REPORT

TO

CADENCE PROPERTY GROUP PTY LTD

ON

DUE DILIGENCE GEOTECHNICAL INVESTIGATION

FOR

PROPOSED WAREHOUSE

AT

128 ANDREWS ROAD, PENRITH, NSW

17 August 2018
Ref: 31675LBrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 17 August 2018
Report No: 31675LBrpt
Revision No:

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For and on behalf of
JK GEOTECHNICS
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1 INTRODUCTION

This report presents the results of a due diligence geotechnical investigation for the proposed warehouse at 128 Andrews Road, Penrith, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Mitchell Kent of Cadence Property Group Pty Ltd, and was carried out in accordance with our proposal dated 10 July 2018, Ref: P47587B. This report confirms and amplifies preliminary information emailed to Cadence Property Group on 10 August 2018.

The development is only at concept stage, however we understand that a 50,000m^2 warehouse is proposed within the 85,000m^2 site. Two options for the development have been supplied to us as shown in the drawings by Cadence (Drawing No. 1805-142-SK-001, Revision A, dated 22/3/18, and Drawing No. 1805-142-SK-011, Revision A, dated 18/4/18). These options show the same size warehouse located slightly differently within the site. The warehouse will be accessed by pavements adjacent to the warehouse, and by a driveway constructed within the right of way off Lambridge Place. At this stage we have not been provided with a survey plan for the site or design levels. Therefore, for the purposes of this due diligence investigation we have assumed that the development will be constructed at about the existing surface level, possibly with some cut and fill earthworks of about 1m.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for preliminary comments and recommendations on geotechnical issues for the proposed development, such as earthworks, footings, floor slabs and pavements.

This geotechnical investigation was carried out in conjunction with a preliminary environmental site assessment by our specialist division, Environmental Investigation Services (EIS). Reference should be made to the separate report by EIS, Ref: E31675KR, for the results of the environmental assessment.

2 INVESTIGATION PROCEDURE

The assessment initially comprised a desktop study of previous geotechnical investigations we have carried out within the vicinity of the site to gain an understanding of the likely subsurface conditions. Following that, a limited scope geotechnical investigation of the site was carried out, and this comprised the auger drilling of five boreholes (BH1 to BH5) using our track mounted JK300 drilling rig. BH1 to BH4 were drilled as close as practical to the locations nominated by Cadence, however, as some time was available on the day of drilling, an additional borehole, BH5, was also drilled. The boreholes were drilled to refusal within gravel at depths ranging from 3.0m to 4.9m below the existing ground surface.

The borehole locations, as shown on Figure 2, were set out using a Topcon GRS-1 differential GPS surveying unit. The measured surface levels are shown on the borehole logs and are based on the Australian Height Datum (AHD).

The strength and relative density of the alluvial soils were obtained from Standard Penetration Test (SPT) ‘N’ values, augmented where possible with hand penetrometer tests on more clayey samples recovered from the SPT split tube sampler. Due to the friable nature of the silty soils, hand penetrometers were not possible on the majority of SPT samples. The relative density of the gravels, which caused refusal of the auger, were assessed from the results of SPT and Solid Cone Penetration tests as well as from the resistance to penetration of a Tungsten Carbide (TC) bit attached to the augers.

Groundwater observations were made during and on completion of drilling. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Mr Arthur Billingham, was present on site full-time during the borehole drilling and set out the borehole locations, nominated the sampling and in-situ testing locations, and prepared logs of the strata encountered. The borehole logs are attached to this report, together with a set of explanatory notes which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd, a NATA accredited laboratory, for testing to determine moisture contents, Atterberg limits, linear shrinkages, standard compaction properties, and four day soaked CBR values. The results of the laboratory testing are summarised in the attached STS Tables A and B. Samples were also collected from the boreholes for testing as part of the preliminary environmental site assessment by EIS.

3 RESULTS OF INVESTIGATION

3.1 Desktop Study

The Penrith 1:100,000 Geological Series Sheet 9030 indicates that the site is underlain by the Cranebrook Formation comprising Quaternary deposits of *“gravel, sand, silt and clay”*. A search of our project database has revealed that we have carried out several geotechnical investigations within the vicinity of the site as detailed below.

126 Andrews Road

This site is located about 400m to the north-east of the subject site and the investigation comprised the drilling of boreholes to a maximum depth of 5m. The boreholes encountered fill covering alluvial silty clays that were assessed to be mostly of very stiff to hard strength with some firm to stiff layers. Groundwater was only encountered in one borehole at a depth of about 2m.

Lambridge Place

An investigation was completed for the subdivision either side of Lambridge Place, which is to the north of the subject site. The boreholes drilled close to the subject site encountered predominantly clayey sand with some sandy clay and refused on alluvial gravels at depths ranging from 2.8m to 6.0m. The clayey sands were assessed to generally be of medium dense relative density. Groundwater seepage was encountered in one borehole at a depth of 5m.

2115 Castlereagh Road

An investigation was carried out at the western end of this site, which is to the south-west of the subject site. Those boreholes were drilled to depths of 4.25m and encountered surface fill covering silty clay, which was generally assessed to be of very stiff strength with some stiff and hard layers. These boreholes were terminated within the clays and did not encountered any gravels. No groundwater was encountered during drilling of the boreholes.

Penrith Treatment Plant

We have carried out several investigations within the Penrith Treatment Plant, located about 350m south of the subject site. Those boreholes encountered fill covering alluvial silty clay, generally assessed to be of stiff to very stiff strength. Most boreholes were terminated within the clays, but some boreholes were drilled deeper and refused within gravel at depths ranging from about 9m to 11m. Grounder was encountered at depths ranging from about 5m to 7m.

3.2 Site Description

The site is located in relatively level alluvial topography associated with the Nepean River floodplain. The site is relatively level, with the surface levels of the boreholes measured by the GPS only varying by about 1.5m.

The site is vacant and largely overgrown with tall grass, together with a number of small trees scattered across the site. At the western end of the site is a predominantly bare area comprising an overflow swale sloping towards stormwater pipes at the southern boundary. Water was observed ponding within a locally deeper excavation adjacent to the pipes and in other low points along the swale. Within the base of the swale, the surface soil appeared to be predominantly silty in nature and was soft underfoot, even where it was dry. Along the eastern edge of the swale vegetated mounds, presumably excavated material from within the swale, were observed to heights of about 1m to 3m.

The site is bound to the north, west and south by industrial lots containing large on-grade warehouses. The warehouses to the north and west are located about 10m to 15m from the common boundary, but to the south the buildings are located about 50m from the boundary. The warehouses are generally surrounded by pavements of predominantly concrete construction. To the east of the site is vacant land.

3.3 <u>Subsurface Conditions</u>

In summary, the boreholes drilled for this investigation encountered silty alluvial soils overlying alluvial gravels, which caused refusal of the auger. Further comments on the subsurface conditions are provided below. A graphical summary of the borehole information is provided as Figure 3. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered.

Topsoil

In BH1 and BH4 a distinct topsoil layer was encountered to depths of 0.2m and 0.5m, respectively. Within the remaining boreholes although a distinct topsoil layer was not encountered roots were encountered within the upper soils to depths ranging from 0.15m to 0.2m.

Silty Alluvial Soils

The alluvial soils predominantly comprised silt, with some clayey silt in BH4 and BH5, and sandy silt and silty sand with depth in BH3 and BH4. The silty soils were assessed to be of low plasticity and predominantly stiff to very stiff based on the SPT test results. We note that within the boreholes where hand penetrometer tests could not be carried out due to the friable nature of the silty soils, the SPT 'N' values were similar to those measured in BH4 where hand penetrometer tests could be completed.

Alluvial Gravels

Alluvial gravels were encountered at depths ranging from 2.6m to 4.5m and were contained within and sandy silt matrix, with a trace of cobbles. The auger was only able to penetrate the gravel for a short distance, with refusal at depths ranging from 3.0m to 4.9m. The gravels were assessed to be of dense relative density based on the high resistance to penetration of the TC bit attached to the auger and the limited SPT and solid Cone tests.

Groundwater

No groundwater was encountered during, on completion or up to 3 hours after completion of drilling.

3.4 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the clayey silt tested is of low plasticity and is assessed to have a slight potential for shrink/swell movements with changes in moisture content.

The four day soaked CBR tests on samples of the soil compacted to 98% of their Standard Maximum Dry Density (SMDD) gave CBR values of 14% and 8%.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues and Further Geotechnical Investigation

Since this geotechnical investigation was carried out for due diligence purposes only a wide spacing of boreholes was carried out. Those boreholes encountered alluvial silty soils overlying alluvial gravels, which caused refusal of the auger. Based on these results we consider that the main geotechnical issues for the proposed warehouse are as follows:

- The site is underlain by silty soils and any earthworks to adjust surface levels will involve working with the silty soils. Silts are very moisture sensitive and can be very difficult to place and compact efficiently. Silty soils will soften rapidly if they become wet making reuse virtually impossible. If the silty soils become wet they will soften and may require stripping and replacement at any stage during the earthworks. Therefore, if the material is to be reused allowance should be made for difficulties with reusing such silty materials and careful control of the moisture of the material will be required. The earthworks must be carried out by an earthworks contractor who is experienced in working with such materials.

- Following completion of bulk earthworks we recommend that a layer of good quality crushed rock should be placed over the completed earthworks platform to protect the silty soils from moisture change and softening during construction.
- An existing overland flow path is present on the western side of the site and if the proposed development extends over this area of the site the flow will need to be redirected to allow flow to continue into the pipes that are present at the southern boundary. We expect that a pipe line will need to be constructed below the site.
- Stockpiles are present adjacent to the existing swale on the western side of the site and these will need to be assessed and managed during earthworks.
- The most appropriate footing system for the warehouse will comprise shallow footings founded within the alluvial soils. Insufficient information on the deeper subsurface profile is available at this time to recommend the use of piles. If piles were to be considered further geotechnical investigations to determine the thickness of the gravel and what material underlies the gravel will be required to assess the feasibility of piling and appropriate bearing pressures for piles.
- The measured CBR values were high and as such, provided the earthworks are carried out adequately, a reasonably good subgrade for the proposed pavements and slabs should be present. We recommend that a reduced design CBR value be adopted due to the limited testing completed to date and the potential for variability within the soils.
- Development of the site must also be carried out in accordance with the recommendations provided by EIS within their preliminary environmental site assessment.

Further comments on these issues are provided within the following sections of this report. However, the comments and recommendations are of a preliminary nature due to the wide spacing of the boreholes and should only be used for planning and preliminary design. To allow final design, we recommend that a detailed geotechnical investigation of the site be carried out once the final development details are known. The geotechnical investigation should comprise the drilling of additional boreholes at a closer spacing, targeted where the proposed warehouse will be located. As part of the detailed geotechnical investigation additional laboratory testing should be carried out to characterise the nature of the alluvial soils, in particular the silt present. If the use of piles is to be investigated specialist drilling equipment will be required to penetrate the gravels as discussed below. The comments and recommendations provided herein must be reviewed and amplified as part of the detailed geotechnical investigation.

Overall, we consider that the site is suitable for the proposed warehouse development and will be similar to other warehouses constructed within adjoining properties.

4.2 Earthworks

Due to the relatively level nature of the site, we expect that earthworks for the proposed warehouse and pavements will be minor, possibly involving excavation on the south-eastern portion of the site and filling in the western portion. We expect that such earthworks would be to depths of no more than about 1m.

As mentioned above, the site is underlain by silty soils, which will be moisture sensitive. Careful control of soil moisture will be required, otherwise the soils will become unworkable. We recommend that only the services of an earthworks contractor who is experienced in working with such silty soils be considered. Careful control of drainage will also be required, together with sealing off of the filled surface at the end of each day to reduce infiltration if overnight rain occurs. Even with good moisture control during the earthworks there will be a risk that layers will become over-wet, soften and become unsuitable. This may require stripping of the softened material and replacement with imported fill. To reduce such risk consideration could be given to the importation of more suitable filling material for the entire earthworks for this project.

Following completion of the earthworks, we recommend that a layer of good quality crushed rock be placed over the subgrade to provide a good base for construction and to protect the silty soils from moisture infiltration during construction.

Excavation of the soil will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators or scrapers, depending on the extent of the excavations required.

Initial earthworks should comprise stripping of the vegetation and root affected soils. The boreholes did not encounter a distinct topsoil layer throughout the site, but the upper root affected zone should be removed, which in the boreholes was generally to depths of about 0.2m. This topsoil material would not be suitable to reuse as engineered fill on its own, but could be used within landscaped areas. As part of the detailed geotechnical investigation it may be possible to sample and test the organic content of the topsoil to assess if it can be blended with other materials for reuse as engineered fill. Further advice on this would need to be provided as part of the detailed geotechnical investigation if this is to be investigated.

The stockpiles located adjacent to the drainage swale towards the western end of the site should be stripped of vegetation and then inspected by a geotechnical engineer to assess the suitability of the material for reuse. If the material can be reused it should be fully excavated to expose the natural soils before placement of additional fill. Any water within the swale at the time of the earthworks should be removed and any the water softened material removed. It is unlikely that this soft material would be suitable for reuse.

Following stripping the exposed subgrade should be proof rolled with at least 7 passes of a minimum 8 tonne smooth drum roller. The final pass should be carried out in the presence of a geotechnical engineer to detect any weak or unstable subgrade areas. As detailed above, silty soils are moisture sensitive and heaving may occur if the moisture content of the subgrade is slightly outside of optimum. Moisture conditioning of the subgrade may be an effective method of subgrade improvement if heaving occurs, however, if unsuitable material is encountered, excavation and replacement with engineered fill may be necessary. The final subgrade improvement works should be determined by the geotechnical engineer during the proof rolling inspection. Allowance should be made for at least a moderate amount of subgrade improvement or replacement with engineered fil.

Following treatment of any unsuitable subgrade areas, engineered fill may be placed in thin layers in accordance with the recommendations provided in Section 4.3.

4.3 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated alluvial soils may be reused as engineered fill, provided they are free of deleterious materials. Any silt or clay fill should be compacted to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of Standard Optimum Moisture Content (SOMC). However, for the silts a narrower moisture limit may need to be adopted. Careful control of moisture content and compaction control will be required when reusing the silty soils from site.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. Where the fill is to support building loads it should be placed under Level 1 control, as defined by AS3798. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.4 Drainage

Due to the moisture sensitivity of the soils, careful control of drainage during earthworks and in the long term will be required. The principal aim of the drainage should be to promote run-off and reduce ponding. Any exposed surfaces should be sealed and graded at the end of each day of works to reduce infiltration if overnight rainfall occurs, with a fall on the surface to promote run-off. Placement of a blinding layer of durable granular fill or subbase material to provide a trafficable surface during construction may be necessary or desirable. The earthworks should be carefully planned and scheduled to maintain cross-falls during construction.

4.5 Batters and Retaining Walls

Since details of the proposed development have not been finalised at this time it is unknown if batters or retaining walls will be required, but the following general advice is provided on such batters and walls of no more than 3m in height away from adjoining properties. This advice should be reviewed once details of any proposed batters and retaining walls are known and specific advice provided if excavations are proposed close to the site boundaries.

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

Permanent retaining walls constructed at the toe of temporary batters, where some resulting ground movements behind the walls are acceptable, may be designed based on a triangular earth pressure distribution, using an active earth pressure coefficient, K_a, of 0.3 and bulk unit weight of $20kN/m^3$. Where walls are restrained from some movement by other structural elements in front of the wall, or where movements are to be reduced, an 'at rest' earth pressure coefficient, K_0, of 0.5 should be used.

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

The space between temporary batters and permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The excavated alluvial soils will be difficult to properly compact within the limited space available behind the walls and our recommendation is that more readily compactable materials, such as ripped or crushed rock, be used. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas a lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. An alternative and our preferred material for backfill would be to use a uniform granular material, such as crushed concrete of 30mm to 70mm in size, surrounded in a geofabric, with a clay or concrete cap to reduce infiltration.

4.6 Footings

The most suitable footing system for this site is the use of shallow footings founded within the alluvial soils or engineered fill. It is likely that pad or strip footings to support the external walls of the warehouse with independent floor slab between the walls would be used. However, if office buildings or similar are also proposed the use of stiffened raft slabs may also be appropriate.

Shallow footings founded within the alluvial soils of at least stiff strength or engineered fill may be designed based on an allowable bearing pressure of 100kPa. Such footings should be designed to accommodate the shrink/swell movements of the soils, which will depend on the reactivity and depth of any fill placed. We expect that the natural silty soils would undergo shrink/swell movements similar to a Class S site in accordance with AS2870-2011, but this would increase to Class M or possibly H1 where clay engineered fill is used. The final assessment of the likely shrink/swell movements should be determined once the development plans have been finalised and the material types for filling determined.

The footing excavations should be inspected by a geotechnical engineer to confirm that adequate foundation material has been encountered.

Based on the information obtained to date the suitability of piles cannot be determined since the boreholes refused shortly after encountering the gravels. The suitability of piles and the appropriate bearing pressures will depend on the material that underlies the gravel. If the gravel is underlain by weak soils then piles may need to extend through the gravel and the weak material and into any underlying better soils or bedrock, which may be deep. Alternatively, if the gravel is underlain by good quality material, such as weathered rock, then piles founded within the gravel may be feasible. The allowable end bearing pressure for the design of piles may range from a few hundred kPa if the gravels are underlain by soils, to say 1000kPa or more if the gravel is underlain by good quality bedrock. To determine the suitability of piles and the appropriate design parameters an additional geotechnical investigation would be required to prove the thickness of the gravel and the material below. Percussive drilling equipment would be required to break through the gravels and if the gravel is underlain by rock, coring of the rock would be required to determine the rock quality. Such investigation methods are time consuming and costly and should be carefully considered before proceeding. Similarly, if the piles need to extend through the gravels percussive drilling equipment would be required resulting in a high cost for the piling. We would expect that groundwater would be present within the gravels and this would add to the difficulties of drilling piles through the gravels.

4.7 Pavements and Floor Slabs

The pavement and floor slab subgrade should be prepared as recommended above.

The two CBR tests carried out for this investigation measured CBR values of 14% and 8%. Given the size of the site we recommend that additional CBR tests be carried out once the location and level of the pavements are known. If the pavements are constructed on engineered fill, such testing may be best carried out following completion of the earthworks so the actual subgrade soils can be tested.

Based on the testing carried out to date, we recommend that provisional design of the pavement thickness be based on a soaked CBR of 5%, or an estimated modulus of subgrade reaction of 30kPa/mm (750mm plate). Where fill, such as a granular layer to protect the subgrade is used, the CBR and thickness of the granular layer may be taken into account as part of the pavement design.

Surface and subsoil drainage should be provided on the high side of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 300mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain. In addition, the surface of the adjacent pavement subgrade should be provided with a uniform cross fall towards the subsoil drain to assist with drainage.

Concrete pavements should have a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2014) unbound base material (or similar good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the detailed design and construction phases of the project. For example, a detailed geotechnical investigation of the site should be carried out once the final details of the proposed development have been determined. In the event that any of the detailed design or 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.

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 Warehouse
Location: 128 Andrews Road, Penrith, NSW

Ref No: 31675B
Report: A
Report Date: 9/08/2018

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 %
4	0.50-0.95	14.5	22	13	9	4.0
5	0.50-0.95	6.5	24	12	12	5.0

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: 20/7/18

NATA
NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.

Authorised Signature / Date
(A. Tatikonda) 9/8/18

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31675B
Project:	Proposed Warehouse	**Report:**	B
Location:	128 Andrews Road, Penrith, NSW	**Report Date:**	9/08/2018
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BOREHOLE NUMBER	1	4
DEPTH (m)	0.40 - 1.00	0.40 - 1.00
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.96 STD	1.86 STD
Optimum Moisture Content (%)	9.1	12.7
Moulded Dry Density (t/m^3)	1.92	1.83
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	104	98
Moisture Contents		
Insitu (%)	6.1	14.7
Moulded (%)	9.4	12.4
After soaking and		
After Test, Top 30mm(%)	12.1	17.3
Remaining Depth (%)	10.3	13.9
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	0.0	0.0
C.B.R. value: @2.5mm penetration		8
@5.0mm penetration	14	

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample:20/7/18

NATA

NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.

Authorised Signature / Date
(A. Tatikonda) 9/8/18

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

1/1

Client: CADENCE PROPERTY GROUP PTY LTD
Project: PROPOSED WAREHOUSE
Location: 128 ANDREWS ROAD, PENRITH, NSW

Job No. 31675B **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 25.1m
Date: 26/7/18 **Datum:** AHD
Logged/Checked by: A.B./D.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 AND AFTER 1.5 HRS	ES		0			TOPSOIL: Silt, low plasticity, brown, trace of roots.	w<PL			GRASS COVER
	DB, DS	N = 8 9,5,3			ML	SILT: low plasticity, orange brown, trace of fine grained sand, and clay.	w<PL	(St-VSt)		ALLUVIAL TOO FRIABLE FOR HP TESTING
	DS	N = 7 3,3,4	1			as above, but with fine grained sand, and clay.				TOO FRIABLE FOR HP TESTING
	DS	N = 10 4,4,6	3			SILT: low plasticity, light brown, with fine grained sand, and clay.				TOO FRIABLE FOR HP TESTING
					GM	Sandy silty GRAVEL: medium to coarse grained, dark grey and grey sub rounded and sub angular gravel in a sandy silt, low plasticity, orange brown, fine grained sand matrix, trace of sub rounded and sub angular, dark grey cobbles.	D	D		HIGH 'TC' BIT RESISTANCE 'TC' BIT REFUSAL
			4			END OF BOREHOLE AT 3.7m				
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: CADENCE PROPERTY GROUP PTY LTD
Project: PROPOSED WAREHOUSE
Location: 128 ANDREWS ROAD, PENRITH, NSW

Job No. 31675B
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 25.6m
Date: 26/7/18
Datum: AHD
Logged/Checked by: A.B./D.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 AND AFTER 2 HRS	ES		0		ML	SILT: low plasticity, brown, trace of roots.	w<PL	(St-VSt)		GRASS COVER
						as above, but without roots.				ALLUVIAL
	DS	N = 16 16,9,7				SILT: low plasticity, orange brown, with fine grained sand, and clay.				TOO FRIABLE FOR HP TESTING
			1							
	DS	N = 9 5,5,4								TOO FRIABLE FOR HP TESTING
			2		SM	Sandy SILT: low plasticity, orange brown, fine grained sand, with clay.				
		Nc= 7/20 REF	3		GM	Sandy silty GRAVEL: medium to coarse grained, dark grey and grey, sub rounded and sub angular gravel in a sandy silt, low plasticity, orange brown matrix, with sub rounded and sub angular cobbles.	D	D		HIGH 'TC' BIT RESISTANCE 'TC' BIT REFUSAL
						END OF BOREHOLE AT 3.1m				
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: CADENCE PROPERTY GROUP PTY LTD
Project: PROPOSED WAREHOUSE
Location: 128 ANDREWS ROAD, PENRITH, NSW

Job No. 31675B
Date: 26/7/18
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 24.6m
Datum: AHD
Logged/Checked by: A.B./D.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	ES		0		ML	SILT: low plasticity, brown, trace of roots.	w<PL	(St-VSt)		GRASS COVER
						as above, but without roots.				ALLUVIAL
	DS	N = 15 8,8,7				SILT: low plasticity, orange brown, trace of fine to coarse grained sand, and clay.				TOO FRIABLE FOR HP TESTING
			1							
	DS	N = 6 2,3,3				as above, but with fine grained sand, and clay.				TOO FRIABLE FOR HP TESTING
			2							
	DS	N = 9 6,4,5	3		SM	Sandy SILT: low plasticity, light brown, fine to medium grained sand, with clay.				
					GM	Sandy silty GRAVEL: medium to coarse grained, dark grey and grey, sub rounded and sub angular gravel in a sandy silt, low plasticity, light brown matrix, fine to medium grained sand.	M	D		HIGH 'TC' BIT RESISTANCE
			4			END OF BOREHOLE AT 3.75m				'TC' BIT REFUSAL
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: CADENCE PROPERTY GROUP PTY LTD
Project: PROPOSED WAREHOUSE
Location: 128 ANDREWS ROAD, PENRITH, NSW

Job No. 31675B
Date: 26/7/18
Method: SPIRAL AUGER JK300
Logged/Checked by: A.B./D.B.
R.L. Surface: ≈ 24.1m
Datum: AHD

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 AND AFTER 3 HRS	ES		0		-	TOPSOIL: Silty clay, low plasticity, dark brown, trace of roots, and ash.	w<PL			GRASS COVER
	DB					as above, but without roots.				
	DS	N = 5 3,2,3			ML	Clayey SILT: low plasticity, light brown.	w>PL	St	150 130 110	ALLUVIAL
			1							
						as above, but light brown mottled orange brown, trace of fine to medium grained ironstone gravel, and fine grained sand.		VSt		
	DS	N = 14 6,6,8							230 250 310	
			2							
			3					St-VSt		
	DS	N = 15 5,6,9							170 190 220	
			4		SM	Silty SAND: fine to medium grained, light grey and brown.	M	MD		
	DS	N > 11 11,11/ 60mm REFUSAL			GM	Sandy silty GRAVEL: course grained, dark grey and grey sub rounded and sub angular gravel, in a sandy silt, low plasticity, orange brown, fine to medium grained sand matrix with grey sub rounded and sub angular cobbles.	M	D		HIGH 'TC' BIT RESISTANCE
			5			END OF BOREHOLE AT 4.9m				'TC' BIT REFUSAL
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

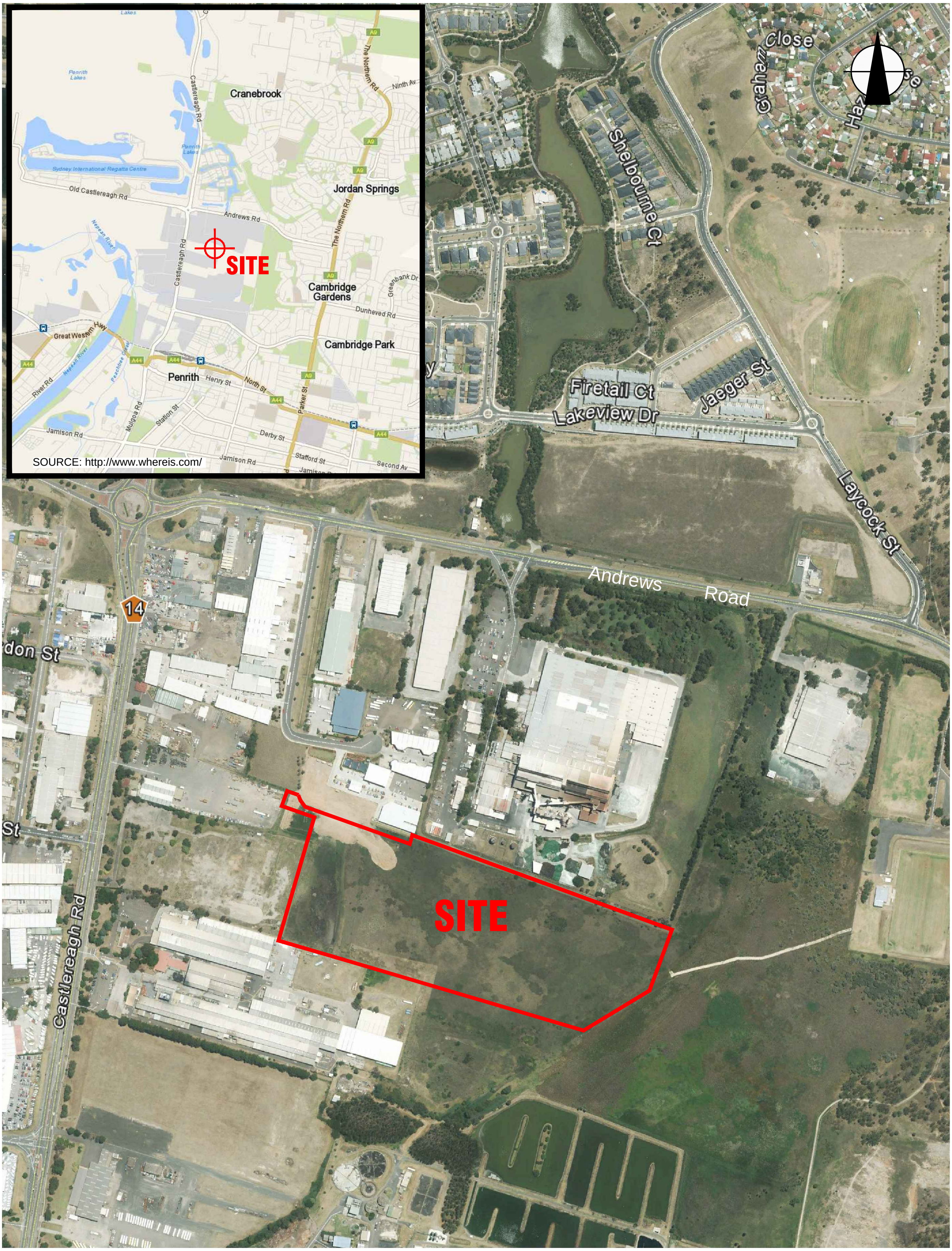
Borehole No.
5
1/1

Client: CADENCE PROPERTY GROUP PTY LTD
Project: PROPOSED WAREHOUSE
Location: 128 ANDREWS ROAD, PENRITH, NSW

Job No. 31675B
Date: 26/7/18
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 24.8m
Datum: AHD
Logged/Checked by: A.B./D.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		ML	SILT: low plasticity, brown, trace of roots.	w<PL	(St-VSt)		GRASS COVER
						Clayey SILT: low plasticity, orange brown.				ALLUVIAL
	DS	N = 16 9,10,6								TOO FRIABLE FOR HP TESTING
			1			SILT: low plasticity, orange brown, with clay, trace of fine grained sand, and clay.				
	DS	N = 7 3,3,4								TOO FRIABLE FOR HP TESTING
			2							
					GM	Sandy silty GRAVEL: medium to coarse grained, dark grey and grey sub rounded and sub angular gravel in a sandy silt, low plasticity, orange brown, fine to medium grained sand matrix, trace of dark grey, sub rounded and sub angular cobbles.	D	D		HIGH 'TC' BIT RESISTANCE
			3			END OF BOREHOLE AT 3.0m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

Document Set ID: 8449091
Version: 1, Version Date: 05/11/2018



AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
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This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	128 ANDREWS ROAD PENRITH, NSW
Report No: 31675B	Figure No: 1
	JK **Geotechnics**

PLOT DATE: 10/08/2018 11:34:18 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31675B PENRITH\CAD\31675B.DWG

AERIAL IMAGE SOURCE: NEARMAP.COM

0 20 40 60 80 100

SCALE 1:2000 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: 128 ANDREWS ROAD PENRITH, NSW

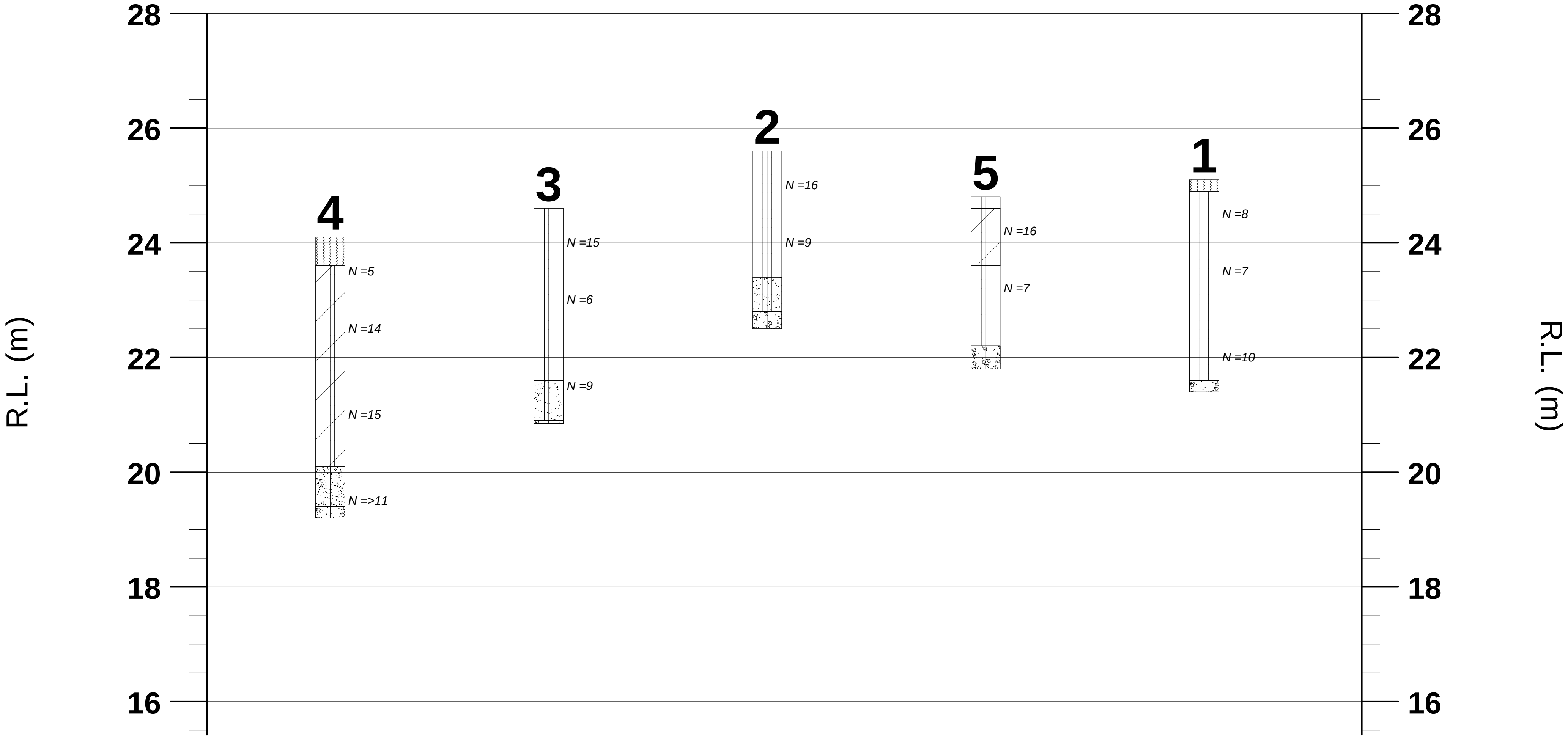
Report No: 31675B

Figure No: 2

J K **Geotechnics**

PLOT DATE: 10/08/2018 11:34:07 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31675B PENRITH\CAD\31675B.DWG

GRAPHICAL BOREHOLE SUMMARY



Topsoil
Clayey Silt
Silty Sand
Silty Clayey Gravel
Silt
Sandy silt

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 31675B

Figure No.: 3

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

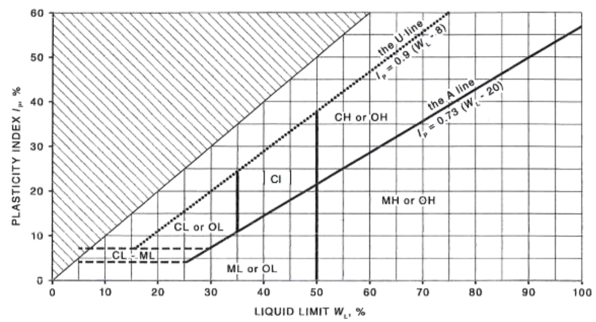
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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