REPORT TO

WILLIAM CAREY CHRISTIAN SCHOOL

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MODULAR BUILDINGS

AT

WILLIAM CAREY CHRISTIAN SCHOOL,
38 - 44 BUMBERA STREET, PRESTONS, NSW

Date: 24 September 2024
Ref: 36929PErpt

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
36929PErpt	Final Report	24 September 2024

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed modular buildings located at William Carey Christian School, 38 – 44 Bumbera Street, Prestons, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Gregory Eyears of William Carey Christian School by return of a signed 'Acceptance of Proposal' form dated 16 August 2024. The commission was on the basis of our fee proposal, Ref. 'P61164PE' dated 14 August 2024.

We have received two sets of architectural drawings (Project No. 270, Drawing Nos. SK1.01 to SK1.04, all Revision B, & Project No. 2408, SK1C.00, SK1C.02, SK1C.02.1, SK1C.02.5, SK1C.02.6, SK1C.03, SK1C.04 to SK1C.07, SK1C.09 & SK1C.10, all Revision E) prepared by Marathon Modular. Based on a review of the supplied drawings, we understand the proposed development will comprise the construction of a single storey modular 'amenities' building and a two storey modular 'classroom' building. The ground floor finished floor level (FFL) of the amenities building at RL2.4m (assumed datum) and classroom building at RL40.9m (AHD) is a maximum height of approximately 0.7m above surrounding surface levels.

We have not been supplied with any structural loads and have assumed typical loadings for this type of development will apply.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at six borehole locations, and to use this information for providing our comments and recommendations on footing design and exposure classification.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was caried out on 26 August 2024 and comprised the drilling and testing of six boreholes (BH1 to BH6) to approximately 6m depth below existing surface levels using spiral augering techniques with our track mounted JK305 drilling rig. BH1 to BH4 were drilled within the footprint of the proposed two storey classroom (i.e. Site 1) and the remaining BH5 and BH6 were drilled within the footprint of the proposed single storey amenities building (i.e. Site 2).

Prior to the commencement of the fieldwork, a specialist sub-consultant reviewed available 'Before You Dig Australia' information and electro-magnetically scanned the borehole locations for buried services.

The fieldwork for the investigation was carried out in the full time presence of our geotechnical engineer (John Lo), who set out the borehole locations, coordinated the electro-magnetic scanning, nominated testing and sampling, and prepared the borehole logs. The borehole logs are attached to this report, together with a glossary of terms and symbols used.

The borehole locations, as shown on the attached Figure 2 (Site 1) and Figure 3 (Site 2), were set out using taped measurements from existing surface features. The surface RL's shown on borehole logs (BH1 to BH4 only) were obtained by interpolation between spot heights and contours shown on a drawing prepared by

Marathon Modular (i.e. Drawing No. SK1C.02.6[E]). The datum is the Australian Height Datum (AHD). We note that a survey plan surrounding the proposed amenities building was not provided and the surface RL's at BH5 & BH6 could not be estimated.

The concrete pavement encountered at the ground surface (BH1 & BH2 only) was penetrated with a thin walled diamond tipped tube, with water flush. Below the pavement and from ground surface (BH3 to BH6), the relative compaction of the fill and strength of the natural soils was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on cohesive soils recovered in the SPT split-spoon sampler, and by tactile examination.

The strength of the underlying bedrock was assessed by observation of the auger penetration resistance when using a tungsten carbide (TC) drill bit, together with examination of the recovered rock cuttings and correlations with subsequent laboratory moisture content test results. The assessment of rock strength in this way is subjective, and variations of say one order of strength should not be unexpected.

Groundwater observations were made in the boreholes during and on completion of drilling. Long term groundwater level monitoring was outside the agreed scope of the geotechnical investigation. For details of the adopted investigation techniques employed, and their limitations, reference should be made to the attached Report Explanation Notes.

Selected soil and rock chip samples were returned to Soil Test Services (STS), a NATA accredited laboratory, for moisture content, Atterberg Limits and linear shrinkage testing. The results of these tests are summarised in the attached STS Table A. Additional soil samples were returned to a second NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, sulphate and chloride content and resistivity testing. The results are presented in the attached 'Certificate of Analysis 361138'.

3 RESULTS OF INVESTIGATION

3.1 Site Description

William Carey Christian School is located on the eastern side of a localised gully which slopes gently down toward Cabramatta Creek at less than approximately 2°. The school generally consists of one and two storey brick buildings, concrete and asphaltic concrete (AC) surfaced carparks, hardstands, sports courts, and walkways, as well as grass ovals/playgrounds and garden beds comprising small and medium size bushes and trees.

Based on a cursory inspection, the buildings and structures adjacent to the borehole locations generally appeared to be in reasonable condition, with the exception of some minor crocodile and longitudinal cracking up to approximately 5mm wide noted in the AC pavement beside the proposed amenities building.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates that 'Site 1' (i.e. the proposed two storey classroom building) is underlain by alluvial deposits comprising medium grained sand, silt and clay, whereas 'Site 2' (i.e. the proposed single storey amenities building) is underlain by Bringelly Shale of the Wianamatta Group.

The boreholes disclosed a generalised subsurface profile comprising fill overlying alluvial soils at Site 1 and residual clay soils underlain by siltstone bedrock at Site 2. Groundwater was encountered at shallow to moderate depth at Site 1 only. Reference should be made to the attached borehole logs for specific details at the investigation locations. A summary of the subsurface conditions encountered at the borehole locations is presented below.

Pavement

Unreinforced concrete was encountered from surface level in BH1 and BH2 and was 100mm thick.

Fill

Fill comprising sand, silt and clay was encountered below the pavements in BH1 & BH2 and from surface level in the remaining boreholes (i.e. BH3 to BH6) and extended to depths between 0.4m (BH1) and 1.0m (BH4). The fill contained minor inclusions of fine to medium grained ironstone gravel, fine to medium grained sand, slag, and root fibres. Where the fill was of sufficient thickness for testing (i.e. BH4 only), the fill was assessed to be well compacted.

Natural Soils

At Site 1, alluvial soils comprising clayey silt, sandy clayey silt, silty clay and sandy clay were encountered below the fill and were generally of low to medium plasticity and of firm, stiff and very stiff strength. However, soils of soft to firm strength were encountered in BH2 (between 2.0m and 3.3m depth), BH3 (between 3.0m and 4.5m depth) and BH4 (between 2.5m and 4.0m depth).

A relatively consistent silty gravelly clay layer was encountered below the aforementioned weaker soils at respective depths of 3.4m (BH1), 3.3m (BH2), 4.5m (BH3) and 4.0m (BH4) and was generally of low to medium plasticity and of stiff to very stiff strength. It is possible that the hand penetrometer tests could be artificially low due to disturbance caused by the presence of the gravel, and it is possible that the strength is very stiff. Clay soils of medium and high plasticity and of very stiff to high strength were encountered below the gravelly clays and extended to the borehole termination depths between approximately 6m and 6.3m; it appeared these were residual soils, though this could not be determined due to the limited penetration.

At Site 2, residual clay soils were encountered underlying the fill and were of medium to high plasticity and of very stiff strength. The clayey soils contained various proportions of fine to medium grained ironstone gravel and root fibres.

Weathered Bedrock

At Site 1, weathered siltstone bedrock was encountered below the residual clay soils in BH5 & BH6 at depths of 4.0m and 3.6m, respectively. The siltstone was initially extremely weathered and of 'soil' strength, becoming distinctly weathered and of low rock strength below depths of approximately 5.5m (BH5) and 5.0m (BH6). BH5 & BH6 were terminated within the low strength bedrock at 6m depth.

At Site 2, weathered bedrock was not encountered within the boreholes. However, extremely weathered siltstone bands were noted within some of the residual clays and it is possible that the bedrock surface may be located a short depth below the borehole termination depths. In a previous project about 80m to the south, siltstone bedrock was encountered at depths between about 5m and 7m.

Groundwater

At Site 1, groundwater seepage was recorded at respective depths of approximately 3.0m (BH1), 2.0m (BH2 & BH3) and 2.5m (BH4) during spiral auger drilling. Prior to backfilling, the groundwater level in BH1 to BH4 was approximately 1.9m (RL38.3m), 2.1m (RL38.1m), 2.1m (RL38.2m) and 2.7m (RL37.6m), indicating groundwater levels appear to be sloping down to the north-west toward Cabramatta Creek. We note that groundwater levels may not have stabilised within the limited observation period prior to backfilling the boreholes.

At Site 2, BH5 & BH6 were 'dry' during and on completion of auger drilling.

No longer-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The moisture content tests completed on the rock chip samples correlated well with our field logging of rock strength. The results of the Atterberg Limits confirm the natural alluvial silt and clay samples from BH1 and BH3 are of low plasticity and the natural residual clay sample from BH6 is of high plasticity. The tested silt/clay samples were all slightly 'wet' of their plastic limit. The linear shrinkage tests indicated the alluvial clay and silt soils were of low reactivity and the residual clayey soil was of moderate reactivity with changes in moisture content.

The soil pH test results were 5.4, 7.3 and 7.5 which showed the samples were slightly acidic to near neutral. The sulphate and chloride test results were less than 390mg/kg which indicates low sulphate and chloride contents. The resistivity test results were between 2,300 ohm.cm and 30,000 ohm.cm which indicates moderate to high resistivity.

4 COMMENTS AND RECOMMENDATIONS

4.1 Amenities Building

The proposed amenities building is expected to be constructed up to a maximum height of approximately 0.7m above current surface levels within the existing garden bed. An access ramp which grades up at approximately 1 Vertical (H) in 14 Horizontal (H) will also be constructed at the western end of the building. The footprint of the proposed building and ramp is shown on Figure 3.

Due to the presence of clayey fill with depths greater than 0.4m and considering the boreholes were located within a garden bed, the existing fill was likely end dumped with limited compaction and must be considered to be uncontrolled. As such, the existing fill is not suitable to support any structural loads, the site classifies as 'Class P' in accordance with AS2870-2011 'Residential Slabs and Footings', and the design of footings must be by engineering principles. The presence of the garden bed and adjacent building/pavements may also lead to abnormal moisture conditions which would further result in a 'Class P' site. Notwithstanding this, AS2870 allows reclassification if assessed in accordance with engineering principles, and characteristic surface movements at the proposed amenities building could be in the order of about 40mm (i.e. in the range of a Class 'M' site).

Due to the presence of the existing fill, there are two options that could be considered for the proposed building. If the uncontrolled fill is left in place, apart from removing any obvious deleterious, organic or contaminated material and/or vegetation, trees etc. which must be removed, then the proposed single storey building could be supported on piles founded below the fill and within the underlying natural clay soils. The ground floor slab and ramp would then need to be designed as fully suspended and supported on the above mentioned piles. Void formers must be used below the walls and slab to isolate them from damage as a result of possible heave.

As an alternative to the above, to allow the use of shallow footings, all of the existing fill would need to stripped and replaced with controlled, engineered fill. However, due to the risk of the earthworks being carried out to a poor standard, which could possibly lead to unacceptable total and differential movements in the long-term, our preference is to suspended the building and ramp from piles founded in the underlying clayey soils. It is also possible that compaction of the new fill would cause vibration damage to the nearby structure.

For the subsurface conditions encountered at the borehole locations, piled footings (bored or screw piles) founded in the underlying natural residual clays of at least very stiff strength and below a depth of 1.8m or 4 pile diameters (whichever is greater) may be designed for an allowable end bearing pressure of 300kPa.

If screw piles are to be adopted, the helix diameter for screw piles should be designed based on the provided bearing pressure outlined above and not based upon empirical correlations with installation torque. The contribution of a 'secondary helix' or 'skin friction' should also be ignored. We note that screw piles are typically designed and constructed by the piling contractor. As such, the contractor must provide appropriate certification of both the structural and geotechnical load capacity for the screw piles, if adopted.

If bored piles are preferred, they should be drilled, cleaned out and inspected by a geotechnical engineer (prior to the installation of the reinforcement cages) and poured on the same day as drilling. Geotechnical inspection will likely only be required for the initial stages of piling works to confirm that a satisfactory being stratum is being achieved, provided the piling contractor is adequately drilling and cleaning out each of the pile holes.

Another alternative would be to use pad footings through the existing garden bed and founded at least 0.8m depth and at least 0.3m into residual clays of at least very stiff strength, and then suspending the structure with a void below. such footings could be designed for an allowable bearing pressure of 150kPa subject to geotechnical inspection of representative footings. Appropriate reactive movements would need to be accounted for in the design.

4.2 Classroom Building

The ground floor FFL of the proposed two storey classroom building at RL40.9m is approximately 0.7m above current surface levels within the existing courtyard. In addition, a first floor walkway will be constructed and link the south-western end of the proposed building with the adjacent two storey building. The footprint of the proposed building and walkway are shown on the attached Figure 2.

Due to the presence of clayey/silty fill with depths greater than 0.4m and the soft to firm strength soils encountered in each borehole, the site classifies as 'Class P' in accordance with AS2870-2011, and the design of footings must be by engineering principles, similar to the proposed amenities building. Again, the presence of the adjacent buildings and concrete pavements may also lead to abnormal moisture conditions which would further result in a 'Class P' site.

Due to the possibility of long term total and differential settlements of the underlying alluvial silts and clays of soft to firm strength, we do not recommend the use of the high level footings or on-grade floor slabs at this site. As such, we recommend the proposed two storey building is supported on piles founded in the underlying alluvial gravelly clays encountered at moderate depth. Screw piles founded at least 4 pile diameters into the underlying natural gravelly clays of at least stiff strength may be designed for an allowable end bearing pressure of 180kPa. As mentioned in Section 3.2, it is likely that the gravelly clays are in fact very stiff. If that can be proven, such as by completing Cone Penetration Tests (CPT's) though the soils if access is provided (access was not possible at the time of the investigation), it is likely that the allowable bearing pressure could be increased to 300kPa.

Bored piles are not considered suitable for this site due to the groundwater encountered at shallow and moderate depths. The advice presented in Section 4.1 for screw piles must also be adopted for this site.

4.3 Soil Aggression

The soil aggression test results have indicated slightly acidic to near neutral conditions, low sulphate and chloride contents, and moderate to high resistivity values. In accordance with Table 6.4.2 (C) and

Table 6.5.2 (C) of AS2159-2009 'Piling-Design and Installation', the exposure classification to concrete and steel piles are 'mild and 'non-aggressive', respectively. In accordance with Table 4.8.1 of AS3600-2018 'Concrete Structures', the exposure classification to buried concrete elements is 'A2'.

5 GENERAL COMMENTS

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

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

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

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

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

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Modular Buildings
Location: 38-44 Bumbera Street, Prestons, NSW

Report No.: 36929PE - A
Report Date: 24/09/2024

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	0.50 - 0.95	18.1	30	16	14	4.5*
1	6.00 - 6.01	17.7	-	-	-	-
3	1.50 - 1.95	18.9	21	16	5	1.5*
3	3.00 - 3.45	21.3	-	-	-	-
4	3.00 - 3.45	20.4	-	-	-	-
4	6.00 - 6.27	18.7	-	-	-	-
5	4.00 - 4.50	11.9	-	-	-	-
6	0.50 - 0.95	18.8	50	16	34	12.0**
6	5.00 - 6.00	9.2	-	-	-	-

Notes:
• The test sample for liquid and plastic limit was air-dried & dry-sieved
• The linear shrinkage mould was 125mm
• Refer to appropriate notes for soil descriptions
• Date of receipt of sample: 06/09/2024.
• Sampled and supplied by client. Samples tested as received.
• * Denotes Linear Shrinkage cracked.
• ** Denotes Linear Shrinkage curled.

NATA

NATA Accredited Laboratory Number:1327

24/09/2024
Authorised Signature / Date
(M. Knights)

Envirolab Services Pty Ltd
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12 Ashley St Chatswood NSW 2067
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CERTIFICATE OF ANALYSIS 361138

Client Details

Client	JK Geotechnics
Attention	John Lo
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**36929PE 38-44 Bumbera Street, PRESTONS**
Number of Samples	3 Soil
Date samples received	06/09/2024
Date completed instructions received	06/09/2024

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.

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

Report Details

Date results requested by	13/09/2024
Date of Issue	12/09/2024

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

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

Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		361138-1	361138-2	361138-3
Your Reference	UNITS	BH1	BH4	BH5
Depth		3.0-3.4	0.5-0.95	1.5-1.95
Date Sampled		26/08/2024	26/08/2024	26/08/2024
Type of sample		Soil	Soil	Soil
Date prepared	-	06/09/2024	06/09/2024	06/09/2024
Date analysed	-	10/09/2024	10/09/2024	10/09/2024
pH 1:5 soil:water	pH Units	7.3	7.5	5.4
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	300
Sulphate, SO4 1:5 soil:water	mg/kg	35	260	390
Resistivity in soil*	ohm m	300	45	23

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	[NT]
Date prepared	-			06/09/2024	3	06/09/2024	06/09/2024		06/09/2024	[NT]
Date analysed	-			10/09/2024	3	10/09/2024	10/09/2024		10/09/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	3	5.4	5.3	2	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	300	310	3	104	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	390	440	12	110	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	3	23	22	4	[NT]	[NT]

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

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments

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

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL
Project: PROPOSED MODULAR BUILDINGS
Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 40.2m
Date: 26/8/24 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			CONCRETE: 100mm.t				NO OBSERVED REINFORCEMENT
					-	FILL: Clayey sandy silt, low plasticity, brown, fine to medium grained sand.	w<PL			POSSIBLY NATURAL ALLUVIAL
		N = 8 3,3,5			ML	Clayey SILT: low plasticity, brown and orange brown, with fine grained sand.	w>PL	St-VSt	200 160 250	
			1							
ON COMPLET-ION		N = 11 3,5,6				Sandy clayey SILT: low plasticity, red brown and grey, fine to medium grained sand.		F-St	180 190 80	
			2		CL-CI	Silty CLAY: low to medium plasticity, brown and light grey, with fine to medium grained sand.			80 60 50	HP TESTS ON DISTURBED SAMPLE
		N = 16 4,6,10	3						110 90 80	HP TESTS ON REMOULDED SAMPLE
					CI	Silty gravelly CLAY: medium plasticity, brown and light grey, fine to coarse grained ironstone gravel.		St	120 170 160	HP TESTS AT 3.0m DEPTH AND 4.5m DEPTH LIKELY INFLUENCED BY GRAVEL CONTENT
			4							
		N = 17 11,9,8						St-VSt	130 120 220	
			5							
		N = SPT 5/10mm REFUSAL	6			Silty CLAY: medium plasticity, grey.	w<PL	(VSt-Hd)		RESIDUAL?
						END OF BOREHOLE AT 6.05m				
			7							

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BOREHOLE LOG

Borehole No.
2
1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL
Project: PROPOSED MODULAR BUILDINGS
Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE | **Method:** SPIRAL AUGER | **R.L. Surface:** ≈ 40.2m
Date: 26/8/24 | **Datum:** AHD
Plant Type: JK305 | **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 100mm.t				
			-	FILL: Sandy silt, low plasticity, grey, fine to medium grained sand.	w>PL			POSSIBLY NATURAL
	N = 13 5,6,7		CI	Silty CLAY: medium plasticity, red brown and brown, trace of fine to medium grained sand.	w>PL	St-VSt	200 200 230	ALLUVIAL
	N = 11 3,4,7	1		Silty CLAY: medium plasticity, brown and grey, trace of fine to medium grained sand.		VSt	320 380 330	
ON COMPLETION		2	CL	Sandy silty CLAY: low plasticity, brown and grey, fine to medium grained sand.		S-F	50 30	
	N = 5 0,0,5	3	CL-CI	Silty gravelly CLAY: low to medium plasticity, brown, fine to coarse grained ironstone gravel.		St	110	
	N = 24 9,11,13	4		as above, but brown, red brown and light grey.		VSt-Hd	430 380	
	N = SPT 7/10mm REFUSAL	6	CH	Silty CLAY: high plasticity, light grey and brown.	w<PL	VSt	320 310 300	RESIDUAL?
				END OF BOREHOLE AT 6.01m				
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. **3**

1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL

Project: PROPOSED MODULAR BUILDINGS

Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 40.3m

Date: 26/8/24 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, low plasticity, brown, trace of fine grained sand and root fibres.	w<PL			GRASS COVER
	N = 6 5,3,3		ML	Clayey SILT: low plasticity, brown, with fine grained sand.	w>PL	F-St	70 110 60	ALLUVIAL
	N = 12 6,6,6	1	CL	Silty CLAY: low plasticity, brown and red brown.		VSt	300 330 230	
ON COMPLETION		2						
	N = 2 0,0,2	3		Silty CLAY: low plasticity, brown and grey, trace of fine grained sand.		S-F	40 50 40	
		4		as above, but with fine to medium grained ironstone gravel.				
	N = 29 15,15,14	5	CL-CI	Silty gravelly CLAY: low to medium plasticity, brown and red brown, fine to coarse grained ironstone gravel.		VSt	350 340 270	
	N > 4 9,4/10mm REFUSAL	6	CH	Silty CLAY: high plasticity, light grey and brown.	w<PL	VSt-Hd	420 280 250	RESIDUAL?
				END OF BOREHOLE AT 6.16m				
		7						

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BOREHOLE LOG

Borehole No. 4
1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL
Project: PROPOSED MODULAR BUILDINGS
Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE **Method:** SPIRAL AUGER **R.L. Surface:** N/A
Date: 26/8/24 **Datum:** -
Plant Type: JK305 **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Sandy clay, low plasticity, trace of root fibres.	w<PL			GRASS COVER
	N = 13 7,8,5			FILL: Clayey silt, low plasticity, brown, with fine grained sand, trace of slag.				APPEARS WELL COMPACTED
		1	CL	Silty CLAY: low plasticity, red brown, trace of fine grained sand.	w>PL	St-VSt		ALLUVIAL
	N = 11 5,5,6	2	ML	Clayey sandy SILT: low plasticity, red brown, fine to medium grained sand.			110 180 240	
ON COMPLETION	N = 3 0,1,2	3	CL	Silty sandy CLAY: low plasticity, brown.		S-F	40 60 60	
	N = 21 7,8,13	4 5	CI	Silty gravelly CLAY: medium plasticity, brown and red brown, fine to coarse grained ironstone gravel.		VSt		TOO GRAVELLY FOR HP TESTING
	N > 17 6,17/ 120mm REFUSAL	6	CH	Silty CLAY: high plasticity, light grey and light brown.	w<PL	Hd	450 550 580	RESIDUAL?
		7		END OF BOREHOLE AT 6.27m				

SAMPLES: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No. **5**

1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL

Project: PROPOSED MODULAR BUILDINGS

Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE **Method:** SPIRAL AUGER **R.L. Surface:** N/A

Date: 26/8/24 **Datum:** -

Plant Type: JK305 **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty sandy clay, low plasticity, brown, trace of ironstone gravel and root fibres.	w<PL			MULCH COVER
		N = 16 2,8,8	1		CI-CH	Silty CLAY: medium to high plasticity, grey mottled red brown, trace of fine grained ironstone gravel, and root fibres.	w>PL	VSt	320 390 290	RESIDUAL
		N = 19 7,9,10				Silty CLAY: medium to high plasticity, grey and red brown, with fine to medium grained ironstone gravel.			240 240 230	
		N = 19 7,9,10	2, 3		CH	Silty CLAY: high plasticity, light grey mottled red brown.			280 300 240	
						as above, but light grey.				
			4		-	Extremely Weathered siltstone: silty CLAY, medium plasticity, grey.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
			5			as above, but dark grey and grey.				VERY LOW TO LOW RESISTANCE
						SILTSTONE: dark grey.	DW	L		
			6, 7							

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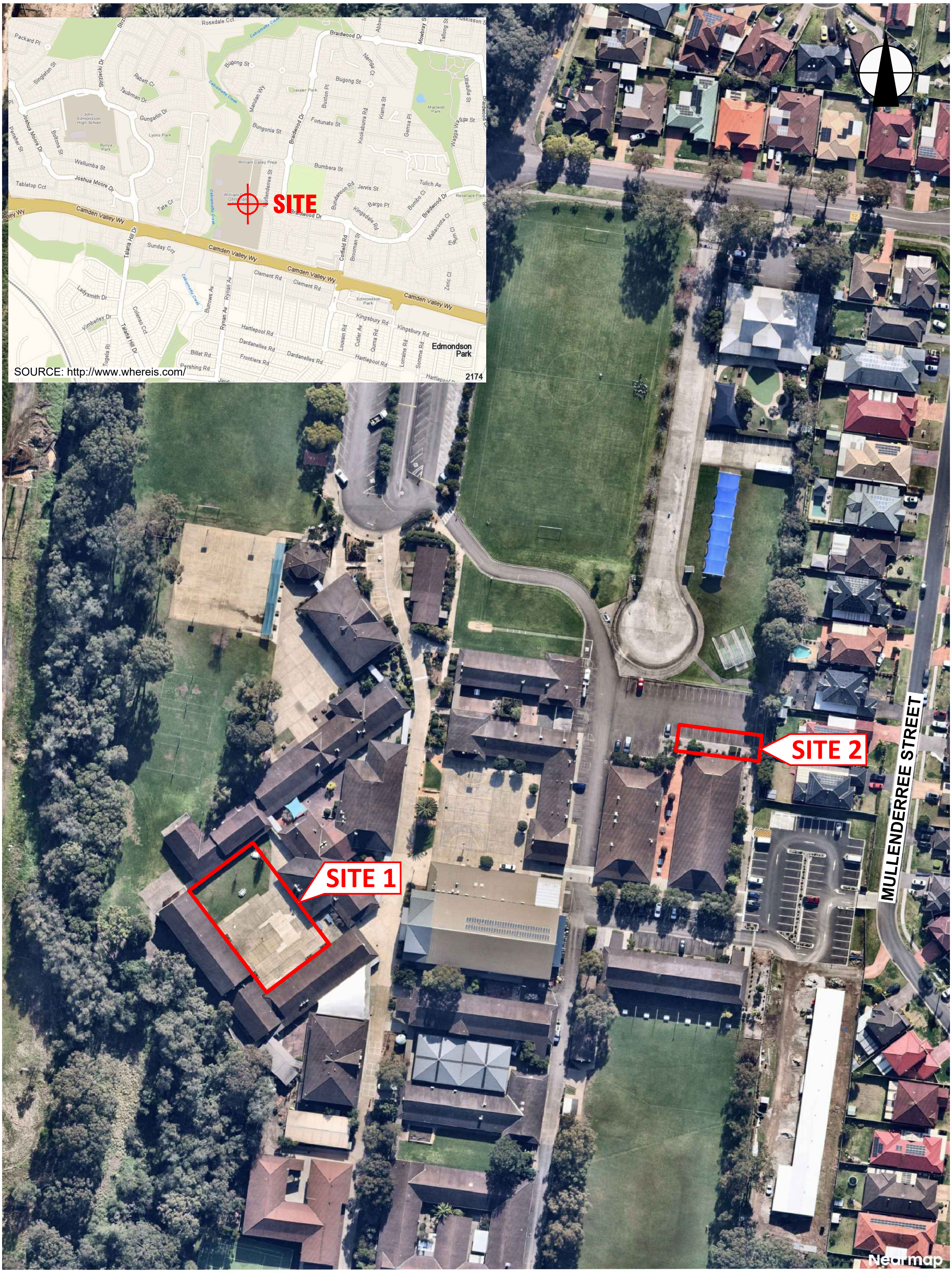
BOREHOLE LOG

Borehole No.
6
1/1

Client: WILLIAM CAREY CHRISTIAN SCHOOL
Project: PROPOSED MODULAR BUILDINGS
Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Job No.: 36929PE **Method:** SPIRAL AUGER **R.L. Surface:** N/A
Date: 26/8/24 **Datum:** -
Plant Type: JK305 **Logged/Checked by:** J.L.O./M.E.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty sandy clay, low plasticity, brown, trace of root fibres.	w>PL			MULCH COVER
	DS	N = 14 5,5,9			CI-CH	Silty CLAY: medium to high plasticity, grey mottled orange brown.	w>PL	VSt	350 280 280	RESIDUAL
			1							
	DS	N = 14 5,7,7				Silty CLAY: medium to high plasticity, grey mottled red brown, trace of fine to medium grained ironstone gravel, and root fibres.			320 330 370	
			2							
	DS					Silty CLAY: medium to high plasticity, grey, trace of fine to coarse grained ironstone gravel.				
	DS	N = 20 8,9,11	3						250 260	
	DS				-	Extremely Weathered siltstone: silty CLAY, medium plasticity, grey.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
			4							
	DS		5			SILTSTONE: dark grey.	DW	L		
			6			END OF BOREHOLE AT 6.0m				
			7							



PLOT DATE: 24/09/2024 10:04:10 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36929PE PRESTONS\CAD\36929PE.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	SITE LOCATION PLAN
Location:	38-44 BUMBERA STREET, PRESTONS, NSW
Report No: 36929PE	Figure No: 1

JKGeotechnics

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 2.5 5 7.5 10 12.5

SCALE 1:250 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN (SITE 1)**

Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Report No: 36929PE

Figure No: 2

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PLOT DATE: 24/09/2024 10:04:18 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36929PE PRESTONS\CAD\36929PE.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN (SITE 2)**

Location: 38-44 BUMBERA STREET, PRESTONS, NSW

Report No: 36929PE

Figure No: 3

JKGeotechnics

PLOT DATE: 24/09/2024 10:05:23 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36929PE PRESTONS\CAD\36929PE.DWG

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 or where only a limited investigation has been completed or 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:

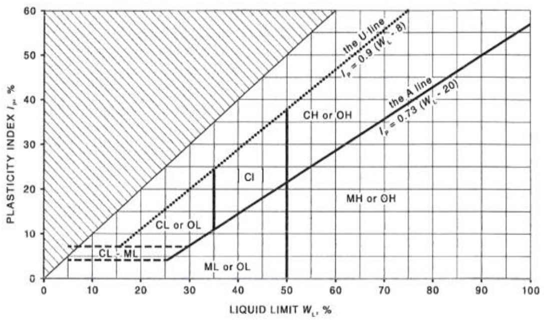
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



JKGeotechnics

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* ≈ PL *w* < PL *w* ≈ 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)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)** VERY LOOSE ≤ 15 0 – 4 LOOSE > 15 and ≤ 35 4 – 10 MEDIUM DENSE > 35 and ≤ 65 10 – 30 DENSE > 65 and ≤ 85 30 – 50 VERY DENSE > 85 > 50 Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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