REPORT TO

**KINCOPPAL - ROSE BAY SCHOOL**

ON

# GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

FOR

**PROPOSED ELEVATED WALKWAY AND ROAD**

AT

**CNR NEW SOUTH ROAD AND VAUCLUSE ROAD, VAUCLUSE, NSW**

Date: 20 April 2021
Ref: 32915SH2rpt Rev1

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## DOCUMENT REVISION RECORD

| Report Reference | Report Status | Report Date |
|---|---|---|
| 32915SH2rpt | Final Report | 26 February 2020 |
| 32915SH2rpt Rev1 | Revised report based on updated architectural drawings | 20 April 2021 |
| | | |

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## 1 INTRODUCTION

This report presents the results of a geotechnical and hydrogeological investigation for the proposed elevated walkway and road at Kincoppal-Rose Bay, School of the Sacred Heart (KRB), cnr New South Head Road and Vaucluse Road, Vaucluse, NSW. The location of the site is shown approximately in Figure 1.

We previously carried out a geotechnical and hydrogeological investigation at the site for the proposed development, and the results were presented in our report, Ref. 32915SH2rpt, dated 26 February 2020. The development details have since been revised. We have used the results of our previous investigation in the preparation of the current report.

We were also commissioned to carry out geotechnical and hydrogeological investigations for two other proposed developments at the school. The investigation results for the other two projects are presented in our separate reports, Ref. 32915PH1rpt Rev1, and Ref. 32915PH3rpt Rev1.

Based on the supplied architectural drawings prepared by BVN Architecture Pty Ltd (Drawing Nos. AR-A-A1-00$^5$, AR-A-B1-00(A)$^3$, AR-A-B1-00(B)$^2$), we understand that an elevated walkway and an entry road off Vaucluse Road are proposed. The entry road will connect to the existing concrete driveway in the area which will be widened to accommodate a new kiss and drop off zone. The surface level of the new entry road is not shown, but as it will connect to the existing concrete driveway, we assume that the surface of the new road will be at, or very close to, existing grade. The architectural drawings show the new road to be 'free draining permeable driveway paving'. The outlines of the proposed elevated walkway and road are shown on the attached Figure 2. Structural loads typical for this type of development have been assumed.

The purpose of the investigation was to assess the subsurface conditions at four borehole locations and, based on the information obtained, present our comments and recommendations on footings, soil aggression, earthworks, external pavements 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.

## 2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 28 January 2020 and 3 February 2020 and comprised the auger drilling of four boreholes (BH4 to BH8) to depths ranging from 0.4m (BH7) to 3.2m (BH4). BH4 and BH5 were auger drilled using our track mounted JK205 drill rig, whilst BH6 and BH7 were drilled to refusal using a hand auger, as there was no rig access to the latter locations. Dynamic Cone Penetration (DCP) tests were carried out at BH6 and BH7 to refusal depths of 0.5m and 0.4m, respectively. BH1 to BH3 were drilled elsewhere on site and are not part of this report.

The borehole locations as shown on the attached Figure2, were set out by tape measurements from existing surface features in consultation with Mr Mahady of Mahady Management prior to the commencement of drilling. The surface reduced levels (RLs) of BH6 and BH7 were obtained using a Topcon GRS-1 differential GPS unit, however, due to the presence of trees, it was not possible for the GPS to obtain surface RLs for BH4 and BH5. The surface RL to the Australian Height Datum (AHD) of BH6 and BH7 are shown on the respective borehole logs. At the time the GPS coordinates were recorded, the accuracy of the levels was about ±0.1m. Figure 2 is based on a recent Nearmap image of the site.

The relative compaction and density of the soil profile were assessed from the Standard Penetration Test (SPT) results, as well as interpretation of the DCP test results. The strength of the underlying bedrock in BH4 and BH5 was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings and correlation with subsequent laboratory moisture content test results. Groundwater observations were also made in the boreholes.

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

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

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

## 3 RESULTS OF THE INVESTIGATION

### 3.1 Site Description

The proposed elevated walkway and road (ie. the site) is located mid-slope on a moderately sloping west facing hillside, along the eastern side of KRB. The hillside slopes were generally in the order of 10° to 15° down to the west, though there were some flatter areas. Vaucluse Road bounds KRB along its eastern side.

At the time of the fieldwork, the site was mostly covered with garden beds, scattered large trees and shrubs, several relatively level grassed areas, concrete pathways and driveways and an asphaltic concrete (AC) surfaced roads. The eastern side of the site, which coincided with the eastern boundary of KRB, was lined by a concrete block boundary wall with an external sandstone facing.

A sandstone cliff face up to about 5m high with an approximate north-south orientation was located in the central portion of the site, just to the east of BH4 and BH5. The cliff face exposed sub-horizontally bedded, distinctly weathered sandstone of at least low to medium strength. There were some overhangs within the cliff face, the underside of which extended back a horizontal distance of about 2m. Groundwater seepage stains were visible over the bedrock surface. Several drill and blast holes were evident within the cliff face suggesting that some excavation into the cliff face has occurred in the past. There were several detached sandstone boulders along the toe of the cliff face.

Several school buildings were present around the southern and northern ends of the site.

## 3.2 Subsurface Conditions

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Hawkesbury Sandstone.

In summary, the boreholes encountered silty sand fill overlying residual silty sands (BH6 and BH7 only) then sandstone bedrock at shallow and moderate depth. Groundwater was not encountered within the maximum 3.2m depth of investigation. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. A summary of the subsurface conditions encountered in our investigations is provided below.

***Fill***

Silty sand fill was encountered from the surface of each borehole and extended to depths ranging from 0.2m (BH7) to 2.4m (BH4) below existing surface levels. Inclusions of sandstone cobbles and boulders were present within the fill. The fill was assessed to be variably compacted, which suggests the fill has not been placed and compacted in a controlled manner.

***Residual Silty Sand***

A thin layer (0.2m) of residual silty sand of medium dense or dense relative density was encountered below the fill in BH6 and BH7.

***Sandstone Bedrock***

Sandstone bedrock was encountered or inferred in each borehole at depths ranging from 0.4m (BH7) to 2.4m (BH4).

In BH4, the sandstone bedrock was assessed to be extremely and distinctly weathered and of hard (soil) and very low strength from first contact, improvising to slightly weathered and fresh and of medium to high strength below 3.0m depth. ‘TC’ bit refusal occurred within the bedrock profile at 3.2m depth.

In BH5, the sandstone bedrock was assessed to be distinctly weathered and of medium strength. ‘TC’ bit refusal occurred within the bedrock profile at 2.5m depth.

In BH6 and BH7 the depth of bedrock is inferred from the refusal depth of the DCP tests.

***Groundwater***

All boreholes were 'dry' during and on completion of drilling. No long term groundwater level monitoring has been undertaken.

## 3.3 Laboratory Test Results

The results of the moisture content tests carried out on recovered rock cutting samples from BH4 and BH5 correlated well with our field assessment of bedrock strength.

The soaked CBR test on a sample from BH6 returned a result of 12%, which suggests that a relatively good subgrade is present.

The soil pH test results were 6.3 and 7.1, which show the samples tested from BH6 and BH7 to be slightly acidic or near neutral. The soil sulphate and chloride content test results were less than 36mg/kg, which indicates low sulphate and chloride contents. The resistivity test results were relatively high (4,000 ohm.cm and 27,000 ohm.cm).

# 4 COMMENTS AND RECOMMENDATIONS

## 4.1 Footing Design

Due to the expected presence of relatively shallow sandstone bedrock across the site in areas where the sandstone does not already outcrop, the proposed elevated walkway should be uniformly supported by footings founded in, or on, the underlying sandstone bedrock.

Pad footings will be suitable where the depth to the bedrock is relatively shallow, say less than 1m depth. Where the rock is deeper than 1m, piled footings would be more appropriate. Due to the presence of sandy soils, the piles could comprise hand augered or bored piles (with an allowance for temporary or permanent liners), but preferably continuous flight auger (CFA) piles. However, given the expected relatively shallow depth to bedrock and noting the limited site access at least in some areas, it will not be economic to mobilise a CFA piling rig, and so the use of casing/liners with a pendulum auger fitted to an excavator or small bored piling rig could be attempted.

Pad footings and piles founded in the underlying sandstone bedrock should be designed for a maximum allowable end bearing pressure of 600kPa. The allowable bearing pressure may be increased to 1,000kPa, provided the footings/ piles are founded in at least low strength bedrock and a representative number of the footing excavations and piles are inspected by a geotechnical engineer. For piles only, sockets formed below a minimum 0.3m length requirement in at least low strength rock may be designed for allowable shaft adhesion values of 100kPa in compression and 50kPa in tension, on condition that the pile shaft is suitably roughened. Due to the presence of medium and high strength sandstone, the design of long rock socket lengths should be avoided where possible due to the expected difficulty in penetrating the competent sandstone.

Any footings located directly behind the crest of the sandstone cliff face should be designed for a maximum allowable bearing pressure of 600kPa and the cliff face below the toe of the footing inspected by a geotechnical engineer to identify any adverse defects or overhangs that may require stabilisation or underpinning.

All pad footings should be excavated, cleaned out, dewatered, inspected, and poured with minimal delay.

A Hazard Factor (Z) of 0.08 and a Site Subsoil Class Be should be adopted for earthquake design in accordance with AS1170.4-2007 ‘Structural Design Actions, Part 4: Earthquake Actions in Australia’, including Amendment Nos 1 & 2. It should be noted that if soil depths exceed 3m then the subsoil class would be Ce.

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

## 4.2 Earthworks

### 4.2.1 Site Preparation

Following demolition of any existing structures or pavements within the footprint of the proposed road and where the existing driveway is to be widened, all vegetation, topsoil, root affected soils and any deleterious or contaminated fill should be stripped from below the footprint of where the pavements are proposed. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for soft landscaping purposes, subject to approval from JKE. Reference should be made to the JKE report for guidance on the offsite disposal of soil.

Excavation of the soil profile down to the design subgrade level can be completed using buckets on a tracked hydraulic excavator.

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

### 4.2.2 Subgrade Preparation and Engineered Fill

Within the footprint of where new pavements are proposed, we recommend that all existing fill be stripped and recompacted as engineered fill. The fill must be free from organic matter and any particles greater than 75mm.

Following stripping of the existing fill, the subgrade should be proof rolled with at least six passes of a static smooth drum roller of at least 10 tonnes deadweight. The final passes of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of any 'unstable' areas.

Subgrade heaving during proof rolling should be expected in areas where the subgrade has become 'saturated'. The heaving areas can typically be improved by locally removing the heaving material down to a stable base and replacing with engineered fill, as outlined below. In this regard, the presence of an undulating bedrock surface can result in water becoming trapped in the hollows which will cause heaving and/or premature pavement failure. It is therefore important to carefully review the drainage during construction.

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

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

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

### 4.2.3 Design

Based on the investigation results and to account for some variability of the subgrade, we recommend that the proposed new pavements be designed on the basis of a CBR value of 10%, provided that the subgrade is prepared as per our advice above.

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

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

We note the architectural drawings show the new road off Vaucluse Road to comprise ‘free draining permeable driveway paving’. Whilst we expect the majority of the subgrade to comprise sandy soils which could have a reasonably high permeability allowing water infiltration, in some areas residual soils with a high clay content or even bedrock could be present. The clay soils and bedrock are expected to be have a very low permeability, and therefore do not readily allow water infiltration. Further, where clayey soils are present, allowing water into the subgrade could soften the soils, leading to possible poor performance of the pavement. We therefore recommend permeable pavers be used with caution on this site, and either flexible or rigid pavements may be more appropriate.

### 4.2.4 Subsoil Drains

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

## 4.3 Hydrogeology

Based on the investigation results, we would expect intermittent seepage over the bedrock surface from the higher lying areas to the east, and through defects within the cliff face on site, following prolonged or heavy rainfall.

Vaucluse Road is located directly to the east of the site, and may intercept any existing intermittent groundwater seepage over the bedrock surface, but service trenches and the like may bring in other seepage flows.

Noting that the only expected excavation required will be for new footings to support the proposed elevated walkway, in our opinion, the proposed development should not adversely affect the existing transient groundwater seepage flows in and around the site, provided the recommendations presented in this report are adopted in their entirety.

## 4.4 Geotechnical and Hydrogeological Monitoring Program (GHMP)

As the site is located within the Woollahra Council LGA, and excavation of rock (even just for footings) is proposed, Council may impose a DA Condition that a GHMP be prepared, prior to the CC being issued. The GHMP would need to address any vibration and survey monitoring requirements, and will impose a number of hold points on the project. We can complete the GHMP if commissioned to do so.

### 4.5 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:

- Advice on controlling vibrations if rock hammers are used.
- Footing/pile inspections.
- Proof roll inspections.
- Density testing of all engineered fill to at least Level 2 control by a GTA, if appropriate.
- Density testing of all granular pavement materials to at least Level 2 control by a GTA.

## 5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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 |
| | | **Page 1 of 1** | |

| **AS 1289** | **TEST METHOD** | **2.1.1** |
|---|---|---|
| **BOREHOLE NUMBER** | **DEPTH m** | **MOISTURE CONTENT %** |
| 1 | 1.80 - 2.00 | 3.5 |
| 3 | 7.00 - 8.00 | 8.0 |
| 3 | 9.10 - 9.30 | 3.5 |
| 4 | 2.60 - 2.80 | 5.9 |
| 4 | 3.00 - 3.20 | 4.9 |
| 5 | 1.80 - 2.40 | 6.0 |

**Notes:**

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

NATA

NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
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the items tested or sampled.

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

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**SOIL TEST SERVICES**
**ABN 43 002 145 173**

# TABLE B
# FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

**Client:** JK Geotechnics
**Project:** Proposed Developments at Kincoppal - Rose Bay School
**Location:** Cnr New South Head Road & Vaucluse Road, Vaucluse, NSW

**Ref No:** 32915PH
**Report:** B
**Report Date:** 17/02/2020
**Page 1 of 1**

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

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

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

17/02/2020

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Authorised Signature / Date
(T. Finnegan)

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

**Envirolab Services Pty Ltd**
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customerservice@envirolab.com.au
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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.<br>Alternatively determined by colourimetry/turbidity using Discrete Analyser. |

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

## Result Definitions

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

## Quality Control Definitions

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

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

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

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

## Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

**JKGeotechnics**

# BOREHOLE LOG

**Borehole No.**
**4**
1 / 1

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

**Job No.:** 32915PH2
**Method:** SPIRAL AUGER
**R.L. Surface:** N/A
**Date:** 28/1/20
**Datum:** AHD
**Plant Type:** JK205
**Logged/Checked By:** D.A.F./A.J.H.

| Groundwater Record | SAMPLES ES | SAMPLES U50 | SAMPLES DB | SAMPLES DS | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel Density | Hand Penetrometer Readings (kPa) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DRY ON COMPLETION | ■<br>■<br>■ | | | ■ | N = 7<br>3,4,3<br>N = 2<br>5,1,1 | 1 | ▨ | | FILL: Silty sand, fine to coarse grained, brown and light brown, trace of root fibres. | M | | | GRASS COVER<br>APPEARS POORLY COMPACTED |
| | ■ | | | ■ | | 2 | ▨ | | as above, but light orange brown. | | | | |
| | ■ | | | | | | ⁙ | - | SANDSTONE: fine to coarse grained, orange brown. | XW - DW | Hd - VL | | HAWKESBURY SANDSTONE<br>VERY LOW 'TC' BIT RESISTANCE |
| | | | | | | 3 | ⁙ | | as above, but light grey. | SW | M - H | | MODERATE TO HIGH RESISTANCE |
| | | | | | | 4<br>5<br>6 | | | END OF BOREHOLE AT 3.20 m | | | | 'TC' BIT REFUSAL |

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# JKGeotechnics

# BOREHOLE LOG

**Borehole No.**
5
1 / 1

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

**Job No.:** 32915PH2 **Method:** SPIRAL AUGER **R.L. Surface:** N/A
**Date:** 28/1/20 **Datum:** AHD
**Plant Type:** JK205 **Logged/Checked By:** D.A.F./A.J.H.

| Groundwater Record | SAMPLES ES | SAMPLES U50 | SAMPLES DB | SAMPLES DS | Field Tests | Depth (m) | Graphic Log | Unified Classification | DESCRIPTION | Moisture Condition/ Weathering | Strength/ Rel Density | Hand Penetrometer Readings (kPa) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DRY ON COMPLETION | ■<br>■ | | | | | | ▩ | | FILL: Silty sand, fine to coarse grained, brown and dark brown, with sandstone cobbles and boulders, trace of root fibres. | M | | | GRASS COVER<br>APPEARS MODERATELY COMPACTED |
| | ■ | | | ■ | N > 13<br>6,13/ 150mm<br>REFUSAL | 1 | ▩ | | | | | | |
| | ■ | | | ■ | | 2 | ⁚ | - | SANDSTONE: fine to coarse grained, light brown. | DW | M | | HAWKESBURY SANDSTONE<br>MODERATE TO HIGH 'TC' BIT RESISTANCE |
| | | | | | | 3<br>4<br>5<br>6 | | | END OF BOREHOLE AT 2.50 m | | | | 'TC' BIT REFUSAL |

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# BOREHOLE LOG

**Borehole No.**
**6**
1 / 1

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

**Job No.:** 32915PH2
**Method:** HAND AUGER
**R.L. Surface:** 51.4 m
**Date:** 3/2/20
**Datum:** AHD
**Plant Type:**
**Logged/Checked By:** D.A.F./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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DRY ON COMPLETION | ■ |  | ■ | ■ | REFER TO DCP TEST RESULTS |  |  | ▨ |  | FILL: Silty sand, fine to coarse grained, dark brown, trace of roots and root fibres. | M |  |  | GRASS COVER |
|  | ■ |  |  | ■ |  | 51 |  | ⁘ | SM | Silty SAND: fine to medium grained, orange brown, trace of clay. | M | MD |  | RESIDUAL |
|  |  |  |  |  |  |  |  |  |  | END OF BOREHOLE AT 0.50 m |  |  |  | HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK |
|  |  |  |  |  |  |  | 1 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 50 |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | 2 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 49 |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | 3 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 48 |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | 4 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 47 |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | 5 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 46 |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | 6 |  |  |  |  |  |  |  |
|  |  |  |  |  |  | 45 |  |  |  |  |  |  |  |  |

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# BOREHOLE LOG

**Borehole No.**
7
1 / 1

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

**Job No.:** 32915PH2
**Method:** HAND AUGER
**R.L. Surface:** 51.8 m
**Date:** 3/2/20
**Datum:** AHD
**Plant Type:**
**Logged/Checked By:** D.A.F./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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DRY ON COMPLETION | ■ | | | | REFER TO DCP TEST RESULTS | | | ▨ | | FILL: Silty sand, fine to coarse grained, dark brown, trace of roots and root fibres. | M | | | |
| | ■ | | | ■ | | | | ░ | SM | Silty SAND: fine to coarse grained, light orange brown. | M | D | | RESIDUAL |
| | | | | | | | | | | END OF BOREHOLE AT 0.40 m | | | | HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK |
| | | | | | | 51 | | | | | | | | |
| | | | | | | | 1 | | | | | | | |
| | | | | | | 50 | | | | | | | | |
| | | | | | | | 2 | | | | | | | |
| | | | | | | 49 | | | | | | | | |
| | | | | | | | 3 | | | | | | | |
| | | | | | | 48 | | | | | | | | |
| | | | | | | | 4 | | | | | | | |
| | | | | | | 47 | | | | | | | | |
| | | | | | | | 5 | | | | | | | |
| | | | | | | 46 | | | | | | | | |
| | | | | | | | 6 | | | | | | | |
| | | | | | | 45 | | | | | | | | |

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# DYNAMIC CONE PENETRATION TEST RESULTS

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

Job No. 32915PH2
Date: 3-2-20
Tested By: D.A.F.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

| Test Location | **6** | **7** | | | | | |
|---|---|---|---|---|---|---|---|
| Surface RL | 51.4m | 51.8m | | | | | |
| Depth (mm) | Number of Blows per 100mm Penetration | | | | | | |
| 0 - 100 | 3 | 6 | | | | | |
| 100 - 200 | 6 | 11 | | | | | |
| 200 - 300 | 8 | 16 | | | | | |
| 300 - 400 | 6 | 22 | | | | | |
| 400 - 500 | 6 | REFUSAL | | | | | |
| 500 - 600 | REFUSAL | | | | | | |
| 600 - 700 | | | | | | | |
| 700 - 800 | | | | | | | |
| 800 - 900 | | | | | | | |
| 900 - 1000 | | | | | | | |
| 1000 - 1100 | | | | | | | |
| 1100 - 1200 | | | | | | | |
| 1200 - 1300 | | | | | | | |
| 1300 - 1400 | | | | | | | |
| 1400 - 1500 | | | | | | | |
| 1500 - 1600 | | | | | | | |
| 1600 - 1700 | | | | | | | |
| 1700 - 1800 | | | | | | | |
| 1800 - 1900 | | | | | | | |
| 1900 - 2000 | | | | | | | |
| 2000 - 2100 | | | | | | | |
| 2100 - 2200 | | | | | | | |
| 2200 - 2300 | | | | | | | |
| 2300 - 2400 | | | | | | | |
| 2400 - 2500 | | | | | | | |
| 2500 - 2600 | | | | | | | |
| 2600 - 2700 | | | | | | | |
| 2700 - 2800 | | | | | | | |
| 2800 - 2900 | | | | | | | |
| 2900 - 3000 | | | | | | | |

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19

SOURCE: http://www.whereis.com/ NTS

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

## SITE LOCATION PLAN

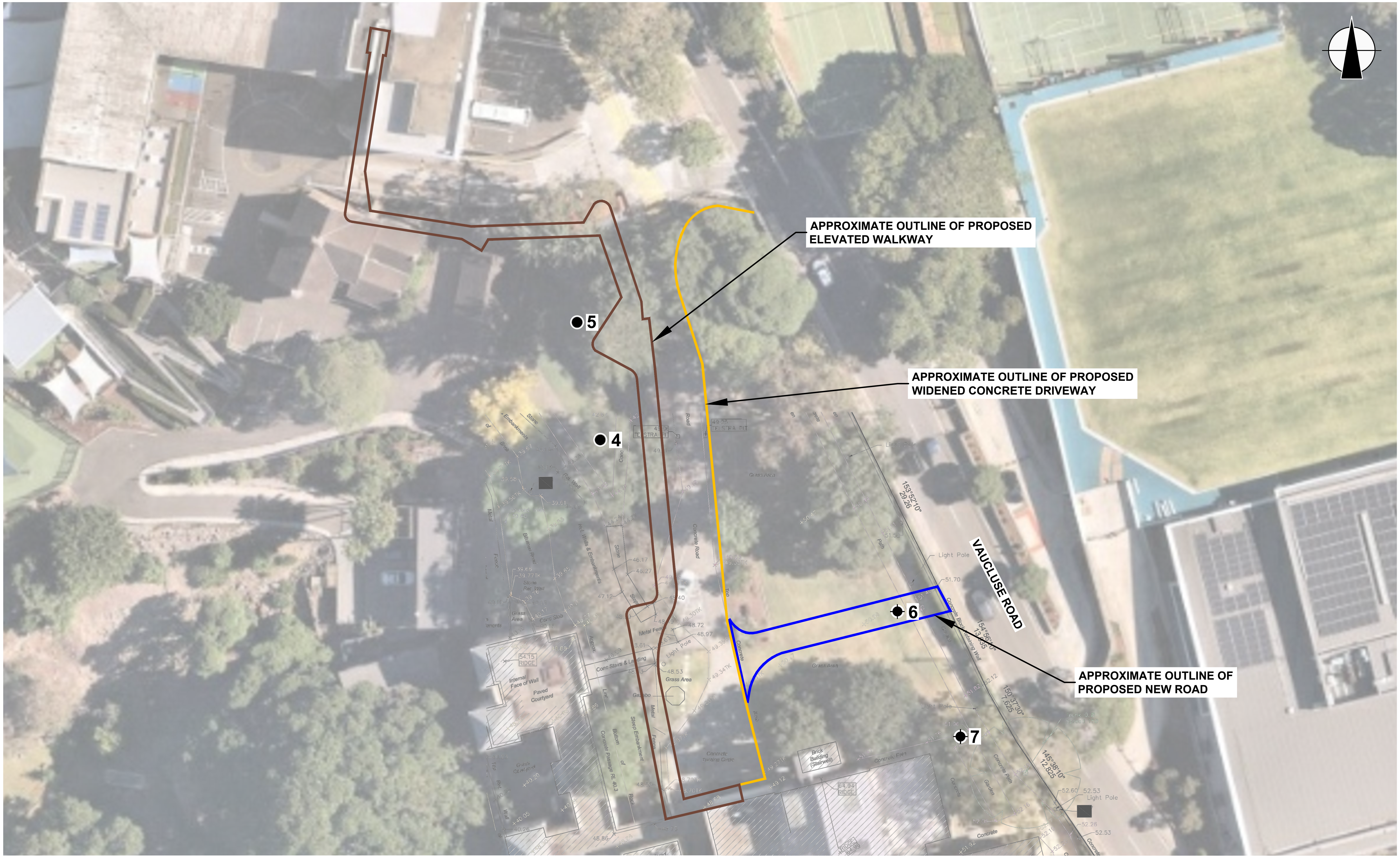
Location: 2 VAUCLUSE ROAD, VAUCLUSE, NSW

Report No: 32915SH2

Figure: 1

**JKGeotechnics**

PLOT DATE: 15/04/2021 2:41:34 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32915PH VAUCLUSE\CAD\32915SH2.DWG



**LEGEND**

● BOREHOLE

⊕ BOREHOLE AND DCP TEST

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

| | | | | | |
|---|---|---|---|---|---|
| 0 | 4.8 | 9.6 | 14.4 | 19.2 | 24 |

SCALE 1:480 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: 2 VAUCLUSE ROAD, VAUCLUSE, NSW

Report No: 32915SH2

Figure: 2

**JKGeotechnics**

PLOT DATE: 15/04/2021 2:43:31 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32915PH VAUCLUSE\CAD\32915SH2.DWG

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# 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 ($\phi$), coefficient of consolidation ($C_h$), coefficient of permeability ($K_h$), unit weight ($\gamma$), and vertical drained constrained modulus (M).

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

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

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

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

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

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

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

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

## LOGS

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

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

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

## GROUNDWATER

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

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

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

## FILL

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

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

## LABORATORY TESTING

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

## ENGINEERING REPORTS

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

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

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

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

### SITE ANOMALIES

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

### REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

### REVIEW OF DESIGN

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

### SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

## SYMBOL LEGENDS

### SOIL

- ▩ FILL
- ≋ TOPSOIL
- ▨ CLAY (CL, CI, CH)
- ▥ SILT (ML, MH)
- ░ SAND (SP, SW)
- ⁘ GRAVEL (GP, GW)
- ▨ SANDY CLAY (CL, CI, CH)
- ▨ SILTY CLAY (CL, CI, CH)
- ▨ CLAYEY SAND (SC)
- ░ SILTY SAND (SM)
- ▨ GRAVELLY CLAY (CL, CI, CH)
- ▨ CLAYEY GRAVEL (GC)
- ▥ SANDY SILT (ML, MH)
- ⁂ PEAT AND HIGHLY ORGANIC SOILS (Pt)

### ROCK

- ⁘ CONGLOMERATE
- ░ SANDSTONE
- ▤ SHALE/MUDSTONE
- ▥ SILTSTONE
- ▤ CLAYSTONE
- ■ COAL
- ▥ LAMINITE
- ▦ LIMESTONE
- ≋ PHYLLITE, SCHIST
- ▤ TUFF
- ⁘ GRANITE, GABBRO
- ✚ DOLERITE, DIORITE
- ∨ BASALT, ANDESITE
- ≋ QUARTZITE

### OTHER MATERIALS

- ▦ BRICKS OR PAVERS
- ░ CONCRETE
- ■ ASPHALTIC CONCRETE

# CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

### Laboratory **Classification** Criteria

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

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

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

NOTES:

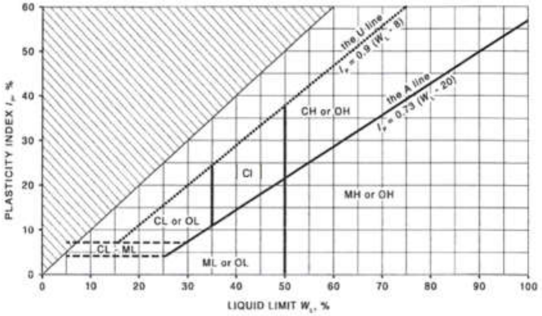
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



# LOG SYMBOLS

| Log Column | Symbol | Definition |
|---|---|---|
| Groundwater Record | ▼ | Standing water level. Time delay following completion of drilling/excavation may be shown. |
| | C | Extent of borehole/test pit collapse shortly after drilling/excavation. |
| | ▶ | Groundwater seepage into borehole or test pit noted during drilling or excavation. |
| Samples | ES | Sample taken over depth indicated, for environmental analysis. |
| | U50 | Undisturbed 50mm diameter tube sample taken over depth indicated. |
| | DB | Bulk disturbed sample taken over depth indicated. |
| | DS | Small disturbed bag sample taken over depth indicated. |
| | ASB | Soil sample taken over depth indicated, for asbestos analysis. |
| | ASS | Soil sample taken over depth indicated, for acid sulfate soil analysis. |
| | SAL | Soil sample taken over depth indicated, for salinity analysis. |
| Field Tests | N = 17<br>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<br>7<br>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<br>(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)<br>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. |

| Log Column | Symbol | Definition | Density Index ($I_D$) Range (%) | SPT 'N' Value Range (Blows/300mm) |
|---|---|---|---|---|
| Density Index/<br>Relative Density<br>(Cohesionless Soils) | 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<br>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

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

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

## Rock Material Strength Classification

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

# Abbreviations Used in Defect Description

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