



REPORT TO
KINCOPPAL – ROSE BAY SCHOOL

ON
SUPPLEMENTARY GEOTECHNICAL AND
HYDROGEOLOGICAL INVESTIGATION

FOR
PROPOSED CAR / BUS PARKING STRUCTURE

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

Date: 16 April 2021
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ATTACHMENTS

Table A: Point Load Strength Index Test Report

STS Table B: Four Day Soaked California Bearing Ratio Test Report

STS Table C: Point Load Strength Index Test Report

Envirolab Services Certificate of Analysis No. 236147

Borehole Logs 8, 9, 10 and 201, 202 and 203 (With Core Photographs)

Dynamic Cone Penetration Test Results

Figure 1: Site Location Plan

Figure 2: Borehole Location Plan

Vibration Emission Design Goals

Report Explanation Notes

1 INTRODUCTION

This report presents the results of a supplementary geotechnical and hydrogeological investigation for the proposed car/bus parking at Kincoppal-Rose Bay School (KRB), cnr New South Head Road and Vaucluse Road, Vaucluse, NSW. The location of the site is shown approximately 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. 32915SH3rpt, dated 28 February 2020. We have included the results from our initial investigation in the preparation of this report.

We were also commissioned to carry out geotechnical and hydrogeological investigations for two other proposed developments at the school. The fieldwork for those investigations was carried out concurrently with the fieldwork for this project. The investigation results for the other two projects are presented in separate reports, Ref. 32915SH1rpt Rev1, and Ref. 32915SH2rpt Rev1.

Based on the supplied architectural drawings prepared by BVN Architecture Pty Ltd (Drawing Nos. AR-B13-A0-00⁴, AR-B13-B0-00³, AR-B13-B0-01³, AR-B13-B0-02¹, AR-B13-B0-03¹, AR-B13-B1-00⁴, AR-B13-B1-01³, AR-B13-B1-02², AR-B13-C1-00³, AR-B13-D1-00³, AR-B13-D1-01² and AR-B13-M1-00³), we understand that a new bus parking area underlain by a basement bus car park is proposed within the south-eastern corner of the school. A vehicle ramp and driveway area is proposed off the south-western corner of the basement. The proposed finished floor level of the proposed basement will be at reduced levels (RL) between RL47.33m and RL48.90m. To achieve these levels, excavation to a maximum depth of about 6.0m will be required along the northern side of the proposed basement. Due to the sloping site, the southern side of the proposed basement will be at, or slightly below existing levels. The far south-western corner of the proposed basement will be slightly above existing levels. Reconfigured on-grade car parking is proposed just to the west of the proposed basement, with the southern extent of the car parking partially suspended over an existing heritage sandstone block retaining wall. The outline of the proposed basement 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 three 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, floor slabs, 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 for the initial investigation was carried out on 3 February 2020, and comprised the hand auger drilling of three boreholes (BH8, BH9 and BH10) to depths of 1.8m, 0.9m and 0.45m, respectively. Dynamic Cone Penetration (DCP) tests were carried out at each borehole to refusal depths of 1.73m (BH8), 0.85m (BH9) and 0.48 (BH10). BH8 and BH9 were extended into the underlying bedrock by diamond core drilling using TT56 coring techniques with our portable Melville coring equipment, to final depths of 5.95m and 4.70m, respectively.

The fieldwork for the current investigation was carried out on 25 March 2021, and comprised the auger drilling of three boreholes (BH201, BH202 and BH203) using our track mounted JK205 drill rig to depths of 1.18m, 2.87m and 2.84m, respectively. Each borehole was extended by diamond core drilling using NMLC coring techniques to final depths of 6.06m, 9.08m and 5.91m, respectively.

The borehole locations, which are shown on the attached Figure 2, 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. 122156-SU-DT-003, dated 5 February 2020). The approximate surface RLs indicated on the attached borehole logs and DCP test results sheet were interpolated between spot level heights and ground contour lines shown on the survey plan. 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, as well as interpretation of the DCP test results. In BH201, BH202 and BH203, 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 made in the boreholes during and on completion of drilling. A groundwater monitoring well was installed into BH202 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. A cast-iron 'gatic' cover was concreted flush with the ground surface to protect the top of the groundwater monitoring well. The installation details are presented on the BH202 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 the borehole locations, nominate the testing and sampling, and prepare the attached borehole logs and DCP test results sheet. 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 four day soaked CBR testing, and the results are provided in the attached STS Table B. Additional soil samples

were returned to another NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, chloride and sulphate content and resistivity testing; the test results are summarised in the attached Envirolab Services Certificate of Analysis 236147.

The recovered rock cores were photographed and 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 A and STS Table C. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table A and STS Table C.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The location of the proposed car / bus parking structure is mid-slope on a south-west facing hillside, which grades at about 10° to 15° down to the south-west. New South Head Road and Vaocluse Road were located uphill to the east and north of the site, respectively. A four to five storey KRB school building was located just to the west of the site and appeared to be in good external condition, based on a cursory inspection.

At the time of the fieldwork, the proposed basement footprint was essentially undeveloped and covered by grass, mulch, garden beds, concrete driveways and footpaths and scattered medium to large trees. The concrete pavements were generally in poor condition, with cracking and shallow subsidence observed. Some areas had been filled and terraced by several sandstone block retaining walls that were up to about 2m high. The area to the west of the proposed basement where the enlarged car parking is proposed had also been raised by filling and was mostly surfaced by concrete and was for parking cars. The southern side of the existing concrete surfacing in that area car park was supported by concrete block and sandstone block retaining walls, which ranged in height from about 1.5m at the eastern end to about 5m adjacent to the proposed development. Only the eastern portion of these walls were visible from within the site, and were generally in good condition. The western portion of the wall could not be inspected, due to a locked gate within the site which prevented access.

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

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

A 2.5m high sandstone block fence ran along the south-western side of the footpath, adjacent to Vacluse Road. The basal portion of the fence, up to about 0.5m, supported the footpath.

Ground surface levels beyond the southern boundary of KRB, immediately adjacent to the proposed development footprint, stepped down several metres to the south and south-west. Due to dense vegetation along the common boundary it was not possible to ascertain whether there was a sandstone cliff face below the boundary, or a retaining wall. However, due to the presence of outcropping sandstone bedrock along the base of the sandstone block retaining wall described above, it is likely that a sandstone cliff face is present. Two neighbouring brick and cement rendered brick buildings (1 Bayview Hill Road) were located just beyond the toe of the step down to the south of the site.

3.2 Subsurface Conditions

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

In summary, the boreholes encountered pavements (BH201, BH202 and BH203 only) and sand fill covering aeolian and residual soils then sandstone bedrock at shallow and moderate depths. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. A summary of the subsurface conditions encountered in our investigations is provided below.

Concrete Pavements

Reinforced concrete pavements ranging from 90mm to 120mm thickness were encountered in BH201 to BH203. A granular sub-base layer was not encountered.

Fill

Fill comprising silty sand, sand and gravelly sand was encountered below the concrete pavements in BH201, BH202 and BH203 and from the surface of BH8, BH9 and BH10 and extended to depths ranging from 0.45m (BH10) to 1.8m (BH203) below existing surface levels. Inclusions of igneous, ironstone and sandstone gravel, plastic, glass and concrete fragments, slag and organic matter were present within the fill. The fill has been assessed to be mostly poorly compacted, which suggests the fill has not been placed and compacted in a controlled manner.

Where a retaining wall is present, fill to a depth equivalent to the retained height should also be expected directly behind the crest of the wall.

Aeolian Silty Sand

A 0.3m thick layer of aeolian silty sand of loose relative density was encountered below the fill in BH202.

Residual Clayey Sand

A 0.2m thick layer of residual clayey sand of very loose relative density was encountered below the fill in BH8. In BH202, residual clayey sand of medium dense relative density was encountered below the aeolian silty sand.

Sandstone Bedrock

With the exception of BH10, sandstone bedrock was encountered in each borehole at depths ranging from 0.9m (BH8) to 2.6m (BH202). In BH10/DCP10, sandstone bedrock was inferred at about 0.45m depth, based on the hand auger and DCP refusal depths.

The sandstone bedrock ranged from distinctly to slightly weathered to fresh and was of medium and high strength. The exception was in BH203, where a 0.4m thick layer of extremely weathered sandstone of dense (soil) strength was encountered.

The sandstone bedrock contained defects including sub-horizontal bedding partings, joints and extremely weathered seams. 'No core' zones occurred in BH8 at depths of 1.8m (0.32m) and 3.2m (0.15m thick) and in BH9 at a depth of 3.8m (0.13m thick), and are inferred to be extremely weathered bands which have been washed out by the drill flush water.

Groundwater

BH9, BH10, BH201 and BH203 were 'dry' during and on the completion of auger drilling. Groundwater seepage was encountered during drilling in BH202 at 2.5m depth. In BH8, groundwater was encountered on completion of hand augering at 1.8m depth, just above the soil/bedrock interface.

In BH201, BH202 and BH203, groundwater was measured on completion of coring at depths of 0.4m, 3.57m and 0.9m, respectively. In BH202, groundwater was measured at a depth of 1.7m depth on 29 March 2021 (ie. four days after the completion of drilling). However, as water is introduced into the boreholes as part of the coring process, these groundwater levels are not considered to be representative of the actual groundwater level, which we consider would be well below the site and base of the proposed excavation. No other groundwater level monitoring was undertaken.

There was a variable, but often near full return, of the recycled drill flush water, which generally indicates a relatively impermeable rock mass. The very low drill flush return in BH203 however, was most likely attributable to leakage of drill flush water from the base of the casing.

3.3 Laboratory Test Results

The Point Load Strength Index test results and correlated Unconfined Compressive Strengths 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. $UCS = 20 \times I_{S(50)}$), generally ranged from 8MPa to 24MPa, with a few values of up to 34MPa recorded.

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

4 COMMENTS AND RECOMMENDATIONS

4.1 Transport for NSW (TfNSW)

New South Head Road bounds the site to east, which we understand is a TfNSW asset. In our experience and for an excavation of this size, depth and proximity to the road, TfNSW may require numerical analyses of the proposed excavation sequencing and shoring system to assess the potential impact of the proposed works on New South Head Road.

TfNSW may also require the installation and subsequent monitoring of borehole inclinometers, though this is considered unlikely as we expect the majority of the eastern side of the basement excavation can be temporarily battered within the site geometry.

Reference should be made to the NSW Government Technical Direction document 'Geotechnology', Reference: GTD2020/001 Version 1 dated 2 July 2020, for further information.

Once any TfNSW requirements have been confirmed, we can prepare a proposal to assist with the geotechnical requirements, if requested.

4.2 Dilapidation Survey

Prior to the commencement of any demolition and excavation on site, a dilapidation survey should be completed on the neighbouring brick buildings to the south of the site (1 Bayview Hill Road).

The dilapidation survey report can be used as a benchmark against which to set vibration limits for rock excavation, and for assessing possible future claims for damage arising from the works.

The owner of the adjoining property to the south should be asked to confirm in writing that the dilapidation survey report presents a fair assessment of the existing conditions. As dilapidation survey reports are relied upon for the assessment of potential future damage claims, they must be carried out thoroughly with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc) and defects photographed where practical.

Consideration should also be given to carrying out a dilapidation survey on the adjoining school building to the west of the proposed development.

4.3 Existing Retaining Walls

There are existing retaining walls on site, and whilst some of these will be demolished, some will remain. Based on a visual inspection, the majority of the existing walls at the site would only be constructed to a landscaping standard, and would generally not have factors of safety associated with an engineer designed structure.

We therefore recommend that prior to carrying out any demolition or excavation on site, a structural engineer be engaged to inspect any existing retaining walls that are to be incorporated into the development to assess their integrity and life expectancy, and to advise whether any strengthening of the walls is required, or if demolition and reconstruction with new engineer designed retaining walls is recommended.

4.4 Site Preparation

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

Care must be taken during site stripping and subsequent excavation not to undermine or remove support from any boundary structures (particularly the sandstone block fence adjacent to Vacluse Road) or retaining walls within the site that are to remain.

The bulk excavation for the proposed basement will extend very close to the north-eastern site boundary (ie. adjacent to Vacluse Road). Whilst the depth to bedrock is expected to be relatively shallow within the majority of the proposed basement footprint, at the north-eastern corner of the site, the depth to bedrock in BH202 was 2.6m. We therefore recommend during the demolition phase and prior to excavation, that some test pits be excavated to assess the depth to bedrock at the north-eastern corner of the site to assist with assessing the extent of shoring required. The test pits should be inspected by JK Geotechnics. Further advice on shoring design is presented in Section 4.7 below.

4.5 Excavation

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 6.0m will extend through the soil profile, but predominantly into sandstone bedrock of medium and high strength.

Excavation of the soil profile, extremely weathered sandstone and sandstone bedrock of very low strength, if present, 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 equipment could include hydraulic rock hammers, rotary grinders, rock saws or ripping tynes. Excavation productivity is expected to be low, as the sandstone bedrock is mostly of medium and high strength. Grid sawing techniques in conjunction with ripping or hammering will help to facilitate excavation.

Rock excavation using hydraulic rock hammers will need to be strictly controlled as there will likely be direct transmission of ground vibrations to the adjoining school building to the west and the neighbouring

structures to the south, all of which are likely to be founded on bedrock. We recommend that continuous quantitative vibration monitoring be carried out whenever hydraulic rock hammers are used during demolition and excavation on this site, as a precaution against possible vibration induced damage. Vibration monitors should be set up on the adjoining structures to the south at 1 Bayview Hill Road and on the adjoining school building to the west, 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 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.

Due to the proximity of nearby structures, further mitigation of transmitted vibrations could be provided by sawing slots around the perimeter of the proposed excavations, ahead of the bulk excavation.

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.

4.6 Drainage

Noting that the site is located part way down a hill well above Sydney Harbour, groundwater is not expected within the depth of excavation.

Notwithstanding, seepage into the excavation may occur as local seepage flows within the fill, at the fill, natural soil and bedrock interfaces and through joints and bedding partings within the bedrock profile, particularly after heavy or prolonged rain. If seepage does occur, it is likely to be intermittent and of a small flowrate, and readily controlled during construction by sump and pump methods or gravity drainage to divert it to the stormwater system for disposal.

In the long term, drainage will need to be provided behind any basement retaining walls and below the basement floor slab to intercept ephemeral seepage and dispose of it directly to the stormwater system. The completed excavation should be inspected by the hydraulic consultant to assess if the designed drainage system is adequate for the seepage flows encountered.

4.7 Shoring Design

4.7.1 Shoring System

Where the site geometry permits, the fill and natural soil profiles may be cut no steeper than 1 Vertical (V) in 1.5 Horizontal (H), provided all surcharge loads are kept well clear of the crest of the temporary batters.

Where temporary batters are not feasible, such as at the north-eastern corner of the proposed basement, the soil and weathered bedrock up to very low strength, will need to be supported by an engineer designed shoring system that must be installed prior to the commencement of excavation. We recommend a contiguous pile wall. The stability of the soils in open pier borings in the vicinity of BH202 will probably not be good. We suggest a trial bore be completed and a decision made as to whether conventional pier borings, perhaps assisted by temporary liners, will be feasible because otherwise continuous flight auger (CFA) piles will have to be used for the shoring system. The latter requires a much more expensive rig to be mobilised.

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 or where piles are founded above bulk excavation level, then the walls will need to be anchored and/or internally propped as excavation proceeds to reduce the deflections.

Contiguous piles may be terminated above bulk excavation level but must be embedded at least 0.5m into sandstone bedrock of at least medium strength, with representative piles drilled in the presence of a geotechnical engineer. Deeper penetration of the piles may be preferred so that the toe restraint can be provided by socketing the piles into bedrock below bulk excavation level. However, we note that the bedrock will generally be of medium and high strength, and therefore penetration rates through the bedrock will be slow and therefore the piling will need to be undertaken with a large piling rig. If the pile toes are not founded below bulk excavation level, then either permanent toe bolts will be required for lateral toe restraint, or the structure designed to provide permanent lateral support of the piles, such as by internal bracing.

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.

Once the founding conditions of any shoring piles founded above bulk excavation level has been confirmed to be suitable by the geotechnical engineer, a vertical face through the sandstone bedrock may then be excavated below the toe of the piles, but must not extend laterally underneath the piles.

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.

Retaining walls can be constructed at the toe of any temporary batters and subsequently backfilled, provided the rock below is proven to be of sufficient quality to be self-supporting. Based on the investigation results, we expect the rock will be of sufficient quality, and so an accessible void to maintain drainage around the basement could be formed, if a 'dry' wall around the basement is proposed. Alternatively, the cut faces could be left exposed inside the basement. Where the bedrock is not of sufficient quality, then the retaining wall will need to be constructed from the toe of the cut face.

4.7.2 Shoring Design Parameters

All cantilevered or anchored retaining walls, including those founded at the crest of rock cuts, should be uniformly founded in sandstone bedrock of at least medium strength.

For free-standing cantilever walls which are retaining areas where movement is of little concern (ie. landscape walls), 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 cantilever wall where the wall is restrained by the permanent structure, or which retain areas 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. A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and weathered rock of up to and including very low strength.

For propped or anchored contiguous pile walls, we recommend the design be based on a rectangular lateral earth pressure distribution of $6H\text{ kPa}$ for the retained profile, where 'H' is the retained height in metres.

Any surcharge affecting the walls (e.g. construction loads, nearby shallow footings founded in soil, inclined backfill, etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.

The basement retaining walls should be designed as a drained system with measures undertaken to induce complete and permanent drainage of the ground behind the walls. Weep hole outlets, also known as spitter pipes, should be provided between contiguous piles just above the soil/rock interface at a horizontal spacing no greater than about 1.5m. These should incorporate a non-woven geotextile filter fabric at the inserted end to reduce soil erosion.

For temporary anchors, permission must be sought from the Council and TfNSW, prior to installation where their assets would be affected. Temporary anchors bonded into sandstone of at least medium strength may be designed for an allowable bond stress of 300kPa . The anchors should have a bond length of at least 3m, with the bond located fully beyond a 45° line inclined up from the depth of the medium or high strength bedrock. All anchors should be proof-loaded to at least 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor, with anchors 'locked off' at about 85% of the design working load. Lift-off tests should be carried out on at least 15% of the anchors 3-4 days following

locking off to confirm that the anchors are holding their load. We recommend that only experienced contractors be considered for the anchor installation and stressing.

Any anchors or toe bolts that will extend below a neighbouring property, will require permission from the respective property owner, prior to installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping.

For lateral toe restraint, the retaining walls should be keyed into sandstone bedrock below bulk excavation level. An allowable lateral stress of 350kPa may be adopted for key design, subject to geotechnical inspection of any adjacent rock excavation below the founding level. For piles, the upper 0.3m depth of the socket should not be taken into account to allow for tolerance effects and possible disturbance during excavation. Where the retaining wall comprises concrete poured directly onto a clean and rough rock surface of at least medium strength, a friction angle of 35° could be adopted in the design.

4.8 Engineered Backfill

The excavated sandstone bedrock of at least low strength is considered suitable for reuse as engineered backfill behind cantilevered retaining walls, on condition the material is well-graded, 'clean', free of organic matter and contains a maximum particle size not exceeding 50mm. These materials should be compacted in maximum 150mm thick loose layers and to a density ratio of at least 98% of Standard Maximum Dry Density (SMDD). The compaction specification may be relaxed to at least 95% of SMDD in soft landscaping areas.

The wall designer must take into account the compaction stresses and surcharge loads from the compaction plant when working behind the retaining walls.

Density tests should be carried out on the engineered backfill to confirm the above specification is achieved, with at least one test per two layers per 50m². The testing should be completed as Level 2, in accordance with AS3798-2007 'Guidelines on earthworks for commercial and residential developments. The Geotechnical Testing Authority (GTA) should be directly engaged by the client, rather than by the excavation contractor.

Compaction of engineered fill behind retaining walls is very difficult. As an alternative, a single sized durable aggregate, such as 'blue metal' or recycled concrete, which do not require significant compactive effort, is often preferred if good performance is a priority. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor or trench roller in maximum 200mm thick loose layers. A non-woven geotextile filter fabric (such as Bidim A34) should be placed as a separation layer immediately on top of the temporary batter slope prior to backfilling, to control subsoil erosion. The geotextile should be wrapped over the surface of the gravel backfill and capped with at least a 0.3m thick compacted layer of engineered fill to reduce infiltration of surface water into the backfill. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required.

4.9 Footing Design

4.9.1 Design

On completion of bulk excavation, sandstone bedrock will be exposed at bulk excavation within the majority of the proposed basement footprint, or be present just below. Therefore, for uniformity of support, the proposed two storey car / bus parking structure should be uniformly supported by footings founded in the underlying sandstone bedrock.

Pad footings will be suitable where the depth to the bedrock is relatively shallow. Where the rock is deeper than about 1m, CFA or lined bored piles would be appropriate, if the soil proves to be too unstable for conventional bored piles.

Pad footings and piles founded in the underlying sandstone bedrock of at least low strength can be designed for an allowable bearing pressure of 1,000kPa, provided representative footing excavations and drilling of piles are inspected by a geotechnical engineer. The allowable bearing pressure may be increased to 3,000kPa, provided all footings/piles are founded in at least medium strength sandstone bedrock and all footing excavations and piles are inspected by a geotechnical engineer. All CFA piles must be certified by the piling contractor.

For piles, sockets below a minimum 0.3m length requirement (below bulk excavation level) in sandstone bedrock may be designed for allowable shaft adhesion values of 100kPa in compression and 50kPa in tension, on condition that the pile shaft is suitably roughened. These values may be increased to 300kPa in compression and 150kPa in tension, provided the sockets are within at least medium strength bedrock and each pile is inspected by a geotechnical engineer. However, due to the presence of medium and high strength sandstone, the design of long rock socket lengths should be avoided due to the expected difficulty in penetrating the sandstone.

All footings/piles must be founded below a 45° line inclined up from the toe of any cliff face adjacent to the southern site boundary or retaining wall. Alternatively, a reduced bearing pressure of 600kPa would apply for footings founded above the 45° line, subject to a geotechnical engineer inspecting any adjacent cliff face to identify any adverse defects or overhangs that may require stabilisation or underpinning.

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

4.9.2 Earthquake Design Parameters

A Hazard Factor (Z) of 0.08 and a Site Subsoil Class Ce (due to the presence of deep fill behind some of the existing retaining walls) should be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia', including Amendment Nos 1 & 2.

4.9.3 Soil Aggression

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

4.10 Basement Floor Slab Construction

On completion of the excavation for the proposed basement, we expect sandstone bedrock will be mostly exposed at bulk excavation level. However, soil could be exposed in some areas along the southern side of the basement. Therefore, to prevent issues of differential settlement, we recommend that the basement floor slab be designed as suspended where it overlies soil.

We also recommend that an underfloor drainage blanket be provided. The drainage material should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead any transient groundwater seepage to a sump for gravity drainage to the stormwater system.

4.11 On-Grade Pavements

4.11.1 Subgrade Preparation and Engineered Fill

The excavation advice above in Section 4.5 applies. Excavation of the soils down to design subgrade level can be completed using buckets on a tracked hydraulic excavator.

Where a rock subgrade is exposed below external pavements, 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 and to provide a more uniform subgrade.

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

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

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

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

Engineered fill could comprise the excavated sandstone bedrock of at least medium strength, provided it is well graded and crushed (so that the maximum particle size does not exceed 75mm), 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 KRB or their representative and not by the contractor.

4.11.2 Design

Based on the investigation results, we recommend that any proposed on-grade pavements be designed on the basis of a CBR value of 10%, provided that the subgrade is prepared as per our advice above and any poor quality soils are removed, bearing in mind there may be pockets of varying quality materials.

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

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

4.11.3 Subsoil Drains

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

4.12 Hydrogeology

Based on the investigation results, we expect only minor seepage into the excavation, more so during or following periods of heavy rainfall.

Settlement occurring due to draining any minor seepage from within the soils and bedrock is almost inconceivable.

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

We consider that design and construction of a 'drained' basement to be appropriate with 'tanking' being unnecessary.

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

4.13 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 monitoring of the basement shoring walls and retained areas for movement, and will impose a number of hold points on the project. We can complete the GHMP if commissioned to do so.

4.14 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 of test pits at the north-eastern corner of the proposed basement excavation to assess the extent where shoring will be required.
- Completion of trial pier boring.
- Anchor inspections, if appropriate.
- Rock cut face inspections.
- Footing/pile inspections.
- Proof rolling inspections.
- Density testing of all engineered fill, base and sub-base layers 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.

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: Kincoppal - Rose Bay School

Ref No: 32915SH3

Project: Proposed Developments at Kincoppal - Rose Bay School

Report: A

Location: Cnr New South Head Road and Vaucluse Road, Vaucluse, NSW

Report Date: 31/03/21

Page of 2

BOREHOLE NUMBER	DEPTH (m)	I _S (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
201	1.33 - 1.37	1.7	34	A
	1.73 - 1.76	0.8	16	A
	2.25 - 2.29	1.4	28	A
	2.78 - 2.82	0.9	18	A
	3.25 - 3.29	0.5	10	A
	3.69 - 3.72	0.5	10	A
	4.14 - 4.16	1.2	24	A
	4.63 - 4.66	0.9	18	A
	5.31 - 5.35	0.8	16	A
	5.86 - 5.90	0.7	14	A
202	2.92 - 2.96	1.5	30	A
	3.26 - 3.29	1	20	A
	3.69 - 3.72	1	20	A
	4.27 - 4.30	0.8	16	A
	4.70 - 4.73	1	20	A
	5.24 - 5.27	0.9	18	A
	5.70 - 5.73	1.2	24	A
	6.13 - 6.17	0.8	16	A
	6.67 - 6.71	1	20	A
	7.22 - 7.26	0.7	14	A
	7.66 - 7.69	0.6	12	A
	8.29 - 8.33	1	20	A
	8.72 - 8.76	0.9	18	A
	9.00 - 9.04	1.1	22	A
203	2.96 - 3.00	0.5	10	A

Note: See Page 2

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: Kincoppal - Rose Bay School **Ref No:** 32915SH3

Project: Proposed Developements at Kincoppal - Rose Bay School **Report:** A

Location: Cnr New South Head Road and Vaucluse Road, Vaucluse, NSW **Report Date:** 31/03/21

Page 2 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
203	3.16 - 3.20	0.4	8	A
	3.75 - 3.79	0.7	14	A
	4.21 - 4.24	0.6	12	A
	4.83 - 4.86	0.9	18	A
	5.09 - 5.12	1.1	22	A
	5.76 - 5.81	0.9	18	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 $I_{s(50)}$.

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

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

Page 1 of 1

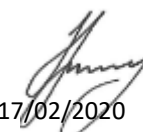
BOREHOLE NUMBER	BH 6	BH 8	BH 10
DEPTH (m)	0.00 - 0.20	0.20 - 0.40	0.20 - 0.40
Surcharge (kg)	9.0	9.0	9.0
Maximum Dry Density (t/m ³)	1.69 STD	1.81 STD	1.80 STD
Optimum Moisture Content (%)	16.5	14.7	14.6
Moulded Dry Density (t/m ³)	1.66	1.77	1.77
Sample Density Ratio (%)	98	98	98
Sample Moisture Ratio (%)	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.



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the items tested or sampled.


17/02/2020

Authorised Signature / Date
(T. Finnegan)

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

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

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

NOTES:

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

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	<u>32915PH, Vaucluse</u>
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
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. Tests not covered by NATA are denoted with *	

Results Approved By

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.

Client Reference: 32915PH, Vaucluse

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.

BOREHOLE LOG

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

Job No.: 32915PH3 **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

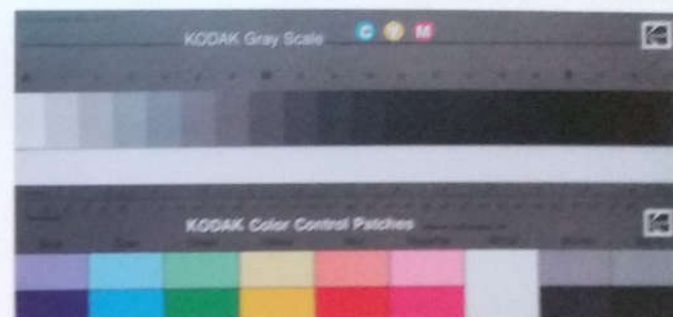
Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS									
ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS	1			FILL: Silty sand, fine to medium grained, brown, trace of roots and root fibres.	M			GRASS COVER
									as above, but grey and light brown, trace of fine to medium grained ironstone gravel, concrete fragments and slag.				APPEARS MODERATELY COMPACTED
													APPEARS POORLY COMPACTED
								SC	Clayey SAND: fine to coarse grained, light brown.	W	VL		RESIDUAL
						2			REFER TO CORED BOREHOLE LOG				
						3							
						4							
						5							
						6							

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW																			
Job No.: 32915PH3 Date: 3/2/20 Plant Type: MELVELLE			Core Size: TT56 Inclination: VERTICAL Bearing: N/A			R.L. Surface: N/A Datum: Logged/Checked By: D.A.F./A.J.H.													
Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$										DEFECT DETAILS		Formation
							VL-0.1	L-0.3	M-1	H-3	VF-10	EH	SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness					
				START CORING AT 1.80m															
		2		NO CORE 0.32m															
				SANDSTONE: fine to coarse grained, light grey, orange brown and red brown, bedded at 0-15°.	DW	M										(2.36m) Be, 0°, Fe Sn	Hawkesbury Sandstone		
		3		as above, but light grey and grey.	FR														
				NO CORE 0.15m															
				SANDSTONE: fine to coarse grained, light grey, with grey laminae, bedded at 0-20°.	FR	M											Hawkesbury Sandstone		
		4																	
		5																	
		6		END OF BOREHOLE AT 5.95 m															
		7																	



Job No: 32915PH
Borehole No: 8
Depth: 1.80m - 5.95m



32915PH BH8 START CORING AT 1.80m

2 | CORE LOSS
0.32m

3 | CORE LOSS
0.15m

4

5

JK Geotechnics

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL

Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL

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


Job No.: 32915PH3

Method: HAND AUGER

R.L. Surface: N/A

Date: 3/2/20

Datum:
Plant Type:
Logged/Checked By: D.A.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING	█			█	REFER TO DCP TEST RESULTS				FILL: Silty sand, fine to coarse grained, brown and dark brown, trace of fine to medium grained sandstone gravel, roots and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
	█			█					FILL: Silty sand, fine to coarse grained, light orange brown and brown, with fine to medium grained sandstone gravel and clay. REFER TO CORED BOREHOLE LOG				
						1							
						2							
						3							
						4							
						5							
						6							

CORED BOREHOLE LOG

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

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

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

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32915PH3 VAUCLUSE.GPJ <<DrawingFile>> 25/02/2020 12:43 10.01.00.01 Dated Log and In Situ Test - DGD Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2018-03-20



Job No: 32915 PH
Borehole No: 9
Depth: 0.90m → 4.70m



32915PH BH9 START CORING AT 0.90m

1

2

3

4

CORE LOSS
0.13m

EOBH AT 4.70m

JK Geotechnics

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL

Project: PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL

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

Job No.: 32915PH3

Method: HAND AUGER


R.L. Surface: N/A

Date: 3/2/20

Datum:

Plant Type:

Logged/Checked By: D.A.F./A.J.H.

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

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Method:** HAND AUGER **R.L. Surface:** ~48.6 m
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to medium grained, brown, trace of roots and root fibres.	M			GRASS COVER
						48	1			as above, but grey and light brown, trace of fine to medium grained ironstone gravel, concrete fragments and slag.				APPEARS MODERATELY COMPACTED
						47			SC	Clayey SAND: fine to coarse grained, light brown.	W	VL		RESIDUAL
							2			REFER TO CORED BOREHOLE LOG				
						46	3							
						45	4							
						44	5							
						43	6							
						42								

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL																
Project: PROPOSED CAR / BUS PARKING STRUCTURE																
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW																
Job No.: 32915SH3																
Core Size: TT56																
R.L. Surface: ~48.6 m																
Date: 3/2/20																
Inclination: VERTICAL																
Datum:																
Plant Type: MELVELLE																
Bearing: N/A																
Logged/Checked By: D.A.F./A.J.H.																
Water Loss/Level	Barrel Lift	RL (m)	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			Formation				
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness						
									600 200 200 60 20							
		47			START CORING AT 1.80m											
100% RETURN			2		NO CORE 0.32m											
			46		SANDSTONE: fine to coarse grained, light grey, orange brown and red brown, bedded at 0-15°.	DW	M									
			3		as above, but light grey and grey.	FR										
			45		NO CORE 0.15m	FR	M									
			44		SANDSTONE: fine to coarse grained, light grey. with grey laminae, bedded at 0-20°.											
			4													
			5													
			43													
			6		END OF BOREHOLE AT 5.95 m											
			42													
			7													
			41													

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Method:** HAND AUGER **R.L. Surface:** ~50 m
Date: 3/2/20 **Datum:**
Plant Type: **Logged/Checked By:** D.A.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to coarse grained, brown and dark brown, trace of fine to medium grained sandstone gravel, roots and root fibres.	M			GRASS COVER
										FILL: Silty sand, fine to coarse grained, light orange brown and brown, with fine to medium grained sandstone gravel and clay. REFER TO CORED BOREHOLE LOG				APPEARS POORLY COMPACTED
						49	1							
						48	2							
						47	3							
						46	4							
						45	5							
						44	6							

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Core Size:** TT56 **R.L. Surface:** ~50 m
Date: 3/2/20 **Inclination:** VERTICAL **Datum:**
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** D.A.F./A.J.H.

Water Loss/Level	Barrel Lift	RL (m)	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 $I_p(50)$	DEFECT DETAILS		Formation
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	
								VL-0.1 L-0.3 M-1 H-3 VH-10 EH	600 200 60 20	Specific General	
					START CORING AT 0.90m						
		49	1		SANDSTONE: fine to coarse grained, red brown and orange brown, bedded at 10-20°.	DW	H				
		48	2			SW - FR	M				
		47	3		SANDSTONE: fe to coarse grained, light grey, with occasional grey laminae, bedded at 5-15°.						
		46	4		NO CORE 0.13m SANDSTONE: fine to coarse grained, light grey, with occasional grey laminae, bedded at 5-15°.	FR	M				
		45	5		END OF BOREHOLE AT 4.70 m						
		44	6								

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL

Project: PROPOSED CAR / BUS PARKING STRUCTURE

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

Job No.: 32915SH3

Method: HAND AUGER

R.L. Surface: ~48.5 m

Date: 3/2/20

Datum:

Plant Type:

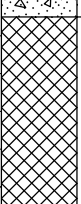
Logged/Checked By: D.A.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to coarse grained, dark brown, trace of fine to coarse grained sandstone gravel.	M			APPEARS POORLY COMPACTED
						48				END OF BOREHOLE AT 0.45 m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
							1							
						47								
							2							
						46								
							3							
						45								
							4							
						44								
							5							
						43								
							6							
						42								

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Method:** SPIRAL AUGER **R.L. Surface:** ~51.6 m
Date: 25/3/21 **Datum:**
Plant Type: JK205 **Logged/Checked By:** B.A./A.J.H.

Groundwater Record	SAMPLES			Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS									
ON COMPLETION OF CORING AUGERING						51			CONCRETE: 100mm.t FILL: Sand, fine to medium grained, brown, with fine to coarse grained sandstone gravel, trace of silt fines.	M			3 x 7mm DIA. REINFORCEMENT 25, 50 & 80mm TOP COVER APPEARS MODERATELY COMPACTED
						1							
									SANDSTONE: fine to medium grained, brown. REFER TO CORED BOREHOLE LOG	DW	M - H		HAWKESBURY SANDSTONE HIGH 'TC' BIT RESISTANCE
						50							
						2							
						49							
						3							
						48							
						4							
						47							
						5							
						46							
						6							
						45							

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Core Size:** NMLC **R.L. Surface:** ~51.6 m
Date: 25/3/21 **Inclination:** VERTICAL **Datum:**
Plant Type: JK205 **Bearing:** N/A **Logged/Checked By:** B.A./A.J.H.

Water Level	Barrel Lift	RL (m)	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 $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
										Specific	General	
					START CORING AT 1.18m							
			50		SANDSTONE: fine to medium grained, dark brown, and brown and red brown, bedded at 0-10°.	MW	M - H	*1.7				
			2					*0.80				
			49					*1.4				
			3		as above, but light brown.	SW		*0.90				
			48					*0.50				
			4					*0.50				
			47		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°.	FR		*1.2				
			5					*0.90				
			46					*0.80				
			6					*0.70				
					END OF BOREHOLE AT 6.06 m							
			45									
			7									
			44									

JK 0.024 LIB GLB Log JK CORED BOREHOLE - MASTER 32915SH3 VAUCLUSE.GPJ <<DrawingFile>> 14/04/2021 15:07 10.01.00.01 D:\git\Lab and in Situ\Tool - DGD\Lib JK 0.024 2019-05-31 Pj1



JK Geotechnics

Job No: 32915SH3

Borehole No: 201

Depth: 1.18m - 6.06m



JOB NO. 32915SH3 BH201 START CORING AT 1.18m

1

2

3

4

5

6

END OF BH AT 6.06m

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL

Project: PROPOSED CAR / BUS PARKING STRUCTURE

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

Job No.: 32915SH3

Method: SPIRAL AUGER

R.L. Surface: ~53.2 m

Date: 25/3/21

Datum:

Plant Type: JK205

Logged/Checked By: B.A./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING ON 25/3/21						53			-	CONCRETE: 120mm.t	M			7mm DIA. REINFORCEMENT 65mm TOP COVER
					N = 4 3,2,2		1			FILL: Sand, fine to medium grained, brown, with fine to coarse grained sandstone gravel, trace of silt fines.				APPEARS POORLY COMPACTED
						52			SM	Silty SAND: fine to medium grained, brown.	M	L		AEOLIAN
					N = 18 4,11,7		2		SC	Clayey SAND: fine to medium grained, light grey and red brown, trace of ironstone gravel, and silt fines.		MD		RESIDUAL
						51			-	SANDSTONE: fine to medium grained, brown.	DW	M - H		HAWKESBURY SANDSTONE
						50	3			REFER TO CORED BOREHOLE LOG				HIGH 'TC' BIT RESISTANCE
						49	4							GROUNDWATER MONITORING WELL INSTALLED TO 9.08m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.08m TO 9.08m. CASING 0.1m TO 3.08m. 2mm SAND FILTER PACK 0.5m TO 9.08m. BENTONITE SEAL 0.1m TO 0.5m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						48	5							
						47	6							

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Core Size:** NMLC **R.L. Surface:** ~53.2 m
Date: 25/3/21 **Inclination:** VERTICAL **Datum:**
Plant Type: JK205 **Bearing:** N/A **Logged/Checked By:** B.A./A.J.H.

Water Loss/Level	Barrel Lift	RL (m)	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 $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
										Specific	General	
					START CORING AT 2.87m							
ON COMPLETION OF CORING	100% RETURN		3		SANDSTONE: fine to medium grained, grey, brown and yellow brown, cross bedded at up to 22°.	MW	M - H	1.5			(3.16m) Be, 0°, P, R, Cn	Hawkesbury Sandstone
			4		SANDSTONE: fine to medium grained, light brown and brown, bedded at 0-10°.	SW		1.0				
			5					1.0			(4.82m) J, 90°, Ir, R, Clay FILLED	
			6		SANDSTONE: fine to medium grained, light grey and yellow brown, bedded at 0-15°.			0.90			(5.10m) Be, 15°, St, R, Clay Ct (5.11m) Be, 17°, C, R, Clay Ct	
			7					1.2			(5.33m) Be, 15°, P, R, Fe Sn	
			8					0.80			(5.55m) Be, 5°, P, R, Fe Sn	
			9		as above, but light grey and light yellow brown, with occasional carbonaceous lenses.	FR		1.0			(6.94m) Be, 15°, P, R, Clay Ct (6.95m) Be, 15°, P, R, Clay Ct (6.96m) Be, 15°, P, R, Clay Ct	
								0.70			(7.55m) Be, 3°, Ir, R, Clay Ct	
								0.60			(7.77m) Be, 15°, P, R, Clay Ct	
								1.0			(8.48m) Be, 2°, P, R, Clay Ct	
					END OF BOREHOLE AT 9.08 m			0.90			(8.77m) Be, 4°, P, R, Clay Ct	
								1.1				



Job No: 32915SH3
Borehole No: 202
Depth: 2.87m - 9.08m



JOB NO. 32915SH3 BH202 START CORING AT 2.87m

2

3

4

5

6

7

8

9

END OF BH AT 9.08m

BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Method:** SPIRAL AUGER **R.L. Surface:** ~49.4 m
Date: 25/3/21 **Datum:**
Plant Type: JK205 **Logged/Checked By:** B.A./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING ON COMPLETION OF CORING						49			-	CONCRETE: 90mm.t FILL: Gravelly sand, fine to medium grained, brown, fine to coarse grained igneous, ironstone and sandstone gravel.	M			NO OBSERVED REINFORCEMENT APPEARS POORLY COMPACTED
					N = 3 3,1,2		1			FILL: Silty sand, fine to medium grained, dark brown, trace of fine to coarse grained ironstone and sandstone gravel, trace of plastic and glass fragments, slag and organic matter.				
					N > 8 1,1,7/ 100mm REFUSAL		48							
							2		-	Extremely Weathered sandstone: clayey SAND, fine to medium grained, brown and yellow brown, trace of silt.	XW	D		HAWKESBURY SANDSTONE
							47			SANDSTONE: fine to medium grained, light grey and orange brown.	DW	M		MODERATE TO HIGH 'TC' BIT RESISTANCE
							3			REFER TO CORED BOREHOLE LOG				
							46							
							4							
							45							
							5							
							44							
							6							
							43							

CORED BOREHOLE LOG

Client: KINCOPPAL - ROSE BAY SCHOOL
Project: PROPOSED CAR / BUS PARKING STRUCTURE
Location: CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW

Job No.: 32915SH3 **Core Size:** NMLC **R.L. Surface:** ~49.4 m
Date: 25/3/21 **Inclination:** VERTICAL **Datum:**
Plant Type: JK205 **Bearing:** N/A **Logged/Checked By:** B.A./A.J.H.

Water Level	Barrel Lift	RL (m)	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 $I_p(50)$	DEFECT DETAILS		Formation
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	
								VL-0.1 L-0.3 M-1 H-3 VH-10 EH	600 200 60 20	Specific General	
		47			START CORING AT 2.84m						
			3		SANDSTONE: fine to medium grained, brown and light brown, bedded at 0-5°	HW	M	0.50			
		46						0.40			
			4		as above, but light grey and light yellow brown.	SW		0.70			
		45						0.60			
			5			FR	M - H	0.90		(4.02m) XWS, 0°, 15mm.t (4.06m) Be, 0°, P, S, Fe, Sn (4.41m) Be, 4°, Ir, R, Fe, Sn (4.51m) Be, 0°, P, R, Fe, Sn (4.62m) J, 32°, P, R, Cn (4.65m) Jh, 22°, Ir (4.71m) XWS, 0°, 20-30mm.t	
		44						1.1			
			6		END OF BOREHOLE AT 5.91 m			0.90			
		43									
			7								
		42									
			8								
		41									



Job No: 32915SH3
Borehole No: 203
Depth: 2.84m-5.91m



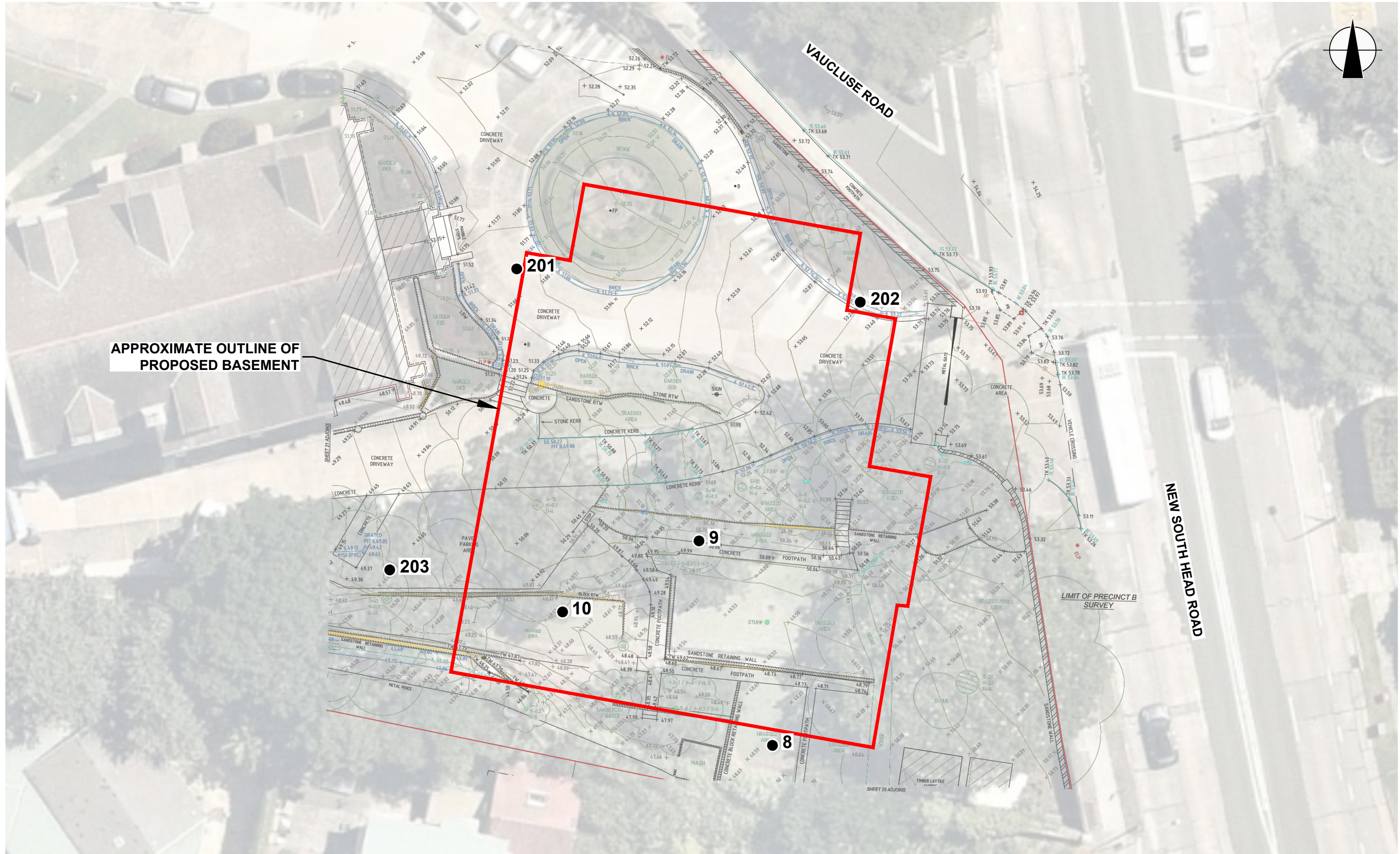
JOB NO 32915SH3 BH203 START CORING AT 2.84m





DYNAMIC CONE PENETRATION TEST RESULTS

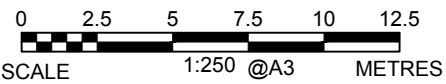
Client:	KINCOPPAL - ROSE BAY SCHOOL						
Project:	PROPOSED DEVELOPMENTS AT KINCOPPAL ROSE BAY SCHOOL						
Location:	CNR NEW SOUTH HEAD ROAD & VAUCLUSE ROAD, VAUCLUSE, NSW						
Job No.	32915PH3			Hammer Weight & Drop: 9kg/510mm			
Date:	3-2-20			Rod Diameter: 16mm			
Tested By:	D.A.F.			Point Diameter: 20mm			
Test Location	8	9	10				
Surface RL							
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	3	3	3				
100 - 200	4	3	2				
200 - 300	4	2	8				
300 - 400	8	1	11				
400 - 500	8	2	18/80mm				
500 - 600	12	3	REFUSAL				
600 - 700	5	3					
700 - 800	1	3					
800 - 900	1	10/50mm					
900 - 1000	↓	REFUSAL					
1000 - 1100	1						
1100 - 1200	↓						
1200 - 1300	1						
1300 - 1400	↓						
1400 - 1500	1						
1500 - 1600	↓						
1600 - 1700	1						
1700 - 1800	3/40mm						
1800 - 1900	REFUSAL						
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						



APPROXIMATE OUTLINE OF
PROPOSED BASEMENT

- NOTES:
1. BOREHOLES 8, 9 AND 10 ARE FROM OUR 2020 INVESTIGATION.
 2. BOREHOLES 201, 202 AND 203 ARE FROM THE CURRENT INVESTIGATION.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM



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:

32915SH3

Figure:

2

JKGeotechnics



VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite ‘safe’, depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are ‘safe limits’, up to which no damage due to vibration effects has been observed for the particular class of building. ‘Damage’ is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the ‘safe limits’, then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the ‘safe limits’ are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

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_0), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), 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_0).

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 60% 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	

Laboratory Classification Criteria

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

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10} D_{60}}$$

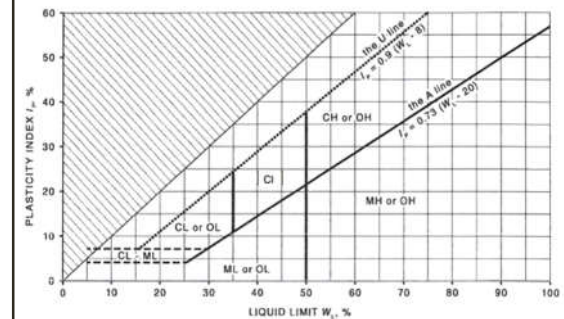
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

- 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.
- 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.
- Clay soils with liquid limits $> 35\%$ and $\leq 50\%$ may be classified as being of medium plasticity.
- The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
fine 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	–	–	–	–

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) (Coarse Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
	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)	VL	VERY LOOSE
	L	LOOSE
	MD	MEDIUM DENSE
	D	DENSE
	VD	VERY DENSE
	()	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.
	250	



Log Column	Symbol	Definition
Remarks	'V' bit 'TC' bit T_{60} Soil Origin	<p>Hardened steel 'V' shaped bit.</p> <p>Twin pronged tungsten carbide bit.</p> <p>Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.</p> <p>The geological origin of the soil can generally be described as:</p> <p>RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.</p> <p>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.</p> <p>ALLUVIAL – soil deposited by creeks and rivers.</p> <p>ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.</p> <p>MARINE – soil deposited in a marine environment.</p> <p>AEOLIAN – soil carried and deposited by wind.</p> <p>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.</p> <p>LITTORAL – beach deposited soil.</p>

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

Abbreviations Used in Defect Description

Cored Borehole Log Column	Symbol Abbreviation	Description
Point Load Strength Index	• 0.6	Axial point load strength index test result (MPa)
	x 0.6	Diametral point load strength index test result (MPa)
Defect Details – Type	Be	Parting – bedding or cleavage
	CS	Clay seam
	Cr	Crushed/sheared seam or zone
	J	Joint
	Jh	Healed joint
	Ji	Incipient joint
	XWS	Extremely weathered seam
	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	P	Planar
	C	Curved
	Un	Undulating
	St	Stepped
	Ir	Irregular
	Vr	Very rough
	R	Rough
	S	Smooth
	Po	Polished
	SI	Slickensided
	Ca	Calcite
	Cb	Carbonaceous
	Clay	Clay
	Fe	Iron
	Qz	Quartz
	Py	Pyrite
	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
	mm.t	Defect thickness measured in millimetres