REPORT TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED TAMS BUILDING

AT

NEPEAN HOSPITAL, DERBY STREET, KINGSWOOD, NSW

Date: 4 July 2022

Ref: 35033LTrpt

JKGeotechnics

www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801

Report prepared by:

Arthur Billingham
Senior Geotechnical Engineer

Report reviewed by:

Linton Speechley
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
35033LTrpt	Final Report	4 July 2022

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Total Asset Management Services (TAMS) 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: P55875LT) dated 8 February 2022, and our variations for weekend work dated 25 March 2022.

We understand from the supplied architectural drawings prepared by BVN (Project No. 1903020.000, Drawing Nos. NHR-BVN-DRW-ARC-TAM-GEN-NL00002, NL00003 and NL00005, Issue A dated 26 February 2021) that the proposed TAMS Building will comprise a ground floor level workshop with a second-storey office level at the southern end of the building footprint. The ground floor will have a finished floor level at RL50.7m which will be close to existing levels along the northern edge of the footprint and above existing surface levels elsewhere. Structural loads for the building have not been provided but are anticipated to be relatively light. 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.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E35033PLrpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

The fieldwork for the current investigation comprised the drilling of four boreholes (BH1 to BH4) to depths ranging from 7.5m to 9m below existing surface levels, using our track-mounted drilling rig. The boreholes were advanced using spiral auger drilling techniques and a Tungsten Carbide (TC) bit.

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, the upper profile was drilled using a hand auger to visually assess whether any services were present at the borehole locations.

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, Sheet No. 50, 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 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.

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. 296912.

Groundwater observations were recorded in all boreholes during and on completion of auger drilling. 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 has a southern frontage with Derby Street with an entry and exit driveway located off Derby Street at the south-western corner of the site.

The site comprises an area, shown on Figure 1, within the south-eastern portion of Nepean Hospital, which is currently used by the Fleet carparking. The car park has areas surfaced with gravel and sprayed seal which appears to be in poor condition.

North and north-west of the site are metal clad buildings supported on brick piers which appear to be in good condition.

Abutting the eastern boundary of the site is the P5 eight-storey carpark. The building appears to be in good condition based on cursory visual observations. Surface levels across the boundary vary due to the slope on the subject site such that the ground floor level of the carpark was 0.5m higher at the southern end grading to 0.6m lower at the northern end. Levels along the boundary are supported by a concrete retaining wall with a thickness of about 0.2m.

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 1.5m 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.

Fill
From the surface in all boreholes, fill was encountered and was generally shallow (0.2m to 0.6m) although deeper fill extending to 1.1m depth was encountered within BH1. The fill within the boreholes generally comprised gravelly sand, although in BH4, silty clay fill was encountered below a depth of 0.2m. The upper gravelly sand fill may comprise a pavement material for the carpark. The fill in BH1 was assessed as well compacted.

Residual Silty Clay
Silty clay, assessed as residual in origin, was encountered below the fill in all boreholes. The clay was assessed as high plasticity and generally of very stiff to hard strength. The residual clay generally contained inclusions of ironstone gravel.

Weathered Claystone Bedrock
Weathered claystone bedrock, was encountered at depths ranging from 1.5m to 2.5m. The level of the bedrock ranged from approximately RL50.1 (BH3) to RL47.8m (BH1). The surface of the bedrock generally appears to dip down to the south-east. The initial 1m to 1.5m of bedrock comprised extremely weathered material. We note that extremely weathered claystone will remould to a material with soil like properties. The bedrock generally appears to be deeply weathered with extremely weathered bands encountered throughout much of the profile and the rock strength improving gradually. Low to medium strength rock was encountered at the termination depths in each of the boreholes, except BH1, where the rock was assessed as low strength.

Groundwater
All boreholes were dry during and on completion of drilling.

3.3 Laboratory Test Results

The Atterberg Limit testing completed on the residual silty clay indicate they are of medium and high plasticity with the high plasticity material encountered in the upper profile whilst the medium plasticity material was encountered closer towards the bedrock horizon. The linear shrinkage results indicate a high potential for shrink-swell movements with changes in moisture content.

The four-day soaked CBR tests on the residual clay returned values of 1.5% and 1.0%. The in-situ moisture contents of the clays are 2.9% and 1.0% 'wet' of their respective optimum moisture contents. During soaking, the samples swelled by 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)
BH1	0.6-0.95	Silty Clay FILL	8.2	45	250	4,500
BH2	1.5-1.95	Silty Clay RESIDUAL	7.6	92	620	2,100
BH4	4.0-4.5	Claystone BEDROCK	4.3	320	1,900	740

Based on these results, the soils would be classified as having a 'Non-aggressive' exposure classification for concrete piles and a 'Moderate' 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 pavements, 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 may apply to the structure if deemed similar to the buildings defined in Section 1.1 of AS2870-2011 by the structural engineer. 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.

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 RL50.7m which will only require minimal excavation around the north-western corner of the building footprint. Elsewhere excavation will only be required for footings, services and to achieve the design subgrade level for new pavements. In this regard, we expect these excavations will be limited in depth to about 1.5m which will encounter the fill and residual clays which should be readily excavated using the buckets of hydraulic excavators.

No groundwater was encountered during drilling and given the anticipated shallow excavation depths we do not expect groundwater will be encountered.

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 batters through the fill and residual clays 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. If this occurs temporary batter slopes may need to be flattened.
- 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, and retaining walls constructed in front of temporary batters, the backfill between the temporary batter slope and the rear of the retaining wall should preferably comprise a uniform sized durable granular material which is surrounded in a geotextile fabric. A capping layer of at least 0.3m 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 retaining 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).

Where permanent batters 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 fill and residual clays should be battered at no steeper than 1V: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 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 4.3 above.

- 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 0.6.
- 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 retaining 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 proposed ground floor level of the TAMS building is above existing levels and we consider that, to avoid issues associated with earthworks, it will be more economical to suspend the building above the slope. However, earthworks will be required for new pavements and, if fill is to be placed to support structural loads from ground floor slabs for the TAMS building, then this must also 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 pavements and any existing pavement materials. The roots 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 300mm loose thickness to at least 98% of Standard Maximum Dry Density (SMDD).
- While not preferred, from a geotechnical perspective, the existing residual soils and possibly some of the existing granular 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 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

We are not aware of the structural loads for the building; however, we anticipate that the building will have relatively light loads. Due to the profile comprising very stiff to hard clay overlying weathered bedrock at relatively shallow depths suitable footing systems could comprise raft slabs, shallow pad and/or strip footings may be founded in the residual clay. Shallow pad/strip footings or the external and internal beams of stiffened raft slabs founded in the residual clay of at least very stiff strength may be designed for an allowable bearing pressure (ABP) of 150kPa.

Alternatively, the building could be constructed with a fully suspended ground floor slab supported on piles founded within the underlying weathered claystone. Piles socketed at least 0.5m into at least very low strength claystone may be designed for an ABP of 600kPa. If the structure is 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 would need to be underlain by a void former. Further advice should be obtained from the geotechnical engineers when details are known more fully.

Prior to pouring concrete, the footing excavations will need to be cleaned of all loose and/or water-softened material from the base, inspected and approved by the geotechnical engineers. Where piles are adopted, the piles will need to be inspected by the geotechnical engineer whilst on site during drilling to confirm the above recommended bearing pressure is achieved. At least the initial stages of footing excavation/pile drilling should be inspected by a geotechnical engineer to ascertain that the recommended founding material has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit. Footings must be dry and free of any loose or water softened materials

4.7 Pavement Design

Following subgrade preparation in accordance with the recommendations in Section 4.5, 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 1% may be adopted. The silty clay subgrade has a low soaked CBR value and with such relatively poor subgrade values and conditions the main options are:

1. Design the pavements for a CBR value of 1% or an estimated subgrade reaction modulus (for concrete slabs or pavements) of 20kPa/mm (750mm diameter plate).

   OR

2. Provide an appropriate select fill layer as part of the overall pavement thickness. The select fill should be well graded ripped or crushed sandstone or good quality sandstone with a minimum soaked CBR value of 10%. The pavement sections where imported fill is used to raise site levels may be designed taking into account the thickness and soaked CBR value of the imported fill material. Consideration may be given to placing geotextile across the surface of the subgrade to improve the compaction of the select fill above and reduce pumping of fines into the select fill layer.

   OR

3. Stabilise the subgrade to a depth of about 300mm by the addition of lime. When thoroughly mixed and re-compacted to a minimum of 98% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved. As a guide, the addition of approximately 4% lime by dry weight of clay should result in a soaked CBR value of around 5% or an equivalent subgrade reaction modulus of 40kPa/mm. This should, however, be confirmed by laboratory testing. If lime stabilisation is undertaken, an experienced contractor with appropriate equipment should complete it and a number of lime demand tests will need to be undertaken prior to commencement to confirm the percentage of lime required to achieve the required soaked CBR value. We note

that use of lime close to residential, hospital and office areas is generally not preferred unless an acceptable method of dust suppression can be adopted.

Flexible pavements should be underlain by a good quality base-course layer comprising crushed rock to TfNSW QA specification 3051 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 have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD. Concrete pavements should be designed with an effective shear transmission of all joints by way of either doweled or keyed joints.

Careful attention to subsurface and surface drainage is required in view of the effect of moisture on the clay subgrade. The surface of the pavement and the subgrade should be sloped to shed water, and adequate subsurface drainage should be installed around the pavement to intercept and dispose of water flows. The subsoil drainage should extend at least 0.3m below the subgrade levels.

The pavement sections where imported fill is used to raise site levels, or replace unsuitable (heaving) subgrade by a depth of at least 0.5m may be designed on the basis of a four-day soaked CBR value of the imported fill material.

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.

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

SOIL TEST SERVICES

ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed TAMS Building
Location: TAMS, Derby Street, Kingswood, NSW

Report No.: 35033LT - A
Report Date: 22/06/2022

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	1.10 - 2.00	22.2	-	-	-	-
1	4.50 - 4.60	11.6	-	-	-	-
2	0.50 - 0.95	18.9	57	17	40	14.0
2	6.50 - 7.00	13.3	-	-	-	-
3	0.50 - 1.50	19.4	-	-	-	-
3	8.00 - 8.50	10.3	-	-	-	-
4	1.50 - 1.95	15.4	42	17	25	11.0
4	5.00 - 5.50	9.0	-	-	-	-

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: 01/06/2022.
- Sampled and supplied by client. Samples tested as received.

NATA
NATA Accredited Laboratory Number:1327

22/06/2022
Authorised Signature / Date
(D. Treweek)

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

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Report No.:**	35033LT - B
Project:	Proposed TAMS Building	**Report Date:**	7/06/2022
Location:	TAMS, Derby Street, Kingswood, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	BH 1	BH 3
DEPTH (m)	1.10 - 2.00	0.50 - 1.50
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.72 STD	1.75 STD
Optimum Moisture Content (%)	19.5	18.4
Moulded Dry Density (t/m^3)	1.69	1.73
Sample Density Ratio (%)	98	99
Sample Moisture Ratio (%)	97	97
Moisture Contents		
Insitu (%)	22.4	19.4
Moulded (%)	19.0	17.8
After soaking and		
After Test, Top 30mm(%)	29.8	31.1
Remaining Depth (%)	22.7	19.8
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	1.5	1.5
C.B.R. value: @2.5mm penetration	1.5	1.0

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: 01/06/2022.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

07/06/2022

Authorised Signature / Date
(D. Treweek)

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 296912

Client Details

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

Sample Details

Your Reference	**35033LT, Kingswood**
Number of Samples	3 Soil
Date samples received	01/06/2022
Date completed instructions received	01/06/2022

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	08/06/2022
Date of Issue	08/06/2022

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

Results Approved By
Nancy Zhang, Laboratory Manager, Sydney

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		296912-1	296912-2	296912-3
Your Reference	UNITS	1	2	4
Depth		0.6-0.95	1.5-1.95	4.0-4.5
Date Sampled		28/05/2022	28/05/2022	28/05/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	01/06/2022	01/06/2022	01/06/2022
Date analysed	-	06/06/2022	06/06/2022	06/06/2022
pH 1:5 soil:water	pH Units	8.2	7.6	4.3
Chloride, Cl 1:5 soil:water	mg/kg	250	620	1,900
Sulphate, SO4 1:5 soil:water	mg/kg	45	92	320
Resistivity in soil*	ohm m	45	21	7.4

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	296912-2
Date prepared	-			01/06/2022	1	01/06/2022	01/06/2022		01/06/2022	01/06/2022
Date analysed	-			06/06/2022	1	06/06/2022	06/06/2022		06/06/2022	06/06/2022
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	8.2	8.4	2	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	250	250	0	103	85
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	45	45	0	100	93
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	45	44	2	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

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BOREHOLE LOG

Borehole No.
1
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED TAMS BUILDING
Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 50.3m
Date: 28/5/22 **Datum:** AHD
Plant Type: JK400 **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		0		FILL: Gravelly sand, fine to coarse grained, dark brown, fine to medium grained sub-angular and angular igneous gravel, trace of silt.	M			APPEARS WELL COMPACTED
	N = 19 11,11,8			as above, but light brown, fine to medium grained sub-angular sandstone gravel.				
		1	CH	Silty CLAY: high plasticity, light brown and light grey, trace of fine to medium grained sub-angular ironstone gravel and roots.	w≈PL	VSt		RESIDUAL
	N = 14 3,6,8	2					290 290 340	
	N = SPT 14/50mm REFUSAL	3	-	Extremely Weathered claystone: silty CLAY, high plasticity, grey, occasional highly weathered, very low strength Claystone bands.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE WITH LOW BANDS
		4		CLAYSTONE: brown and dark grey.	DW	VL		LOW 'TC' BIT RESISTANCE
	N = SPT 17/100mm REFUSAL	5		as above, but dark grey, with extremely weathered bands.				
		6		CLAYSTONE: grey and dark grey.		VL-L		
		7						

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BOREHOLE LOG

Borehole No.
1
2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED TAMS BUILDING
Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 50.3m
Date: 28/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									CLAYSTONE: grey and dark grey.	DW	VL-L		LOW 'TC' BIT RESISTANCE
						8							
									CLAYSTONE: dark grey and grey, occasional grey brown, fine to medium grained sandstone bands.		L		LOW TO MODERATE 'TC' BIT RESISTANCE
						9			END OF BOREHOLE AT 9.0m				
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No.
2
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED TAMS BUILDING
Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 50.8m
Date: 28/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Gravelly sand, fine to coarse grained, dark brown, fine to medium grained, sub-angular and angular igneous gravel.	M			
		N = 7 2,2,5			CH	Silty CLAY: high plasticity, red brown, trace of fine to medium grained sub-angular gravel and roots.	w>PL	VSt	210 230 230	RESIDUAL
			1			as above, but light grey and red brown.				
		N = 17 4,8,9	2				w≈PL	VSt-Hd	300 390 410	
		N = SPT 16/50mm REFUSAL	3		-	Extremely Weathered claystone: silty CLAY: medium to high plasticity, brown and grey, occasional ironstone bands.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE, LOW BANDS
			4			CLAYSTONE: dark grey and grey brown.	DW	VL		LOW 'TC' BIT RESISTANCE
			5			as above, but grey and dark grey.		VL-L		
			6					L		LOW 'TC' BIT RESISTANCE, MODERATE BANDS
			7							

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BOREHOLE LOG

Borehole No.
2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED TAMS BUILDING
Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 50.8m
Date: 28/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									CLAYSTONE: grey and dark grey.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No. 3

1/2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED TAMS BUILDING

Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 51.6m

Date: 28/5/22 **Datum:** AHD

Plant Type: JK400 **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		0		FILL: Gravelly sand, fine to coarse grained, dark brown, fine to medium grained, sub-angular igneous gravel.	M			
	N = 7 2,3,4		CH	Silty CLAY: high plasticity, light brown, with roots, trace of fine to medium grained, sub-angular ironstone gravel.	w>PL	St-VSt	140 160 220	RESIDUAL
		1		Silty CLAY: high plasticity, light brown and red brown, trace of fine to medium grained, sub-angular ironstone gravel.	w≈PL	VSt		
	N > 14 14,14/ 70mm REFUSAL	2	-	Extremely Weathered claystone: silty CLAY: medium to high plasticity, brown mottled grey.	XW	Hd	>600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
				as above, but occasional highly weathered, very low strength bands.				VERY LOW TO LOW 'TC' BIT RESISTANCE
	N > 14 10,14/ 30mm REFUSAL	3		CLAYSTONE: grey brown, with extremely weathered claystone bands.	DW	VL		
		4						
		5						
		6		as above, but grey and dark grey.		VL-L		LOW 'TC' BIT RESISTANCE
		7						

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BOREHOLE LOG

Borehole No. 3

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED TAMS BUILDING

Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 51.6m

Date: 28/5/22 **Datum:** AHD

Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									CLAYSTONE: dark grey and grey.	DW	VL-L		
						8					L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 8.5m				
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 4

1/2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED TAMS BUILDING

Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 51.2m

Date: 28/5/22 **Datum:** AHD

Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Gravelly sand, fine to coarse grained, sub-angular and angular igneous gravel.	M			
						FILL: Silty clay, medium to high plasticity, dark brown and grey, trace of fine to medium grained, sub-angular and angular igneous ironstone gravel.	w>PL			
		N = 11 2,4,7	1		CH	Silty CLAY: high plasticity, light grey and red brown, trace of ironstone bands and roots.	w≈PL	VSt	300 310 400	RESIDUAL
		N = 26 4,11,15			CI	Silty CLAY: medium plasticity, light grey and red brown, trace of ironstone and extremely weathered claystone bands.	w<PL	Hd	>600 >600 >600	
		N > 14 4,14/ 150mm	2 3		-	Extremely Weathered claystone: silty CLAY: high plasticity, light grey and red brown, with clay bands.	XW	Hd	500 540	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE TO SOIL RESISTANCE
						CLAYSTONE: grey and red brown.	DW	VL	>600 >600	VERY LOW 'TC' BIT RESISTANCE
			4			as above, but grey brown.		VL-L		
			5					L		LOW 'TC' BIT RESISTANCE
			6 7			as above, but dark grey and grey.		L-M		LOW TO MODERATE 'TC' BIT RESISTANCE

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BOREHOLE LOG

Borehole No.
4

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED TAMS BUILDING
Location: DERBY STREET, KINGSWOOD, NSW

Job No.: 35033LT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 51.2m
Date: 28/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** B.Z./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									CLAYSTONE: dark grey and grey.	DW	L-M		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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, KINGSWOOD, NSW
Report No: 35033LT	Figure No: 1

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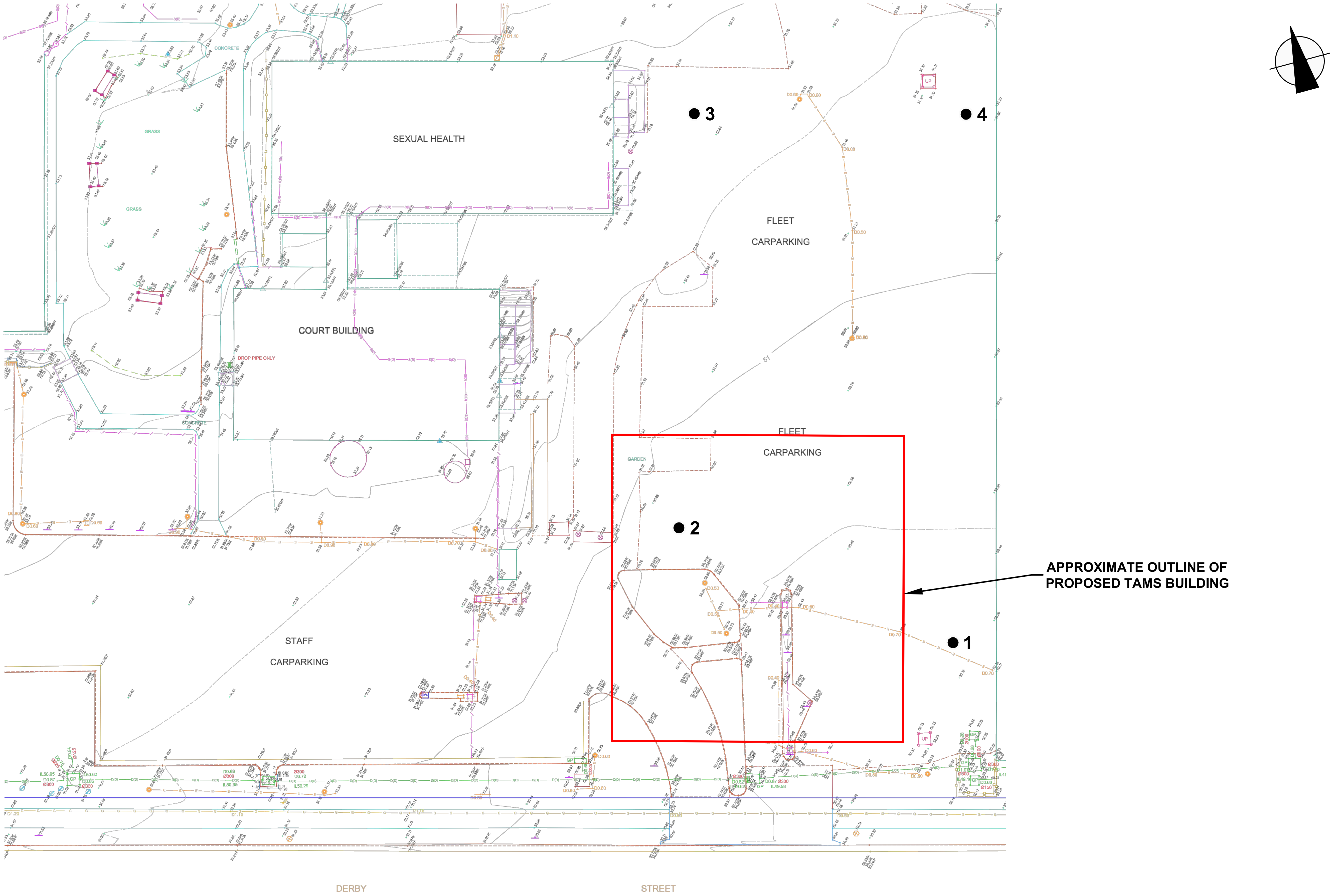
PLOT DATE: 29/06/2022 1:32:33 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35033LT KINGSWOOD\CAD\35033LT.DWG



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, KINGSWOOD, NSW
Report No:	35033LT
Figure No:	2

JKGeotechnics

PLOT DATE: 5/07/2022 9:00:05 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35033LT KINGSWOOD\CAD\35033LT.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		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:

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

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

NOTES:

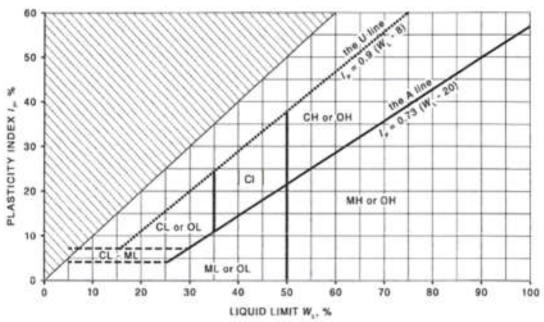
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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