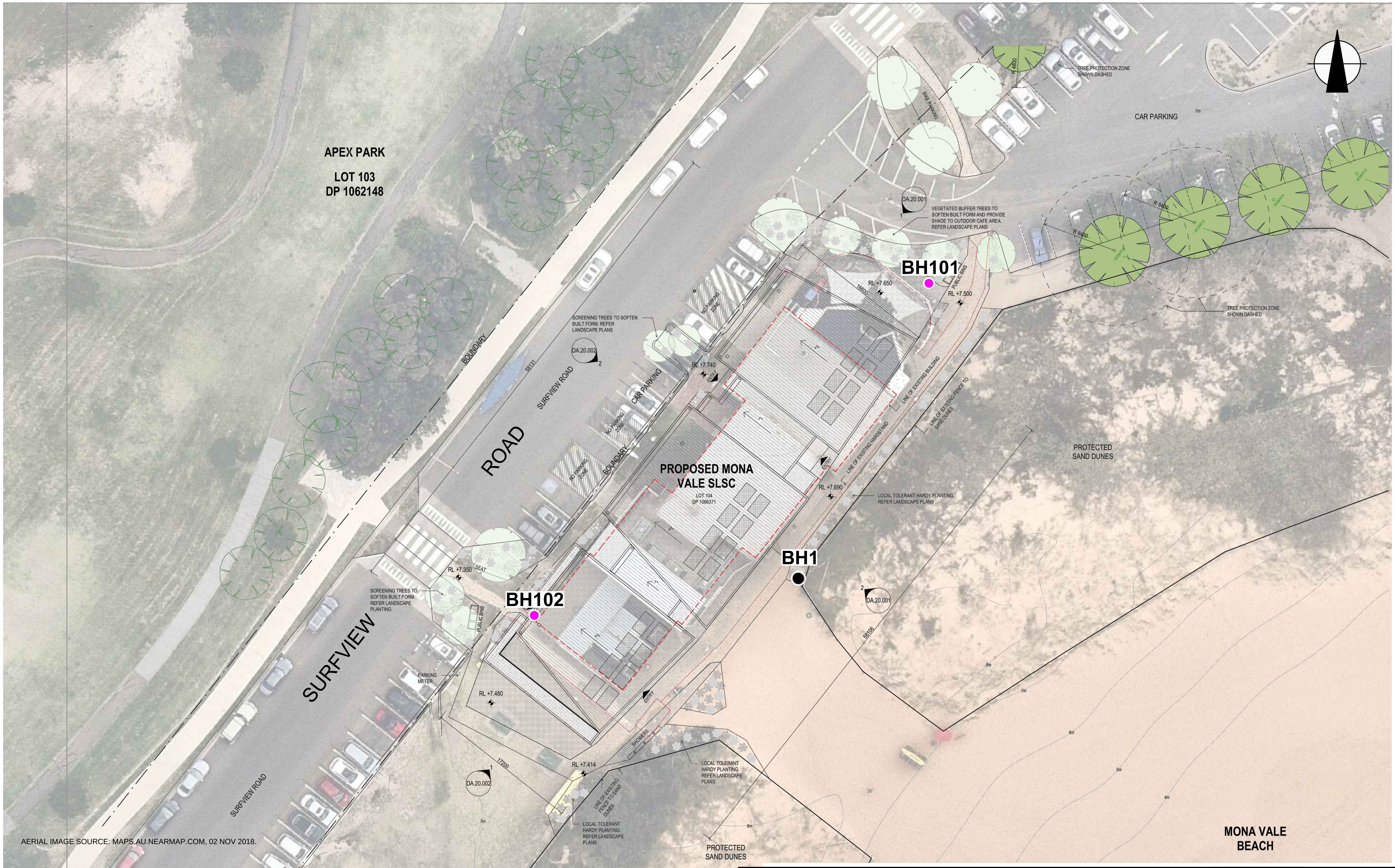
Location: MONA VALE SURF LIFE SAVING CLUB
SURFVIEW ROAD, MONA VALE, NSW

Report No: 28092RRPT2

Figure No: 1

JK **Geotechnics**

PLOT DATE: 13/11/2018 4:59:30 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\28000'S\28092ZR MONA VALE\CAD\28092RRPT2.DWG



LEGEND

- BH101 BOREHOLE LOCATION, (EIS 2018)
- BH1 BOREHOLE LOCATION (JK GEOTECHNICS 2015)

SCALE 0 4 8 12 16 20 METRES 1:400 @A3

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

Title:	BOREHOLE LOCATION PLAN
Location:	MONA VALE SURF LIFE SAVING CLUB SURFVIEW ROAD, MONA VALE, NSW
Report No:	28092Rrpt2
Figure No:	2

JK Geotechnics

PLOT DATE: 13/11/2018 5:05:36 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\28000'S\28092ZR MONA VALE\CAD\28092RRPT2.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

- 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:

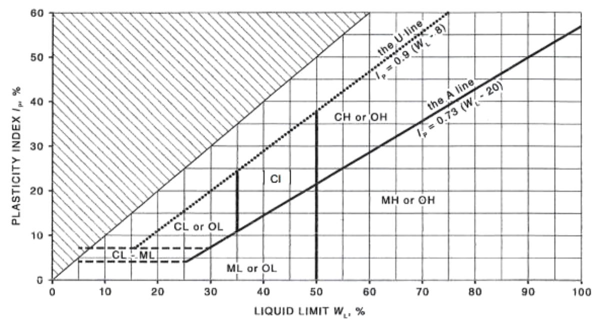
$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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 U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

<table>
<tr><th>Log Column</th><th>Symbol</th><th>Definition</th></tr>
<tr><td>Remarks</td><td>‘V’ bit</td><td>Hardened steel ‘V’ shaped bit.</td></tr>
<tr><td></td><td>‘TC’ bit</td><td>Twin pronged tungsten carbide bit.</td></tr>
<tr><td></td><td>T_{60}</td><td>Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.</td></tr>
<tr><td></td><td>Soil Origin</td><td>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.</td></tr>
</table>

Classification of Material Weathering

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

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

Rock Material Strength Classification

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