REPORT TO

KINCOPPAL - ROSE BAY SCHOOL

ON

GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

FOR

PROPOSED BUS PARKING

AT

CNR NEW SOUTH ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW

Date: 28 February 2020
Ref: 32915SH3rpt

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DOCUMENT REVISION RECORD

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Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical and hydrogeological investigation for the proposed bus parking 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 approximately 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 geotechnical and hydrogeological investigations for two other proposed developments at the school, both of which are at a development application stage. The fieldwork for those investigations was carried out concurrently with the fieldwork for this project. The investigation results for the other two projects are presented in separate reports, Ref. 32915PH1rpt, and Ref. 32915PH2rpt.

From the supplied State Significant Development Application document prepared by BVN Architecture dated 13 November 2019, an email sent on 9 December 2019 by Mr Mahady and our discussions with Mr Mahady during the fieldwork, we understand that a new bus park is proposed over the eastern portion of the development footprint, which will have roof parking for cars above. The western portion of the footprint will incorporate the existing on-grade car parking. The surface levels of the proposed bus parking areas have not been indicated, but Mr Mahady indicated at the proposal stage, that excavation to a maximum depth of about 1m might be required only within the far north-eastern corner of the development footprint; elsewhere we have assumed that the lowest level of parking will be at, or close to, existing surface levels, though some minor filling might be required to raise surface levels where they fall away to the south-west. The approximate location of the proposed development footprint 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 comments and recommendations on excavation, retaining walls, drainage, footings, soil aggression, external pavements 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 completed on 3 February 2020, and comprised the hand auger drilling of three boreholes (BH8, BH9 and BH10) to depths of 1.8m, 0.9m and 0.45m, respectively, below existing surface levels. Dynamic Cone Penetration (DCP) tests were carried out at each borehole to refusal depths of 1.73m (BH8), 0.85m (BH9) and 0.48 (BH10).

BH8 and BH9 were extended into the underlying bedrock by diamond core drilling using TT56 coring techniques with our portable Melvelle coring equipment, to final depths of 5.95m and 4.70m, respectively, below existing surface levels.

In consultation with Mr Mahady at the commencement of the fieldwork, the borehole locations were positioned within the eastern portion of the proposed development footprint, where the two storey portion of the parking area is proposed. The borehole locations, which were set out using tape measurements off existing surface features, are shown on the attached Figure 2, which is based on a recent Nearmap image of the site. We were not provided with a survey plan of the site, so the surface RL of each borehole was not established, and could not be obtained using a GPS due to the presence of trees limiting the satellite signal.

The relative compaction and density of the soil profile were assessed from the Standard Penetration Test (SPT) results, as well as interpretation of the DCP test results. The strength of the bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results. Groundwater observations were made in the boreholes during and on completion of drilling

The borehole logs and DCP test results sheet are attached, together with a set of explanatory notes, which describe the investigation techniques (and their limitations) and define the logging terms and symbols used.

Our geotechnical engineer was present full time during the fieldwork to set out the borehole locations, nominate the testing and sampling, and prepare the attached borehole logs and DCP test results sheet.

Selected soil samples were returned to our NATA accredited laboratory (Soil Test Services Pty Ltd [STS]) for four day soaked CBR testing, and the results are provided in the attached STS Table B. 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.

The recovered rock cores were photographed and returned to STS, for Point Load Strength Index testing. The rock core photographs are enclosed with the borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and are summarised in the attached STS Table C. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in STS Table C.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The approximate location of the proposed parking area (“the site”) is mid-slope on a south-west facing hillside, at the south-eastern corner of KRB. The hillside slopes were in the order of about 10° to 15° down to the south-west. New South Head Road and Vaucluse Road were located uphill to the east and north of the site, respectively. A four to five storey KRB school building was located just to the north of the site and appeared to be in good external condition, based on a cursory inspection from within the site.

At the time of the fieldwork, the eastern portion of the site was mostly undeveloped and covered by grass, mulch, garden beds, a concrete driveway, concrete footpaths and scattered medium to large trees. The concrete pavements were generally in poor condition, with cracking and shallow subsidence observed. The area had been filled and terraced by several sandstone block retaining walls that were up to about 2m high; the retaining walls appeared to be in reasonably good condition. The western portion of the site had also been raised by filling and contained an on-grade car park, the concrete surface of which was in generally good condition. The southern side of the car park was supported by concrete block and sandstone block retaining walls, which ranged in height from about 1.5m at the eastern end to about 5m adjacent to the western end of the proposed footprint. Only the eastern portion of the walls were visible from within the site, and these generally appeared to be in good condition. The western portion of the wall could not be inspected, due to a locked gate within the site which prevented access.

Sandstone bedrock was exposed within a partially concrete lined dish drain along the toe of the sandstone block retaining wall which supported the existing car parking; at the eastern end of the wall, the bedrock was assessed to be of at least medium strength. We did not observe any seepage emanating over the bedrock surface.

The footpath which ran along the western side of New South Head Road adjacent to the site, was supported above the site by a concrete retaining wall approximately 1.5m high. Several concrete buttresses were present to provide additional lateral wall restraint. A sandstone block wall ran along the western side of the footpath, above the retaining wall.

Ground surface levels beyond the southern boundary of KRB, immediately adjacent to the site, stepped down several metres to the south-west. Due to dense vegetation cover along the boundary it was not possible to ascertain whether there was a sandstone cliff face below the boundary, or a retaining wall. However, due to the presence of outcropping sandstone bedrock along the base of the sandstone block retaining wall described above, it is likely that a sandstone cliff face is present. Two neighbouring brick and cement rendered brick buildings were located just beyond the toe of the step down to the south of 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.

In summary, the boreholes encountered silty sand fill overlying a thin layer of residual clayey sand (BH8 only) then sandstone bedrock at shallow and moderate depths. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. Some of the characteristic features of the subsurface conditions encountered in the boreholes is provided below.

Fill

Silty sand fill was encountered from the surface of each borehole and extended to depths ranging from 0.45m (BH10) to 1.6m (BH8) below existing surface levels. Deeper fill should be expected behind some of the retaining walls. Inclusions of ironstone and sandstone gravel, concrete fragments and slag were present within the fill. The fill was assessed to be mostly poorly compacted, which suggests the fill has not been placed and compacted in a controlled manner.

Residual Clayey Sand

A thin layer (0.2m) of residual clayey sand of very loose relative density was encountered below the fill in BH8.

Sandstone Bedrock

Sandstone bedrock was encountered in BH8 and BH9 at depths of about 2.1m and 0.9m, respectively. In BH10 and DCP10, sandstone bedrock was inferred at about 0.45m depth, based on their refusal depths.

The sandstone bedrock was assessed to range from distinctly to slightly weathered and fresh and was generally of medium strength, with the exception of the upper metre in BH9 which was of high strength. Very few defects were encountered in the bedrock and these comprised sub-horizontal bedding partings. 'No core' zones occurred in BH8 at depths of 1.8m (320mm) and 3.2m (150mm thick) and in BH9 at a depth of 3.8m (130mm thick); these zones are most likely extremely weathered bands or clay bands which have been washed out by the drill flush water.

Groundwater

BH9 and BH10 were 'dry' during and on the completion of hand augering. In BH8, groundwater was encountered on completion of hand augering at 1.8m depth, just above the soil/bedrock interface. The introduction of water to flush the boreholes during the coring process prevented further useful assessment of the groundwater conditions in BH8 and BH9 and at the time of the fieldwork. No long term monitoring of the groundwater levels was undertaken.

3.3 Laboratory Test Results

The soaked CBR tests on samples from BH8 and BH10 returned results of 17% and 10%, respectively, which suggest that a relatively good subgrade is present.

The soil pH test results were just below 7, which show the samples tested from BH8, BH9 and BH10 to be slightly acidic. The soil sulphate and chloride content test results were less than 30mg/kg, which indicates low sulphate and chloride contents. The resistivity test results were high (14,000 ohm.cm to 31,000 ohm.cm).

4 COMMENTS AND RECOMMENDATIONS

4.1 Additional Geotechnical Investigation

At the time of preparing this report, the exact details of the proposed development had yet to be finalised and our boreholes were positioned over the eastern portion of the development footprint. The comments and recommendations provided in this report are therefore considered preliminary.

We strongly recommend that once the architectural drawings have been finalised, a supplementary geotechnical investigation including the drilling of at least two additional cored boreholes be completed to confirm the depth to, and quality of, the sandstone bedrock across the development footprint. The supplementary investigation should also include a geotechnical inspection of the southern KRB boundary from within the neighbouring properties to the south to assess the details across the boundary. The recommendations in this report will then need to be reviewed and probably updated following completion of the supplementary investigation. We can provide a fee proposal for this additional work, if requested to do so.

Prior to any works on site, we also recommend that a structural engineer be engaged to inspect any existing retaining walls that are to be incorporated into the development to assess their integrity and life expectancy and to advise whether any strengthening of the walls are required.

4.2 Site Preparation

Construction of the proposed bus parking structure will require some demolition of the existing pavements and retaining walls, removal of trees and other vegetation, stripping of grass, topsoil and root affected soils from the proposed 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 boundary structures or retaining walls within the site that are to remain.

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 1m depth of excavation and that only fill will be encountered. Excavation of the fill profile can be completed using buckets on a tracked hydraulic excavator.

Following review of the architectural drawings and after completion of the recommended additional drilling, should sandstone bedrock be expected within 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 fill 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 retaining walls are founded on bedrock. If there is any doubt as to the adjacent retaining wall 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 or propping is required.

4.4 Drainage

Based on the investigation results, the limited depth of excavation required and noting that the site is located on a hillside, 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.

For lateral toe restraint and assuming bedrock will be encountered at relatively shallow depth below the base of the excavation, any structural retaining wall footings (as opposed to soft landscaping walls) can be keyed into the bedrock, provided the footing is well clear of any cliff lines or steps in the rock surface below. An allowable lateral stress of 200kPa may be adopted for key design. Where the retaining wall comprises concrete poured directly onto a clean and rough rock surface, a friction of 35° could be adopted in the design.

4.6 Footings

4.6.1 Design

Due to the presence of sandstone bedrock at shallow to moderate depth across most of the site and the anticipated structural loads, the proposed two storey bus parking structure should be uniformly supported by footings founded in the underlying sandstone bedrock.

Pad footings will be suitable where the depth to the bedrock is relatively shallow. Where the rock is deeper than about 1m, piled footings would be more appropriate. Due to the presence of sandy soils and the type of structure proposed, our preference would be to use continuous flight auger (CFA) piles, particularly if some of the piles are located just behind an existing retaining wall that is to remain. Depending also on where the piles are located within the site, it may also be feasible to install bored piles, but with an allowance for casing/liners.

Pad footings and piles founded in the underlying sandstone bedrock of at least low strength should be designed for a maximum allowable end bearing pressure of 1,000kPa. Subject to completion of the supplementary geotechnical investigation, the maximum allowable bearing pressure may be increased to 3,000kPa, provided all footings/piles are founded in at least medium strength/Class IV sandstone bedrock and all footing excavations and piles are inspected by a geotechnical engineer and certified by the piling contractor.

For piles, sockets formed below a minimum 0.3m length requirement in at least low strength rock may be designed for allowable shaft adhesion values of 100kPa in compression and 50kPa in tension, on condition that the pile shaft is suitably roughened. These values may increase following completion of the additional drilling. Due to the presence of medium and high strength sandstone, the design of long rock socket lengths should be avoided due to the expected difficulty in penetrating the sandstone. The provided pressures are based upon serviceability criteria of deflections at the pile toe of less than 1% of the pile diameter.

All footings/piles must be founded behind a line inclined up from the toe of any cliff face or retaining wall at 45°.

Subject to completion of the supplementary investigation and depending on the position of the southern KRB boundary relatively to the step down in the bedrock, it may be possible to position footings/piles closer to the suspected cliff face located along the southern KRB boundary. However, a tentative reduced bearing pressure of 600kPa would apply for any footings located just behind the crest of the cliff face and a geotechnical engineer would need to inspect the cliff face to identify any adverse defects or overhangs that may require stabilisation or underpinning.

All pad footings should be excavated, cleaned out, dewatered, inspected, and poured with minimal delay. All bored piles should be cleaned out, inspected and poured on the same day as drilling

4.6.2 Earthquake Design Parameters

A Hazard Factor (Z) of 0.09 and a Site Subsoil Class Ce (due to the presence of deep fill behind some of the existing retaining walls) should be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia', including Amendment Nos 1 & 2.

4.6.3 Soil Aggression

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

4.7 Bus Parking Lowest Slab Level Construction

Due to the sloping site and presence of poorly compacted fill to variable depths, we recommend that the entire lowest bus parking level be designed as suspended to reduce the potential for differential movements occurring.

Where ground surface levels need to be raised, we recommend adopting general site filling procedures. The resulting fill would not be suitable to support footings or for structural support, however, the fill can be used for temporary 'formwork' support for suspended slab construction.

After demolition of any retaining walls or concrete pavements and removal of any vegetation and topsoil, the general fill should be placed in loose layers of say 150mm thickness, with 'nominal' compaction using the tracks of an excavator, or with a trench roller. The excavated fill could be used for general fill, provided the material is free of organic matter and contains a maximum particle size not exceeding 75mm.

4.8 On-Grade Pavements

4.8.1 Subgrade Preparation and Engineered Fill

The advice above in Section 4.2 applies. Excavation of the soil profile down to the design subgrade level can be completed using buckets on a tracked hydraulic excavator.

Where a rock subgrade is exposed, it must be ripped to a depth of 0.3m and recompacted to at least 98% of Standard Maximum Dry Density (SMDD) to allow for drainage below the pavement. Should the bedrock be of sufficient strength that a rock hammer is required for excavation, then further geotechnical advice on controlling vibrations must be sought.

We recommend that all existing fill be stripped below the footprint of ay proposed on-grade pavements and recompacted as engineered fill. The fill must be free from organic matter and any particles greater than 75mm.

Following stripping of the existing fill, and a natural soil subgrade is exposed, the soils should be proof rolled with at least six passes of a static smooth drum roller of at least 12 tonnes deadweight. The final passes of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of any 'unstable' areas. Care must be taken to not surcharge any existing retaining walls with the roller.

Subgrade heaving during proof rolling should be expected in areas where the subgrade has become 'saturated'. The heaving areas can typically be improved by locally removing the heaving material down to a stable base and replacing with engineered fill, as outlined below.

Where site levels need to be raised, engineered fill must be used.

Engineered fill should comprise an imported select, well graded, granular material such as crushed or processed sandstone with a CBR value of 10% or more, and should be compacted in maximum 200mm thick loose layers using a large static roller to achieve a density ratio of at least 98% of SMDD. If lighter compaction plant is proposed, then thinner layers will be required and further geotechnical advice should be sought in this regard.

Density tests should be carried out on each layer of engineered fill at a frequency meeting or exceeding that defined in AS3798-2007 ''Guidelines on earthworks for commercial and residential developments". At least Level 2 control of fill compaction in accordance with AS3798-2007 should be carried out. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by the KRB or their representative and not by the contractor.

4.8.2 Design

Based on the investigation results, we recommend that any proposed on-grade pavements be designed on the basis of a CBR value of 10%, provided that the subgrade is prepared as per our advice above.

Assuming a flexible pavement is constructed, all unbound granular base materials should comprise DGB20 in accordance with RMS QA Specification 3051. The DGB20 material should be compacted in maximum 200mm thick loose layers using a smooth drum roller to at least 98% of Modified Maximum Dry Density (MMDD). All unbound granular sub-base materials should comprise DGS40, DGS20 or DGB20 in accordance with RMS QA Specification 3051. The sub-base material should be compacted in maximum 200mm thick loose layers using a smooth drum roller to at least 95% of Modified Maximum Dry Density (MMDD). For both the base and sub-base layers, adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement so as to reduce the potential for material breakdown during compaction.

Density tests should be carried out on the granular pavement materials at a frequency meeting or exceeding that defined in AS3798-2007, but with a minimum of at least six density tests to be completed on the basecourse and sub-base layers (ie. a minimum of 12 tests in total). Due to a potential conflict of interest, the GTA should be directly engaged by KRB or their representative and not by the contractor.

4.8.3 Subsoil Drains

A subsoil drain should be provided below the upslope edges of the proposed pavement with invert levels at least 200mm below design subgrade level. The drainage trenches should be excavated following the compaction and density testing of base and sub-base materials, with a uniform longitudinal fall to appropriate discharge points, so as to reduce the likelihood of water ponding. Discharge from the subsoil drains should be piped to the stormwater system for disposal.

4.9 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.

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.

Vaucluse Road and New South Head Road are located directly uphill to the north and east of the site, respectively, and therefore may intercept any existing intermittent groundwater seepage over the bedrock surface, but service trenches and the like may bring in other seepage flows.

Noting the limited depth and extent of excavation required, including that for footings to support the proposed structure, in our opinion, the proposed development should not adversely affect the existing transient groundwater seepage flows in and around the site, provided the recommendations presented in this report are adopted in their entirety.

4.10 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:

- Supplementary geotechnical investigation comprising the drilling of additional cored boreholes to confirm the depth to, and quality of, the sandstone bedrock over the western portion of the proposed parking area and further inspection of the suspected cliff face along the southern portion of the site.
- Review of structural drawings for consistency with the geotechnical reports.
- Advice on controlling vibrations if rock hammers are used.
- Footing/pile inspections.

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 B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32915PH
Project:	Proposed Developments at Kincoppal - Rose Bay School	**Report:**	B
		Report Date:	17/02/2020
Location:	Cnr New South Head Road & Vaucluse Road, Vaucluse, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	BH 6	BH 8	BH 10
DEPTH (m)	0.00 - 0.20	0.20 - 0.40	0.20 - 0.40
Surcharge (kg)	9.0	9.0	9.0
Maximum Dry Density (t/m^3)	1.69 STD	1.81 STD	1.80 STD
Optimum Moisture Content (%)	16.5	14.7	14.6
Moulded Dry Density (t/m^3)	1.66	1.77	1.77
Sample Density Ratio (%)	98	98	98
Sample Moisture Ratio (%)	99	101	101
Moisture Contents			
Insitu (%)	14.2	20.4	15.0
Moulded (%)	16.4	14.9	14.7
After soaking and			
After Test, Top 30mm(%)	19.9	17.4	18.8
Remaining Depth (%)	17.9	16.1	16.9
Material Retained on 19mm Sieve (%)	0	0	5*
Swell (%)	0.0	0.0	0.0
C.B.R. value:			
@5.0mm penetration	12	17	10

NOTES: Sampled and supplied by client. Samples tested as received.

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 06/02/2020.
- * Denotes not used in test sample.

17/02/2020

NATA Accredited Laboratory Number:1327

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Authorised Signature / Date
(T. Finnegan)

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SOIL TEST SERVICES

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32915PH
Project:	Proposed Developments at Kincoppal - Rose Bay School	**Report:**	C
		Report Date:	6/02/2020
Location:	Cnr New South Head Road & Vaucluse Road, Vaucluse, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
8	2.42 - 2.46	0.5	10
	2.89 - 2.92	0.8	16
	3.50 - 3.54	0.6	12
	3.89 - 3.93	0.9	18
	4.29 - 4.33	1.0	20
	4.77 - 4.81	1.0	20
	5.20 - 5.23	0.8	16
	5.68 - 5.72	1.0	20
9	1.11 - 1.15	1.2	24
	1.65 - 1.69	1.1	22
	2.10 - 2.14	0.7	14
	2.66 - 2.70	0.9	18
	3.42 - 3.46	0.6	12
	4.17 - 4.21	0.8	16
	4.66 - 4.70	0.8	16

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

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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.

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BOREHOLE LOG

Borehole No.
8

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.: 32915PH3 **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

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
		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, brown, trace of roots and root fibres.	M			GRASS COVER APPEARS MODERATELY COMPACTED
			1			as above, but grey and light brown, trace of fine to medium grained ironstone gravel, concrete fragments and slag.				APPEARS POORLY COMPACTED
ON COMPELTION OF AUGERING					SC	Clayey SAND: fine to coarse grained, light brown.	W	VL		RESIDUAL
			2			REFER TO CORED BOREHOLE LOG				
			3							
			4							
			5							
			6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32915PH3 VAUCLUSE.GPJ <<DrawingFile>> 25/02/2020 12:42 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

CORED BOREHOLE LOG

Borehole No.
8

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

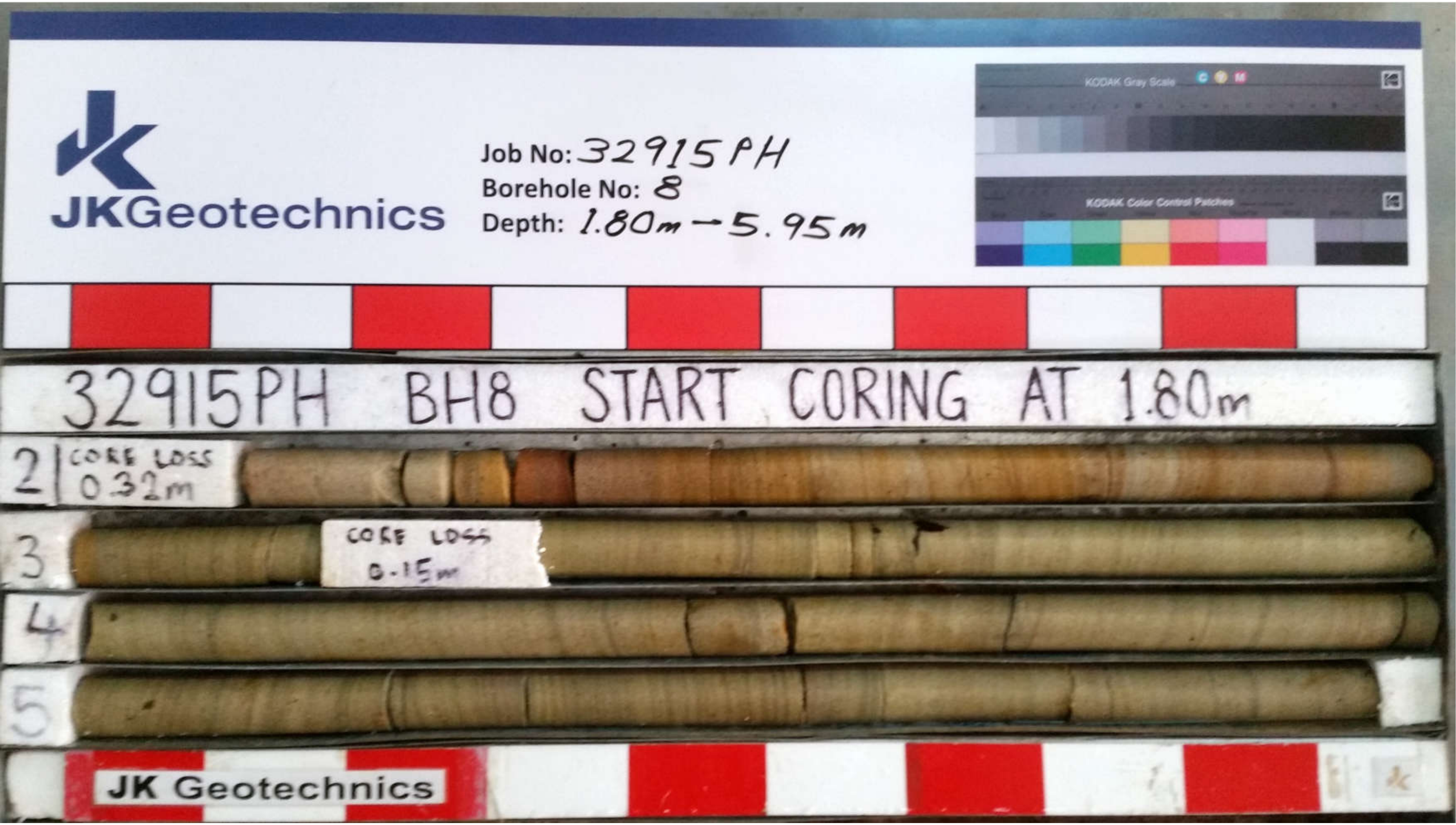
Job No.: 32915PH3 **Core Size:** TT56 **R.L. Surface:** N/A
Date: 3/2/20 **Inclination:** VERTICAL **Datum:**
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** D.A.F./A.J.H.

Water Loss\Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	SPACING (mm)	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
				START CORING AT 1.80m						
100% RETURN		2		NO CORE 0.32m						
				SANDSTONE: fine to coarse grained, light grey, orange brown and red brown, bedded at 0-15°.	DW	M	0.50, 0.80		(2.36m) Be, 0°, Fe Sn	Hawkesbury Sandstone
		3		as above, but light grey and grey.	FR					
				NO CORE 0.15m						
		4, 5		SANDSTONE: fine to coarse grained, light grey. with grey laminae, bedded at 0-20°.	FR	M	0.60, 0.90, 1.0, 1.0, 0.80, 1.0			Hawkesbury Sandstone
		6		END OF BOREHOLE AT 5.95 m						
		7								

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing scale: 600, 200, 60, 20

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32915PH3 VAUCLUSE.GPJ <<DrawingFile>> 25/02/2020 12:42 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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 32915PH
Borehole No: 8
Depth: 1.80m → 5.95m
KODAK Gray Scale
KODAK Color Control Patches
32915PH BH8 START CORING AT 1.80m
2
CORE LOSS
0.32m
3
CORE LOSS
0.15m
4
5
JK Geotechnics

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BOREHOLE LOG

Borehole No.
9

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.: 32915PH3 **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

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 OF AUGERING					REFER TO DCP TEST RESULTS				FILL: Silty sand, fine to coarse grained, brown and dark brown, trace of fine to medium grained sandstone gravel, roots and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
									FILL: Silty sand, fine to coarse grained, light orange brown and brown, with fine to medium grained sandstone gravel and clay.				
						1			REFER TO CORED BOREHOLE LOG				
						2							
						3							
						4							
						5							
						6							

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CORED BOREHOLE LOG

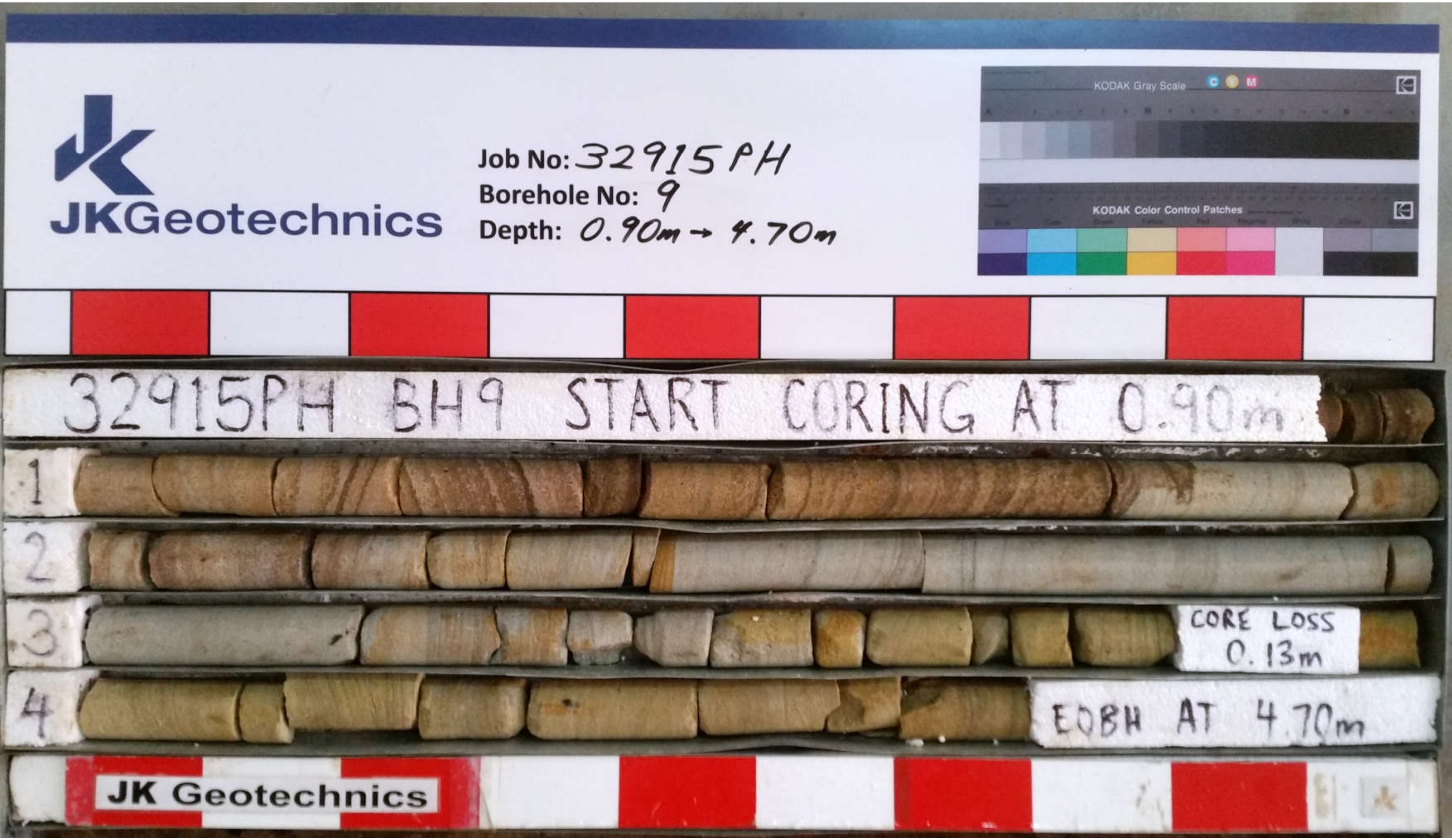
Borehole No.
9
2 / 2

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.: 32915PH3 **Core Size:** TT56 **R.L. Surface:** N/A
Date: 3/2/20 **Inclination:** VERTICAL **Datum:**
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** D.A.F./A.J.H.

Water Loss\Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) 600 200 60 20	DEFECT DETAILS — DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
				START CORING AT 0.90m						
100% RETURN		1		SANDSTONE: fine to coarse grained, red brown and orange brown, bedded at 10-20°.	DW	H	1.2 1.1		(1.40m) Be, 10°, Fe Sn	Hawkesbury Sandstone
		2			SW - FR	M	0.70		(2.42m) Be, 10°, Fe Sn	
		3		SANDSTONE: fe to coarse grained, light grey, with occasional grey laminae, bedded at 5-15°.			0.90 0.60			
				NO CORE 0.13m						
		4		SANDSTONE: fine to coarse grained, light grey, with occasional grey laminae, bedded at 5-15°.	FR	M	0.80 0.80			Hawkesbury Sandstone
				END OF BOREHOLE AT 4.70 m						
		5								
		6								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 32915PH
Borehole No: 9
Depth: 0.90m → 4.70m
KODAK Gray Scale
KODAK Color Control Patches
32915PH BH9 START CORING AT 0.90m
1
2
3
CORE LOSS 0.13m
4
EOBH AT 4.70m
JK Geotechnics

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BOREHOLE LOG

Borehole No.
10
1 / 1

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.: 32915PH3 **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS				FILL: Silty sand, fine to coarse grained, dark brown, trace of fine to coarse grained sandstone gravel.	M			APPEARS POORLY COMPACTED
									END OF BOREHOLE AT 0.45 m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						1							
						2							
						3							
						4							
						5							
						6							

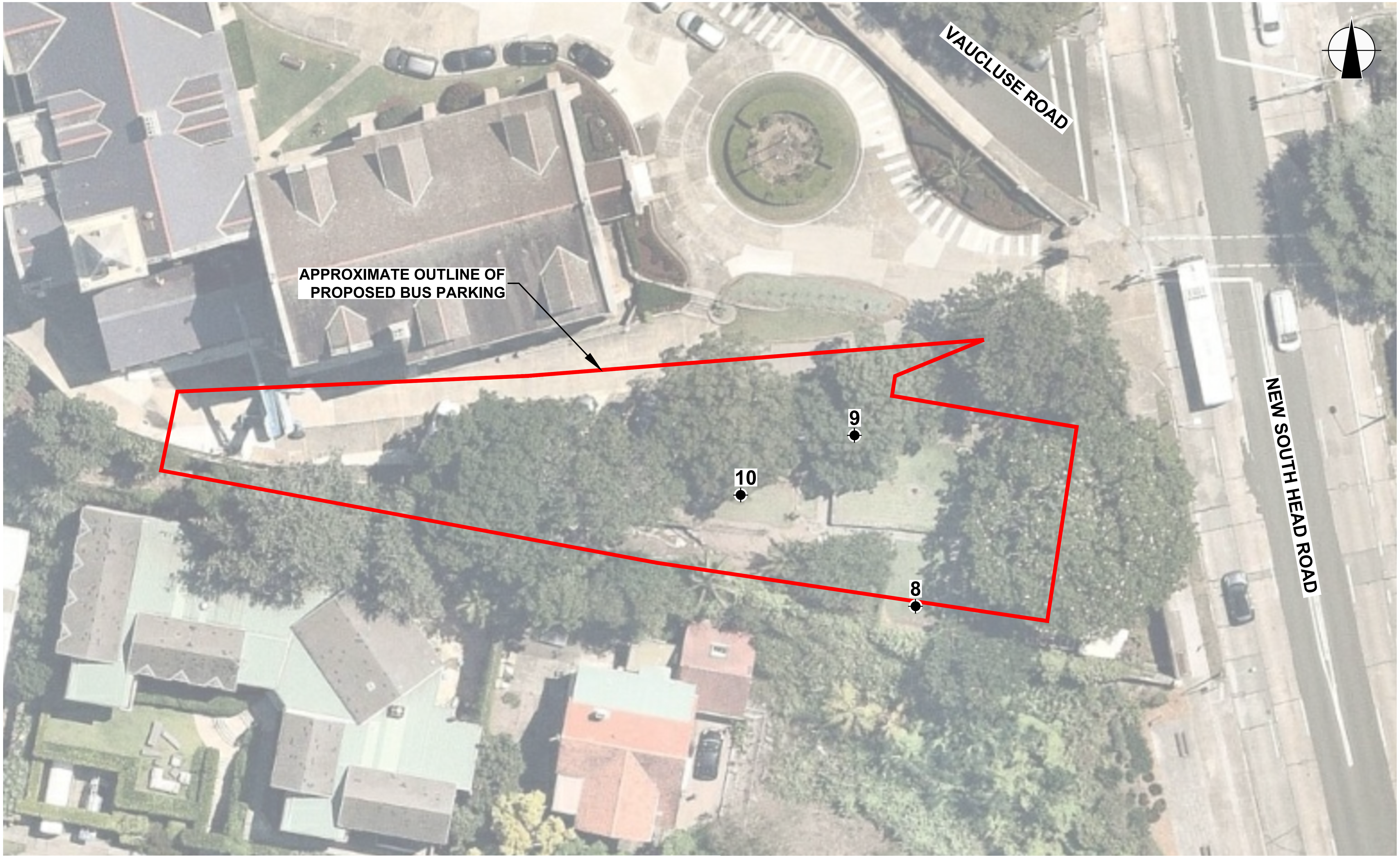
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	SITE LOCATION PLAN
Location:	2 VAUCLUSE ROAD, VAUCLUSE, NSW
Report No: 32915PH3	Figure: 1

JKGeotechnics

PLOT DATE: 24/02/2020 5:38:26 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32915PH VAUCLUSE\CAD\32915PH3.DWG



LEGEND

BOREHOLE AND DCP TEST

0 3 6 9 12 15

SCALE 1:300 @A3 METRES

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

Title:	**BOREHOLE LOCATION PLAN**
Location:	2 VAUCLUSE ROAD, VAUCLUSE, NSW
Report No:	32915PH3
Figure:	2

JKGeotechnics

PLOT DATE: 26/02/2020 1:51:59 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32915PH VAUCLUSE\CAD\32915PH3.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:

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

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

NOTES:

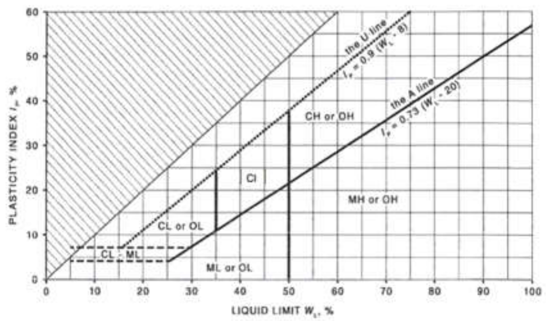
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.		
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.		
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	Sample taken over depth indicated, for environmental analysis.		
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.		
	DB	Bulk disturbed sample taken over depth indicated.		
	DS	Small disturbed bag sample taken over depth indicated.		
	ASB	Soil sample taken over depth indicated, for asbestos analysis.		
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.		
	SAL	Soil sample taken over depth indicated, for salinity analysis.		
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	VNS = 25	Vane shear reading in kPa of undrained shear strength.		
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).		
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ 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

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			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