REPORT TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED CAMHS DEVELOPMENT

AT

NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Date: 17 May 2021

Ref: 33780LTrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Child and Adolescent Mental Health Service (CAMHS) development at Nepean Hospital, Derby Street, Kingswood, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Health Infrastructure and was carried out as a variation to our existing Health Infrastructure Consultancy Agreement (Contract No. HI16465, dated 1 October 2020), our fee proposal (Ref: P53288LT) dated 21 December 2020, and our variations for non-destructive digging dated 28 January 2021, and weekend work dated 25 February 2021.

We have been supplied with the following drawings and information:

- Architectural general arrangement drawings by Silver Thomas Hanley (Aus) Pty Ltd (Project No. 10442, Drawing Set. NHR-STH-DRW-ARC-MHS, Drawing Nos. A02-001[B], A10-001[A], A20-101[A], A20-301[A], A51-001[A] and A51-002[A] dated 10 February 2021 [Rev A] and 11 February 2021 [Rev B]).
- Preliminary column loads supplied by Meinhardt Bonacci dated 11 January 2021.

From the above information we understand that the CAMHS Building will comprise a two-storey structure. The ground floor will have a finished floor level at RL53.08m to tie in with surrounding surface levels. The extent of the proposed building is shown approximately on the attached Figure 2. Unfactored column loads are anticipated to be in the order of 1,600kN. Excavation to achieve the lowest level (Level 01) will be to maximum depths of about 1m at the western end, reducing towards the east, with the eastern third of the building to be above existing surface levels by about 1m. Locally deeper excavation will be required for the proposed lift pit. New pavements will be constructed around the eastern and northern sides of the new building.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention, bearing pressures for footings, and potential settlements.

2 INVESTIGATION PROCEDURE

The fieldwork for the current investigation comprised the drilling of four boreholes (BH601 to BH604) to depths ranging from 9.1m to 11.55m below existing surface levels, using our track-mounted drilling rig. All boreholes were initially advanced through the soils and upper weathered bedrock using spiral auger drilling techniques and a Tungsten Carbide (TC) bit. The boreholes were then extended to the final depths by rotary diamond coring techniques, using an NMLC triple tube core barrel and water flush.

Prior to commencement of the fieldwork, the investigation locations were electromagnetically scanned by a specialist subcontractor so that borehole locations could be located clear of buried services. The services scan was also completed by referencing the 'Dial Before You Dig' plans. Safe work measures and procedures were implemented during the course of the fieldwork. At each location, non-destructive digging with vacuum excavation was completed to visually assess whether any services were present at the borehole locations.

At three locations, trenches were vacuum excavated to allow information on the upper soils to be obtained. However, at one location (BH601), due to the presence of deeper than expected concrete, the upper 1.2m of the borehole was vacuum excavated and therefore no details on the strength of the soils in this initial portion of the profile were obtained.

The borehole locations are shown on the attached Figure 2, and these were set out by taped measurements from existing surface features shown on the survey plan. The approximate reduced level (RL) at each borehole location, as shown on the borehole logs, was interpolated from spot heights and contours from the supplied survey plan prepared by Cardno (Drawing No. 118117502, Revision 05, dated 2 March 2018). The height datum is Australian Height Datum.

The apparent compaction of the fill and strength of the cohesive soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer tests carried out on cohesive samples recovered by the SPT split tube sampler. The strength of the bedrock in the augered portion was assessed from observation of drilling resistance using the TC drill bit attached to the augers, tactile examination of rock cuttings, and correlation with the results of subsequent laboratory moisture content tests. It should be noted that strengths assessed in this way are approximate and variances of at least one strength order should not be unexpected.

For the cored portion of the bedrock, the recovered core was returned to our laboratory for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations, the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is_{50} results. These Point Load Strength test results are summarised in the attached Table C and on the borehole logs.

Selected soil samples were also returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories. STS completed moisture content, Atterberg limits and CBR testing. The results of these tests are provided in the attached STS Tables A and B. Soil aggression testing was completed by Envirolab Services Pty Ltd and the results are provided in the attached Certificate of Analysis No. 266594.

Groundwater observations were recorded in all boreholes during and on completion of auger drilling. Standpipe piezometers were installed in BH601 and BH603 to allow for longer-term groundwater monitoring. A further site visit was conducted on 22 April 2021 to measure the groundwater levels in BH601 and BH603. No further groundwater monitoring has been carried out.

Our geotechnical engineers were present on a full-time basis during the fieldwork, to nominate testing and sampling and prepare the borehole logs. The borehole logs, which include field test results and groundwater observations, are attached, together with a set of explanatory notes which describe the investigation techniques, and their limitations and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently undulating topography defined by slopes of 5° or less and is located near the crest of a low-height ridgeline orientated north-south and roughly followed by Parker Street/The Northern Road. Surface levels across the site slope down overall to the south-east at approximately 2°.

The site comprises an area, shown on Figure 1, within Nepean Hospital which encompasses the northern portion of the south-eastern building cluster of the Hospital. Surface levels across the site have been altered by the existing developments through retaining walls to create a relatively level area in and around the buildings. The existing buildings comprise a single-storey brick building (Nepean 1) within the western half of the site, and a one and two-storey metal clad building (Nepean 2) within the eastern half of the site. Both buildings appear to be in good condition based on cursory external observations. Adjacent to the external walls of the buildings are garden beds and pavements. Medium to large trees are scattered around the perimeter of the eastern half of the site. Set back about 2m from the western edge of Nepean 1 is a concrete ramp which provides access between the lower and upper levels of the Mental Health Centre.

The site is generally surrounded by adjacent areas within Nepean Hospital including Mental Health Centre to the west and NBMLHD Executive and Sexual Health to the south. An on-grade asphalt car park (Fleet Car Park) is situated adjacent to the south-east portion of the site. Surface levels across the boundaries into these surrounding areas of the Hospital are generally similar to those on site.

Along the northern boundary of the site is an asphalt paved internal driveway which provides access to the Emergency entrance. This road slopes down to the east at approximately 2° and appears to be in good condition.

3.2 Subsurface Conditions

The Penrith 1:100,000 Geological Series Sheet 9030 indicates that the site is underlain by Bringelly Shale of the Wianamatta Group consisting of *"shale, carbonaceous claystone, claystone, laminite, fine- to medium-grained lithic sandstone, rare coal and tuff".* This profile does not take into account in-situ weathering or any earthworks that have taken place on the site.

The investigation encountered a generalised profile comprising relatively shallow fill overlying residual silty clay which transitioned to weathered claystone bedrock at depths ranging from 1.9m to 2.5m. The bedrock is highly weathered in the upper portion however generally increases in quality with depth. Some of the more pertinent subsurface observations are discussed below, however for specific details reference should be made to the attached borehole logs.

Pavements and Fill

Concrete pavements were encountered at the surface in BH601, BH602 and BH604 with the concrete at BH602 and BH604 being 100mm and 90mm thick respectively. Pavements were not encountered at the surface of BH603.

Underlying the pavements and from the surface in BH603, fill was encountered and was generally shallow (0.5m to 0.6m) although deeper fill extending to 2.1m depth was encountered within BH601. The deeper fill within BH601 is likely due to backfilling behind the retaining wall to construct the adjoining ramp. The fill within the boreholes generally comprised silty clay with varying proportions of gravel. The fill contained inclusions of igneous, ironstone and sandstone gravel and slag. The fill, where assessed, appears to be poorly compacted. The clays within the fill were assessed to range from low to high plasticity.

Residual Silty Clay

Silty clay, assessed as residual in origin, was encountered below the fill in all boreholes except BH601. The clay was assessed as high plasticity and of very stiff to hard strength. The residual clay generally contained inclusions of ironstone gravel with the proportion of gravel generally appearing to increase with depth. Towards the bedrock horizon, the residual clays contained an extremely weathered structure.

Weathered Bedrock

Weathered bedrock, predominantly comprising claystone, was encountered at depths ranging from 1.9m (BH602) to 2.5m (BH603). The level of the bedrock ranged from approximately RL53.3m (BH601) to RL50.1m (BH603). The surface of the bedrock generally appears to dip down to the east. Although the bedrock predominantly comprised claystone, bands of fine-grained sandstone and laminite were encountered within each of the boreholes. The bedrock generally appears to improve in quality with depth with the upper portions of the rock containing significant quantities of defects as well as being more highly weathered.

The following table provides our rock classification assessment for BH601 to BH604 inclusive. The classification was completed in general accordance with Pells et al (2019). The rock classes are approximate only and will be dependent on footing/pile sizes. Some variability between the boreholes should be expected.

Borehole Number	Depths (Reduced Levels) Class V Rock	Depths (Reduced Levels) Class IV Rock	Depths (Reduced Levels) Class III Rock	Depths (Reduced Levels) Class II or Better Rock
601	2.1m to 2.6m (RL53.3 to RL52.8) Claystone	2.6m to 6.7m (RL52.8 to RL48.7) Claystone	7.7m to 9.1m (RL47.7 to RL46.3) Claystone	6.7m to 7.7m (RL48.7 to RL47.7) Sandstone
602	1.9m to 4.3m (RL51.7 to RL49.3) Claystone	4.3m to 5.2m (RL49.3 to RL48.4) Sandstone 5.2m to 8.1m (RL48.4 to RL45.5) Claystone	8.1m to 10.7m (RL45.5 to RL42.9) Claystone	Not encountered

603	2.5m to 5.3m (RL50.1 to RL47.3) Claystone	5.3m to 9.3m (RL47.3 to RL43.3) Claystone	9.3m to 11.0m (RL43.3 to RL41.6) Claystone 11.0m to 11.5m (RL41.6 to RL41.1) Sandstone	Not encountered
604	2.0m to 5.1m (RL50.8 to RL47.7) Claystone	6.5m to 9.4m (RL46.3 to RL43.4) Claystone	5.1m to 6.5m (RL47.7 to RL46.3) Laminite + Sandstone	Not encountered

Groundwater

All boreholes were dry on completion of auger drilling. We note that during the coring process, water is introduced into the borehole and therefore the water level recorded in BH601 at 3.7m on completion of coring is artificially high. Piezometer standpipes installed within BH601 and BH603 were measured on 22 April 2021 at depths of 3.98m and 4.13m (correlating with levels of RL48.6m and RL51.3m) respectively.

3.3 Laboratory Test Results

The Atterberg Limits testing completed on the residual silty clay and silty clay fill indicate they are of medium and high plasticity. The linear shrinkage results indicate a moderate to high potential for shrink-swell movements with changes in moisture content, with the residual clays being more reactive than the clay fill. The moisture contents of the clay were all below their respective plastic limits.

The four-day soaked CBR tests on the residual clay returned values of 3.0% and 4.0%. The clays are 1.7% and 2.8% 'wet' of their respective optimum moisture contents. During soaking, the samples swelled by 3.0% and 1.5% indicating the clays are reactive with respect to variations in moisture content.

The following table summarises the soil aggression tests.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity (ohm.cm)
BH601	1.8-1.95	Silty Clay FILL	8.0	390	620	1,200
BH602	1.5-1.9	Silty Clay RESIDUAL	8.1	340	240	2,600
BH604	2.2-2.5	Claystone BEDROCK	6.2	190	630	1,600

Based on these results, the soils would be classified as having a 'Non-aggressive' exposure classification for concrete piles and a 'Mild' exposure classification for steel piles in accordance with Table 6.4.2(C) and 6.5.2(C) of AS2159-2009 'Piling – Design and Installation.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Classification

Due to the depth of the fill and the likely abnormal moisture conditions as a result of buildings, pavements and trees, we consider that the proposed building area will classify as Class 'P' in accordance with AS2870-2011 'Residential Slabs and Footings'. Therefore, all footings will need to be designed by engineering principles.

The use of AS2870-2011 will not be relevant to the main building and will only apply to any lightly-loaded structures within the scope defined by the code. For such structures, the laboratory testing of the residual silty clay soils indicates that they will likely have characteristic surface movements in the range equivalent to that of a Class 'H1' site under "normal" conditions. Where footings are designed on the basis of AS2870-2011, consideration will also need to be given to the adverse effect on shrink-swell movements from trees which are scattered around the proposed development area.

If the residual silty clay soils are used as an engineered fill, or if excavations into the residual silty clays are carried out, then it is possible that characteristic surface movements will be greater, and may be closer to Class 'H2' type movements. As such further advice from the geotechnical engineers is recommended when design details and levels are known.

Reference should also be made to Appendix B of AS2870-2011, for guidance on appropriate site maintenance, including site drainage and planting of trees and shrubs.

4.2 Excavation Conditions

The following recommendations should be read in conjunction with the latest version of *'Excavation Work – Code of Practice'* prepared by SafeWork NSW.

The proposed ground floor level of the building is at RL53.08m which will require excavation to depths ranging up to about 1m at the western edge of the building, reducing towards the east with little or no excavation required over the eastern third of the building footprint. Locally deeper excavation will be required for the proposed lift.

Based on the investigation results, excavation to these depths will encounter the fill and residual soils, although the very upper profile of weathered bedrock may be encountered at the western end of the site. Excavation of the soils should be readily achievable using the buckets of hydraulic excavators. Locally deeper excavations e.g. for lift overruns, may encounter extremely weathered claystone though this too should be readily excavatable using the buckets of medium sized excavators.

Groundwater was not encountered during auger drilling in any of the boreholes however, groundwater was measured within the standpipe piezometers at depths of 3.98m and 4.13m which correlate with levels about 1.8m and 4.5m below the proposed Level 01 floor level. We note that the groundwater in these boreholes is unlikely to have stabilised in the short period after drilling, and as such these groundwater levels may be artificially high following coring. Notwithstanding, we do not anticipate that groundwater will be a significant issue given the relatively shallow depths of excavation and the anticipated low permeability of the silty clay. Some seepage may occur through the fill and may be encountered during piling and such flows will likely be controllable by conventional sump and pump techniques.

Seepage may need to be treated prior to disposal into stormwater systems and any requirements should be checked with the environmental and hydraulic consultants.

Material to be disposed of offsite will need to be suitably classified for waste disposal.

4.3 Excavation Batters

Based on the proposed excavation depths and offset of the excavation from adjacent structures there appears to be sufficient space to form temporary batter slopes. The feasibility of temporary batters should be checked based on the following recommendations once the building design and layout has been finalised.

- Temporary batter slopes through any clay fill material should be battered at not steeper than 1 Vertical (V) in 1.5 Horizontal (H). If any sandy fill is exposed temporary batter slopes will need to be flatter at probably 1V in 2H subject to inspection by the geotechnical engineers.
- Temporary batters through the residual clays and any of the upper weathered bedrock should be battered at not steeper than 1 Vertical (V) in 1 Horizontal (H). Seepage may occur at the fill/residual soil interface or at the toe of the batter.
- Surcharge loads, including adjoining buildings, construction loads etc, must be kept well clear of the crest of temporary batters (at least 2H from the crest, where H is the vertical height of the batter slope in metres).

Where temporary batters are formed, consideration needs to be given to the type of backfill to be used against the permanent retaining walls. Uncompacted backfill placed up against retaining walls will result in large settlements which can have adverse effects on structures, paving or landscaping supported above. The backfill placed against the permanent retaining walls should preferably comprise a uniform sized durable granular material which is surrounded in a geotextile fabric. A capping layer of at least 0.5m thickness of clayey site won material should be placed above the geofabric, to reduce water infiltration. A subsoil 'agg' drain surrounded by a geofabric filter sock should also be placed at the base and rear of the basement wall to collect seepage and discharge it to the stormwater system. This type of backfill has the advantage that only nominal compaction is required (such as by the use of a plate attached to the excavator). The alternative (although less preferred) is to use the site won material as backfill, however it will require careful control of moisture content, placement and compaction of material in thin layers, and density testing of each layer to ensure it is placed in a controlled manner as an engineered fill material. Placement and compaction of site won material at the rear of basement walls is difficult and time consuming due to the space limitations. Care should also be taken when compacting fill behind retaining walls, to ensure that compaction stresses do not exceed the design earth pressures. Advice during construction is recommended when the type of equipment proposed is known.

Where permanent batter slopes are being proposed, the formation will be dependent on the height of the cut and the materials exposed. As a guide we suggest the following general recommendations;

- Permanent batters through the residual clays should be battered at not steeper than 1V in 2H.
- Any permanent batters will need to be fully protected from erosion in the long term, by a suitable and approved erosion protection measure. Suitable measures would include revegetation or shotcrete. Where revegetation is being proposed, consideration should be given to flattening the permanent batters even further than recommended above to assist with initial vegetation and topsoil establishment and provide for ease of maintenance.

4.4 Retaining Walls

Where temporary batter slopes are adopted, conventional concrete block retaining walls may be constructed at the toe of the batter slopes. We recommend that the following characteristic parameters may be adopted for shoring wall design. The following parameters are on the basis of either a properly placed and compacted engineered backfill or backfill comprising a uniform sized durable granular material which is surrounded in a geotextile fabric as discussed in Section 5.3 above.d

- For cantilever walls where some movement can be tolerated, we recommend a triangular lateral earth pressure distribution using an 'active' earth pressure coefficient (Ka) of 0.35.
- For cantilever walls which will be propped by floor slabs or where movements are to be reduced, we recommend a triangular lateral earth pressure distribution using an 'at rest' earth pressure coefficient (K_0) of 0.55.
- A bulk unit weight of $20kN/m^3$ may be used for the backfill.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design.

Measures must be taken to provide permanent and effective drainage of the ground immediately behind the basement walls. We recommend the use of a free draining durable aggregate (such as 20mm size blue metal) with 'agg' pipe surrounded by a geotextile at the base and connected to the stormwater drainage system.

4.5 Earthworks

The floor level of the building may require the placement of up to 1m of fill over the eastern third of the site. The fill placed to support structural loads from ground floor slabs or surrounding pavements must be placed as engineered fill and the following recommendations relate to site preparation and placement of engineered fill in these areas.

- Strip off the existing grass, topsoil, root affected material, concrete pavements and any existing fill materials. The root balls of any trees or shrubs should also be fully removed. Stripped materials should be stockpiled separately and from a geotechnical perspective will not be suitable for re-use as engineered fill unless specifically assessed and approved by the geotechnical engineers.

- The exposed subgrade should then be proof rolled with 8 passes of a minimum 10 tonne smooth drum roller to detect any soft or heaving areas. The proof rolling should be carried out in the presence of a geotechnical engineer or experienced earthworks technician. Smaller rollers may be applicable for lightly loaded pavements and advice should be sought from the geotechnical engineers in that regard at the time of proof rolling. The boreholes have generally indicated that the residual silty clays are of very stiff or hard strength and we do not expect significant areas of heaving subgrade within those areas, unless they are allowed to wet up. The subgrade should be well graded to promote runoff and reduce the risk of water ponding on the surface. If the subgrade becomes wet it may be untrafficable.

- Any areas of heaving subgrade should be locally removed to a competent base and replaced with engineered fill. As discussed above, where poorly compacted clayey fill is encountered as the subgrade, further more specific subgrade improvement may be required and this is best determined in consultation with the geotechnical engineers at the time of construction.

- Engineered fill should comprise a good quality granular material, such as crushed sandstone or the existing granular road-base material, and should be compacted in horizontal layers with a maximum 200mm loose thickness to at least 98% of Standard Maximum Dry Density (SMDD).

- While not preferred, and from a geotechnical perspective, the existing residual soils and possibly some of the existing clayey fill materials (subject to approval as discussed above) may also be used as engineered fill, provided they are compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD) and to within ±2% of Standard Optimum Moisture Content (SOMC). If the residual silty clay soils are to be adopted for use as an engineered fill the following needs to be carefully considered.

  (i) Some of the clays (particularly the existing clayey fill materials) may have moisture contents greater than the plastic limit and therefore they will require drying out prior to their use as engineered fill, and

  (ii) Where reactive silty clays are used as an engineered fill, they will undergo greater shrink swell movements with changes in moisture content than the in-situ reactive clays. Therefore, consideration needs to be given to the affect that greater shrink-swell movements will have on the performance of structures founded above.

- Density testing should be regularly carried out on any engineered fill. Regular density testing in accordance with Level 1 requirements of AS3798-2007 *'Guidelines on Earthworks for Commercial and Residential Developments'* are recommended.

Soil may need to be removed from site during earthworks operations or pile drilling which will require a waste classification prior to disposal.

4.6 Footings

Due to the moderate column loads, piled footings into the claystone bedrock will be required for the CAMHS Building. We consider that bored piles will be feasible, however groundwater seepage will occur into pile holes during drilling and therefore allowance will need to be made for pumping of groundwater from the pier holes, or pouring concrete by tremie methods. If this is not preferred then consideration could be given to using grout injected piles.

The bedrock generally ranges from extremely low to medium strength, although there are some high strength bands of sandstone within the bedrock profile. Therefore, considering the rock profile and the likely larger diameter piles required to carry the column loads, this will necessitate the use of piling rigs with rock drilling equipment. We recommend that any potential piling contractors be provided with a copy of this geotechnical report and they should be requested to confirm that their equipment is suitable to penetrate the rock and achieve the required depths.

The table in Section 3.2 provides our assessment of the depth and reduced levels for the various rock classes encountered within the boreholes. Based on the rock classification, the following table presents our recommendations on maximum allowable end bearing pressures, ultimate end bearing pressures, maximum allowable skin friction values and ultimate skin friction values for the various classes of rock.

Summary Table of Maximum Allowable and Ultimate End Bearing Pressures and Skin Friction Values

Rock Class	Maximum Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Maximum Allowable Skin Friction (kPa)	Ultimate Skin Friction (kPa)
Class V Claystone (Shale)	700	1,500	50	70
Class IV Claystone (Shale)	1,000	3,000	100	150
Class III Claystone (Shale)	3,000	20,000	250	500
Class IV Sandstone	1,500	4,000	150	300
Class III Sandstone	3,500	30,000	350	800

Class III bedrock was encountered in each of the boreholes, but at variable depths ranging from 5.1m to 9.3m. Bands of Class IV rock were also encountered within the Class III strata and BH604 was terminated within a profile of Class IV claystone. Due to the variability in the quality of the rock and the magnitude of the proposed loads we recommend that unless further investigation is completed, piles be designed to found within Class IV material. Should footings be designed to found within Class III bedrock then additional cored boreholes must be completed following demolition of the existing structures on site to profile the Class III rock in more detail.

We recommend that all piles be founded on and with a minimum embedment of 0.3m into the appropriate quality of rock. In addition to the maximum allowable and ultimate end bearing pressures, piles can also be designed for skin friction. The boreholes indicate bands of poorer quality rock within some of the better-quality rock. For founding purposes, a single pile must have a thickness of at least 1.5B (where B is the pile diameter) below the toe of the pile and within the required rock class, in order to adopt such a rock class for the founding material.

Where ultimate end bearing and skin friction values are adopted, then the ultimate values recommended in the table above must be reduced by an appropriate geotechnical reduction factor. The geotechnical reduction factor should be based on the risk assessment procedure set out in Table 4.3.2 (A) of AS2159-2009, but should not be greater than 0.5, unless the risk factors producing a higher geotechnical reduction factor can be fully justified. Consideration should also be given to the pile testing requirements when determining a suitable geotechnical strength reduction factor.

In order to achieve the recommended skin friction values nominated in the table above, it is essential that the rock sockets be cleaned of any clay smear and suitably roughened using a side wall grooving tool, and that they be at least as rough as Roughness Class R2. We note that an R2 roughness is equivalent to grooves 1mm to 4mm deep and grooves 2mm wide, which are spaced at 50mm to 200mm down the socket length. It will be the responsibility of the piling contractor to ensure that he has the appropriate equipment and methodology to satisfy this roughness criteria.

Where allowable bearing pressures and skin friction values are adopted, settlement of piles will typically be less than 1% of the pile diameter at the toe of the pile. However where ultimate end bearing and skin friction values are adopted, settlements will be greater and therefore once column loads are known, some detailed settlement analysis of piles is recommended to check that predicted settlements are within acceptable limits.

We recommend that the geotechnical engineers inspect piles during drilling to confirm the above recommended bearing pressures and skin frictions are being achieved. Where the lower quality rock (equivalent to Class IV Claystone) is adopted as the founding material, we consider that only a selection of piles will need to be inspected by the geotechnical engineers. However, if further investigation allows the use of the higher quality rock (equivalent to Class III) for a founding material then all piles should be inspected by the geotechnical engineers. Inspection of piles will require the geotechnical engineer to be on site during the drilling process so that they can inspect both the material being drilled and check the pile's consistency with nearby borehole logs. It is important to note that the geotechnical engineers can only 'sign off' on piles which they have inspected.

Prior to pouring concrete, piles will need to be dewatered, cleaned of all loose debris from the base, inspected and approved by the geotechnical engineers. We recommend the base of piles are cleaned with a cleaning bucket. Piles will need to be poured as soon as possible after drilling, but at least on the day of drilling. If piles are left open overnight, they must be redrilled prior to pouring concrete to remove any softened or other debris from the base of the pile.

If the structure is to be designed with a fully suspended slab on piles, then any portion of the suspended slab and thickening beams where the subgrade, after bulk excavation, exposes residual clays or extremely weathered rock would need to be underlain by a void former. Further advice should be obtained from the geotechnical engineers when details are known more fully.

4.7 Pavements and Slabs On-Grade

Following subgrade preparation in accordance with the recommendations in Section 5.5, new pavements will need to be designed on the basis of the specific subgrade material. Where the subgrade comprises the residual silty clay a design CBR of 3% may be adopted. Where pavements overlie areas of engineered fill, CBR testing of the engineered fill subgrade will be required to confirm design assumptions.

Flexible pavements should be underlain by a good quality base-course layer comprising crushed rock to RTA QA specification 3051 (2010) unbound base material, or equivalent good quality and durable fine crushed rock compacted to at least 100% of Standard Maximum Dry Density (SMDD).

Concrete pavements should also be underlain by a subbase layer of at least 100mm thickness comprising DGB20 compacted to at least 100% of SMDD. This will reduce the risk of pumping of fines where clayey subgrades are encountered.

We recommend that subsoil drains be placed around the perimeter of the new pavements. The subsoil drains should extend to a depth of at least 0.3m below the subgrade level and the drains should have adequate falls to reduce ponding in the drains.

If slabs on grade at level 01 are being considered, then concrete slabs on grade, should be isolated from the structural columns to allow relative movement. Concrete slabs will need to be designed for the potential for shrink-swell movements as discussed in Section 4.1 above. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

4.8 Earthquake Design Parameters

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) = 0.08
- Site Subsoil Class = Class Ce

5 SALINITY

With reference to the 1:100,000 Map of Salinity Potential in Western Sydney prepared by the Department of Natural Resource, the site is located in an area where there is a moderate potential for soil and groundwater salinity to occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. The local council has guidelines relating to salinity issues which should be checked for relevance to this project.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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:**	33780LT
Project:	Proposed CAMHS Building	**Report:**	A
Location:	Nepean Hospital, Derby Street, Kingswood, NSW	**Report Date:**	23/04/2021
		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 %**
602	0.30 - 1.50	21.1	-	-	-	-
602	0.50 - 0.95	19.4	51	17	34	13.5
604	0.50 - 1.50	18.1	-	-	-	-
604	1.50 - 1.95	13.3	44	20	24	10.5

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: 15/04/2021.
- Sampled and supplied by client. Samples tested as received.

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the items tested or sampled.

23/04/2021

Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33780LT
Project:	Proposed CAMHS Building	**Report:**	B
Location:	Nepean Hospital, Derby Street, Kingswood, NSW	**Report Date:**	21/04/2021
		Page 1 of 1	

BOREHOLE NUMBER	BH 602	BH 604
DEPTH (m)	0.30 - 1.50	0.50 - 1.50
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.70 STD	1.81 STD
Optimum Moisture Content (%)	19.4	15.4
Moulded Dry Density (t/m^3)	1.67	1.77
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	102	104
Moisture Contents		
Insitu (%)	21.1	18.2
Moulded (%)	19.7	16.0
After soaking and		
After Test, Top 30mm(%)	27.4	20.6
Remaining Depth (%)	22.4	18.4
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	3.0	1.5
C.B.R. value: @2.5mm penetration	3.0	4.0

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: 15/04/2021.
- BH 604 dried back prior to testing as the sample was too saturated.

NATA

NATA Accredited Laboratory Number:1327

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21/04/2021
Authorised Signature / Date
(D. Treweek)

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Health Infrastructure	**Ref No:**	33780LT
Project:	Proposed CAMHS Building	**Report:**	C
Location:	Nepean Hospital, Derby Street, KINGSWOOD, NSW	**Report Date:**	14/04/21

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
601	2.87 - 2.91	0.3	6	A
	3.18 - 3.21	0.2	4	A
	3.84 - 3.87	0.3	6	A
	4.04 - 4.07	0.3	6	A
	4.72 - 4.76	0.3	6	A
	5.04 - 5.07	0.03	1	A
	5.47 - 5.51	0.2	4	A
	6.32 - 6.35	0.07	1	A
	6.88 - 6.91	1	20	A
	7.28 - 7.31	1.1	22	A
	7.86 - 7.89	0.2	4	A
	8.23 - 8.26	0.3	6	A
	8.73 - 8.76	0.2	4	A
	9.05 - 9.08	0.3	6	A
602	4.58 - 4.61	0.2	4	A
	4.91 - 4.96	0.3	6	A
	5.20 - 5.24	2.6	52	A
	5.89 - 5.94	0.3	6	A
	6.09 - 6.12	0.2	4	A
	6.77 - 6.81	0.3	6	A
	7.30 - 7.34	0.3	6	A
	7.77 - 7.80	0.03	1	A
	8.23 - 8.26	0.3	6	A
	8.83 - 8.86	0.2	4	A
	9.30 - 9.34	0.6	12	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure
Ref No: 33780LT

Project: Proposed CAMHS Building
Report: C

Location: Nepean Hospital, Derby Street, KINGSWOOD, NSW
Report Date: 14/04/21

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
602	9.32 - 9.36	0.6	12	A
	9.71 - 9.74	0.3	6	A
	10.12 - 10.16	0.6	12	A
	10.58 - 10.61	0.7	14	A
603	5.80 - 5.85	0.2	4	A
	6.22 - 6.25	0.1	2	A
	6.76 - 6.79	0.2	4	A
	7.42 - 0.00	0.3	6	A
	7.64 - 0.00	0.2	4	A
	8.30 - 0.00	0.05	1	A
	8.74 - 0.00	0.09	2	A
	9.30 - 0.00	0.3	6	A
	9.66 - 0.00	0.2	4	A
	10.06 - 0.00	0.1	2	A
	10.33 - 0.00	0.7	14	A
	10.84 - 0.00	0.5	10	A
	11.14 - 0.00	0.5	10	A
	11.45 - 0.00	1.5	30	A
604	2.93 - 2.96	0.2	4	A
	3.13 - 3.17	0.2	4	A
	3.72 - 3.75	0.1	2	A
	4.17 - 4.20	0.04	1	A
	4.63 - 4.67	0.05	1	A
	5.19 - 5.23	0.6	12	A
	5.80 - 5.83	0.8	16	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Health Infrastructure	**Ref No:**	33780LT
Project:	Proposed CAMHS Building	**Report:**	C
Location:	Nepean Hospital, Derby Street, KINGSWOOD, NSW	**Report Date:**	14/04/21

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
604	6.19 - 6.23	0.4	8	A
	6.82 - 6.85	0.4	8	A
	7.38 - 7.41	0.1	2	A
	7.72 - 7.75	0.03	1	A
	8.22 - 8.25	0.1	2	A
	8.69 - 8.72	0.03	1	A
	9.28 - 9.31	0.4	8	A
	0.00 -			A
	10.00 - 10.03	49.3	986	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the IS(50) has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 IS(50).

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CERTIFICATE OF ANALYSIS 266594

Client Details

Client	JK Geotechnics
Attention	Bryan Zheng
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33780LT, Kingswood**
Number of Samples	3 Soil
Date samples received	14/04/2021
Date completed instructions received	14/04/2021

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.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	21/04/2021
Date of Issue	21/04/2021

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		266594-1	266594-2	266594-3
Your Reference	UNITS	601	602	604
Depth		1.8-1.95	1.5-1.9	2.2-2.5
Date Sampled		10/04/2021	10/04/2021	11/04/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	20/04/2021	20/04/2021	20/04/2021
Date analysed	-	20/04/2021	20/04/2021	20/04/2021
pH 1:5 soil:water	pH Units	8.0	5.1	6.2
Chloride, Cl 1:5 soil:water	mg/kg	620	240	630
Sulphate, SO4 1:5 soil:water	mg/kg	390	340	190
Resistivity in soil*	ohm m	12	26	16

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	-			20/04/2021	[NT]	[NT]	[NT]	[NT]	20/04/2021	[NT]
Date analysed	-			20/04/2021	[NT]	[NT]	[NT]	[NT]	20/04/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	85	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	84	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[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.

Report Comments

pH
Samples were out of the recommended holding time for this analysis.

JKGeotechnics

BOREHOLE LOG

Borehole No.
601

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Method:** SPIRAL AUGER **R.L. Surface:** ~55.4 m
Date: 10/4/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.Z./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			55	0–1.2			POTHOLED SECTION: MATERIAL REMOVED BY VACUUM EXCAVATION DOWN TO 1.2m DEPTH				CONCRETE AT SURFACE
ON COMPLETION OF CORING	ES, DS	N = 5 1,2,3	54	1.2–2.1		-	FILL: Gravelly clay, medium to high plasticity, brown mottled dark grey and red brown, fine to medium grained sub-angular and angular ironstone and siltstone gravel.	w>PL			APPEARS POORLY COMPACTED
	ES, DS		53	2.1–2.6		-	CLAYSTONE: dark brown, with iron indurated bands.	HW DW	VL VL - L		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
				2.6–2.8			as above, but dark brown and grey, with fine to medium grained sandstone bands.		L - M		LOW TO MODERATE RESISTANCE
			52, 51, 50, 49	3–6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 9.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 9.1m TO 2.6m. CASING 2.6m TO 0.1m. 2mm SAND FILTER PACK 9.1m TO 2.5m. BENTONITE SEAL 2.5m TO 2.0m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
601

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT	**Core Size:** NMLC	**R.L. Surface:** ~55.4 m
Date: 10/4/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.Z./A.B.

Column headings: 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); DESCRIPTION – Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific, General); Formation

RL (m AHD) scale: 53, 52, 51, 50, 49, 48, 47. Depth (m) scale: 3, 4, 5, 6, 7, 8.

Water Loss\Level: ON 22/4/21 (water level); 100% RETURN

START CORING AT 2.82m

Core Description	Weathering	Strength	Point Load $I_s(50)$
SANDSTONE: fine to medium grained, grey and brown, bedded sub-horizontally.	MW	L - M	0.30
Extremely Weathered claystone: silty CLAY, high plasticity, brown, with fine to medium grained angular claystone gravel.	XW	(Hd)	
LAMINITE: fine to medium grained, grey sandstone (50%), sub-horizontally laminated with dark grey and brown claystone (50%).	MW	L - M	0.20
Extremely Weathered claystone: silty CLAY, high plasticity, dark brown.	XW	(Hd)	
LAMINITE: fine to medium grained, grey sandstone (50%), sub-horizontally laminated with dark grey and brown claystone (50%).	MW	L - M	0.30, 0.30, 0.30
Extremely Weathered claystone: silty CLAY, high plasticity, dark brown, with highly weathered claystone bands.	XW / HW	Hd / VL	0.030
CLAYSTONE: dark grey and brown, sub-horizontally laminated.	MW	L - M	0.20
Extremely Weathered claystone: silty CLAY, high plasticity, dark brown.	XW	(Hd)	
CLAYSTONE: dark grey, bedded sub-horizontally, carbonaceous.	HW	VL	0.070
Extremely Weathered claystone: silty CLAY, high plasticity, dark grey and red brown.	XW	(Hd)	
SANDSTONE: fine to medium grained, grey mottled light brown, bedded at 0-5°, with dark grey claystone laminae.	SW	M - H	1.0, 1.1
CLAYSTONE: dark grey mottled brown, bedded sub-horizontally.	FR	L - M	0.20, 0.30, 0.20

Defect Details – Description (Specific):

- (2.87m) Be, 0°, P, R, Fe Sn
- (2.92m) Be, 0°, P, R, Fe Sn
- (3.15-3.50m) Numerous Be and XWS, typically 0°, P, R to S, Fe or Clay, Sn or FILLED up to 2mm.t
- (3.55m) J, 73°, P, R, Fe Sn
- (3.59m) CS, 0°, 12 mm.t
- (3.62-3.66m) CS x 3, 0°, 2 mm.t
- (3.90m) J, 87°, Un, R, Fe Sn
- (4.12m) Be, 0°, P, R, Fe Sn
- (4.16m) Be, 0°, P, R, Fe Sn
- (4.43m) Be, 3°, Un, R, Fe Vn
- (4.55m) XWS, 0°, 20 mm.t
- (4.65m) J, 72°, Un, R, Fe Vn
- (4.68m) Be, 0°, P, R, Fe Sn
- (4.77m) Be, 4°, P, R, Fe Sn
- (4.85m) Be, 0°, P, R, Fe Sn
- (5.24m) J, 90°, Un, S, Cn
- (5.38m) J, 23°, Un, R, Fe Sn
- (5.45m) J, 24°, Un, R, Fe Sn
- (5.50m) Be, 0°, Un, S, Fe Vn
- (5.54m) Ji, 90°, P
- (5.62m) J, 30°, Un, R, Fe Vn
- (5.67m) J, 20°, Un, R, Fe Sn
- (5.70m) J, 23°, Un, R, Fe Sn
- (5.80m) J, 70°, P, R, Fe Sn
- (6.83m) CS, 0°, 28 mm.t
- (7.19m) Be, 4°, Un, R, Fe Vn
- (7.69m) CS, 9°, 32 mm.t
- (7.74m) XWS, 9°, 20 mm.t
- (7.80m) CS, 0°, 4 mm.t
- (7.83m) CS, 0°, 10 mm.t
- (8.19m) Be, 2°, Un, S, Clay Vn
- (8.37m) Be, 2°, Un, S, Clay Vn

Formation: Bringelly Shale

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

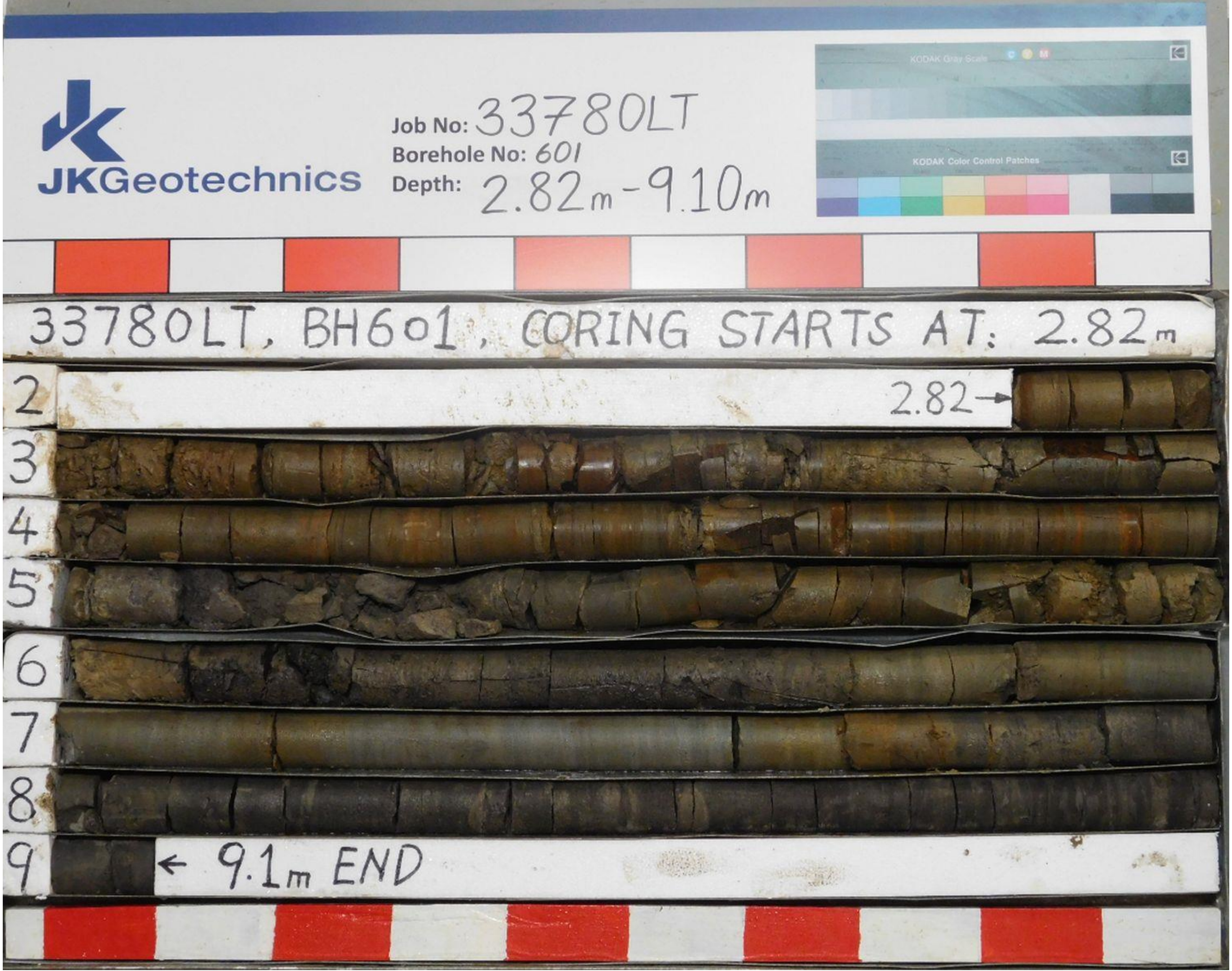
CORED BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT	**Core Size:** NMLC	**R.L. Surface:** ~55.4 m
Date: 10/4/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.Z./A.B.

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
					CLAYSTONE: as above	FR	L - M	0.30			
					END OF BOREHOLE AT 9.10 m						
		46									
			10								
		45									
			11								
		44									
			12								
		43							600 200 60 20		
			13								
		42									
			14								
		41									
			15								
		40							600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33780LT
Borehole No: 601
Depth: 2.82m - 9.10m
KODAK Gray Scale
KODAK Color Control Patches
33780LT, BH601, CORING STARTS AT: 2.82m
2
2.82→
3
4
5
6
7
8
9
← 9.1m END

JKGeotechnics

BOREHOLE LOG

Borehole No.
602
1 / 2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Method:** SPIRAL AUGER **R.L. Surface:** ~53.6 m
Date: 10/4/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING							CONCRETE: 100mm.t				
						-	over ROADBASE: 50mm.t	w~PL			
		N = 8 1,4,4	53				FILL: Silty clay, medium to high plasticity, brown mottled dark grey and red brown, trace of fine to medium grained ironstone gravel, ash and slag.				
				1		CH	Silty CLAY: high plasticity, grey mottled brown and red brown, trace of fine to medium grained ironstone gravel, and roots.	w~PL	VSt	200 280 290	RESIDUAL
		N > 33 12,15,18/ 100mm REFUSAL	52			CI	Silty CLAY: medium plasticity, red brown and grey, with ironstone bands grading into extremely weathered bedrock.	w<PL	Hd	>600 >600 >600	
			51	2		-	Extremely Weathered claystone: silty CLAY, high plasticity, red brown and grey, with ironstone bands.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE BANDS
			50	3							
			49	4			CLAYSTONE: dark grey and brown, with iron indurated and sandstone bands.	DW	L - M		LOW TO MODERATE RESISTANCE
			48	5			REFER TO CORED BOREHOLE LOG				
			47	6							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
602

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Core Size:** NMLC **R.L. Surface:** ~53.6 m
Date: 10/4/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** B.S./A.B.

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)	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 4.50m						
100% RETURN		49	5		SANDSTONE: fine to medium grained, grey, bedded at 0-5°, with dark grey claystone laminae.	MW	L - M	0.20, 0.30		(4.55m) Be, 0°, P, S, Clay Vn (4.58m) J, 85°, Un, R, Fe Ct (4.76m) J, 84°, Un, R, Fe Ct (5.05m) J, 79°, Un, R, Fe Ct	Bringelly Shale
					CLAYSTONE: dark grey, bedded sub-horizontally.			2.6		(5.15m) CS, 0°, 20 mm.t (5.25m) Cr, 0°, 50 mm.t	
		48	6		CLAYSTONE: dark grey, bedded sub-horizontally, with fine grained grey sandstone laminae.		L	0.30		(5.57m) XWS, 0°, 8 mm.t (5.79m) J, 90°, Un, S, Cn (5.82m) CS, 0°, 1 mm.t (5.96m) J, 90°, Un, S, Cn	
							VL	0.20			
		47	7				L	0.30, 0.30	200, 60, 20	(6.14m) Be, 0°, P, S, Fe Sn (6.20m) CS, 0°, 1 mm.t (6.28m) CS, 0°, 2 mm.t (6.34m) CS, 0°, 2 mm.t (6.37m) CS, 0°, 1 mm.t (6.49m) CS, 0°, 1 mm.t (6.51m) CS, 0°, 1 mm.t (6.65m) J, 90°, P, S, Clay Vn (6.70m) XWS, 0°, 100 mm.t (6.85m) J, 42°, Un, S, Clay Ct (7.22m) J, 47°, Un, S, Clay Ct	
		46	8		CLAYSTONE: dark grey, bedded sub-horizontally.	HW	VL	0.030		(7.62m) J, 38°, Un, S, Clay Ct	
		45				MW	L	0.30		(8.10m) J, 42°, Un, S, Clay Ct (8.50m) J, 40°, P, S, Clay Ct (8.68m) J, 45°, P, S, Clay Ct	
		44, 43	9, 10		CLAYSTONE: dark grey, bedded sub-horizontally, with fine grained grey sandstone laminae.	FR	L - M	0.20, 0.60, 0.60, 0.30, 0.60, 0.70		(8.76m) J, 67°, Un, R, Clay Vn (9.92m) CS, 0°, 3 mm.t (10.48m) J, 47°, C, S, Clay Vn	
					END OF BOREHOLE AT 10.70 m				600, 200, 60, 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33780LT KINGSWOOD.GPJ <<DrawingFile>> 04/05/2021 12:50 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
Job No: 33780LT
Borehole No: 602
Depth: 4.50m - 10.70m
KODAK Gray Scale
KODAK Color Control Patches
33780LT, BH602, CORING STARTS AT: 4.5m
4
4.5 →
5
6
7
8
9
10
← 10.7m END

JKGeotechnics

BOREHOLE LOG

Borehole No.
603
1 / 2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Method:** SPIRAL AUGER **R.L. Surface:** ~52.6 m
Date: 17/2/21 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** B.S./A.B.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					FILL: Sandy silty clay, low plasticity, dark brown and brown, fine to medium grained sand, with fine to medium grained igneous gravel, trace of roots.	w~PL			
	N = 16 4,6,10	52	1	CI-CH	Silty CLAY: medium to high plasticity, orange brown, trace of fine to medium grained ironstone gravel and roots.	w~PL	Hd	590 600 >600	RESIDUAL
	N = 9 3,4,5	51	2	CH	Silty CLAY: high plasticity, light grey and red brown, trace of fine to coarse grained ironstone gravel and roots.	w<PL		410 450 550	
	N=SPT 14/ 150mm REFUSAL	50	3	-	Extremely Weathered claystone: silty CLAY, medium to high plasticity, grey, with iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE BANDS
ON 22/4/21		49	4		as above, but with very low strength bands.				
		48	5		CLAYSTONE: dark grey and brown, with sandstone and iron indurated bands.	DW	VL - L L		VERY LOW TO LOW BANDED RESISTANCE
		47, 46	6		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 11.55m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 11.55m TO 2.55m. CASING 2.55m TO 0m. 2mm SAND FILTER PACK 11.55m TO 2.0m. BENTONITE SEAL 2.0m TO 1.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

SAMPLES: ES, U50, DB, DS; Graphic Log

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 603

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Core Size:** NMLC **R.L. Surface:** ~52.6 m
Date: 17/2/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** B.S./A.B.

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
		47			START CORING AT 5.80m						
75% RETURN		46	6		CLAYSTONE: grey and brown, bedded sub-horizontally.	MW	L	0.20, 0.10		(6.38m) Ji, 90°, Un	Bringelly Shale
			7		CLAYSTONE: grey, bedded sub-horizontally.	HW	VL	0.20		(6.74m) CS, 0°, 4 mm.t (6.90m) XWS, 0°, 50 mm.t (6.95m) J, 60°, P, S, Fe Sn (6.99m) J, 30°, P, S, Fe Sn (7.09m) XWS, 0°, 70 mm.t (7.29m) XWS, 0°, 100 mm.t	
		45				MW	L	0.30, 0.20			
			8		Extremely Weathered claystone: silty CLAY, medium plasticity, grey, with very low strength bands.	HW	VL			(7.76m) XWS, 0°, 25 mm.t (7.85m) J, 80°, P, S, Cn (7.94m) XWS, 0°, 120 mm.t	
		44	9		CLAYSTONE: grey, bedded at 0-5°.			0.050, 0.090		(8.70m) XWS, 0°, 15 mm.t (8.90m) J, 90°, P, S, Cn	
		43, 42	10		as above, but with fine grained sandstone laminae.	FR	L	0.30, 0.20, 0.10		(9.70m) XWS, 0°, 40 mm.t	
							M	0.70, 0.50			
			11		SANDSTONE: fine grained, grey, with dark grey claystone laminae.		M - H	0.50, 1.5		(11.02m) Be, 0°, P, R, Clay Ct (11.43m) XWS, 0°, 20 mm.t	
		41			END OF BOREHOLE AT 11.55 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33780LT
Borehole No: BH603
Depth: 5.80m - 11.55m
KODAK Gray Scale
C Y M
KODAK Color Control Patches
JOB No. 33780LT, BH603, CORING STARTS AT 5.80m
5
6
7
8
9
10
11
EOH AT 11.55m

JKGeotechnics

BOREHOLE LOG

Borehole No.
604

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Method:** SPIRAL AUGER **R.L. Surface:** ~52.8 m
Date: 11/4/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.Z./A.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		0		CONCRETE: 90mm.t				
			-	over ROADBASE: 210mm.t				
				FILL: Silty clay, medium to high plasticity, brown mottled dark grey, trace of fine to medium grained sub-angular ironstone gravel, glass fragments, ash and slag.	w~PL			
	N = 8 3,4,4		CH	Silty CLAY: high plasticity, brown mottled yellow brown, trace of fine to medium grained sub-angular ironstone gravel.	w~PL	VSt	300 330 320	RESIDUAL
					w<PL	Hd		
	N = 38 7,11,27		CI	as above, but medium plasticity, brown and grey, with ironstone bands grading into extremely weathered bedrock.			450 550 >600	
		2	-	Extremely Weathered claystone: silty CLAY, high plasticity, grey and brown.	XW	Hd		BRINGELLY SHALE
				CLAYSTONE: grey and dark brown.	DW	L		VERY LOW 'TC' BIT RESISTANCE
								LOW RESISTANCE WITH VERY LOW BANDS
		3		REFER TO CORED BOREHOLE LOG				

RL (m AHD): 52, 51, 50, 49, 48, 47, 46
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 604

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT **Core Size:** NMLC **R.L. Surface:** ~52.8 m
Date: 11/4/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** B.Z./A.B.

Columns: 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) | DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness; Specific / General) | Formation

RL scale: 50, 49, 48, 47, 46, 45, 44. Depth scale: 3, 4, 5, 6, 7, 8.

Water Loss\Level: 100% RETURN

START CORING AT 2.93m

Core Description	Weathering	Strength	Point Load $I_s(50)$
CLAYSTONE: grey and dark brown, bedded sub-horizontally.	MW	L	0.20, 0.20
NO CORE 0.25m			
CLAYSTONE: grey and dark brown, bedded sub-horizontally.	MW	L	0.10
	HW	VL	0.040, 0.050
SANDSTONE: fine to medium grained, grey, bedded sub-horizontally at 0-5°, with sub-horizontally grey claystone laminae.	MW	M	0.60
LAMINITE: fine grained grey sandstone (50%) sub-horizontally laminated with dark grey claystone (50%).			0.80, 0.40
Extremely Weathered claystone: silty CLAY, medium plasticity, dark brown and dark grey.	XW	Hd	
CLAYSTONE: dark grey, sub-horizontally bedded with fine to medium grained grey, sandstone laminae.	MW	L	0.40
CLAYSTONE: dark grey, bedded sub-horizontally.	FR	L - M	0.10, 0.030
	HW	VL	
	FR	L - M	0.10
as above, but with extremely weathered bands.	HW	VL	0.030

Defect Details – Description (Specific):

- (3.05m) J, 36°, P, S, XW Infill
- (3.15m) Be, 0°, Un, R, Fe Sn
- (3.23m) J, 47°, Un, R, Fe Sn
- (3.32m) Cr, 150 mm.t
- (3.83m) J, 90°, P, R, Fe Sn
- (3.90m) J, 90°, P, S, Fe Sn
- (4.40m) XWS, 0°, 50 mm.t
- (4.87m) XWS, 0°, 50 mm.t
- (5.05m) XWS, 0°, 100 mm.t
- (5.25m) XWS, 0°, 2 mm.t
- (5.41m) XWS, 0°, 2 mm.t
- (5.70m) Jh, Un, Fe Vn
- (6.12m) J, P, R, Cn
- (6.18m) Cr, 20 mm.t
- (6.35m) Be, Un, R, Fe Sn
- (6.37m) CS, 3 mm.t
- (7.20m) XWS, 0°, 100 mm.t
- (7.70m) CS
- (7.80m) J, P, R, Cn

Formation: Bringelly Shale

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
604
3 / 3

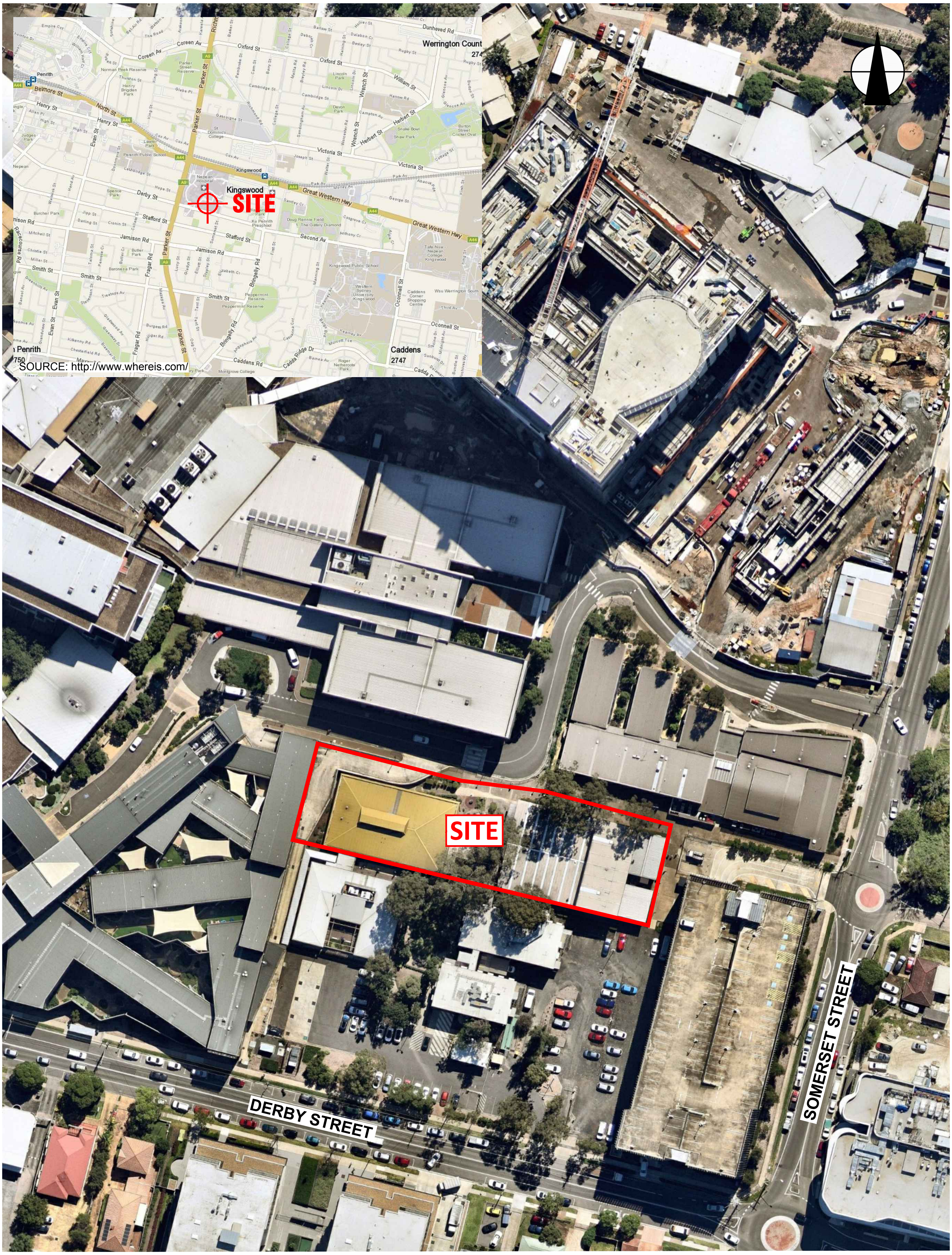
Client: HEALTH INFRASTRUCTURE
Project: PROPOSED CAMHS BUILDING
Location: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Job No.: 33780LT	**Core Size:** NMLC	**R.L. Surface:** ~52.8 m
Date: 11/4/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.Z./A.B.

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
					CLAYSTONE: as above	HW	VL				
					CLAYSTONE: dark grey, bedded sub-horizontally.	FR	L - M	0.40		(9.20m) CS, 2 mm.t	
					END OF BOREHOLE AT 9.40 m						
		43									
			10								
		42									
			11								
		41									
			12								
		40									
			13								
		39									
			14								
		38									
			15								
		37									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33780LT
Borehole No: 604
Depth: 2.93m - 9.40m
KODAK Gray Scale
KODAK Color Control Patches
33780LT, BH604, CORING STARTS AT: 2.93m
2
2.93 →
3
NO CORE
250mm
4
5
6
7
8
9
← 9.4m END



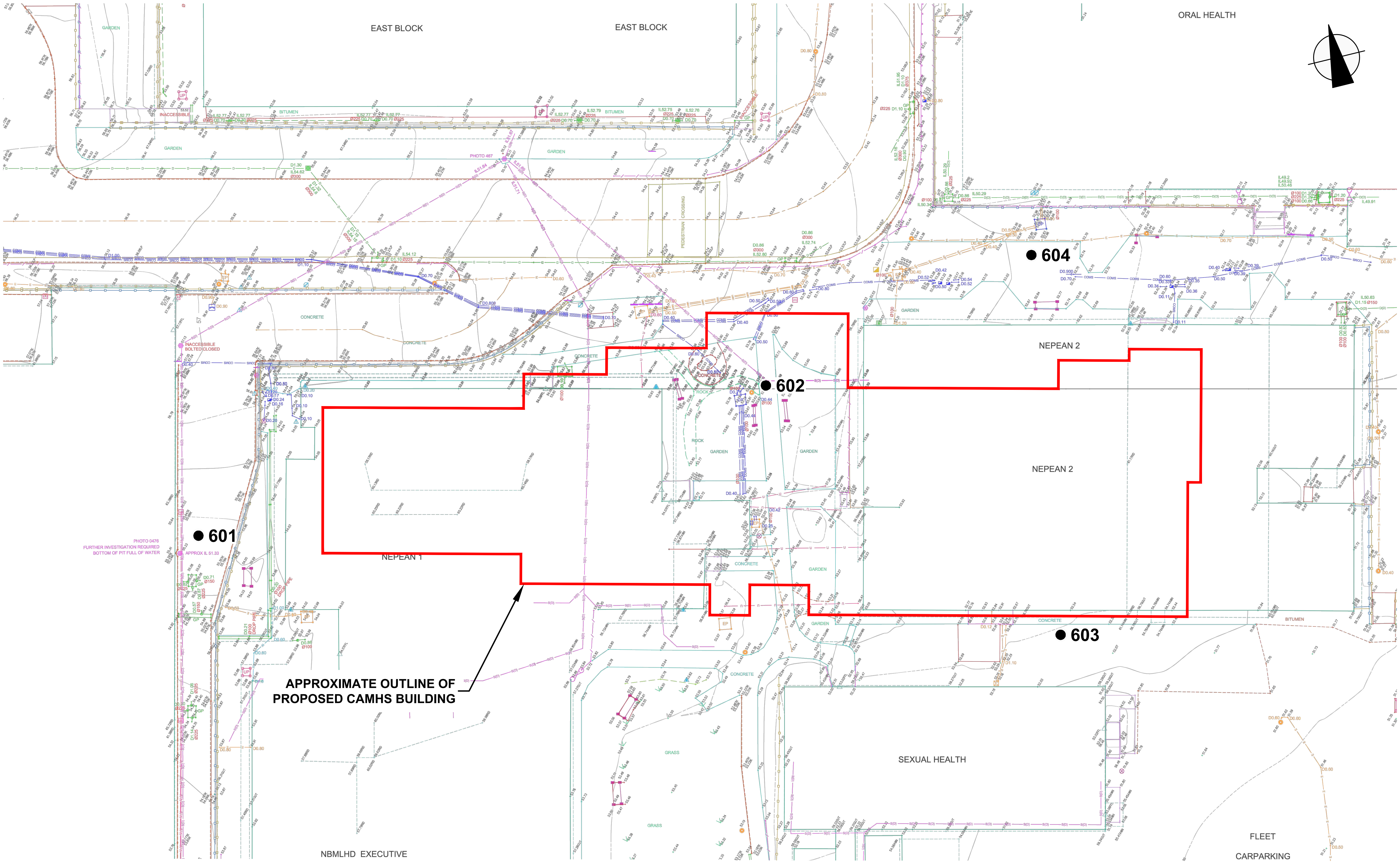
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW
Report No:	33780LT
Figure No:	1

JKGeotechnics

PLOT DATE: 22/04/2021 11:33:10 AM DWG FILE: Y:\33000'S\33780LT KINGSWOOD\CAD\33780LT.DWG



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: NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Report No: 33780LT

Figure No: 2

JKGeotechnics

PLOT DATE: 4/05/2021 1:35:57 PM DWG FILE: Y:\33000'S\33780LT KINGSWOOD\CAD\33780LT.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

The DMT is used to measure material index (I_D), horizontal stress index (K_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:

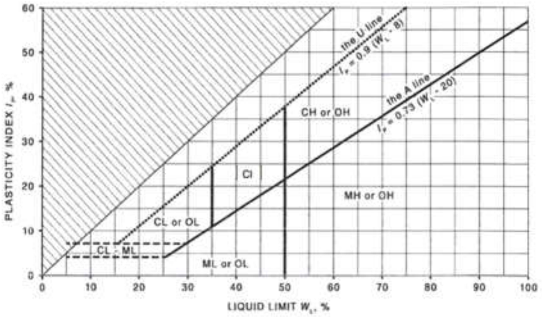
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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 U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. 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 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT ‘N’ Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (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