REPORT TO

KINCOPPAL - ROSE BAY SCHOOL

ON

GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

FOR

PROPOSED ELC BUILDING

AT

CNR NEW SOUTH ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW

Date: 25 February 2020
Ref: 32915SH1rpt

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1 INTRODUCTION

This report presents the results of a geotechnical and hydrogeological investigation for the proposed Early Learning Centre (ELC) building at Kincoppal-Rose Bay, School of the Sacred Heart (KRB), cnr New South Head Road and Vaucluse Road, Vaucluse, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Terry Mahady of Mahady Management, on behalf of KRB, by a signed 'Acceptance of Proposal' form dated 13 December 2019. The commission was on the basis of our fee proposal, Ref: 'P50877PH, dated 9 December 2019.

We were also commissioned to carry out a geotechnical and hydrogeological investigation for two other proposed developments at the school, all of which are at a development application stage. The fieldwork for those investigations were carried out concurrently with the fieldwork for this project. The investigation results for the other two projects are presented in separate reports, Ref. 32915PH2rpt, and Ref. 32915PH3rpt.

The supplied architectural drawing extracts by BVN contained within the State Significant Development Application document dated 13 November 2019, show that the proposed development comprises construction of a one and two storey building. The finished floor level of its lowest level (Level -03) is not shown on the drawings. We were advised by Mr Mahady at proposal stage, that excavation to a maximum depth of about 2m is envisaged on the high side of the proposed building. The approximate outline of the proposed ELC building is shown on the attached Figure 2. Structural loads typical for this type of development have been assumed.

The purpose of the investigation was to assess the subsurface conditions at three borehole locations and, based on the information obtained, present our preliminary comments and recommendations on excavation, retaining wall design, footings, soil aggression, the Level -03 floor slab and hydrogeology.

Our environmental consulting division, JK Environments (JKE), was commissioned to undertake a Preliminary Site Investigation, and this report should be read in conjunction with the JKE report, Ref. E32915BDrpt, dated February 2020.

2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 28 January 2020 and comprised the auger drilling of three boreholes (BH1, BH2 and BH3) to depths of 2.0m, 9.2m and 9.3m, respectively, below existing surface levels. The boreholes were drilled using our track mounted JK205 drilling rig.

The borehole locations, which were set out in consultation with Mr Mahady prior to the commencement of drilling, are shown on the attached Figure 2 and were recorded using a Topcon GRS-1 differential GPS unit. The surface RL to the Australian Height Datum [AHD] of each borehole location is shown on the respective borehole log. At the time the GPS coordinates were recorded, the accuracy was about 100mm in all directions. Figure 2 is based on a recent Nearmap image of the site.

The relative compaction and density of the soil profile were assessed from the Standard Penetration Test (SPT) results. The strength of the underlying bedrock was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings and correlation with subsequent laboratory moisture content test results.

Groundwater observations were also made in the boreholes. A groundwater monitoring well was installed into BH2 and comprised a 50mm diameter Class 18 PVC standpipe. The annulus between the borehole and the slotted length was backfilled with 2mm filter sand. Above the sand backfill, the borehole was sealed with bentonite and then the drilling cuttings. A cast-iron 'Gatic' cover was concreted flush with the ground surface to protect the top of the well. The installation details are presented on the BH2 borehole log.

The borehole logs are attached, together with a set of explanatory notes, which describe the investigation techniques and define the logging terms and symbols used.

Our geotechnical engineer was present full time during the fieldwork to observe site conditions, record the borehole locations using the GPS, nominate the testing and sampling, and prepare the attached borehole logs.

Selected soil samples were returned to our NATA accredited laboratory (Soil Test Services Pty Ltd [STS]) for moisture content testing, and the results are provided in the attached STS Table A. Additional soil samples were returned to another NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, chloride and sulphate content and resistivity testing; the test results are summarised in the attached Envirolab Services Certificate of Analysis 236147.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The proposed ELC building ("the site") is located mid-slope on the northern flank of a gully, which graded down to the south-west at about 15°. The overall hillside on which the site is located also sloped down to the west towards Sydney Harbour. Vaucluse Road is located about 60m to the east of the site.

At the time of the fieldwork, the site was occupied by an elevated walkway structure and several garden beds. Some of the garden beds were supported by low height timber retaining walls and rock mattresses.

An asphaltic concrete (AC) surfaced internal road was located on the southern and eastern sides of the site; the AC surfacing was in good condition. Further to the south of the site, was a large fill batter slope that graded down to the south at about 15°; we infer the maximum height of the fill batter was about 10m. The surface of the fill batter was covered with sandstone cobbles and boulders.

To the north of the site, was a three-storey brick school building. Just to the south-west of the building was a sandstone cliff face up to about 3m high. The bedrock was distinctly weathered and generally of low to medium strength. No seepage was observed through, or over, the cliff face. At the crest of the cliff face was a dry-stacked sandstone block retaining wall up to about 1.5m high.

To the west of the site was a single storey brick building (Sophie's Cottage). On the south-eastern, south-western and north-western sides of the Cottage were several playground areas with the ground surface covered by Softfall.

Possible sandstone outcrops were visible within the playground areas, as well as on the southern flank of the gully. We also observed several detached sandstone blocks on the flanks of the gully, which were up to several metres in size.

The buildings surrounding the site which were all within the school grounds, all appeared to be in good external condition, based on a cursory inspection from within the site.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Hawkesbury Sandstone. The geological map also indicates an igneous dyke to pass through the proposed building footprint in an approximate east-south-east to west-north-west orientation. A dyke is a sub-vertical igneous intrusion through the sedimentary bedrock. The width of the dyke can range from centimetres to many metres and can extend laterally for very large distances.

The subsurface conditions within, and adjacent to, a dyke can be extremely variable. Where there is a dyke present, the bedrock in contact with the dyke can vary considerably in terms of its quality and depth, which is the case with the subject site.

There may also be other dykes present in close proximity to the dyke referred to above, and which may have formed the gully feature on which the proposed building is located. We infer that the gully most likely formed by preferential erosion of the weaker infill materials within the dyke, which generally comprise a residual clay, though no such material was encountered in this investigation. Further, the erosion processes can also result in infilling of the dyke with material from the host rock, or allow other materials to be deposited within the dyke by water, gravity or wind action.

In summary, the boreholes encountered AC pavements then moderate to deep fill overlying natural sandy soils (BH2 only) then sandstone bedrock (BH1 and BH3 only). Groundwater was encountered at depth in BH2 only. Reference should be made to the attached borehole logs for specific details at each location. Some of the characteristic features of the subsurface conditions encountered in the boreholes is provided below.

Pavement and Fill

The three boreholes were drilled through an AC surfacing which had a thickness of either 50mm (BH1 and BH3) or 90mm (BH2). The fill below the AC extended to depths of 1.75m (BH1), 4.2m (BH2) and 6.2m (BH3) below existing surface levels and comprised gravelly silty sand and silty sand, with inclusions of brick fragments, sandstone gravel, cobbles and boulders. The fill was assessed to be variably compacted, ranging from poorly to well compacted, which suggests the fill has not been placed and compacted in a controlled manner.

Natural Soils

Natural silty sand and gravelly sand was encountered below the fill in BH2 and extended to the borehole termination depth. BH2 was terminated within the natural soil profile.

Due to the suspected presence of the dyke and the appearance of the natural soils compared to the overlying fill, it is possible that the natural soils in BH2 may in fact be fill.

Sandstone Bedrock

Sandstone bedrock was encountered in BH1 and BH3 at depths of 1.75m and 6.2m, respectively.

In BH1, the sandstone bedrock was assessed to be distinctly weathered and of medium to high strength. The bedrock was only proven for a limited depth of 0.25m before 'TC' bit refusal occurred at 2.0m depth. Due to the limited penetration into the rock, and presence of boulders within the gully, it is possible that the bedrock in BH1 could be a detached sandstone boulder within the fill profile.

In BH3, the sandstone bedrock was assessed to be predominantly extremely and distinctly weathered and of hard (soil) and very low strength, but with medium to high strength iron indurated bands. 'TC' bit refusal occurred within the bedrock profile at 9.3m depth.

Groundwater

BH1 and BH3 were 'dry' during and on completion of drilling. In BH2, groundwater seepage was noted at 8.5m depth during drilling, with groundwater measured at 8.5m depth on completion of drilling.

On 3 February 2020, we returned to site and the groundwater level in BH2 had risen slightly to 8.0m depth. No long term groundwater level monitoring has been undertaken.

3.3 Laboratory Test Results

The results of the moisture content tests carried out on recovered rock cutting samples from BH1 and BH3 correlated well with our field assessment of bedrock strength.

The soil pH test results were 6.2, 9.0 and 9.9, which show the samples tested to be slightly acidic to alkaline. The soil sulphate and chloride content test results were less than 70mg/kg, which indicates low sulphate and chloride contents. The resistivity test results were relatively high (7,600 ohm.cm to 28,000 ohm.cm).

4 COMMENTS AND RECOMMENDATIONS

4.1 Additional Geotechnical Investigation

The comments and recommendations provided in this report are considered preliminary and based on three augered boreholes, only two of which encountered bedrock. Further, the bedrock in BH1 was only proven for a very limited depth and could be a detached boulder buried within the fill profile rather than bedrock.

There is also expected to be a dyke present at the site, and so the subsurface conditions at the site are likely to be inherently variable.

We strongly recommend that once the architectural drawings have been finalised, an additional geotechnical investigation including the drilling of cored boreholes be completed to confirm the depth to, and quality of, the underlying bedrock. The recommendations in this report will then need to be reviewed and probably updated following completion of the additional investigation. We can provide a fee proposal for this additional work, if requested to do so.

4.2 Site Preparation

Construction of the proposed ELC building will require demolition of the existing structures and pavements, removal of any trees and other vegetation, stripping of grass, topsoil and root affected soils from the proposed building footprint and the removal of any deleterious or contaminated fill. Reference should be made to the JKE report for guidance on the offsite disposal of site soils.

Care must be taken during site stripping and subsequent excavation not to undermine or remove support from any structures or landscaping on, or beyond, the footprint of the proposed ELC building.

Due to the presence of poorly compacted silty sand fill which most likely extends beyond the footprint of the proposed building, we recommend that tracking of hydraulic excavators or other tracked plant be carried out with caution. Sudden stop/start movements may result in ground vibration leading to settlement of these soils and possible damage to any surrounding structures not founded on bedrock.

4.3 Excavation and Temporary Batters

Prior to any excavation commencing, reference should be made to the Safe Work Australia 'Excavation Work Code of Practice' dated July 2015.

For the purpose of this report, we have assumed that bedrock will not be encountered within the maximum 2.0m depth of excavation.

Excavation of the soil profile can be completed using buckets on a tracked hydraulic excavator.

Should the additional recommended drilling show that bedrock will be encountered in the excavation, then further geotechnical advice on rock excavation and controlling vibrations will be provided as part of the additional geotechnical report.

Excavations through the soil profile may be temporarily battered no steeper than 1 Vertical in 1.5 Horizontal, provided all surcharge loads are kept well clear of the crest of these batters and any nearby school buildings are founded on bedrock. If there is any doubt as to the adjacent building footing details and foundation materials, then test pits should be excavated in the presence of a geotechnical engineer prior to excavation to assess whether any underpinning is required. If the site geometry does not allow sufficient space for batter slopes then a shoring system will be required; in these ground conditions a contiguous pile wall is generally preferred.

4.4 Drainage

Based on the investigation results, the depth of excavation required and noting that the site is located on a hillside well above Sydney Harbour, we do not expect that any significant groundwater seepage into the excavation will occur. However, if some minor seepage is experienced during and following rainfall periods, this should be able to be controlled using gravity drainage, or conventional sump and pump techniques.

4.5 Retaining Walls

For free-standing cantilever walls which are retaining areas where movement is of little concern (i.e. landscaped or grassed areas), a triangular lateral earth pressure distribution may be adopted using an 'active' earth pressure coefficient, K_a, of 0.35, assuming a horizontal backfill surface. For a propped cantilever wall or where the wall retains an area where only minor movements can be tolerated, a triangular lateral earth pressure distribution should be adopted using an 'at rest' earth pressure coefficient, K_o, of 0.6, assuming a horizontal backfill surface.

Appropriate surcharge loads must be taken into account in the design of the retaining walls, and the design should incorporate drainage measures to reduce any pore water pressures.

The retaining walls must be backfilled with either engineered fill placed, compacted and tested in thin layers, or with a single sized, hard and durable drainage gravel tamped into place in thin layers behind the wall. Where gravel backfill is used, a layer of non-woven geofabric should be placed between the soil and the gravel backfill, and this should then be wrapped over the top of the gravel backfill. A less permeable strata should then be placed over the geofabric to reduce the amount of surface water entering the retaining wall backfill; this could comprise either a pavement, or a layer of about 0.3m thickness of compacted clayey soil.

Lateral toe restraint may be achieved by the resistance of the ground in front of the walls. For embedment depth design, a triangular lateral earth pressure distribution should be assumed with a 'passive' lateral earth pressure coefficient (K_p) of 2.8, assuming horizontal ground in front of the walls. All localised excavations in front of the wall, such as for buried services, footings, etc, must be taken into account in the embedment depth design. We note that significant movement is required in order to mobilise the full passive pressure and so a Factor of Safety of at least 2.0 should be adopted in order to reduce such movement.

Due to the presence of moderate to deep fill and moderate ground slopes, structural retaining walls (as opposed to soft landscaping walls) should be supported by piled footings founded in the underlying sandstone bedrock. Allowable bearing pressures for piles are presented in Section 4.6 below.

4.6 Footings

4.6.1 Design

Following excavation, we expect that the proposed building footprint will generally be underlain by a moderate to deep uncontrolled fill profile, and based on the anticipated structural loads, the proposed building should therefore be uniformly supported by piled footings founded in the underlying sandstone or basalt (dyke) bedrock.

Due to the presence of sandy fill, groundwater at depth and nearby buildings, the proposed building should be supported by continuous flight auger (CFA) piles.

CFA piles founded a nominal 0.3m into sandstone bedrock should be tentatively designed for a maximum allowable end bearing pressure of 600kPa. We note that bedrock was only proven in BH1 and BH3, and in BH1, the bedrock was only proven for a very limited depth. At this stage, and subject to completion of the additional geotechnical investigation, we recommend that the piles be designed in end bearing only unless they are socketed into the rock. Higher bearing pressures may be feasible following completion of the additional geotechnical investigation, however, the presence of the dyke, may limit the bearing pressures, due to the variability of the sandstone bedrock quality and whether any weathered dyke materials are present at depth.

Due to the expected variability of the sandstone bedrock depth and quality and possible buried obstructions/boulders within the fill, we recommend that all pile drilling be witnessed by a geotechnical engineer to check that the pile embedment depths are consistent with the borehole results.

We assume that the piling contractor would be responsible for certifying the load capacity of the piles.

4.6.2 Earthquake Design Parameters

A Hazard Factor (Z) of 0.09 and a Site Subsoil Class De should be tentatively adopted for earthquake design in accordance with AS1170.4-2007 ‘Structural Design Actions, Part 4: Earthquake Actions in Australia’, including Amendment Nos 1 & 2.

The earthquake design parameters must be confirmed following completion of the additional geotechnical investigation.

4.6.3 Soil Aggression

Based on the soil aggression test results, concrete and steel elements in contact with the soil should be designed for 'mild' and 'non-aggressive' exposure classifications, respectively, in accordance with AS2159-2009 'Piling-Design and Installation.

If basaltic rock is found in future boreholes then additional testing should be completed.

4.7 Level -03 Floor Slab Construction

Due to the sloping site and expected presence of moderate to deep fill, we recommend that the entire Level -03 floor slab be designed as suspended to reduce the potential for differential movements occurring.

4.8 Hydrogeology

Based on the investigation results, we would only expect very limited seepage into the excavation, if any, during or following periods of heavy rainfall. Further, the base of the excavation will almost certainly be well above the groundwater level.

Any settlements associated with draining any minor seepage from the existing fill into any backfill behind the proposed retaining walls would be expected to be extremely small and immeasurable.

Based on our hydrogeological assessment, we consider design and construction of a 'drained' structure to be appropriate for the proposed development, with tanking being unnecessary.

In view of the above, the proposed development should not adversely affect the existing groundwater flows to the extent that there will be any noticeable impact on the surrounding structures, provided the recommendations presented in this report are adopted in their entirety.

4.9 Further Geotechnical Input

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

- Additional geotechnical investigation comprising the drilling of cored boreholes to confirm the depth to, and quality of, the sandstone bedrock.
- Test pits to expose adjacent school building footings ad their foundation materials, if appropriate.
- Review of structural drawings for consistency with the geotechnical reports.
- Witnessing of CFA pile installations.

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

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

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

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A

MOISTURE CONTENT TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32915PH
Project:	Proposed Developments at Kincoppal - Rose Bay School	**Report:**	A
Location:	Cnr New South Head Road & Vaucluse Road, Vaucluse, NSW	**Report Date:**	14/02/2020
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AS 1289	**TEST METHOD**	**2.1.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
1	1.80 - 2.00	3.5
3	7.00 - 8.00	8.0
3	9.10 - 9.30	3.5
4	2.60 - 2.80	5.9
4	3.00 - 3.20	4.9
5	1.80 - 2.40	6.0

Notes:
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 13/02/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

14/02/2020
Authorised Signature / Date
(D. Treweek)

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CERTIFICATE OF ANALYSIS 236147

Client Details

Client	JK Geotechnics
Attention	David Fisher, Adrian Hulskamp
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32915PH, Vaucluse**
Number of Samples	9 Soil
Date samples received	06/02/2020
Date completed instructions received	06/02/2020

Analysis Details

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

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

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

Report Details

Date results requested by	13/02/2020
Date of Issue	13/02/2020

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		236147-1	236147-2	236147-3	236147-4	236147-5
Your Reference	UNITS	BH1	BH2	BH2	BH6	BH6
Depth		1.5-1.5	2.8-3.0	8.7-8.9	0.4-0.5	0.0-0.1
Date Sampled		28/01/2020	28/01/2020	28/01/2020	03/02/2020	03/02/2020
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	11/02/2020	11/02/2020	11/02/2020	11/02/2020	11/02/2020
Date analysed	-	11/02/2020	11/02/2020	11/02/2020	11/02/2020	11/02/2020
pH 1:5 soil:water	pH Units	9.9	9.0	6.2	7.0	6.3
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	<10	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	69	48	42	20	32
Resistivity in soil*	ohm m	76	85	280	270	40

Misc Inorg - Soil					
Our Reference		236147-6	236147-7	236147-8	236147-9
Your Reference	UNITS	BH7	BH8	BH9	BH10
Depth		0.2-0.3	0.2-0.4	0.6-0.7	0.2-0.4
Date Sampled		03/02/2020	03/02/2020	03/02/2020	03/02/2020
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	11/02/2020	11/02/2020	11/02/2020	11/02/2020
Date analysed	-	11/02/2020	11/02/2020	11/02/2020	11/02/2020
pH 1:5 soil:water	pH Units	7.1	6.5	6.5	6.8
Chloride, Cl 1:5 soil:water	mg/kg	36	<10	29	30
Sulphate, SO4 1:5 soil:water	mg/kg	20	<10	21	20
Resistivity in soil*	ohm m	150	310	200	140

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	236147-6
Date prepared	-			11/02/2020	3	11/02/2020	11/02/2020		11/02/2020	11/02/2020
Date analysed	-			11/02/2020	3	11/02/2020	11/02/2020		11/02/2020	11/02/2020
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	3	6.2	6.4	3	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	<10	<10	0	91	95
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	42	52	21	106	110
Resistivity in soil*	ohm m	1	Inorg-002	<1	3	280	240	15	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 1

PROPOSED ELC BUILDING

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915PH1 **Method:** SPIRAL AUGER **R.L. Surface:** 35.1 m
Date: 28/1/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION		35			ASPHALTIC CONCRETE: 50mm.t				
				-	FILL: Gravelly silty sand, fine to coarse grained, light grey, fine to medium grained igneous gravel.	M			APPEARS MODERATELY COMPACTED
	N = 11 5,6,5	34	1		FILL: Silty sand, fine to coarse grained, light brown, trace of clay and fine to medium grained sandstone gravel.				
	N > 3 2,3/ 100mm REFUSAL								
				-	SANDSTONE: fine to coarse grained, light grey and red brown.	DW	M - H		HAWKESBURY SANDSTONE
		33	2		END OF BOREHOLE AT 2.00 m				HIGH 'TC' BIT RESISTANCE
									'TC' BIT REFUSAL
		32	3						
		31	4						
		30	5						
		29	6						

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32915PH1 VAUCLUSE.GPJ <<DrawingFile>> 25/02/2020 11:45 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

PROPOSED ELC BUILDING

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915PH1 | **Method:** SPIRAL AUGER | **R.L. Surface:** 38.6 m
Date: 28/1/20 | **Datum:** AHD
Plant Type: JK205 | **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				0			ASPHALTIC CONCRETE: 90mm.t				
						-	FILL: Silty sand, fine to coarse grained, brown, with brick fragments, trace of sandstone gravel and clay.	M			APPEARS POORLY COMPACTED
		N = 4 2,2,2	38								
				1							
		N = 7 4,4,3	37								
				2							
			36								
		N = 4 6,2,2		3							
			35								
				4							
						SM	Silty SAND: fine to coarse grained, light orange brown, with clay, trace of fine to coarse grained ironstone gravel.	M	L		CONTINUAL SPIRAL AUGER DRILLING (i.e. NO INSITU TESTING) BELOW 4.95m IN ORDER TO ATTEMPT TO PROVE BEDROCK
		N = 8 4,5,3	34								
				5							
			33								
				6							
			32								

JKGeotechnics

BOREHOLE LOG

Borehole No.
2

PROPOSED ELC BUILDING

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915PH1 **Method:** SPIRAL AUGER **R.L. Surface:** 38.6 m
Date: 28/1/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON 3/2/20						31	8		SM	Silty SAND: fine to coarse grained, light orange brown, with clay, trace of fine to coarse grained ironstone gravel.	M	L		
ON COMPLETION OF AUGERING	■			■		30	9		SW	Gravelly SAND: fine to coarse grained, light orange brown, fine to coarse grained ironstone gravel, with clay .	W			
						29, 28, 27, 26, 25	10, 11, 12, 13			END OF BOREHOLE AT 9.20 m				GROUNDWATER MONITORING WELL INSTALLED TO 9.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.2m TO 9.2m. CASING 3.2m TO 0.2m. 2mm SAND FILTER PACK 2.8m TO 9.2m. BENTONITE SEAL 2.4m TO 2.8m. BACKFILLED WITH CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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BOREHOLE LOG

Borehole No.
3

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915PH1 **Method:** SPIRAL AUGER **R.L. Surface:** 40.6 m
Date: 28/1/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			0	-	ASPHALTIC CONCRETE: 50mm.t				
					FILL: Silty sand, fine grained, brown, trace of clay.	M			APPEARS WELL COMPACTED
					FILL: Silty sand, fine to medium grained, brown and light brown.				
	N = 21 10,11,10	40			FILL: Silty sand, fine to coarse grained, brown and dark brown, trace of igneous gravel and brick fragments.				
	N = 6 5,4,2	39	1–2						APPEARS POORLY COMPACTED
	N = 15 7,4,11	38–37	3						APPEARS WELL COMPACTED
		36–35	4–6		as above, but with sandstone boulders and cobbles.				BANDS OF LOW 'TC' BIT RESISTANCE
	Nc = 12 15 11 /100mm REFUSAL	34		-	SANDSTONE: fine to coarse grained, light grey, grey and red brown, with occasional medium strength bands.	XW - DW	Hd - VL		HAWKESBURY SANDSTONE VERY LOW RESISTANCE WITH MODERATE BANDS

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
2 / 2

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915PH1 **Method:** SPIRAL AUGER **R.L. Surface:** 40.6 m
Date: 28/1/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						33, 32	8, 9		-	SANDSTONE: fine to coarse grained, light grey and grey, with occasional medium strength iron indurated bands.	XW - DW	Hd - VL		VERY LOW RESISTANCE WITH MODERATE BANDS
											DW	M - H		HIGH RESISTANCE
						31, 30, 29, 28, 27	10, 11, 12, 13			END OF BOREHOLE AT 9.30 m				'TC' BIT REFUSAL

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title:	**SITE LOCATION PLAN**
Location:	CNR NEW SOUTH HEAD ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW
Report No: 32915PH1	Figure: 1

JKGeotechnics

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

PLOT DATE: 18/02/2020 1:47:11 PM DWG FILE: Y:\32000'S\32915PH VAUCLUSE\CAD\32915PH1.DWG

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
Location:	CNR NEW SOUTH HEAD ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW
Report No:	32915PH1
Figure:	2

JKGeotechnics

PLOT DATE: 18/02/2020 1:49:18 PM DWG FILE: Y:\32000'S\32915PH VAUCLUSE\CAD\32915PH1.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 <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:

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

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

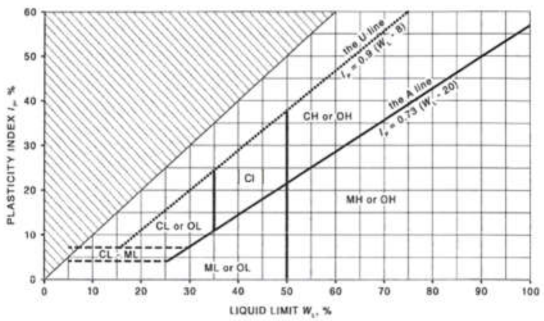
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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