REPORT

TO

NSW DEPARTMENT OF EDUCATION

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED JORDAN SPRINGS PUBLIC SCHOOL

AT

14-28 CULLEN AVENUE, JORDAN SPRINGS, NSW

15 February 2019

Ref: 30718AH2rpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Jordan Springs Public School at 14-28 Cullen Avenue, Jordan Springs, NSW. The location of the site is shown on Figure 1.

Based on the supplied architectural drawings prepared by Group GSA (Drawing Nos. A-L0000^{D}, A-1000^{D}, A-1001^{D}, A-1100^{D}, A-1101^{D}, A-1120^{D}, A-1121^{D}, A-1122^{D}, A-3020, A-3021^{D}, A-6202^{D}, A-7500^{D}, A-7501^{D}, A-7502^{D} and L-1000^{D} to L-1004^{D}, L-500^{D}, L6001^{D} and L6002^{D}, dated 25 January 2019), we understand that the proposed new school will include construction of several two storey buildings. The ground floor levels will be constructed with a finished floor level at reduced level (RL) 40.7m. We have assumed that structural loads typical for a two storey building apply. A basketball court and an on-grade car park are proposed at the south-western corner of the site and will have finished surface levels at RL40.8m and RL40.75m, respectively. Cut and fill earthworks to a maximum depth/height of about 1m and 0.5m, respectively, are expected. A sports oval is proposed over the north-western portion of the site.

In 2017, JK Geotechnics investigated the site for a similar proposed development (report Ref. 30718Zrpt, dated 25 August 2017). The original development details have since been revised. We have used the results of our previous investigation in the preparation of the current report.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at three borehole locations, as a basis for preliminary comments and recommendations on earthworks, retaining wall design, footings, earthquake design parameters, on-grade floor slabs, external concrete pavements and additional investigations.

Our environmental consulting division, Environmental Investigation Services (EIS), was also commissioned to undertake a Preliminary Environmental Site Assessment. This report should therefore be read in conjunction with the EIS report, Ref. E30719KPrpt-rev1, dated 23 January 2019.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 27 July 2017 and comprised three boreholes (BH1, BH2 and BH3) which were completed using a combination of auger drilling and push tube techniques to 6.5m depth using our four wheel drive Eziprobe rig. Prior to the commencement of works, a specialist sub-consultant reviewed available 'Dial Before You Dig' information and scanned the borehole locations for buried services using electro-magnetic techniques.

The borehole locations are shown on the attached Figure 2 and were set out using tape measurements off existing surface features. The approximate surface RL's indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on a supplied unreferenced survey plan. The survey datum was not shown on the drawing and as such has been assumed.

The relative compaction/strength of the subsoils was assessed from hand penetrometer readings on clay samples recovered in the push tube sampler, and by tactile examination. The strength of the bedrock was assessed based on auger penetration resistance, examination of the recovered rock cuttings and correlation with subsequent laboratory moisture content test results. Groundwater observations were also made in the boreholes during the fieldwork. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineer was present full time during the fieldwork to set out the borehole locations, direct the electro-magnetic scanning, nominate the testing and sampling, and prepare the attached borehole logs. The Report Explanation Notes define the logging terms and symbols used.

Selected soil and rock cutting samples were recovered from site and submitted to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for moisture content, Atterberg Limits and linear shrinkage testing. The test results are summarised in the attached STS Table A.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The following site description was prepared at the time of our fieldwork in July 2017. With reference to recent Nearmap aerial images of the site, the site appears essentially the same as it was when the fieldwork was carried out.

The site is located in gently undulating topography and generally falls towards the south-east at approximately 1° to 2°. The site is bound by Cullen Avenue to the south, community buildings and car parks to the south-west, Lakeside Parade to the west, a creek to the east, and residential properties to the north.

At the time of the fieldwork, the site was covered with patchy grass and the mid-southern portion was occupied by Landscape Solutions' compound and storage area. The compound and driveway entrance off Cullen Avenue was gravel covered. An earth embankment about 1m to 1.5m high was present just inside the western end of the southern site boundary. An electrical kiosk was located at the south-western corner of the site.

The neighbouring properties to the north of the site which were vacant at the time of our fieldwork are now occupied by residences. A two storey child care centre and a single storey community centre, both with external car parks ad driveways were located to the south-west of the site. A creek was located about 30m to the east of the site.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith indicates the site is underlain by Bringelly Shale of the Wianamatta Group, which consists of *'shale, carbonaceous claystone, claystone, laminite, finite, medium grained lithic sandstone, rare coal and tuff'*.

Generally, the boreholes encountered moderately deep clay fill overlying residual silty clay, then shale bedrock at depth. Groundwater was not encountered in the boreholes. Reference should be made to the attached borehole logs for details at each specific location. A summary of the encountered subsurface characteristics is provided below:

- Fill comprising silty clay, gravelly clay and sandy clay was encountered in each borehole to depths of 2.3m (BH1), 2.8m (BH2) and 2.9m (BH3). The fill was assessed to be moderately to well compacted.

- Residual silty clay of high plasticity and of generally stiff to hard strength was encountered below the fill in each borehole.
- Weathered shale bedrock was encountered at depths of 4.6m (BH1) and 6.0m (BH2 and BH3). The shale was generally extremely and distinctly weathered and of extremely low (‘hard’ soil strength), very low and low strength.
- All three boreholes were ‘dry’ during and on completion of drilling. We note that groundwater levels may not have stabilised within the short observation period. No long-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The Atterberg Limits and linear shrinkage test results confirmed the clayey fill samples from BH1 and BH2 and the residual silty clay samples from BH1 and BH3 to be of high plasticity, and indicated a high potential for shrink-swell reactivity with changes in moisture content.

The moisture content tests carried out on recovered rock cutting samples correlated well with our field assessment of rock strength.

4 PRELIMINARY COMMENTS AND RECOMMENDATIONS

4.1 Additional Geotechnical Investigation

The comments and recommendations provided in this report are based on three boreholes located at the south-eastern corner of the site. As such, the advice provided is preliminary and generalised, and will need to be reviewed and updated following completion of a more comprehensive investigation.

We strongly recommend that prior to finalising the structural and civil designs, nine additional boreholes (i.e. five for the proposed buildings and four for the proposed on-grade car park and basketball court) be completed with a drilling rig to confirm the subsurface conditions across the development area. We can provide a fee proposal for this additional work, if requested to do so.

We also recommend that an attempt be made to obtain information on the existing fill, to assess whether it has been placed and compacted in a ‘controlled’ manner. Refer to our comments below in Section 4.2.

4.2 Existing Fill

The investigation has indicated that the site, at least over its south-eastern corner, has been filled. The depth of the fill in the boreholes was between 2.3m (BH1) and 2.9m (BH3).

Whilst the fill was assessed to be moderately to well compacted based on hand penetrometer readings, these tests do not provide a direct quantitative assessment of the insitu density of the fill. The hand penetrometer readings are affected by the presence of gravel and the moisture content of the fill.

We are unaware of records that document the manner of placement, compaction specification and control of the fill. Hence, the fill is deemed to be not a controlled fill as defined in Clause 1.8.13 of AS2870-2011 'Residential Slabs and Footings'. The site classification is therefore considered to be Class 'P'. Notwithstanding, the standard footing designs provided in AS2870 2011 do not apply for the proposed development and a Class 'P site, and therefore the footing design will need to be carried out using engineering principles.

As the site is located within a relatively new subdivision, we recommend that records be obtained from Council or the developer on the specification, placement, compaction and testing of the existing fill. If the records show that the fill has been placed to Level 1 control in accordance with AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments' and to an appropriate specification, then it may be possible to reclassify the site, in accordance with AS2870-2011. We would be happy to prepare a proposal to review available information on the existing fill, if requested.

4.3 Earthworks

All earthworks recommendations provided below should be complemented by reference to AS3798-2007.

The earthworks for the proposed buildings will depend on whether the ground floor slabs are designed as suspended or as stiffened rafts, as discussed further below in Section 4.5. A stiffened raft slab will only be suitable, if records show that the existing fill is a 'controlled' fill that has been placed and compacted to Level 1 control in accordance with AS3798-2007 and to an appropriate specification.

4.3.1 Site Preparation

All vegetation, topsoil, root affected soils and any fill containing deleterious or contaminated soil should be stripped from below the proposed development footprint. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes, subject to approval from EIS. Reference should be made to the EIS report for guidance on the offsite disposal of soil.

Excavation through the soil profile down to design subgrade levels can be completed using hydraulic excavators and dozers.

4.3.2 Batter Slopes

Space permitting, temporary batter slopes through the soil profile and of fill embankments are feasible and should be cut no steeper than 1 Vertical (V) on 1 Horizontal (H), provided surcharge loads are kept well clear from the crests of the batters. Retaining walls can then be constructed along the toes of the temporary batter slopes and subsequently backfilled.

Where spatial constraints do not permit temporary batter slopes, then further geotechnical advice should be sought from JK Geotechnics.

If permanent batter slopes can be accommodated within the site, these should be graded at no steeper than 1V on 2H. Surface erosion protection, for example, quick establishing grass and/or proprietary systems (such as those provided by Geofabrics Australasia or Global Synthetics) should be provided to the permanent batter slopes. Dish drains should also be provided along the crest of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system for disposal.

4.3.3 Site Drainage

The subgrade at the site is expected to undergo a substantial loss in strength when wet. Furthermore, the subgrade is expected to have a high shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained subgrade may become untrafficable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

Due to the potential for the subgrade to soften in the presence of water, consideration should be given to the provision of a select subgrade ('working platform') layer comprising a well graded, durable granular material, such as crushed or processed sandstone.

4.3.4 Subgrade Preparation

The following advice should be followed for any stiffened raft slabs, on-grade floor slabs and external pavements.

Following stripping and excavation down to design subgrade levels, the subgrade should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of any 'unstable' areas.

Subgrade heaving during proof rolling should be expected in areas where the subgrade has become 'saturated' or where 'under-compacted' existing fill exists. The heaving areas can typically be improved by locally removing the heaving material to a stable base and replacing with engineered fill, as outlined below. Options and detailed design of subgrade improvement works must be provided by the geotechnical engineer following the proof rolling inspection.

If soil softening occurs after rainfall periods, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the subgrade exhibits shrinkage cracking, then the surface must be moistened and rolled until the shrinkage cracks are no longer evident. Care must be taken not to over-water the subgrade as this will result in softening.

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

If the slabs are designed as suspended, then proof rolling of the subgrade is considered to be unwarranted.

4.3.5 Engineered Fill

General

From a geotechnical perspective, the excavated clay fill is considered suitable for reuse as engineered fill, on condition that it is 'clean', free of organic matter and contain a maximum particle size of 100mm. Some moisture conditioning of the clays in order to conform to the specification provided below should be expected.

If there is a short fall in site-won material, then all imported material must be classified as Virgin Excavated Natural Material (VENM) and our preference would be for a select well graded, granular material such as crushed or processed sandstone, free of organic matter or other deleterious substances, with a maximum particle size not exceeding 100mm.

Engineered fill comprising clay materials should be compacted in maximum 300mm thick loose layers using a large static pad-foot roller (say, at least 15 tonnes deadweight) to a density ratio strictly between 98% and 102% of Standard Maximum Dry Density (SMDD) and at a moisture content within 2% of Standard Optimum Moisture Content (SOMC). Engineered fill comprising an imported select, well graded, granular material such as crushed or processed sandstone, should be compacted in maximum 300mm thick loose layers using a large static roller to achieve a minimum density ratio of 98% of SMDD.

If the engineered fill is located in landscaped areas, then the minimum density ratio can be relaxed to at least 95% of SMDD.

If lighter compaction plant is proposed, then thinner placement layers will be required. If the earthworks contractor wishes to use the vibratory mode on the roller then trials will need to be completed to assess vibration levels of the nearby residences to the north, and community buildings to the south-west. Further geotechnical advice should be sought in respect to both scenarios.

Edge Compaction

In order to achieve adequate edge compaction where fill platforms are proposed, we recommend that the outer edge of each fill layer extend a horizontal distance of at least 1m beyond the design geometry. The roller must extend just over the edge of each placed layer in order to seal the batter surface. On completion of filling, the excess under-compacted edge fill should be trimmed back to the design geometry.

Service Trenches

Backfilling of service trenches must be carried out using engineered fill to reduce post-construction settlements. Due to the reduced energy output of compaction plant that can be placed in trenches, backfilling should be carried out in maximum 150mm thick loose layers and compacted using a trench roller, a pad-foot roller attachment fitted to an excavator and/or a vertical rammer compactor, also known as a ‘Wacker Packer’. Due to the reduced loose layer thickness, the maximum particle size of the backfill material should reduce to 50mm. The compaction specifications provided above are applicable.

Retaining Wall Backfill

Backfilling behind retaining walls must also be carried out using engineered fill to reduce post-construction settlements. Compaction of the engineered backfill should be carried out using a vertical rammer compactor for the lower layers and immediately behind the wall in the upper layers. Elsewhere a small static roller should be used. As for services trenches, backfilling should be carried out in maximum 150mm thick loose layers and the maximum particle size of the backfill material should be no more than 50mm. The compaction specifications provided above are applicable.

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

Earthworks Inspection and Testing

Density tests should be carried out on all engineered fill to confirm the above compaction specifications are being achieved. Following completion of the additional investigation, we will nominate testing frequencies for the various aspects of the earthworks.

Due to the potential for the subgrade to soften in the presence of water and the nature of the proposed development, we recommend that Level 1 control of fill placement and compaction in accordance with AS3798-2007 be carried out, including for the trench and retaining wall backfill. Due to a potential conflict of interest, the geotechnical inspection and testing authority (GITA) should be directly engaged by the Department of Education or their representative, and not by the contractor.

Construction of high level footings, ground floor slabs and on-grade pavements supported on engineered fill platforms should only commence once the Level 1 earthworks report has been submitted by the GITA and reviewed and approved by the Project Superintendent and/or JK Geotechnics.

4.4 Retaining Walls

Cantilevered retaining walls located in areas where some movement can be tolerated and which are independent of the proposed structures, should be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient (K_a) of 0.35 for the soil profile, assuming a horizontal backfill surface.

Cantilevered retaining walls located in areas where movements are to be reduced, or where they are restrained by the proposed structures, should be designed using a triangular lateral earth pressure distribution and an 'at-rest' earth pressure coefficient (K_0) of 0.55 for the soil profile, assuming a horizontal backfill surface.

A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.

Any surcharge affecting the walls (eg. construction traffic, pavement and slab loads, compaction stresses during backfilling, inclined backfill surface, etc.) should be allowed in the design using the appropriate earth pressure coefficient provided above. The retaining walls should be designed as permanently drained. Subsurface drains should incorporate a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion. Discharge should be piped to the stormwater system for disposal.

Retaining walls independent of the proposed structures and founded in engineered fill (to Level 1 control) may be designed for maximum allowable bearing pressure of 100kPa. If the retaining walls are founded in the existing fill (with no documentation that it was placed and compacted to Level 1 control and to an-appropriate specification), then a maximum allowable bearing pressure of 50kPa, should be adopted. The passive lateral toe resistance for footings founded in existing fill or engineered fill may be estimated using a 'passive' earth pressure coefficient (K_p) of 2.5 or 3.0 respectively (but with a Factor of Safety of at least 2 to limit deformations associated with achieving a full passive condition), assuming horizontal ground in front of the wall.

All retaining wall footing excavations should be cleaned out, inspected by a geotechnical engineer to confirm that a satisfactory bearing stratum has been achieved, and poured on the same day as excavation.

4.5 Footings

4.5.1 Site Classification

Based on the investigation results and due to the depth of the existing fill, the site is Class ‘P’ in accordance with AS2870-2011. The footings will therefore need to be designed using engineering principles.

4.5.2 High Level Footings

High level strip and/or pad footings or stiffened raft slab edge and internal beams founded in the existing fill which has been confirmed to be a ‘controlled’ fill and/or in engineered fill (to Level 1 control in accordance with AS3798-2007) may be adopted. These footings should be tentatively designed for a maximum allowable bearing pressure of 100kPa and should be founded at least 0.8m below the adjacent finished ground surface level to reduce the effects of potential shrink-swell movements of the clay soils. The shrink-swell movements below each structure will be a function of the fill depth and age. As a guide, we expect that characteristic surface movements will be in the order of at least 60mm (ie. equivalent to a Class ‘H2’ or Class ‘E’ site).

All building footing excavations must be inspected and tested by a geotechnical engineer prior to pouring to confirm that satisfactory founding material has been exposed.

We recommend that the footing excavations be cleaned out, inspected and poured with minimum delay to avoid deterioration. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration. Water should be avoided from ponding in the base of footing excavations as this will soften the foundation material, resulting in further excavation and cleaning being required.

4.5.3 Pile Footings

An alternative to high level footings or if the existing fill cannot be confirmed to be ‘controlled’, would be to support the proposed structures on conventional bored piles. Steel screw (helix) piles may be feasible subject to trials, to assess if the piles can adequately penetrate the existing fill.

Bored piles founded at a minimum depth of 2m and at least two pile diameters into the residual silty clays of at least stiff strength may be tentatively designed for a maximum allowable end bearing pressure of 300kPa.

If higher bearing pressures are required, then bored piles socketed at least 0.3m into shale bedrock may be designed for an allowable bearing pressure of 700kPa. Pile sockets formed below the nominal 0.3m length requirement may be designed for maximum allowable shaft adhesion values of 70kPa (in compression) and 35kPa (in tension), on condition that the pile shaft is suitably roughened using a grooving tool fitted to the side of the auger.

The feasibility of higher bearing pressures on deeper, more competent bedrock (if present), will be assessed following completion of the additional investigation recommended in Section 4.1.

Bored piles should be cleaned-out, 'dry', inspected by a geotechnical engineer and poured on the same day as drilling.

If screw piles are installed, then we assume that the piling contractor would be responsible for certifying the load capacity of the piles.

Due to the potential for swell pressures from the clay soils, we recommend that ground beams between piles and any suspended floor slabs be poured over void formers, which can tentatively accommodate heave movements of at least 50mm so as to isolate the structural members from the underlying clays. The void former performance criteria for each structure will be assessed following completion of the additional investigation.

4.5.4 Earthquake Design Parameters

A Hazard Factor (Z) of 0.09 and a Site Subsoil Class C_e can be adopted for earthquake design in accordance with AS1170.4-2007 ('Structural Design Actions, Part 4: Earthquake Actions in Australia', including Amendment Nos 1 & 2).

4.6 On-Grade Floor Slabs

Unless incorporated into raft slabs, we recommend that the ground floor slabs be designed as suspended on the footings and poured over void formers as discussed in Section 4.5.3.

Alternatively, slab-on-grade construction is considered feasible provided the subgrade is prepared as discussed above in Section 4.3.4 and the existing fill has been confirmed to be a 'controlled' fill. On-grade floor slabs must be isolated from the building walls, columns and footings. Joints should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints. However, there will be differential movements between the walls/columns and ground floor slabs due to shrink-swell movements of the underlying clays. Careful detailing between the floor slabs and walls/columns will therefore be required. To reduce the effects of shrink-swell movements in the underlying clays on the proposed buildings, we recommend that the external walls of the buildings be protected with perimeter apron slabs at least 2m wide, which grade away from the buildings. The gap between the building and apron slab, as well as any transverse joints in the slab, must be appropriately sealed to prevent water ingress.

4.7 External Concrete Pavements

We recommend that the proposed concrete car park pavements be tentatively designed on the basis of a CBR value of 3.0% or a Short Term Young's Modulus of 22MPa, provided that the subgrade is prepared as per our advice above in Section 4.3.4. We strongly recommend that CBR testing of the subgrade be carried out as part of the additional geotechnical investigation.

We recommend that all unbound granular sub-base materials comprise DGB20 in accordance with RMS QA Specification 3051. The DGB20 material should be compacted in maximum 200mm thick loose layers using a static smooth drum roller to at least 98% of Modified Maximum Dry Density. Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content should be provided during placement so as to reduce the potential for material breakdown during compaction.

The sub-base material aims to provide uniform slab support and reduce 'pumping' of subgrade 'fines' at joints due to vehicular movements.

Density tests should be carried out on the granular sub-base materials to confirm the above specification is achieved. Level 2 control of sub-base compaction is the minimum permissible under AS3798-2007. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by the Department of Education or their representative.

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

4.8 Further Geotechnical Input

We summarise below the previously recommended additional work that needs to be carried out:

1. An additional geotechnical investigation including the drilling and testing of nine boreholes and CBR testing and updating of this report.
2. Review of available records on the existing fill.
3. Proof rolling inspections.
4. Inspection and testing of all engineered fill to Level 1 control by a GITA.
5. Review of the Level 1 report.
6. Density testing of all granular sub-base materials to at least Level 2 control by a GTA.
7. Footing/pile inspections.

5 SALINITY

The site is located in an area where soil and groundwater salinity may 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 preliminary advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications should only be prepared based on our final report following completion of the additional investigation.

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 LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed New School
Location: 14-28 Cullen Avenue, Jordan Springs, NSW

Ref No: 30718Z
Report: A
Report Date: 7/08/2017

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	0.20-0.40	17.2	52	21	31	14.5
1	2.60-2.80	14.8	57	20	37	15.0
1	5.00-6.00	7.3				
2	0.60-0.80	18.4	56	21	35	15.0
2	6.00-6.50	8.5				
3	2.90-3.00	20.6	55	20	35	15.5
3	6.20-6.50	14.5				

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: 31/07/2017

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

1/1

Client:

Project: PROPOSED NEW SCHOOL

Location: 14-28 CULLEN AVENUE, JORDAN SPRINGS, NSW

Job No. 30718Z **Method:** SPIRAL AUGER/ EZIPROBE **R.L. Surface:** ≈ 40.2m

Date: 27/7/17 **Datum:** ASSUMED

Logged/Checked by: D.A.F./A.Z.

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: Silty clay, high plasticity, brown, light grey and orange brown, with bands of fine to coarse grained shale gravel.	MC<PL		500 570 480 380 340	APPEARS MODERATELY TO WELL COMPACTED
			1							
			2							
					CH	SILTY CLAY: high plasticity, light grey and orange brown, trace of fine to coarse grained ironstone gravel.	MC<PL	H	460 500 530 450 450	
			3							XW SANDSTONE BAND, 0.1mm.t
			4							
					-	SHALE: brown and grey.	DW	VL-L		LOW 'TC' BIT RESISTANCE
			5							
			6							
						END OF BOREHOLE AT 6.5m				
			7							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client:
Project: PROPOSED NEW SCHOOL
Location: 14-28 CULLEN AVENUE, JORDAN SPRINGS, NSW

Job No. 30718Z
Date: 27/7/17
Method: SPIRAL AUGER/ EZIPROBE
R.L. Surface: ≈ 40.2m
Datum: ASSUMED
Logged/Checked by: D.A.F./A.Z.

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: Silty clay, low to medium plasticity, brown and orange brown, with fine to coarse grained sandstone and ironstone gravel, trace of sand.	MC≈PL			APPEARS MODERATELY COMPACTED
						FILL: Silty clay, high plasticity, orange brown and brown, trace of fine to medium grained ironstone gravel, and fine to coarse grained shale gravel.	MC>PL		370 200 280 420	
			1			FILL: Gravelly clay, low plasticity, orange brown and brown, fine to coarse grained shale gravel, with sand.	MC<PL		340 500 450	
			2			FILL: Silty clay, high plasticity, light grey and brown, trace of medium to coarse grained igneous gravel and concrete fragments.	MV>PL		120 140 160	
			3		CH	SILTY CLAY: high plasticity, light grey and red brown, trace of fine to medium grained ironstone gravel.	MC≈PL	VSt	250 250 270 350 270 330	RESIDUAL
			4				MC>PL	St	200 200 180 190 170 180	
			5							
			6		-	SHALE: brown and grey.	DW	VL		LOW 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 6.5m				
			7							

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

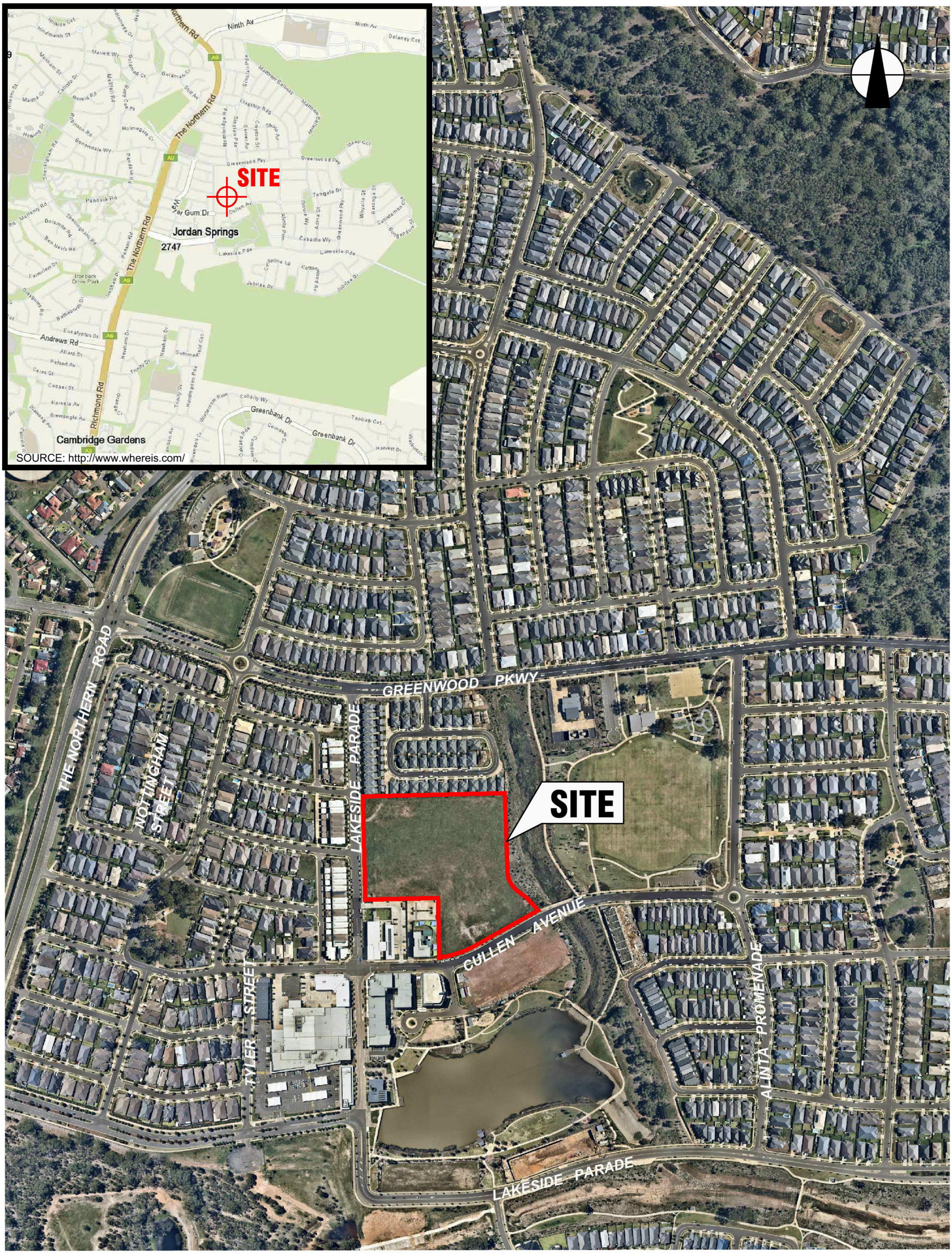
BOREHOLE LOG

Borehole No.
3
1/1

Client:
Project: PROPOSED NEW SCHOOL
Location: 14-28 CULLEN AVENUE, JORDAN SPRINGS, NSW

Job No. 30718Z
Date: 27/7/17
Method: SPIRAL AUGER/ EZIPROBE
R.L. Surface: ≈ 39.9m
Datum: ASSUMED
Logged/Checked by: D.A.F./A.Z.

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: Silty clay, medium to high plasticity, light grey and red brown, trace of sand, fine to coarse grained igneous and ironstone gravel.	MC<PL			TOO FRIABLE FOR HP TESTING
						FILL: Sandy clay, low plasticity, with fine to medium grained igneous gravel, trace of concrete fragments.				APPEARS MODERATELY TO WELL COMPACTED
			1			FILL: Silty clay, high plasticity, brown grey and orange brown, trace of fine to coarse grained ironstone gravel.	MC>PL		320 350 350 380 380 380 360	
			2				MC≈PL		420 420 500 420 460 520	
						SILTY CLAY: low to medium plasticity, dark grey, brown and red brown, trace of fine to medium grained ironstone, shale and igneous gravel.				
			3		CH	SILTY CLAY: high plasticity red brown and light grey, trace of fine to medium grained ironstone gravel.	MC>PL	St-VSt	200 230 200 220 200 230 280	RESIDUAL
			4							
			5							
			6		-	SHALE: brown and grey.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 6.5m				
			7							



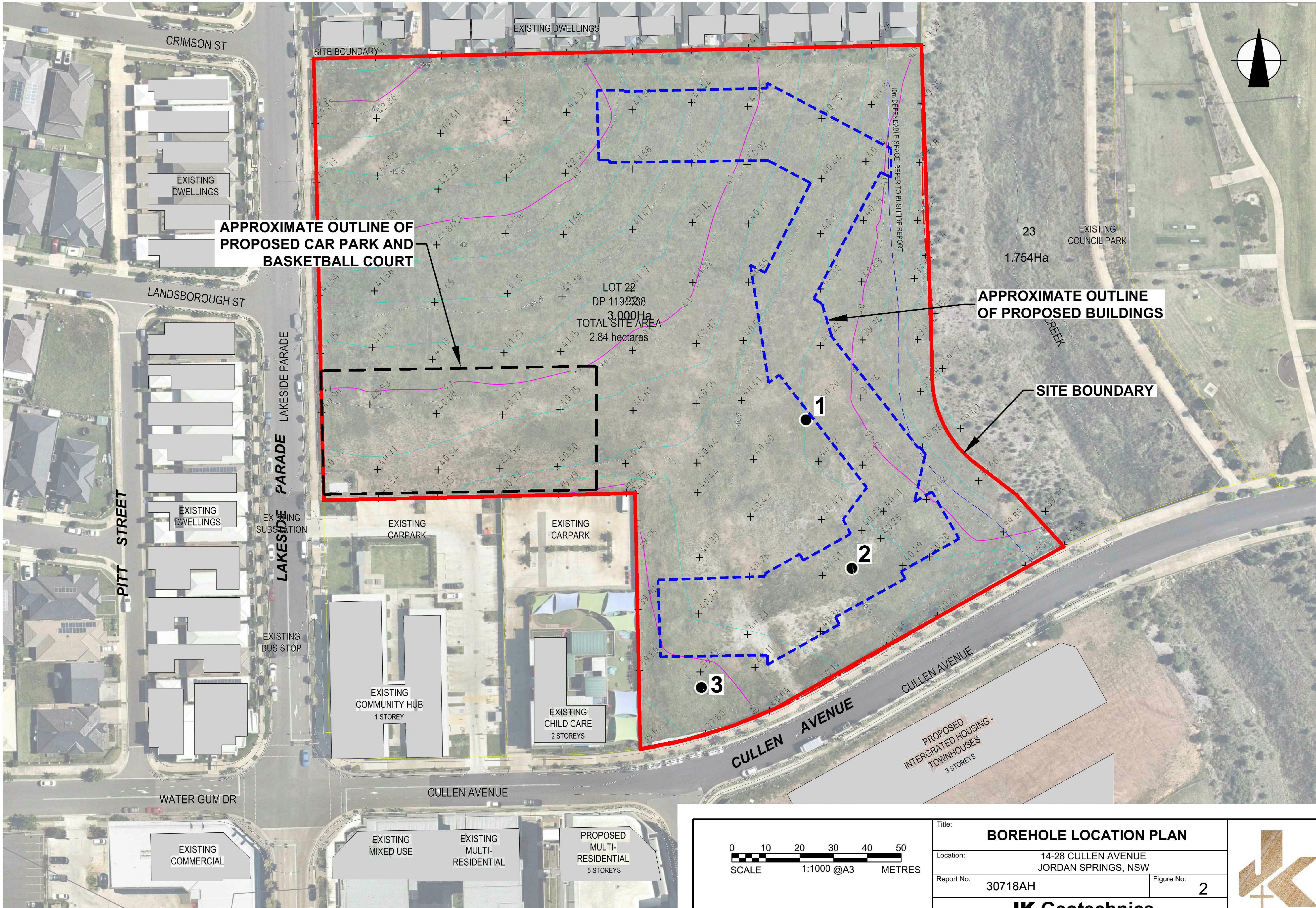
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 29 DEC 2018.

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

Title:	**SITE LOCATION PLAN**
Location:	14-28 CULLEN AVENUE JORDAN SPRINGS, NSW
Report No: 30718AH	Figure No: 1

JK Geotechnics

PLOT DATE: 4/02/2019 1:19:31 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30718Z JORDAN SPRINGS\CAD\30718AH.DWG



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

Title:	**BOREHOLE LOCATION PLAN**
Location:	14-28 CULLEN AVENUE JORDAN SPRINGS, NSW
Report No: 30718AH	Figure No: 2
	JK Geotechnics

PLOT DATE: 4/02/2019 1:16:39 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30718Z JORDAN SPRINGS\CAD\30718AH.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.

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

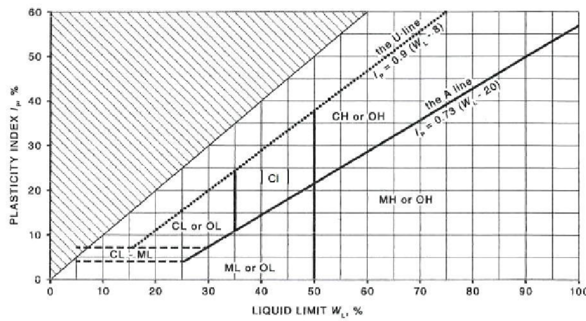
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w = PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ 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

<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 $I_{s(50)}$ (MPa)	Guide to Strength: 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
		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