REPORT

TO

ELDESO GROUP PTY LTD

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

215-235 O'RIORDAN STREET AND EWAN PLACE, NSW

13 December 2018

Ref: 31721VJrpt

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Date: 13 December 2018
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Revision No: 0

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<u>**TABLE OF CONTENTS**</u>

1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed mixed use development at 215-235 O'Riordan Street and 1-3 Ewan Place, Mascot, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Chris Mavrocoratos of Eldeso Group Pty Ltd and was carried out in accordance with our proposal dated 18 October 2018 (Ref: P47251VJRev1-Mascot). The environmental component of the works was carried out in conjunction with the geotechnical investigation; however, the results will be presented in a separate report by our environmental division, Environmental Investigation Services (EIS).

We have been provided with the following information:

- Architectural drawings by Mode Design Corp. Pty Ltd (Ref: Project No. 17689, Drawing Nos. AR-0000, AR-1001 to AR-1015, AR-1017, AR-1201 to AR-1203, AR-1301 to AR-1304, AR1501, all Revision D and dated 11 December 2018);
- Elevation and plan tunnel drawings by Hard & Foster Consulting Surveyors (Ref: Drawing Nos. 1098590009, Rev 00, dated 5 November 2001 and 83370103, drawing titled 'Tunnel Notification Zone Botany Council', Sheet 3 of 4, dated November 2001); and,
- Survey drawing by Eric Scerri & Associates Pty Ltd Land Survey Consultants (Eric Scerri & Associates, Plan Ref: 2573/18, dated 18 April 2018).

Based on these drawings, we understand that the proposed works will include the following:

- Demolition of the existing warehouse buildings, pavements and other structures on site.
- Construction of commercial/hotel building with up to 12 levels over 3 basement levels. From the basement level 3 floor plan provided, the proposed basement car park extends to the common boundary along the west side and is setback between 3m and 9m on the south, east and north sides. The basement level 3 level is shown as RL-2.5m. We anticipate about a 9.5m deep excavation to be required. At this stage structural drawings and design loads are not available. We expect column loads to be in the high range for this type of development.
- The site is near the first reserve and over the second reserve of the Sydney Airport Link (APL) tunnel and special considerations must be made during the design and construction stage to satisfy the Transport for NSW (TfNSW) requirements, as set out in the following:
    - TfNSW document titled '*Development Near Rail Tunnels*' (Ref: T HR CI 12051 ST, Version 1.0, dated 14 November 2016);
    - TfNSW Technical Note – TN 026: 2017 (Ref: Subject: Amendments to T HR CI 12051 ST '*Development Near Rail Tunnels*', Version 1.0); and,

- *Airport Line Tunnel Protection Guidelines – Guidelines for Development Within the Vicinity of the Airport Line* (Ref: Part A: Planning and Design Matters, Part B: Technical Matters, Rail Access Corporation, Rev A, August 2000).

The purpose of the preliminary investigation was to obtain geotechnical information on subsurface conditions at the test locations as a basis for comments and recommendations on excavation, hydrology, retention, footings and basement floor slab. Investigation locations were limited due to access restrictions, primarily due to existing buildings and structures on site. Therefore, further investigations are warranted, including analyses to address geotechnical issues of the impacts on the APL tunnel and groundwater modelling to estimated water extraction volumes during construction.

2 INVESTIGATION PROCEDURE

Our preliminary geotechnical investigation was carried out over a number of days between 22 October and 13 November 2018 and in summary comprised the following:

- Carry out a 'Dial Before You Dig' (DBYD) search and the borehole locations were electromagnetically scanned by a specialist sub-contractor for buried services (carried out on 22 November 2018);
- Three (3) cored boreholes, BH1 to BH3, to depths between 30.5m and 30.9m (carried out on 24, 25, 26, 29 and 30 November 2018);
- Installation of standpipes in each of the boreholes upon completion and re-visiting the site to measure groundwater levels (13 November 2018); and,
- Four (4) Cone Penetration Tests (CPTs), CPT4 to CPT7, to depths between 20.05m and 25.0m, including two (2) piezocone tests (CPT4 and CPT5) (carried out on 22 October and 2 November 2018).

The test location plan is included as Figure 2, which used the Eric Scerri & Associates survey drawing as a base plan for the borehole and CPT locations. The locations were set out by taped measurements from surface features. The approximate surface levels, as shown on the borehole logs and CPT result sheets, were estimated by interpolation between spot levels shown on the survey drawing and are, therefore, only approximate. The datum for the levels is Australian Height Datum (AHD), as noted in the survey drawing.

The boreholes were initially drilled by spiral auger, washboring and casing advancer techniques with our truck mounted JK500 rig to depths ranging from between 21.2m to 22.2m, then extended to termination depths by rock coring techniques and an NMLC triple tube core barrel. In the boreholes, the degree of compaction of the fill and relative density/material strength of the subsurface soils were assessed from the Standard Penetration Test (SPT) 'N' values undertaken. We do note that 'N' values may reflect higher than normal values when testing below the water table in sands as the SPT split spoon sampler may fill up with the sands prior to testing. This was likely to be the case in SPTs at depths of 9m, 10.5 and 15m in BH3, and may be the case in other tests.

Further assessment of relative density/material strength of the subsurface soils was carried out using Cone Penetration Testing (CPT). The CPT method involves continuously pushing a 35mm diameter rod with a conical tip into the subsurface materials using hydraulic rams fitted to our truck mounted rig. Measurements of the end resistance on the cone tip and the frictional resistance on a separate sleeve, immediately behind the cone, are taken. A 'dummy cone' was used in CPT4 and CPT5 initially, to protect the test cone, in the fill materials. The subsurface material identification, including the inferred fill layer and the material strength/relative density, is by interpretation of the test results based on past experience, empirical correlations and correlations with geotechnical information from the geotechnical boreholes carried out nearby. We note that the CPTs do not provide sample recovery. Piezocones were used in CPT4 and CPT5 to measure pore pressures within the soils to provide an indication of the likely groundwater level at the time of testing.

The strength of the cored rock from the boreholes was assessed with reference to Point Load Strength Index ($I_{s(50)}$) test results carried out on the recovered rock core. The results of the point load strength index tests are summarised in the attached STS Table A and on the cored borehole logs. Using established correlations the approximate unconfined compressive strength of the bedrock was interpreted from the Point Load test results.

Groundwater observations were made in the boreholes during auger drilling. Some additional measurements were made during the investigation as shown on the borehole logs. Standpipes were installed in the three boreholes at depths of 13.2m (BH1), 14.2m (BH2) and 11.5m (BH3) to allow for longer term groundwater monitoring and infiltration testing (not included in this scope of preliminary works). Subsequent groundwater readings in the three standpipes were carried out on 13 November 2018. A summary of groundwater levels measured is provided below in Section 3.2.

The investigation was carried out under the supervision of our Senior Geotechnical Engineer, Mr Jarett Mones. The fieldwork was completed in the full time presence of our geotechnical engineer, Mr Bryan Zheng, who set out the boreholes, nominated sampling and testing locations and prepared logs of the strata encountered. The borehole (BH) logs and CPT results sheets, which include field test results and groundwater observations/inferred levels, are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques, and their limitations, and define the logging terms and symbols used. Figure 3 includes a cross section of the site with a graphical investigation summary of the BHs and CPTs.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site location is shown in Figure 1.

The site is located in a relatively flat area near to the Sydney Airport, north of Botany Bay.

The site is currently developed over a number of lots, mainly with one to three storey warehouses surrounded by parking areas with asphaltic concrete (AC) and concrete pavements. There are no basement levels that we are aware of. The site is generally flat with a level variation of about 1m.

In summary, the subject site comprises an agglomeration of the following lots with existing development:

- 215 O'Riordan Street contained a two-storey brick building with a flat metal roof and a concrete paved car parking area. The building appeared to be in good condition when viewed externally. The pavements appeared to be in good condition.
- 217 O'Riordan Street was used as a vehicle parking area and was surfaced with AC pavement. The pavement appeared to be in poor to moderate condition with several cracks. There were also three shade cloths on metal poles covering most of the parking area.
- 219 O'Riordan Street was occupied by a two-storey brick & metal warehouse with a metal roof. A concrete floor slab was observed inside the warehouse. The building appeared to be in moderate to good condition when viewed externally.
- 221-233 O'Riordan Street contained a car park in the eastern half and a two to three-storey brick warehouse with a metal roof on its western half. The car park was surfaced with a reinforced concrete pavement. The southern to central portion of pavements appeared to be in fair to moderate condition, with cracking and spalling. The pavements in the northern portion (i.e. 'Hertz Car Rental') appeared to be in good condition. A concrete floor slab was observed inside the warehouse. The warehouse building appeared to be in moderate to good condition when viewed externally and internally, with some cracking within the concrete floor slab. Along the eastern boundary there was a low height (about 0.5m) brick

retaining wall supporting the site. The brick wall is tilting outwards (i.e. towards the east) at around 2° to 5°.

- 235 O'Riordan Street is occupied by a two-storey brick warehouse with a metal roof. A concrete floor slab was observed inside the warehouse. The warehouse appeared to be in moderate condition when viewed externally.
- 1-3 Ewan Street was currently used as a vehicle parking area and was surfaced with AC pavement. The pavement was assessed to be in moderate condition. There were also three shade clothes on metal poles. There were timber soldier pile walls retaining the site on the western and northern sides near the boundary. These walls were setback, from the property boundary, by a distance of about 0.4m. The walls ranged in heights from between 0.3m and 1m.

The site is bound by Ewan Street to the south, O'Riordan Street and the APL tunnels to the east, King Street to the north and a multi-level parking garage to the west. Below is a more detailed description of the site surrounds.

The multi-level parking garage (to the west) has a lower ground level, below the subject site level. We were unable to access the garage to review if there were additional basement levels. Set-in approximately 24m from the southern boundary, the parking garage extends to the site boundary for about a 17m length (to north). The concrete ramp partially abuts the western site boundary towards the northern end. Otherwise the building appears to be setback by a distance of about 3m to 15m to the boundary. However, basement or lower ground levels of the basement may extend closer to the boundary than that observed from the subject site; therefore the as-built basement layout/drawings should be reviewed during the design stage.

Ewan Street (to the south) is a two-lane street surfaced with AC pavement. Ewan Street falls slightly towards the west at about 1° to 2° and the pavement was assessed to be in moderate condition.

O'Riordan Street (to the east) is a four-lane street surfaced with AC pavement. The pavement was assessed to be in moderate condition

King Street (to the north) is a two-lane street surfaced with AC pavement. King Street falls slightly towards the west at about 1° to 2° and the pavement was assessed to be in moderate condition.

3.2 Geology and Subsurface Conditions

The 1:100,000 geological series sheet for Sydney shows the site to be underlain by a Quarternary aged deposits of fine to medium grained 'marine' sand with podsols.

The boreholes (BHs) and CPTs have disclosed subsurface conditions to generally comprise pavements and granular fill covering natural silty sand with thin silty clay/clayey silt layers (marine soil deposits) with a transition to residual silty clay, grading into siltstone bedrock at depth. Some of the more pertinent features of the strata encountered are described below. For further details of the conditions encountered at each location, reference should be made to the attached borehole logs and CPT results sheets.

Pavements

The boreholes (BH1 to BH3) and CPT6, carried along the eastern end of the site disclosed reinforced concrete pavement between about 100mm to 150mm thick. The warehouse floor slabs were observed to be concrete and are likely to have thicknesses exceeding 100mm. The AC pavements were estimated to be about 50mm thick from the site inspection and results of the CPTs.

Fill

Fill encountered in BH1 to BH3 typically comprised silty sand and gravelly sand to depths of 1.5m (BH1), 1.0m (BH2) and 0.3m (BH3). It was noted that in BH3 it is possible that fill might be deeper, possibly about 1.0m. As the CPTs do not allow for sample extraction inferred depths of fill were made as 0.4m (CPT4), 1.4m (CPT5), 1.8m (CPT6) and 1.5m (CPT7). Inferred fill materials/depths around the CPT locations and in the warehouse areas should be confirmed by further geotechnical investigations. The fill appeared to be poorly to moderately compacted.

Marine Soils

Marine soils, generally described as silty sand, were found to directly underlay the fill. The sands were overall assessed as medium dense to dense; however, there were some dense to very dense and very dense layers. Some very stiff to hard silty clay to clayey silt bands were observed (in the boreholes) and assessed (in the CPTs) within this unit. We also observed thinner organic clay layers that had traces of timber fragments. It should be noted that it is not unusual to encounter organic and peaty layers within the Mascot area. The marine soils extended to depths assessed between approximately 16.2m (RL-9.2m) and 19.0m (RL-12.3m).

Residual Soils

Residual soils, generally as silty clay were assessed to underlay the marine soils. In general, the residual soils were assessed as high plasticity and firm to stiff initially, increasing in strength with depth to very stiff, very stiff to hard or hard.

Siltstone Bedrock

The boreholes, BH1 to BH3, drilled on the eastern end of the site were carried out to prove bedrock and extend below the existing APL tunnel invert level, whereas CPT4 was drilled to refusal. Refusal in the CPT may indicate top of very low strength or better bedrock; however, refusal could also occur on harder bands in the clay. In this case we inferred the top of very low strength bedrock at the refusal depth (in CPT4). However, further geotechnical investigations should be carried out to confirm this.

With respect to the borehole drilling, the siltstone bedrock, assessed as extremely weathered and with strength similar to a very stiff to hard clay, was encountered at depths of 22.5m (RL-16m) at BH1, 22.9m (RL-16.2m) at BH2 and 22.06m (RL-15.36m) at BH3. Inferred depths in the CPT holes for the extremely weathered siltstone unit were assessed as 21.4m (RL-14.4m) at CPT4 and 20.45m (RL-13.65m) at CPT7. However, the assessments of top of extremely weathered siltstone were based on cone tip resistance and should be verified by carrying out additional cored boreholes.

The siltstone improved to very low strength at depths of 23.7m (RL-17.2m) at BH1, 23.2m (RL-16.5m) at BH2 and 23.8m (RL-17.1m) at BH3. CPT4 refusal occurred at a depth of about 24.3m (RL-17.3m), which we infer to be about the depth of the very low strength bedrock (and as discussed above requires additional boreholes to confirm). The RLs for the assessed top of very low strength bedrock are quite uniform between these holes.

Each of the boreholes generally improved with depth, with strengths of low to medium, then medium to high strength.

In the upper portion of rock assessed as very low to medium strength, there were numerous clay seams and bedding partings. The clay seams were typically less than 10mm thick; however thicker bands up to 10mm to 40mm were not uncommon. Some steeply inclined joints (greater than 45°) were observed.

An indicative engineering classification of the bedrock (in accordance with Pells et al., 1998) has been carried out and is tabulated below:

Borehole	Indicative Engineering Classification of Top of 'Shale' Bedrock Class Depths (m) and Approximate RLs (m)							
	Class V		Class IV		Class III		Class II	
	Depth	RL	Depth	RL	Depth	RL	Depth	RL
BH1	23.7m	-17.2m	25.0m	-18.5m	25.0m	-18.5m	28.2m	-21.7m
BH2	23.2m	-16.5m	24.2m	-17.5m	25.5m	-18.8m	29.2m	-22.5m
BH3	23.8m	-17.1	25.4m	-18.7m	26.5m	-19.8m	28.0m	-21.3m
CPT4	24.3	-17.3	-	-	-	-	-	-

Notes on above table: CPT4 inferred top of Class V and requires confirmation by carrying out additional boreholes.

Groundwater

Groundwater seepage was encountered in each of the boreholes during drilling, at a depth of 3m at BH1 and BH3, and a depth of 3.5m at BH2. We revisited the site on 13 November 2018 (about 2 to 3 weeks from completion of drilling) to measure the groundwater in each of the standpipes installed in the boreholes. Below is a summary of the depths and RLs for these measurements in the standpipes and the groundwater levels assessed at the CPT locations.

Test Location (Surface ~RLm)	Groundwater Depth in m (~RLm)
BH1 (6.5m)	2.9 (RL3.6m)
BH2 (6.7m)	1.9 (RL4.8m)
BH3 (6.7m)	3.1 (RL3.6m)
CPT4 (7.0m)	3.45m (RL3.55m)
CPT7 (RL6.8m)	2.7m (RL4.1m)

Notes on above table: Borehole groundwater depths from measurements on 13 November 2018. CPT groundwater depths assessed from CPT data.

3.3 Laboratory Test Results

The point load strength index test results correlate well with our field rock strength assessment on site. $I_{s\ (50)}$ values ranged generally from 0.04MPa to 3.0MPa. We estimated the Unconfined Compressive Strength (UCS) based on the relationship, UCS = 20 x $I_{s\ (50)}$. The UCS values for Class IV rock ranged from 2MPa to 12MPa, for Class III rock ranged from 4MPa to 38MPa and for Class II rock ranged from 10MPa to 60MPa. We do not consider the test at BH3, depth range of 26.18m to 26.21m, to be representative of the rock mass (i.e. $I_{s\ (50)}$ value of 0.01MPa). Results are included as STS Table A (3 Pages) and are shown on our borehole logs. Table A also shows the approximate Unconfined Compressive Strengths of the rock core as interpreted from published correlations.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Design Issues

Based on the results of this investigation, the following principal geotechnical issues have been identified:

1. The proposed development may require excavations to depths on the order of up to about 9.5m for the construction of 3 basement levels. The APL tunnel is nearby to the site (under O'Riordan Street) and the site is within its zone of influence, i.e. second reserve for the tunnel. Good engineering design, construction and maintenance practices should be adopted to maintain stability during excavation and in the long term. TfNSW will require additional analyses be carried out to assess the potential impact of the excavation, shoring and foundation loads on the APL tunnel. Ground anchors (if used in the design) within the second reserve will need to be assessed for their effects on the underground infrastructure. Finite element numerical analyses will be required to satisfy TfNSW that the proposed development will not adversely impact on their infrastructure.
2. The proposed basement levels will be below the water table and therefore, the building will need to be tanked, i.e. fully waterproofed and designed against water pressures. Significant volumes of groundwater will require removal for the temporary construction works. Groundwater infiltration testing and modelling will be required to estimate extraction volumes and to review groundwater drawdowns.
3. Based on the APL tunnel and requirements from TfNSW, it is recommended that proposed building loads be transferred to the siltstone bedrock below the invert of the tunnel. This will require significantly deep pile footings to bedrock. Based on sandy soils and a high groundwater table, CFA piles are recommended to support building loads. Based on the shallow groundwater table and sands at bulk excavation level, piling platforms would likely be required to satisfy short term stability of the large piling rigs that would be required. As high building loads are anticipated, it likely that the design would include piles into Class III siltstone (or better). Additional deep cored boreholes will be required to assess the depth (and RLs) for this class of material.
4. Prior to commencement of demolition and construction, and considering the deep excavation and temporary dewatering during construction, which may drawdown the groundwater table, we recommend that dilapidation survey reports be carried out on the neighbouring buildings/structures within the zone of influence of the excavation and APL tunnel to the east.
5. Temporary shoring will be required to support the basement excavation around the site. Based on the site geometry and proposed development, temporary batters will not suffice for the bulk

excavation. A number of retaining wall options are discussed below, including advantages/disadvantages of each option.

6. The excavated fill, natural soils and groundwater will need to be disposed of appropriately. Reference shall be made to the EIS environmental investigation report. A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report. The recommendations provided in this report should be reviewed once details of the proposed development, such as floor levels and bulk excavation levels and structural loads, are revised or determined and also, following any additional geotechnical work. It is likely that further advice/input will be required during the structural and civil engineering design to address issues that may not have been addressed in this report. To some degree, this is an "iterative" process between evaluation of the geotechnical site conditions and the structural and civil designs.

4.2 Tunnel Infrastructure

The APL was completed in 2000 and comprised a 10km underground railway from Central Station to East Hills rail line near Turrella. The approximate primary (first) reserve and second reserve of the APL tunnel are shown on Figure 1. The second reserve extends into the eastern portion of the site, whereas the first reserve is east of the site, below O'Riordan Street. The TfNSW guidelines define the zone of influence for the tunnel as all land within 25m of its centre line.

Based on the Airport Line Tunnel Protection Limits drawing, the APL tunnel has been designed as a 'Soft Ground Tunnel'. This tunnel has been constructed as a circular bored tunnel using Tunnel Boring Machines supported by precast segments. Each circular ring comprises eight 450mm thick segments. The external diameter of the tunnel is about 10.4m.

The upper limits (i.e. crown) of the tunnel adjacent to the site were approximated to range between depths of 16.0m (to RL-9.2m, to north) and 17.3m (to RL-10.6m, to the south). The lower limits (i.e. invert) of the tunnel were approximated to range between depths of 26.8m (to RL-20m, to the north) and 27.7m (to RL-21m, to the south). Based on the above and an estimated bulk excavation depth of 9.5m (to RL-2.8m), we provide the following comments:

- Once bulk excavation for the proposed basement is reached/floor slab constructed, the tunnel will be between roughly 6.4m and 7.8m below the new building.

- The upper portion of the tunnel is likely to be within sand and clay; whereas the base is likely to be founded within the siltstone bedrock.
- According to the TfNSW Technical Note – TN 026:2017, Table 7 – Load Limits on APL Tunnels the following is stated:
    - For 'Soft Ground Tunnel' elements, applied pressures on the tunnel lining exceeding 30kPa require foundation loads to be transferred beyond the tunnel and load limits and associated ground movements should not compromise the function of waterproof gaskets in the tunnel lining.
    - An assessment of the ground anchor effects on the underground infrastructure are to be carried out.
    - Excavations adjacent to the APL tunnel are not to adversely impact the track alignment/rail operation. The tolerance of track alignment is distortion less than 3mm in the vertical and horizontal direction.

To abide by the TfNSW requirements, we recommend that 2D finite element analysis, using a program such as PLAXIS 2D, be carried out to review the impact the proposed development, including the construction stages, would have, if any, on the existing rail infrastructure. To do so, as-built drawings of the tunnel and the design (shoring, footings, structural loads, final floor levels, etc.) would be required for review.

The guidelines from TfNSW also specify additional requirements for developments. The guidelines suggest that potential developments could suffer ground borne noise and vibration, electromagnetic radiation and stray electrical currents from the operation of APL, and that the development design must take these factors into account. Potential developers must review the guidelines to assess the effect of these requirements on their development. A meeting of the design team, once the design has been further advanced, would be of benefit to discuss the geotechnical issues in more detailed and determine the scope of the further geotechnical work. The following recommendations have incorporated the geotechnical aspects of the requirements.

4.3 Dilapidation Survey

Prior to the commencement of excavation, dilapidation reports should be prepared for structures within a distance of 2H of the proposed excavation (where H is the height of the excavation). A dilapidation survey is required to be carried out in the tunnel adjacent to the proposed development, below O'Riordan Street, prior to the commencement of works and post construction. The dilapidation report should note the location, length, and width of each crack in those structures. A copy of the report should be signed by the owners/TfNSW to confirm it presents a fair record of the existing condition of these buildings/structures and tunnel. The reports could then be used as a benchmark for assessing potential claims for damage caused by the excavation. It is in the interest of both the owner/TfNSW and contractors that this report is compiled with a high degree of diligence rather than being based upon a cursory inspection, as the report could be essential in defending claims against damage.

4.4 Excavation

Excavation recommendations provided below should be complemented by reference to the Code of Practice 'Excavation Work', Code of Practice prepared by Safe Work Australia July 2015 and to the TfNSW guidelines for excavations near rail tunnels.

4.4.1 Excavation Conditions

The basement 3 finish floor level is shown as RL-2.5m. Based on an average existing ground level at RL6.7m we have estimated a depth of excavation of about 9.5m. Based on the boreholes and CPTs, we expect that the excavation will be through soils, most of which are below groundwater levels, comprising the granular fill, natural sandy soils of medium dense to very dense relative density and some very stiff to hard clay bands. We would consider that conventional hydraulic excavators will be suitable for excavation through these soils.

The excavated fill and natural soils (and groundwater) will need to be disposed of appropriately. Reference shall be made to the EIS environmental investigation report. A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal.

4.5 Hydrogeology

Currently no longer term groundwater monitoring has been carried out other than during the drilling/CPT stage and short period thereafter (2 to 3 weeks later). Groundwater seepage was encountered in each of the boreholes during drilling at depths of 3m at BH1 and BH3 and 3.5m at BH2 and inferred at depths between 2.7m and 3.45m in CPT7 and CPT4, respectively. Groundwater measurements after the 2 to 3 week period were measured as 2.9m in BH1, 1.9m in BH2 and 3.1m in BH3.

Based on our groundwater monitoring and noted depths during drilling we anticipate that the groundwater table will likely be intersected at a depth of about 3m; however, groundwater seepage may be encountered between depths of about 1.5m and 3.5m. Water levels will be about 6m to

8m above basement level 3. Further groundwater monitoring is recommended to allow appropriate lateral water pressures on shoring walls and hydrostatic uplift pressures on the floor slab to be nominated for design purposes, however, some preliminary pressures are provided in Sections 4.6 and 4.8, below.

Based on sandy soils the permeability rate will be high, resulting in very fast inflow rates and large volumes of water to extract. To reduce groundwater inflows during bulk excavation, an option that can be considered would be extending the perimeter shoring walls down into the underlying clay layers. Groundwater permeability rates will be high for sands, in the order of 1×10^{-4} m/second, whereas clays may be in the order of 1×10^{-7} m/second. However as discussed above, testing is required to better estimate the rate in sands. We recommend that longer term groundwater monitoring, infiltration testing and groundwater modelling be carried out. Data loggers can be installed in the existing three standpipes to monitor the groundwater fluctuation over time and sensitivity to rainfall.

During construction allowance must be made for temporary removal of groundwater. Temporary dewatering, within the excavation, using spear points, sumps and pumps or other approved methods will be required to allow effective excavation. All collected groundwater should be directed to the stormwater system or an infiltration pit. A dewatering licence will need to be obtained from the Department of Primary Industries Water (formerly the NSW Office of Water) for all temporary dewatering activities.

In the long term the basement will need to be designed as a tanked structure to accommodate lateral water pressures and hydrostatic uplift pressures. Full waterproofing will be required below the floor slabs and walls of the basement structure.

Further groundwater monitoring wells will also need to be installed outside the basement excavations to ensure that groundwater levels are not being drawn down during bulk excavation and dewatering activities.

4.6 Excavation Retention

Due to the sandy soils, high groundwater table and proposed excavation, temporary batters will not be feasible as a shoring option for the perimeter of the site. It is anticipated that the perimeters of the bulk excavation will need to be supported by a properly designed in-situ shoring system, installed prior to excavation commencing. Consideration of the tunnel to the east of the site must also be made in selection of the shoring method. If shoring walls are designed to incorporate building loading and are within the zone of the influence of the tunnel, they should extend below the tunnel invert to avoid transferring any loading to the tunnel infrastructure.

Based on an excavation of about 9.5m we consider suitable shoring systems may include cutter soil mix, diaphragm, or anchored secant pile walls. Contiguous pile walls will not be appropriate as it is not practical to prevent water from infiltrating the site as their will be gaps between piles. Bored piles will not be suitable as they will collapse in the sandy soils. Sheet pile walls are not recommended due to the dense to very dense sands and harder clay bands and potential to cause vibrations that may cause damage to nearby buildings/structures and existing tunnel. The shoring system or combination of systems will need to extend sufficiently below the proposed bulk excavation level and have anchors and/or props to satisfy stability.

Even with good design and construction, some vertical and lateral ground movements beyond the limits of the excavation may occur. The magnitude of the movements is directly related to the stiffness of the shoring system and construction techniques employed. Therefore, during shoring wall design, the wall designer must make an assessment of the likely shoring wall movements and associated adjoining ground movements, so that an assessment of the risk to adjoining structures can be made.

The shoring system will be installed in sandy soils of medium dense to very dense relative density and through thinner bands of silty clay/clayey silt assessed as very stiff to hard, within groundwater. The designer may extend the shoring system into the underlying residual clay unit, assessed as firm to hard.

The use of cutter soil mix walls (CSM) and diaphragm walls are preferred options for this project as they provide a good internal finish, are largely waterproof and do not suffer misalignment issues which affect piled walls. The CSM is the “equivalent” to a secant piled wall.

Diaphragm walls are considered to have similar advantages to that of the CSM wall. Diaphragm walls in the form of concrete cast-in-place in a trench supported by bentonite slurry are practically impervious and if taken down to clay layers or the bedrock will seal the basement excavation so that an extensive dewatering system may not be necessary. However, the provision of drainage wells inside the excavation may still be required. The diaphragm wall may require anchors for lateral stability. We note, however, that the diaphragm wall must be free of any openings such as joints or holes; if these are noted after construction these should be immediately patched up by say, injection grouting. The diaphragm wall could be used a permanent wall, supporting peripheral columns, combined with top-down construction techniques (which would then avoid temporary anchors for lateral stability). Diaphragm walls because of their rigid design and the avoidance of severe ground disturbance, are advantageous forms of support where the yielding and settlement of the adjacent ground surface or property have to be reduced to a minimum. The installation and construction of the diaphragm wall is critically dependent on construction techniques and should only be carried out by contractors of recognised competence in this field of work.

Another shoring system that could be considered, although it has its disadvantages, includes grout injected secant pile walls using CFA grout injected piles. In the secant pile wall, a closed joint may be produced by constructing alternate piles, followed by boring or cutting away part of the first phase piles while the concrete is still weak and thus constructing intermediate piles overlapping with the first phase piles. Secant pile walls rely on drilling 'overlapping' hard and soft piles; however, there is a risk that gaps can develop between piles which will allow higher groundwater inflows and also possibly drop groundwater levels on adjoining sites. It is also important to maintain verticality of the secant piles that may extend to significant depths. Therefore, for these reasons secant piles may not be preferred or would need to be drilled as double rotary cased secant piles. Grouting of gaps between piles, if they may occur, would be required during excavations. Further advice from shoring contractors should be obtained on vertical tolerances for the secant pile system.

Secant pile walls also have the potential to cause damage to adjoining structures as a result of 'decompression', which is a loosening of the ground as a result of the shoring wall installation process. Decompression can cause settlements of the adjoining ground which can affect nearby buildings/structures. This effect can be greatly decreased if a casing system is used as discussed above, though this is more expensive. Since the shoring wall will generally be some metres from the boundaries on the south, east and north ends, these may be considered in these locations. However, since the west side of the site is near the existing parking garage, caution for decompression will need to be taken if the piling option is adopted. Also if piles extend to depths near the APL tunnel there are also risks of decompression around the tunnel. We recommend competent shoring contractors are used for the works.

The shoring system will need to be laterally supported in the short term and in the long term. We assume that in the long term the shoring wall will be laterally supported by the structural floor slabs. In the temporary case anchors and/or props will be required or alternatively, top-down construction methods may be adopted.

Shoring may be required for the construction of the lift pits. Where space permits temporary batters may be formed, as set out above. Temporary batters should not be steeper than 1 Vertical (V) in 1.75 Horizontal (H).

As the excavation will be within the second reserve and near the first reserve of the APL tunnel, finite element analysis will be required to demonstrate the impact the shoring system may have (if any) on the existing tunnel.

4.6.2 Shoring Design

For preliminary shoring design purposes of propped or anchored walls, we recommend that an apparent rectangular lateral earth pressure distribution of 6H or 9H be adopted (where H is the retained height of the soils in metres). The 6H should only be used where some movements of the shoring wall can be tolerated and there are no sensitive services within 2H of the wall. The 9H should be used where movements are to be reduced and/or there are movement sensitive services or structures within a distance of 2H from the wall. In all cases specific shoring wall analysis must be undertaken, including an assessment of the likely ground movements beyond the shoring walls. The shoring wall design engineers should then be requested to provide comment on whether such movements will be problematic to any adjoining structures or services.

Hydrostatic pressures and surcharge loads (such as from adjoining buildings/parking garage, construction equipment etc.) are additional to the above earth pressure recommendations. Shoring designs should also be checked during each stage of the analysis for the effect of a rise in groundwater level.

Toe support for that portion of the shoring system extending below the bulk excavation level will be provided by passive resistance of the underlying soils, as bedrock is much deeper. The passive resistance through at least medium dense sands may be calculated using a passive earth pressure co-efficient (Kp) of 3.2 and a triangular distribution. A factor of safety of at least 2 should be applied to the passive resistance, as relatively large deflections are required to reach the full passive resistance. The upper 0.5m below the proposed bulk excavation level should be ignored in the calculation of passive resistance to allow for possible over-excavation and loosening.

We would be pleased to carry out WALLAP or PLAXIS analyses of the proposed wall system if commissioned. The 2D finite element software (i.e. PLAXIS) can calculate factors of safety for given toe embedments, bending moments, shear forces, anchor loads and wall deflections. We have found from experience that the use of this 2D finite element analysis software can produce significantly more economical shoring wall designs than by adopting the simplified earth pressure envelopes above. We consider that the following geotechnical design parameters could be adopted for preliminary shoring wall design using such software packages.

Preliminary Shoring Wall Design Parameters

Material Type	Unit Weight (above GWL) (kN/m^3)	Unit Weight (below GWL) (kN/m^3)	Effective Friction Angle (degrees)	Effective cohesion (kPa)	Elastic Modulus (MPa)
Silty Sand and Gravelly Sand Fill	19	21	27	0	10
Sand (Loose)	18	20	27	0	10
Sand (Medium Dense)	19	20.5	30	0	30
Sand (Dense)	20	21.5	33	0	80
Sand (Very Dense)	21	22	35	0	100
Clay (Firm)	17	19	18	2	3
Clay (Stiff)	17.5	19.5	20	5	10
Clay (Very Stiff to Hard)	18	20	23	10	50

Anchors used for lateral support must be bonded into either medium dense to dense sandy soils. Where anchors are bonded into sandy soils then they could be designed based on an effective average friction angle of 32°. All anchors should have a minimum free length of 4m (to allow adequate stressing) and a minimum bond length of 4m (although longer bond lengths will probably be required in order to carry the design loads). The bond length of any anchor must be entirely behind a line drawn up at 45° from the bulk excavation level. All anchors will need to be proof loaded to at least 1.3 times their design working load under the supervision of an engineer independent of the anchor contractor. It is normal industry practice for anchors to be a design and construct sub-contract to avoid contractual disputes in the event any anchors fail their test load.

Where anchors extend beyond the site boundaries then it will be necessary to obtain permission from the adjoining property owners before placing anchors below their property. On the eastern side of the site anchors may be restricted by the tunnel, in some locations its setback from the proposed excavation is about 7.5m. Review of the anchor locations with respect to the tunnel will be required during the design process. Impacts the anchors may have on the tunnel will also need to be reviewed. We recommend that the parking garage to the west be reviewed for additional basement levels, since our assessment observed a lower ground level and we were not able to access the garage to review if there were additional basement levels. We recommend that these checks for the anchors be carried out at an early stage of design development as in our experience this permission can often take a considerable period of time to obtain. Alternatively props may be adopted.

Deflection monitoring should be carried out to ensure that deflections of the shoring under load do not exceed design values. This should include both inclinometer and survey monitoring.

4.7 Footings

Based on the expected high loading and loading limits on tunnels (from TfNSW), as discussed above in Section 4.2, it is recommended that all footings loads are transferred below the invert level of the tunnel. To do so, piled footings should be adopted. Based on an approximated bulk excavation level of RL-2.8m sands of medium to very dense would likely be exposed at subgrade level. Based on tunnel invert levels of between roughly RL-20m and RL-21m, piles would need to extend to depths exceeding 17m to 18m, when drilled from bulk excavation level.

Piles are likely to be needed to extend to Class III (or better) siltstone bedrock (Ref: Pells) to optimise the design. Further investigations are required to review the depth and quality of the siltstone bedrock across the remainder of the site. Where the design is based on Class III bedrock, we consider a minimum of 5 additional cored boreholes be carried out. Where the design is based on Class II bedrock cored boreholes are to be carried out in at least 50% of the footings, into the design bearing strata. The geotechnical drilling cost must be considered, in conjunction with the cost saving in the design, if the Class II bedrock ABP is selected. It may be preferable to keep geotechnical costs to a minimum by adopting the Class III bedrock ABP.

Based on the high groundwater level and sands, CFA grout injected piles should be used. Due to the collapsing nature of the sands and the presence of groundwater bored piers are not recommended for this site.

Reference should be made to the table in Section 3.2 for classification of rock in each borehole location. The following table may be referenced for the allowable bearing pressures for the representative 'Class' of the 'shale' bedrock and testing required. 'Shale' bedrock includes the siltstone unit encountered on the site.

Preliminary Allowable Bearing Pressures and Shaft Adhesions

Classification of 'Shale' (Pells et al., 1998)	Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion (kPa)*
V	700	70
IV	1000	100
III	3500	350
II	6000	600

Notes on above table: When calculating the shaft adhesions in tension (i.e. uplift) for the above rock materials, the allowable shaft adhesion values above are to be halved. **A design that adopts the 3500kPa Allowable Bearing Pressure (ABP) must allow for minimum of 5 additional cored boreholes prior to construction. A design that allows for 6000kPa ABP must allow for cored boreholes in at least 50% of the footings.**

If relatively heavy column loads are expected, the use of pile groups would probably be required with minimum pile spacing adopted from AS2159.

The bearing pressures given above are based on a serviceability criteria of deflections at the pile toe of less than or equal to 1% of the pile diameter. Alternatively, if the designer wishes to adopt the limit state design methods of the piling code, AS2159-2009, then the ultimate values of end bearing pressures and shaft adhesions may be estimated by multiplying the above recommended allowable values by Factors of Safety of 3 and 1.5, respectively. We recommend that a geotechnical strength reduction factor, Φ_g, of 0.5 be applied to the maximum pile load applied.

Piled footings should be nominally socketed at least 300mm into the rock unit on which the design is based. During footing construction we recommend that all piles be visually inspected by a geotechnical engineer to confirm that they are founded on a founding stratum consistent with these recommendations and the design structural requirements. We note however that piles will be drilled using grout injected techniques, and as such inspection will essentially only be able to deduce that the pile is founded at a level consistent with the borehole logs. Therefore a suitable number of boreholes will need to be drilled across the site to enable a reasonable judgement on the bearing stratum.

4.8 Lowest Basement Floor Slab

As discussed in Sections 4.1 and 4.5 above, in the long term, the basement will need to be designed as a tanked basement with its lowest floor slab to resist hydrostatic uplift pressures. From review of the current groundwater observations, the hydrostatic uplift pressure on the underside of the basement slab may be in the order of 60kPa to 80kPa during ‘normal’ conditions for three levels of basement. Higher groundwater levels and therefore, higher hydrostatic uplift pressures will likely occur during periods of rainfall and deeper structures constructed below basement level 3, such as lifts, etc. Further groundwater monitoring will need to be carried out during the detailed design stage of the project to allow appropriate groundwater uplift pressures to be nominated for design purposes. We recommend installation of some groundwater data loggers in boreholes so that changes in groundwater level with rainfall can be assessed.

In the absence of specific groundwater level monitoring using data loggers, and assessment of groundwater level rises with rainfall, initial preliminary design should allow for groundwater level rises. The designer should also consider that future developments of a similar scale nearby may result in new flow channels and even higher localised short term water levels. Short term ground water fluctuations could be in the order of 1m to 2m from long term levels. It may be possible to install pressure relief valves to deal with extreme rainfall events. Considering a building up to 12 levels with 3 basement levels this may provide adequate resistance to uplift forces. However, this should be reviewed by the designer. Screw piles/anchors in sands could be adopted where additional resistance is required to secure the basement slab.

Bulk excavation will likely expose subgrade comprising sandy marine soils, possibly with some clay bands. The current investigations indicate that the sandy soils will likely be of at least medium dense relative density and will therefore be suitable to support independent basement slabs (i.e. slabs separated from shoring walls and footings). However, the subgrade will need to be properly prepared, including proof rolling and replacement of unstable areas with engineered fill. Basement slabs, while independent from the shoring walls and footing, will need to be appropriately waterproofed to resist hydrostatic pressures. Prior to the slabs being constructed, the exposed subgrade should be proof-rolled using a two to four tonne minimum dead-weight roller. Density tests should be carried out on the subgrade to ensure that a density of insitu soils of at least 98% of Standard Maximum Dry Density (SMDD) or a density index of at least 70% for the sands is achieved. As the slab is to act as a road or garage pavement then the slab should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD.

A working platform of good quality granular material placed as engineered fill will likely be required to support construction plant, i.e. piling rigs, etc., at subgrade level.

4.9 Further Geotechnical Input

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

- The recommendations provided in this report should be reviewed once details of the proposed development, such as floor levels and bulk excavation levels and structural loads, are revised or determined and also, following any additional geotechnical work. It is likely that further advice/input will be required during the structural and civil engineering design to address issues that may not have been addressed in this report. To some degree, this is an "iterative" process between evaluation of the geotechnical site conditions and the structural and civil designs.
- Dilapidation reports of all structures including tunnel in zone of influence of the excavation.
- Further cored boreholes prior to construction. A minimum of 5 cored boreholes are recommended if design is based on Class III bedrock and where the design is based on Class II bedrock cored boreholes are to be carried out in at least 50% of the footings.
- Assessing response of groundwater with rainfall, carrying out infiltration testing and groundwater modelling to assess extraction rates and groundwater drawdowns.
- 2D finite element analysis, using PLAXIS 2D, to review potential impacts of the proposed works on the existing APL tunnel.
- Piling inspections during construction.
- Proof rolls and density testing of subgrade.
- Review as-built drawings of the basement level to the west.
- Working platform designs for piling rigs.
- it will be essential during earthworks, foundation and pavement works that geotechnical inspections be commissioned to check initial assumptions about any excavation, including for footings, foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for earthworks and foundation works. We recommend that all spread footings be inspected and tested by geotechnical engineer prior to placing steel and pouring concrete. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections.

5 GENERAL COMMENTS

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

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

Occasionally, the subsurface conditions between the completed boreholes and CPTs 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

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31721VJ
Project:	Proposed Mixed Use Development	**Report:**	A
Location:	215-235 O'Riordan Street, Mascot, NSW	**Report Date:**	5/11/2018
		Page 1 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	24.10 - 2412	0.05	1
	24.18 - 24.21	0.07	1
	24.40 - 24.43	0.04	1
	25.04 - 25.06	0.5	10
	25.46 - 25.48	0.6	12
	25.76 - 25.78	1.9	38
	26.05 - 26.07	1.3	26
	26.49 - 26.52	0.7	14
	26.77 - 26.80	0.4	8
	27.26 - 27.29	0.7	14
	27.73 - 27.75	0.6	12
	28.11 - 28.13	0.6	12
	28.27 - 28.30	1.5	30
	28.57 - 28.60	0.6	12
	28.86 - 28.88	1.7	34
	29.17 - 29.19	1.0	20
	29.47 - 29.50	1.3	26
	29.84 - 29.87	1.8	36
	30.07 - 30.10	0.8	16
	30.20 - 30.23	1.3	26
	30.27 - 30.30	2.2	44
2	23.03 - 23.05	0.2	4
	23.38 - 23.41	0.06	1
	23.56 - 23.59	0.1	2
	23.87 - 23.90	0.09	2

NOTES: See Page 3 of 3

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Project:	Proposed Mixed Use Development	**Report:**	A
Location:	215-235 O'Riordan Street, Mascot, NSW	**Report Date:**	5/11/2018
		Page 2 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	24.12 - 24.14	0.2	4
	24.36 - 24.39	0.2	4
	24.66 - 24.69	0.3	6
	24.86 - 24.88	0.6	12
	25.11 - 25.14	0.2	4
	25.43 - 25.47	0.2	4
	25.72 - 25.75	1.0	20
	26.26 - 26.28	0.7	14
	26.76 - 26.79	0.3	6
	27.25 - 27.28	1.0	20
	27.80 - 27.83	0.8	16
	28.16 - 28.19	0.5	10
	28.46 - 28.48	0.7	14
	28.80 - 28.83	1.7	34
	29.20 - 29.23	0.7	14
	29.36 - 29.38	0.9	18
	29.62 - 29.65	0.7	14
	29.83 - 29.86	0.9	18
	30.13 - 30.16	2.8	56
	30.87 - 30.90	3.0	60
3	23.81 - 23.64	0.05	1
	24.27 - 24.30	0.05	1
	24.68 - 24.71	0.07	1
	25.17 - 25.19	0.03	1
	25.73 - 25.76	0.1	2

NOTES: See Page 3 of 3

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Client:	JK Geotechnics	**Ref No:**	31721VJ
Project:	Proposed Mixed Use Development	**Report:**	A
Location:	215-235 O'Riordan Street, Mascot, NSW	**Report Date:**	5/11/2018

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
3	26.18 - 26.21	0.01	<1
	26.52 - 26.54	0.3	6
	26.79 - 26.82	0.2	4
	27.00 - 27.02	0.4	8
	27.90 - 27.93	0.8	16
	28.07 - 28.10	0.9	18
	28.43 - 28.45	1.5	30
	28.91 - 28.93	1.1	22
	29.29 - 29.31	1.3	26
	29.78 - 29.81	0.5	10
	29.72 - 29.74	1.0	20
	30.22 - 30.25	2.1	42
	30.48 - 30.51	0.6	12
	30.82 - 30.84	1.3	26

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

1 / 6

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ | **Method:** SPIRAL AUGER, WASHBORE & CASING ADVANCER | **R.L. Surface:** ~6.5 m
Date: 24/10/18 TO 25/10/18 | **Datum:** AHD
Plant Type: JK500 | **Logged/Checked By:** B.Z./J.M.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		0		CONCRETE: 130mm.t				
			-	FILL: Silty sand, fine to medium grained, light brown.	M			APPEARS POORLY TO MODERATELY COMPACTED
	N = 8 5,3,5			as above, but with fine to coarse grained sub angular ironstone and sandstone gravel.				
				FILL: Silty sand, fine to medium grained, dark brown.				
	N = 11 4,5,6	1.5	SM	Silty SAND: fine to medium grained, light brown and dark brown.	M	MD		MARINE
								SLIGHT ORGANIC ODOUR
AFTER 12 HRS AND 5 DAYS	N = 24 7,11,13	3.0			W			
	N = 19 3,9,10	4.5		as above, but brown.				
	N = 25 3,10,15	6.0						

RL (m AHD) scale: 6, 5, 4, 3, 2, 1, 0

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Method:** SPIRAL AUGER, WASHBORE & CASING ADVANCER **R.L. Surface:** ~6.5 m
Date: 24/10/18 TO 25/10/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./J.M.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SM	as above, but brown. *(continued)*	W	MD		
	DS	N = 33 9,13,20	-1						D		
				8							
			-2								
	DS	N = 31 6,10,21		9							
			-3								
				10							
	DS	N > 30 18,30/ 150mm REFUSAL	-4						D - VD		
				11							
			-5								
				12			as above, but with organic silty clay/sandy silt bands.		D		
			-6								
				13							
	DS	N = 38 16,16,22	-7								

BOREHOLE LOG

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**Method:** SPIRAL AUGER, WASHBORE & CASING ADVANCER	**R.L. Surface:** ~6.5 m
Date: 24/10/18 TO 25/10/18		**Datum:** AHD
Plant Type: JK500	**Logged/Checked By:** B.Z./J.M.	

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SM	as above, but with organic silty clay/sandy silt bands. *(continued)*	W	D		
			-8								
				15							
			-9								
				16		CH	Silty CLAY: high plasticity, grey.	w>PL	F		
	DS	N = 4 0,2,2	-10							80 90 90	
				17							
			-11								
				18							
			-12								
				19							
	DS	N = 6 0,0,6	-13				as above, but light grey mottled yellow brown.			90 90 90	
				20							
			-14								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ
Method: SPIRAL AUGER, WASHBORE & CASING ADVANCER
R.L. Surface: ~6.5 m
Date: 24/10/18 TO 25/10/18
Datum: AHD
Plant Type: JK500
Logged/Checked By: B.Z./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■					CH	as above, but red brown.	w>PL	(St)		
						-15				REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 13.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 13.2m TO 10.2m. CASING 10.2m TO 0.1m. 2mm SAND FILTER PACK 13.2m TO 9.7m. BENTONITE SEAL 9.7m TO 9.0m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER
							22							
						-16								
							23							
						-17								
							24							
						-18								
							25							
						-19								
							26							
						-20								
							27							
						-21								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
1

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**Core Size:** NMLC	**R.L. Surface:** ~6.5 m
Date: 24/10/18 TO 25/10/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK500	**Bearing:** N/A	**Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT SPACING (mm) 600 200 60 20	DEFECT DESCRIPTION Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness Specific / General	Formation
		-14	21		START CORING AT 21.20m						
		-15			NO CORE 0.45m						
			22		Silty CLAY: high plasticity, light grey.	RS	VSt Hd			(21.65-22.50m) RESIDUAL HP: 350, 300, 350, 320, 360 kPa (22.10m) HP: 400, 400, 450, 460, 500, 520 kPa	
		-16			Extremely Weathered siltstone: silty CLAY, high plasticity, grey and light grey, trace of very low strength iron indurated bands.	XW				(22.50m) HP: 460, 500, 520, 550, 480 kPa	
			23		as above, but grey and yellow brown, with very low strength iron indurated bands.		VSt - Hd			(23.10m) HP: 380, 400, 420, 460, 390 kPa	
100% RETURN		-17			SILTSTONE: dark grey, with light grey laminae and extremely weathered bands.	XW - HW	VL				Ashfield Shale
		-18	24					0.050 0.070 0.040			
			25			MW - SW	M	0.50		(25.10m) J, 35°, P, R, Cn (25.25m) CS, 0°, 50 mm.t	
		-19						0.60		(25.45m) Be, 0°, P, S, Cn (25.50m) Be, 0°, P, S, Cn (25.54m) Be, 0°, P, S, Cn (25.58m) Be, 0°, P, S, Cn (25.65m) CS, 0°, 20 mm.t (25.73m) Be, 0°, P, S, Cn (25.83m) XWS, 0°, 10 mm.t (25.90m) CS, 0°, 5 mm.t	
			26				M - H	1.9 1.3		(25.97m) CS, 0°, 3 mm.t (26.02m) CS, 0°, 10 mm.t (26.04m) CS, 0°, 6 mm.t (26.12m) CS, 0°, 10 mm.t (26.17m) CS, 0°, 8 mm.t (26.26m) Be, 0°, P, S, Cn (26.28m) Be, 0°, P, S, Cn	
		-20					M	0.70 0.40		(26.30m) CS, 0°, 3 mm.t (26.37m) CS, 0°, 10 mm.t (26.41m) Be, 0°, P, S, Cn (26.57m) Be, 0°, P, S, Cn (26.65m) CS, 0°, 10 mm.t (26.70m) CS, 0°, 7 mm.t	

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

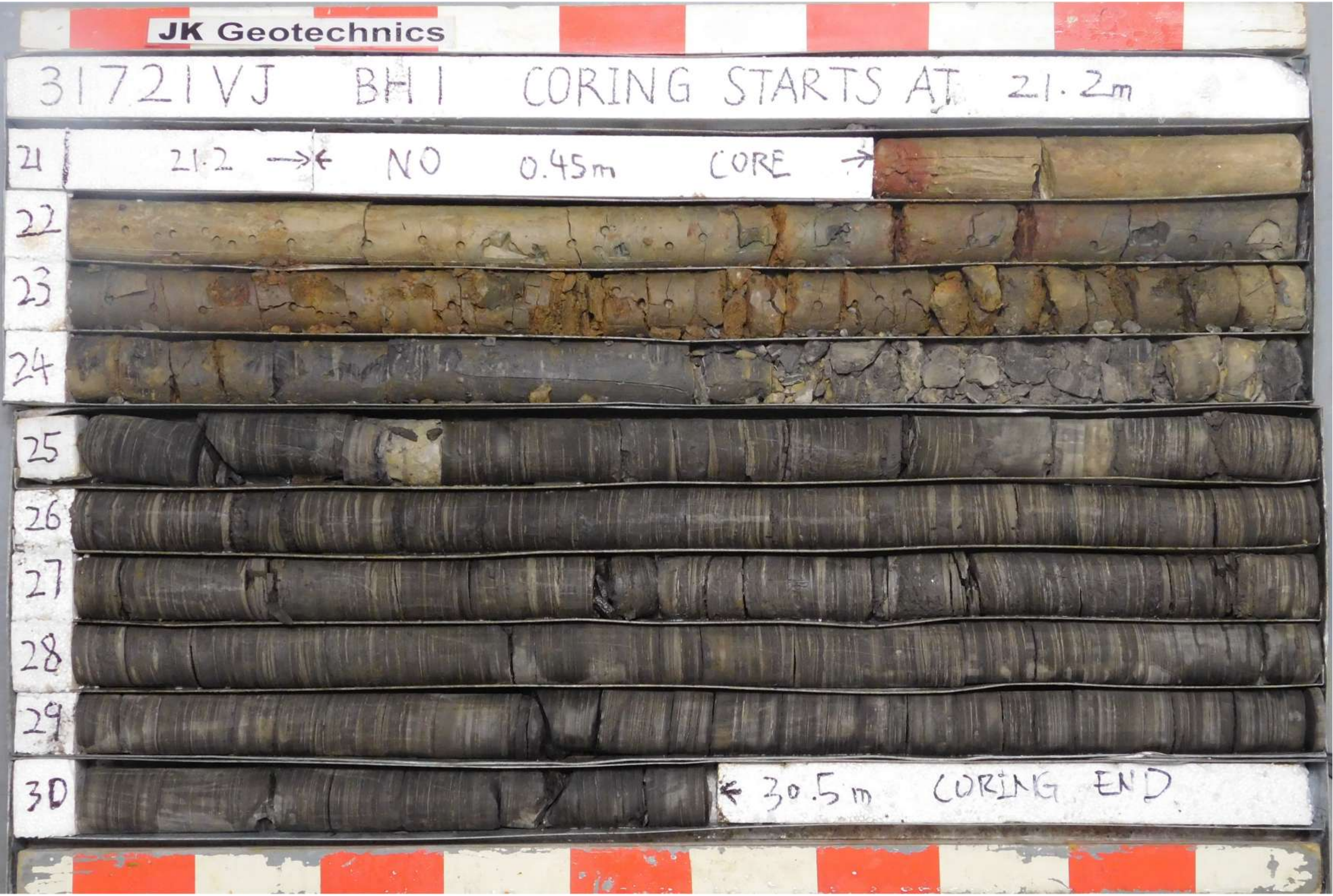
Borehole No.
1

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**Core Size:** NMLC	**R.L. Surface:** ~6.5 m
Date: 24/10/18 TO 25/10/18	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK500	**Bearing:** N/A	**Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness Specific / General	Formation
100% RETURN		-21, -22, -23	28, 29, 30		SILTSTONE: dark grey, with light grey laminae and extremely weathered bands. *(continued)*	MW - SW	M	0.70 0.60 0.60		(26.72m) Be, 0°, P, S, Cn (26.76m) Be, 2°, P, S, Cn (26.88m) CS, 0°, 10 mm.t (27.04m) Be, 0°, P, S, Cn (27.16m) Be, 0°, P, S, Cn (27.19m) Be, 0°, P, S, Cn (27.34m) Be, 0°, P, S, Cn (27.39m) Be, 0°, P, S, Cn (27.40m) CS, 0°, 3 mm.t (27.47m) CS, 0°, 3 mm.t (27.50m) Be, 0°, P, S, Cn (27.53m) CS, 0°, 3 mm.t (27.55m) Be, 0°, P, S, Cn (27.57m) CS, 0°, 5 mm.t (27.62m) Be, 0°, P, S, Cn (27.63m) XWS, 0°, 40 mm.t	Ashfield Shale
					as above, but without extremely weathered bands.	FR	M - H	1.5 0.60 1.7 1.0 1.3 1.8 0.80 1.3 2.2	60, 20	(27.73m) CS, 2°, 7 mm.t (27.77m) CS, 3°, 6 mm.t (27.81m) CS, 0°, 5 mm.t (27.83m) Be, 0°, P, S, Cn (27.84m) Be, 0°, P, S, Cn (27.90m) Be, 1°, P, S, Cn (27.93m) Be, 1°, P, S, Cn (27.99m) Be, 1°, P, S, Cn (28.07m) Be, 2°, P, S, Cn (28.10m) Be, 2°, P, S, Cn (28.35m) Be, 0°, P, S, Cn (28.76m) Be, 2°, P, S, Cn (29.03m) Be, 0°, P, S, Cn (29.07m) Be, 0°, P, S, Cn (29.12m) Be, 0°, P, S, Cn (29.37m) J, 33°, C, Vr, Cn (29.72m) J, 47°, Cn (30.13m) J, 50°, Cn (30.23m) Be, 0°, P, S, Cn (30.36m) J, 45°, P, R, Cn (30.38m) J, 40°, P, R, Cn (30.43m) Be, 0°, P, S, Cn	
		-24, -25, -26, -27	31, 32, 33		END OF BOREHOLE AT 30.50 m				600, 200, 60, 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
31721VJ BH1 CORING STARTS AT 21.2m
21
21.2 → ← NO 0.45m CORE →
22
23
24
25
26
27
28
29
30
← 30.5m CORING END

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ
Method: SPIRAL AUGER, WASHBORE & CASING ADVANCER
R.L. Surface: ~6.7 m
Date: 26/10/18 TO 29/10/18
Datum: AHD
Plant Type: JK500/WASHBORE
Logged/Checked By: B.Z./J.M.

Depth (m)	Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
0 – 0.15				CONCRETE: 150mm.t				
0.15		N = 8 3,3,5	-	FILL: Gravelly sand, fine to medium grained, light brown, fine to coarse grained sub angular and angular igneous and sandstone gravel, trace of sub angular and angular igneous and sandstone cobbles.	M			APPEARS POORLY TO MODERATELY COMPACTED
				FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained sub angular ironstone gravel.				
1.0			SM	Silty SAND: fine to medium grained, light grey.	M	L		MARINE
1.5		N = 13 3,7,6				MD		
2.2				as above, but brown.				
3.0		N = 21 5,7,14		as above, but dark brown.				
~3.6	▶				W			
4.5		N > 4 7,4/ 100mm REFUSAL				MD - D		
6.0		N = 20 5,8,12				MD		

SAMPLES (ES, U50, DB, DS): samples taken between approx. 0.15–1.95 m, 3.0–3.45 m, 4.5–4.75 m and 6.0–6.45 m.

RL (m AHD): 6, 5, 4, 3, 2, 1, 0

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ
Method: SPIRAL AUGER, WASHBORE & CASING ADVANCER
R.L. Surface: ~6.7 m
Date: 26/10/18 TO 29/10/18
Datum: AHD
Plant Type: JK500/WASHBORE
Logged/Checked By: B.Z./J.M.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SM	as above, but dark brown. *(continued)*	W	MD		
	DS	N = 31 6,11,20	-1	8					D		
	DS	N > 30 10,18,12/ 50mm REFUSAL	-2	9					D - VD		
	DS	N > 30 6,18,12/ 60mm REFUSAL	-3, -4	10, 11							
			-5, -6, -7	12, 13							

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ
Method: SPIRAL AUGER, WASHBORE & CASING ADVANCER
R.L. Surface: ~6.7 m
Date: 26/10/18 TO 29/10/18
Datum: AHD
Plant Type: JK500/WASHBORE
Logged/Checked By: B.Z./J.M.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SM	as above, but dark brown. *(continued)*	W	D - VD		
	DS	N > 30 13,20,10/ 30mm REFUSAL	-8								
				15							
			-9								
				16							
			-10								
				17							
	DS	N = 29 5,11,18	-11				as above, but with silty clay bands, trace of timber fragments.		MD		
				18							
			-12								
				19		CH	Silty CLAY: high plasticity, light grey mottled yellow brown.	w>PL	(VSt)		RESIDUAL
			-13								
				20							
	DS	N = 34 9,12,22	-14					w~PL	VSt - Hd	350 400 450	
							see description below,				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Client: ELDESO GROUP

Project: PROPOSED MIXED USE DEVELOPMENT

Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ
Method: SPIRAL AUGER, WASHBORE & CASING ADVANCER
R.L. Surface: ~6.7 m

Date: 26/10/18 TO 29/10/18
Datum: AHD

Plant Type: JK500/WASHBORE
Logged/Checked By: B.Z./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						-15	22		CH	as above, but light grey and red brown, trace of fine to medium grained sub angular ironstone gravel.	w~PL	VSt - Hd		
						-16 -17 -18 -19 -20 -21	23 24 25 26 27			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 14.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 14.2m TO 9.2m. CASING 9.2m TO 0.0m. 2mm SAND FILTER PACK 14.2m TO 10.0m. BENTONITE SEAL 10.0m TO 9.5m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER

CORED BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Core Size:** NMLC **R.L. Surface:** ~6.7 m
Date: 26/10/18 TO 29/10/18 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK500/WASHBORE **Bearing:** N/A **Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness (Specific / General)	Formation
		-15	22		START CORING AT 22.20m						
100% RETURN		-16			NO CORE 0.48m						
					Silty CLAY: high plasticity, light grey and rd brown, with medium to high strength ironstone bands.	RS	VSt			(22.80m) HP: 200, 250, 300 kPa	
			23		Extremely Weathered siltstone: silty CLAY, high plasticity, grey and brown.	XW	Hd	0.20		(23.00m) HP: 400, 450, 460, 450 kPa	Ashfield Shale
		-17			SILTSTONE: grey and dark brown, with low strength iron indurated bands and clay seams.	XW - HW	VL	0.060 0.10			
			24		as above, but dark grey, without iron indurated bands.			0.090			
		-18			SILTSTONE: dark grey, with light grey laminae.	MW	L	0.20 0.20 0.30		(24.41m) Be, 0°, P, S, Cn (24.79m) Be, 0°, P, S, Cn	
			25		as above, but with extremely weathered bands.		L - M	0.60 0.20 0.20		(24.90-25.50m) NUMEROUS XWS & CS	
		-19	26		as above, but trace of extremely weathered bands.		M	1.0		(25.79m) Be, 0°, P, S, Cn (25.83m) Be, 0°, P, S, Cn (25.90m) Be, 0°, P, S, Cn	
		-20				SW		0.70 0.30		(26.26m) Be, 0°, P, S, Cn (26.45m) XWS, 0°, 60 mm.t (26.68m) Be, 0°, P, S, Cn	
		-21	27					1.0 0.80	20	(27.07m) XWS, 0°, 12 mm.t (27.11m) Be, 0°, P, S, Cn (27.13m) Be, 0°, P, S, Cn (27.18m) Be, 0°, P, S, Cn (27.20m) Be, 0°, P, S, Cn (27.23m) Be, 0°, P, S, Cn (27.29m) Be, 3°, P, S, Cn (27.32m) CS, 0°, 12 mm.t (27.36m) CS, 0°, 6 mm.t (27.40m) Be, 0°, P, S, Cn (27.42m) Be, 3°, P, S, Cn (27.49m) Be, 0°, P, S, Cn (27.51m) CS, 0°, 17 mm.t	

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
2

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ | **Core Size:** NMLC | **R.L. Surface:** ~6.7 m
Date: 26/10/18 TO 29/10/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK500/WASHBORE | **Bearing:** N/A | **Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific / General	Formation
100% RETURN		-22, -23, -24	29, 30		as above, but trace of extremely weathered bands. *(continued)*	SW	M	0.50, 0.70, 1.7		(27.61m) Be, 0°, P, S, Cn (27.71m) Be, 0°, P, S, Cn (27.75m) Be, 2°, P, S, Cn (27.78m) CS, 2°, 6 mm.t (27.83m) CS, 0°, 13 mm.t (27.90m) CS, 0°, 6 mm.t (27.93m) Be, 0°, P, S, Cn (28.08m) CS, 0°, 16 mm.t (28.12m) CS, 0°, 5 mm.t (28.13m) Be, 0°, P, S, Cn (28.16m) Be, 0°, P, S, Cn (28.18m) Be, 0°, P, S, Cn (28.19m) Be, 0°, P, S, Cn (28.21m) XWS, 0°, 30 mm.t (28.25m) XWS, 0°, 25 mm.t (28.29m) Be, 0°, P, S, Cn	Ashfield Shale
							M - H				
						FR		0.70, 0.90, 0.70, 0.90, 2.8, 3.0		(28.32m) CS, 0°, 10 mm.t (28.37m) CS, 0°, 11 mm.t (28.41m) XWS, 0°, 30 mm.t (28.54m) Be, 0°, P, S, Cn (28.61m) Be, 0°, P, S, Cn (28.68m) XWS, 0°, 26 mm.t (28.71m) Be, 0°, P, S, Cn (28.74m) Be, 0°, P, S, Cn (28.87m) XWS, 0°, 30 mm.t (28.97m) Be, 0°, P, S, Cn (29.01m) CS, 0°, 13 mm.t (29.08m) Be, 0°, P, S, Cn (29.13m) Be, 0°, P, S, Cn (29.15m) Be, 0°, P, S, Cn (29.18m) Be, 0°, P, S, Cn	
		-25, -26, -27, -28	31, 32, 33, 34		END OF BOREHOLE AT 30.90 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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31721 VJ BH2 CORING STARTS AT 22.2m
22
22.2 → ← NO 0.48m CORE →
23
24
25
26
27
28
29
30
← 30.90m
END

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Method:** SPIRAL AUGER **R.L. Surface:** ~6.7 m
Date: 30/10/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./J.M.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				0		CONCRETE: 100mm.t				
	ES, DS				-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained sub angular igneous and ironstone gravel.	M			
	ES, DS	N = 10 3,5,5	6		SM	Silty SAND: fine to medium grained, grey.	M	L - MD		POSSIBLE FILL
				1		Silty SAND: fine to medium grained, dark brown.	M	MD		MARINE ORGANIC ODOUR
	ES, DS	N = 20 5,7,13	5	2						
▶ (approx. 3.0 m)	DS	N = 22 5,8,14	4	3			W			
			3	4						
	DS	N = 21 5,7,14	2	5						
	DS	N = 12 2,2,10	1	6		as above, but with silty clay to clayey silt bands.				
			0							

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31721VJ MASCOT.GPJ <<DrawingFile>> 27/11/2018 12:20 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3
2 / 6

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Method:** SPIRAL AUGER **R.L. Surface:** ~6.7 m
Date: 30/10/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
										Silty SAND: fine to medium grained, dark brown.	W	MD		
						-1								
							8		SM	as above, but light grey.				
						-2								
					N=SPT 18/ 50mm REFUSAL		9					MD - VD		SPT SAMPLER MAY HAVE BEEN FILLED PRIOR TO TESTING AND THEREFORE 'N' COUNTS MAY BE HIGHER THAN USUAL; FOR TESTS AT 9.0m, 10.5m AND 15.0m DEPTHS
						-3								
							10							
					N=SPT 11/ 150mm REFUSAL	-4								
							11							
						-5								
							12							
						-6								
							13							
						-7								

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 3

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Method:** SPIRAL AUGER **R.L. Surface:** ~6.7 m
Date: 30/10/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./J.M.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						SM	as above, but light grey. *(continued)*	W	MD - VD		
			-8								
	DS	N=SPT 24/ 150mm REFUSAL		15			as above, but with high plasticity, dark grey organic clay bands, trace of timber fragments.				
			-9	16							
			-10	17							
			-11								
	DS	N > 25 2,15,10/ 50mm REFUSAL		18		CH	Sandy CLAY: high plasticity, grey, with fine to medium grained dark brown clayey sand bands.	w>PL	F	80 70 70	RESIDUAL
			-12	19							
			-13	20							
			-14								

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Borehole No.
3

BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **Method:** SPIRAL AUGER **R.L. Surface:** ~6.7 m
Date: 30/10/18 **Datum:** AHD
Plant Type: JK500 **Logged/Checked By:** B.Z./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						-15	22		CH	Silty CLAY: high plasticity, light grey.	w>PL	(VSt)		
						-16 -17 -18 -19 -20 -21	23 24 25 26 27			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 11.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 11.5m TO 8.5m. CASING 8.5m TO 0.0m. 2mm SAND FILTER PACK 11.5m TO 8.0m. BENTONITE SEAL 8.0m TO 7.0m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER

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CORED BOREHOLE LOG

Borehole No.
3

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ | **Core Size:** NMLC | **R.L. Surface:** ~6.7 m
Date: 30/10/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK500 | **Bearing:** N/A | **Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: SPACING (mm)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness (Specific / General)	Formation
		-15	22		START CORING AT 22.06m						
100% RETURN		-16			Extremely Weathered siltstone: silty CLAY, high plasticity. light grey and red brown.	XW	Hd			(22.20m) HP: 450, 500, 550 kPa	Ashfield Shale
			23		as above, but light grey and brown, with very low strength bands and very low strength iron indurated bands.	XW - HW	Hd - VL			(22.50m) HP: 550, 600, >600 kPa	
					NO CORE 0.50m					~~(23.60m) HP: 400, 420, 450 kPa~~	
		-17			Extremely Weathered siltstone: silty CLAY, high plasticity, grey and red brown, with very low to low strength iron indurated bands..	XW	Hd				Ashfield Shale
			24			XW - HW	VL	0.050		(23.80m) HP: 550, 560, 580 kPa	
		-18	25		SILTSTONE: dark grey, with brown and light grey laminae, and clay bands.			0.050 0.070 0.030	600 200 60 20	(24.80m) HP: 300, 350, 420, 450, 500 kPa	
		-19	26		SILTSTONE: dark grey, with light grey laminae, and extremely weathered bands.	HW	VL - L	0.10 0.010		(25.40-26.50m) NUMEROUS XWS & CS (26.15m) J, 80°, Un, R, Cn	
		-20	27		as above, but trace of extremely weathered bands.	SW	L - M	0.30 0.20		(26.59m) J, 80°, Un, R, Cn (26.61m) Be, 0°, P, S, Cn (26.63m) Be, 0°, P, S, Cn (26.66m) Be, 0°, P, S, Cn (26.72m) Be, 0°, P, S, Cn (26.89m) J, 65°, P, R, Cn (26.91m) CS, 0°, 5 mm.t (26.94m) CS, 0°, 10 mm.t	
							M	0.40		(27.18m) Be, 0°, P, S, Cn (27.31m) J, 25°, P, R, Cn (27.35m) J, 40°, P, R, Cn (27.40m) XWS, 0°, 40 mm.t (27.50m) J, 85°, P, R, Cn (27.40-27.76m) FRAGMENTED ZONE	
		-21				MW		0.80	200 60 20	(27.76-28.00m) CORE LIKELY TO HAVE BEEN DAMAGED DUE TO DRILLING	

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.01.2 LIB.GLB Log JK CORED BOREHOLE - MASTER 31721VJ MASCOT.GPJ <<DrawingFile>> 27/11/2018 12:20 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ | **Core Size:** NMLC | **R.L. Surface:** ~6.7 m
Date: 30/10/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK500 | **Bearing:** N/A | **Logged/Checked By:** B.Z./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Core Description (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Point Load Strength Index $I_s(50)$	Defect Spacing (mm)	Defect Description – Specific (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness)	Defect Description – General	Formation
100% RETURN		-22, -23, -24	29, 30	as above, but trace of extremely weathered bands. *(continued)*	FR	M - H	0.90, 1.5, 1.1, 1.3, 0.50, 1.0, 2.1, 0.60, 1.3		(28.10m) XWS, 0°, 30 mm.t (28.28m) J, 30°, Un, R, Cn (28.30m) Be, 0°, P, S, Cn (28.32m) Be, 0°, P, S, Cn (28.34m) J, 35°, Un, R, Cn (28.46m) J, 65°, Un, Cn (28.81m) J, 80°, P, R, Cn (29.04m) J, 12°, Un, R, Cn (29.15m) Be, 0°, P, S, Cn (29.39m) Be, 0°, P, S, Cn (30.15m) J, 33°, Un, R, Cn (30.38m) J, 70°, P, R, Cn (30.68m) Be, 0°, P, S, Cn		Ashfield Shale
		-25, -26, -27, -28	31, 32, 33, 34	END OF BOREHOLE AT 30.84 m							

Strength scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing scale: 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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31721 VJ BH3 CORING STARTS AT: 22.06m
22
23
NO CORE
24
25
26
27
28
29
30
30.84 END

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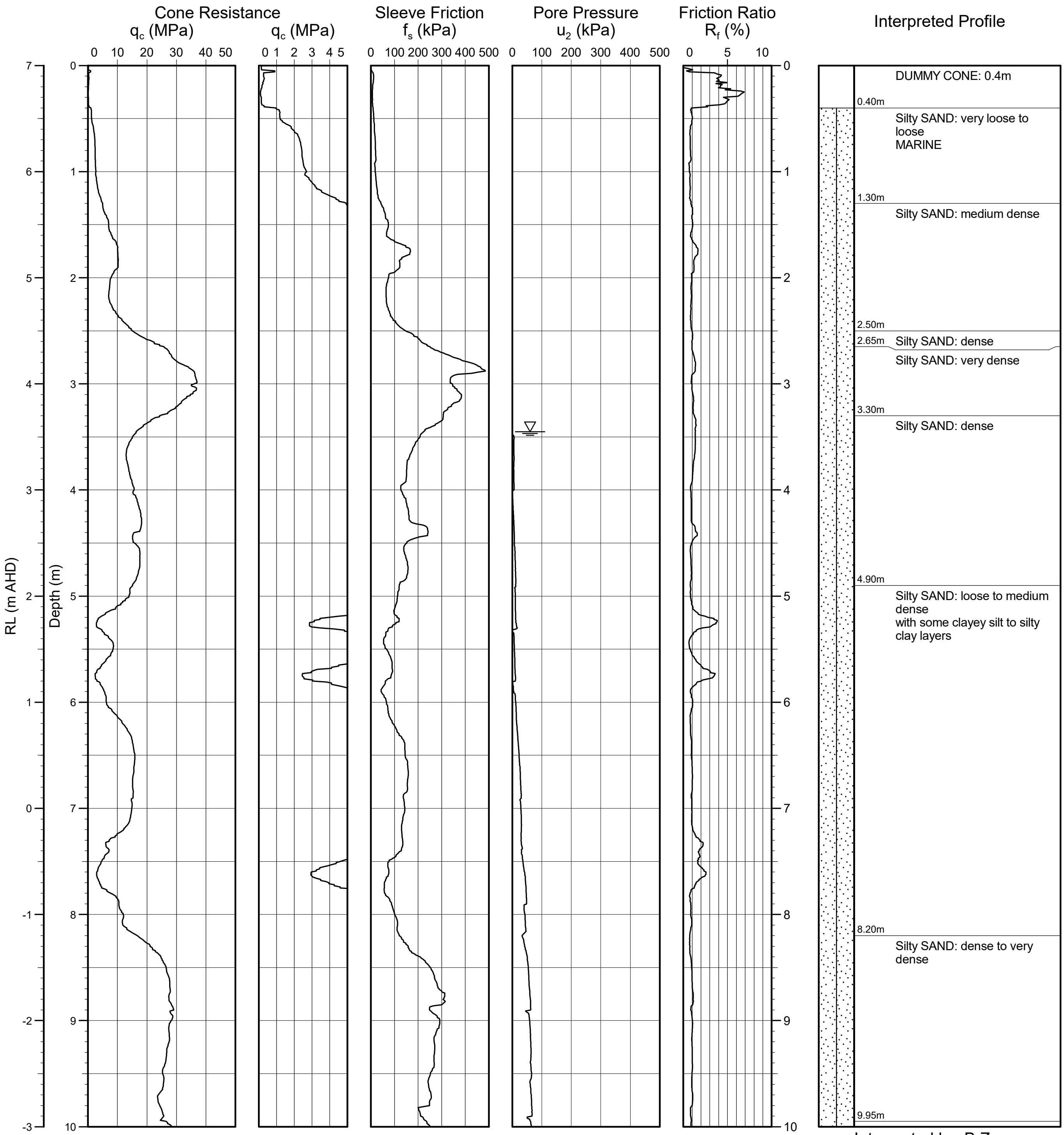
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
4

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~7.0 m	**Data File:** 31721VJ_4.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

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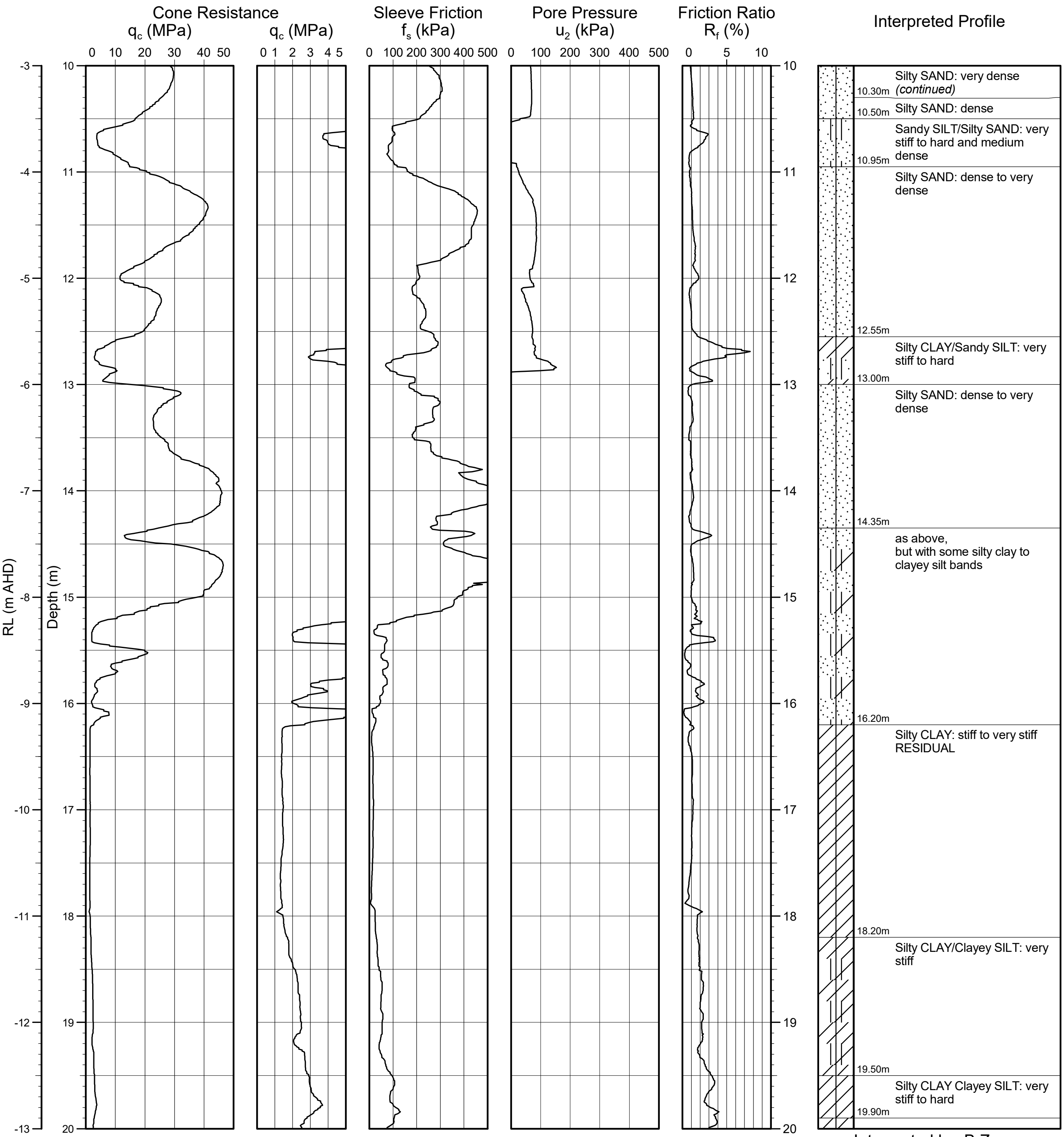
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
4
2 / 3

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~7.0 m	**Data File:** 31721VJ_4.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

JK 9.01.2 LIB.GLB Log JK CPTU MATERIAL - MASTER 31721VJ MASCOT.GPJ <<DrawingFile>> 26/11/2018 11:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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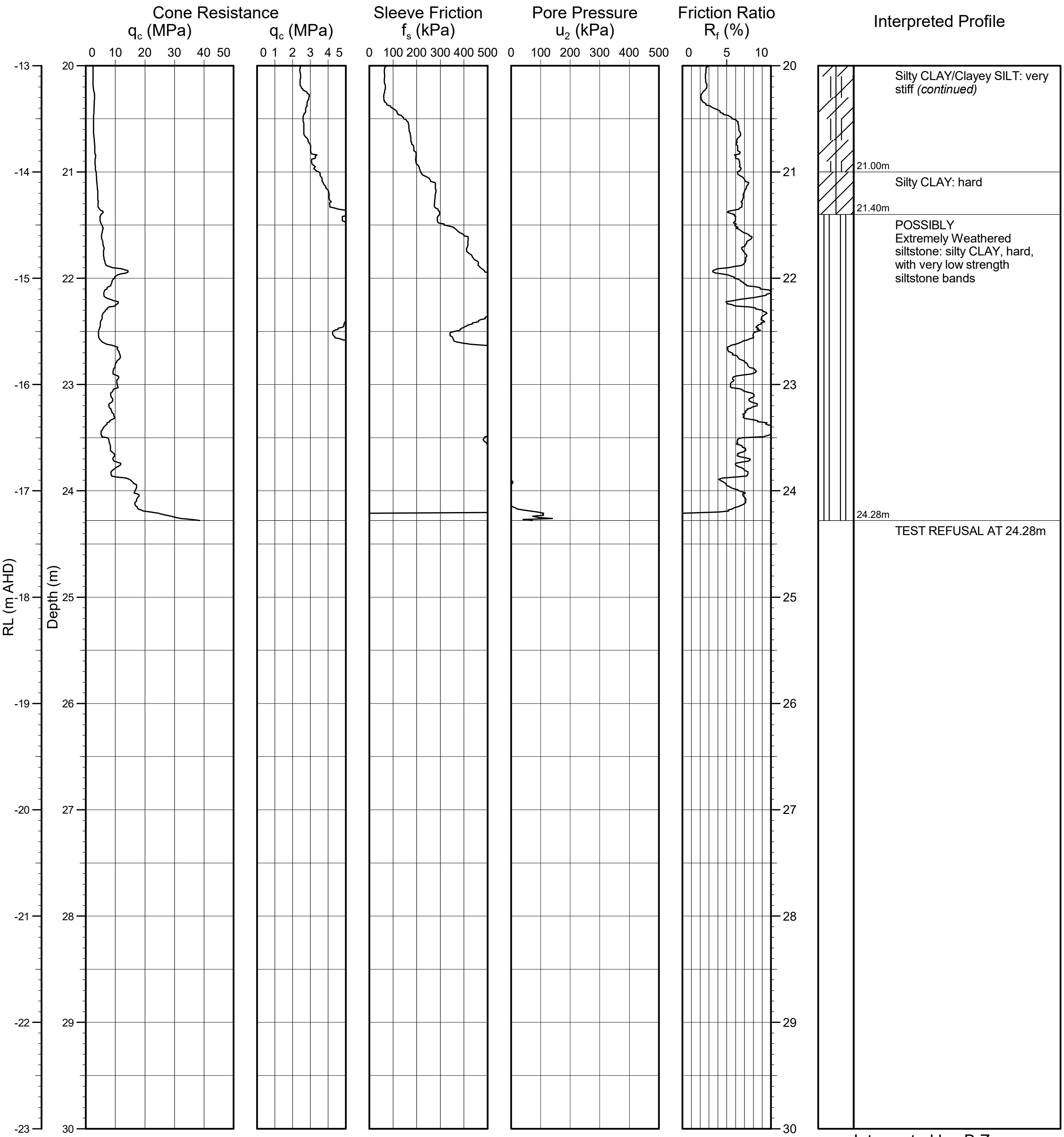
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
4

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~7.0 m	**Data File:** 31721VJ_4.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

JK 9.01.2 LIB.GLB Log JK CPTU MATERIAL - MASTER 31721VJ MASCOT.GPJ <<DrawingFile>> 26/11/2018 11:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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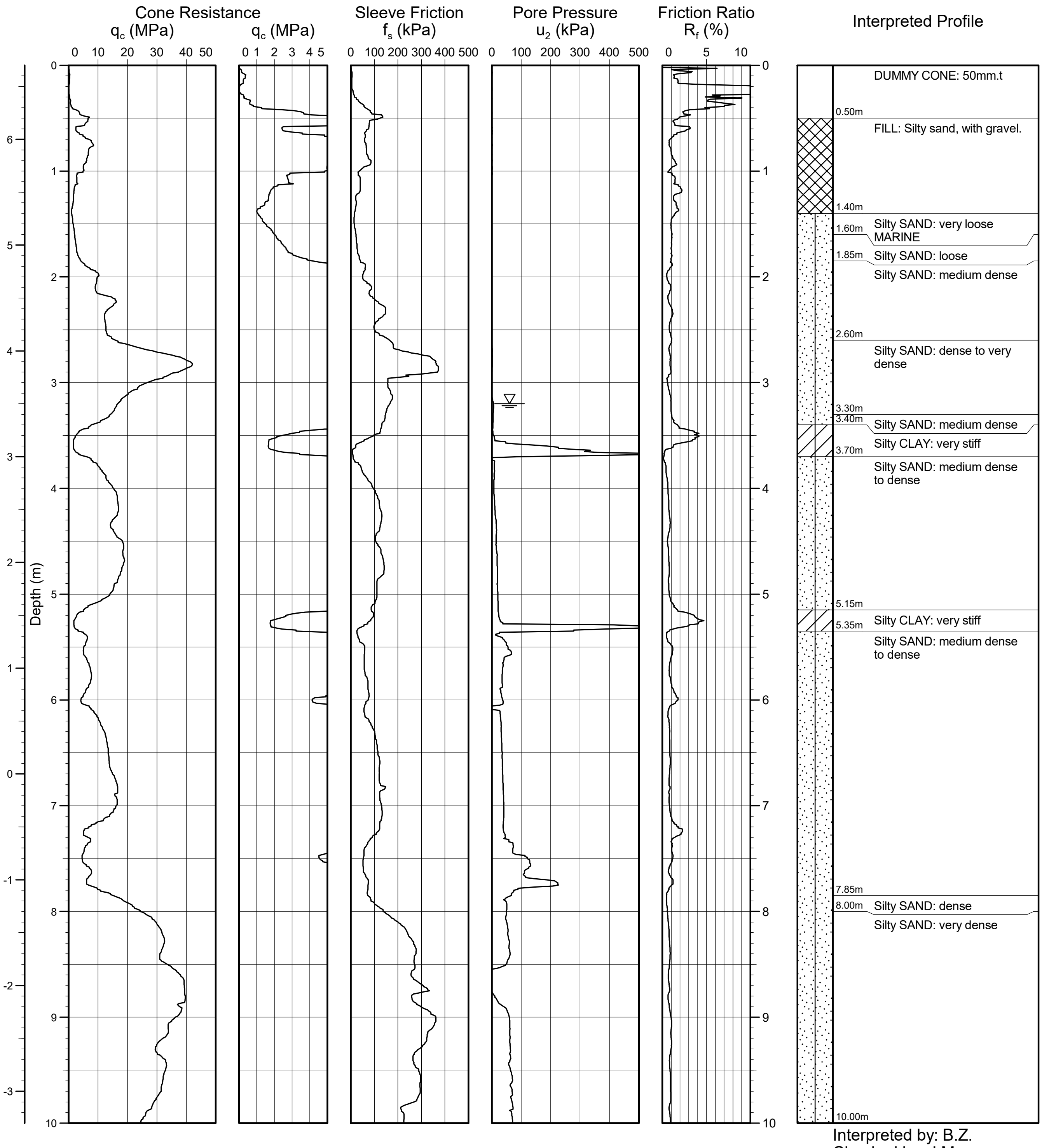
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No. 5

CONE PENETROMETER TEST RESULTS

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **R.L. Surface:** ~6.7 m **Data File:** 31721VJ_5.GEF
Date: 2/11/18 **Datum:** AHD **Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

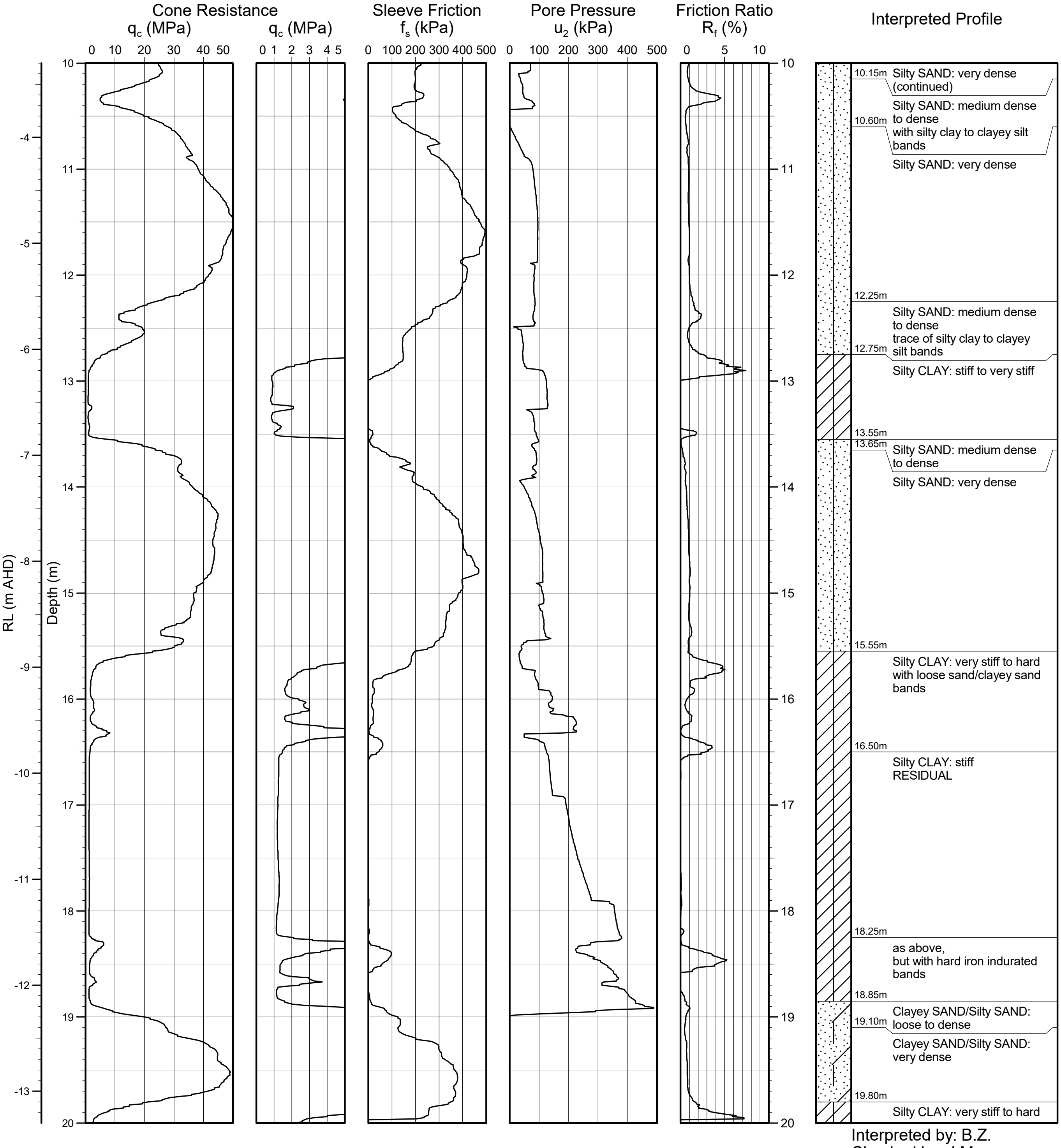
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
5
2 / 3

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~6.7 m	**Data File:** 31721VJ_5.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

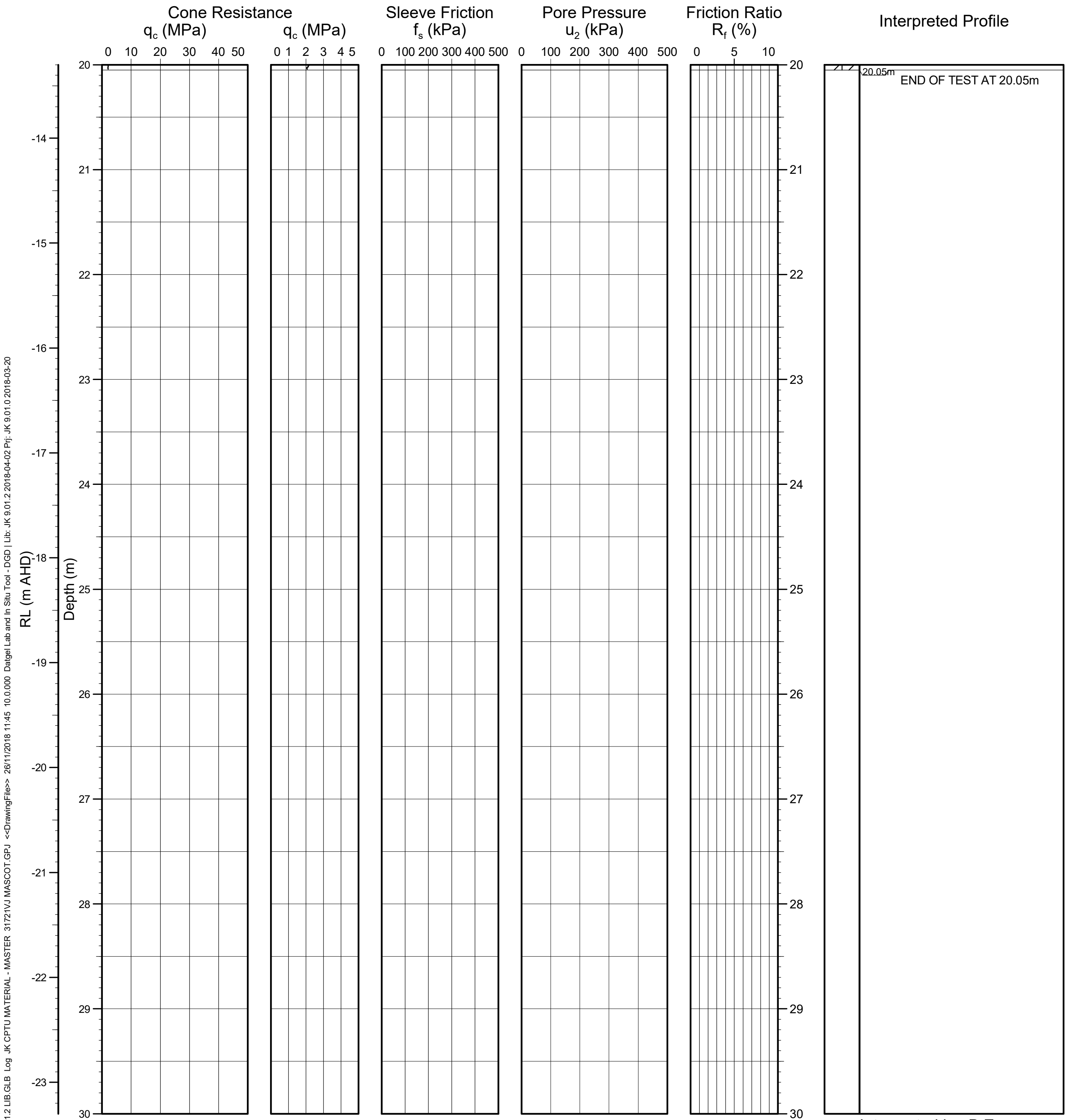
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
5

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~6.7 m	**Data File:** 31721VJ_5.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

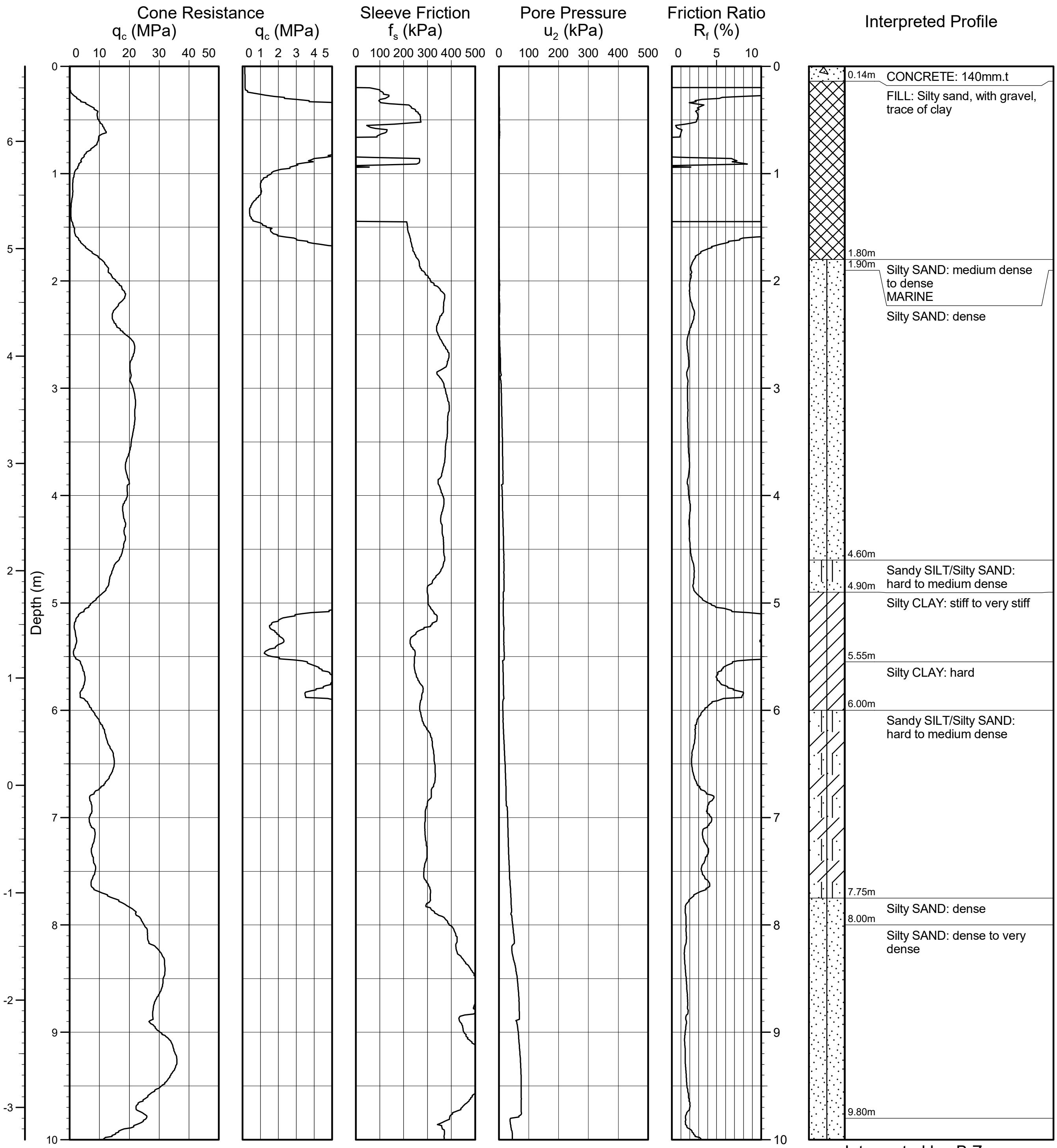
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
6

CONE PENETROMETER TEST RESULTS

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **R.L. Surface:** ~6.7 m **Data File:** 31721VJ_6.GEF
Date: 2/11/18 **Datum:** AHD **Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

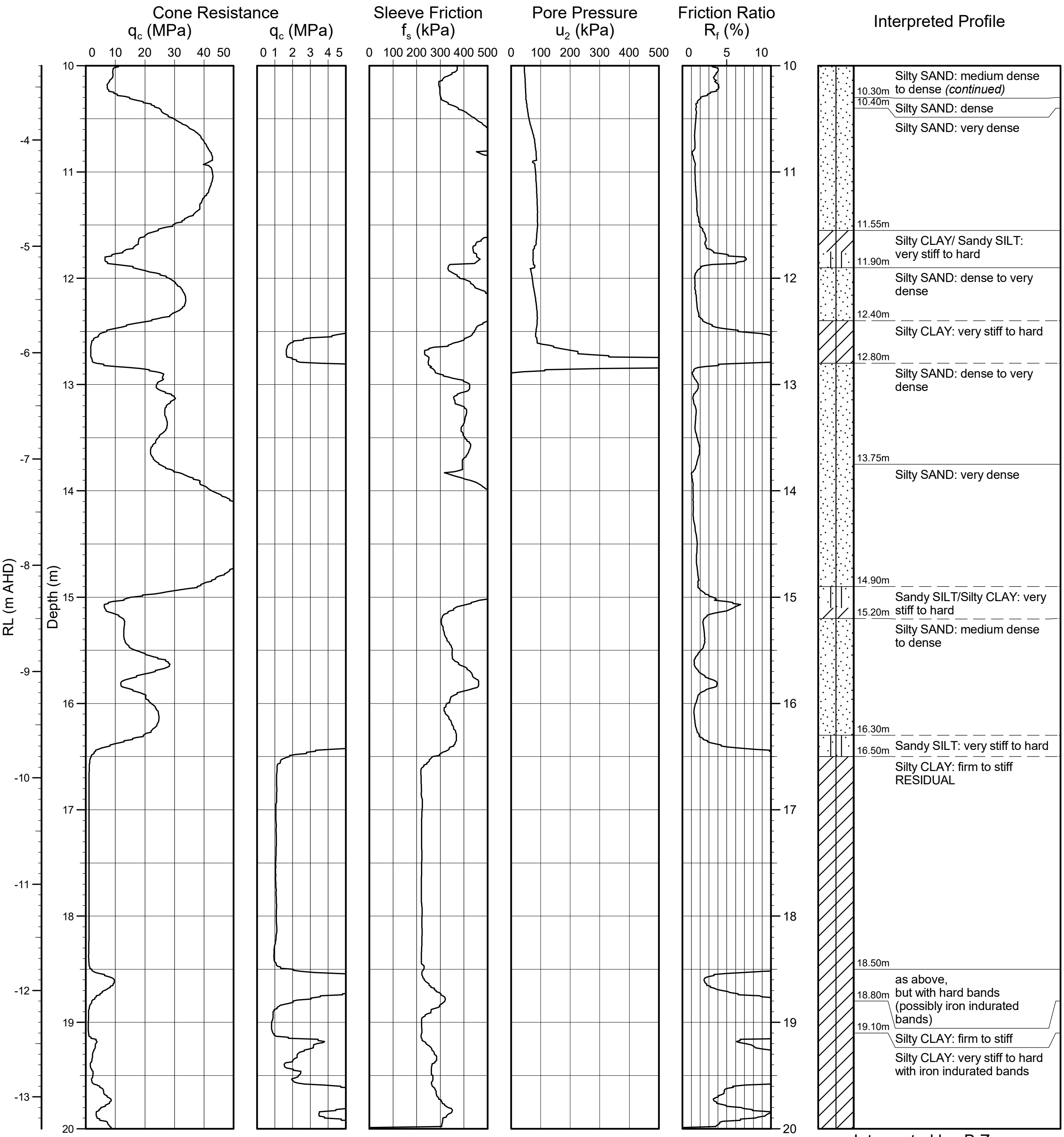
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No. 6

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~6.7 m	**Data File:** 31721VJ_6.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

JK 9.01.2 LIB.GLB Log JK CPTU MATERIAL - MASTER 31721VJ MASCOT.GPJ <<DrawingFile>> 26/11/2018 11:45 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

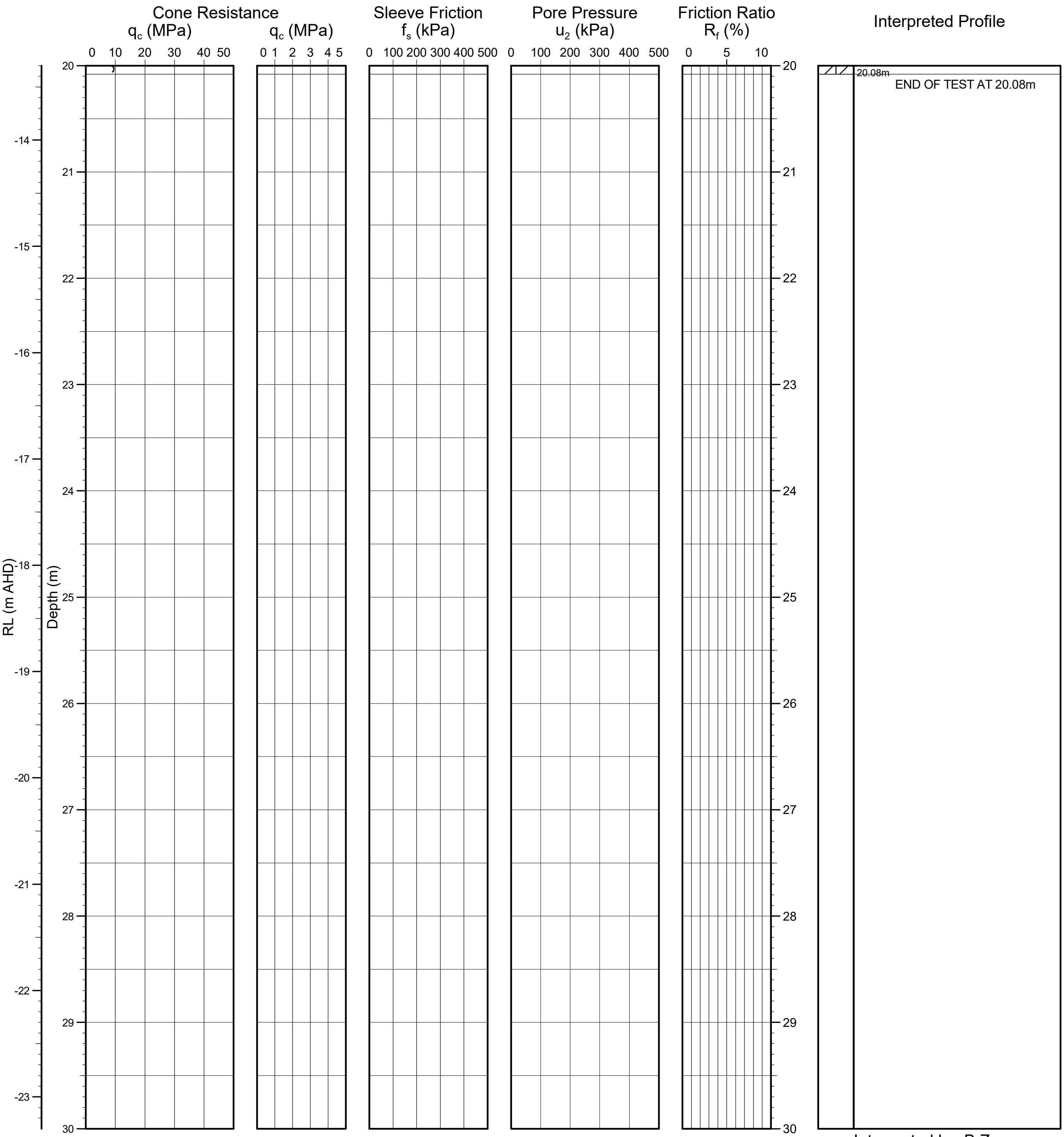
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
6

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~6.7 m	**Data File:** 31721VJ_6.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

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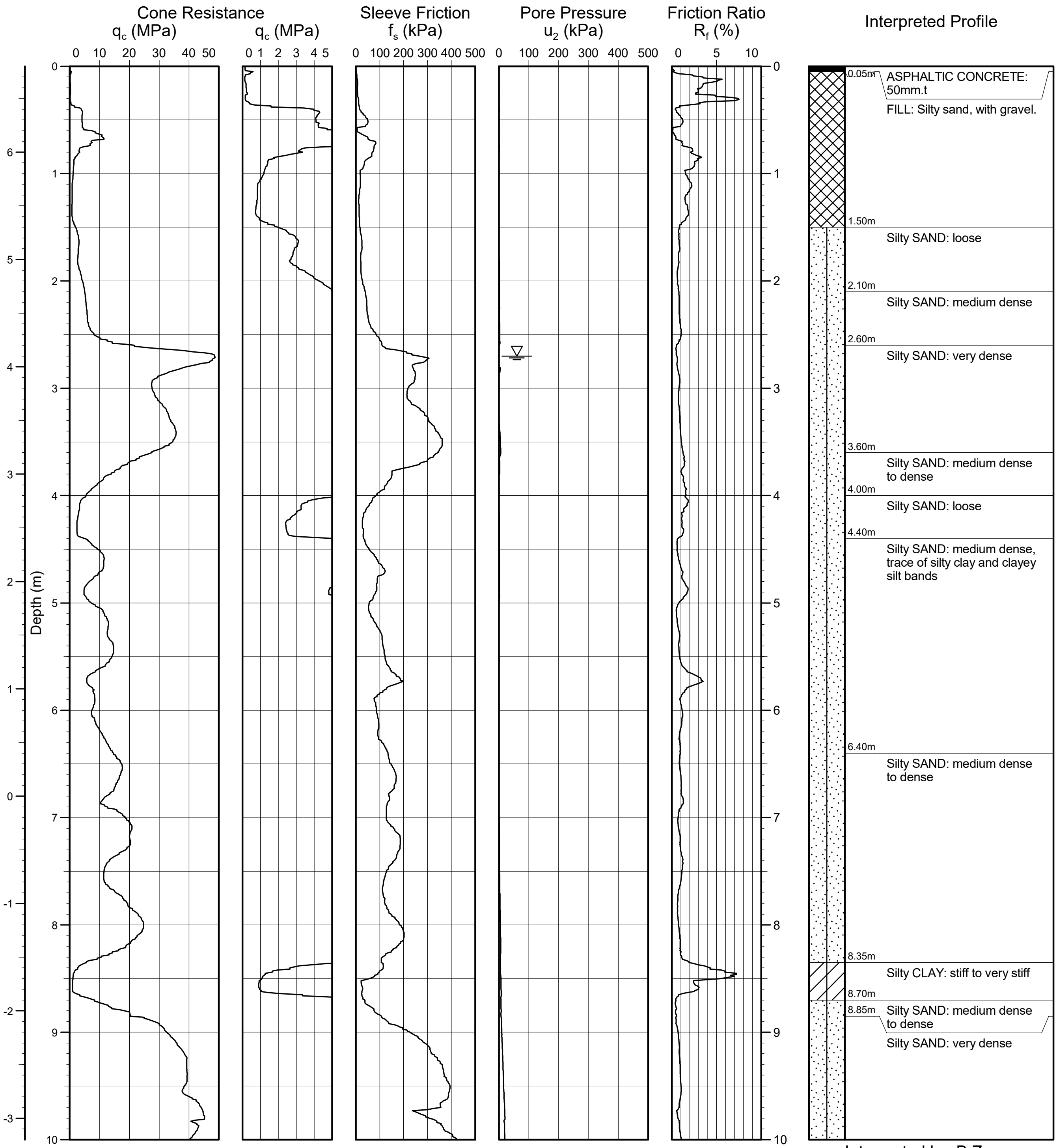
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
7
1 / 3

CONE PENETROMETER TEST RESULTS

Client: ELDESO GROUP
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ **R.L. Surface:** ~6.8 m **Data File:** 31721VJ_7.GEF
Date: 2/11/18 **Datum:** AHD **Operator:** LEON



Interpreted by: B.Z.
Checked by: J.M.

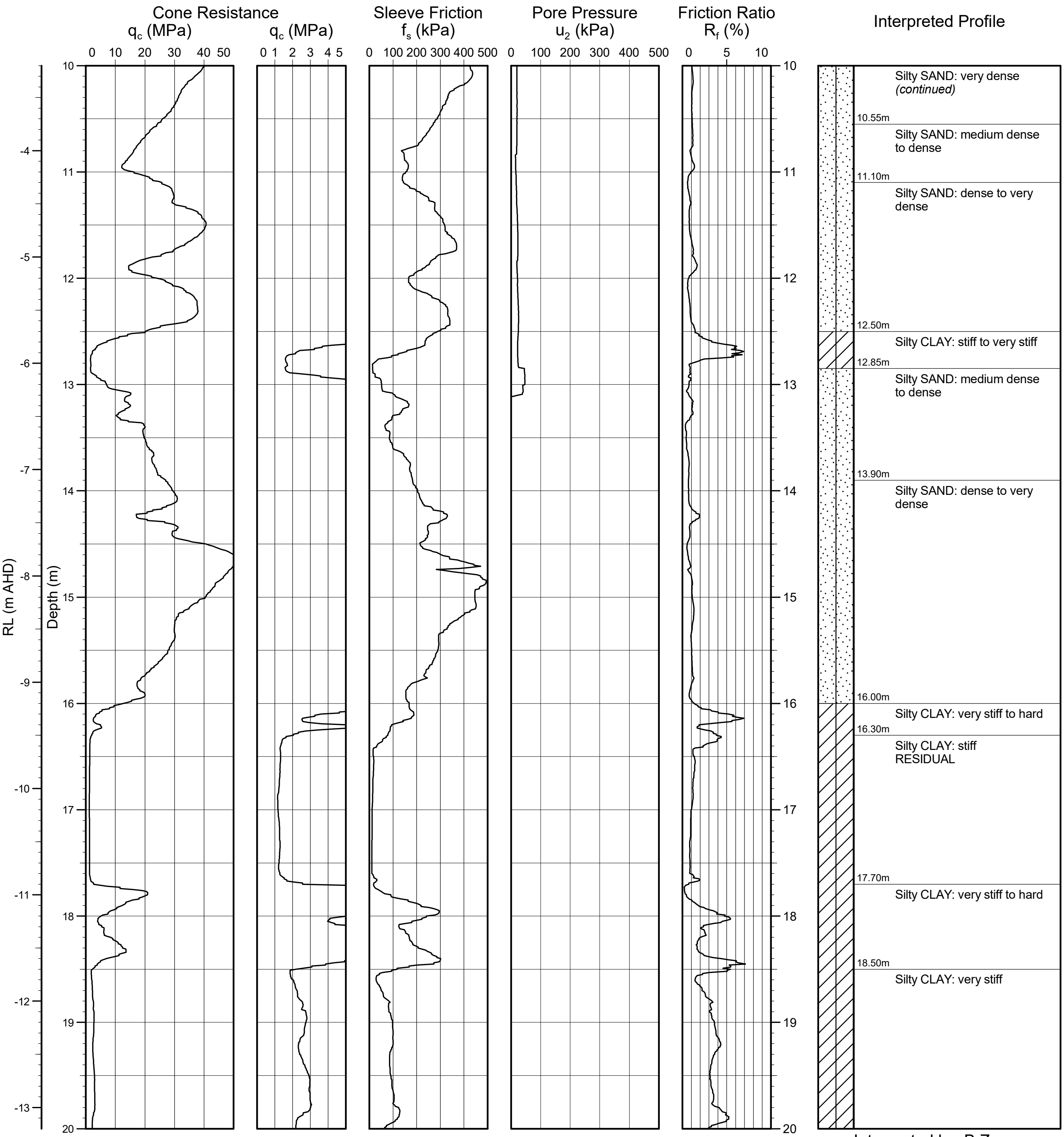
JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CPT No.
7

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW

Job No.: 31721VJ	**R.L. Surface:** ~6.8 m	**Data File:** 31721VJ_7.GEF
Date: 2/11/18	**Datum:** AHD	**Operator:** LEON



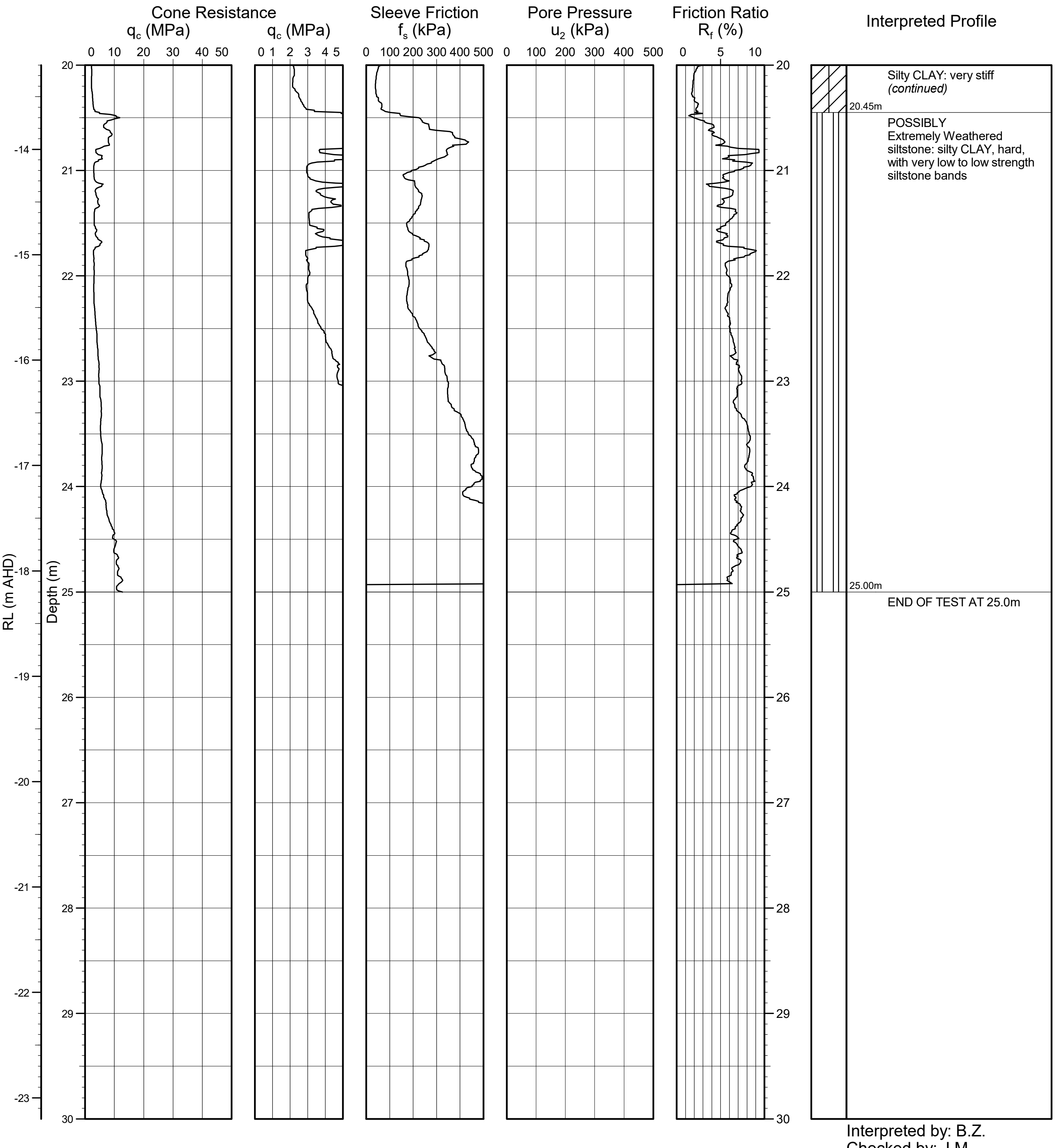
Interpreted by: B.Z.
Checked by: J.M.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

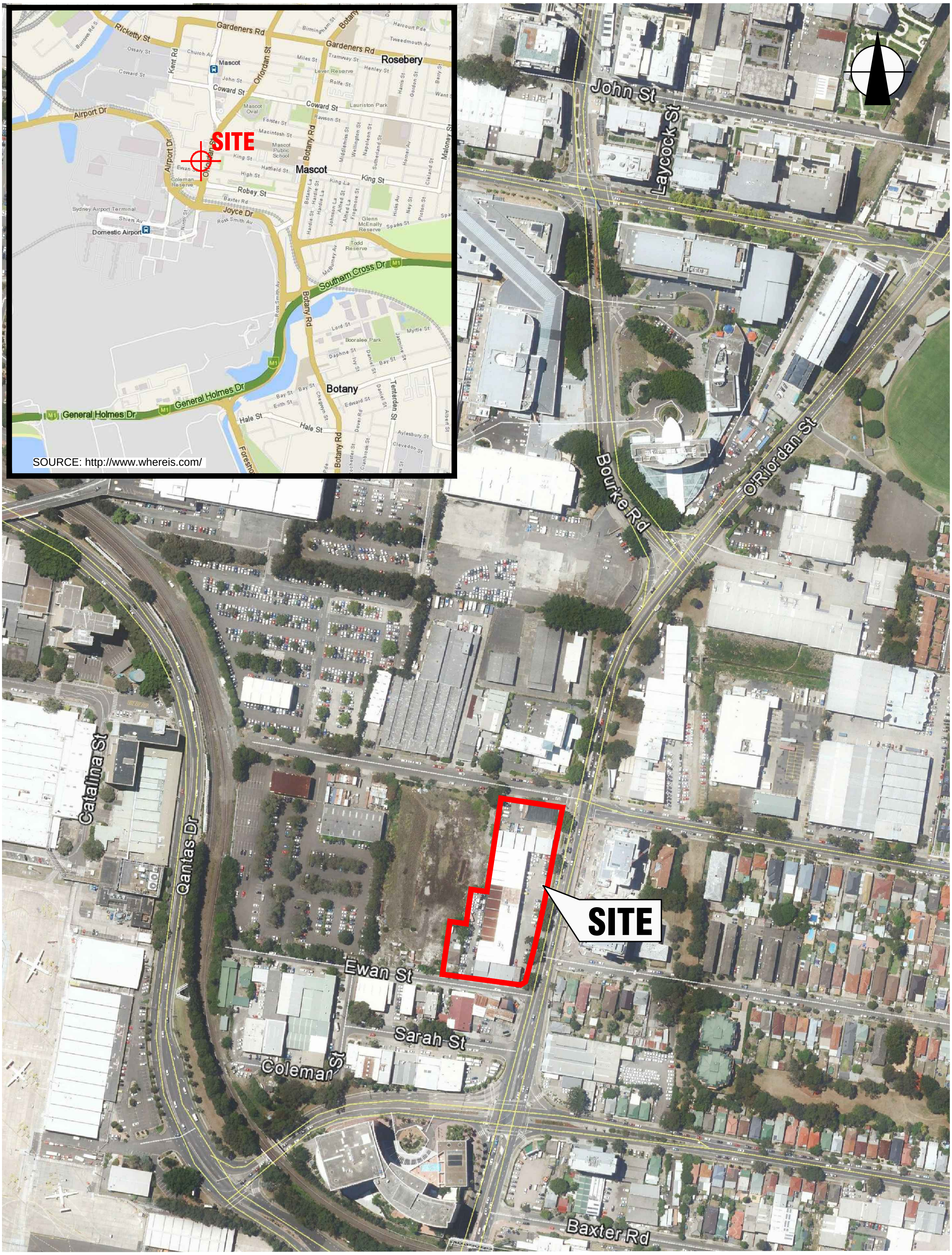
CPT No. 7

CONE PENETROMETER TEST RESULTS

Client:	ELDESO GROUP				
Project:	PROPOSED MIXED USE DEVELOPMENT				
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW				
Job No.:	31721VJ	**R.L. Surface:**	~6.8 m	**Data File:**	31721VJ_7.GEF
Date:	2/11/18	**Datum:**	AHD	**Operator:**	LEON



Interpreted by: B.Z.
Checked by: J.M.



AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title: **SITE LOCATION PLAN**

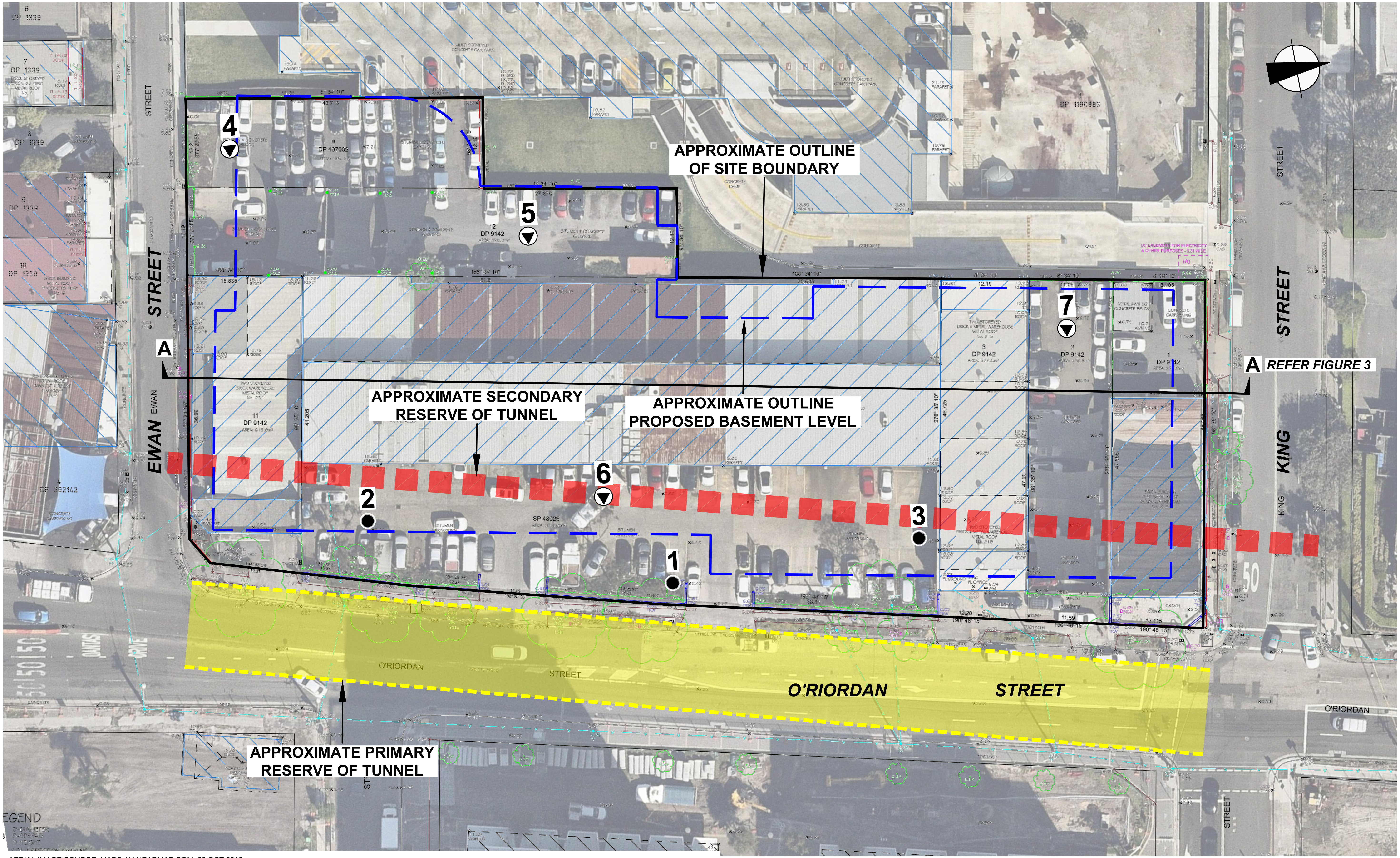
Location: 215-235 O'RIORDAN STREET & 1-3 EWAN STREET MASCOT, NSW

Report No: 31721VJ

Figure No: 1

JK **Geotechnics**

PLOT DATE: 7/11/2018 4:45:44 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31721V MASCOT\CAD\31721VJ.DWG



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 23 OCT 2018.

LEGEND

- ● BOREHOLE
- ▼ ELECTRICAL FRICTION CONE PENETRATION TEST

0 5 10 15 20 25

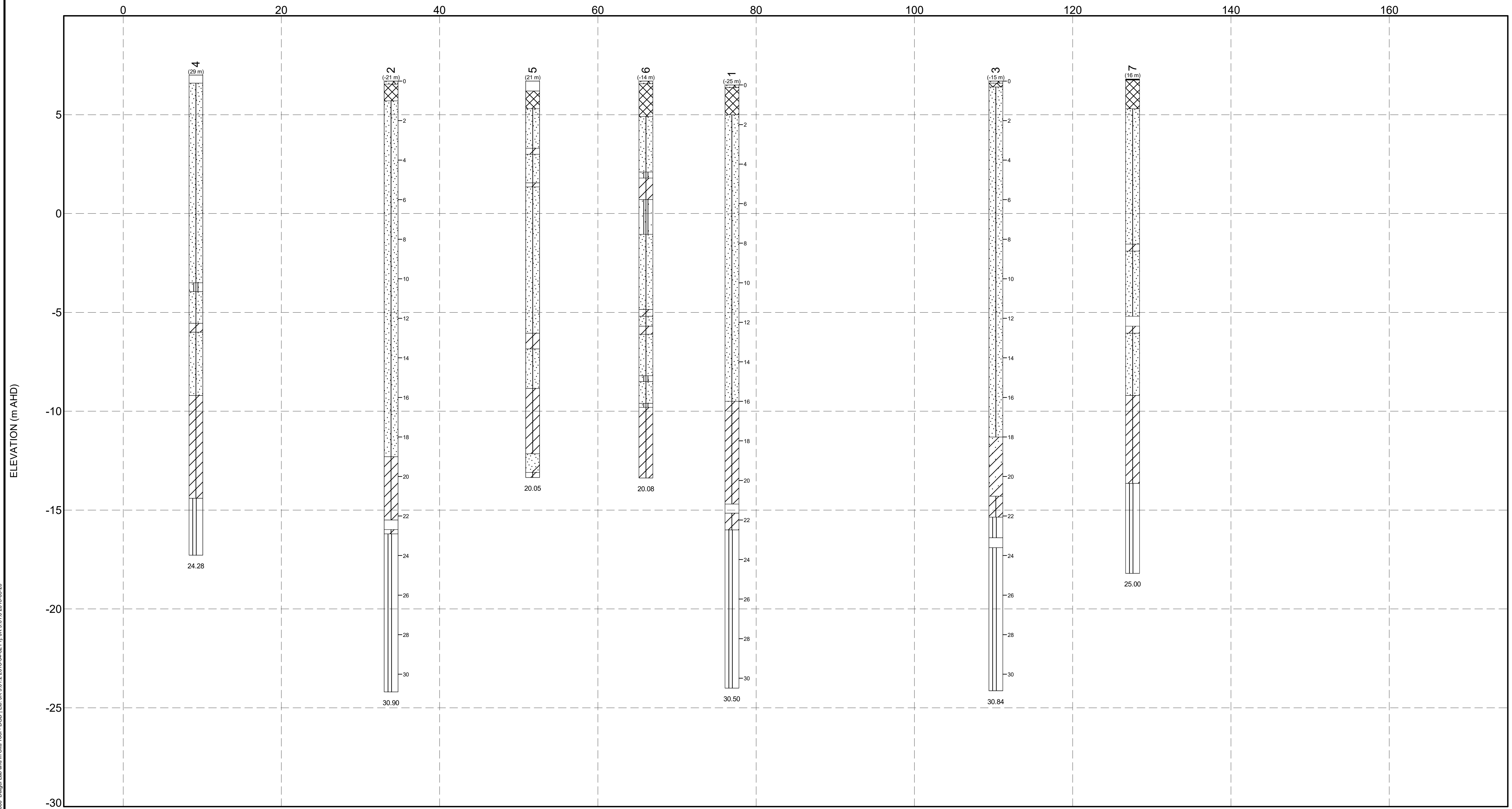
SCALE 1:500 @A3 METRES

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

Title:	TEST LOCATION PLAN
Location:	215-235 O'RIORDAN STREET & 1-3 EWAN STREET MASCOT, NSW
Report No: 31721VJ	Figure No: 2

JK Geotechnics

PLOT DATE: 20/11/2018 10:12:16 AM DWG FILE: S:\6 GEOTECHNICAL JOBS\31000'S\31721V MASCOT\CAD\31721VJ.DWG



MATERIAL GRAPHIC

- ASPHALTIC CONCRETE
- BLANK
- CLAYEY SAND (SC)
- NO CORE
- SANDY CLAY (CL, CI, CH)
- SANDY SILT (ML, MH)
- SILTY CLAY (CL, CI, CH)
- SILTY SAND (SM)
- CONCRETE
- FILL
- SILTSTONE

TITLE: GRAPHICAL INVESTIGATION SUMMARY	DRAWN: D.M.	DATE: 20/11/2018
ELDESO GROUP	CHECKED: J.M.	DATE: 20/11/2018
215-235 O'RIORDAN STREET & 1-3 EWAN PLACE, MASCOT, NSW	SCALE: H 1:500 V 1:200	A3
PROPOSED MIXED USE DEVELOPMENT	PROJECT No: 31721VJ	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:

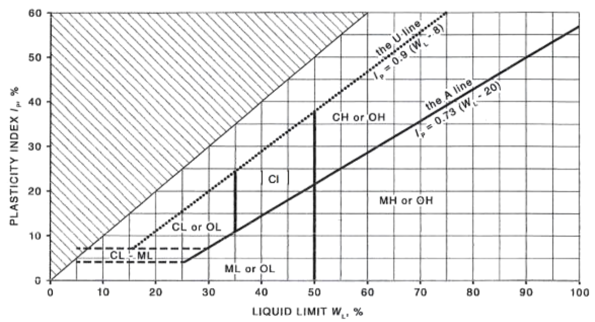
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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 $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)	 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