REPORT TO

ERILYAN PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED GENESISCARE CAMPBELLTOWN

AT

CNR KELLICAR & CAMDEN ROADS,
CAMPBELLTOWN, NSW

Date: 7 October 2020
Ref: 33438Arpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed GenesisCare Campbelltown development, located at the corner of Kellicar and Camden Roads, Campbelltown, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Richard Curtis of Erilyan Pty Ltd in an email dated 17 August 2020. The commission was based on our fee proposal, Ref. P52313A dated 24 July 2020.

We have been provided with the following information:

- 'Geotechnical + Environmental Investigation – Brief + Offer of Service' prepared by Taylor Thomson Whitting (NSW) Pty Ltd [TTW];
- Preliminary architectural drawings prepared by Team 2 Architects (Project No. P936, Drawing Nos. DA-101 to DA-104, Revision 1, dated 17 September 2020).

Based on the supplied information, we understand that a four-storey building, with pile loads up to 5000kN, is proposed. A lift core is proposed towards the north-western side of the new building. We have assumed that the proposed lift pit will require excavation to a maximum depth of 1.5m below existing grade. A new on-grade, asphaltic concrete surfaced car park is also proposed. From an email prepared by Richard Curtis on 21 September 2020, we understand that the short length of Camden Road adjacent to the site has been de-commissioned as a public road and is to be incorporated into the car park. Based on an email from Eirian Crabbe of TTW on 21 September 2020, we understand that the design traffic load for the proposed car park is 1 x 10^4 Equivalent Standard Axles (ESA).

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at ten nominated borehole locations as a basis for comments on mine subsidence, and recommendations on earthworks, retaining walls and lift pit walls, piled footings, soil aggression, earthquake design parameters, and the car park pavement.

This report confirms and amplifies the preliminary advice provided in our letter report, Ref. 33438Alet dated 18 September 2020.

We have been provided with a previous geotechnical investigation report prepared by Geotechnique Pty Ltd; report Ref. 14038/1-AA dated 22 June 2017 [Geotechnique 2017]. The previous investigation included three boreholes and limited laboratory testing. Only one of the three boreholes (BH1) was completed on the subject site. For the purpose of this report, this previous borehole will be referred to as 'G BH1'. G BH1 was drilled and tested to 7.6m depth below existing grade. The bedrock was diamond core drilled for a length of 1.7m. The log for G BH1 is presented in Appendix A, and its location has been plotted onto the attached Figure 2.

This geotechnical investigation was carried out in conjunction with a 'Preliminary (Stage 1) Site Investigation (Contamination Assessment and Waste Classification)' by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref. E33438PLrpt dated 30 September 2020.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 24 & 25 August 2020 and comprised the drilling and testing of ten boreholes (BH1 to BH10), at the locations shown on Figure 2, to depths between 1.5m (BH10) and 10.15m (BH1) below existing surface levels. The boreholes were completed using our track mounted JK308 drill rig, which is equipped for site investigation purposes.

Prior to the commencement of the fieldwork, a specialist sub-consultant reviewed available 'Dial Before You Dig' information, and electro-magnetically scanned the borehole locations for buried services.

The borehole locations were set out using a differential survey system, and the measured grid coordinates and surface RL's are presented on the attached borehole logs. The measured grid coordinates and surface RL's are to Map Grid Australia (MGA) and the Australian Height Datum (AHD), respectively. The order of accuracy in all directions is expected to be within 50mm. An available aerial image forms the basis of Figure 2.

The soil and upper weathered bedrock profiles were spiral auger drilled. The relative compaction/strength of the soil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split-spoon sampler and off the auger (ie. on remoulded samples), and by tactile examination. The strength of the underlying bedrock was assessed by observation of auger penetration resistance when using a twin-pronged tungsten carbide (TC) bit, together with examination of recovered cuttings and correlations with subsequent laboratory moisture content test results.

BH1 and BH4, at depths of 7.25m and 7.12m, respectively, were extended into the bedrock to their final depths by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results.

Groundwater observations were made in the boreholes. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

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

Selected soil and rock cutting samples were returned to a NATA accredited laboratory, Soil Test Services Pty Ltd (STS), for moisture content, Atterberg Limits, linear shrinkage, Standard compaction and four day soaked CBR testing. The results are summarised in the attached STS Tables A & B.

Selected soil samples were returned to another NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH and sulfate testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis 250021'.

The recovered rock cores from BH1 and BH4 were photographed and returned to STS for Point Load Strength Index testing. The rock core photographs are enclosed with the borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and summarised in the attached STS Table C. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in STS Table C.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located towards the toe of a gently sloping hillside, within relatively flat alluvial topography associated with Bow Bowing Creek. The creek is located approximately 100m to the north-west of the site. Within the site, the ground surface generally grades down to the north-west at about 1°. The site, which is triangular in plan area, is bound by Kellicar Road to the south-east, Camden Road to the north, and Narellan Road to the south-west.

At the time of the fieldwork, the site was vacant and grass covered. Scattered medium to large trees were located along the northern site boundary, adjacent to Camden Road. On the south-western side of the site was a chainlink fence, which presumably extended along the south-western boundary. Immediately outside the chainlink fence was a strip of dense bushland, comprising medium to large size tree. Some of these trees were located within the site. Within the strip of bushland immediately beyond the south-western site boundary was a shallow unlined stormwater drain, which contained stagnant water. Beyond the stormwater drain, was the Narellan Road embankment which gradually increased in height in a north-westerly direction. Adjacent to the north-western apex of the site, the embankment was up to approximately 3m high and the maximum grade of the fill batter slope was about 25°.

Kellicar Road to the south-east of the site was surfaced with asphaltic concrete (AC), which appeared to be in good condition based on a cursory inspection. Camden Road was also surfaced with AC, which was in fair to poor condition with crocodile and longitudinal cracking, patchwork repairs, and subsidence of an AC resurface of a trench backfill.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Wollongong-Port Hacking (Geological Survey of NSW, Geological Series Sheet 9029-9129) indicates the site to be underlain by Quaternary alluvium associated with Bow Bowing Creek, then Ashfield Shale of the Wianamatta Group. Generally, the boreholes encountered fill, overlying alluvial clays, then siltstone at moderate depths. Reference should be made to the attached borehole logs for specific details at each location. Graphical borehole summaries for Sections A-A & B-B, as shown on Figure 2, are presented as Figures 3 & 4, respectively. A summary of the encountered subsurface characteristics is provided below:

Fill

Granular unbound roadbase materials were encountered in BH5 and BH6 to 0.08m depth. Clayey fill was encountered below the roadbase in BH5 and from the surface in BH1 to BH4, and BH7 to BH10 to depths generally between 0.2m and 0.5m. In BH5 however, the fill was 1.5m deep. In all boreholes except BH5 and BH6, the fill was grass covered. Inclusions of igneous gravel were found in the fill. In BH2, asbestos fragments were found in the fill. Based on the SPT result and limited hand penetrometer readings, the deeper fill in BH5 was assessed to be poorly compacted.

Alluvial Clays

Alluvial clays comprising predominantly silty clays, and to a lesser extent, sandy clays, were encountered below the fill in all boreholes. The alluvial clays were mostly of high plasticity and of very stiff or hard strength. BH5 to BH10 were terminated within the alluvial clay profile.

Siltstone Bedrock

Siltstone (formerly referred to as shale) bedrock was encountered in BH1 to BH4, and in the previous G BH1, at the depths and RL's tabulated below:

Borehole	Surface RL (mAHD)	Depth to Weathered Bedrock below Ground Surface Level (m)	Approximately Weathered Bedrock Surface RL (mAHD)
BH1	67.85	5.7	62.2
BH2	67.60	6.4	61.2
BH3	68.23	6.8	61.4
BH4	68.07	6.9	61.2
G BH1	67.8	5.8	62.0

In BH1 and the previous G BH1, the siltstone bedrock on first contact was extremely weathered and of hard (soil) strength. The thickness of this 'weak' profile in BH1 and G BH1 was 0.3m and 0.1m, respectively. The underlying bedrock in BH1 and G BH1, and the bedrock from first contact in BH2, BH3 and BH4 was generally moderately weathered, slightly weathered or fresh, and of low, medium or high strength.

From the cored lengths of BH1, BH4 and G BH1, the siltstone profile contained numerous rock defects (ie. clay seams, crushed seams, extremely weathered seams and joints). In BH4, a 310mm thick 'no core' (core loss) zone was encountered at 7.69m depth; presumably a 'weaker' band washed out by the drill flush water.

An indicative engineering classification of the siltstone (shale) bedrock has been carried out for the current boreholes (in accordance with 'Classification of Sandstones and Shales in the Sydney Region: A Forty Year Review' by Pells et al., Australian Geomechanics, June 2019) and is tabulated below. We were unable to complete a rock classification for the previous G BH1 as insufficient information was provided on the borehole log.

Borehole	**Surface RL (mAHD)**	**Indicative Engineering Classification of Siltstone (Shale) Bedrock Depths (m) [Approx. RL at top of Unit (mAHD)]**			
		Class V	**Class IV**	**Class III**	**Class II/I**
BH1	67.85	-	6.0 – 9.1 [61.9]	9.1 – 10.2 [58.8]	-
BH2[1]	67.60	6.4 – 7.5 [61.2]	7.5 – 9.2 [60.1]	-	-
BH3[1]	68.23	6.8 – 7.4 [61.4]	7.4 – 10.0 [60.8]	-	-
BH4	68.07	6.9 – 8.8 [61.2]	-	8.8 – 10.1 [59.3]	-

Note 1: Engineering classification of bedrock estimated from augered boreholes only.

Groundwater

All boreholes were 'dry' during and on completion of auger drilling. In BH2 and BH3, groundwater was measured at depths of 5.4m and 4.9m, respectively, at a short time after completion of auger drilling. In BH1 and BH4, groundwater was measured at depths of 3.3m and 3.1m, respectively, at a short time after completion of rock core drilling, however, we expect that these levels would have been influenced by the introduced drill flush water. In the previous G BH1, groundwater was recorded at 5.0m depth. We note that the groundwater levels would not have stabilised within the limited observation period. No long-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The moisture content and Atterberg Limits test results confirmed our field classification of the site soils. The Atterberg Limits and linear shrinkage test results indicated that the sampled alluvial silty clay of high plasticity from BH1 and BH4 to have a moderate to high potential for shrink-swell reactivity with changes in moisture content.

The four-day soaked CBR tests carried out on a clayey fill sample from BH5 and on alluvial silty clay samples from BH6 to BH10 resulted in values between 4.5% and 9%, when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The samples were compacted prior to CBR testing at close to their Standard Optimum Moisture Contents (SOMC), which were generally within 1.3% of their respective insitu moisture contents, except for the clayey fill sample from BH5 where its SOMC was 8.5% 'dry' of its insitu moisture content. During the four day soaking period, swells of either 0.5% or 1% were measured on the BH6, BH8, BH9 and BH10 clayey samples.

The results of the soil aggression testing are tabulated below:

Borehole	Sample Depth (m)	Soil Description	Soil pH	Soil Sulfate (mg/kg)
BH2	4.2-4.65	Alluvial Silty Clay	8.3	29
BH3	0.5-0.95	Alluvial Silty Clay	7.3	<10
	2.7-3.15	Alluvial Silty Clay	8.0	70
	5.7-6.15	Alluvial Silty Clay	8.6	40
G BH1*	2.5-2.95	Alluvial Silty Clay	6.8	110

* Laboratory test results obtained from Geotechnique 2017.

The results of the moisture content tests carried out on recovered rock cuttings correlated well with our field assessment of bedrock strength. The results of the Point Load Strength Index tests carried out on the recovered rock cores from BH1 and BH4 correlated well with our field assessment of bedrock strength. The estimated UCS's, based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (ie. UCS = 20 x $I_{S(50)}$), generally ranged from 4MPa to 22MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

We consider the following to be the primary geotechnical issues for the proposed GenesisCare Campbelltown development:

- Presence of clayey fill and alluvial silty clays with a moderate to high potential for shrink-swell movements with changes in moisture content.
- Deeply weathered bedrock profile for pile design and construction.
- Low CBR values for the clayey subgrade.

The effects of the above geotechnical issues on design and construction are detailed in the sections which follow.

4.2 Mine Subsidence

Based on the 'South Campbelltown Mine Subsidence District' map (Ref. PP5205), which is available for viewing at the NSW Government Subsidence Advisory website (**www.subsidenceadvisory.nsw.gov.au**), the site is located outside a potential mine subsidence area.

4.3 Earthworks

All earthworks recommendations provided below for the proposed GenesisCare Campbelltown development should be complemented by reference to AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.3.1 Subgrade Drainage

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

4.3.2 Site Preparation

Following demolition and removal of the existing Camden Road pavement, all grass, topsoil, root affected soils and any deleterious fill or contaminated soil should be stripped. Stripped topsoil and root affected soils should be stockpiled separately as they are generally considered unsuitable for reuse as engineered fill. The stripped topsoil and root affected soils may be reused for landscaping purposes subject to confirmation by JKE. Reference should be made to the JKE report for guidance on the offsite disposal of soil.

Care must be taken not to undermine or remove support from the site boundaries during stripping.

We note that the existing trees have likely caused localised 'drying out' of the surrounding clayey soils. Removal of the trees, if required, will lead to the recovery of the soil moisture content, resulting in differential swell movements in the vicinity of the trees and their root systems (which can extend for a significant distance from the trunk). The swell movements generated by the removal of the trees are in addition to the shrink-swell movements which can occur in the clayey soils due to weather related natural moisture changes and by the reduction in surface evaporation subsequent to covering the site with the proposed building and car park pavement.

It is likely that moisture equilibrium in the clayey soils, following removal of the tree stumps and roots, could take at least one to two years to develop. In order to reduce the effects that removal of the trees will have on the proposed development, we strongly recommend they be removed as early as possible ahead of construction. We recommend that all soils located within their primary root structures be boxed out and replaced with engineered fill, as discussed in Section 4.3.4 below.

4.3.3 Subgrade Preparation

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

Subgrade heaving during proof-rolling may occur in areas where the clays have become 'saturated' and/or where under-compacted existing fill exists (eg. in the vicinity of BH5). Small areas can typically be improved by locally removing the heaving/'soft' material to a stable base and replaced with engineered fill, as outlined below. Alternatively, bridging layer support using high tensile geogrids and appropriately sized well graded durable crushed rock could be considered to support the new fill. Other subgrade improvement options, as appropriate, should be provided by the geotechnical engineer following the proof rolling inspection.

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

Engineered fill must be used to raise site levels.

4.3.4 Engineered Fill

General

From a geotechnical perspective, excavated clayey soils are considered suitable for reuse as engineered fill on condition that they are 'clean', free of organic matter, and contain a maximum particle size not exceeding 75mm. Excavated granular materials from the Camden Road pavement should be thoroughly blended with the clayey soils in order to improve the workability of the latter soil type.

Raising of site levels if required, will involve the importation of Virgin Excavated Natural Material (VENM) or Excavated Natural Material (ENM), as approved by JKE. To simplify the earthworks, our preference would be to import a well graded, durable granular material (eg. crushed sandstone), which is free of organic matter and contains a maximum particle size not exceeding 75mm.

Engineered fill comprising site won clayey soils should be compacted in maximum 250mm thick loose layers using a large vibratory pad-foot roller (say, at least 12 tonnes deadweight) to a density ratio of at least 98% of SMDD and at a moisture content within 2% of SOMC. If boxed out material from the vicinity of BH5 is to be reused as engineered fill, then it will need to be dried back in order to meet the moisture specification.

Engineered fill comprising imported crushed sandstone should also be compacted in maximum 250mm thick loose layers using a large vibratory roller to a density ratio of at least 98% of SMDD.

Service Trenches

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

Earthworks Inspection and Testing

Density tests must be carried out on the engineered fill to confirm the above specifications are achieved, as outlined below:

- The frequency of density testing for general engineered fill should be at least one test per layer per 1,000m^2, or one test per 200m^3 distributed reasonably evenly throughout the full depth and area, or 3 tests per visit, whichever requires the most tests (assumes maximum 250mm thick loose layers).
- The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 150mm thick loose layers). This implies that at each test location, two compacted layers will be tested simultaneously.

We consider that Level 2 testing of fill compaction, in accordance with Section 8 of AS3798-2007, is appropriate for this development. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by the GenesisCare (or their representative), and not by the earthworks contractor or service installation sub-contractors.

4.3.5 Lift Pit Excavation

Prior to any excavation commencing, we recommend that reference be made to the NSW Government 'Code of Practice, Excavation Work' dated January 2020.

We have assumed that the proposed lift pit will require excavation to a maximum depth of 1.5m below existing ground surface levels. Excavation of the soil profile can be completed using the bucket of a hydraulic excavator. For stability considerations, the cut faces should be temporarily battered or benched back at an overall grade of no steeper than 1 Vertical (V) on 1 Horizontal (H). Surcharge loads (including plant and stockpile loads) must be kept well back from the crests of the cut faces. Alternatively, vertically cut sides can be appropriately shored using a trench box or an alternative (engineered) temporary shoring system.

Groundwater inflows into the lift pit excavation may occur as local seepage flows at the base of the fill, and/or through gravel bands or fissures within the alluvial silty clays, particularly after heavy rain. Seepage volumes into the excavation are expected to be localised, of limited volume and controllable by conventional sump and pump discharge systems. Discharge from the drainage system should be piped to the stormwater system.

4.4 Retaining Walls and Lift Pit Walls

If any ground level structural elements are to retain a soil profile (eg. perimeter edge beams, loading dock walls, etc.), then they should be designed using a triangular lateral earth pressure distribution, with an 'at-rest' earth pressure coefficient (K_0) of 0.55, assuming a horizontal backfill/retained surface. The lift pit walls should be designed using this K_0 value. A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.

Any surcharge loads affecting the retaining walls (eg. construction traffic, pavement loads, compaction stresses during backfilling, inclined backfill/retained surfaces, etc.) should be allowed in the design using the above K_0 value.

The retaining walls should be designed as permanently drained. Similarly, the lift pit should be designed as externally drained. Subsurface drains behind retaining walls and lift pit walls should incorporate (1) an appropriately sized 'ag' pipe with filter sock, surrounded by (2) free draining, single size, durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate, and encapsulated within (3) a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion. All drainage water should be piped to the stormwater system.

If no drainage is to be provided behind the retaining walls, then they should be designed to withstanding full lateral hydrostatic pressures. Similarly, if the lift pit is not externally drained, then the walls and base should be designed to withstand full lateral and uplift hydrostatic pressures, respectively.

4.5 Footings

4.5.1 Geotechnical Design

Based on the results of the investigation, we recommend that the proposed building be supported on conventional bored piles socketed into the underlying siltstone bedrock. The primary geotechnical issue in relation to pile design is presence of a deeply weathered bedrock profile.

Conventional bored piles socketed at least 0.3m into Class IV or better quality siltstone bedrock may be designed for an allowable end bearing pressure of 1,000kPa. Sockets formed below the minimum 0.3m length requirement may be designed for allowable shaft adhesion values of 100kPa in compression, and 50kPa in tension, on condition that the pile shaft is suitably roughened using a grooving tool fitted to the side of the auger. The provided pressures are based upon serviceability criteria of deflections at the pile toe of less than 1% of the pile diameter. These pile settlements will be of an elastic nature and are expected to occur as construction proceeds.

From BH1 and BH4, Class III siltstone was encountered at approximately 9.0m depth, however it was only proven in each borehole for a 1m length. The Class III siltstone is suitable for an allowable bearing pressure in the order of 3,500kPa, however, two additional cored boreholes would need to be completed to at least 15m depth to confirm the presence of the Class III (or better quality) siltstone at depth. We can provide a fee proposal for this additional work if TTW consider that the higher bearing pressure will greatly optimise pile design.

Due to the presence of groundwater, and medium and high strength siltstone, the prospective piling contractors should be provided with a full copy of this report to ensure that appropriate drill rigs and equipment are brought to site.

We have assumed that the ground floor slab will be movement sensitive and therefore will be fully suspended on the footings. Due to the shrink-swell nature of the clayey soils, we recommend that all ground beams between pile heads and the suspended ground floor slab be poured over void formers. The void formers must be able to accommodate heave movements of 50mm in order to protect the structural elements. A minimum 75mm thick cardboard void former should be detailed in design. If slab-on-grade construction is proposed for the ground floor slab, then further advice should be sought from JK Geotechnics.

All bored piles should be cleaned out, inspected and poured on the same day as drilling. As the piles will be deep and groundwater seepage is expected, the concrete must be tremie poured. All pile holes should be cleaned out using a cleaning bucket (for all pile diameters) for effective removal of loose material and sludge. Due to the expected groundwater seepage, the piles should only be cleaned out when concrete is ready to be tremie poured. For a design bearing pressure of 1,000kPa, the piling should be inspected by a geotechnical engineer and compared to the borehole information during the initial stages and then periodically during the works to confirm that a satisfactory bearing stratum has been achieved. For higher design bearing pressures, full-time inspections will most likely be warranted.

4.5.2 Soil Aggression

The laboratory test results have indicated slightly acidic to alkaline soil conditions, as well as low soil sulfate contents. In accordance with Table 6.4.2(C) of AS2159-2009 ‘Piling – Design and Installation’, the exposure classification to concrete piles is ‘non-aggressive’.

4.5.3 Earthquake Design Parameters

The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 ‘Structural Design Actions, Part 4: Earthquake Actions in Australia’ (including Amendments 1 & 2):

- Hazard Factor (Z) = 0.09 (due to the proximity of the site to Camden)
- Site Subsoil Class = Class C_e

4.6 Car Park Pavement

4.6.1 Design CBR Value

Based on the results of the investigation, including the CBR test results, an established correlation between Plasticity Index, linear shrinkage and CBR, and our experience with similar soils, we recommend that the design of the proposed car park pavement be based on a CBR value of 4% for the compacted clay subgrade.

4.6.2 AC Pavement Layer Thickness Designs

Our recommended AC pavement layer thickness designs (3 options), on the basis of a subgrade CBR value of 4% and a design traffic load of 1 x 10^4 ESA's are provided below. The mechanistic designs have been carried out using the computer based CIRCLY (Version 6.0) program, in accordance with Austroads 2017 'Guide to Pavement Technology Part 2: Pavement Structural Design'. The analyses have taken into account potential failure modes of subgrade rutting and tensile fatigue of the AC wearing course.

Option 1 – Unbound Basecourse & Sub-Base Layers

Pavement Layer	Material Type	Minimum Thickness (mm)
Wearing Course	AC10	40
Basecourse	DGB20	100
Sub-Base	DGS40	185
Total Pavement Thickness		*325*

Option 2 – Unbound Basecourse Layer & Select Subgrade

Pavement Layer	Material Type	Minimum Thickness (mm)
Wearing Course	AC10	40
Basecourse	DGB20	150
Select Subgrade	Crushed Sandstone (soaked CBR ≥ 10%)	160
Total Pavement Thickness		*350*

Option 3 – Unbound Basecourse Layer & Select Subgrade

Pavement Layer	Material Type	Minimum Thickness (mm)
Upper Wearing Course	AC10	40
Basecourse	DGB20	120
Select Subgrade	Crushed Sandstone (soaked CBR ≥ 10%)	200
Total Pavement Thickness		*360*

For the above AC pavement thickness design options, we provide the following additional advice:

1. The upper wearing course must comprise AC10. The AC10 is to be compacted in one lift.
2. A tack coat or a 7mm primer seal is to be used immediately above the basecourse layer.
3. All basecourse materials to comprise DGB20, in accordance with TfNSW QA Specification 3051, and compacted using a large smooth drum roller to a density ratio of at least 98% of Modified Maximum

Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement so as to reduce the potential for material breakdown during compaction.

4. All sub-base materials (ie. Option 1) to comprise DGS40, in accordance with TfNSW QA Specification 3051, and compacted using a large smooth drum roller to a density ratio of at least 95% of MMDD. Adequate moisture conditioning to within 2% of MOMC should be provided during placement.
5. All select subgrade materials (ie. Options 2 & 3) to comprise well graded, durable crushed sandstone with a maximum particle size not exceeding 50mm, and with a four day soaked CBR value of at least 10% when compacted to a density ratio of 100% of SMDD. The crushed sandstone shall be compacted in one lift using a large smooth drum roller to a density ratio of at least 100% of SMDD. Representative soaked CBR test results shall be provided to JK Geotechnics prior to carting to site to confirm the suitability of the material.
6. The tabulated layer thicknesses are minimum thicknesses.
7. The layer thickness designs assume that good surface and subsurface drainage will be provided.

A cost comparison and an assessment of material availability should be made for each option. We can carry out additional layer thickness design iterations based on nominated variables (eg. AC types, bituminous binders, cementing binders, select fill quality, etc.), if commissioned to do so.

4.6.3 Density Testing

Density tests must be carried out immediately after placement and compaction of each granular pavement layer to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 1,000m^2, or three tests per visit, whichever requires the most tests. Level 2 testing of pavement layer compaction is considered appropriate. The geotechnical testing authority (GTA) should be directly engaged by GenesisCare (or their representative).

4.6.4 Subsoil Drains

Subsoil drains should be provided along the edges of the Camden Road pavement and car park pavement, including around any internal landscaping features, with invert levels of at least 200mm below subgrade level. The drainage trenches should be excavated with a uniform and consistent longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.7 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:

1. Additional investigation comprising two cored boreholes to 15m depth in an attempt to optimise pile design, if required.
2. Proof rolling inspections.
3. Insitu density testing of all engineered fill and granular pavement layers by a GTA to Level 2 control.
4. Bored pile inspections.
5. Review of laboratory test information on all crushed sandstone intended to be imported onto site for reuse as a select subgrade.

5 SALINITY

The site is located in an area where soil and groundwater salinity may occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. Campbelltown City Council has guidelines relating to salinity issues which should be checked for relevance to this project.

6 GENERAL COMMENTS

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

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

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed GenesisCare Campbelltown
Location: Cnr Kellicar & Camden Roads, Campbelltown, NSW

Ref No: 33438A
Report: A
Report Date: 21/09/2020
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	0.50 - 0.95	25.3	58	20	38	15.0
1	6.50 - 7.00	5.5	-	-	-	-
2	7.50 - 8.00	3.2	-	-	-	-
3	7.00 - 7.40	5.2	-	-	-	-
3	8.50 - 9.00	4.7	-	-	-	-
3	9.50 - 10.00	3.8	-	-	-	-
4	0.50 - 0.95	21.3	52	17	35	13.5
5	1.50 - 1.95	22.8	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 28/08/2020 & 14/09/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
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the items tested or sampled.

21/09/2020
Authorised Signature / Date
(D. Treweek)

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STS
SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Propose GensisCare Campbelltown
Location: Cnr Kellicar & Camden Roads, Campbelltown, NSW

Ref No: 33438A
Report: B
Report Date: 21/09/2020
Page 1 of 1

BOREHOLE NUMBER	BH 5	BH 6	BH 7	BH 8	BH 9	BH 10
DEPTH (m)	0.20 - 1.00	0.50 - 1.50	0.20 - 1.00	0.40 - 1.50	0.20 - 1.00	0.50 - 1.50
Surcharge (kg)	9.0	9.0	9.0	9.0	9.0	9.0
Maximum Dry Density (t/m^3)	1.88 STD	1.75 STD	1.82 STD	1.67 STD	1.73 STD	1.77 STD
Optimum Moisture Content (%)	15.4	18.3	16.5	21.2	20.9	18.0
Moulded Dry Density (t/m^3)	1.84	1.72	1.78	1.64	1.70	1.72
Sample Density Ratio (%)	98	98	98	98	98	98
Sample Moisture Ratio (%)	99	99	104	100	99	103
Moisture Contents						
Insitu (%)	23.9	19.1	17.0	20.6	20.4	19.3
Moulded (%)	15.3	18.2	17.1	21.1	20.7	18.6
After soaking and After Test, Top 30mm(%)	22.0	23.3	21.5	27.1	25.9	23.2
Remaining Depth (%)	19.2	21.0	18.7	22.5	21.4	19.7
Material Retained on 19mm Sieve (%)	0	0	0	0	0	0
Swell (%)	0.0	0.5	0.0	1.0	1.0	0.5
C.B.R. value: @2.5mm penetration	9	6	5	4.5	6	6

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 28/08/2020 & 14/09/2020.

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Number:1327

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

Approved Signatory / Date
(D. Treweek)
21/9/20

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SOIL TEST SERVICES

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33438A
Project:	Proposed GenesisCare Campbelltown	**Report:**	C
Location:	Cnr Kellicar & Camden Roads, Campbelltown, NSW	**Report Date:**	1/09/2020
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	7.31 - 7.34	0.2	4
	7.81 - 7.85	0.3	6
	8.27 - 8.30	0.6	12
	8.66 - 8.69	0.5	10
	9.04 - 9.07	0.4	8
	9.85 - 9.88	0.3	6
	10.06 - 10.09	0.2	4
4	7.57 - 7.60	0.5	10
	8.00 - 8.03	0.5	10
	8.79 - 8.84	1.1	22
	9.02 - 9.05	1.1	22
	9.60 - 9.63	0.7	14
	10.03 - 10.06	0.6	12

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 $I_{S\ (50)}$

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CERTIFICATE OF ANALYSIS 250021

Client Details

Client	JK Geotechnics
Attention	Kartik Singh
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33438A, Campbelltown**
Number of Samples	4 Soil
Date samples received	28/08/2020
Date completed instructions received	28/08/2020

Analysis Details

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

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

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

Report Details

Date results requested by	04/09/2020
Date of Issue	03/09/2020

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference		250021-1	250021-2	250021-3	250021-4
Your Reference	UNITS	BH2	BH3	BH3	BH3
Depth		4.2-4.65	0.5-0.95	2.7-3.15	5.7-6.15
Date Sampled		24/08/2020	24/08/2020	24/08/2020	24/08/2020
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	02/09/2020	02/09/2020	02/09/2020	02/09/2020
Date analysed	-	02/09/2020	02/09/2020	02/09/2020	02/09/2020
pH 1:5 soil:water	pH Units	8.3	7.3	8.0	8.6
Sulphate, SO4 1:5 soil:water	mg/kg	29	<10	70	40

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			02/09/2020	[NT]	[NT]	[NT]	[NT]	02/09/2020	[NT]
Date analysed	-			02/09/2020	[NT]	[NT]	[NT]	[NT]	02/09/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	109	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

BOREHOLE LOG

Borehole No.
1
1 / 3

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 67.85 m
Date: 25/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	●							▩		FILL: Silty clay, medium plasticity, brown, trace of root fibres and fine grained igneous gravel.	w<PL			GRASS COVER
	●			●	N = 11 4,6,5	67	1	▨	CH	Silty CLAY: high plasticity, orange brown mottled yellow, trace of fine grained ironstone gravel, root fibres and ash.	w<PL	VSt	330 350 360	ALLUVIAL
				●	N = 19 6,10,9	66	2	▨				Hd	480 520 550	
								▨		Silty CLAY: high plasticity, brown mottled grey, trace of fine grained ironstone gravel.				
ON COMPLETION OF CORING AND AFTER 3 HRS				●	N = 14 5,6,8	65	3	▨					590 550 >600	
						64	4	▨		as above, but orange brown mottled grey.				
				●	N = 14 5,6,8	63	5	▨					500 470 580	
				●	N > 10 8,10/ 100mm REFUSAL	62		▥	-	Extremely Weathered siltstone: Silty CLAY, medium plasticity, orange brown and grey, trace fine to medium grained ironstone gravel.	XW	Hd	>600 >600 >600	ASHFIELD SHALE
				●		61	6	▥		SILTSTONE: grey, with extremely weathered bands.	MW	L - M		LOW TO MODERATE 'TC' BIT RESISTANCE WITH VERY LOW BANDS

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33438A CAMPBELLTOWN.GPJ <<DrawingFile>> 18/09/2020 15:53 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

BOREHOLE LOG

Borehole No.
1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A
Method: SPIRAL AUGER
R.L. Surface: 67.85 m
Date: 25/8/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: K.K.S/A.J.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
									-	SILTSTONE: grey, with extremely weathered bands. *(continued)*	MW	L - M		
						60 59 58 57 56 55 54	8 9 10 11 12 13			REFER TO CORED BOREHOLE LOG				

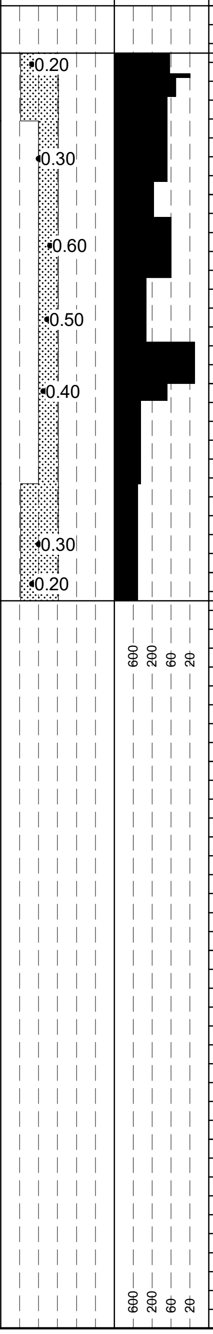
CORED BOREHOLE LOG

Borehole No.
1
3 / 3

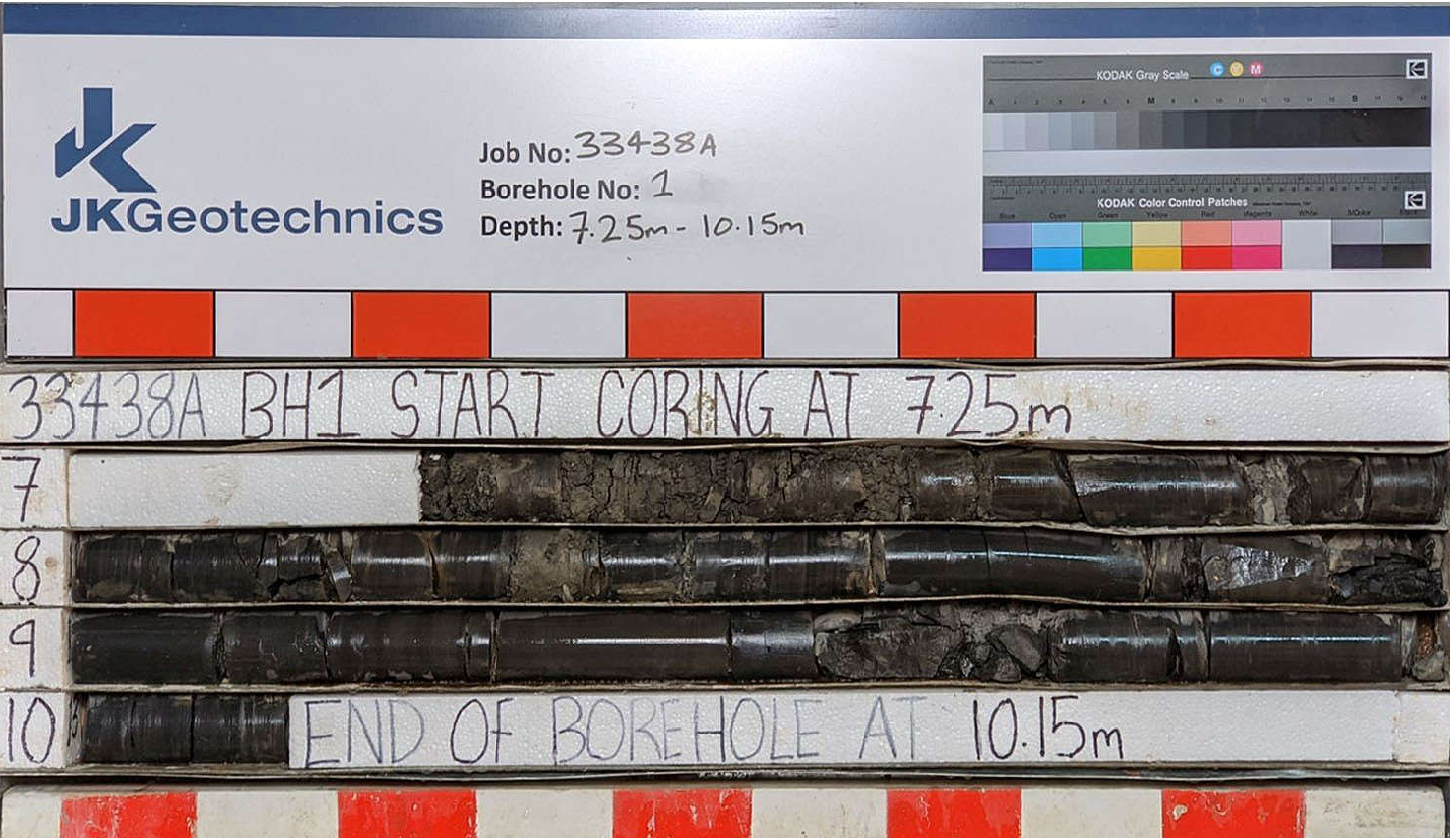
Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A
Core Size: NMLC
R.L. Surface: 67.85 m
Date: 25/8/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.K.S/A.J.

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 DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
					START CORING AT 7.25m					
90% RETURN		60	8		SILTSTONE: grey and brown, bedded at 0-10°.	MW	L - M	0.20	(7.25m) ~~XWS, 0°, 55 mm.t~~ (7.36m) XWS, 0°, 20 mm.t (7.38m) CS, 0°, 35 mm.t (7.42m) XWS, 0°, 60 mm.t (7.48-7.57m) J, 60°, P, R, Cn	Ashfield Shale
					as above, but dark grey with occasional iron indurated bands.	SW	M	0.30	(7.57m) XWS, 0°, 40 mm.t (7.65m) XWS, 0°, 15 mm.t (7.74m) J, 28°, Un, R, Cn (7.86m) XWS, 0°, 40 mm.t (7.93m) Be, 0°, Un, R, Cn (8.12m) J, 35°, C, R, Cn (8.17m) J, 90°, Un, R, Cn	
		59	9		SILTSTONE: dark grey, bedded at 0-10°.	FR		0.60 0.50 0.40	(8.26m) J, 35°, C, R, Fe Sn (8.31m) CS, 0°, 55 mm.t (8.38m) Be, 0°, Un, R, Fe Sn (8.44m) XWS, 0°, 10 mm.t (8.78m) Be, 0°, Un, R, Cn (8.81m) XWS, 0°, 10 mm.t (8.82-9.00m) J, 80 - 90°, Un, R, Cn (9.09m) XWS, 0°, 5 mm.t	
		58	10			SW - FR	L - M	0.30 0.20	(9.53-9.75m) J, 80 - 50°, Un, R, Cn	
		57 56 55 54	11 12 13		END OF BOREHOLE AT 10.15 m					



FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 33438A
Borehole No: 1
Depth: 7.25m - 10.15m

33438A BH1 START CORING AT 7.25m

END OF BOREHOLE AT 10.15m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 67.60 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■			■				⊠		FILL: Silty clay, medium plasticity, dark grey and brown, trace of root fibres, fine grained sand and asbestos fragments.	w~PL			GRASS COVER SCREEN: 0-0.5m 10.02kg F1 at 0-0.5m
	■			■	N = 9 4,4,5	67		▨	CH	Silty CLAY: high plasticity, orange brown mottled yellow, trace of fine grained ironstone gravel.	w<PL	VSt	350 390 360	ALLUVIAL
							1	▨						
	■			■	N = 13 5,6,7	66		▨		as above, but trace of fine grained sand and ash.			310 370 360	
							2	▨						
						65		▨		Silty CLAY: high plasticity, orange brown mottled yellow and grey, trace of fine to medium grained ironstone gravel, fine grained sand and ash.		Hd		
				■	N = 27 8,12,15		3	▨					>600 >600 >600	
						64	4	▨						
								▨		as above, but without fine grained sand.				
				■	N = 29 8,14,15	63		▨					420 >600 580	
AFTER 3.75HRS ▽						62	5	▨						
				■	N = 24 5,10,14		6	▨						
						61		▥		SILTSTONE: grey.	MW	L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH VERY LOW BANDS
								▥				L - M		

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BOREHOLE LOG

Borehole No.
2
2 / 2

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 67.60 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
										SILTSTONE: grey. *(continued)*	MW	L - M		LOW TO MODERATE RESISTANCE
				■		60 59	8 9				SW - FR	H		HIGH RESISTANCE
						58 57 56 55 54	10 11 12 13			END OF BOREHOLE AT 9.20 m				'TC' BIT REFUSAL

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BOREHOLE LOG

Borehole No.
3
1 / 2

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 68.23 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■			■		68				FILL: Silty clay, medium plasticity, brown, trace of root fibres.				GRASS COVER SCREEN: 0-0.4m 10.64kg NO FCF
	■			■	N = 9 5,4,5				CH	Silty CLAY: high plasticity, orange brown mottled yellow, trace of fine grained ironstone gravel and root fibres.	w<PL	VSt	330 350 380	ALLUVIAL
						67	1			as above, but without root fibres.				
	■			■	N = 17 3,8,9							Hd	510 540 >600	
						66	2							
										Silty CLAY: high plasticity, orange brown mottled yellow and grey, trace of fine grained ironstone gravel.				
				■	N = 20 7,8,12	65	3						>600 >600 >600	
						64	4							
				■	N = 17 5,7,10								>600 >600 540	
AFTER 2 HRS ▽						63	5							
				■	N = 15 5,6,9	62	6						420 440 440	
										SILTSTONE: grey.	SW	M		ASHFIELD SHALE

BOREHOLE LOG

Borehole No.
3
2 / 2

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 68.23 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■		61				SILTSTONE: grey. *(continued)*	SW	M		ASHFIELD SHALE LOW TO MODERATE 'TC' BIT RESISTANCE
				■		60	8				SW - FR	M - H		MODERATE TO HIGH RESISTANCE
				■		59	9					H		HIGH RESISTANCE
						58 57 56 55	10 11 12 13			END OF BOREHOLE AT 10.00 m				

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BOREHOLE LOG

Borehole No.
4
1 / 3

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 68.07 m
Date: 25/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■ ■			■		68	0			FILL: Silty clay, medium plasticity, brown, trace of root fibres, fine grained igneous and ironstone gravel.	w<PL			GRASS COVER
				■	N = 9 3,4,5	67	1		CH	Silty CLAY: high plasticity, orange brown mottled grey, trace of root fibres and fine grained ironstone gravel.	w<PL	VSt - Hd	350 480 450	ALLUVIAL
				■	N = 13 5,6,7	66	2		CL	Sandy CLAY: low plasticity, orange brown, fine grained sand, trace of fine grained ironstone gravel and ash.		Hd	550 460 500	
▽ ON COMPLETION OF CORING				■ ■	N = 27 10,13,14	65	3		CH	Silty CLAY: high plasticity, orange brown, trace of fine grained ironstone gravel.			450 480	HP READINGS ON DISTURBED AUGER SAMPLE
				■	N = 13 4,5,8	64 63	4 5		CI-CH	Silty CLAY: medium to high plasticity, orange brown mottled grey, trace of fine grained ironstone gravel.	w~PL	VSt	330 350 360	
				■ ■	N = 22 6,9,13	62	6			as above, but trace of fine to medium grained ironstone gravel.			300 280 350	

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BOREHOLE LOG

Borehole No.
4
2 / 3

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 68.07 m
Date: 25/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

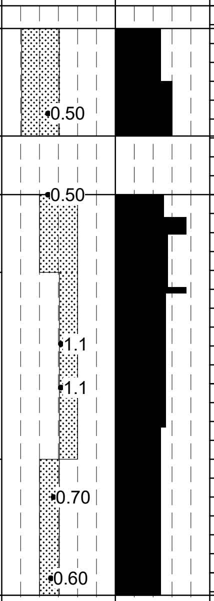
Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
				■		61		▥		SILTSTONE: grey and brown. *(continued)*	MW	L - M		ASHFIELD SHALE LOW TO MODERATE 'TC' BIT RESISTANCE
										REFER TO CORED BOREHOLE LOG				
							8							
						60								
							9							
						59								
							10							
						58								
							11							
						57								
							12							
						56								
							13							
						55								

CORED BOREHOLE LOG

Borehole No.
4
3 / 3

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

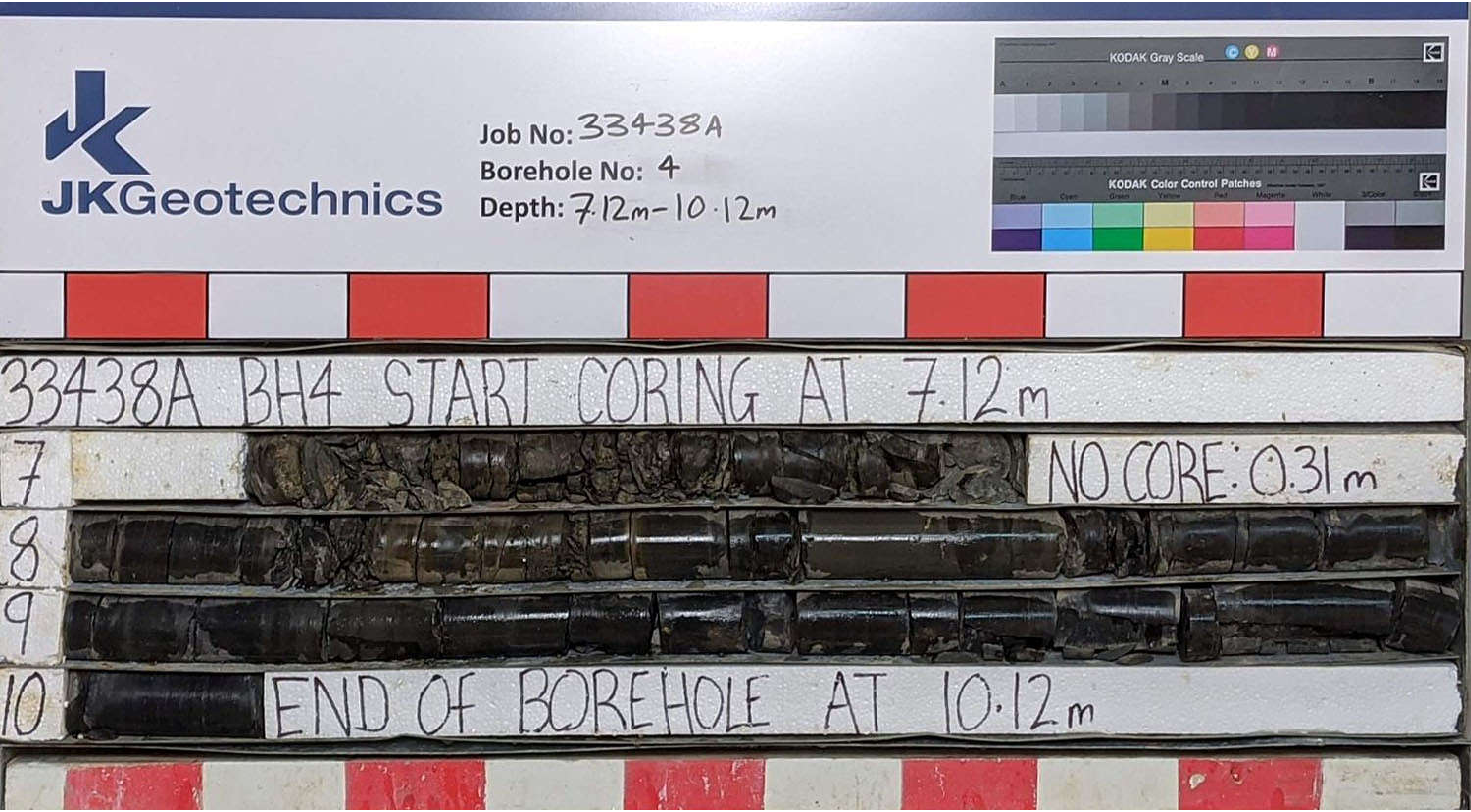
Job No.: 33438A
Core Size: NMLC
R.L. Surface: 68.07 m
Date: 25/8/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: K.K.S/A.J.



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 START CORING AT 7.12m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
75% RETURN		61			SILTSTONE: grey and brown, bedded at 0-10°.	MW	L - M	0.50	(7.18m) XWS, 0°, 100 mm.t (7.33-7.47m) J, 90°, Un, R, Clay FILLED, 2 mm.t (7.47m) XWS, 0°, 20 mm.t (7.52m) CS, 0°, 5 mm.t (7.54-7.62m) Jx 2, 75°, P, R, Clay FILLED, 2 mm.t (7.62m) Cr, 0°, 70 mm.t	Ashfield Shale
					NO CORE 0.31m					
		60	8		SILTSTONE: dark grey, with occasional iron indurated bands, bedded at 0-10°.	SW	M - H	0.50	(8.12m) Be, 0°, Un, R, Cn (8.14m) XWS, 0°, 30 mm.t (8.18m) J, 90°, St, R, Cn (8.21m) XWS, 0°, 100 mm.t	Ashfield Shale
		59	9		as above, but without iron indurated bands.	FR	H	1.1 1.1	(8.36m) CS, 0°, 20 mm.t (8.41m) Be, 0°, P, R, Fe Sn (8.49m) Be, 0°, Un, R, Cn (8.52m) XWS, 0°, 5 mm.t (8.65m) Ji, 30°, Un (8.72m) Be, 0°, Un, R, Cn (8.75m) XWS, 0°, 30 mm.t (8.90m) Be, 0°, P, R, Cn (9.09m) XWS, 0°, 5 mm.t (9.18-9.27m) Ji, 90°, Un	
		58	10				M	0.70 0.60	(9.37m) J, 90°, Un, R, Cn (9.48m) J, 15°, Un, R, Cn (9.63-9.79m) Jx 2, 80 - 90°, Un, R, Cn (9.81-9.95m) J, 80 - 90°, Un, R, Cn	
		57	11		END OF BOREHOLE AT 10.12 m					
		56	12							
		55	13							

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33438A CAMPBELLTOWN.GPJ <<DrawingFile>> 18/09/2020 15:53 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 33438A

Borehole No: 4

Depth: 7.12m–10.12m

33438A BH4 START CORING AT 7.12m

7

NO CORE: 0.31m

8

9

10

END OF BOREHOLE AT 10.12m

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BOREHOLE LOG

Borehole No.
5
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A
Method: SPIRAL AUGER
R.L. Surface: 67.15 m
Date: 24/8/20
Datum: AHD
Plant Type: JK308
Logged/Checked By: K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION						67				FILL: Sandy gravel, fine to coarse grained, igneous, grey, fine to coarse grained sand.	M			ROADBASE
			■							FILL: Silty clay, medium plasticity, orange brown mottled yellow, trace of fine to medium grained sand, fine grained ironstone gravel and ash.	w>PL			APPEARS POORLY COMPACTED
			■	■	N=0 0,0,0								40 40 90	
						66	1							
				■	N = 9 4,4,5				CH	Silty CLAY: high plasticity, orange brown mottled yellow, trace of fine to medium grained sand.	w>PL	St - VSt	250 140 220	ALLUVIAL
						65	2			END OF BOREHOLE AT 1.95 m				

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BOREHOLE LOG

Borehole No.
6
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 66.99 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				■				▨		FILL: Sandy gravel, fine to coarse grained, igneous, grey, fine to coarse grained sand.	M			ROADBASE
				■				▨	CI-CH	Silty CLAY: medium to high plasticity, orange brown mottled yellow, trace of fine grained sand, fine grained ironstone gravel and ash.	w<PL	Hd		ALLUVIAL
			■	■	N = 12 4,5,7			▨					460 510 >600	
			■			66	1	▨						
				■	N = 20 5,9,11			▨					550 570 580	
						65	2			END OF BOREHOLE AT 1.95 m				
						64	3							
						63	4							
						62	5							
						61	6							

BOREHOLE LOG

Borehole No.
7
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 66.73 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■							☒		FILL: Silty clay, medium plasticity, brown, trace of root fibres.	W			GRASS COVER
	■		■	■				▨	CI-CH	Silty CLAY: medium to high plasticity, orange brown, trace of fine grained sand, and fine grained ironstone gravel.	w~PL	VSt - Hd		ALLUVIAL
			■	■	N = 10 4,5,5	66		▨					350 440 490	
							1	▨						
				■	N = 20 5,8,12	65		▨		as above, but orange brown mottled yellow and grey.		Hd	410 450 430	
						64 63 62 61 60	2 3 4 5 6			END OF BOREHOLE AT 1.95 m				

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BOREHOLE LOG

Borehole No.
8
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 67.35 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			67				FILL: Silty clay, medium plasticity, brown, trace of fine grained sub angular igneous gravel, and root fibres.	w<PL			GRASS COVER SCREEN: 0-0.4m 9.68kg NO FCF
		N = 8 3,4,4 N = 18 6,8,10	66	1		CH	Silty CLAY: high plasticity, orange brown mottled grey, trace of fine grained ironstone gravel and fine grained sand.	w<PL	Hd	420 430 450 500 410 450	ALLUVIAL
			65 64 63 62 61	2 3 4 5 6			END OF BOREHOLE AT 1.95 m				

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BOREHOLE LOG

Borehole No.
9
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 67.48 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION	■			■						FILL: Silty clay, medium plasticity, brown, trace of fine grained, igneous gravel and root fibres.				GRASS COVER
	■		■	■	N = 11 3,5,6	67	1		CH	Silty CLAY: high plasticity, orange brown mottled grey, trace of root fibres and ash.	w<PL	Hd	470 450 500	ALLUVIAL
	■			■	N = 17 3,7,10	66				as above, but trace of fine grained sand.		VSt - Hd	350 410 480	
						65 64 63 62 61	2 3 4 5 6			END OF BOREHOLE AT 1.95 m				

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33438A CAMPBELLTOWN.GPJ <<DrawingFile>> 18/09/2020 15:54 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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BOREHOLE LOG

Borehole No.
10
1 / 1

Client: ERILYAN
Project: PROPOSED GENESISCARE CAMPBELLTOWN
Location: CNR KELLICAR & CAMDEN ROADS, CAMBELLTOWN, NSW

Job No.: 33438A **Method:** SPIRAL AUGER **R.L. Surface:** 68.27 m
Date: 24/8/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** K.K.S/A.J.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION	■					68		▩		FILL: Silty clay, medium plasticity, brown, trace of root fibres.	w<PL			GRASS COVER
	■		■	■	N = 9 5,4,5	67	1	▨	CH	Silty CLAY: high plasticity, orange brown mottled yellow, trace of fine grained ironstone gravel.	w<PL	Hd	530 510 550	ALLUVIAL
						66 65 64 63 62	2 3 4 5 6			END OF BOREHOLE AT 1.50 m				

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AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:
SITE LOCATION PLAN

Location: CNR OF KELLICAR AND CAMDEN ROADS,
CAMPBELLTOWN, NSW

Report No: 33438A

Figure No: 1

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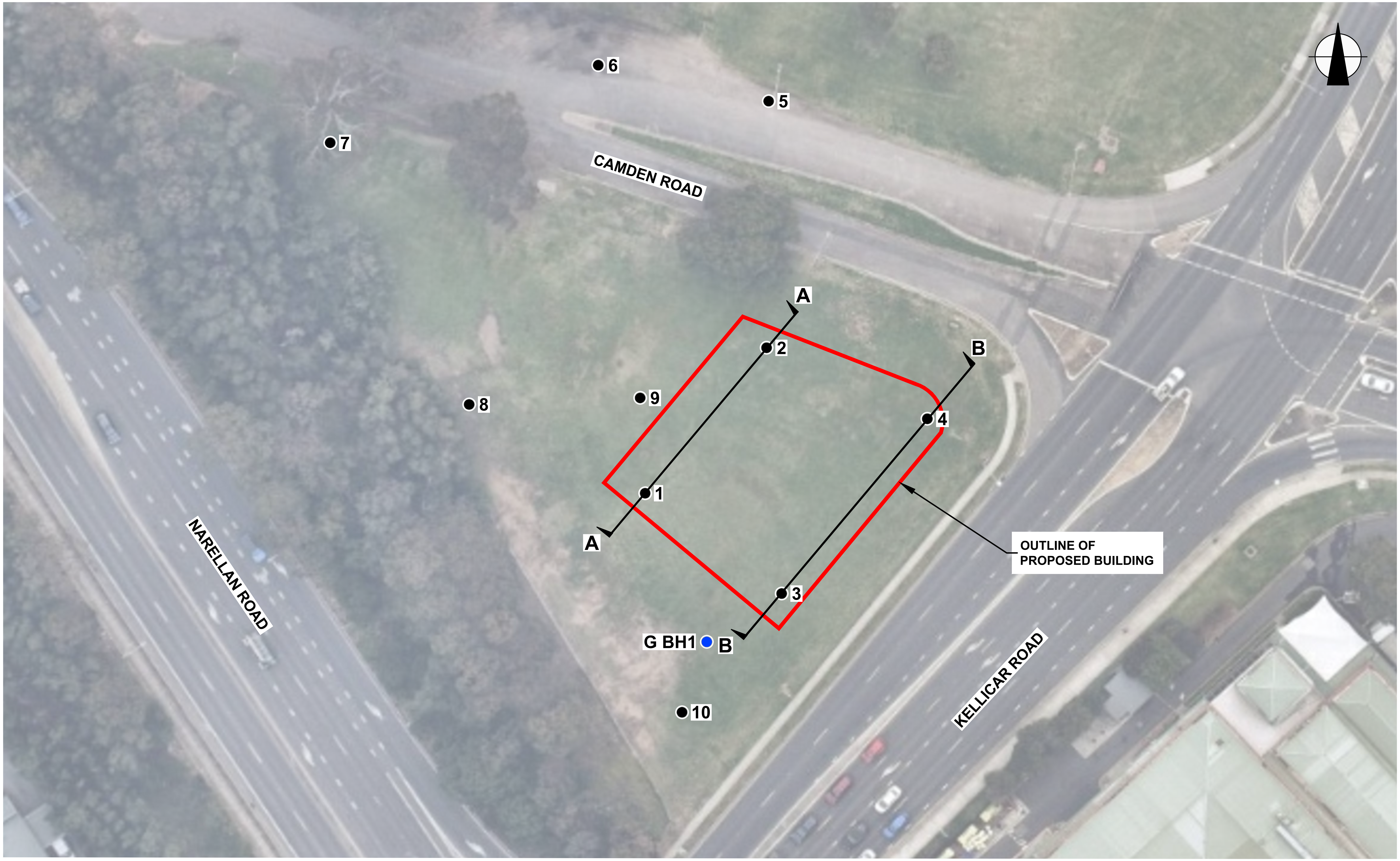
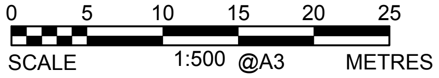
PLOT DATE: 18/09/2020 3:39:54 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33438A CAMPBELLTOWN\CAD\33438A.DWG



NOTES:
1. BOREHOLES 1 TO 10 ARE FROM THE CURRENT INVESTIGATION.
2. G BH1 IS FROM A PREVIOUS GEOTECHNICAL INVESTIGATION BY GEOTECHNIQUE PTY LTD (REPORT Ref:14038/1-AA, DATED 22 JUNE 2017).

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM



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

Title: **BOREHOLE LOCATION PLAN**

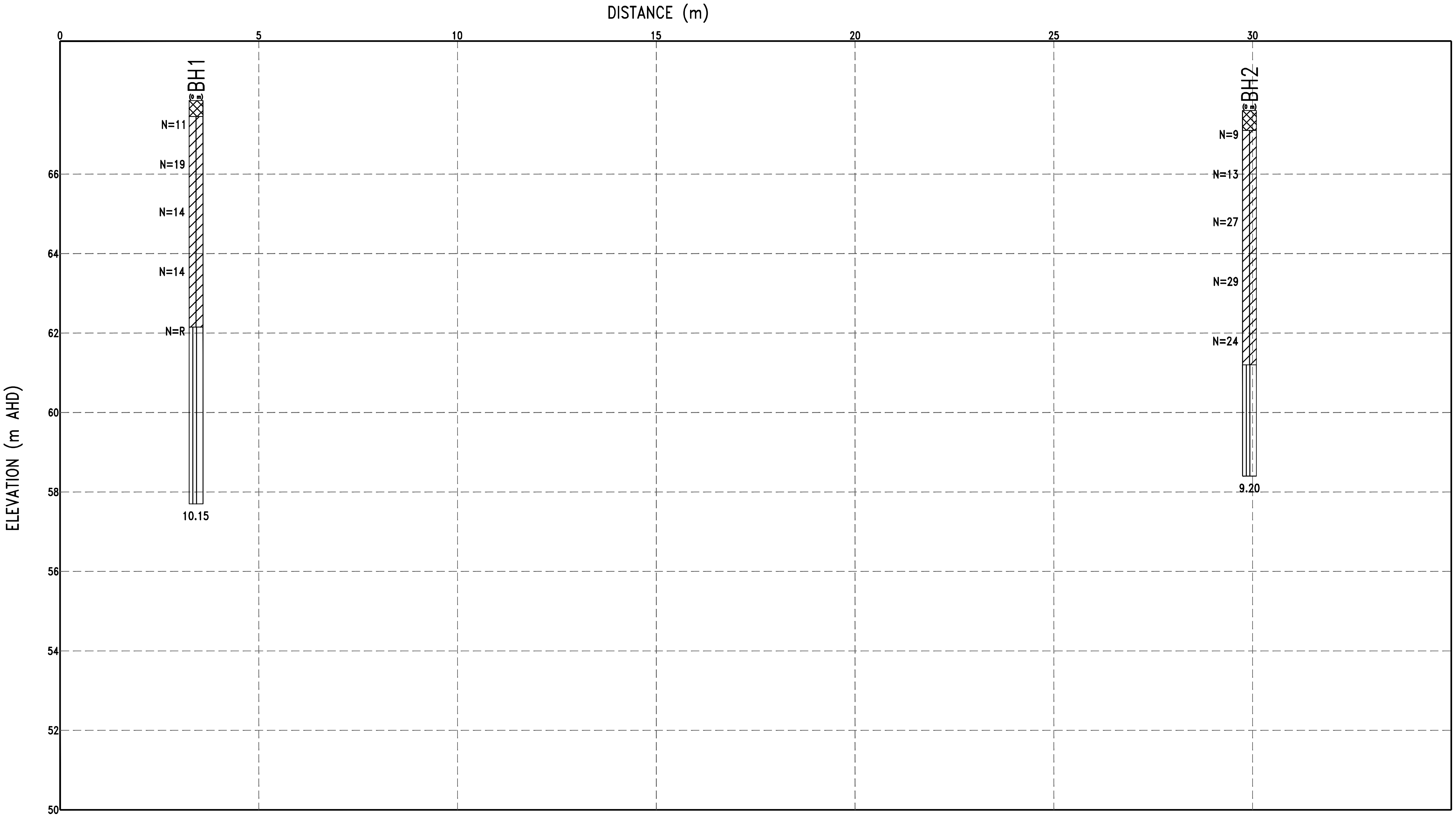
Location: CNR OF KELLICAR AND CAMDEN ROADS, CAMPBELLTOWN, NSW

Report No: 33438A

Figure No: 2

JKGeotechnics

PLOT DATE: 21/09/2020 9:21:18 AM DWG FILE: Y:\33000'S\33438A CAMPBELLTOWN\CAD\33438A.DWG



MATERIAL GRAPHIC

▨ SILTY CLAY (CL, CI, CH)

▩ FILL

▥ SILTSTONE

0 1 2 3 4 5

SCALE 1:100 @A3 METRES

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

Title: **SECTION A-A**
GRAPHICAL BOREHOLE SUMMARY

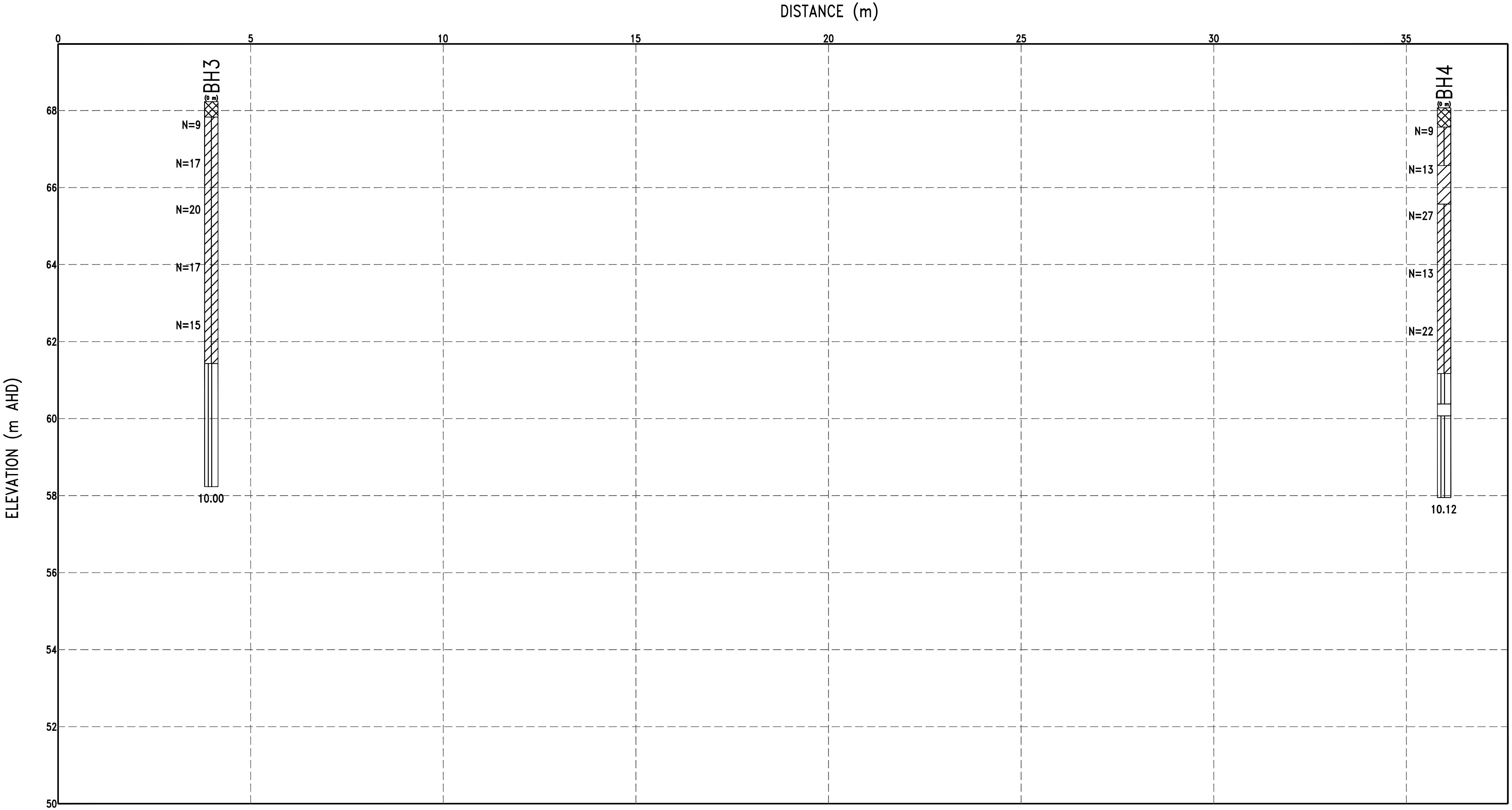
Location: CNR OF KELLICAR AND CAMDEN ROADS,
CAMPBELLTOWN, NSW

Report No: 33438A

Figure No: 3

JKGeotechnics

PLOT DATE: 18/09/2020 3:36:12 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33438A CAMPBELLTOWN\CAD\33438A.DWG



MATERIAL GRAPHIC

- ☐ NO CORE
- ▨ SANDY CLAY (CL, CI, CH)
- ▨ SILTY CLAY (CL, CI, CH)
- ☒ FILL
- ▥ SILTSTONE

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title:
SECTION B-B
GRAPHICAL BOREHOLE SUMMARY

Location: CNR OF KELLICAR AND CAMDEN ROADS,
CAMPBELLTOWN, NSW

Report No: 33438A

Figure No: 4

JKGeotechnics

PLOT DATE: 18/09/2020 3:36:25 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33438A CAMPBELLTOWN\CAD\33438A.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed <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:

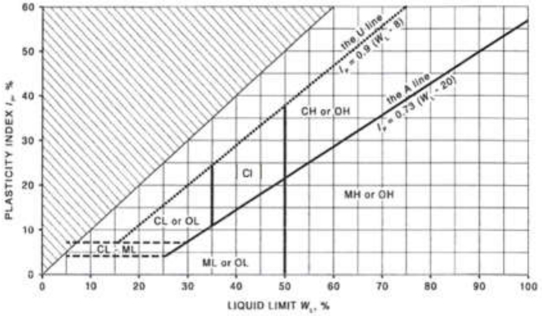
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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	▼ C ▶	Standing water level. Time delay following completion of drilling/excavation may be shown. 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) (Coarse Grained Soils)	$w > \mathrm{PL}$ $w \approx \mathrm{PL}$ $w < \mathrm{PL}$ $w \approx \mathrm{LL}$ $w > \mathrm{LL}$ D M W	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. 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

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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

APPENDIX A

Borehole Log 1 (G BH1) from a Previous Geotechnical Investigation completed by Geotechnique Pty Ltd (Report Ref. 14038/1-AA dated 22 June 2017)

engineering log - borehole

Client : Campbelltown City Council
Project : Possible Commerical Development
Location : Corner Kellicar and Camden Roads
Campbelltown

Job No. : 14038/1
Borehole No. : 1
Date : 06/06/2017
Logged/Checked by: MT

drill model and mounting : Commacchio MC200 Truck Mounted **slope :** deg. **R.L. surface :** ≅67.8
hole diameter : 125 mm **bearing :** deg. **datum :** AHD

method	groundwater	env samples	PID reading (ppm)	geo samples	field test	depth or R.L. in meters	graphic log	classification symbol	MATERIAL DESCRIPTION soil type, plasticity or particle characteristic, colour, secondary and minor components.	moisture condition	consistency density index	hand penetrometer kPa	Remarks and additional observations
						0			TOPSOIL: Silty Clay, low plasticity, brown, with some grass roots				
								CI-CH	Silty CLAY, medium to high plasticity, brown orange	M<PL	St-VSt		Alluvium
					N=19 5,8,11	1							
						2							
					N=15 4,6,9			CI-CH	Silty CLAY, medium to high plasticity, yellow brown	M<PL	VSt		
						3							
					N=18 4,7,11	4							
	▼					5							Groundwater at 5.0m
					4,6, 20/50								
									SHALE, grey, extremely weathered, low strength				Extremely weathered bedrock
						6			Started coring at 5.9m				
						7							
						8							
						9							

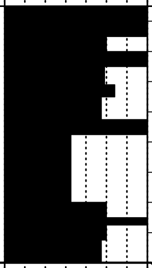
engineering log
cored borehole

Client : Campbelltown City Council
Project : Possible Commerical Development
Location : Corner Kellicar and Camden Roads
Campbelltown

Job No. : 14038/1
Borehole No. : 1
Date : 06/06/2017
Logged/Checked by : MT

drill model and mounting : Commacchio MC200 Truck
slope : **deg.**
R.L. surface : ≅67.8
core size: NMLC
bearing : **deg.**
datum : AHD

barrel lift	water loss/level	depth of R.L. in meters	graphic log	CORE DESCRIPTION rock type, grain characteristics, colour, structure, minor components.	weathering	strength	point load index strength $I_{S}(50)$ EL VL L M H VH	DEFECT DETAILS defect spacing (mm) 2000 1000 500 300 100 50	DEFECT DETAILS DESCRIPTION type, inclination, thickness, planarity, roughness, coating. Specific	General
				Started coring at 5.9m						
		6 7		SHALE, grey, thinly laminated, with some clay bands	DW-SW	M	×		6.0m, Crushed Seam 6.2m, Infilled Seam 6.4m, Bp=0° planar 6.55m,Jo=25° plannar 6.7m, Infilled Seam 6.8m, Infilled Seam 7.2m,Bp=0° planar 7.3m,Crushed Seam 7.45m,Infilled Seam	
		8 9 10 11 12 13 14 15		Borehole No 1 terminated at 7. 6m						



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Job No 14038/1 BH1 Started Coring at 5.9m

6.0m

7.0m

End at 7.6m

BH1 terminated at 7.6m