REPORT TO

MOON INVESTMENTS PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED CABRAMATTA EAST PRECINCT

AT

BROOMFIELD STREET, CABRAMATTA, NSW

Date: 10 September 2019

Ref: 32430YFrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Cabramatta East precinct at Broomfield Street, Cabramatta, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Stephen Castagnet of Moon Investments Pty Ltd by signed Acceptance of Proposal form dated 24 May 2019. The investigation was carried out in general accordance with our fee proposal, Ref: P49291VF Rev1 dated 30 May 2019.

We understand from the supplied architectural drawings prepared by Plus Architecture (Job No. 20073 dated 11/02/2019) that it is proposed to demolish the existing site structures and construct five towers ranging from 8 and 15 storeys high. The towers will be constructed over four basement levels which we estimate will require excavation to about 12m depth. The proposed development will be constructed over four stages given the size of the site which implies the basements will also be constructed in stages. There is also the option that in the future one of the 15 storey towers will be increased to 19 storeys. We have not been provided with footing loads but we expect high loads for structures of this type.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at the borehole locations. Based on this we have provided comments and recommendations on excavation conditions, retention systems, hydrogeological considerations, footings, subgrade preparation and basement slabs.

2 INVESTIGATION PROCEDURE

The investigation was carried out on 24 to 27 June 2019 and comprised five boreholes drilled with our track mounted JK300 and JK305 drilling rigs. The boreholes were drilled to depths between 7.36m and 8.90m below existing surface levels using spiral auger techniques and a Tungsten Carbide ('TC') bit. These boreholes were then extended to depths ranging from 14.73m and 15.75m using an NMLC triple tube barrel fitted with a diamond coring bit and water flush.

The strength of the subsurface soils was assessed from Standard Penetration Test (SPT) 'N' values augmented by hand penetrometer tests on the SPT split tube samples. The strength of the siltstone and sandstone bedrock was assessed by observation of the auger penetration resistance using a tungsten carbide 'TC' drill bit, together with examination of the recovered rock cuttings. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Where bedrock was diamond cored, the recovered core was returned to our NATA registered laboratory (Soil Test Services (STS)) for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the I_{s50} results. The results are summarised in the attached Table A. Copies of the colour photographs are provided with the borehole logs. Selected soil samples were also tested by STS to determine California Bearing Ratio (CBR) values. The results are summarised in the attached Table B.

Groundwater observations were made in the boreholes during and on completion of drilling and at the end of the field work. We note that water is introduced into the borehole during coring and therefore the water levels measured at completion of coring may be artificially high as the water levels have not had time to stabilise. In BH1, BH4 and BH5, machine slotted PVC standpipes were installed to depth of 14.37m to 15.52m and finished with a cast iron gatic cover to allow longer term groundwater monitoring to be completed. The groundwater levels were measured during a return visit to site on 3 September 2019.

The fieldwork was completed in the full-time presence of our geotechnical engineer who set out the borehole locations, nominated the testing and sampling, and prepared the attached borehole logs. The borehole locations are shown on the attached Figure 2, and these were set out by taped measurements from assumed site boundaries and features as shown on survey plans by Veris (Job No. 173669, Dwg. DETL-001/A, Sheets 1 to 3, dated 03/05/17). The relative levels shown on the attached logs were interpolated from spot heights shown on the survey plan and are therefore only approximate. The height datum used is the Australian Height Datum (AHD). For more details of the investigation procedures and their limitations, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site lies in relatively low-lying topography with a very gentle northerly slope. The site itself slopes down towards the north at less than 1°.

At the time of fieldwork, the site contained numerous single and two storey brick, cement rendered and concrete building that generally appeared in moderate to good condition based upon a cursory external inspection. The site also contains external asphaltic concrete (AC) and concrete pavements. The AC pavements were generally in poor condition showing signs of distress in the form of extensive cracking, whereas the concrete pavements were generally in moderate condition with only occasional cracking.

The site has northern, southern and western street frontages onto Fisher Street, Cabramatta Road East and Broomfield Street, respectively. The adjacent roads comprised AC pavements that generally appeared in good condition with occasional longitudinal cracking observed. We noted the presence of buried services within the roads and footpaths. On the opposite side of Broomfield Street is Cabramatta train station and associated infrastructure including the rail track, pedestrian overpass and lift.

The neighbouring eastern properties comprised a two storey brick building, two storey concrete car park and an external AC paved car park. The structures typically appeared in good condition with no visible defects based upon a cursory inspection from within the subject site. The structures typically abut the common boundary and have similar levels to the subject site.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Penrith indicates the site to be underlain by Bringelly Shale of the Wianamatta Group comprising shale, carbonaceous claystone, claystone, laminite, lithic sandstone, rare coal and tuff.

The boreholes encountered a profile comprising pavement, fill and residual clay overlying siltstone bedrock at moderate depths that in turn overlies sandstone bedrock. The bedrock was generally initially extremely weathered and of hard soil strength and very low rock strength but improved to medium to high strength with depth. Groundwater was measured within the soil profile. A summary of the subsurface profile is given below. For a more detailed description of the subsurface conditions at each location, reference should be made to the attached borehole logs.

Pavement and Fill

Asphaltic concrete (AC) of 100mm thickness was encountered at the surface in BH1. A thin, 5mm thick AC overlay was encountered at the surface in BH2 and overlay a 100mm thick concrete pavement. Concrete pavement was encountered at the surface for the remaining boreholes (BH3, BH4 and BH5) and ranged in thickness between 110mm and 180mm.

Fill was encountered below the pavement in all boreholes and extended to 0.4m depth below surface level. The fill material generally comprised silty clay containing varying amounts of fine to medium grained ironstone, igneous and sandstone gravel. The clayey fill generally had a moisture content less than the plastic limit. We noted the presence of a thin levelling sand layer below the pavement in BH4.

Residual Silty Clay

Residual silty clay was encountered below the fill in all boreholes. The silty clay initially ranged from firm to very stiff strength but increased to very stiff to hard strength with depth. The clays contained varying amounts of fine to medium grained ironstone gravel. The silty clay was assessed as medium to high plasticity and the moisture content was generally less than the plastic limit.

Bedrock

In all boreholes siltstone bedrock was encountered below the silty clays and overlay sandstone bedrock at depth.

Weathered siltstone bedrock was encountered at depths ranging from 2.5m (BH5) to 4.7m (BH1) and levels ranging from approximately RL11.2m (BH1) to RL13.1m (BH5) indicating the bedrock slopes down towards the south-east. The bedrock comprised siltstone and was generally initially highly weathered and of very low to low strength, with the exception of BH1 which was extremely weathered and of hard soil strength when first encountered. With depth the siltstone bedrock increased in strength, at least to low to medium strength although this increase in strength was not uniform with lower strength bands typically present within this higher strength material.

Sandstone bedrock was encountered in all boreholes below the siltstone at depths ranging from 9.15m (BH3) and 12.1m (BH4) and reduced levels varying from RL3.89m (BH1) to RL6.25m (BH3). Once encountered the sandstone bedrock was either of medium or medium to high strength. The bedrock in all boreholes contained numerous near horizontal clay and extremely weathered seams of generally less than 20mm thickness and also occasional angular joints.

Within each of the boreholes the rock encountered has been classified in general accordance with the classification system given in Pells et al. We note that this classification system was formulated to assist with design of footings and as such the classification should take into account the footing width, pile diameter and pile socket length, which are not known at the time of preparing this report. The classification given below is based on representative lengths of core and some judgement and should be treated as approximate only. In addition, within each rock class there may be some subsections of rock being a rock class higher or lower than the overall rock classification. These classifications can be further refined once the footing widths, pile diameters and pile socket lengths are known.

Borehole	Depth and Level to the Top of Each Rock Class									
	Class V Shale		Class IV Shale		Class III Shale		Class III Sandstone		Class II Sandstone or Better	
	Depth	RL (AHD)	Depth	RL (AHD)	Depth	RL (AHD)	Depth	RL (AHD)	Depth	RL (AHD)
1	4.7m*	11.2*	6.0m*	9.9*	-	-	10.43m	5.47	12.01m	3.89
2	3.5m*	11.7*	-	-	-	-	-	-	9.35m	5.85
3	4.0m*	11.4*	-	-	-	-	-	-	9.15m	6.25
4	3.9m*	12.1*	-	-	7.0m*	9.0*	-	-	11.10m	4.90
5	2.5m*	13.1*	4.0m*	11.6*	-	-	-	-	9.40m	6.20

NOTE: Rock Classification in accordance with Foundations on Sandstone and Shale in the Sydney Region, Pells, Mostyn and Walker, Australian Geomechanics, Dec 1998

* Partially or wholly based on augered portion of borehole and so variation may occur. Further proving through additional geotechnical investigations required.

Groundwater

No groundwater seepage was encountered within the auger portion of the boreholes. On a return visit to site, standing water was measured between 3.39m and 3.52m depth below existing surface levels, or about RL12.08m to RL12.58m.

3.3 Laboratory Test Results

The point load strength index test results correlated reasonably well with our field assessment of the rock strength. The estimated UCS values based on a correlation of 20 times the $I_{s(50)}$ value generally ranging from less than 4MPa to 40MPa, although occasional higher values up to 114MPa were measured.

The CBR test on residual clay samples of medium to high plasticity were compacted to 98% of their Standard Maximum Dry Density (SMDD) returning CBR values of 2% and 4%.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings and Issues

As discussed in more detail in Section 3.2, the boreholes penetrated pavements and a thin fill layer that overlay residual clays and then weathered siltstone and sandstone bedrock. The upper portion of the siltstone bedrock was typically of hard soil strength or very low to low rock strength before improving to medium to high and high strength bedrock with depth.

Based on the results of the boreholes and our understanding of the proposed development (refer to Section 1), we have summarised the principal geotechnical findings, issues and recommendations to be considered in the planning, design and construction of the development.

1. Prior to demolition or excavation, we recommend a detailed dilapidation survey be completed on the adjacent building to the east. The remaining boundaries of the site are formed with roadways and council or the NSW RMS may require dilapidation surveys on the roads and footpaths prior to the start of excavation. Dilapidations will may be required along the rail corridor.
2. Excavation for the proposed basement will be through soils and then predominantly through siltstone bedrock of very low to low strength and sandstone bedrock of medium to high strength. Excavation of the bedrock of greater than low strength will require the use of "hard rock" excavation equipment for effective excavation, which may transmit vibrations through the rock mass that could affect adjoining movement sensitive structures.
3. Retention systems will be required to support the proposed excavations, but may be terminated above bulk excavation level (BEL) in the good quality sandstone bedrock which should be suitable to be excavated vertically and left unsupported provided no adverse defects are present. Retention systems may comprise soldier pile walls with shotcrete infill panels where some movements are tolerable or more rigid contiguous pile walls if movements are to be kept low. Lateral support for the piles comprising temporary ground anchors or internal propping will be required. Computer modelling is likely to be required by RMS and Sydney Trains to assess the potential movements induced by the development below RMS roads and the rail corridor.
4. The proposed basements will extend below the water levels measured in the wells. Given the relatively low permeability of the soils and bedrock, we expect seepage rates to be manageable using conventional sump and pump methods. In the long term, we consider that the use of a drained basement will be appropriate for this development but further analysis may be required to satisfy the relevant authorities, such as Council and Department of Primary Industries (DPI) Water.
5. The proposed structure may be supported on pad or pile footings founded within the sandstone bedrock. Suitable geotechnical inspections and testing of the footing excavations will be required, but the extent will depend on the design allowable bearing pressure (ABP) adopted.
6. The proposed basement will overlie bedrock and therefore no particular subgrade preparation will be required for the basement slabs. However, if pavements are proposed external to the basement and these pavements will be supported on the soils subgrade preparation works will be required.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report. Given the current investigation only targeted a portion of the overall site, additional geotechnical investigations will be required to obtain information on the remaining site.

4.1.1 Dilapidation Surveys

Prior to the commencement of demolition and excavation, we recommend that dilapidation surveys be completed on the Council/RMS assets along Cabramatta Road East, Broomfield Street and Fisher Street, as well as the neighbouring structures to the east. We consider it likely that Sydney Trains will require a dilapidation survey on their assets within the rail corridor.

The dilapidation surveys should include internal and external inspection of the buildings, roadways and footpaths, where all defects including defect location, type, length and width are described and photographed. The respective owners of the assets should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.2 Retention

Excavation within the soils and weathered siltstone bedrock will not be self-supporting and therefore retention systems will be required prior to the start of excavation. Excavation for the proposed basement will be required to depths of about 12m and will extend up to the site boundaries. Therefore, insufficient space will be available for temporary batters and are not appropriate for this development.

The upper soil profile and siltstone bedrock must be supported by a suitable shoring system prior to excavation. While the site is bounded to the north, south and west by roads it is understood that the site will be developed in stages. Consequently, excavation of the site will also be completed in a staged manner. Where movement sensitive structures are not present within the zone of influence of the excavation (defined by a distance 2H back from the crest of the wall where H is the retained height) a soldier pile will with infill concrete panels is likely to be suitable. Where movement sensitive structures are present within the zone of influence of the excavation the use of a more rigid contiguous pile wall may be necessary. . The shoring wall may extend full depth, or alternatively, the shoring wall may be terminated once self-supporting sandstone bedrock is encountered.

Unsupported vertical excavations within the sandstone bedrock of greater than low strength may be feasible, provided no adverse defects are present but further geotechnical investigations and regular geotechnical inspections will be required during excavation to confirm this. Excavation should be terminated above the pile toe level to allow the installation of anchors for temporary lateral support and at this time a geotechnical inspection of the conditions exposed should be carried out. Following this, the geotechnical engineer should inspect all every 1.5m of unsupported cut through the sandstone bedrock so that adverse defects such as weak zones, inclined joints etc may be identified and remedial measures initiated. Remedial measures are likely to include rock bolts, shotcrete and mesh. Any additional support required should be installed prior to further excavation.

We recommend retention of the medium to high and high strength siltstone as the bedrock has the potential to contain large continuous inclined joints which can adversely affect stability. Therefore we do not recommend vertical unsupported excavations within the siltstone. Inclined joints within the siltstone may not become apparent until the bulk excavation is reached and at that time it would be too late to install the necessary lateral support to retain the rock wedges isolated by the inclined joints.

Bored piles should be feasible for the site, however given the presence of groundwater, we recommend that all piles are either tremie poured or pumped dry prior to pouring concrete (although tremie methods are likely to be required due to the depth of the piers to prevent concrete segregation). If conditions prove difficult, allowance for casing of the upper soil profile could be considered or, alternatively, Continuous Flight Auger (CFA) piling techniques may be required. The piles may need to be socketed into high strength sandstone and as such piling rigs with adequate capacity to penetrate such rock should be used. We do not recommend sheet piles for the site due to the potential damage to nearby structures and infrastructure caused by the installation process and their inability to penetrate the poor quality bedrock and be installed down to the underlying sandstone bedrock.

During excavation, if soldier piles are adopted, reinforced shotcrete panels should be sprayed progressively as the excavation deepens to support the soils and weathered siltstone between the piles. In this regard we recommend that no more than 1.5m of vertical face be left exposed between the piles at any one time. It will be necessary to install strip drains behind each panel of shotcrete to dissipate the pore pressures from immediately behind the shotcrete facing.

Due to the close proximity of the rail corridor and the expected excavation depth for the proposed basement, Sydney Trains will likely request finite element modelling to assess the potential impact of the development on the rail corridor. Furthermore, a geotechnical monitoring plan would also be required to satisfy Sydney Trains and would detail the required geotechnical instrumentation and monitoring during construction. Where excavation extends to RMS roads, it is similarly likely that RMS will require numerical modelling to predict the potential impact of the excavation and a monitoring program to verify the validity of this analysis. We can complete such modelling, but it would be an iterative process with the structural engineer. In addition we can also prepare the monitoring programs for both Sydney Trains and RMS if required.

4.2.1 Retaining Wall Design Parameters

Propped or anchored walls may be designed based on a trapezoidal earth pressure distribution of 6H kPa , where H is the retained height of soils and weathered bedrock up to and including low strength, where some resulting ground movement is tolerable and adjacent structures or services are located beyond a horizontal distance of 2H from the wall. Where structures or movement sensitive services are located within 2H of the wall, a higher trapezoidal lateral pressure of 8H kPa should be used. These maximum lateral pressures should be held constant for the central 50% of the trapezoidal lateral pressure distribution.

Where the shoring system supports medium and high strength weathered bedrock, then shotcrete panels may be designed for a uniform pressure of 10kPa to support small local wedges of rock.

As discussed above weathered siltstone, in particular when of medium strength or better, have the potential for large continuous defects. Therefore the full shoring system must be designed to be able to support a large sliding wedge of rock inclined at about 45° to the horizontal and daylighting just above BEL, and including the weight of any soils above. The design may be undertaken by adopting a sliding wedge initially assumed to have an effective friction angle of 25° although geotechnical inspections could be carried out to assess if above design value is reasonable for any specific defect encountered. The design will vary along the shoring wall as it is dependent on the height of good quality bedrock above BEL, as well as the depth of the soils above.

Alternatively, to take into account the presence of these sliding wedges of rock, the pressure distribution adopted for the soils and poor quality bedrock could be extended down into the good quality rock, i.e. adopt trapezoidal earth pressure distribution of 6H/8H for full height of wall regardless of rock quality. Adopting this method may lead to a more onerous design and so should be checked by the designer. Allowance for the sliding wedges of rock may be somewhat conservative if such joints are not present, but the installation of anchors only if such joints are encountered during excavation is generally not practical as by the time the base of the joint is exposed it is often too late to install the anchors and the wedge may then be fully isolated.

Appropriate surcharge loads (such as adjoining buildings, traffic, sloping backfill, footing loads etc.) are additional to the above earth pressures and should be allowed for in the design. The additional earth pressures from surcharge loads may be calculated using an 'at rest' earth pressure coefficient of 0.35 where movement sensitive structures are not present within the zone of influence of the excavation and 0.5 where they are.

Based on the water measurements, we expect water to be encountered above BEL. Provided a drained basement is adopted, behind wall drainage such as strip drains should be installed behind the shotcrete facing between the soldier piles and discharge into the stormwater drainage system. Appropriate hydrostatic pressures should be adopted for the soldier pile wall. If contiguous pile walls are proposed then they must be designed for hydrostatic pressures, unless measures can be undertaken to provide complete and permanent drainage. Based on the groundwater monitoring, the water levels ranged from about 3.39m and 3.52m depth below existing surface levels, or about RL12.08m to RL12.58m. We recommend adopting a design level 1m higher than the measured groundwater levels provided above to allow for a potential rise in groundwater levels.

Passive toe resistance of the retention system below the base of the bulk excavation may be estimated based on a maximum allowable lateral resistance of 400kPa for piles socketed into sandstone bedrock of medium or higher strength. The passive resistance should be ignored to at least 0.5m below the base of the excavation, including footing, lift pit and service trench excavations etc due to the potential for fracturing of the bedrock during bulk excavation.

Where adopted anchors should have their bond length formed within the siltstone and sandstone bedrock and may be provisionally designed based on an allowable bond stresses of 100kPa for very low strength siltstone, 150kPa for low to medium strength siltstone or 400kPa for siltstone/sandstone of medium strength or higher strength. The anchor bond should be formed below a line drawn up at 45° from the BEL, with a minimum free and bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load. We have assumed that the long term lateral support will be provided by the floor slabs for the proposed structure.

Where shoring piles are terminated above BEL pile toe restraint in the form of rock bolts/anchors may be required if adequate lateral restraint is not achieved by the upper anchor rows. The rock bolts/anchors may be designed based on the bond stresses provided above. It will be important for the excavation to terminate above the toe of the piles to allow for installation of the rock bolts/anchors prior to excavating further.

Where temporary bolts/anchors run below neighbouring properties permission from the adjoining owners must be obtained prior to their installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping. We note that Sydney Trains do not allow anchors to extend below their property and therefore it may be possible that internal propping may be necessary along the western boundary, unless the anchors can achieve their design load without extending into the rail corridor easement.

Specific shoring wall analysis should be undertaken, including an assessment of the likely ground movements beyond the shoring walls, which we consider will be necessary to satisfy the relevant authorities, such as Sydney Trains due to the nearby rail corridor. We can assist with such analysis if required. The structural engineer should then be requested to provide comment on whether such movements will be problematic to any adjoining structures or services.

4.3 Excavations

All earthworks recommendations should be complemented by reference to the latest edition of Safe Work Australia's 'Excavation Work Code of Practice'.

4.3.1 Excavation Methods

For the proposed basement, we expect excavations of about 12m depth will be required to achieve BEL. The excavations will encounter pavements, soils and siltstone and sandstone bedrock of up to high strength.

Excavation of the soils and rock of up to low strength may be achievable using conventional excavation equipment, such as the buckets of large hydraulic excavators, possibly with some light ripping from a ripping hook fitted to the excavator. Excavation of siltstone and sandstone bedrock of greater than low strength will represent 'hard rock' excavation conditions and will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping tynes or rock saws. The excavator contractor should be made aware of this by being supplied with all geotechnical information, particularly the borehole logs and point load strength test results. Low productivity and increased equipment wear should be expected due to the rock strength.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there could be direct transmission of ground vibrations to nearby structures and buried services. We recommend that initial quantitative vibration monitoring be carried out when using hydraulic rock hammers to determine if the transmitted vibrations are within acceptable limits for the nearby structures and services. Sydney Trains may request full-time vibration monitoring along the western boundary and within their property. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations. Where the transmitted vibrations are excessive, it would be necessary to change to alternative excavation methods, such as smaller rock hammers, rotary grinders, ripping tynes or rock saws. If there are concerns regarding the magnitude of transmitted vibrations, monitoring may need to be carried out full time during rock hammer use.

Alternatively, rock excavations using low vibration emitting equipment, such as rock saws and rock grinders fitted to a hydraulic excavator may be used. If rock saws or rock grinders are used, the resulting dust should be suppressed with water. Use of this low vibration emitting equipment would reduce the likelihood of vibration induced damage to the neighbouring structures and services. With the use of the low vibration equipment we do not consider that it will be necessary to carry out any quantitative vibration monitoring.

The use of excavation contractors with appropriate experience and with a competent supervisor who is aware of vibration damage risks, is also recommended. The contractor should have all appropriate statutory and public liability insurances.

The excavated material will need to be disposed off-site and therefore will need to be suitably classified for waste disposal purposes.

4.4 Hydrogeological Considerations

Groundwater levels were measured to range from about 3.39m and 3.52m depth below existing surface levels, or about RL12.08m to RL12.58m, and therefore are expected to be encountered above BEL. Given the inferred low permeability of the clay soils and bedrock, we consider that in the long-term a drained basement would be appropriate for the site provided the relevant authorities are comfortable with the predicted pump out rates required to maintain the site in a drained condition over the design life of the building. While the soils and bedrock are anticipated to be of low permeability, the size of the proposed excavation may mean that pump out rates may exceed those volumes that are considered acceptable to the relevant authorities.

Groundwater seepage is expected to occur through the soils, at the soil-rock interface and through open joints or bedding planes of the bedrock, particularly during and following rainfall events.

We expect that during construction the seepage would be able to be controlled using conventional sump and pump techniques. In the long term, drainage should be provided around the basement perimeter and below the lowest basement slab to direct seepage into sumps with automatic pumps to remove water from the basement. If rock faces are exposed in the basement (i.e. no walls are constructed in front of them) then access to clean out dish drains should not be a problem. We caution against placing dry walls in front of the rock face as removal of the debris that inevitably frets from the rock faces and falls into drains at the toe of the cut becomes problematic if adequate space is not provided between the cut face and the back of the wall.

To further assist with the hydraulic design and to confirm our preliminary comments, we recommend that a detailed groundwater investigation be undertaken. As a minimum, we recommend that longer-term monitoring be undertaken, say at least 3 months, to further assess the groundwater level and potential fluctuations. Furthermore, infiltration testing should be carried out to assess the permeability of the subsurface conditions to allow for seepage modelling to assess the potential dewatering volumes into the basement.

We expect that dewatering licenses obtained from DPI Water will be required during construction and in the long-term. We understand acquiring these licenses can be a long and timely process. To obtain these licenses, the above recommended additional groundwater investigations will be required to satisfy DPI Water. Should groundwater inflows exceed those volumes considered acceptable to DPI Water a tanked basement may be required. For an excavation of this size there would be significant cost in the construction of a tanked basement.

Actual seepage rates should be observed and measured during construction to confirm the sizing of such sumps and associated pumping systems. The completed excavation should be inspected by the geotechnical and hydraulic engineers to confirm that the designed drainage is sufficient for the actual seepage flows.

4.5 Footings

Based on the results of the investigation and the rock classifications given in Section 3.2, we expect that medium to high strength sandstone bedrock will be exposed at the BEL for the proposed basement level. The proposed structure may be supported on pad or strip footings founded within the exposed sandstone given the expect Class III or better sandstone that will be present. We note the current investigation only targeted a portion of the overall site and therefore we recommend additional cored boreholes to obtain further subsurface information across the remainder of the site.

The design of footings founded within the rock may be based on the following parameters. We note that the serviceability parameters given are based on settlement of less than 1% of the pile diameter or footing width. The ultimate parameters may be used for limit state design on the understanding that settlement of the footing may be up to 5% of the pile diameter or footing width. Differential settlements of about half the total settlements would be expected. The designer may use the modulus values given below to estimate the settlements of particular footings.

Rock Class	Allowable Bearing Pressure	Allowable Shaft Adhesion in Compression	Ultimate Bearing Pressure	Ultimate shaft Adhesion in Compression	Elastic Modulus
Shale Class V	700kPa	70kPa	3000kPa	100kPa	70MPa
Shale Class IV	1000kPa	100kPa	3000kPa	150kPa	300MPa
Shale Class III	3500kPa	350kPa	12,500kPa	500kPa	700MPa
Sandstone Class III	6000kPa	600kPa	40,000kPa	1500kPa	1200MPa
Sandstone Class II or better	8000kPa	800kPa	80,000kPa	2000kPa	1500MPa

Appropriate load factors and geotechnical reduction factors, in accordance with AS2159-2009, must be used in the design. The geotechnical strength reduction factor must be determined by the designer once all details of the design methods and installation requirements are known.

Any piles should be founded with a nominal socket of at least 0.3m into the appropriate class of bedrock. For the design of sockets into the rock below BEL and all surrounding localised excavations, the shaft adhesion should be ignored within the 0.3m nominal socket. For the design of piles in uplift, shaft adhesions of half the shaft adhesions in compression may be used. The shaft adhesion values assume that adequate socket roughness and cleanliness is maintained.

Prior to pouring concrete all footings should be free from all loose and softened materials and should be inspected by a geotechnical engineer to confirm that the design ABP's have been achieved. It should be noted that where footings are founded in Class IV/V siltstone that the siltstone is sensitive to moisture and where water ponds in the base of the footing the siltstone will soften and no longer be suitable for the design ABP. Where this occurs the footing excavation must first be pumped dry and then re-excavated to remove all loose and softened materials.

Where footings are founded within Class III shale or sandstone, we recommend spoon testing be carried out within at least one third of the footing locations, where pad footings are adopted. Spoon testing involves drilling a 50mm diameter hole in the base of the footing excavation to a depth of at least 1.5 times the minimum footing width, but no less than 1.5m deep. The hole is scraped with a grooving tool to assess the location and thickness of any defects below the base of the footing.

4.6 Subgrade Preparation

Earthworks recommendations in this report should be read in conjunction with AS3798-2007: '*Guidelines on Earthworks for Commercial and Residential Developments*'.

Since bedrock will be present at BEL no particular subgrade preparation will be required. However, a subbase layer should be provided below the slabs as recommended in Section 4.7 below.

If pavements for access roads are required outside the basement footprint, preparation of those pavement subgrades will be required. The final subgrade preparation required will depend on the subgrade materials and the recommendations given herein and should be reviewed once the extent and level of any pavements is known.

We assume that the finish levels for the proposed access roads will be constructed essentially at the same level as current levels. As a result, the subgrade will comprise residual clay. Therefore, poor subgrade areas may be identified that will require treatment prior to the placement of pavements.

Following excavation to the design subgrade level, the following subgrade preparation measures should be followed:

- Strip the subgrade of all existing pavements, vegetation, root affected soils or other deleterious materials.
- Following stripping, proof roll the subgrade with a minimum of 6 passes of a smooth drum non-vibratory roller of no less than 12 tonnes static weight. All proof-rolling should be completed in the presence of an experienced geotechnical engineer under static loads.
- The purpose of proof rolling is to improve the near surface density of the soils and identify any soft or unstable areas. Any soft or unstable areas identified should be excavated down to a sound base and reinstated with engineered fill as described below or as directed by the geotechnical engineer during the inspection.
- Care must be taken when rolling close to existing structures or services and the vibrations may need to be reduced or ceased where they are of concern.

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 70mm. We expect that only some of the excavated soils may be suitable for reuse as engineered fill due to the presence of deleterious materials. However, if the deleterious materials are sieved and picked out, then the material may be reused following an inspection by a geotechnical engineer. Engineered fill should be placed in layers of maximum loose thickness of about 200mm, although this layer thickness may be varied provided the layer is being uniformly compacted over its full depth to the required compaction specification. Engineered fill should be compacted to a density of between 98% and 102% of standard maximum dry density (SMDD) and within 2% of Standard Optimum Moisture Content (SOMC).

Density tests should be carried out at a frequency of one test per layer per 500m^2 or three tests per visit, whichever requires the most tests, to confirm the above specification has been achieved. For backfilling of localised excavations, such as service trenches or localised soft spots, testing should consist of one test per two layers per 50m^2.

4.7 Basement Slab

Based on the investigation results, the exposed subgrade below the basement slab will comprise sandstone bedrock. In these areas the basement slab should be underlain by a layer of durable igneous granular material such as DGB20 or other approved material to act as a separation layer between the rock and the basement slab. Although unlikely, it is possible that poor quality bedrock may be present at BEL and consequently, we recommend that the subgrade be inspected by a geotechnical engineer to confirm the above assessment and provide further advice as required.

If a drained basement is permitted then drainage should be provided around the basement perimeter and below the basement slab to direct seepage into sumps with permanent and fail safe automatic pumps to remove water from the basement. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage is sufficient for the actual seepage flows. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel and may comprise a drainage blanket or grid of drains.

4.8 Pavement Design Parameters

The following pavement design assumes that the subgrade preparation and engineered fill recommendations have been implemented on the site as per the recommendations provided in Section 4.6 above.

The four-day soaked CBR tests returned values of 2% and 4% for the residual clays. Based on the test results, we recommend that a value of no more than 2% be used for pavement design. For the CBR value of 2% an estimated modulus of subgrade reaction of 20kPa/mm (750mm plate) may be adopted.

Once the final layout and design levels of any pavements are known we recommend that additional CBR testing be carried out to assess the final design value. This will allow the sampling and testing to be targeted to the actual subgrade soils. If low values of 2% are confirmed by such testing, then some form of subgrade treatment may be required in order to reduce the thickness of the overlying pavement materials. This may comprise placement of a select layer of good quality granular material, such as crushed sandstone, or lime stabilisation of the subgrade soils. Further geotechnical advice on this should be obtained if the additional testing indicate low CBR values.

If concrete pavements are adopted, we recommend that the pavements be underlain by at least 100mm of DGB20, which is compacted to at least 100% SMDD. Concrete pavements should be designed with keyed or dowelled joints to transmit shear forces but not bending moments.

The need for subsoil drainage should be carefully assessed at both design and construction stages. In general it is beneficial to provide a subsoil drain at least on the upslope side of pavements to intercept seepage flows which may soften the subgrade which may lead to premature failure of the pavement. It is important that any subsoil drains have an invert level at least 0.3m below the subbase layer and have adequate falls to collection points to avoid any ponding of water.

4.9 Earthquake Design Parameters

Based upon AS1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia" including Amendment 1 and 2, the following design parameters may be adopted:

- Hazard Factor (Z) = 0.08;
- Class C_e – Shallow soil site

4.10 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:

- Additional geotechnical investigations, in the form of cored boreholes to investigate the remainder of the site.
- Longer-term groundwater monitoring, permeability testing and seepage modelling to assess expected dewatering volumes.
- Dilapidation surveys for the neighbouring structures and infrastructure, especially if rock hammers are to be used.
- Finite element analysis of the potential retention system deflections and impact on the adjoining Sydney Trains and RMS infrastructure, if required.
- Preparation of a geotechnical monitoring program to verify the predicted deflections where required by Sydney Trains and RMS,
- At least initial quantitative vibration monitoring during bulk excavation using rock hammers. Periodic or continuous vibration monitoring will be necessary depending on the level of assurance required,
- Witnessing installation and proof testing of anchors.
- Inspection by a geotechnical engineer of every 1.5m of unsupported vertical cut such that adverse defects may be identified and remedial measures initiated,
- Regular groundwater observations during and on completion of excavation.
- Inspection of all footings by a geotechnical engineer to confirm that the design ABP's have been achieved and, where required spoon tests,
- Additional CBR testing to test subgrade at specific pavement locations,

- Proof rolling of the subgrade in the presence of an experienced geotechnical or geotechnical engineer prior to the placement of engineered fill or pavements,
- Density testing of all fill placed as engineered fill.

Given no structural drawings have been issued and only conceptual architectural drawings are available, we recommend a review by a geotechnical engineer after the initial structural design has been completed to confirm that our recommendations have been correctly interpreted. It is possible that further advice/input will be required during the structural design to address issues that may not have been addressed in this report. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

5 SALINITY

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

6 GENERAL COMMENTS

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

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32430YF
Project:	Proposed Cabramatta East Precinct	**Report:**	A
Location:	Broomfield Street, Cabramatta, NSW	**Report Date:**	11/07/2019
		Page 1 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	9.06 - 9.09	0.2	4
	9.84 - 9.87	0.3	6
	10.15 - 10.18	0.5	10
	10.82 - 10.85	0.9	18
	11.12 - 11.15	0.6	12
	11.85 - 11.88	1.0	20
	12.04 - 12.07	0.5	10
	12.83 - 12.86	1.2	24
	13.06 - 13.09	0.9	18
	13.69 - 13.72	5.7	114
	14.11 - 14.14	5.3	106
	14.83 - 14.86	1.2	24
	15.06 - 15.09	0.7	14
	15.72 - 15.75	1.9	38
2	8.04 - 8.07	0.4	8
	8.88 - 8.91	0.3	6
	9.18 - 9.21	0.2	4
	9.77 - 9.80	0.8	16
	10.11 - 10.15	0.4	8
	10.44 - 10.47	1.2	24
	10.84 - 10.87	1.5	30
	11.25 - 11.28	0.7	14
	11.94 - 11.97	1.1	22
	12.22 - 12.25	1.2	24

NOTES: See Page 3 of 3

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SOIL TEST SERVICES

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POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32430YF
Project:	Proposed Cabramatta East Precinct	**Report:**	A
Location:	Broomfield Street, Cabramatta, NSW	**Report Date:**	11/07/2019
		Page 2 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	12.67 - 12.70	1.2	24
	13.31 - 13.34	0.7	14
	13.77 - 13.80	0.9	18
	14.12 - 14.15	0.9	18
	14.67 - 14.70	1.8	36
3	9.30 - 9.33	0.2	4
	9.58 - 9.61	0.5	10
	9.81 - 9.84	0.7	14
	10.07 - 10.10	0.5	10
	10.81 - 10.84	1.0	20
	11.12 - 11.15	0.7	14
	11.79 - 11.82	0.8	16
	12.13 - 12.16	1.4	28
	12.66 - 12.69	1.4	28
	13.40 - 13.43	0.8	16
	13.91 - 13.94	0.6	12
	14.14 - 14.17	0.8	16
	14.60 - 14.63	2.2	44
4	8.86 - 8.90	0.5	10
	9.15 - 9.18	0.3	6
	9.76 - 9.79	0.4	8
	10.18 - 10.21	0.5	10
	10.88 - 10.91	0.6	12
	11.06 - 11.09	0.3	6
	11.72 - 11.75	0.9	18

NOTES: See Page 3 of 3

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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32430YF
Project:	Proposed Cabramatta East Precinct	**Report:**	A
Location:	Broomfield Street, Cabramatta, NSW	**Report Date:**	11/07/2019
		Page 3 of 3	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	12.15 - 12.18	2.4	48
	12.76 - 12.79	2.5	50
	13.18 - 13.21	1.8	36
	13.81 - 13.84	1.3	26
	14.16 - 14.19	1.7	34
	14.88 - 14.90	1.5	30
5	8.96 - 9.00	0.7	14
	9.32 - 9.35	0.7	14
	9.93 - 9.97	1.1	22
	10.18 - 10.22	0.4	8
	10.67 - 10.70	2.0	40
	11.20 - 11.23	2.1	42
	11.88 - 11.91	0.5	10
	12.28 - 12.31	2.0	40
	12.88 - 12.91	1.0	20
	13.10 - 13.14	2.2	44
	13.75 - 13.78	1.2	24
	14.10 - 14.13	2.5	50
	14.75 - 14.78	3.5	70

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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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32430YF
Project:	Proposed Cabramatta East Precinct	**Report:**	B
Location:	Broomfield Street, Cabramatta, NSW	**Report Date:**	7/08/2019
		Page 1 of 1	

BOREHOLE NUMBER	BH 2	BH 4
DEPTH (m)	0.50 - 1.50	0.50 - 1.50
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.78 STD	1.70 STD
Optimum Moisture Content (%)	19.3	21.0
Moulded Dry Density (t/m^3)	1.75	1.67
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	102	100
Moisture Contents		
Insitu (%)	26.2	26.6
Moulded (%)	19.7	21.0
After soaking and		
After Test, Top 30mm(%)	31.9	30.1
Remaining Depth (%)	27.2	26.4
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	4.5	2.5
C.B.R. value: @2.5mm penetration	4	2

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: 29/07/2019.

07/08/2019

Authorised Signature / Date
(D. Treweek)

NATA
NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
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In full without approval of the laboratory. Results relate only to
the items tested or sampled.

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BOREHOLE LOG

Borehole No. 1

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.9 m
Date: 24/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			0		ASPHALTIC CONCRETE: 100mm.t				
				-	FILL: Silty clay, medium to high plasticity, brown and grey, with fine to medium grained igneous and ironstone gravel.				
	N = 12 5,6,6	15	1	CH	Silty CLAY: high plasticity, with fine to medium grained ironstone gravel.	w<PL	Hd	490 150 500	RESIDUAL
	N = 14 6,6,8	14	2		as above, but red brown and light grey, trace of fine to medium grained ironstone gravel.		VSt - Hd	530 350 350	
	N = 20 9,10,10	13 12	3 4		as above, but light grey mottled red brown, trace of iron indurated bands.		Hd	420 530 450	
	N > 11 10,11/ 70mm REFUSAL	11 10	5	-	Extremely Weathered siltstone: Silty CLAY, medium to high plasticity, light grey mottled red brown, trace of fine to medium grained ironstone gravel.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
		9	6		SILTSTONE: dark grey and grey, bedded sub-horizontally.	HW	L M		LOW TO MODERATE RESISTANCE WITH OCCASIONAL LOW STRENGTH BANDS

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BOREHOLE LOG

Borehole No.
1

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.9 m
Date: 24/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

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
									-	SILTSTONE: dark grey and grey, bedded sub-horizontally. *(continued)*	HW	M		LOW TO MODERATE RESISTANCE WITH OCCASIONAL LOW STRENGTH BANDS
						8	8				SW	M - H		
						7	9			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 15.52m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 9.52m TO 15.52m. CASING 0m TO 9.52m. 2mm SAND FILTER PACK 1.5m TO 15.52m. BENTONITE SEAL 0.15m TO 1.5m. COMPLETED WITH A CONCRETED GATIC COVER.
						6	10							
						5	11							
						4	12							
						3	13							
						2								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 32430YF CABRAMATTA.GPJ <<DrawingFile>> 23/08/2019 11:33 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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CORED BOREHOLE LOG

Borehole No.
1
3 / 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF	**Core Size:** NMLC	**R.L. Surface:** ~15.9 m
Date: 24/6/19	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK300	**Bearing:** N/A	**Logged/Checked By:** J.L./O.F.

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
		7			START CORING AT 8.90m					
80% RETURN			9		NO CORE 0.06m					Bringelly Shale
					SILTSTONE: dark grey and grey, bedded sub-horizontally.	MW	L	0.20	(9.06m) J, 40°, P, R, Cn	
					Extremely Weathered siltstone: silty CLAY, medium to high plasticity, grey mottled red and orange brown, trace of fine to medium grained ironstone gravel.	XW	Hd		(9.22m) Be, 0°, P, R, Cn	
		6	10		SILTSTONE: dark grey and grey, bedded sub-horizontally.	MW	L	0.30	(9.65m) Be, 0°, P, S, Cn	
		5	11		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded sub-horizontally.	SW	M - H	0.50, 0.90, 0.60	(10.10m) CS, 0°, 3 mm.t (10.13m) CS, 0°, 10 mm.t (10.20m) CS, 0°, 10 mm.t (10.26m) CS, 0°, 5 mm.t (10.37m) CS, 0°, 20 mm.t (10.40m) CS, 0°, 10 mm.t (10.42m) CS, 10°, 5 mm.t (11.04m) CS, 0°, 3 mm.t (11.07m) CS, 0°, 3 mm.t (11.10m) Be, 0°, Un, R, Cn (11.12m) Be, 30°, P, R, Cn (11.48m) CS, 0°, 5 mm.t (11.84m) Bex 2, 0°, P, R, Cn (11.89m) Be, 0°, P, R, Cn (11.94m) CS, 0°, 5 mm.t (11.97m) CS, 0°, 40 mm.t (12.01m) Be, 0°, P, R, Cn (12.08m) Be x 2, 0°, P, R, Cn (12.18m) Be, 0°, Un, R, Cn (12.51m) CS, 0°, 10 mm.t	
		4, 3, 2, 1	12, 13, 14		as above, but bedded 0-20°, with siltstone laminae.			1.2	(12.70m) J, 70 - 90°, C, R, Cn (13.06m) Be, 15°, P, R, Cn (13.10m) Be, 0°, P, R, Cn (13.12m) Be, 0°, P, R, Cn (13.13m) Be, 0°, P, R, Cn (13.44m) Be, 0°, P, R, Cn (13.68m) XWS, 20°, 10 mm.t (13.78m) XWS, 0 - 30°, 30 mm.t (13.85m) J, 50°, Un, R, Fe Sn (13.93m) Jh, 30°, Un, Cn (14.12m) Be, 30°, P, R, Fe Sn (14.45m) XWS, 20°, 10 mm.t (14.68m) Be, 0°, P, R, Cn (14.72m) Be, 0°, P, R, Cn	

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

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JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32430YF CABRAMATTA.GPJ <<DrawingFile>> 23/08/2019 11:33 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

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Core Size:** NMLC **R.L. Surface:** ~15.9 m
Date: 24/6/19 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK300 **Bearing:** N/A **Logged/Checked By:** J.L./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (600, 200, 60, 20)	DEFECT DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
80% RETURN					as above, but bedded 0-20°, with siltstone laminae. *(continued)*	SW	M - H	0.70; 1.9		(15.27m) Be, 10°, P, R, Cn (15.33m) Be, 10°, P, R, Cn	Bringelly Shale
		0	16		END OF BOREHOLE AT 15.75 m						
		-1	17								
		-2	18								
		-3	19								
		-4	20								
		-5	21								
		-6									

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JOBNO 32430YF BH1 START CORING AT 8.9m
NO CORE 60mm
9
10
11
12
13
14
15
END OF BOREHOLE AT 15.75m
JK Geotechnics

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BOREHOLE LOG

Borehole No.
2

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.2 m
Date: 25/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

Groundwater Record	SAMPLES: ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING								-	ASPHALTIC CONCRETE: 5mm.t				
						15		-	over CONCRETE: 100mm.t	w<PL			
				DS					FILL: Silty clay, low to medium plasticity, grey and brown, trace of fine to medium grained igneous and ironstone gravel.				
			DB	DS	N = 4 2,2,2	14	1	CI-CH	Silty CLAY: medium to high plasticity, brown, red brown and orange brown, trace of fine to medium grained ironstone gravel.	w<PL	St	140 150 160	RESIDUAL
				DS	N = 8 2,4,4	13	2		as above, but light grey, red brown and orange brown.		VSt	200 260 270	
				DS	N = 18 5,8,10	12	3		as above, but light grey mottled red brown and orange brown.		VSt - Hd	350 400 280	
						11 10 9	4 5 6	-	SILTSTONE: light grey and light orange brown, with clay bands.	HW	VL - L		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE

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BOREHOLE LOG

Borehole No.
2
2 / 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.2 m
Date: 25/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

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
						8			-	SILTSTONE: grey.	DW	L - M		MODERATE RESISTANCE
						7	8			REFER TO CORED BOREHOLE LOG				
						6	9							
						5	10							
						4	11							
						3	12							
						2	13							

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CORED BOREHOLE LOG

Borehole No.
2

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF | **Core Size:** NMLC | **R.L. Surface:** ~15.2 m
Date: 25/6/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK300 | **Bearing:** N/A | **Logged/Checked By:** J.L./O.F.

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
		8			START CORING AT 7.36m					
80% RETURN					SILTSTONE: dark grey, bedded sub-horizontally, with very low to low strength bands.	XW	Hd		(7.36-8.00m) DEFECTS CANNOT BE TAGGED INDIVIDUALLY, FRAGMENTED	Bringelly Shale
		7	8				L - M	0.40	(8.36m) CS, 0°, 200 mm.t; (8.53m) J, 40°, Ir, R, Cn; (8.66m) J, 40°, Ir, R, Cn; (8.76m) CS, 0°, 150 mm.t; (8.83m) CS, 0°, 3 mm.t; (8.85m) CS, 0°, 3 mm.t	
		6	9		as above, but grey.	SW	L	0.30; 0.20	(9.17m) CS, 0°, 15 mm.t; (9.22m) CS, 0°, 20 mm.t	
		5	10		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°, trace of sandstone laminae.		M - H	0.80; 0.40; 1.2	(9.93m) CS, 0°, 10 mm.t; (10.08m) CS, 0°, 10 mm.t	
		4	11					1.5; 0.70	(10.88m) Be, 0°, P, R, Cn	
		3	12					1.1; 1.2; 1.2	(11.88m) Be, 0°, P, R, Cn; (11.97m) Be, 10°, P, R, Cn	
		2	13					0.70; 0.90	(13.72m) Be, 0°, P, R, Cn	

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CORED BOREHOLE LOG

Borehole No.
2
4 / 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Core Size:** NMLC **R.L. Surface:** ~15.2 m
Date: 25/6/19 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK300 **Bearing:** N/A **Logged/Checked By:** J.L./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
80% RETURN		1		▨	SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°, trace of sandstone laminae. *(continued)*	SW	M - H	0.90 1.8			Bringelly Shale
		0 -1 -2 -3 -4 -5	15 16 17 18 19 20		END OF BOREHOLE AT 14.80 m						

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JOB NO 32430YF BH2 START CORING AT 7.36m
7
8
9
10
11
12
13
14
END OF BOREHOLE AT 14.8m
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BOREHOLE LOG

Borehole No.
3

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.4 m
Date: 25/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				0		CONCRETE: 180mm.t				NO OBSERVED REINFORCEMENT
					-	FILL: Silty clay, medium plasticity, brown and grey, trace of fin eto medium grianed igneous and ironstone gravel.	w<PL			
			15		CI-CH	Silty CLAY: medium to high plasticity, brown, red brown and orange brown, trace of ironstone gravel.	w<PL	St - VSt		RESIDUAL
		N = 5 2,3,2							250 220 180	
			14	1				St		
		N = 8 2,3,5							180 170 160	
			13	2		as above, but light grey mottled red brown and orange brown.				
		N = 24 7,12,12	12	3				Hd	500 410 550	
			11	4	-	SILTSTONE: grey and brown.	HW	VL		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
			10	5				VL - L		LOW RESISTANCE
			9	6						

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BOREHOLE LOG

Borehole No.
3
2 / 3

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.4 m
Date: 25/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

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
						8	8		-	SILTSTONE: grey and brown. *(continued)*	HW	VL - L		
						7				SILTSTONE: grey.	SW	L - M H		MODERATE BANDS
						6, 5, 4, 3, 2	9, 10, 11, 12, 13			REFER TO CORED BOREHOLE LOG				

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CORED BOREHOLE LOG

Borehole No.
3

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Core Size:** NMLC **R.L. Surface:** ~15.4 m
Date: 25/6/19 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK300 **Bearing:** N/A **Logged/Checked By:** J.L./O.F.

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: Specific	Formation
		7			START CORING AT 8.90m					
80% RETURN		6, 5, 4, 3, 2, 1	9		Extremely Weathered siltstone: silty CLAY, low plasticity, light grey.	XW	Hd			Bringelly Shale
			9–14.73		SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded ast 0-10°.	SW	M - H	0.20, 0.50, 0.70, 0.50, 1.0, 0.70, 0.80, 1.4, 1.4, 0.80, 0.60, 0.80, 2.2	(9.20m) J, 70°, P, R, Cn; (9.34m) Be, 0°, Ir, R, Cn; (9.55m) Be, 10°, P, R, Cn; (9.91m) CS, 0°, 10 mm.t; (10.71m) Be, 10°, Un, R, Cn; (13.10m) Be, 20°, P, R, Cn; (13.40m) Be, 0°, P, R, Cn; (13.43m) Be, 0°, P, R, Cn; (14.18m) Be, 0°, P, R, Cn; (14.30m) Be, 0°, P, R, Cn; (14.69m) Be, 0°, P, R, Cn	
					END OF BOREHOLE AT 14.73 m					

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JOB NO 32430YF BH3 START CORING AT 8.9m
9
10
11
12
13
14
END OF BOREHOLE AT 14.73m
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BOREHOLE LOG

Borehole No. 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~16 m
Date: 26/6/19 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** J.L./O.F.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING						CONCRETE: 110mm.t				6mm DIA. REINFORCEMENT 80mm TOP COVER
					-	FILL: Silty sand, fine to medium grained, light brown.	D w<PL			
					CH	FILL: Silty clay, medium to high plasticity, grey and brown, trace of fine to medium grained ash.	w<PL	St - St		RESIDUAL
		N = 7 1,3,4	15	1		Silty CLAY: high plasticity, brown mottled red brown and grey, trace of fine to medium grained ironstone gravel.			30 80 120	
		N = 16 3,7,9	14	2		as above, but grey mottled red brown and orange brown.		(St)	150	
		N = 37 10,15,22	13	3		as above, but light grey mottled red brown and orange brown.		Hd	>600 >600 >600	
			12	4	-	SILTSTONE: grey and brown, with iron indurated bands.	HW	VL - L		BRINGELLY SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
			11	5						
			10	6						

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BOREHOLE LOG

Borehole No. 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~16 m
Date: 26/6/19 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** J.L./O.F.

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
						8	8		-	as above, but grey and dark grey.	DW	M		MODERATE RESISTANCE MODERATE TO HIGH RESISTANCE
						7 6 5 4 3	9 10 11 12 13			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 14.37m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 8.37m TO 14.37m. CASING 0.15 TO 8.37m SURFACE. 2mm SAND FILTER PACK 6.2m TO 14.37m. BENTONITE SEAL 0.15m TO 0.5m. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No. 4

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Core Size:** NMLC **R.L. Surface:** ~16 m
Date: 26/6/19 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** J.L./O.F.

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 – Specific (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness)	Formation
					START CORING AT 8.86m					
80% RETURN		7	9		SILTSTONE: dark grey, bedded sub-horizontally.	MW	L - M	0.50, 0.30, 0.40, 0.50, 0.60, 0.30	(9.09m) Be, 0°, Ir, S, Cn (9.14m) Be, 0°, P, S, Cn (9.25m) Be, 0°, Ir, S, Cn (9.43m) XWS, 0°, 20 mm.t (9.74m) Be, 0°, Un, S, Cn (10.80m) Jh, 50°, Un, Fe Sn (10.94m) XWS, 0°, 15 mm.t (11.06m) CS, 0°, 10 mm.t (11.17m) CS, 0°, 10 mm.t	Bringelly Shale
		6	10							
		5	11		SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-20°.	SW	M	0.90		
		4	12				H	2.4, 2.5, 1.8	(13.12m) CS, 20°, 2 mm.t	
		3	13					1.3, 1.7, 1.5	(13.53m) Be, 0°, P, R, Cn (14.64m) CS, 10°, 5 mm.t	
		2	14							

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

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END OF BOREHOLE AT 14.90 m

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JOB NO 32430YF BH4 START CORING AT 8.86m
9
10
11
12
13
14
END OF BOREHOLE AT 14.9m
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BOREHOLE LOG

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.6 m
Date: 27/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			0		CONCRETE: 130mm.t				5mm DIA. REINFORCEMENT 30mm TOP COVER
				-	FILL: Silty clay, medium to high plasticity, brown and grey, trace of fine to medium grained igneous and ironstone gravel, and ash.	w<PL			
	N = 7 3,3,4	15		CH	Silty CLAY: high plasticity, light grey mottled red brown and orange brown, trace of fine to medium grained ironstone gravel, and roots.	w~PL	St		RESIDUAL
	N = 13 4,5,8	14	1			w<PL	VSt	300 310 210	
		13	2.5	-	SILTSTONE: grey and brown, with iron indurated bands.	HW	VL - L		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
		12	4			DW	L - M		MODERATE BANDS
		11, 10, 9	5, 6						

Samples columns: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No.
5
2 / 3

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF **Method:** SPIRAL AUGER **R.L. Surface:** ~15.6 m
Date: 27/6/19 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** J.L./O.F.

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
						8, 7	8		-	as above, but with clay bands.	DW	L - M		
											SW	M - H		MODERATE TO HIGH RESISTANCE
						6, 5, 4, 3, 2	9, 10, 11, 12, 13			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 14.6m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 8.6m TO 14.6m. CASING 0.15m TO 8.6m. 2mm SAND FILTER PACK 0.15m TO 14.6m. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
5

Client: MOON INVESTMENTS PTY LTD
Project: PROPOSED CABRAMATTA EAST PRECINCT
Location: BROOMFIELD STREET, CABRAMATTA, NSW

Job No.: 32430YF	**Core Size:** NMLC	**R.L. Surface:** ~15.6 m
Date: 27/6/19	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK300	**Bearing:** N/A	**Logged/Checked By:** J.L./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION – Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		7			START CORING AT 8.90m						
50% RETURN		6, 5, 4, 3, 2, 1	9 – 15		SILTSTONE: dark grey, with light grey laminae, bedded sub-horizontally.	SW	M	0.70			Bringelly Shale
					SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded 0-15°.	FR	M - H / L / M - H / H	1.1, 0.40, 2.0, 2.1, 0.50, 2.0, 1.0, 2.2, 1.2, 2.5, 3.5			
			15		END OF BOREHOLE AT 15.00 m						

Defects:
- (9.40m) J, 40°, P, R, Cn
- (9.77m) XWS, 0°, 5 mm.t
- (9.92m) Be, 0°, P, R, Cn
- (10.24m) Be, 15°, P, R, Cn
- (10.42m) J, 90°, P, R, Cn
- (10.54m) CS, 0°, 3 mm.t
- (10.67m) CS, 0°, 3 mm.t
- (11.53m) CS, 0°, 3 mm.t
- (11.59m) CS, 0 - 10°, 5 mm.t
- (11.70m) J, 90°, Ir, R, Cn
- (12.09m) Be, 10°, P, R, Cn
- (12.11m) Be, 10°, P, R, Cn
- (12.91m) CS, 30°, 5 mm.t
- (13.33m) Be x 2, 0°, P, R, Cn
- (14.02m) J, 30°, P, R, Cn
- (14.15m) Bex 2, 10°, P, R, Cn
- (14.30m) J, 50°, P, R, Cn

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32430YF CABRAMATTA.GPJ <<DrawingFile>> 23/08/2019 11:34 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


JK Geotechnics
JOB NO 32430YF BH5 START CORING AT 8.86m
9
10
11
12
13
14
END OF BOREHOLE AT 15m
JK Geotechnics

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 29 DEC 2019.

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

Title:	**SITE LOCATION PLAN**
Location:	BROOMFIELD STREET CABRAMATTA, NSW
Report No: 32430YF	Figure No: 1

JKGeotechnics

PLOT DATE: 2/09/2019 12:28:10 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32430YF CABRAMATTA\CAD\32430YF.DWG

© JK GEOTECHNICS

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

Title:	BOREHOLE LOCATION PLAN
Location:	BROOMFIELD STREET CABRAMATTA, NSW
Report No:	32430YF
Figure No:	2

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

JKGeotechnics

PLOT DATE: 12/09/2019 10:36:34 AM DWG FILE: S:\6 GEOTECHNICAL JOBS\32000'S\32430YF CABRAMATTA\CAD\32430YF.DWG

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VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

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:

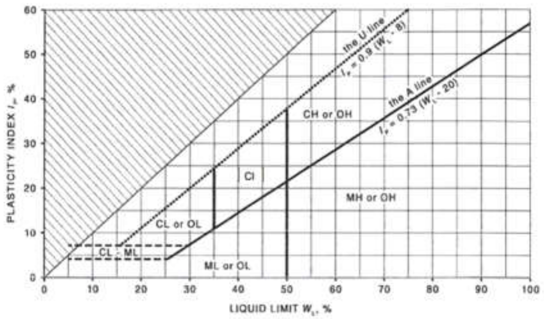
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



JKGeotechnics

LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (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