REPORT TO

KINCOPPAL – ROSE BAY SCHOOL

ON

SUPPLEMENTARY GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

FOR

PROPOSED ELC BUILDING

AT

CNR NEW SOUTH ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW

Date: 20 April 2021

Ref: 32915SH1rpt Rev1

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a supplementary 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 supplementary investigation was commissioned by Mr Terry Mahady of Mahady Management, on behalf of KRB, by a signed 'Acceptance of Proposal' form dated 8 March 2021. The commission was on the basis of our fee proposal, Ref: P53690SH, dated 8 March 2021.

We previously carried out a geotechnical investigation at the site for the proposed development when it was at a concept design stage, and the results were presented in our report, Ref. 32915SH1rpt, dated 25 February 2020. We have included the results from our initial investigation in the preparation of this report.

We were also commissioned to carry out a geotechnical and hydrogeological investigation for two other proposed developments at the school. 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 Rev1, and Ref. 32915PH3rpt Rev1.

Based on the supplied architectural drawings prepared by BVN Architecture Pty Ltd (Drawing Nos. AR-A-B1-01[7], AR-A-B1-02[6], AR-A-B1-03[5], AR-A-C1-06[4], AR-A-C1-07[4]), we understand that a one and two storey building is proposed. The lowest proposed finished floor level (Level -03) will be at a reduced level (RL) of 37.2m. To achieve this level, excavation to a maximum depth of about 4.0m will be required within the north-eastern portion of the proposed building. Due to the sloping site, the depth of excavation will taper to zero at the south-western corner of the proposed building. The outline of the proposed building is shown on the attached Figure 2. Structural loads typical for this type of development have been assumed.

The purpose of the supplementary investigation was to further assess the subsurface conditions at four additional borehole locations and, based on the information obtained, to present our updated comments and recommendations on excavation, shoring design, drainage, footing design, soil aggression, the Level -03 floor slab and hydrogeology.

Our environmental consulting division, JK Environments (JKE), was commissioned to undertake an Additional Site Investigation and Remediation Action Plan (RAP), and this report should be read in conjunction with the JKE reports, Ref. E32915BDrptRev1 and E32915BArptRev1-RAP, dated April 2021.

We understand that the proposed ELC building is part of a larger proposed development within the junior school, however, we were only requested to provide advice for the proposed ELC building. Provision of geotechnical advice in relation to other aspects of the proposed junior school development was outside our agreed scope of work.

2 INVESTIGATION PROCEDURE

The fieldwork for the initial investigation was carried out on 28 January 2020 and comprised the auger drilling of three boreholes (BH1, BH2 and BH3) using our track mounted JK205 drill rig to depths of 2.0m, 9.2m and 9.3m, respectively.

The fieldwork for the current investigation was carried out on 14 and 29 March 2021, and comprised the auger drilling of four boreholes (BH101, BH102, BH103 and BH103A) using our track mounted JK205 drill rig to depths ranging from 3.4m (BH102) to 11.5m (BH103). BH101 and BH103A were extended by diamond core drilling using NMLC coring techniques to final depths of 12.44m and 11.39m, respectively. The purpose of the current boreholes was to further assess the depth to the bedrock surface and the quality of the bedrock.

The borehole locations are shown on the attached Figure 2. The locations of BH1, BH2 and BH3 were set out in consultation with Mr Mahady of Mahady Management prior to the commencement of drilling and were recorded using a Topcon GRS-1 differential GPS unit. The locations of BH101, BH102, BH103 and BH103A were set out by tape measurements from existing surface features. Figure 2 is based on the supplied survey plan by Crux Surveying Australia Pty Ltd (Crux Drawing No. 120244-SU-DT-007, dated 12 October 2016) laid over a recent Nearmap image of the site. The surface RLs indicated on the attached borehole logs and DCP test results sheet were obtained from the GPS unit (BH1, BH2 and BH3) or interpolated between spot level heights and ground contour lines shown on the survey plan (BH101, BH102, BH103 and BH103A). The accuracy of the surface levels is about ±0.1m. The survey datum is the Australian Height Datum (AHD).

The relative compaction and density of the fill and natural soil profiles were assessed from the Standard Penetration Test (SPT) results, and tactile examination. The strength of the upper bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings. The strength of the cored 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 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.

Further details of the techniques and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineers were present full time during the fieldwork to set out and record/measure the borehole locations, nominate the testing and sampling, and prepare the attached borehole logs. The Report Explanation Notes define the logging terms and symbols used.

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.

The recovered rock cores were photographed and Point Load Strength Index testing carried out. The rock core photographs are enclosed with the respective borehole log. The Point Load Strength Index test results are plotted on the borehole logs and summarised in Table B. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table B.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The footprint of the proposed ELC building (ie. the site) is located mid-slope on the northern flank of a gully, which graded down to the south-west at about 15°. The 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 timber 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 road and raised AC surfaced parking area were located on the southern and eastern sides of the site, respectively. Further to the south of the site, was a large fill batter slope that was up to about 10m high and graded down to the south at about 15°. 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 this building was a sandstone cliff face up to about 3m high. The bedrock was distinctly weathered and generally of low to medium strength. Minor seepage was observed over the surface of the cliff face. At the crest of the cliff face was a dry-stacked sandstone block retaining wall up to about 1.5m high. A

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 passes through the area in an approximate east-south-east to west-north-west orientation. A dyke is a sub-vertical igneous intrusion through the sedimentary bedrock.

The subsurface conditions within, and adjacent to, a dyke can be extremely variable. Where a dyke is present, the bedrock in contact with the dyke can also be extremely variable in terms of its quality (ie. weathering, strength and presence of defects). The depth to the bedrock surface can also be extremely variable over relatively short distances, which is the case with this site.

There may be other dykes present in close proximity to the dyke referred to above, and these may have formed the gully feature on which the proposed building is to be located. We infer that the gully was most likely formed by preferential erosion of the weaker weathered materials within the dyke, which generally comprise residual clay; such material appeared to be encountered within BH103 below 7.3m depth down to the bedrock surface. 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 previous and current boreholes generally encountered AC pavements then moderate to deep fill overlying natural sandy soils (BH2, BH101 and BH103), residual sandy clay in BH103 (weathered dyke material) then sandstone bedrock at variable depths. Groundwater was encountered at depth in BH2. Reference should be made to the attached borehole logs for specific details at each location. A summary of the subsurface conditions encountered in our investigations is provided below.

Pavements and Fill

With the exception of BH102, each borehole was drilled through an AC surfacing which ranged in thickness from 50mm (BH1, BH3 and BH101) to 90mm (BH2 and BH103A). The fill below the AC extended to depths ranging from 1.75m in BH1 to 6.9m in BH103 and BH103A and comprised gravelly silty sand and silty sand, with inclusions of brick, tile, metal, plastic and glass fragments, sandstone, ironstone and igneous gravel, cobbles and boulders. In BH102, the fill between 0.5m and 2.1m depth comprised mostly sandstone and ironstone gravel. 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, gravelly sand and sand were encountered below the fill in BH2, BH101, BH103 and BH103A. The sandy soils were very loose and loose relatively density, where tested. BH2 was terminated within the natural soil profile.

Due to the suspected presence of a dyke and the appearance of the natural soils compared to the overlying fill, it is possible that some of the natural sandy soils could be fill.

Residual Soils
Residual light grey sandy clay of low plasticity and stiff to very stiff strength was encountered in BH103 at 7.3m depth. This soil has been interpreted to be comprise residual material derived from weathering of an igneous dyke.

Sandstone Bedrock
With the exception of BH2, sandstone bedrock was encountered in each borehole at depths ranging from 1.75m (BH1) to 11.1m (BH103). Sandstone bedrock also outcropped at various locations within and adjacent to the site. In BH103, which we suspect was drilled through a dyke, the bedrock was only proven for a limited depth and may comprise weathered dolerite, rather than sandstone bedrock.

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 bedrock, and presence of sandstone 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.

In the current boreholes, the sandstone bedrock was generally distinctly and highly weathered with some fresh bands, and was generally of medium and high strength. The dark brown colouring of some of the bedrock is unusual, and may have been influenced by the presence of an inferred igneous dyke (as discussed below). In BH101, a 1.4m thick capping of extremely weathered sandstone of hard (soil) strength was encountered from first contact. In BH102 and BH103, 'TC' bit refusal occurred within the bedrock profile.

The sandstone bedrock contained defects including sub-horizontal bedding partings, joints and extremely weathered seams. A 0.24m thick 'no core' zone occurred in BH103A at a depth of 8.0m, and is inferred to be an extremely weathered band which has been washed out by the drill flush water.

Inferred Igneous Dyke
With reference to the relevant geological map of the site and based on investigation results, the trace of an igneous dyke may cross the central portion of the proposed building footprint, as shown on Figure 2. BH103 appears to have extended into the inferred dyke, as well as possibly BH2. Based on our previous experience on nearby sites, the dyke may manifest as a sub-vertical open defect within the sandstone bedrock with an upper portion of sandy infill overlying residual clay, that may include corestones of higher strength dolerite rock. The open defect representing the dyke and/or dyke infill can range from centimetres to many metres and can extend laterally for very large distances. Alternatively, the material in BH103, interpreted to be weathered igneous rock, could be a horizontal offshoot of the known dyke to the north of the school; it is known for such offshoots to penetrate along major bedding partings in the sandstone.

Groundwater

With the exception of BH2, each borehole was 'dry' during and on completion of auger 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, the groundwater level in BH2 had risen slightly to 8.0m depth. On 29 March 2021, the groundwater level in BH2 was at 8.6m depth.

In BH101 and BH103A, groundwater was measured on completion of coring at depths of 5.6m and 8.6m, respectively. As water is introduced into the boreholes as part of the coring process, the groundwater level in BH101 is not considered to be representative of the actual groundwater level, though the groundwater level in BH103A which was at the same level as measured in BH2, could be representative of the actual groundwater level at that location.

No other long term groundwater level monitoring was 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 Point Load Strength Index test results and correlated Unconfined Compressive Strengths also correlated well with our field assessment of rock strength. The estimated UCS's, based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (ie. UCS = 20 x $I_{S(50)}$), generally ranged from 8MPa to 24MPa, with a few values of up to 38MPa recorded.

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 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 around 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.

4.2 Excavation and Shoring

All excavation recommendations should be complemented by reference to the current NSW Government 'Code of Practice Excavation Work'.

Excavation to a maximum depth of about 4.0m is expected to extend mostly through the soil profile, however, sandstone bedrock of up to medium strength will be encountered within the north-western portion of the proposed building footprint.

Excavation of the soil profile may be completed using a 'digging' bucket fitted to a hydraulic excavator.

Bedrock of low and higher strength will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, rock saws or ripping tynes.

Rock excavation using hydraulic rock hammers will need to be strictly controlled as there will likely be direct transmission of ground vibrations to nearby school buildings. We recommend that continuous quantitative vibration monitoring be carried out on the adjacent school buildings whenever hydraulic rock hammers are used, as a precaution against possible vibration induced damage. The vibration monitors should be set up on the adjoining structures and the monitors should be fitted with flashing warning lights and sirens which would warn if vibrations exceeded the pre-set limits. We recommend a peak particle velocity of 5mm/sec be applied. The Structural Engineer should advise if any of the adjoining structures are sufficiently sensitive to require a lower vibration limit. It should be noted that when vibration limits are exceeded, they should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable depending on the associated vibration frequency.

If it is confirmed that transmitted vibrations are excessive, it would be necessary to change to alternative lower vibration emitting excavation equipment, such as a smaller rock hammer, rotary grinder, rock saw or drill and split.

We recommend the use of excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances and should be provided with a full copy of this report including the attachments.

Where the site geometry permits, 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 that are to be retained as part of the proposed development are founded on bedrock. If the crest of a temporary batter slope encroaches close to an adjacent school building (say within about 2m), we recommend some test pits be excavated to assess the adjacent footing details and foundation materials. The test pits should be excavated in the presence of a geotechnical engineer prior to excavation commencing to assess whether any localised temporary shoring or underpinning of the buildings is required.

Where temporary batters are not feasible within the site geometry, or are not preferred, the soil profile will need to be supported by an engineer designed shoring system that must be installed prior to the commencement of excavation. Due to the presence of deep sandy soils, we recommend a contiguous pile wall.

The wall designer must calculate the deflections associated with a cantilevered wall and confirm whether those deflections are acceptable for any adjoining structures/roadways or nearby buried services. Where the structural engineer confirms that the deflections are not acceptable, then the walls will need to be anchored and/or internally propped as excavation proceeds to reduce the deflections. We assume permanent lateral support of the shoring system will be provided by the new floor slabs and walls.

The contiguous pile walls must be founded below bulk excavation level (including below detailed excavations such as for footings, buried services, etc) at suitable depths to satisfy founding and stability considerations.

Construction of the contiguous pile walls must be of high quality taking care to prevent soil loss through gaps that will most likely occur between the piles as this would add to the possibility of ground subsidence occurring outside the excavation. Such gaps should be rectified progressively during excavation such as by dry packing the gaps with non-shrink, cementitious mortar.

Sandstone bedrock of at least low strength can be cut vertically, subject to geotechnical inspection of the cut faces at not more than 1.5m depth increments, and on completion of excavation. The purpose of the inspections is to identify adversely orientated defects which could isolate blocks or wedges of sandstone that would then require stabilisation, such as with rock bolts. Provision should be made in the construction program and budget for the inspections and possible stabilisation of the rock cuts.

4.3 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, the seepage should be able to be controlled using gravity drainage, or conventional sump and pump techniques.

4.4 Retaining Walls

For 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 simple 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. If the walls will propped or anchored at more than one level, the shoring should be designed for a uniform lateral earth pressure of 6HkPa, where H is the retained height in metres.

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. Weep hole outlets, also known as spitter pipes, should be provided between contiguous piles at a horizontal spacing no greater than about 1.8m and should incorporate a non-woven geotextile filter fabric at the inserted end to reduce soil erosion.

Apart from a contiguous pile wall, other retaining walls must be backfilled with either engineered fill that is 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 engineered fill.

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. If ground slopes fall away in front of a footing the passive restraint will be greatly diminished.

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 or dyke bedrock. Allowable bearing pressures for piles are presented in Section 4.5 below.

Anchors bonded into extremely weathered sandstone (or better quality) bedrock may be designed based on an allowable bond stress of 50kPa. All anchors should be proof tested to 1.3 times the design working load and locked-off at about 85% of their design working load, with this process being witnessed by an experienced engineer independent of the anchoring contractor. Approximately four days after lock off, at least 20% of the anchors should be subjected to lift-off testing to confirm that the anchors are continuing to hold their load. We recommend that only experienced 'top tier' contractors be considered for the anchor installations and that anchors be a design and construct subcontract to avoid disputes if the anchors fail test load.

4.5 Footings

4.5.1 Design

Following excavation, we expect that the proposed building footprint will generally be underlain by a moderate to deep uncontrolled fill, and based on the anticipated structural loads, the proposed building should therefore be uniformly supported by piled footings founded in the underlying sandstone or dyke bedrock. Sandstone bedrock will be encountered at bulk excavation level towards the north-western corner of the proposed building footprint and high level footings should be possible there.

Due to the presence of sandy soils and groundwater within the soil profile at depth, the proposed building should be supported by continuous flight auger (CFA) piles. Pad or strip footings could be used where the rock is less than 1m deep, which will be the case within the north-western corner of the building.

Pad and strip footings, as well as CFA piles founded a nominal 0.3m into sandstone or dyke bedrock of at least medium strength may be designed for an allowable bearing pressure of 1,500kPa. For piles, sockets below a minimum 0.3m length requirement in at least low strength bedrock may be designed for allowable shaft adhesion values of 150kPa in compression and 75kPa in tension, on condition that the pile shaft is suitably roughened.

Noting the variability of the bedrock depth and quality, and possible buried obstructions/boulders within the fill profile, we recommend that any shallow footing excavations be inspected and all pile drilling be witnessed by a geotechnical engineer. All CFA piles must be certified by the piling contractor.

4.5.2 Earthquake Design Parameters

A Hazard Factor (Z) of 0.08 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.

4.5.3 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.

4.6 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.

4.7 Hydrogeology

Based on the investigation results, we would only expect limited seepage into the excavation 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.

4.8 Geotechnical and Hydrogeological Monitoring Program (GHMP)

As the site is located within the Woollahra Council LGA, and excavation is proposed, Council will almost certainly impose a DA Condition that a GHMP be prepared, prior to the CC being issued. The GHMP will need to address any vibration and survey monitoring required, and will impose a number of hold points on the project. We can complete the GHMP if commissioned to do so.

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:

- Preparation of a GHMP.
- Excavation and inspection of test pits to expose adjacent school building footings and their foundation materials, if appropriate.
- Rock cut face inspections, if rock is encountered in the excavation.
- 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 and below the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

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

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the items tested or sampled.

14/02/2020

Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: KINCOPPAL - ROSE BAY SCHOOL **Ref No:** 32915SH1

Project: PROPOSED ELC BUILDING **Report:** B

Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD,VAUCLUSE, NSW **Report Date:** 15/04/21

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
101	9.24 - 9.28	0.8	16	A
	9.72 - 9.76	0.4	8	A
	10.14 - 10.18	0.9	18	A
	10.69 - 10.73	1.1	22	A
	11.31 - 11.35	0.7	14	A
	11.89 - 11.92	1.4	28	A
	12.25 - 12.29	1.6	32	A
103A	7.74 - 7.77	0.4	8	A
	8.34 - 8.38	0.8	16	A
	8.84 - 8.87	0.7	14	A
	9.35 - 9.38	0.6	12	A
	9.79 - 9.82	1.2	24	A
	10.23 - 10.26	1.1	22	A
	10.72 - 10.75	1	20	A
	11.13 - 11.17	1.9	38	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
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 based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' 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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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.
1
1 / 1

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

Job No.: 32915SH1 **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	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
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							

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BOREHOLE LOG

Borehole No.
2
1 / 2

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

Job No.: 32915SH1 **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 – 0.09			ASPHALTIC CONCRETE: 90mm.t				
		N = 4 2,2,2	38	1		-	FILL: Silty sand, fine to coarse grained, brown, with brick fragments, trace of sandstone gravel and clay.	M			APPEARS POORLY COMPACTED
		N = 7 4,4,3	37	2							
		N = 4 6,2,2	36 35	3 4							
		N = 8 4,5,3	34 33 32	5 6		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

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BOREHOLE LOG

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

Job No.: 32915SH1 **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 ELC BUILDING
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH1 **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	M			
					FILL: Silty sand, fine grained, brown, trace of clay.				
		40			FILL: Silty sand, fine to medium grained, brown and light brown.				
	N = 21 10,11,10		1		FILL: Silty sand, fine to coarse grained, brown and dark brown, trace of igneous gravel and brick fragments.				
	N = 6 5,4,2	39	2						APPEARS POORLY COMPACTED
	N = 15 7,4,11	38	3						APPEARS WELL COMPACTED
		37	4		as above, but with sandstone boulders and cobbles.				BANDS OF LOW 'TC' BIT RESISTANCE
	Nc = 12 15 11 /100mm REFUSAL	36 35	5 6						
		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

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BOREHOLE LOG

Borehole No.
3
2 / 2

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

Job No.: 32915SH1 **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

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BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL

Project: PROPOSED ELC BUILDING

Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~40.6 m

Date: 14/3/21 **Datum:** AHD

Plant Type: JK205 **Logged/Checked By:** B.A./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
DRY ON COMPLETION OF AUGERING							ASPHALTIC CONCRETE: 50mm.t				
						-	FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained sandstone, ironstone and igneous gravel.	M			APPEARS WELL COMPACTED
		N = 23 10,10,13	40				FILL: Silty sand, fine to medium grained, brown and light grey, trace of fine to coarse grained sandstone, ironstone gravel, trace of brick and glass framents and ash.				
				1							
		N = 8 5,5,3	39								APPEARS POORLY COMPACTED
				2							
			38								
		N = 29 6,17,12		3							APPEARS WELL COMPACTED
			37								
				4							
		N = 5 5,2,3	36								APPEARS POORLY COMPACTED
AFTER COMPLETION OF CORING				5							
			35								
						SP	SAND: fine to medium grained, brown, with silt fines.	M	VL		AEOLIAN
		N = 2 2,1,1		6							
			34								

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BOREHOLE LOG

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

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~40.6 m
Date: 14/3/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.A./A.J.H.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									SP	SAND: fine to medium grained, brown, with silt fines. *(continued)*	M	VL		AEOLIAN
	■			■		33	8		-	Extremely Weathered sandstone: Sandy CLAY, low plasticity, brown, fine to medium grained sand, with silt fines.	XW	(Hd)		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
						32								
						31	9			REFER TO CORED BOREHOLE LOG				
						30	10							
						29	11							
						28	12							
						27	13							

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CORED BOREHOLE LOG

Borehole No.
101
3 / 3

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

Job No.: 32915SH1 **Core Size:** NMLC **R.L. Surface:** ~40.6 m
Date: 14/3/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK205 **Bearing:** N/A **Logged/Checked By:** B.A./A.J.H.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components. START CORING AT 9.00m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: 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
0% RETURN		31, 30, 29	10, 11, 12		SANDSTONE: fine to medium grained, light grey with occassional grey laminae, bedded at 0-10°.	FR	M - H	0.80, 0.40		(9.85m) XWS, 0°, 60 mm.t	Hawkesbury Sandstone
					SANDSTONE: medium to coarse grained, light grey, yellow brown and dark brown, with quartz gravel, trace of occassional carbonaceous lenses, bedded at 0-15°.	HW	M - H / H	0.90, 1.1, 0.70, 1.4, 1.6		(10.86m) Be, 10°, P, R, Clay Ct (11.16m) Be, 3°, Ir, R, Fe Sn (11.75m) Be, 2°, Ir, R, Fe Sn (11.78m) Be, 3°, Ir, R, Fe Sn	
		28, 27, 26, 25	13, 14, 15		END OF BOREHOLE AT 12.44 m						

Strength scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 32915SH1
Borehole No: BH101
Depth: 9.00m-12.44m
KODAK Gray Scale
KODAK Color Control Patches
JOB NO. 32915SH1 BH101 START CORING AT 9.00m
9
10
11
12
END OF BH AT 12.44m

JKGeotechnics

BOREHOLE LOG

Borehole No.
102

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

Job No.: 32915SH1	**Method:** SPIRAL AUGER	**R.L. Surface:** ~37.0 m
Date: 14/3/21		**Datum:** AHD
Plant Type: JK205	**Logged/Checked By:** B.A./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
DRY ON COMPLETION							FILL: Gravelly sand, fine to medium grained, brown, fine to medium grained ironstone, sandstone and igneous gravel.	M			APPEARS WELL COMPACTED
		N = 21 9,15,6	36	1			FILL: Sandstone and ironstone gravel, fine to coarse grained, light grey and orange brown.				
		N = 15 7,11,4	35	2							
						-	SANDSTONE: fine to medium grained, light grey.	DW	M - H		HAWKESBURY SANDSTONE MODERATE TO HIGH 'TC' BIT RESISTANCE
			34	3			as above, but yellow brown and orange brown.		H		HIGH RESISTANCE
							END OF BOREHOLE AT 3.40 m				'TC' BIT REFUSAL
			33	4							
			32	5							
			31	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32915SH1 VAUCLUSE.GPJ <<DrawingFile>> 15/04/2021 15:46 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

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

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~38.6 m
Date: 29/3/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.A./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
DRY ON COMPLETION							ASPHALTIC CONCRETE: 60mm.t				
		N = 7 7,2,5	38			-	FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sandstone gravel, brick, tile and metal fragments, slag and ash.	M			APPEARS POORLY COMPACTED
				1							
		N = 5 3,2,3	37								
				2							
			36								
		N = 3 1,2,1		3							
			35								
				4			FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sandstone gravel, with clay fines, trace of ash.				
		N = 5 1,3,2	34								
				5							
			33								
		N = 11 2,4,7		6							APPEARS MODERATELY COMPACTED
			32								

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BOREHOLE LOG

Borehole No.
103
2 / 2

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

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~38.6 m
Date: 29/3/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.A./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
						SM	Silty SAND: fine to medium grained, dark brown, with clay fines. *(continued)*	M	(L)		AEOLIAN
			31	8		CL	Sandy CLAY: low plasticity, light grey.	w>PL	(St - VSt)		RESIDUAL POSSIBLE WEATHERED DYKE? NOTE: CONTINUOUS SPIRAL AUGER DRILLING (i.e. NO IN-SITU TESTING) BELOW 6.95m DEPTH IN ORDER TO PROVE BEDROCK
			30	9							
			29	10							
			28	11							
						-	INFERRED BEDROCK?	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
			27	12			END OF BOREHOLE AT 11.50 m				'TC' BIT REFUSAL
			26	13							
			25								

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BOREHOLE LOG

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

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~38.6 m
Date: 29/3/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.A./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
DRY ON COMPLETION OF AUGERING										ASPHALTIC CONCRETE: 90mm.t				
									-	FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sandstone gravel, trace of brick, tile and metal fragments, slag, concrete, ash and plastic.	M			CONTINUOUS SPIRAL AUGER DRILLING (i.e. NO IN-SITU TESTING) IN ORDER TO PROVE BEDROCK
						38								
							1							
						37								
							2							
						36								
							3							
						35								
							4			FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sandstone gravel, with clay fines, trace of ash.				
						34								
							5							
						33								
							6							
						32								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32915SH1 VAUCLUSE.GPJ <<DrawingFile>> 15/04/2021 15:47 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.
103A
2 / 3

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

Job No.: 32915SH1 **Method:** SPIRAL AUGER **R.L. Surface:** ~38.6 m
Date: 29/3/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.A./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
									SM	Silty SAND: fine to medium grained, brown. *(continued)*	M	L		AEOLIAN
									-	SANDSTONE: fine to medium grained, yellow brown.	DW	M		HAWKESBURY SANDSTONE
						31				REFER TO CORED BOREHOLE LOG				HIGH 'TC' BIT RESISTANCE
							8							
						30								
							9							
						29								
							10							
						28								
							11							
						27								
							12							
						26								
							13							
						25								

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CORED BOREHOLE LOG

Borehole No.
103A

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

Job No.: 32915SH1 | **Core Size:** NMLC | **R.L. Surface:** ~38.6 m
Date: 29/3/21 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK205 | **Bearing:** N/A | **Logged/Checked By:** B.A./A.J.H.

Water Loss\Level	Barrel Lift	RL (m AHD)	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)	DEFECT DETAILS: 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
		31			START CORING AT 7.59m						
					SANDSTONE: fine to coarse grained, yellow brown, bedded at 0-10°.	HW	M	0.40		(7.80m) Be, 10°, P, R, Clay Ct (7.93m) J, 40°, Ir, R, Fe Sn (7.97m) XWS, 0°, 50 mm.t	
			8		NO CORE 0.24m						
0% RETURN; ON COMPLETION OF CORING		30			SANDSTONE: medium to coarse grained, brown yellow brown and light grey, with quartz gravel, bedded at 10-20°.	HW	M	0.80		(8.56m) XWS, 15°, 20 mm.t	Hawkesbury Sandstone
			9					0.70		(9.07m) Be, 5°, P, R, Clay Ct	
								0.60		(9.31m) Be, 7°, Ir, R, Cn	
		29					H			(9.58m) Be, 2°, Cn	
								1.2			
			10					1.1			
		28						1.0		(10.56m) Jh, 78°, P	
			11					1.9		(11.26m) J, 75°, P, R, Fe Sn	
		27			END OF BOREHOLE AT 11.39 m						
			12								
		26									
			13								
		25									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 32915SH1
Borehole No: 103A
Depth: 7.59m - 11.39m
KODAK Gray Scale
KODAK Color Control Patches
JOB NO. 32915SH1 BH 103A START CORING AT 7.59m
7
8
NO CORE
9
10
11
END OF BOREHOLE AT 11.39m

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title:

SITE LOCATION PLAN

Location: CNR NEW SOUTH HEAD ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW

Report No: 32915SH1

Figure: 1

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

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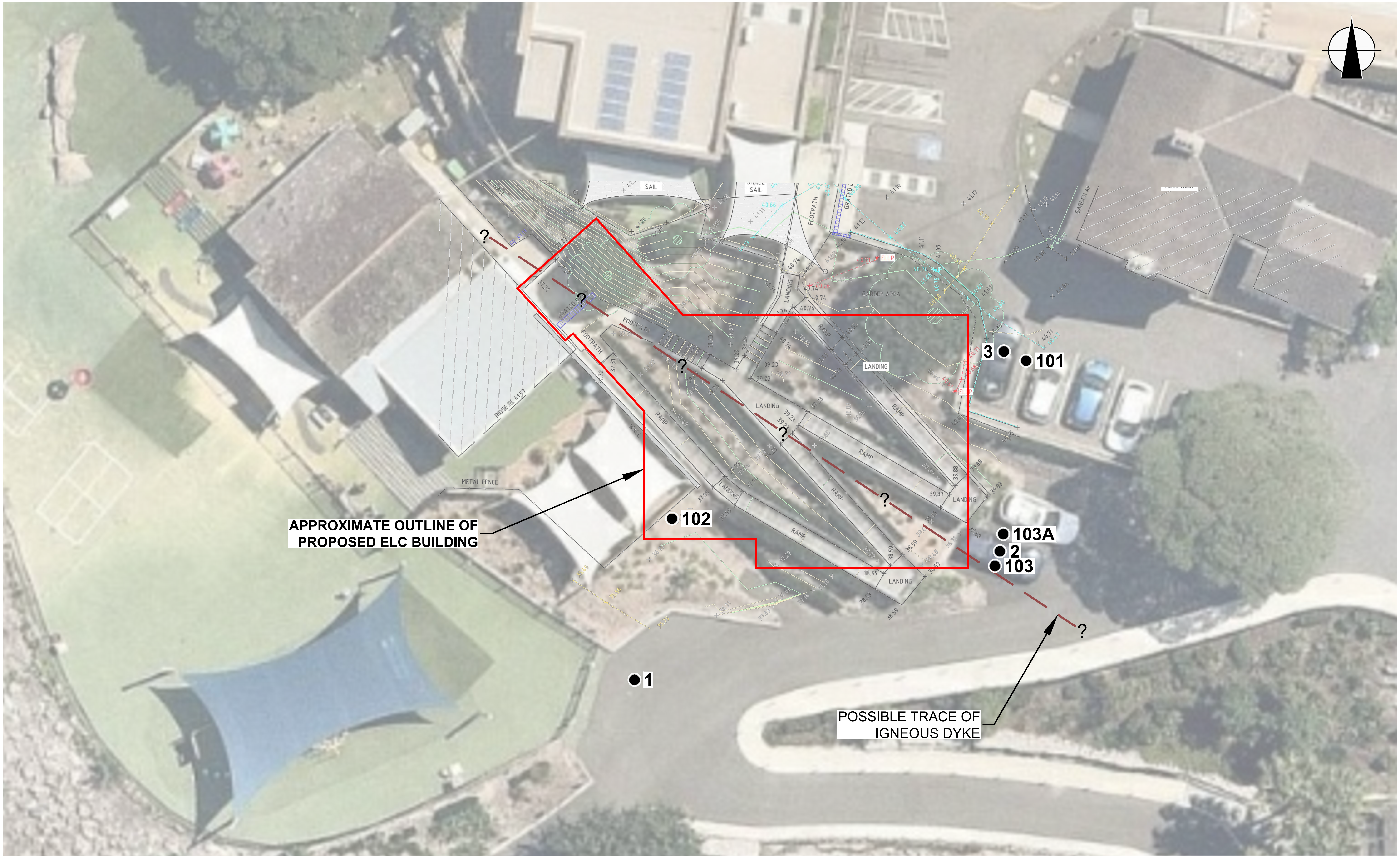
PLOT DATE: 15/04/2021 10:30:49 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32915PH VAUCLUSE\CAD\32915SH1.DWG



NOTES:
1. BH1, BH2 AND BH3 ARE FROM OUR 2020 INVESTIGATION.
2. BH101, BH102, BH103 AND BH103A ARE FROM THE CURRENT INVESTIGATION.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title:	**BOREHOLE LOCATION PLAN**
Location:	CNR NEW SOUTH HEAD ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW
Report No: 32915SH1	Figure: 2

JKGeotechnics

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

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

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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:

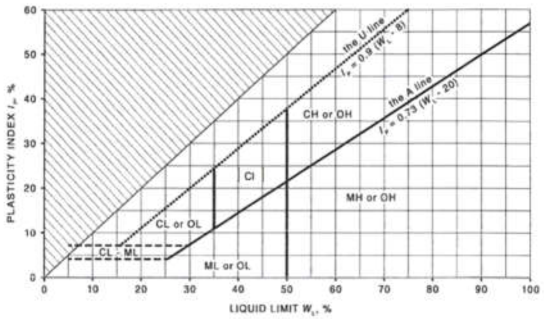
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE — ≤ 15 — 0 – 4
	L	LOOSE — > 15 and ≤ 35 — 4 – 10
	MD	MEDIUM DENSE — > 35 and ≤ 65 — 10 – 30
	D	DENSE — > 65 and ≤ 85 — 30 – 50
	VD	VERY DENSE — > 85 — > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $Is_{(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>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.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>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.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>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.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

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