REPORT TO

RICHARD CROOKES CONSTRUCTIONS PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

NEW PRIMARY SCHOOL IN EDMONDSON PARK

AT

BUCHAN AVENUE, EDMONDSON PARK, NSW

Date: 20 May 2021
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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed New Primary School in Edmondson Park at Buchan Avenue, Edmondson Park, NSW. The location of the site is shown on Figure 1. The investigation was commissioned by a Richard Crookes Constructions (RCC) Purchase Order (No. 1236/643835) dated 20 April 2021. The commission was on the basis of our fee proposal, Ref. P53281PH Rev1, dated 25 March 2021.

Based on the supplied architectural drawings prepared by TKD Architects (Job No. 21002, Dwg Nos. AR-W-DW-0000^{A}, AR-W-DW-1001^{A}, AR-W-DW-2000^{B}, AR-W-DW-2001^{B}, AR-W-DW-2002^{B}, AR-W-DW-2100$^{\mathbf{B}}$ to AR-W-DW-2103^{B}, AR-W-DW-3100^{A}, AR-W-DW-3101^{A}, AR-W-DW-3102^{A}, AR-W-DW-3400^{A} and AR-W-DW-3401^{A}, dated 29/4/21) the proposed development includes the construction of two, one to three level school buildings, an on-grade car park and waste pad along the south-western side of the site, and a basketball court in the south-eastern corner of the site. The approximate outlines of these are shown on Figure 2. We have also been provided with a cut to fill plan (Job No. 210040, Drawing No. 0301, Rev A, dated 18 May 2021) prepared by Northrop. The bulk earthworks plan shows the south-eastern building will have bulk earthworks levels at RL64.25m and RL64.70m, which will require fill at each end to about 2m, but centrally to a maximum of about 5m. The north-western building will have a bulk earthworks level for the north-eastern half at RL65.15m, requiring minor excavation along the north-western side to a maximum depth of about 1.5m and filling to a maximum of about 2.5m in the southern corner. The bulk earthworks level for the south-western half of the north-western building is at RL69.05m, requiring fill to a maximum of about 4m. Fill will also be required between the proposed buildings to a maximum of about 5m. The bulk earthworks level for the proposed basketball court is at RL66.80m, requiring cut and fill earthworks to a maximum of about 1m. The proposed car park and waste pad located along the south-western side of the site will require excavation to a maximum depth of about 1m at the western corner of the site and fill to a maximum of about 1.5m.

We were advised by Rory Dale of Northrop in an email sent on 25 March 2021, that the expected working column loads for the buildings will be in the order of about 450kN.

In 2019, Pells Sullivan Meynink (PSM), carried out a geotechnical investigation at the site, which at that time including the area to the south-east of the site, when the development was at a concept stage. The results of the PSM investigation are presented in their report, Ref. PSM3750-006L, dated 14 February 2019. Two of the PSM boreholes (BH04 and BH05) are relevant to the current project and have been used in preparing this report. Copies of the borehole logs are included in Appendix A for ease of reference.

The purpose of this investigation was to assess the subsurface conditions at four nominated additional borehole locations, but during the fieldwork a fifth borehole was also drilled. Based on the information available, this report presents our comments and recommendations on earthworks, batters and retaining walls, footing design, soil aggression, floors slabs and pavements.

2 INVESTIGATION PROCEDURE

Boreholes BH04 and BH05 by PSM were drilled on 22 January 2019 to auger refusal at depths of 3.4m and 3.0m, respectively. The current fieldwork was carried out on 14 May 2021 and included the drilling of five additional boreholes (BH101 to BH105) using our track mounted JK205 drilling rig. BH101 to BH104 were drilled to depths of 6m, with BH105 terminated at a depth of 4.5m due to refusal of the auger.

BH101 to BH104 were drilled as close as practical to the locations nominated by the structural engineer (Northrop). BH105 was drilled to the south-west of the previous BH04 to further prove the bedrock profile, as BH04 encountered 'TC' bit refusal at a depth of 3.4m and all the rock encountered prior to refusal was logged as extremely weathered.

The current borehole locations were set out by tape measurements from the inferred site boundaries and are shown on the attached Figure 2, which is based on a recent Nearmap aerial image of the site and the supplied survey plan prepared by Total Surveying Solutions (Job No. 210607, Plan No. 210607-1, dated 25 March 2021). Figure 2 also shows the location of the previous BH04 and BH05. The approximate surface levels of BH101 to BH105, as shown on the attached borehole logs, were estimated by interpolation between spot level heights and ground contours shown on the survey plan. The survey datum is the Australian Height Datum (AHD).

In BH101 to BH105, the strength of the soil profile was assessed from the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on clayey samples obtained in the SPT sampler, together with tactile examination. The strength of the bedrock profile was assessed by observation of the auger penetration resistance when using a Tungsten Carbide (TC) bit attached to the augers, together with examination of the recovered rock cuttings and correlation with subsequent laboratory moisture content test results. We note when rock strengths are assessed in this way, the strength may vary by one order of rock strength.

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

Our geotechnical engineer (Joanne Lagan) was present on a full time basis during the current fieldwork to set out the borehole locations, nominate the testing and sampling, and prepare the borehole logs. The borehole logs are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected soil and rock cutting samples were returned to a NATA accredited laboratory (Soil Test Services Pty Ltd [STS]) for moisture content, Atterberg limits, linear shrinkage, standard compaction and four day soaked CBR testing. The test results are summarised in the attached STS Tables A and B. Additional soil samples were returned to another NATA accredited analytical laboratory (Envirolab Services Pty Ltd) for soil pH, sulphate content, chloride content and resistivity testing. The latter test results are summarised in the attached Envirolab Services Certificate of Analysis 267493.

Testing for possible contamination of the site soils and groundwater was outside the agreed scope of the investigation.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located within gently sloping topography. The site is a large rectangular area being about 100m wide and 210m long. Buchan Avenue, Faulkner Way and the rail corridor bound the site to the north-east, north-west and south-west, respectively. Faulkner Way slopes down to the north-east at about 2°, whilst Buchan Avenue slopes down to the south-east at about 3°. A vacant undeveloped property bounds the site to the south-east, and generally exposed clayey soils and patchy grass cover at the surface.

At the time of the fieldwork, the site was vacant and undeveloped. The ground surface was covered by grass or exposed clayey soil. Trafficability for our drilling rig between boreholes locations was good, with no obvious 'soft' ground observed. There is an elevation relief of up to about 8m across the proposed development footprint, with the lowest portions of the site being in between the two proposed building footprints, near BH105. The south-western half of the site generally sloped down to the north-east at about 5°, while the north-eastern portion of the site sloped down to the south-east at about 4°. The ground surface along the north-western side of the site (adjacent to Faulkner Way) sloped down to the south-east at a maximum of about 15°.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates the site is mapped to be underlain by Bringelly Shale of the Wianamatta Group. The Bringelly Shale comprises 'shale, carbonaceous claystone, claystone, laminite, fine to medium-grained lithic sandstone, rare coal and tuff'.

Generally, the current boreholes encountered a thin clay fill layer covering residual silty clay that graded into siltstone bedrock at shallow to moderate depths. The boreholes were 'dry' during and on completion of drilling. Reference should be made to the attached borehole logs for details at each specific location. A summary of the encountered subsurface conditions, including PSM boreholes BH04 and BH05, is provided below.

Fill

Silty clay fill was encountered in each of the current boreholes to depths ranging from 0.3m to 0.4m. Inclusions of igneous, siltstone and ironstone gravel and root fibres were present in the fill. At BH101, BH102 and BH104 the ground surface was grass covered. In BH04 and BH05, a layer of silty clay with root fibres was logged to depths of 0.2m and 0.1m, respectively, that included a comment that it was topsoil. This layer would be the same as the fill layer logged in the current boreholes.

Residual Silty Clays

The residual silty clay and clay was assessed to be of high plasticity in the current boreholes and in BH04 and BH05 ranged from low plasticity to medium to high plasticity.

The residual soils were predominantly of very stiff and hard strength and often contained ironstone gravel.

Siltstone Bedrock

Siltstone bedrock was encountered in all borehole at depths ranging from 1.0m (BH104) to 3.2m (BH105). In BH04 and BH05 the siltstone was logged as being extremely weathered to the termination depths of the boreholes. In BH101, extremely weathered siltstone was encountered to a depth of 2.4m, but then improved to be distinctly weathered and of low strength. Within the remaining boreholes, the siltstone was generally assessed to be distinctly weathered and of very low to low strength on first contact, becoming of low, medium and occasionally high strength with depth.

'TC' bit refusal occurred within medium or high strength bedrock in BH105 at a depth of 4.5m, as well as at depths of 3.4m and 3m in BH04 and BH05, respectively. The refusal depths of BH04 and BH05 possibly indicates the surface of at least medium strength siltstone.

Groundwater

No groundwater seepage was encountered within the boreholes during and on completion of drilling.

With reference to the supplied PSM report, a groundwater monitoring well was installed into BH01, which was located about 150m to the south-east of the proposed basketball court location. A water level data logger was installed into the well and recorded the groundwater level between 21 January 2019 and 12 February 2019. During the monitoring period, which was during a time of limited rainfall, the groundwater level stabilised to about RL53.8m, or about 6m depth. Based on that level, we expect any groundwater at the site to be well below existing surface levels.

3.3 Laboratory Test Results

The Atterberg limits and linear shrinkage test results completed as part of this current investigation indicated the residual silty clay samples from BH101 and BH103 to be of high plasticity, and are therefore assessed to have a high potential for shrink-swell movements with changes in moisture content.

The results of the moisture content tests carried out recovered rock cutting samples correlated reasonably well with our field assessment of bedrock strength.

The four day soaked CBR tests carried out on residual silty clay samples from BH103 and BH105, resulted in CBR values of 1.0% and 2.5%, respectively, when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The insitu moisture contents of the samples were 4.9% 'dry' (BH103) and 0.4% 'dry' (BH105) of their respective Standard Optimum Moisture Contents (SOMC). Swells of 3.5% (BH103) and 4.0% (BH105) were measured during the four day soaking period.

The soil aggression test results indicated acidic (pH 4.9 to 5.9) conditions, low sulphate and chloride contents (maximum 280mg/kg) and variable resistivity values (41ohm.m to 230ohm.m).

4 COMMENTS AND RECOMMENDATIONS

4.1 Primary Geotechnical Issues

Based on the nature of the proposed development, we consider the following are the primary geotechnical issues that should be considered in the design and construction of the proposed development.

- Fill was encountered within the current boreholes to a maximum depth of 0.4m. Whilst the fill is only relatively shallow, we are unaware of any records of placement or compaction control and as such the fill must be considered 'uncontrolled'. PSM also noted that some of the upper residual soils may have comprised topsoil. 'Uncontrolled' fill and topsoil are not considered suitable to support footings, movement sensitive floor slabs and pavements, so will need to be excavated and replaced as part of the earthworks.
- The residual silty clays have a high potential for shrink-swell movements with changes in moisture content.
- Fill will need to be imported to site and placed to a maximum depth of about 5m and will need to be placed and compacted as a controlled fill to reduce the potential for long term settlement of the fill. Consideration will need to be given to the type of fill, as clayey soils will be susceptible to shrink-swell movements, compared to granular materials, such as crushed sandstone.
- Based on the supplied working column loads, we expect that footings will need to be founded within the siltstone bedrock. Due to the depth of fill to be placed, piles will be required to reach the siltstone bedrock.
- The CBR tests measured low CBR values of 1% and 2.5%, which may require a thicker pavement (compared to higher CBR values) and some form of subgrade treatment. We note that the nominated boreholes were located away from the proposed pavement areas and if CBR values are critical additional sampling and testing of the subgrade soils at the actual proposed pavement locations may be warranted.
- The site is bound to the south-west by a railway line. Although the development close to the railway line is minor comprising a car park and waste pad, Sydney Trains may still require review and approval of the proposed development. Given the nature of the development we do not expect that the proposed development will have an adverse effect on the railway line, but Sydney Trains should be contacted to determine their requirements.

Further comments on these issues are provided within the following sections of this report.

4.2 Existing Fill

Fill was encountered in the current boreholes to a maximum depth of 0.4m, though could potentially be locally deeper in other areas of the site. No details on the existing fill (i.e. placement method, compaction specification, density test records, etc.) are available and, therefore, we consider that the existing fill is 'uncontrolled' and not a 'structural' fill, as defined in Clause 1.2.13 of AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

The existing uncontrolled fill is considered to be an unsuitable subgrade to support the footings and floor slabs of the new buildings. Therefore, we strongly recommend that all existing fill be stripped and replaced with engineered fill. Within pavement areas, the fill may remain in place, provided all root affected soils are removed and the fill performs satisfactorily during proof rolling.

4.3 Earthworks

All earthworks recommendations provided below for the proposed development should be complemented by reference to AS3798-2007.

4.3.1 Excavation

To achieve the proposed bulk earthworks levels, excavation will be required to a maximum depth of about 1.5m.

All excavation recommendations should be complemented by reference to current the NSW Government 'Code of Practice Excavation Work'.

Excavation to a maximum depth of about 1.5m is expected to extend into the soil profile only, but if the siltstone is shallower in the areas of the proposed excavation it could be encountered. Excavation of the soil profile may be completed using a 'digging' bucket fitted to a large (i.e. at least 20 tonne) hydraulic excavator. Though unlikely, if bedrock is encountered, then further geotechnical advice should be sought on suitable excavation equipment and controlling vibrations. However, we would expect that only the upper extremely weathered rock will be encountered and this is likely to be able to be excavated using an excavator bucket.

Care must be taken during excavation to not undermine the site boundaries.

4.3.2 Subgrade Preparation

Initially, all grass, root affected soils and any deleterious fill should be stripped from the footprint of the proposed development. The root affected soils should be stockpiled separately as they are generally considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes, from a geotechnical perspective. Reference should be made to Section 5 below for guidance on the offsite disposal of soil.

The investigations have confirmed that some of the upper the subsurface profile comprises clay soil which has "topsoil" properties and contains organic material. If required, a geotechnical assessment of this upper profile to assess its suitability for reuse (possibly blended with better quality material) as engineered fill could be carried out. Such an assessment would comprise the excavation of test pits to better assess the depth of the root affected soils and testing to determine the organic content of the material. Advice would then be provided on the suitability of the material for reuse, including blending of the material with better quality material. If blending was carried out the materials would need to be thoroughly mixed prior to use.

Where a structure is proposed all existing fill should also be stripped to expose the underlying residual silty clay. Care must be taken not to undermine or remove support from the site boundaries during the stripping works. Where pavements are proposed, or where fill is to be placed outside of the building areas, the existing fill may remain in place provided it is proof rolled as recommended below.

Following stripping, the subgrade areas should be proof rolled with at least six passes of a smooth drum vibratory roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out without vibration and in the presence of an experienced geotechnical engineer for the detection of any 'unstable' subgrade areas.

Subgrade heaving during proof rolling may occur in areas where the clays have become 'saturated'. Small areas can typically be improved by locally removing the heaving material to a stable base and replacing with engineered fill. Though unlikely, if the area is large or deep, then a 'bridging' layer of good quality granular material may be required. Options and detailed design of subgrade improvement works must be provided by the geotechnical engineer following the proof rolling inspection.

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

Following preparation of the subgrade, engineered fill may then be used to raise site levels. Engineered fill should be placed and compacted in thin horizontal layers as recommended below.

4.3.3 Engineered Fill and Compaction Control

General

Based on existing surface levels compared to the design levels, material will need to be imported to site. All imported material must comprise Virgin Excavated Natural Material (VENM), and preferably a well graded 'inert' granular material, such as crushed sandstone, free of deleterious materials and having a maximum particle not exceeding 100mm. Such granular material should be compacted in layers of maximum 300mm loose thickness using a large pad-foot roller (say, at least 15 tonnes deadweight) to achieve a density ratio of at least 98% of Standard Maximum Dry Density (SMDD).

The excavated residual silty clay may be reused as engineered fill, but should preferably be used within the lower fill layers. If clay fill is imported to site it will require a greater control of fill compaction and moisture content and should preferably be avoided. In particular, clays of high plasticity should be avoided. Any clay fill should be compacted in layers of maximum 300mm loose thickness using a large pad-foot roller (say, at least 15 tonnes deadweight) to a density ratio strictly between 98% and 102% of SMDD and at a moisture content within 2% of Standard Optimum Moisture Content (SOMC).

If lighter compaction plant are proposed, then thinner layers will be required and further geotechnical advice should be sought once the proposed compaction plant is known.

If a both clay fill and granular fill is sourced, we recommend that the clay fill be used within the lower fill layers and then capped with the granular fill.

Edge Compaction

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

Service Trenches

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of compaction plant that can be placed in trenches, backfilling should be carried out in maximum 100mm loose thickness layers and compacted using a trench roller, a pad-foot roller attachment fitted to an excavator and/or a vertical rammer compactor, also known as a 'Wacker Packer'. Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also be reduced to 50mm. The compaction specifications provided above should still be followed.

Retaining Wall Backfill

Backfilling behind retaining walls must also be carried out using engineered fill in order to reduce post-construction settlements. Noting the size of the site, compaction of the engineered backfill should be carried out using a small static roller or trench roller to reduce the surcharge loads imposed on the walls. Backfilling should be carried out in maximum 150mm thick loose layers and the maximum particle size of the backfill material should not exceed 50mm. The compaction specifications provided above should be followed.

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

Earthworks Inspection and Testing

Density tests should be carried out on all engineered fill to confirm the above compaction specifications are being achieved. The frequency of testing should be in accordance with the guidelines provided in Table 8.1 in AS3798-2007.

Based on the nature of the proposed development, we recommend that Level 1 control of fill placement and compaction in accordance with AS3798-2007 be carried out, including for the trench and retaining wall backfill. Due to a potential conflict of interest, the Geotechnical Inspection and Testing Authority (GITA) should be directly engaged by the client, and not by the earthworks contractor.

Pouring of any on-grade floor slabs should only be completed once the Level 1 earthworks report has been submitted by the GITA and reviewed and approved by the Project Superintendent and/or the geotechnical engineer.

4.3.4 Site Drainage

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

4.4 Batters and Retaining Walls

Temporary excavation batters, if required, of no more than 3m in height should be cut not steeper than 1 Vertical in 1 Horizontal (1V:1H), provided all surcharge loads are kept well away from the crests of the temporary batters.

Permanent batters of no more than 4m in height should be no steeper than 1V:2H, but flatter batters in the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Surface erosion protection, for example, quick establishing grass and/or proprietary systems such as those provided by Geofabrics Australasia or Global Synthetics, should be provided to the permanent batters. Dish drains should also be provided along the crest of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system for disposal.

For permanent batter slopes in excess of 4m in height, we strongly recommend that global stability analyses be completed to confirm the geometric design. It is likely that batters of 1V:3H or 1V:4H would be achievable, but this should be confirmed by such analysis once the fill material that will make up the batters is known.

Cantilevered retaining walls supporting a height of no more than about 3m, located in areas where some movement can be tolerated and which are independent of the proposed buildings, may be designed using a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient (K_a) of 0.35 for the soil profile, assuming a horizontal backfill surface. Cantilevered retaining walls located in areas where movements are to be reduced, or where they are propped by other structural elements in front of the wall, may be designed using a triangular lateral earth pressure distribution with an 'at-rest' earth pressure coefficient (K_0) of 0.55 for the soil profile, assuming a horizontal backfill surface.

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

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

If retaining walls are to support more than about 3m additional geotechnical advice should be obtained. The advice required will depend on the wall type that will be adopted. Consideration could be given to the use of a reinforced earth wall to support the deeper fill, the design of which will require global stability analysis of the proposed geometry and specification of the material that would need to be used within the reinforced block and the reinforcing layers.

4.5 Footings

Following completion of the earthworks, variable conditions will be present at subgrade level, ranging from residual clays and possible weathered siltstone in the areas of cut to engineered fill to considerable depth. Each of the proposed structures should be supported on uniform material. We expect that given the loads of the proposed buildings that they would need to be supported on footings founded within the siltstone. More lightly loaded structures separate to the buildings supported on footings founded within the siltstone may be supported on shallow footings founded within the engineered fill or residual silty clay, such as stiffened raft slabs. However, if excavation is carried out for such structures and encounters the siltstone bedrock then all footings should be founded within the siltstone to provide uniform support.

Where siltstone is exposed or is at shallow depths, pad or strip footings may be used. However, given the depth of fill proposed, we expect that for the proposed buildings, piles will need to be used to reach the siltstone. Bored piles are likely to be suitable for this site.

Bored piles socketed at least 0.3m into siltstone bedrock of at least very low to low strength may be designed based on an allowable bearing pressure of 1,000kPa. Rock sockets formed below the nominal 0.3m socket may be designed for allowable shaft adhesion values of 100kPa (in compression) and 50kPa (in tension), on condition that the pile shaft is suitably roughened using a grooving tool fitted to the side of the auger.

Higher bearing pressures may be appropriate within siltstone of low strength or higher strength, however, additional cored boreholes would be required to determine the appropriate bearing pressures.

For support of lightly loaded structures separate to the main building, such as retaining walls or other minor structures, shallow pad or strip footings or raft slabs founded within engineered fill or residual silty clay of at least very stiff strength could be used. Footings founded within engineered fill may be designed based on allowable bearing pressure of 100kPa. Where footings are founded within residual silty clay of at least very stiff strength an allowable bearing pressure of 200kPa may be used. Such footing must also be designed to accommodate the potential shrink-swell movements of the soils. The magnitude of such movements will depend on the nature of the fill used and may range from movements similar to a Class M site in accordance with AS2870-2011 where granular fill is used, to Class H2 type movements if clay fill is used. The expected movements should be assessed by the geotechnical engineer following completion of the earthworks when the fill material used is known. Settlement of the fill should also be considered as discussed in Section 4.7 below.

Representative footings excavations and bored piles should be inspected by a geotechnical engineer to confirm that the appropriate foundation material has been encountered. Where piles are drilled the geotechnical engineer will need to be present during drilling to assess the material encountered and rock socket lengths. Pad and strip footings and conventional bored piles should be cleaned out, 'dry', inspected by a geotechnical engineer and poured on the same day. If water is allowed to pond within bored pile holes softening of the base may occur and piles may need to be redrilled to remove the softened material prior to pouring.

A Hazard Factor (Z) of 0.08 and a Site Subsoil Class C_e should be adopted for earthquake design in accordance with AS1170.4-2007 ('Structural Design Actions, Part 4: Earthquake Actions in Australia', including Amendment Nos 1 & 2).

4.6 Soil Aggression

Based on the soil pH, sulphate content, chloride content and resistivity test results, concrete and steel piles should be designed for 'Mild' and 'Non-aggressive' exposure classifications, respectively, in accordance with AS2159-2009 'Piling – Design and installation'.

4.7 Floor Slabs and Pavements

Where the floor slabs are supported on engineered fill or residual silty clay they should be constructed independent of the building footings and walls (i.e. designed as 'floating' slabs) to permit relative movements. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

For slab-on-grade construction, the proposed buildings will need to be surrounded by concrete pavements or footpaths that are least 2m wide (unless the slab is supported by a retaining wall), in order to reduce the risk of shrink-swell movements affecting the slabs, unless the upper 1.8m of the soil profile comprises a fill material, such as crushed sandstone. The gap between the building and perimeter concrete pavements or paths must be appropriately sealed to prevent water ingress. Landscaping must be kept well away from the buildings as they provide a source of moisture ingress below the slab. If landscaping features are required, then further geotechnical advice should be sought. Reference should be made to Section 4.5 on potential shrink-swell movements.

Consideration in the slab design should also be made to the potential for settlement of the placed fill. Fill to depths of about 5m may be required and even when properly compacted will undergo future creep settlement, which may be significant given the depth of the fill. Any such settlement will be differential between areas of no fill and areas of deep fill. The amount of settlement that may occur will depend on the type of fill material used, and may range from negligible where granular fill is used possibly to 0.5% of the fill thickness over a log cycle of time where clay fill is used. If the floor slabs cannot accommodate such potential settlement, or shrink-swell movements, fully suspended slabs supported on the piled footings system may be required. Void formers may also be required, but the need for these can only be determined after the type of fill placed is known.

The laboratory CBR tests measured very low CBR values (1% and 2.5%) for the residual silty clay. Design of pavements and floor slabs based on such low values may result in thick pavements, and so some form of subgrade improvement may be warranted to reduce the thickness of the overlying layers. However, given the earthworks that will be required and the depth of fill to be placed, the most practical method would comprise placement of a select layer of good quality granular material, such as crushed sandstone, within at least the upper fill layer. Where on-grade floor slabs or pavements are proposed, we recommend the select layer have a thickness of at least 0.5m, with this thickness and the CBR value of the material then be taken into account in the design.

Alternatively, any clayey engineered fill or residual silty clay subgrade could be stabilised with the addition of lime, but care would be required that the lime is thoroughly mixed with the clay and that the lime does not become air borne as it may affect the neighbouring properties. Laboratory testing would be required to assess the amount of lime required to achieve the desired outcome.

We note that the proposed car park and court pavements are located within an area where boreholes were not drilled and if CBR values are critical to the pavement design then further sampling and testing may be of benefit.

Slabs and concrete pavements should have an unbound subbase layer of at least a 100mm thickness of good quality fine crushed rock such as DGB20 (TfNSW QA Specification 3051 unbound granular material) compacted to at least 100% of SMDD.

In order to protect the pavement edge, subsoil drains should be provided along the perimeter of all proposed pavements, with invert levels of at least 200mm below design subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system for disposal.

4.8 Further Geotechnical Input

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

1. If required, a geotechnical assessment of the upper subsurface profile which has topsoil properties to assess its suitability for reuse/blending.
2. For permanent batter slopes in excess of 4m in height, global stability analyses to confirm the geometric design.
3. Additional CBR testing in the area of the proposed pavements.
4. Proof rolling inspections.
5. Inspection and testing of all engineered fill to Level 1 control by a GITA.
6. Review of the Level 1 report by a geotechnical engineer.
7. Footing/pile inspections.
8. Density testing of all unbound granular pavement materials to at least Level 2 control by a GTA.

5 SALINITY

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

6 GENERAL COMMENTS

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

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

Occasionally, the subsurface conditions between and below 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 is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten 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) could be expected. We strongly recommend that this requirement 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
ABN 43 002 145 173

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

Client:	JK Geotechnics	**Ref No:**	33963PH
Project:	Proposed Edmondson Park Primary School	**Report:**	A
Location:	Gallipoli Drive, Edmondson Park, NSW	**Report Date:**	4/05/2021
		Page 1 of 1	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
101	0.50 - 0.95	16.8	64	20	44	17.5
101	2.50 - 3.00	6.5	-	-	-	-
101	5.00 - 6.00	8.0	-	-	-	-
102	3.00 - 4.00	7.9	-	-	-	-
103	0.50 - 0.95	19.0	57	19	38	15.5
103	3.00 - 4.00	7.2	-	-	-	-
103	5.50 - 6.00	5.6	-	-	-	-
104	3.00 - 3.80	8.7	-	-	-	-
105	3.20 - 3.80	6.3	-	-	-	-
105	4.00 - 4.50	4.2	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/04/2021.
- Sampled and supplied by client. Samples tested as received.

NATA
NATA Accredited Laboratory Number:1327

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

04/05/2021
Authorised Signature / Date
(D. Treweek)

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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:**	33963PH
Project:	Proposed Edmondson Park Primary School	**Report:**	B
Location:	Gallipoli Drive, Edmondson Park, NSW	**Report Date:**	28/04/2021
		Page 1 of 1	

BOREHOLE NUMBER		BH 103	BH 105
DEPTH (m)		0.50 - 1.50	0.50 - 1.50
Surcharge (kg)		9.0	9.0
Maximum Dry Density (t/m^3)		1.83 STD	1.75 STD
Optimum Moisture Content (%)		14.9	16.8
Moulded Dry Density (t/m^3)		1.79	1.72
Sample Density Ratio (%)		98	98
Sample Moisture Ratio (%)		100	99
Moisture Contents			
Insitu (%)		10.0	16.4
Moulded (%)		14.9	16.7
After soaking and			
After Test, Top 30mm(%)		25.7	26.1
	Remaining Depth (%)	20.3	20.0
Material Retained on 19mm Sieve (%)		0	0
Swell (%)		3.5	4.0
C.B.R. value:	@2.5mm penetration	1.0	2.5

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: 15/04/2021.

NATA Accredited Laboratory Number:1327

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the items tested or sampled.

28/04/2021
Authorised Signature / Date
(D. Treweek)

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CERTIFICATE OF ANALYSIS 267493

Client Details

Client	JK Geotechnics
Attention	Joanne Lagan
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**33963PH, Edmondson Park**
Number of Samples	3 Soil
Date samples received	23/04/2021
Date completed instructions received	23/04/2021

Analysis Details

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

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

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

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	30/04/2021
Date of Issue	29/04/2021

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		267493-1	267493-2	267493-3
Your Reference	UNITS	BH102	BH103	BH104
Depth		0.2-0.3	1.5-1.95	0.5-0.95
Date Sampled		15/04/2021	15/04/2021	15/04/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	26/04/2021	26/04/2021	26/04/2021
Date analysed	-	26/04/2021	26/04/2021	26/04/2021
pH 1:5 soil:water	pH Units	5.9	4.9	5.3
Chloride, Cl 1:5 soil:water	mg/kg	28	280	130
Sulphate, SO4 1:5 soil:water	mg/kg	25	170	75
Resistivity in soil*	ohm m	230	41	72

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			26/04/2021	[NT]	[NT]	[NT]	[NT]	26/04/2021	[NT]
Date analysed	-			26/04/2021	[NT]	[NT]	[NT]	[NT]	26/04/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	109	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	108	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments

pH ran outside of recommended holding time.

JKGeotechnics

BOREHOLE LOG

Borehole No.
101
1/1

Client: RICHARD CROOKES CONSTRUCTIONS PTY LTD

Project: PROPOSED EDMONDSON PARK PRIMARY SCHOOL

Location: BUCHAN AVENUE, EDMONDSON PARK, NSW

Job No.: 33963PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 66.0m

Date: 14/04/2021 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** J.L./A.J.H.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium plasticity, brown, trace of fine grained igneous and ironstone gravel and root fibres.	w≈PL			GRASS COVER
	N = 17 4,7,10		CH	Silty CLAY: high plasticity, red brown and light grey, trace of fine to medium grained ironstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
	N = 29 5,12,17	1						
		2	-	Extremely Weathered siltstone: silty CLAY, medium plasticity, grey and light grey, with iron indurated seams.	XW	Hd		BRINGELLY SHALE
				SILTSTONE: brown grey, with extremely weathered seams.	DW	L		VERY LOW TO LOW 'TC' BIT RESISTANCE
		3		SILTSTONE: grey and grey brown.		L-M		LOW TO MODERATE RESISTANCE
		4						
		5						
		6		END OF BOREHOLE AT 6.0m				
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
102
1/1

Client: RICHARD CROOKES CONSTRUCTIONS PTY LTD
Project: PROPOSED EDMONDSON PARK PRIMARY SCHOOL
Location: BUCHAN AVENUE, EDMONDSON PARK, NSW

Job No.: 33963PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 63.8m
Date: 14/04/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.L./A.J.H.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained ironstone and igneous gravel and root fibres.	w≈PL			GRASS COVER
	N = 13 2,5,8		CH	Silty CLAY: high plasticity, red brown and light grey, with fine to medium grained ironstone gravel bands.	w≑PL	VSt- Hd	500 510 390	RESIDUAL
	N = 23 5,9,14	1		Silty CLAY: high plasticity, light grey mottled red brown, with fine to medium grained ironstone gravel bands.	w<PL	Hd	>600 >600 >600	
		2						
			-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey and red brown.	XW	Hd		BRINGELLY SHALE
		3		SILTSTONE: brown grey and grey.	DW	L		VERY LOW 'TC' BIT RESISTANCE
								LOW RESISTANCE
		4				L-M		LOW TO MODERATE RESISTANCE
		5						
		6		END OF BOREHOLE AT 6.0m				
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
103
1/1

Client: RICHARD CROOKES CONSTRUCTIONS PTY LTD
Project: PROPOSED EDMONDSON PARK PRIMARY SCHOOL
Location: BUCHAN AVENUE, EDMONDSON PARK, NSW

Job No.: 33963PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 64.3m
Date: 14/04/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.L./A.J.H.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION			0			FILL: Silty clay, medium plasticity, brown, with fine to coarse grained siltstone, ironstone and igneous gravel, trace of sand.	w<PL			
		N = 16 3,6,10			CH	Silty CLAY: high plasticity, red brown, light grey and orange brown, with fine to medium grained ironstone gravel bands.	w≈PL	Hd	480 580 >600	RESIDUAL
		N = 38 16,16,22	1–2			as above, but with occasional extremely weathered siltstone bands.	w<PL		>600 >600 >600	VERY LOW 'TC' BIT RESISTANCE
			3		-	SILTSTONE: brown grey and red brown.	DW	L		BRINGELLY SHALE LOW 'TC' BIT RESISTANCE
			4			as above, but grey.	SW	L-M		LOW TO MODERATE RESISTANCE
								M		MODERATE RESISTANCE
			6			END OF BOREHOLE AT 6.0m				
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
104
1/1

Client: RICHARD CROOKES CONSTRUCTIONS PTY LTD
Project: PROPOSED EDMONDSON PARK PRIMARY SCHOOL
Location: BUCHAN AVENUE, EDMONDSON PARK, NSW

Job No.: 33963PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 63.1m
Date: 14/04/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.L./A.J.H.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low to medium plasticity, brown, trace of fine grained igneous and ironstone gravel and root fibres.	w<PL			GRASS COVER
		N = 18 4,7,11			CH	Silty CLAY: high plasticity, red brown and light grey, with fine to medium grained ironstone gravel and iron indurated bands.	w≈PL	Hd	>600 >600 >600	RESIDUAL
			1		-	SILTSTONE: grey brown, with occasional iron indurated bands and extremely weathered siltstone bands.	DW	VL-L		BRINGELLY SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
			2							
			3					L		
			4			SILTSTONE: grey and grey brown.		L-M		LOW TO MODERATE RESISTANCE
			5					M		MODERATE RESISTANCE
			6			END OF BOREHOLE AT 6.0m				
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
105
1/1

Client: RICHARD CROOKES CONSTRUCTIONS PTY LTD
Project: PROPOSED EDMONDSON PARK PRIMARY SCHOOL
Location: BUCHAN AVENUE, EDMONDSON PARK, NSW

Job No.: 33963PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 61.6m
Date: 14/04/2021 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** J.L./A.J.H.

Groundwater Record	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION	0		FILL: Silty clay, low to medium plasticity, brown, trace of fine to medium grained igneous and ironstone gravel and root fibres.	w<PL			
		CH	Silty CLAY: high plasticity, red brown and light grey, with occasional fine to medium grained ironstone and iron indurated bands.	w≈PL	(Hd)		RESIDUAL
	1						
	2						
			as above, but with occasional extremely weathered siltstone bands.	w<PL			BANDS OF VERY LOW 'TC' BIT RESISTANCE
	3						
		-	SILTSTONE: grey and brown grey.	DW	L		BRINGELLY SHALE LOW BIT RESISTANCE
	4				M-H		MODERATE TO HIGH RESISTANCE
			END OF BOREHOLE AT 4.5m				'TC' BIT REFUSAL
	5						
	6						
	7						

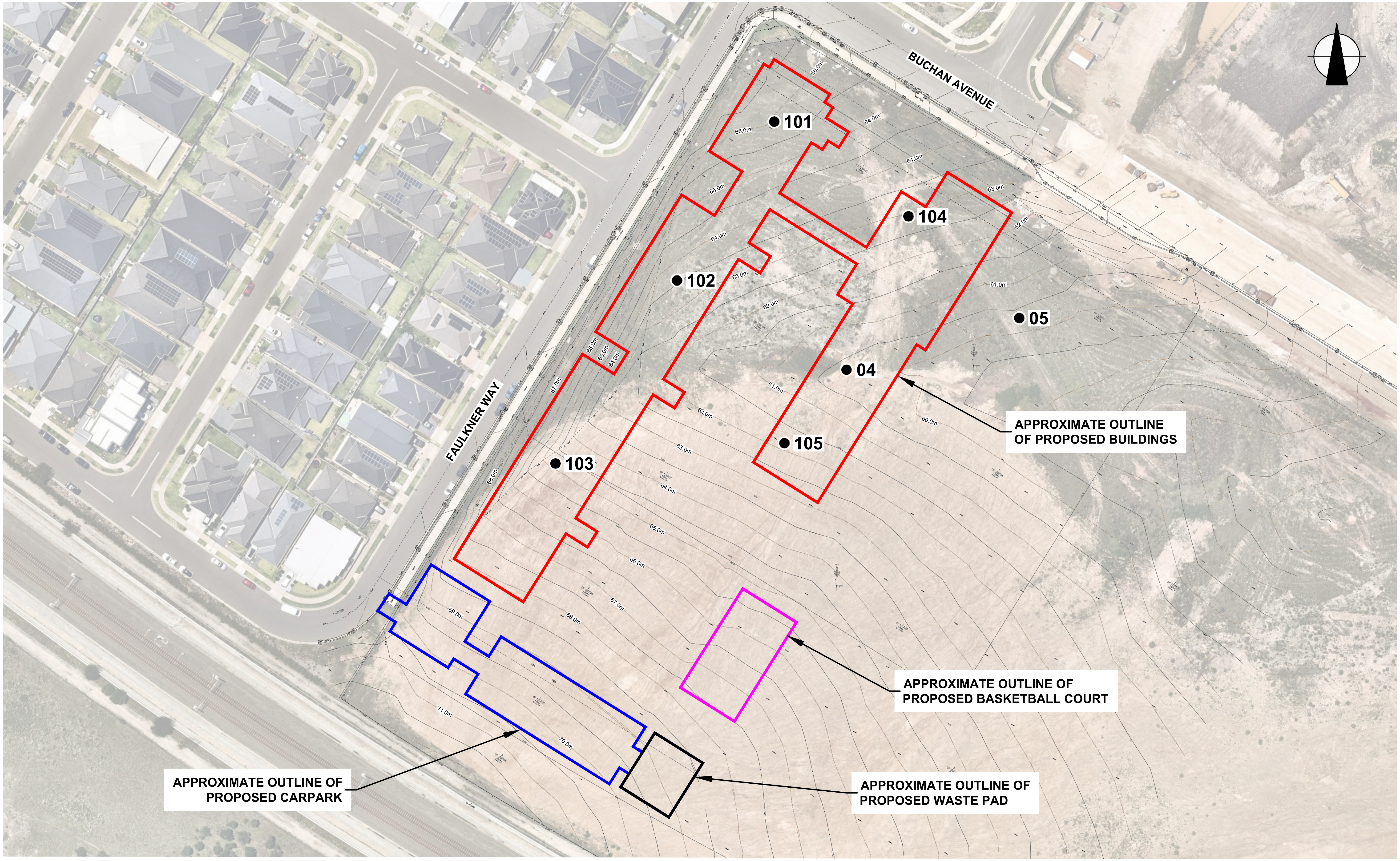
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	BUCHAN AVENUE, EDMONDSON PARK, NSW
Report No: 33963PH	Figure No: 1

JKGeotechnics

PLOT DATE: 16/04/2021 9:59:38 AM DWG FILE: Y:\33000'S\33963PH EDMONDSON PARK\CAD\33963PH.DWG



NOTES:
1. BOREHOLES 04 AND 05 ARE FROM A PREVIOUS 2019 GEOTECHNICAL INVESTIGATION CARRIED OUT BY PSM.
2. BOREHOLES 101 TO 105 ARE FROM THE CURRENT INVESTIGATION.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 10 20 30 40 50
SCALE 1:1000 @A3 METRES

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

Title:	BOREHOLE LOCATION PLAN
Location:	BUCHAN AVENUE, EDMONDSON PARK, NSW
Report No: 33963PH	Figure No: 2

JKGeotechnics

PLOT DATE: 4/05/2021 9:37:10 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33963PH EDMONDSON PARK\CAD\33963PH.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or 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:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

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

NOTES:

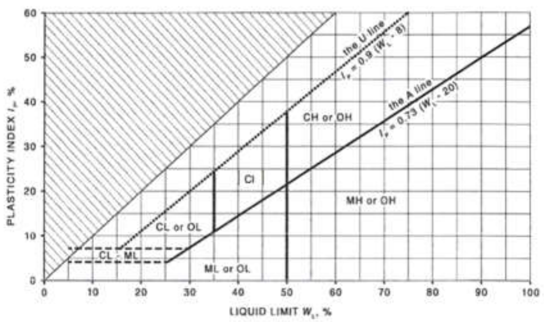
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Abbreviations Used in Defect Description

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

APPENDIX A

PSM

Borehole ID
BH04
Page 1 of 1

Engineering Log - Non Cored Borehole

Project No.: PSM3750

Client:	SINSW	Commenced:	22/01/2019
Project Name:	Geotechnical Investigation on Landcom 6.0 Ha Land	Completed:	22/01/2019
Hole Location:	Lot 375 Edmondson Park	Logged By:	MB
Hole Position:	301622.0 m E 6239539.0 m N	Checked By:	YB

Drill Model and Mounting:	Hanjin Track Mounted	Inclination:	-90°	RL Surface:	60.50 m
Hole Diameter:	110 mm	Bearing:		Datum:	AHD Operator: B&G Drilling

Method	Support	Water	Samples Tests Remarks	RL (m)	Depth (m)	Classification Symbol	Material Description SOIL NAME: Colour, structure, plasticity, additional	Moisture Condition	Consistency / Relative Density	Structure and Additional Observations
AD/V	N	Not Observed			0.00	CL	CLAY: pale brown and orange, low plasticity, roots and rootlets observed.	D	St	0.00: Topsoil
AD/T	N		Atterberg 0.20-0.50 m; ES 0.50 m			CL	CLAY: pale brown, medium to high plasticity.			0.30: V-bit refusal.
			SPT 1.00 - 1.45 m 4, 11, 14 N = 25	59.5	1				VSt	1.00: SPT recovered: 0.45 m
				58.5	2		SILTSTONE: Grey and orange, extremely low strength, extremely weathered.			2.00: Inferred Bedrock, strength and weathering inferred from cuttings.
			SPT 2.50 - 2.90 m 13, 19, Refusal							2.50: SPT recovered: 0.40 m
				57.5	3					
					3.40		Hole Terminated at 3.40 m Refusal			3.40: TC-bit refusal.
				56.5	4					

Hand Penetrometer UCS (kPa): 100 200 300 400 500

Method
AD/T - Auger drilling TC bit
AD/V - Auger drilling V bit
WB - Washbore
SPT - Standard penetration test
PT - Push tube
AS - Auger Screwing

Penetration
No resistance through to refusal

Water
Inflow
Partial Loss
Complete Loss

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
ES - Environmental Sample
TW - Thin Walled
LB - Large Disturbed Sample

Classification symbols and soil descriptions based on Unified Soil Classification System

Moisture Condition
D - Dry
M - Moist
W - Wet

Consistency/Relative Density
VS - Very soft
S - Soft
F - Firm
St - Stiff
VSt - Very stiff
H - Hard
VL - Very loose
L - Loose
MD - Medium dense
D - Dense
VD - Very dense
Ce - Cemented
C - Compact

See Explanatory Notes for details of abbreviations and basis of descriptions.

PSM 3.00.2 LIB V2.GLB Log IS_AU_NONCORE_BH_NZ_AU PSM3750 GINT LOGS.GPJ <<DrawingFile>> 08/02/2019 15:46 8.30.003 Datgel Lab and In Situ Tool - DGD | Lib: PSM 3.00.2 2015-10-23 Prj: PSM 2.01 2015-04-07

PSM

Borehole ID
BH05
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Engineering Log - Non Cored Borehole

Project No.: PSM3750

Client:	SINSW	Commenced:	22/01/2019
Project Name:	Geotechnical Investigation on Landcom 6.0 Ha Land	Completed:	22/01/2019
Hole Location:	Lot 375 Edmondson Park	Logged By:	MB
Hole Position:	301677.0 m E 6239566.0 m N	Checked By:	YB

Drill Model and Mounting:	Hanjin Track Mounted	Inclination:	-90°	RL Surface:	60.50 m		
Hole Diameter:	110 mm	Bearing:		Datum:	AHD	Operator:	B&G Drilling

Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description SOIL NAME: Colour, structure, plasticity, additional	Moisture Condition	Consistency / Relative Density	Hand Penetrometer UCS (kPa) 100 200 300 400 500	Structure and Additional Observations
AD/V			Not Observed	CBR 0.00-1.50 m					CL	SILTY CLAY: pale brown, low plasticity, roots and rootlets observed.	D to M	St		0.00: Topsoil
									CL	CLAY: grey and red, low plasticity.		VSt		0.10: Inferred Natural Soil.
		N		SPT 1.00 - 1.45 m 6, 8, 17 N = 25		59.5	1							1.00: SPT recovered: 0.45 m
				ES 1.50 m					CI-CH	CLAY: pale brown and grey, medium to high plasticity.	D	H		
						58.5	2							2.20: V-bit refusal.
AD/T		N		SPT 2.50 - 2.75 m 5, Refusal					2.7					2.50: SPT recovered: 0.25 m
										SILTSTONE: Grey and orange, extremely low strength, extremely weathered				2.70: Inferred Bedrock, strength and weathering inferred from drill cuttings.
				ES 3.00 m		57.5	3			Hole Terminated at 3.00 m Refusal				3.00: TC-bit refusal.
						56.5	4							

Method
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