REPORT TO

HEALTH INFRASTRUCTURE NSW

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED OPERATING THEATRE UPGRADE

AT

SUTHERLAND HOSPITAL
CORNER OF KINGSWAY & KAREENA ROAD
CARINGBAH, NSW

Date: 9 June 2020
Ref: 33141LXrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed operating theatre upgrade at Sutherland Hospital on the corner of The Kingsway & Kareena Road, Caringbah, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Health Infrastructure NSW by Letter of Award (Contract No. HI20115, dated 14 April 2020) and was carried out in accordance with our fee proposal (Ref: P51464-Rev3, dated 6 April 2020).

Douglas Partners have completed geotechnical investigations to the north-east of the site in 2014. Th relevant boreholes from that previous investigation have also been utilised, and are included in Appendix B.

We have been supplied with the following information:

- Architectural drawings prepared by HDR Architects (Project No: 10192314, Drawing Nos: AR-SK-3015, Revision 2, dated 22 April 2020).
- Request for Tender document prepared by Health Infrastructure NSW (Ref: HI20115, dated 17 March 2020)

Based on the provided information, we understand the operating theatre upgrade will comprise a three-storey western extension to the existing main hospital building with the ground floor level at, or close to the ground floor levels of the existing hospital. It is likely reconstruction of sections of the existing 'Carpark 3' would be required as part of the works.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on site preparation, methodologies for compaction and re-use of existing in-situ materials, excavation, retention, footings, groundwater, earthquake design parameters and soil aggression.

2 INVESTIGATION PROCEDURE

The investigation was carried out on the 7 and 8 May 2020 and comprised the following:

- Four boreholes drilled using a spiral auger to depths of between about 4.2m (BH1 and BH2) and 5.8m (BH3 and BH4) below existing surface levels.
- Each borehole was then extended by diamond core drilling to final depths ranging from 7.22m (BH1) and 8.86m (BH4).

The boreholes were drilled using our track mounted JK305 drilling rig. The borehole locations, as shown on the attached investigation location plan (Figure 2), and the surface reduced levels (RL's) shown on the attached borehole logs were recorded by a Sokkia satellite GPS unit and the accuracy is of the order of ±20mm. The survey datum is the Australian Heights Datum (AHD).

The strengths of the fill and natural clayey soils were assessed from the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on cohesive samples obtained in the SPT split spoon sampler. The strength of the initial augered portion of the bedrock profile was assessed from observation of drilling resistance when using a Tungsten Carbide ('TC') bit, examination of the recovered rock chips and subsequent laboratory moisture content testing. The strength of the cored portion of the weathered bedrock was assessed by examination of the recovered rock core and correlation with subsequent Point Load Strength Index tests.

Groundwater observations were made in the boreholes during drilling and, on completion of auger and core drilling. In BH2, BH3 and BH4, groundwater monitoring wells were installed on completion of drilling and groundwater measurements were made during the fieldwork and approximately 1 month after the completion of the fieldwork. We note that water is introduced as part of the coring process and therefore groundwater measurements immediately following core drilling of the boreholes is often artificially high.

Our geotechnical engineer (Mr Kartik Singh) was present on a full-time basis during the fieldwork to set out the boreholes, log the encountered subsurface profile, nominate in-situ testing and sampling and direct the installation of the standpipes. The borehole logs (which also include field test results, Point Load Strength Index test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used. For more details of the investigation procedures, reference should be made to the attached Report Explanation Notes.

Selected soil samples were returned to a NATA accredited laboratory, Soil Test Services Pty Ltd (STS), for moisture content, Atterberg limits & linear shrinkage testing and Standard compaction and four day soaked CBR testing. The test results are summarised in the attached STS Tables A and B. The recovered rock core was also returned to STS where it was photographed and Point Load Strength Index tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are presented in the attached STS Table C. Soil samples were also sent to a NATA registered laboratory (Envirolab Services Pty Ltd) for soil pH, chloride, sulphate and resistivity testing and the results are presented in Appendix A 'Certificate of Analysis' attached to this report.

As part of the geotechnical investigation JK Environments (JKE), our specialist environmental division, completed a preliminary waste classification and contamination screening of selected soil samples. The results are presented in a separate report (Ref. E33141PA dated 5 June 2020).

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on a local hillside which grades to the south at approximately 3˚ in the area of the site. The site itself is relatively level and is located at the western end of Sutherland Hospital. More specifically the site is located (and includes) the eastern portions of Carpark 3 and its surrounds, the western end of the existing emergency building and COVID 19/Flu Assessment clinic, and the concrete and asphalt paved access roads and foot path which divide the carpark and building.

Carpark 3, located adjacent to Kareena Road, which forms the western boundary of the site, is an asphaltic concrete (AC) pavement which appears in good condition. The carpark was retained by a small height concrete block retaining wall which ranged between 0.5m to 0.9m high. A grassed area was located immediately south of the carpark with an apparent fill embankment approximately 0.5-1.0m high lining the full extent of the southern perimeter of the carpark. This grassed embankment contained scattered medium to large trees.

East of Carpark 3 is an asphaltic concrete paved internal road and a concrete paved ambulance bay. The concreted section which mainly serviced the ambulance bay appeared to be in good condition. The surrounding AC pavements however were noted to be in fair condition with localised sections of rutting and associated crocodile cracking observed particularly around the entrance to the COVID 19/Flu Assessment clinic.

The existing hospital building, and existing ambulance depot to the south of the site comprised one and two storey buildings respectively which appeared to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Wollongong indicates the site is close to the contact between a capping unit comprising claystone, siltstone and laminate and the underlying Hawkesbury Sandstone.

The boreholes disclosed a generalised subsurface profile comprising shallow to moderately deep fill overlying residual clays, with weathered siltstone and then sandstone at depth. For details of the encountered subsurface profile, reference should be made to the attached borehole logs. A summary of the subsurface conditions encountered in the boreholes is presented below.

Pavement

A concrete pavement of 150mm thickness was encountered at the surface in BH1, and the concrete was underlain by a 200mm thickness of sand/gravel (roadbase). An asphaltic concrete (AC) pavement of 60mm thickness was encountered at the surface in BH2 and BH3 and was underlain by a 50mm (BH2) and 440mm (BH3) thickness of sandy gravel (roadbase). BH4 was carried out through the grass embankment on the southern side of Carpark 3 and had no surface pavements. Clayey sand fill was encountered below a concrete pavement in DP-BH4 and an AC pavement DP-BH7 and extended to depths of 1.2m and 0.4m respectively. Shale bedrock was present immediately below the fill in DP-BH7.

Fill

Silty clay fill of medium plasticity was encountered below the pavement in BH1, BH2 and BH3 and from the surface in BH4, extending to depths ranging between 0.8m (BH3) and 1.4m (BH4). The silty clay fill was assessed to be variable and to range from poorly compacted to well compacted.

Residual Soils

Residual silty clay was encountered beneath the fill in each borehole and extended to depths ranging from 2.1 (BH1) to 3.05 (BH3). The clays were assessed to be of medium plasticity and were of very stiff to hard strength. We note that previous laboratory testing by Douglas partners in the nearby BH7 assessed the residual clays as high plasticity. Therefore, some variability in the plasticity should be expected.

Bedrock

Bedrock comprised siltstone overlying sandstone, with the top of siltstone encountered at depths ranging from 2.1m (BH1) to 3.05m (BH3). Sandstone was not encountered within boreholes DP-BH4 and DP-BH7 and was likely present below the borehole termination depths. The siltstone ranged from extremely weathered and hard (soil) strength to very low or low strength and extended to depths ranging from about 5.8m (BH3) and 7.3m (BH2). The siltstone within DP-BH4 and DP-BH7 improved to medium strength with depth however significant core loss zones were encountered within the medium strength shale. The underlying sandstone encountered in BH1 to BH4 bedrock ranged from low to medium strength on first contact improving to medium or high strength with depth. A band of very low strength siltstone of about 300mm thickness was encountered at the base of BH3. Defects within the core typically comprised horizontal bedding partings, extremely weathered/clay seams ranging from 20mm to 140mm thickness and occasional inclined and subvertical joints. Bedrock classification based on Pells et al (1998) is provided in Section 3.4 below.

Groundwater

The boreholes were 'dry' during, and on completion of auger drilling. Standing water levels on completion of drilling ranged between 2.2m (BH3 and BH4) and 4.15m (BH1) however, as water was introduced as part of the coring process, the measured levels are unlikely to be representative of true groundwater levels. Full water-flush returns were recorded whilst core drilling which have been interpreted to indicate a relatively low permeability rock mass. On 1 June 2020 (about 1 month after drilling), groundwater levels were measured at depths of 4.15m (RL36.7m), 3.15m (RL37.7m) and 3.9m (RL38.0m) within the monitoring wells installed within BH2, BH3 and BH4, respectively. This indicates a general groundwater gradient down to the south-east.

3.3 Laboratory Test Results

The moisture content testing completed on recovered rock chips, collected whilst auger drilling, correlated well with our field assessment of the rock strength. The moisture content testing completed within the residual clay samples taken from BH1 and BH2 returned values of 10.7% and 25.3%, respectively. Based on the Liquid Limit and Linear Shrinkage determinations the residual silty clay samples from BH1 and BH2 were of medium plasticity, and were assessed to have a moderate potential for shrink/swell reactivity with changes in moisture content.

The four-day soaked CBR tests on silty clay fill samples from BH3 and BH4 returned CBR values of 17% and 6% respectively when compacted to 98% of their respective Standard Maximum Dry Density (SMDD) at about their respective Standard Optimum Moisture Content (SOMC). The higher 17% test is higher than we would expect fopr such a material and we suspect that the result may have been affected by gravel within the sample. The in-situ moisture contents of the samples were 4.7% (BH3) and 3.2% (BH4) 'dry' of SOMC. Samples exhibited a 0% (BH3) or 0.5% (BH4) swell following the four-day soaking period.

The results of the laboratory point load strength index tests generally correlated well with our field assessment of the in-situ bedrock strength. The Unconfined Compressive Strength (UCS) of the upper siltstone generally ranged from 2MPa to 8MPa and the underlying sandstone ranged from 14MPa to 36MPa, as estimated from the Point Load Strength Index tests.

A summary of the laboratory chemical test results is provided in the table below:

Borehole	Sample Depth (m)	Description	pH Units	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity Ohm.cm
BH1	0.5 - 0.8	Silty Clay Fill	9.6	140	10	6,100
BH1	2.1 - 2.5	VL/L Siltstone	5.7	25	<10	43,000
BH2	1.5 – 1.95	Silty Clay	4.8	74	78	7,400
BH3	3 - 3.05	XW Siltstone	5.8	110	<10	11,000
BH4	1.5 - 1.95	Silty Clay	4.7	240	61	5,100

The above results, indicate that in accordance with Table 6.4.2(C) and Table 6.5.2 (C) of AS2159-2009 – Pile Design and Installation.

- The clayey soils may be classified as having a 'Mild' and 'Non-aggressive' exposure classification for concrete and steel, respectively,
- The siltstone may be classified as having a Non-aggressive exposure classification for both concrete and steel.

3.4 Bedrock Classification

Based on the borehole logs and laboratory test results, the bedrock classifications in accordance with the Pells *et* al (1998) system, in the table below, apply to the siltstone/sandstone bedrock encountered.

Borehole	Approximate Depth Interval (m) / RL (mAHD)		
	Class V (Shale)	Class IV/III (Sandstone)	Class III (Shale)
1	2.1 – 6.25[1] / 39.0 – 34.87	6.25 – 7.22 / 34.87 – 33.9	
2	3.8 – 7.40[1] / 37.1 – 33.45	7.40 – 8.38 / 33.57 – 32.45	
3	4.1 – 5.78[1] / 37.8 – 36.11	5.78 – 8.84 / 36.11 – 33.05	
4	2.6 – 6.35[1] / 39.49 – 35.74	6.35 – 8.86 / 35.74 – 33.23	
DP-BH4	1.2 – 5.5 / 40.3 – 36.0		5.5 – 6.1 / 36.0 – 35.4
DP-BH7	1.6 – 5.2 / 39.6 – 36.0		5.2 – 6.1 / 36.0 – 35.1

[1]Augered portion of bedrock profile assumed, not classified using the Pells et al (1998) system

It should be noted that the above general classification is based on arbitrary lengths of core and does not relate to footing widths or zone of influence as required to determine bearing capacities. The sandstone encountered in BH2 was of lower initial strength than the other boreholes and only a very small length of likely Class III medium strength sandstone bedrock was encountered at the bottom of the borehole.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

The following comments and recommendations should be complemented by reference to AS3798-2007: 'Guidelines on Earthworks for Commercial and Residential Developments'.

In existing pavement areas, all existing asphaltic concrete and concrete pavements should be stripped from the site. In existing grass areas, all grass and other vegetation, and topsoil/root affected soils should be stripped. For all trees, the root balls must be thoroughly grubbed out at this stage. No particular topsoil profile was encountered in any of the boreholes, however, for budgeting purpose, we recommend an allowance be made for a nominal topsoil thickness in all grass surfaced areas. Following stripping these upper surface pavement and grass areas, any deleterious or contaminated existing fill exposed should also be removed and stockpiled separately for site disposal.

Stripped topsoil/root affected soils must be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes. Reference should be made to the environmental report by JK Environments (E33141PA) for guidance on the offsite disposal of soil.

Based on the investigation results, a basecourse layer was encountered below the pavement areas (although it was quite thin at BH2). Where possible this layer should be kept in place as it will provide a more trafficable pavement for plant and equipment. Where this layer must be removed then it is likely that a clayey subgrade will be exposed and such a material is expected to undergo a loss in strength when wet. Furthermore, the clay subgrade is expected to have a moderate shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untrafficable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

4.2 Excavation

Prior to any excavation we recommend that information be obtained on the footing system of any adjoining structures. This information should be passed on to the geotechnical and structural engineers, along with the proposed depths of excavation, so that footings of adjoining structures are not undermined by any excavation.

Significant excavation is not expected for the proposed development. Any minor excavation will require removal of the existing concrete and asphaltic concrete pavements and the underlying soils. Due to the close proximity of existing structures, we do not recommend the use of hydraulic impact hammers to remove concrete pavements. We recommend that concrete pavements be saw cut into manageable pieces and lifted out using the bucket of an excavator. Any soil excavation will be able to be undertaken using the buckets of conventional earth moving equipment.

4.3 Batters and Retaining Walls

We assume that sufficient space is available on site for the formation of temporary batters and construction of any retaining walls at the base of the batters as required for long term support. If this is not the case and insufficient space is available for batters or deeper excavations are proposed additional geotechnical advice should be obtained, but the use of retention systems installed prior to excavation and additional lateral support will be required for such walls.

Temporary batters should be no steeper than 1 Vertical in 1 Horizontal (1V:1H), for the relatively minor excavations envisaged. Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters. Permanent batters should be no steeper than 1V:2H, but flatter batters in the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent batters to also reduce erosion.

Permanent gravity retaining walls of less than about 2m in height may be designed as cantilevered walls based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.35 and a bulk unit weight of $20kN/m^3$, assuming some resulting movements are tolerable. Where walls are restricted from some lateral movements, such as by other structural elements in front of the wall, or where movements are to be reduced, a triangular distribution and an 'at rest' earth pressure coefficient, K_0, of 0.6 should be used.

The above coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic loads, unless measures are undertaken to provide complete and permanent drainage behind the wall.

Where gravity retaining walls are adopted, care will need to be exercised in backfilling between the temporary batter slope and the new retaining wall. Uncontrolled backfilling will lead to large settlements which may adversely affect pavements, structures or landscaping areas. It is often difficult to achieve adequate compaction of backfill due to limited access and the need to use small hand compaction equipment. We therefore recommend the use of a single-sized durable gravel, such as "blue metal" gravel or crushed concrete (free of fines and with less than 10% brick), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer between the cut batter slope and the gravel backfill to control subsoil erosion. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required. The geotextile should then be wrapped over the surface of the gravel backfill and capped with at least a 0.5m thick compacted layer of clayey engineered fill.

4.4 Footings

Pile Footings

Given the anticipated column loads for the proposed new structures, piles uniformly founded on the underlying shale or sandstone bedrock will be required. We consider that bored piles will be suitable.

Bored piles socketed a minimum of 0.3m into the appropriate stratum may be designed based on the parameters outlined in the table below. For rock sockets longer than this 0.3m, the shaft frictions outlined in the table below may be adopted provided the socket is satisfactorily cleaned and roughened to roughness category R2 or better.

Rock Unit (Class)	Allowable End Bearing Pressure (kPa)	Allowable Shaft Friction (kPa) Compression	Ultimate End Bearing Pressure (kPa)	Ultimate Shaft Friction (kPa) Compression	Elastic Modulus (MPa)
Class V Siltstone	700	50	3,000	100	150
Class IV/III Sandstone	3500	350	15,000	800	700

NOTES

1. Allowable bearing pressures assume a settlement of approximately 1% of the least footing dimension for footings in rock.
2. Ultimate bearing pressures assume a settlement of approximately 5% of the least footing dimension for footings in rock.
3. The Allowable and Ultimate values for Class IV/III in the table above should only be adopted where additional geotechnical investigations confirm a uniform founding strata below the site.

For piles designed using ultimate values, an appropriate "*Geotechnical Strength Reduction Factor*" (ϕ_g), as defined in Clause 4.3.1 of AS2159-2009 ('Piling – Design and Installation') must be used. Settlement limitations will also need to be satisfied and can be estimated using the Elastic Modulus values in the table above. At this stage the borehole results indicate that there is a reasonably uniform layer of sandstone bedrock equivalent to Class IV/III rock. However, if this layer is to be utilised as a founding stratum, we recommend some further geotechnical investigations be carried out to confirm its consistency across the site, particularly in the area of BH2. Further geotechnical investigations may enable even higher bearing pressures to be adopted which would assist in more economical footing design.

Based on our investigation and the expected piling depths required, allowance should be made to account for some groundwater seepage/inflow whilst piling. Pouring of concrete using tremie methods to prevent concrete segregation from 'drop pouring' onto water is likely to be required. At least the initial stages of bored pile drilling should be inspected by a geotechnical engineer to confirm the groundwater conditions and that the appropriate foundation material is being achieved. Where the higher bearing pressures associated with Class IV/III are adopted all piles should be inspected by a geotechnical engineer during drilling.

Shallow Footings

For smaller structures, e.g. low height retaining walls or light poles in the carpark, footings in the soil profile could be considered.

For high level footings founded in natural silty clays of at least very stiff strength, an allowable bearing pressure of 150kPa can be adopted. We note that the clayey soils on site (both natural and fill materials) are reactive to moisture content change, as evidenced from the Linear Shrinkage Limits tests. We therefore recommend that any shallow footings be designed to be able to accommodate movements similar to a Class 'M' site as detailed in AS2870-2011: 'Residential Slabs and Footings'. Where such structures adjoining existing buildings or new structures founded on the underlying bedrock, then suitable movement control joints should eb adopted.

We recommend that all high level footings be excavated, cleaned out, inspected and poured with minimum delay to avoid deterioration. If delays in pouring concrete are anticipated, we recommend that the base of the footings be protected with a layer of blinding concrete. Water should be prevented from ponding in the base of footing excavations as this will tend to soften the foundation material, resulting in further excavation and cleaning required.

4.5 Ground Floor Slabs

The ground floor slabs for the proposed new building will overlie a soil profile which will be subject to shrink-swell movements similar to that encountered on a 'Class M' site. Our experience on other hospital projects is also that ground floor slabs may be required to support sensitive hospital equipment where excessive slab movements would be detrimental. Therefore, due to the presence of these reactive soils and uncontrolled fill', we recommend that ground floor slabs be designed as fully suspended structures founded on piled footings to rock. The ground floor slabs should be underlain by a void former of at least 100mm thickness to reduce the risk that reactive clays will 'jack' slabs or footing beam off piers. Where floor slabs are designed as suspended, only light compaction such as by track rolling would be required to form a stable base for formwork and reinforcement.

4.6 Car Park Subgrade Preparation

Following the above stripping works and completion of excavation down to design subgrade level, we recommend that the exposed soil subgrade over any proposed car park areas be proof rolled with at least eight passes of a smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer or earthworks technician for the detection of unstable or soft areas.

If subgrade heaving is detected during proof rolling, then the heaving areas should be locally removed down to a stable base and replaced with engineered fill, as outlined in Section 4.7 below or further geotechnical advice should be sought. Further guidance on the treatment of heaving areas must be provided by the geotechnical engineer during the proof rolling inspection. Based on the investigation results, we do not expect significant heaving areas to be encountered provided the remaining earthworks are carried out during dry weather and not immediately following a period of wet weather.

If soil softening occurs after prolonged rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clayey subgrade exhibits shrinkage cracking, then the surface should be watered and rolled until the shrinkage cracks are no longer evident.

4.7 Engineered Fill

Engineered fill must be used where ground surface levels are to be raised.

The excavated natural silty clay may be re-used as engineered fill, on the condition the materials are "clean", free of organic matter and free of particle sizes greater than 75mm. The re-use of the fill materials is acceptable from a geotechnical perspective, but will be subject to approval from a contamination perspective by the environmental engineers.

All clayey fill should be compacted in maximum 200mm thick loose layers to a density ratio strictly between 98% and 102% of SMDD and within 2% of SOMC. We expect that some moisture conditioning of the clay soils on site will be required in order to meet this compaction specification. Engineered fill comprising well graded granular materials, such as crushed sandstone, should be compacted in maximum 200mm thick loose layers to achieve a density ratio of at least 98% of SMDD. The compaction specification (for both clayey and granular engineered fill) may be relaxed to achieve a density ratio of at least 95% of SMDD, if the engineered fill is within grassed or garden areas.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. All density testing must be completed over the full thickness of each compacted fill layer.

- The frequency of density testing for engineered fill should be at least one test per layer per 500m^2 or one test per 100m^3 distributed reasonably evenly throughout the full depth and area, whichever requires the most tests.
- The frequency of density testing for engineered backfill behind retaining walls and trenches should be at least one test per two layers per 50 linear m.

Due to potential conflict of interest, the geotechnical inspection and testing authority (GITA) should be directly engaged by the client and not by the earthworks contractor or sub-contractors.

4.8 Car Park Construction

Based on the investigation results, we recommend that the proposed flexible pavement for the car park be designed on the basis of a CBR value of 5% or an estimated modulus of subgrade reaction of 30kPa/mm (750mm plate) to take into account the inherent variability of the fill, and provided that the subgrade is prepared as per our advice above. Where engineered fill is placed to raise site levels the thickness and soaked CBR of the engineered fill should be taken into account in the pavement design.

Subsoil drains should be provided along the edges of the proposed pavements, with invert levels at least 300mm below subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.9 Earthquake Design Parameters

Based on the results of the investigation, the following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 ('Structural Design Actions, Part 4: Earthquake Actions in Australia'):

- Hazard Factor (Z) = 0.09
- Site Subsoil Class = Class C_e

4.10 Further Geotechnical Input

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

- Additional drilling to determine the nature and extent of poorer quality siltstone encountered during current geotechnical investigation and the uniformity of the better quality sandstone bedrock for adoption of the higher bearing pressures.
- Proof rolling inspections.
- Density testing of all engineered fill, sub-base and base course materials.
- Geotechnical inspection of footing excavations/pile drilling.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

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

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33141LX
Project:	Proposed Operating Theatre Upgrade	**Report:**	A
Location:	Cnr Kingsway & Kareena Road, Caringbah, NSW	**Report Date:**	28/05/2020
		Page 1 of 1	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	1.50 - 1.95	10.7	37	18	19	6.5
1	2.10 - 2.50	8.1	-	-	-	-
1	3.00 - 3.50	8.5	-	-	-	-
2	0.50 - 0.95	25.3	44	21	23	9.5
2	3.80 - 4.00	10.5	-	-	-	-
3	4.10 - 4.50	7.7	-	-	-	-
3	5.00 - 5.50	13.6	-	-	-	-
4	2.60 - 3.00	11.7	-	-	-	-
4	4.00 - 4.50	6.9	-	-	-	-
4	5.00 - 5.50	7.6	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 19/05/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

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the items tested or sampled.

28/05/2020
Authorised Signature / Date
(D. Treweek)

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33141LX
Project:	Proposed Operating Theatre Upgrade	**Report:**	B
Location:	Cnr Kingsway & Kareena Road, Caringbah, NSW	**Report Date:**	1/06/2020
		Page 1 of 1	

BOREHOLE NUMBER		BH 3	BH 4
DEPTH (m)		0.50 - 0.80	0.10 - 1.00
Surcharge (kg)		4.5	4.5
Maximum Dry Density (t/m^3)		1.83 STD	1.79 STD
Optimum Moisture Content (%)		15.3	16.4
Moulded Dry Density (t/m^3)		1.79	1.75
Sample Density Ratio (%)		98	98
Sample Moisture Ratio (%)		102	97
Moisture Contents			
Insitu (%)		10.6	13.2
Moulded (%)		15.5	15.9
After soaking and			
After Test, Top 30mm(%)		17.9	20.1
	Remaining Depth (%)	16.0	18.6
Material Retained on 19mm Sieve (%)		0	0
Swell (%)		0.0	0.5
C.B.R. value:	@2.5mm penetration	17	6

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: 19/05/2020.
- BH 3 had insufficient material provided so compaction curve was recycled.

NATA

NATA Accredited Laboratory Number:1327

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01/06/2020

Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33141LX
Project:	Proposed Operating Theatre Upgrade	**Report:**	C
Location:	Cnr Kingsway & Kareena Road, Caringbah, NSW	**Report Date:**	13/05/2020
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	4.89 - 4.92	0.2	4
	6.05 - 6.10	0.2	4
	6.39 - 6.42	0.4	8
	6.71 - 6.75	0.8	16
	7.12 - 7.16	0.7	14
2	5.42 - 5.46	0.1	2
	5.75 - 5.79	0.1	2
	6.70 - 6.73	0.3	6
	7.10 - 7.14	0.04	1
	7.42 - 7.46	0.3	6
	7.73 - 7.76	0.2	4
	8.18 - 8.22	0.8	16
3	5.88 - 5.92	1.5	30
	6.35 - 6.40	1.1	22
	6.84 - 6.88	1.3	26
	7.21 - 7.25	1.8	36
	7.66 - 7.69	1.3	26
	8.28 - 8.33	0.8	16
4	6.40 - 6.43	1.0	20
	6.77 - 6.80	1.1	22
	7.19 - 7.24	0.9	18
	7.68 - 7.71	1.2	24
	8.12 - 8.16	0.9	18
	8.70 - 8.74	1.4	28

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 IS (50)

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CERTIFICATE OF ANALYSIS 242650

Client Details

Client	JK Geotechnics
Attention	Kartik Singh
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33141LX, Caringbah**
Number of Samples	5 Soil
Date samples received	11/05/2020
Date completed instructions received	11/05/2020

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	18/05/2020
Date of Issue	14/05/2020

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		242650-1	242650-2	242650-3	242650-4	242650-5
Your Reference	UNITS	1	1	2	3	4
Depth		0.5-0.82	2.1-2.5	1.5-195	3-3.05	1.5-1.95
Date Sampled		07/05/2020	07/05/2020	07/05/2020	08/05/2020	08/05/2020
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	12/05/2020	12/05/2020	12/05/2020	12/05/2020	12/05/2020
Date analysed	-	12/05/2020	12/05/2020	12/05/2020	12/05/2020	12/05/2020
pH 1:5 soil:water	pH Units	9.6	5.7	4.8	5.8	4.7
Chloride, Cl 1:5 soil:water	mg/kg	10	<10	78	<10	61
Sulphate, SO4 1:5 soil:water	mg/kg	140	25	74	110	240
Resistivity in soil*	ohm m	61	430	74	110	51

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			12/05/2020	3	12/05/2020	12/05/2020		12/05/2020	[NT]
Date analysed	-			12/05/2020	3	12/05/2020	12/05/2020		12/05/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	3	4.8	4.7	2	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	78	88	12	96	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	3	74	72	3	99	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	3	74	67	10	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

JKGeotechnics

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX **Method:** SPIRAL AUGER **R.L. Surface:** 41.12 m
Date: 7/5/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** K.K.S./A.F.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			41	0			CONCRETE: 150mm.t				18mm DIA. REINFORCEMENT 75mm TOP COVER
						-	FILL: Sandy gravel, fine to coarse grained, brown, sub-angular igneous gravel.	M			
			40	1			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sub-angular igneous gravel, fine to medium grained sand.	w<PL			
		N = 19 3,3,16				CI	Silty CLAY: high medium plasticity, orange brown and grey, trace of fine to medium grained ironstone gravel	w<PL	Hd	>600 >600 >600	RESIDUAL
			39	2		-	SILTSTONE: grey and brown, with iron indurated bands.	DW	VL - L		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE WITH VERY LOW BANDS
ON COMPLETION			38	3							
			37	4							
			36	5			REFER TO CORED BOREHOLE LOG				
			35	6							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 1

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

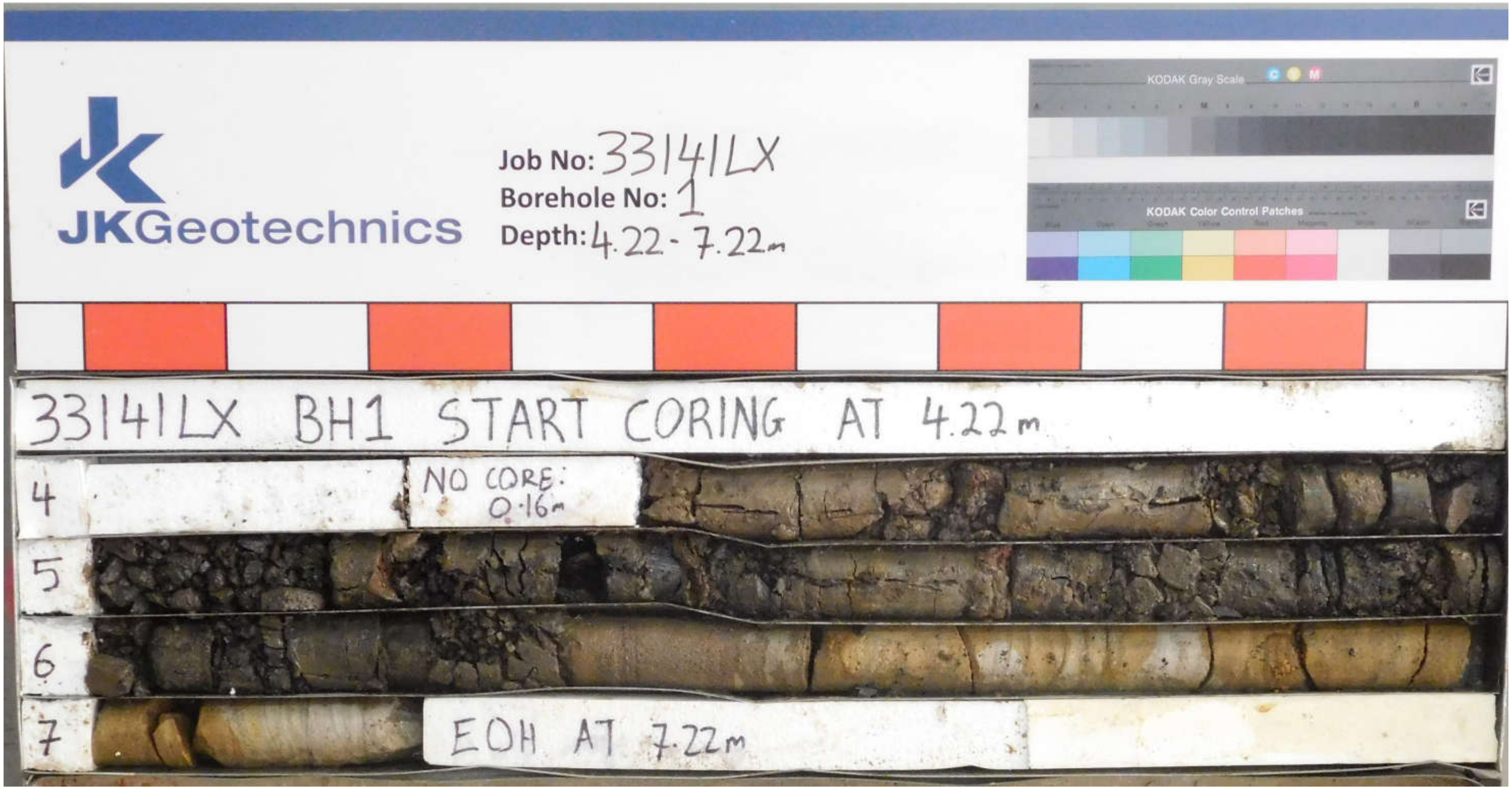
Job No.: 33141LX	**Core Size:** NMLC	**R.L. Surface:** 41.12 m
Date: 7/5/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** K.K.S./A.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: Specific	Formation
		37			START CORING AT 4.22m					
90% RETURN					NO CORE 0.16m					Hawkesbury Sandstone
					Extremely Weathered siltstone: silty CLAY, high plasticity, grey and orange brown.	XW	Hd			
		36	5		SILTSTONE: dark grey, with iron indurated bands, bedded at 0-10°.	MW	L	0.20		
					Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey and orange brown, with iron indurated bands and clay bands.	XW	Hd			
		35	6		SILTSTONE: dark grey, bedded at 0-10°.	MW	VL	0.20	(5.70m) XWS, 0°, 100 mm.t (5.80m) CS, 0°, 60 mm.t (5.89m) Be, 0°, Un, R, Cn (5.94m) XWS, 0°, 100 mm.t (6.09m) CS, 0°, 50 mm.t (6.18m) J, 80 - 90°, R, Cn (6.25m) XWS, 0°, 110 mm.t	
		34	7		SANDSTONE: fine to medium grained, bedded at 0-20°.	MW	M	0.40 0.80	(6.89m) J, 35°, Un, R, Cn	
					SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.	FR	H	0.70		
					END OF BOREHOLE AT 7.22 m					
		33	8							
		32	9							
		31	10							

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

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33141LX CARINGBAH.GPJ <<DrawingFile>> 02/06/2020 13:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33141LX
Borehole No: 1
Depth: 4.22 - 7.22m
KODAK Gray Scale
KODAK Color Control Patches
33141LX BH1 START CORING AT 4.22m
4
NO CORE: 0.16m
5
6
7
EOH AT 7.22m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX **Method:** SPIRAL AUGER **R.L. Surface:** 40.85 m
Date: 7/5/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** K.K.S./A.F.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		0	-	ASPHALTIC CONCRETE: 60mm.t				
				FILL: Sandy gravel, fine to medium grained, grey, fine to medium grained sub-angular igneous gravel.	M			APPEARS MODERATELY COMPACTED
	N = 8 2,4,4			FILL: Silty clay, medium plasticity, grey and brown, trace of fine to medium grained sub-angular igneous and ironstone gravel, trace of fine to medium grained sand.	w<PL		110 180 120	
	N = 19 7,7,12	1	CI	Silty CLAY: high medium plasticity, orange brown and grey, trace of fine to medium grained ironstone gravel	w<PL	Hd	>600 >600 >600	RESIDUAL
		2.5	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey and orange brown, with iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE
ON COMPLETION		3.7		SILTSTONE: dark grey and brown, with iron indurated bands.	DW	VL - L		LOW RESISTANCE WITH VERY LOW BANDS
1 MONTH LATER		4.2		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.38m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 5.38m TO 8.38m. CASING 0m TO 5.38m. 2mm SAND FILTER PACK 5.05m TO 8.38m. BENTONITE SEAL 4.0m TO 5.05m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33141LX CARINGBAH.GPJ <<DrawingFile>> 02/06/2020 13:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

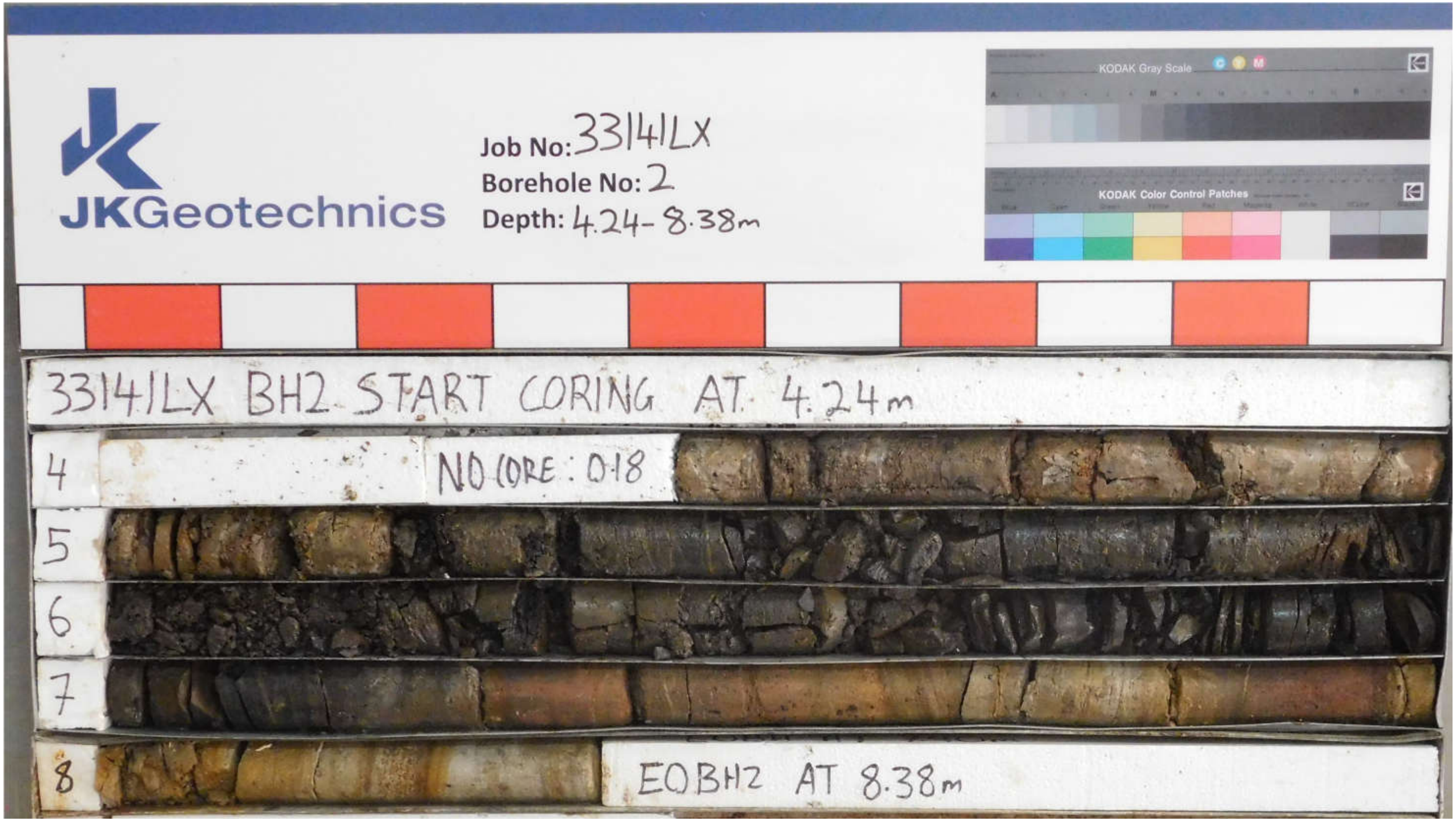
CORED BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX | **Core Size:** NMLC | **R.L. Surface:** 40.85 m
Date: 7/5/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** K.K.S./A.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
					START CORING AT 4.24m						
90% RETURN					NO CORE 0.18m						Hawkesbury Sandstone
		36	5		Extremely Weathered siltstone: silty CLAY, high plasticity, grey and brown, with iron indurated bands.	XW	Hd				
		35	6		SILTSTONE: dark grey, with iron indurated bands, bedded at 0-10°.	SW	VL	0.10 0.10		(5.20m) XWS, 0°, 50 mm.t (5.46m) XWS, 0°, 140 mm.t (5.64m) J, 80 - 90°, Un, R, Cn (5.78m) XWS, 0°, 20 mm.t (5.90m) J, 55°, St, R, Fe Sn	
					Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey and brown, with iron indurated bands.	XW	Hd				
		34	7		SILTSTONE: dark grey, bedded at 0-10°.	SW	L	0.30 0.040		(6.76m) XWS, 0°, 120 mm.t (6.96m) Be, 0°, Un, R, Cn (7.02m) Be, 0°, Un, R, Cn (7.08m) J, 50°, P, R, Fe Sn (7.28m) Be, 0°, Un, R, Fe Sn	
		33	8		SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-10°.	HW	L M	0.30 0.20 0.80		(7.38m) Be, 0°, Un, S, Clay (7.80m) Be, 0°, Un, R, Fe Sn (7.89m) Ji, 90°, Un, Cn (7.97m) Be, 0°, Un, R, Cn (8.05m) J, 75°, Un, Cn	
		32 31 30	9 10		END OF BOREHOLE AT 8.38 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33141LX
Borehole No: 2
Depth: 4.24-8.38m
KODAK Gray Scale
KODAK Color Control Patches
33141LX BH2 START CORING AT 4.24m
4
NO CORE : 0.18
5
6
7
8
EOBH2 AT 8.38m

JKGeotechnics

BOREHOLE LOG

Borehole No. 3

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX **Method:** SPIRAL AUGER **R.L. Surface:** 41.89 m
Date: 8/5/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** K.K.S./A.F.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		0		ASPHALTIC CONCRETE: 60mm.t				
			-	FILL: Sandy gravel, fine to coarse grained, grey, sub-angular igneous gravel.	M			APPEARS WELL COMPACTED
	N = 19 16,12,7			FILL: Silty clay, medium plasticity, brown, with fine to medium grained sub-angular igneous gravel.	w<PL			
			CH	Silty CLAY: high plasticity, orange brown and grey, trace of fine to coarse grained ironstone gravel.		VSt	230 260 250	RESIDUAL
	N = 29 12,13,16					Hd	>600 >600 >600	
ON COMPLETION								
	N=SPT 12/ 50mm REFUSAL	3	-	Extremely Weathered siltstone: silty clay, high plasticity, grey and orange brown, with iron indurated and clay bands.	XW	Hd		HAWKESBURY SANDSTONE SOIL RESISTANCE WITH BANDS OF VERY LOW 'TC' BIT RESISTANCE
1 MONTH LATER				SILTSTONE: dark grey, with iron indurated bands.	DW	VL - L		LOW RESISTANCE WITH VERY LOW BANDS
		5.7		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.84m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 5.84m TO 8.84m. CASING 0m TO 5.84. 2mm SAND FILTER PACK 5.3m TO 8.84m. BENTONITE SEAL 4.3m TO 5.3m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

RL (m AHD): 41, 40, 39, 38, 37, 36, 35
Depth (m): 1, 2, 3, 4, 5, 6

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33141LX CARINGBAH.GPJ <<DrawingFile>> 02/06/2020 13:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
3

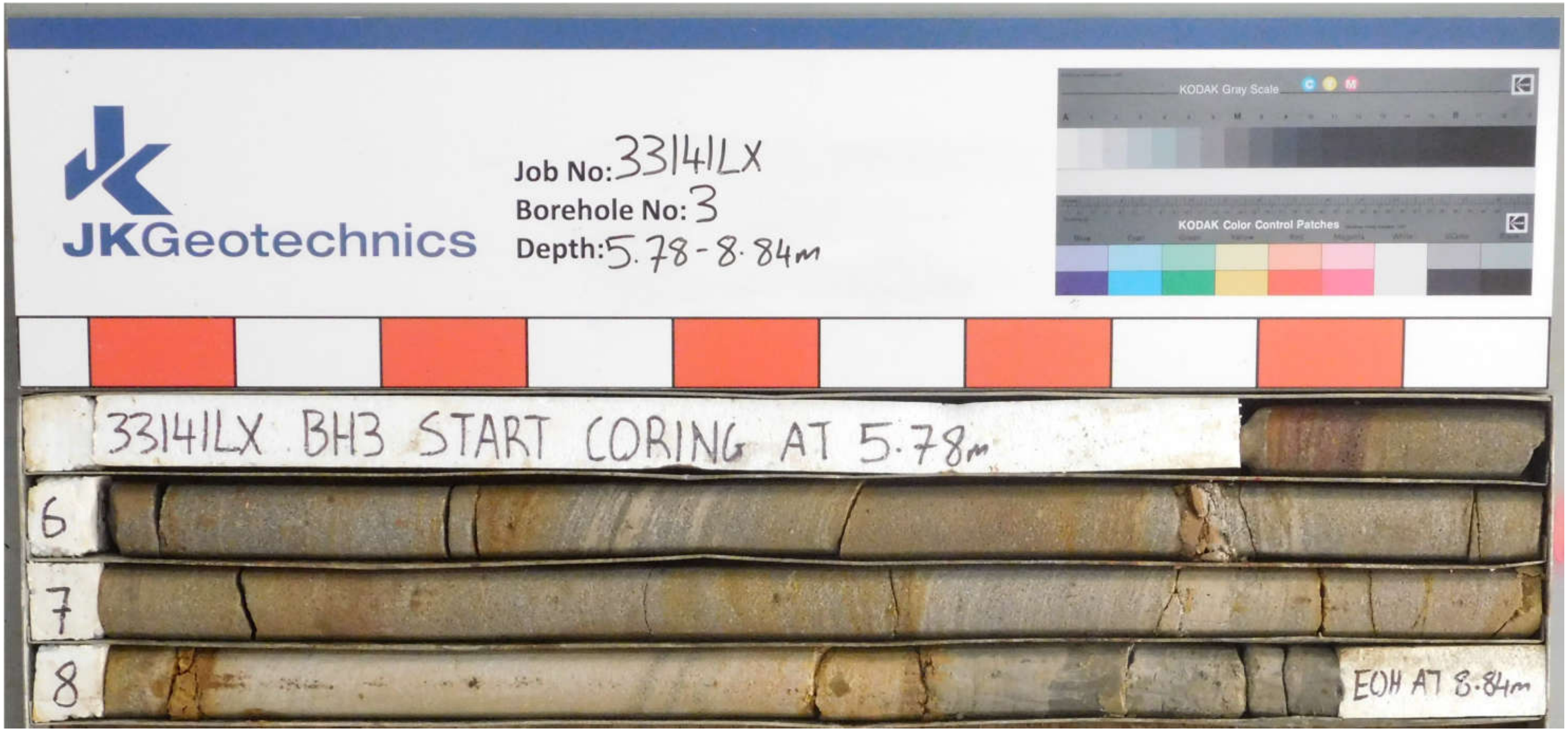
Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX | **Core Size:** NMLC | **R.L. Surface:** 41.89 m
Date: 8/5/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** K.K.S./A.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	General	Formation
					START CORING AT 5.78m						
90% RETURN		36	6		SANDSTONE: fine to medium grained, grey and orange brown, bedded at 0-10°.	HW / SW	L / M - H	1.5			Hawkesbury Sandstone
					as above, but bedded at 0-15°.			1.1			
		35	7		as above, but bedded t 0-10°.			1.3, 1.8, 1.3	(6.73m) CS, 0°, 40 mm.t (6.79m) Be, 10°, Un, R, Fe Sn		
		34	8					0.80	(7.84m) Be, 0°, Un, R, Fe Sn (7.88m) Be, 0°, Un, R, Fe Sn (8.05m) CS, 0°, 20 mm.t (8.51m) Be, 5°, Un, R, Fe Sn (8.56m) Be, 0°, Un, S, Cn		
					SILTSTONE: grey and dark grey, with sandstone lenses, bedded at 0-10°	HW	VL				
		33	9		END OF BOREHOLE AT 8.84 m						
		32	10								
		31	11								
		30									

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33141LX
Borehole No: 3
Depth: 5.78 - 8.84m
KODAK Gray Scale
KODAK Color Control Patches
33141LX BH3 START CORING AT 5.78m
6
7
8
EOH AT 8.84m

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1 / 2

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

Job No.: 33141LX **Method:** SPIRAL AUGER **R.L. Surface:** 42.09 m
Date: 8/5/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** K.K.S./A.F.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		N = 11 5,5,6	42 41	1			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sub-angular igneous gravel, fine to medium grained sand, brick fragments.	w<PL			GRASS COVER APPEARS POORLY COMPACTED
ON COMPLETION		N = 17 4,7,10	40	2		CH	Silty CLAY: high plasticity, orange brown and grey, trace of fine to medium grained ironstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
			39	3		-	SILTSTONE: grey and brown, with iron indurated bands.	DW	VL		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
1 MONTH LATER			38 37	4 5			as above, but dark grey.		VL - L		LOW RESISTANCE WITH VERY LOW BANDS
			36	6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.86m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.36m TO 8.86m. CASING 0m TO 4.36. 2mm SAND FILTER PACK 3.95m TO 8.86m. BENTONITE SEAL 2.5m TO 3.95m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
4

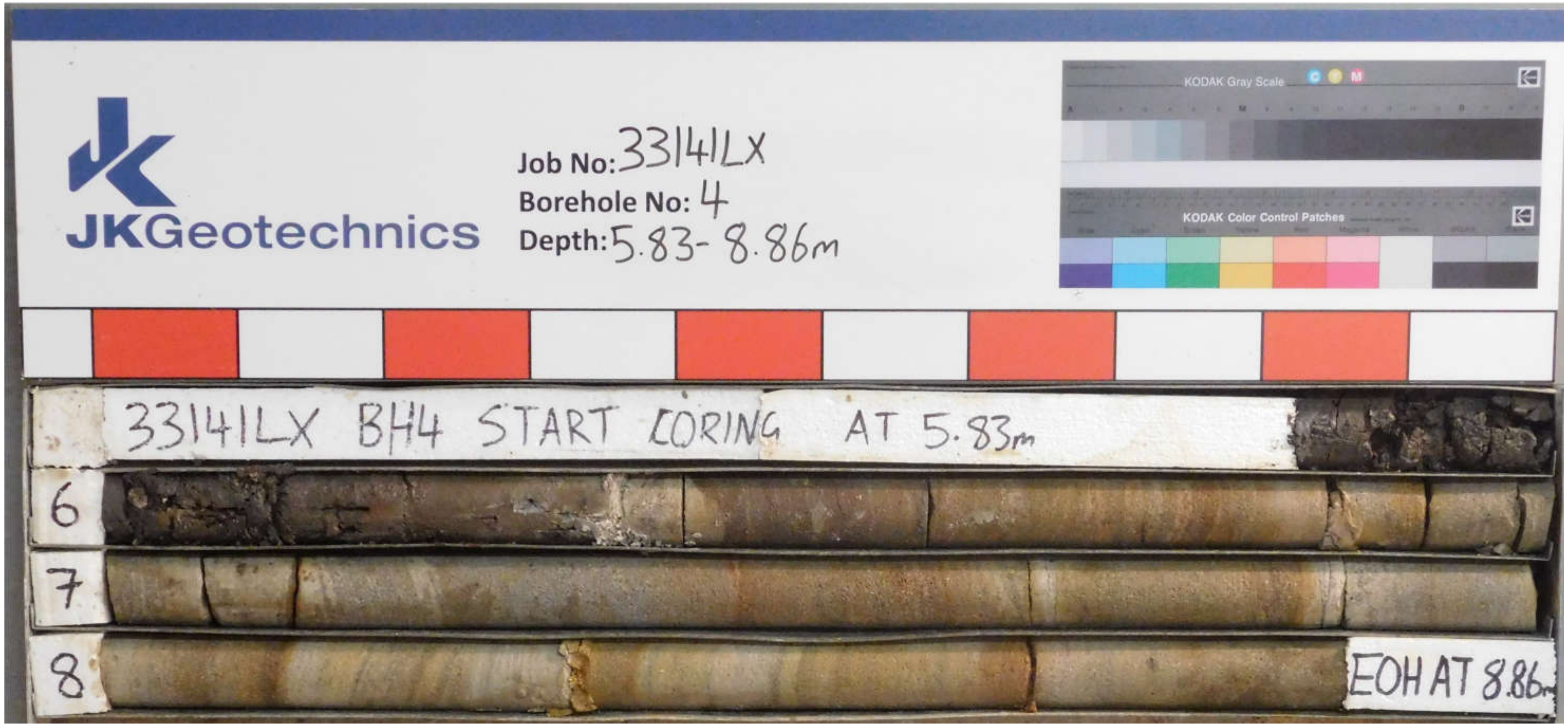
Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED OPERATING THEATRE UPGRADE
Location: CNR KINGSWAY & KAREENA ROAD, CARINGBAH, NSW

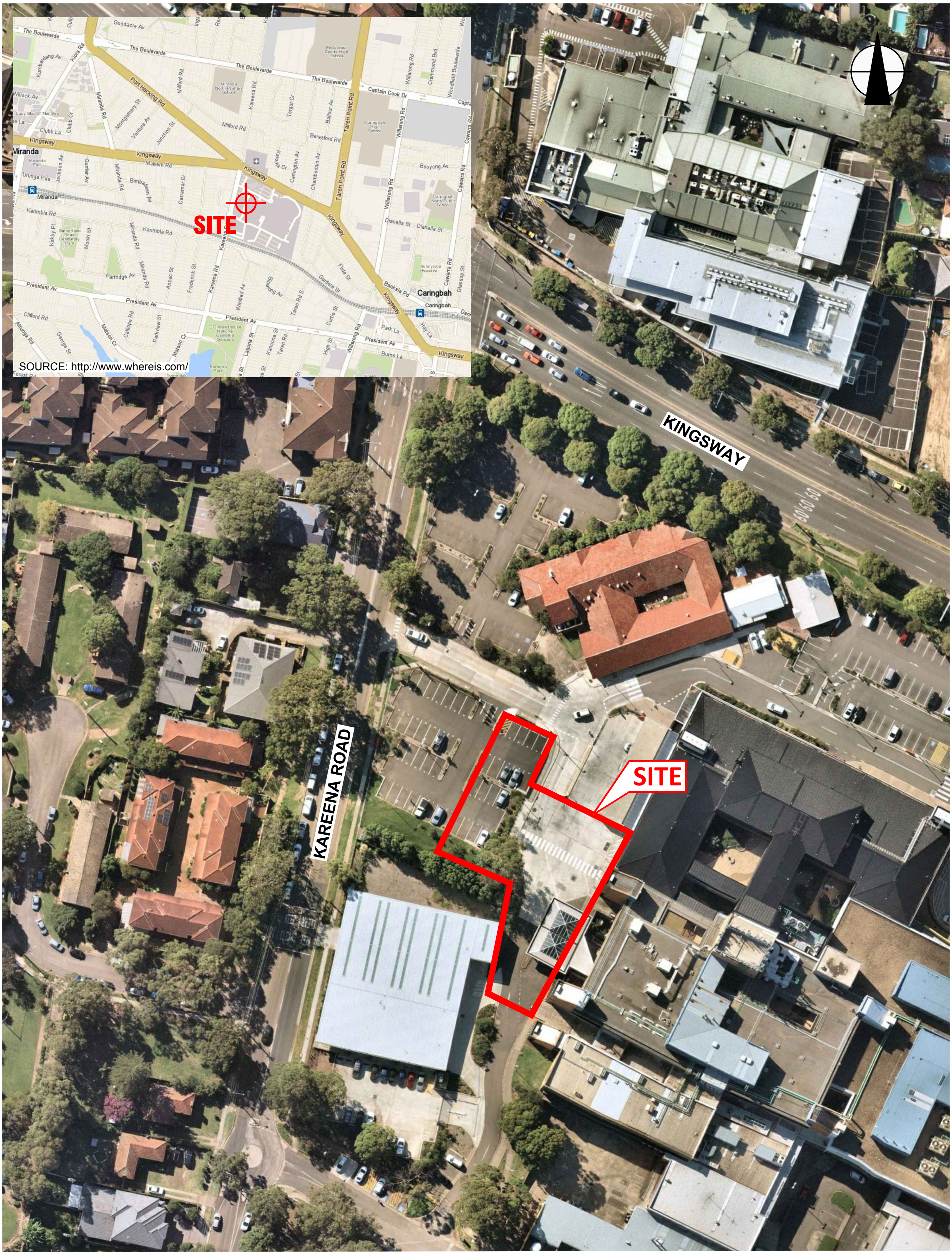
Job No.: 33141LX **Core Size:** NMLC **R.L. Surface:** 42.09 m
Date: 8/5/20 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** K.K.S./A.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Core Description (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Point Load Strength Index $I_s(50)$	Defect Details – Description (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) – Specific	Formation
		37		START CORING AT 5.83m					
90% RETURN		36	6	Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey, with iron indurated and clay bands.	XW	Hd			Hawkesbury Sandstone
		35	7	SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-10°.	MW	M - H	1.0 1.1 0.90 1.2	(6.84m) CS, 0°, 15 mm.t (6.91m) Be, 0°, Un, R, Fe Sn (7.14m) Be, 0°, Un, R, Fe Sn (7.64m) Be, 0°, Un, R, Fe Sn	
		34	8	as above, but bedded at 0-15°.			0.90 1.4	(8.33m) XWS, 0°, 10 mm.t (8.65m) Be, 0°, P, S, Fe Sn	
		33	9	END OF BOREHOLE AT 8.86 m					
		32	10						
		31	11						

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20
Defect details: Specific / General

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33141LX
Borehole No: 4
Depth: 5.83- 8.86m
KODAK Gray Scale
KODAK Color Control Patches
33141LX BH4 START CORING AT 5.83m
6
7
8
EOH AT 8.86m



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	CNR KINGSWAY AND KAREENA ROAD, CARINGBAH, NSW
Report No:	33141LX
Figure No:	1

JKGeotechnics

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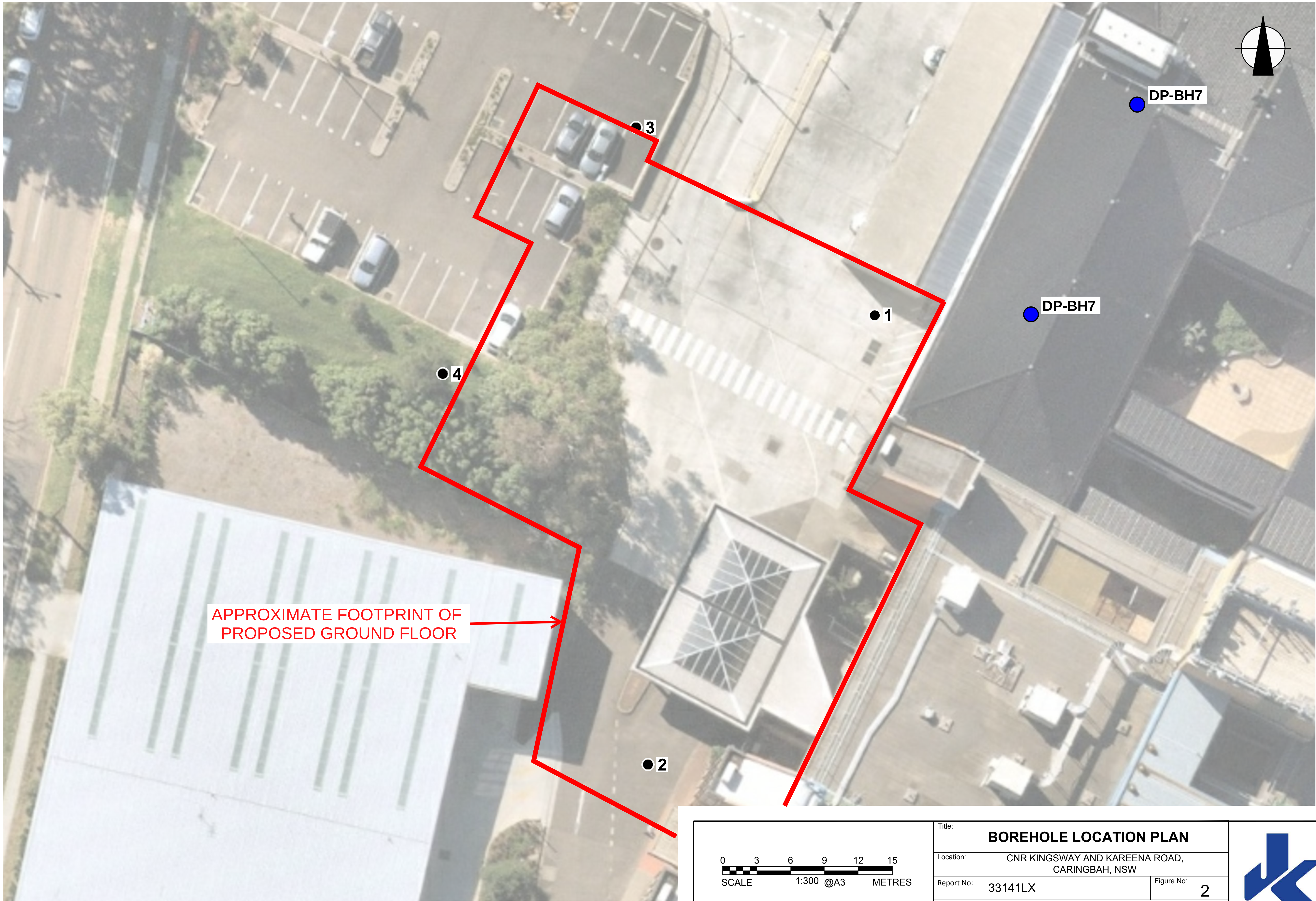


0 3 6 9 12 15
SCALE 1:300 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: CNR KINGSWAY AND KAREENA ROAD, CARINGBAH, NSW

Report No: 33141LX

Figure No: 2

JKGeotechnics

PLOT DATE: 13/05/2020 1:34:32 PM DWG FILE: Y:\33000's\33141LX SUTHERLAND HOSPITAL\CAD\33141LX.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or
iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

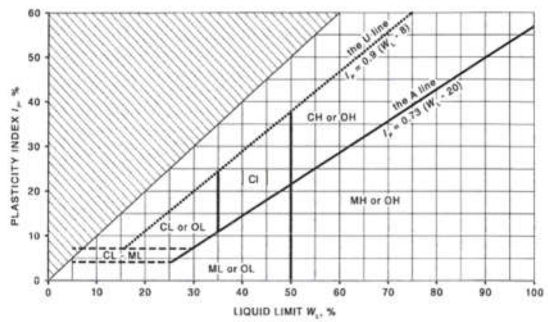
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $Is_{(50)}$ (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
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

APPENDIX A

BOREHOLE LOG

CLIENT: Health Infrastructure
PROJECT: Sutherland Hospital Planning Project
LOCATION: Kareena Road, Caringbah

SURFACE LEVEL: 41.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 84311
DATE: 11/6/2014
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.14	CONCRETE					
0.3	FILLING - grey, medium-grained gravelly sand filling, damp		D			
1.2	FILLING - brown, fine to medium-grained clayey sand filling with brick and concrete fragments, moist to damp		D			
			D			
			S			3,5,10 N = 15
2.9	SHALE - very low strength, red and orange-grey shale	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			16,25/100mm refusal
4.34	SHALE - extremely low and very low strength, extremely to highly weathered, fractured, grey and red-grey shale with some medium strength bands	3.4m: J75°, cln, ro, 100mm 3.73m: J90°, 40mm, cly co, 1-2mm 4.0-4.07m: B0°, cly co, 1-2mm	C	100	0	PL(A) = 0.6
			C	100	0	
5.5	SHALE - medium strength, highly and moderately weathered, fragmented to fractured, grey-brown shale with some extremely low strength bands	4.11m: J85°, cln, ro 4.14m: J30°, cln, sm 4.25m: CORE LOSS: 90mm 4.82m: CORE LOSS: 680mm	C	50	0	PL(A) = 0.4
6.1	6.0-6.1m: high strength sandstone band	5.72m: J90°, cln, ro, 50mm	C	83	0	PL(A) = 0.7 PL(A) = 2.6
	Bore discontinued at 6.1m					

Column headings: RL; Depth (m); Description of Strata; Degree of Weathering (EW, HW, MW, SW, FS, FR); Graphic Log; Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High); Water; Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00); Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault); Sampling & In Situ Testing (Type, Core Rec. %, RQD %, Test Results & Comments)

RL scale: 41, 40, 39, 38, 37, 36, 35, 34, 33, 32
Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9

RIG: DT250 **DRILLER:** LC **LOGGED:** AL/SI **CASING:** HW to 2.9m
TYPE OF BORING: Diatube to 0.14m; Solid flight auger to 2.9m; NMLC-Coring to 6.1m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U,	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: Health Infrastructure
PROJECT: Sutherland Hospital Planning Project
LOCATION: Kareena Road, Caringbah

SURFACE LEVEL: 41.2 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 84311
DATE: 13/6/2014
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.04	ASPHALT		D			
0.25	FILLING - dark grey, fine to medium-grained sandy gravel filling, damp (roadbase)		D			
0.4	FILLING - brown, medium-grained clayey sand filling with sandstone and ironstone gravel, moist		D			
	CLAY - very stiff to hard, red and orange-grey clay with some ironstone gravel, moist		S			5,14,20 N = 34
1.6	SHALE - extremely low strength, grey and brown shale	Note: Unless otherwise stated, rock is fractured along planar bedding dipping at 0°- 10°	S			23,25/60mm refusal
2.9	SHALE - medium strength, highly weathered, fragmented, grey-brown shale with some extremely low and very low strength bands	2.8m: CORE LOSS: 100mm	C	80	0	PL(A) = 0.6
4.0	SHALE - medium strength, slightly weathered, fragmented to fractured, grey shale	3.68m: J90°, fe stn, ro, 40mm 4.25m: J90°, cln, sm, 40mm 4.37m: J60°, fe stn, sm	C	100	0	PL(A) = 0.5
5.2		4.9m: CORE LOSS: 300mm 5.3m: J90°, fe stn, ro, 60mm	C	75	0	PL(A) = 0.6
6.1	Bore discontinued at 6.1m					

RIG: DT250 **DRILLER:** LC **LOGGED:** AL/SI **CASING:** HW to 2.8m

TYPE OF BORING: Solid flight auger to 2.8m; NMLC-Coring to 6.1m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed in bore - refer to seperate log

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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