REPORT TO

NSW DEPARTMENT OF EDUCATION

ON

GEOTECHNICAL INVESTIGATION

FOR

PARRAMATTA EAST PUBLIC SCHOOL UPGRADE

AT

PARRAMATTA EAST PUBLIC SCHOOL, 30-32 BRABYN STREET, NORTH PARRAMATTA, NSW

Date: 4 March 2025
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JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Arthur Billingham
Associate | Geotechnical Engineer

Report reviewed by:

Paul Stubbs
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

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Table of Contents

ATTACHMENTS

STS Table A: Moisture Content, Atterberg Limits & Linear Shrinkage Test Report

STS Table B: Four Day Soaked California Bearing Ratio Test Report

Table C: Point Load Strength Index Test Report

Envirolab Services Certificate of Analysis No. 300802

Borehole Logs 1 to 8 Inclusive (With Core Photographs for BH2 and BH5)

Figure 1: Site Location Plan

Figure 2: Borehole Location Plan

Vibration Emission Design Goals

Report Explanation Notes

1 CLIENT SUPPLIED INTRODUCTION

This geotechnical report been prepared by JK Geotechnics on behalf of the NSW Department of Education to support a Review of Environmental Factors (REF) for the Parramatta East Public School (PEPS) upgrade (the Proposal) at 30-32 Brabyn Street, North Parramatta (the site). The works are proposed by the NSW Department of Education to meet the growth in educational demand in Collet Park precinct, and the broader North Parramatta area.

This report has been prepared to provide comments and recommendations on excavation, groundwater, retention, earthworks, footings, floor slabs and pavements.

1.1 Summary of the Activity

The activity comprises upgrades to PEPS to provide replacement teaching facilities in place of the existing temporary and permanent facilities that are no longer fit for purpose, involving the following works:

- Site preparation and required earthworks;
- Demolition of existing Buildings C, D, E and F, and associated structures including adjacent ramps and walkways;
- Construction of the following:
  - A new 3-storey school building (referred to as Block R) including teaching spaces, library/administration, and staff/student amenities;
  - Upgrade of soft and hard landscape and playground areas;
  - A new at-grade parking area;
  - Formalised waste area, with access being retained from Gaggin Street;
  - Public Domain Works with upgrades to the pedestrian access south of the school, and new kiss and ride zone on Albert Street East;
  - Entrance and School logo signage along the Northern Albert Street East frontage of Block R;
- Refurbishment works to existing buildings;
- Removal of trees as required and retention where possible; and
- Installation and augmentation of services and infrastructure as required.

Refer to the Review of Environmental Factors prepared by Ethos Urban for a full description of works.

1.2 Site Description

The site is located at Brabyn Street within the City of Parramatta Local Government Area. Parramatta East Public School is located in the suburb of North Parramatta, within the City of Parramatta Local Government Area (LGA).

The site is approximately 1.5km northeast of the Parramatta CBD, and 24km west of the Sydney CBD. The site currently comprises a single lot to make up Parramatta East Public School, referred to as Lot 100, DP1312418, and the land is owned by the Minister for Education and Early Learning.

The site has an area of approximately 1.782Ha, is of an irregular shape, and is bounded by Brabyn Street to the West, Albert Street East to the North, and Gaggin Street/Webb Street to the East. The project area is contained within the site and represents where the proposed works will be undertaken, with an area of approximately 1.492ha.

An aerial image of the site and project area is shown at Plate 1 below.

Plate 1. Site Aerial (Source: Nearmap, Ethos Urban)

2 INTRODUCTION

This report presents the results of a geotechnical investigation for the PEPS upgrade at 30-32 Brabyn Street, North Parramatta, NSW. The location of the site is shown in Figure 1. The geotechnical investigation was commissioned by the NSW Department of Education and was carried out in accordance with our proposal dated 10 January 2020, Ref: P50937LT, and the signed Part D Standard Form Agreement.

We have been supplied with the architectural drawings prepared by JDH Architects (Job No. 1291, Sheet Nos. PEPS-JDH-003 to PEPS-JDH-011, PEPS-JDH-101 to PEPS-JDH-107 and PEPS-JDH-410 to PEPS-JDH-412, Revision B dated 25 February 2025). From these drawings, we understand that following demolition of Buildings C, D, E and F, and associated structures the proposed activity will comprise:

- Construction of a three-storey school building (Block R) within the northern portion of the site. The proposed ground floor level of the building is RL25.3m. To achieve the proposed floor level will require excavation and filling no more than 1m from existing levels. A lift pit and buried rainwater tank are proposed on the southern side of the building. The building is anticipated to have moderate structural loads.
- Construction of an on-grade carpark with access from Brabyn Street within the south-western portion of the school. From the bulk earthworks plan prepared by Woolacotts (Job No. 22-107,

Drawing No. PEPS-WCE-00-00-DR-C-0401, Revision C dated 3 March 2025) excavation to achieve the proposed surface levels are generally not anticipated to be greater than 0.8m.

- Construction of a buried OSD tank within the eastern portion of the site with an overlying hardstand area accessed from Gaggin Street. From the bulk earthworks plan excavation for the OSD tank is anticipated to extend to maximum depths of approximately 2.5m at the northern end, reducing towards the south.
- Construction of new footpaths and ramps around the proposed Block R and existing buildings.
- Upgrade of the games court within the central portion of the site. Surface levels on the court are proposed to range from RL24.02m to RL23.74m which are close to the existing levels.

The purpose of the site investigation was to obtain geotechnical information on the subsurface conditions within the Project Area as shown in Plate 1. Based on the information obtained from the subsurface investigation we have provided our comments and recommendations for the proposed development.

This geotechnical assessment was carried out in conjunction with a detailed site investigation by our environmental division, JK Environments (JKE). Reference should be made to the separate reports by JKE, Ref: E35073BR, for the results of the environmental site assessments.

3 GEOTECHNICAL ASSESSMENT PROCEDURE

3.1 Phase 1: Preliminary Assessment

The geotechnical assessment comprised the following:

- A walkover assessment of the site and its surrounds by our Senior Geotechnical Engineer on 16 June 2022. Geotechnical mapping and observations have been carried out using hand held clinometer and tape measure techniques and are therefore only approximate. A summary of our observations is presented in Section 3 below.
- A search of the JK Geotechnics project database to search for geotechnical investigations completed in the vicinity of the site, and the likely subsurface conditions on the subsurface site.
- A review of the Sydney 1:100,000 Geological Series Sheet 9130.

3.2 Phase 2: Site Investigation

The fieldwork was completed between 12 and 15 July 2022, and comprised the drilling of eight boreholes (BH1 to BH8) to depths ranging from 3.6m to 8.0m below existing surface levels, using our track-mounted JK309 and JK205 drilling rigs. Six of the boreholes were advanced through the soils and weathered bedrock using spiral auger drilling techniques and a Tungsten Carbide (TC) bit. Two boreholes (BH2 and BH5) were initially extended through the soil and upper weathered rock using spiral augers and were then extended to the final depths by rotary diamond coring techniques, using an NMLC triple tube core barrel and water flush.

The borehole locations are shown on the attached Figure 2, and were set out by taped measurements from existing surface features shown on the survey plan. Based on discussions with DoE, the borehole locations were nominated by JK Geotechnics to provide general coverage of the Project Area prior to confirmation of the development scope. The approximate Reduced Level (RL) at each borehole location, as shown on the borehole logs, was interpolated from spot heights and contours from the supplied survey plan prepared by Astrea (Job No. A2545 dated May 2022). The height datum is Australian Height Datum.

The apparent compaction of the fill and strength of the cohesive soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer tests carried out on cohesive samples recovered by the SPT split tube sampler. The strength of the bedrock in the augered portion was assessed from observation of the drilling resistance using the TC drill bit attached to the augers, tactile examination of rock cuttings, and correlation with the results of subsequent laboratory moisture content tests. It should be noted that strengths assessed in this way are approximate and variances of at least one strength order should not be unexpected.

For the cored portion of the bedrock, the recovered core was returned for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations, the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is_{50} results. These Point Load Strength test results are summarised in the attached Table C and on the borehole logs.

Selected soil samples were also returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories. STS completed moisture content, Atterberg limits and California Bearing Ratio (CBR) testing and the results of these tests are provided in the attached STS Tables A and B. Soil aggression testing was completed by Envirolab Services Pty Ltd and the results are provided in the attached Certificate of Analysis No. 300802.

Groundwater observations were recorded in all boreholes during and on completion of auger drilling. Standpipe piezometers was installed in BH1, BH3 and BH6 to allow for longer-term groundwater monitoring. No further groundwater monitoring has been carried out since the fieldwork was completed.

Our geotechnical engineer, Mr Tom Foster, was present on a full-time basis during the fieldwork, to nominate testing and sampling and prepare the borehole logs. The borehole logs, which include field test results and groundwater observations, 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.

4 SITE OBSERVATIONS

4.1 Site History

From a review of historical imagery and maps it appears that until the late 1940s the northern portion of the site primarily contained vacant land with residential properties in the southern portion. Over the next few decades, the school expanded and houses within the former residential properties to the south were demolished. No major cut or fill operations appear to have taken place on the site.

4.2 Site Description

The site is located in gently undulating topography on the southern flank of a local hill whose crest is located on Albert Street East, roughly halfway along the frontage with the site. Surface levels generally appear to follow the natural hill which slopes down to the south at approximately 3° overall. The site is bound to the west, north and east by local roads, namely Brabyn Street, Albert Street East and Webb/Gaggin Street respectively.

The site comprises an irregularly shaped property with numerous one-storey brick and metal clad buildings generally arrayed around a central asphalt paved courtyard in the northern portion of the site. South-west of the main building cluster are a few rows of demountable buildings. Buildings across the site generally appeared to be in good condition upon our cursory external observations, although some vertical cracking was observed in brick buildings towards the north-western corner of the site. A small asphalt paved on-grade carpark was located on the western site boundary with access from Brabyn Street. The south-eastern portion of the site comprises a grassed outdoor play area. Medium to large trees are located around the perimeter of and within the site.

The southern boundary of the site is shared with five residential properties (Nos. 22, 24, 26 and 28 Brabyn Street and No. 7 Gaggin Street) which each contain a one or two-storey residence. The residences are of brick or fibro construction. The buildings are generally set back more than 10m from the boundary however the residences at No. 28 Brabyn Street and No. 7 Gaggin Street are set back approximately 2.5m and 0.9m respectively from the boundary. The residence at No. 26 Brabyn Street appears to have a basement level carpark which is approximately 2.5m to 3m below the ground surface however the lateral extent of the basement towards the site is unknown.

5 SUBSURFACE CONDITIONS

The NSW Seamless Geology Version 2.4 indicates that the site is underlain by Ashfield Shale comprising *"black to dark-grey shale and laminite"*. This profile does not account for any earthworks that have occurred on the site or in-situ weathering of the bedrock. The Ashfield Shale is underlain by Hawkesbury Sandstone which is mapped to the north and west of the site.

The investigation encountered a generalised profile comprising relatively shallow fill overlying residual silty clay which transitioned to weathered sandstone bedrock at depths ranging from 1.0m to 2.2m. The bedrock generally comprised sandstone although bands of siltstone and laminite were encountered within the profile. A summary of the subsurface conditions encountered in the boreholes is provided below, however reference should be made to the attached borehole logs for specific details at each location.

Pavements and Fill

Asphaltic concrete pavements 40mm thick, were encountered from the surface in BH2 to BH5 inclusive. Underlying the pavements, and from the surface in the other boreholes, fill was encountered which extended to depths ranging from 0.1m to 0.75m. The fill below the pavements and within BH1 comprised gravelly sand, sandy gravel or sand with gravel which are typical of materials used as a base course for pavement construction. In BH6 the fill comprised silty sand, whilst in BH7 and BH8, the fill comprised low plasticity silty clay. The fill contained inclusions of igneous gravel. In BH7 the fill was assessed as poorly compacted.

Residual Silty Clay

Silty clay, assessed as residual in origin, was encountered below the fill in all boreholes. The clay was assessed as high plasticity and generally of hard strength, although some stiff to very stiff strength clay was encountered in the upper residual profile in BH7 and BH8. The residual clay generally contained inclusions of ironstone gravel.

Weathered Bedrock

Weathered bedrock, predominantly comprising sandstone, was encountered at depths ranging from 1m to 2.2m below existing surface levels. The level of the bedrock ranged from approximately RL22.6m (BH3) to RL19.6m (BH7) and generally appears to dip down towards the south and south-east from a relatively level shelf in the north-western portion of the site. The upper bedrock typically comprised extremely weathered sandstone which increased in strength with depth. Within the upper sandstone profile bands of laminite (interbedded siltstone and sandstone) and siltstone laminae were generally encountered. Such a profile is typical of bedrock near the geological contact between the Ashfield Shale and Hawkesbury Sandstone.

Sandstone bedrock of low or higher strength was encountered at depths ranging from 2.1m to 4.5m in the boreholes and refusal occurred on high strength bedrock at depths ranging from 5.6m to 6.5m in BH1, BH3 and BH4. Refusal at depths of 3.8m, 3.6m and 3.7m in BH6, BH7 and BH8 respectively may have occurred on medium to high strength bedrock due to the smaller rig used to drill these boreholes.

The following table provides our rock classification assessment for BH2 and BH5. The classification was completed in general accordance with Pells et al (2019). The rock classes are approximate only and will be dependent on footing/pile sizes.

Borehole Number	Depths (Reduced Levels) Class V Rock	Depths (reduced Levels) Class IV Rock	Depths (Reduced Levels) Class III or better Rock
2	4.0m to 5.6m (RL20.2 to RL18.6)	Not encountered	5.6m to 8.0m (RL18.6 to RL16.2)
5	2.8m to 5.1m (RL19.6 to RL17.3)	Not encountered	5.1m to 8.0m (RL17.3 to RL14.4)

Groundwater

All boreholes were dry on completion of auger drilling, except BH3 where seepage was measured at a depth of 3.3m during drilling, and standing water was measured at a depth of 3.2m on completion. Piezometer standpipes were installed within BH1, BH3 and BH6, and the following groundwater levels were measured.

Borehole	Depth to Groundwater (m)	Approximate Groundwater Level (mAHD)	Comment
BH1	4.4	16.8	Measured 3 days after drilling
BH3	3.2	21.2	Measured on completion of drilling
BH6	'Dry'	n/a	On completion of drilling

The groundwater levels recorded on the logs for BH2 and BH5 were measured on completion of coring and are therefore considered to be artificially high due to the introduction of water during the coring process.

5.1 Laboratory Test Results

The Atterberg Limit testing completed on the residual silty clay indicate they are of high plasticity. The linear shrinkage results indicate a high potential for shrink-swell movements with changes in moisture content.

The four-day soaked CBR tests on the residual clay returned values of 4% and 5%. The in-situ moisture contents of the clays were equal to their respective optimum moisture contents. During soaking, the samples swelled by 0.5% and 2% indicating the clays are reactive with respect to variations in moisture content.

The following table summarises the soil aggression tests.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity (ohm.cm)
BH1	1.5-1.8	XW Sandstone	4.7	130	<10	12,000
BH2	1.2-1.5	RESIDUAL Silty Clay	5.3	10	<10	39,000
BH3	1.5-1.92	RESIDUAL Silty Clay	5.8	20	<10	59,000
BH4	0.5-0.95	RESIDUAL Silty Clay	5.5	<10	10	19,000
BH7	0.5-0.75	RESIDUAL Silty Clay	6.3	28	<10	30,000
BH8	1.5-1.95	RESIDUAL Silty Clay	5.3	25	27	23,000

6 COMMENTS AND RECOMMENDATIONS

6.1 Site Classification

Due to the likely abnormal moisture conditions as a result of buildings, pavements and trees, we consider that the proposed building area will classify as Class 'P' in accordance with AS2870-2011 'Residential Slabs and Footings'. Therefore, all footings will need to be designed by engineering principles.

The use of AS2870-2011 will only be relevant to lightly-loaded structures within the scope defined by the code. For such structures, the laboratory testing of the residual silty clay soils indicates that they will likely have characteristic surface movements in the range equivalent to that of a Class 'H1' site under 'normal' conditions. Where footings are designed on the basis of AS2870-2011, consideration will also need to be given to the adverse effect on shrink-swell movements from trees which are scattered around the proposed development area.

If the residual silty clay soils are used as an engineered fill, or if excavations into the residual silty clays are carried out, then it is possible that characteristic surface movements will be greater, and may be closer to Class ‘H2’ type movements. As such further advice from the geotechnical engineers is recommended when details regarding the fill to be used to raise site levels is confirmed.

Reference should also be made to Appendix B of AS2870-2011, for guidance on appropriate site maintenance, including site drainage and planting of trees and shrubs.

6.2 Excavation Conditions

Based on the bulk earthworks plan, excavation for the proposed upgrades will generally not exceed 1m depth, although locally deeper excavation for the proposed OSD tank and Block R lift pit and buried rainwater tank will be required.

Based on the subsurface conditions encountered, excavation is expected to predominantly extend through soils. Within locally deeper excavations the upper weathered bedrock may be encountered. Excavation of the soils and any upper extremely weathered sandstone should be achievable using conventional earthmoving equipment, such as the buckets of hydraulic excavators. Ripping hooks or tynes may be required to break-up indurated bands within the extremely weathered profile.

Any excavation through sandstone bedrock of low or higher strength would require rock excavation techniques such as rock saws, rock grinders and/or hydraulic impact hammers. Hydraulic impact hammers should only be used if quantitative vibration monitoring is carried out at the commencement of percussive excavation. Where the vibration monitoring indicates that vibration limits are being exceeded at the boundaries or at existing buildings within the school then further advice will be required to reduce ground borne vibrations to acceptable levels or alternatively non-percussive excavation techniques could be adopted. The attached Vibration Emissions Design Goals sheet provides further information on ‘safe limits’ for ground-borne vibrations.

Only excavation contractors with experience in similar work using a competent supervisor who is aware of vibration damage risks should be used. The contractor should have all appropriate insurances.

Based on the results of the limited monitoring completed we do not consider that the proposed excavations will encounter the groundwater table, however groundwater seepage will likely occur at the soil/rock interface or through joints and defects within the rock within locally deeper excavations, particularly during or immediately following periods of wet weather. We expect that any seepage encountered will be controllable using gravity drainage and/or conventional sump and pump techniques.

Excavated spoil for off-site disposal will need to be suitably classified for waste disposal purposes. Reference should be made to the reports prepared by JK Environments.

6.3 Excavation Batters and Retaining Walls

6.3.1 Excavation Batters

Based on the proposed setbacks temporary batters should be feasible for the proposed excavations, which are less than 3m deep.

Temporary batters formed through fill, natural clayey soils and the upper weathered rock may be formed no steeper than 1 Vertical (V) in 1 Horizontal (H), subject to inspection by a geotechnical engineer.

Surcharge loads such as construction traffic, site sheds etc. should be no closer than 2H from the crest of any temporary batter, where H is the vertical height of the batter in metres. Surface drainage should not be allowed to flow over the crest of temporary batters, and should be directed and discharged in a manner which avoids concentrated flows and erosion.

Where retaining walls are to be constructed in front of temporary batters, care will be required in backfilling between the temporary batter slope and the new retaining wall. Uncontrolled backfilling will lead to large settlements which may adversely affect pavements, structures or landscaping areas. It is often difficult to achieve adequate compaction of backfill due to limited access and the need to use small hand compaction equipment. We recommend therefore the use of a single-sized durable gravel, such as “blue metal” gravel or crushed concrete (free of fines and with less than 10% brick), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope to control subsoil erosion. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required. The geotextile should then be wrapped over the surface of the gravel backfill and capped with at least a 0.5m thick compacted layer of clayey engineered fill.

Where permanent batter slopes are being proposed, the formation will be dependent on the height of the cut and the materials exposed. As a guide we suggest that for permanent batters through the residual clays and upper weathered sandstone these should be battered at not steeper than 1 Vertical (V) in 2 Horizontal (H).

Any permanent batters will need to be fully protected from erosion, in the long term, by a suitable and approved erosion protection measure. Suitable measures would include revegetation or shotcrete. Where revegetation is being proposed, consideration should be given to flattening the permanent batters even further than recommended above to assist with initial vegetation and topsoil establishment, to reduce the risk of topsoil washing from the face during heavy rainfall, and to provide for ease of maintenance.

6.3.2 Retaining Walls

Where temporary batter slopes are adopted and permanent retaining walls constructed within the excavation, we recommend that the following characteristic parameters may be adopted for preliminary shoring wall design. The following parameters are on the basis of either a properly placed and compacted engineered backfill or backfill comprising a uniform sized durable granular material which is surrounded in a geotextile fabric as discussed in Section 5.3.1 above.

- For cantilever walls where some movement can be tolerated, we recommend a triangular lateral earth pressure distribution using an 'active' earth pressure coefficient (K_a) of 0.35.
- For cantilever walls which will be propped by floor slabs or where movements are to be reduced, we recommend a triangular lateral earth pressure distribution using an 'at rest' earth pressure coefficient (K_0) of 0.6.
- A bulk unit weight of 20kN/m^3 may be used for the backfill.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design.

Measures must be taken to provide permanent and effective drainage of the ground immediately behind the basement walls. We recommend the use of a free draining durable aggregate (such as 20mm size blue metal) with 'agg' pipe surrounded by a geotextile at the base and connected to the stormwater drainage system.

6.4 Earthworks

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

For the proposed carpark pavements, games court and where slab on-grade construction is adopted for the proposed Block R the following subgrade preparation is recommended.

- Strip off the existing, grass, topsoil, root affected material, asphaltic concrete and any obvious deleterious fill materials. The root balls of any trees or shrubs should also be fully removed. Stripped materials will not be suitable for re-use as engineered fill and should be stockpiled separately. Such materials may be suitable for re-use within landscaped areas.
- Where new pavements are to be constructed close to existing levels the underlying granular roadbase material should also be stripped to expose the underlying soil subgrade. The existing granular roadbase material will not be suitable as a base-course below new pavements, however it would be suitable for use as an engineered fill to be placed in any areas of poor subgrade.
- Where fill is to be used to raise site levels and support building loads all existing fill would need to be stripped from the surface.
- The exposed subgrade should be proof rolled with 8 passes of a minimum 10 tonne smooth drum roller to detect any soft or heaving areas. The proof rolling should be carried out in the presence of a geotechnical engineer or experienced earthworks technician. The subgrade should be well graded to promote runoff and reduce the risk of water ponding on the surface. If the subgrade becomes wet it may become untrafficable.

- Any areas of heaving subgrade should be locally removed to a competent base and replaced with engineered fill, or stabilised by the use of bridging layers and geogrid reinforcement. Furthermore, specific subgrade improvement may be required and this is best determined in consultation with the geotechnical engineers at the time of construction.
- Engineered fill should comprise a good quality granular material, such as crushed sandstone or any existing granular road-base material, and should be compacted in horizontal layers with a maximum 200mm loose thickness to at least 98% of Standard Maximum Dry Density (SMDD). The weathered sandstone excavated from the site would be suitable for use as an engineered fill.
- While not preferred, the existing residual soils may also be used as engineered fill, provided they are compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD) and to within ±2% of Standard Optimum Moisture Content (SOMC). If the residual clay soils are to be adopted for use as an engineered fill the following needs to be carefully considered.
  - (i) Where clays have moisture contents greater than the plastic limit they will require drying out prior to their use as engineered fill, and
  - (ii) Where reactive clays are used as an engineered fill, they will undergo greater shrink swell movements with changes in moisture content than the in situ reactive clays. Therefore, consideration needs to be given to the effect that greater shrink-swell movements will have on the performance of structures founded above.

Density testing should be regularly carried out on any engineered fill. Regular density testing in accordance with Level 2 requirements of AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments' are recommended for pavements, however if engineered fill is required to support structural loads from buildings, then Level 1 supervision and testing should be carried out.

6.5 Footings

6.5.1 Block R

For the proposed Block R due to the anticipated structural loads, footings uniformly founded within the underlying sandstone bedrock will likely be required.

As minimal excavation is proposed for Block R piles will be required to found within the better-quality sandstone encountered at depth. Bored piers are likely to be feasible in the most part, provided significant groundwater seepage does not occur above the rock. In those circumstances the side walls of bored piles may collapse and alternative grout injected piles would be required.

For piled footings within sandstone bedrock of at least Class V quality an allowable bearing pressure (ABP) and allowable shaft adhesion (in compression) of 1,000kPa and 100kPa respectively may be adopted for preliminary design. Higher bearing pressures will be possible within the better-quality Class III sandstone bedrock which was encountered within BH2 and BH5, however additional cored boreholes would be required to confirm the depth and continuity of these materials. As a guide only, piles founded within Class III sandstone would typically be suitable for an ABP of at least 3,500kPa.

Piled footings designed for allowable end bearing pressures typically induce settlements at the pile toe of less than 1% of the pile diameter.

6.5.2 Lightweight Structures

Lightly loaded structures may be designed to found on the underlying residual soils. Shallow pad/strip footings or stiffened raft slabs would be feasible. Provided the structures are within the scope of AS2870-2011 'Residential Slabs and Footings' these footing systems could then be designed in accordance with that code. Other structures outside the scope of AS2870-2011, will need to be designed on the basis of engineering principles, taking into account the reactivity of the soils and the site conditions.

Where shallow footings are founded on residual silty clays of at least very stiff strength, we consider that a maximum allowable bearing pressure of 150kPa would be applicable. The settlement of shallow pad/strip footings will be dependent on the size of the footing, the strength of the founding material and the depth to any underlying rigid material (such as rock). As a guide, settlements in the order of 1% of the footing width can be assumed to apply, however shrink-swell movements as a result of the reactive clays are likely to be most critical to the design of shallow footings. Particular consideration will also need to be given to the effect of trees or reactive engineered fill as greater surface movements may apply.

If adopting shallow footings founded on the residual clays, consideration will need to be given to the potential for differential movements between other structural elements that may be founded on the underlying bedrock. We strongly recommend that these structures include good articulation to allow relative movements to occur. Reference should also be made to Appendix B of AS2870-2011 which provides further guidance on foundation performance and maintenance for structures on reactive silty clay soils.

6.5.3 Footing Inspections

During construction we recommend that all footings including piles be visually inspected by a geotechnical engineer to confirm that they are supported on a founding stratum consistent with these recommendations and structural design requirements. We note however that if piles need to be drilled using grout injected techniques, inspection will essentially only be able to deduce that the pile is founded at a level consistent with nearby borehole logs. In this regard, a suitable number of cored boreholes will need to be drilled across the site to provide a reasonable assessment on the bearing stratum.

Prior to pouring concrete, piles/footings will need to be dewatered, cleaned of all loose debris from the base, inspected and approved by the geotechnical engineers. We recommend the base of piles are cleaned with a cleaning bucket. Piles/footings will need to be poured as soon as possible after drilling. If piles are left open overnight, they must be redrilled prior to pouring concrete to remove any softened or other debris from the base of the pile.

6.6 Ground Floor Slabs

Following bulk excavation and earthworks, the subgrade for Block R is likely to comprise a combination of residual silty clay or engineered fill. Options for support of ground floor slabs include:

- Constructing the slabs on grade, or
- Designing the slabs as fully suspended.

Where the residual silty clays are encountered at subgrade level, and slabs on grade are proposed, we recommend that the subgrade be prepared in accordance with the recommendations outlined in Section 6.4 above. Similarly, if slabs on-grade are proposed and site levels are to be raised, then the fill below the slabs must comprise engineered fill. Where existing uncontrolled fill is present, and this fill is not removed, then the floor slabs will need to be constructed as suspended slabs.

Wherever slabs on-grade are supported on soils, the slabs should be separated from the structural footings and columns supported on the bedrock to allow relative movement (i.e. designed as floating slabs). These movements will likely largely be due to shrink-swell movements where the slabs are underlain by residual clay or clay fill. The extent of shrink-swell movements, as noted in Section 6.1 will depend on the earthworks completed at the building location and should be assessed following confirmation of the cut and fill depths and material to be used as engineered fill. To reduce the differential movements between the floor slabs and the building structure consideration could be given to replacing the residual clay subgrade with a non-reactive fill material and using such material where engineered fill is required to raise site levels.

For suspended slabs, the slabs will need to be founded on piers supported on the underlying bedrock as recommended above. For the portion of suspended slabs above a clay subgrade this will need to be underlain by degradable void formers of at least 50mm thickness to reduce the risk of swelling soils ‘jacking’ the slabs off the piles. Where fill is used to raise site levels and the slabs are designed as suspended slabs then the fill would not need to be placed as engineered fill.

6.7 Pavements

Following satisfactory preparation of the subgrade (as detailed in Section 6.4 above), new pavements will need to be designed on the basis of the specific subgrade material and in this regard, where pavements are to be supported on engineered fill then sampling and testing of the material should be completed to confirm the design CBR. Where pavements are supported on the underlying residual silty clays then they may provisionally designed on the basis of a soaked CBR of 4%. The location of a new carpark had not been confirmed at the time of our 2022 investigation therefore no testing within the current proposed carpark area was completed. In this regard, we recommend that CBR testing of the soils at the design subgrade level within the carpark be completed to confirm the design CBR value for this pavement.

Flexible pavements should be underlain by a good quality base-course layer comprising crushed rock to TfNSW QA specification 3051 unbound base material, or equivalent good quality and durable fine crushed rock compacted to at least 100% of Standard Maximum Dry Density (SMDD).

Concrete pavements should also be underlain by a subbase layer of at least 100mm thickness comprising DGB20 compacted to at least 100% of SMDD. This will reduce the risk of pumping of fines where clayey subgrades are encountered. Concrete pavements should be isolated from the structural columns to allow relative movement.

Consideration could be given to the use of subsoil drains along the high side of pavements. The subsoil drains should extend to a depth of at least 0.3m below the subgrade level and the drains should have adequate falls to reduce ponding in the drains.

6.8 Earthquake Classification

In accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia' the following parameters would apply to the site:

- Hazard factor (Z) = 0.08
- Site Subsoil Class = Class Ce

6.9 Exposure Classification

The soil aggression test results have indicated slightly acidic conditions, low sulfate and chloride contents, and relatively high resistivity values. In accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009 'Piling – Design and Installation' the exposure classification to concrete and steel piles is 'Mild and 'Non-aggressive' respectively. In accordance with Table 4.8.1 of AS3600:2018 'Concrete Structures', the exposure classification to concrete footings is 'A2'.

6.10 Additional Work

No mitigation measures are required to mitigate environmental impacts arising from the proposed activity. Geotechnical recommendations are made throughout this report and should be considered and adopted where appropriate by the design team and contractor:

- Drilling of additional cored boreholes to confirm the depth and consistency of sandstone bedrock, if higher bearing pressures are required.
- CBR testing of the subgrade within the proposed carpark.
- Assessment of shrink-swell movements by geotechnical engineer following confirmation of material to be used as fill.
- Inspection of proof rolling by an experienced geotechnical engineer or geotechnician.
- Vibration monitoring, if percussive excavation is carried out.

- In-situ density testing of all materials placed as engineered fill, including pavement materials.
- Inspection of footings by geotechnical engineer.

7 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 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.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed School Upgrade
Location: Parramatta East Public School, Parramatta, NSW

Report No.: 35073L - A
Report Date: 28/07/2022
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	0.50 - 0.95	15.6	56	19	37	15.0
1	3.00 - 4.00	11.8	-	-	-	-
1	4.50 - 5.00	6.7	-	-	-	-
2	1.50 - 1.95	13.3	-	-	-	-
2	3.00 - 3.45	7.9	-	-	-	-
3	0.50 - 0.95	24	70	22	48	14.0
3	2.40 - 3.00	5.3	-	-	-	-
3	4.20 - 5.00	6.4	-	-	-	-
4	2.80 - 3.00	4.4	-	-	-	-
4	5.20 - 5.50	6.3	-	-	-	-
4	6.00 - 6.50	4.2	-	-	-	-
5	0.40 - 1.30	16.9	68	24	44	18.5
5	2.80 - 3.00	5.8	-	-	-	-
5	3.20 - 3.50	4.5	-	-	-	-
6	2.10 - 2.80	6.3	-	-	-	-
6	3.40 - 3.50	5.0	-	-	-	-
7	3.20 - 3.40	5.9	-	-	-	-
8	3.00 - 3.20	7.6	-	-	-	-
8	3.30 - 3.40	5.9	-	-	-	-

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: 19/07/2022.
- 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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In full without approval of the laboratory. Results relate only to
the items tested or sampled.

28/07/2022
Authorised Signature / Date
(D. Treweek)

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Report No.:**	35073L - B
Project:	Proposed School Upgrade	**Report Date:**	26/07/2022
Location:	Parramatta East Public School, Parramatta, NSW	**Page 1 of 1**	

BOREHOLE NUMBER		BH 1	BH 5
DEPTH (m)		0.20 - 1.30	0.40 - 1.30
Surcharge (kg)		9.0	9.0
Maximum Dry Density (t/m^3)		1.66 STD	1.75 STD
Optimum Moisture Content (%)		21.9	16.9
Moulded Dry Density (t/m^3)		1.63	1.72
Sample Density Ratio (%)		98	98
Sample Moisture Ratio (%)		97	98
Moisture Contents			
Insitu (%)		21.9	16.9
Moulded (%)		21.3	16.6
After soaking and After Test, Top 30mm(%)		30.2	27.5
Remaining Depth (%)		23.7	22.9
Material Retained on 19mm Sieve (%)		0	0
Swell (%)		0.5	2.0
C.B.R. value:	@2.5mm penetration	5	
	@5.0mm penetration		4.0

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: 19/07/2022.
- Insufficient sample mass provided to complete a 4-point compaction curves.

NATA

NATA Accredited Laboratory
Number:1327

26/07/2022
Authorised Signature / Date
(D. Treweek)

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: School Infrastructure NSW

Ref No: 35073L

Project: Proposed School Upgrade

Report: C

Location: Parramatta East Public School, PARRAMATTA, NSW

Report Date: 14/07/22

Page 1 of 1

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	4.30 - 4.34	1.2	24	A
	4.96 - 4.99	1	20	A
	5.09 - 5.12	1.2	24	A
	5.96 - 6.00	0.8	16	A
	6.42 - 6.45	1	20	A
	6.96 - 7.00	0.6	12	A
	7.44 - 7.47	1.4	28	A
	7.96 - 8.00	1.9	38	A
5	3.88 - 3.90	1.1	22	A
	4.11 - 4.13	0.2	4	A
	4.65 - 4.68	0.5	10	A
	5.19 - 5.22	2.1	42	A
	5.75 - 5.78	1	20	A
	6.14 - 6.18	0.9	18	A
	6.71 - 6.73	2.3	46	A
	7.20 - 7.23	1.3	26	A
	7.67 - 7.71	1.9	38	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 300802

Client Details

Client	JK Geotechnics
Attention	Tom Foster
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35073LT, Parramatta**
Number of Samples	6 Soil
Date samples received	19/07/2022
Date completed instructions received	19/07/2022

Analysis Details

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

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

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

Report Details

Date results requested by	26/07/2022
Date of Issue	26/09/2022
Reissue Details	This report replaces R00 created on 26/07/2022 due to: sample ID error

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Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		300802-1	300802-2	300802-3	300802-4	300802-5
Your Reference	UNITS	BH1	BH2	BH3	BH4	BH7
Depth		1.5-1.8	1.2-1.5	1.5-1.92	0.5-0.95	0.5-0.75
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	25/07/2022	25/07/2022	25/07/2022	25/07/2022	25/07/2022
Date analysed	-	25/07/2022	25/07/2022	25/07/2022	25/07/2022	25/07/2022
pH 1:5 soil:water	pH Units	4.7	5.3	5.8	5.5	6.3
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10	10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	130	10	20	<10	28
Resistivity in soil*	ohm m	120	390	590	190	300

Misc Inorg - Soil		
Our Reference		300802-6
Your Reference	UNITS	BH8
Depth		1.5-1.95
Type of sample		Soil
Date prepared	-	25/07/2022
Date analysed	-	25/07/2022
pH 1:5 soil:water	pH Units	5.3
Chloride, Cl 1:5 soil:water	mg/kg	27
Sulphate, SO4 1:5 soil:water	mg/kg	25
Resistivity in soil*	ohm m	230

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	-			25/07/2022	[NT]	[NT]	[NT]	[NT]	25/07/2022	[NT]
Date analysed	-			25/07/2022	[NT]	[NT]	[NT]	[NT]	25/07/2022	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	99	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	93	[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.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

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.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~21.2 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			21				FILL: Sandy gravel, fine to coarse grained igneous, grey, fine to coarse grained sand, with concrete fragments.	M			
		N = 12 3,6,6	20	1		CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel, and root fibres.	w<PL	Hd	500 520 430	RESIDUAL
							Silty CLAY: high plasticity, light grey mottled red brown, trace of fine grained sand.				
		N > 11 9,11/ 150mm REFUSAL	19	2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey, trace of fine grained sand, with occasional ironstone bands.	XW	Hd	550 >600 >600	HAWKESBURY SANDSTONE
							SANDSTONE: fine to medium grained, grey and red brown.	DW	L		LOW RESISTANCE
ON 15/7/22			18 17	3 4			as above, but light grey and red brown, with extremely weathered bands and ironstone bands.		VL		VERY LOW TO LOW RESISTANCE WITH OCCASIONAL MODERATE STRENGTH BANDS
						-	LAMINITE: SANDSTONE: fine to medium grained, brown, interbedded with SILTSTONE: dark grey.		L - M		MODERATE RESISTANCE
			16	5		-	SANDSTONE: fine to medium grained, grey.		M - H		MODERATE TO HIGH RESISTANCE
									H		HIGH RESISTANCE
			15	6			END OF BOREHOLE AT 5.60 m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 5.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 5.5m TO 2.5m. CASING 2.5m TO 0m. 2mm SAND FILTER PACK 5.5m TO 2.3m. BENTONITE SEAL 2.3m

BOREHOLE LOG

Borehole No.
1
2 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~21.2 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
						14 13 12 11 10 9 8	8 9 10 11 12 13							TO 0.8m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~24.2 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	ASPHALTIC CONCRETE: 40mm.t	M			
	■			■		24				FILL: Mixture of Sand and Gravel, fine to coarse grained sand, and fine to coarse grained igneous gravel.				
	■			■	N = 13 5,6,7				CH	FILL: Gravelly sand, fine to medium grained, brown and grey, fine to medium grained igneous gravel.	w~PL	Hd	>600 400 460	RESIDUAL
				■			1			Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.	w<PL			
	■			■		23				as above, but light grey and red brown.				
	■			■	N = 26 7,12,14	22	2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey, trace of fine grained sand.	XW	Hd	580 >600 >600	HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
▽ ON COMPLETION OF CORING	■			■	N = 26 7,9,17	21	3			Extremely Weathered sandstone: silty CLAY, low plasticity, light grey, with fine to medium grained sand, interbedded with extremely weathered siltstone and occasional ironstone bands.			>600 >600 >600	
						20 19 18	4 5 6			REFER TO CORED BOREHOLE LOG				

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CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT
Core Size: NMLC
R.L. Surface: ~24.2 m
Date: 12/7/22
Inclination: VERTICAL
Datum: AHD
Plant Type: JK309
Bearing: N/A
Logged/Checked By: T.F./A.B.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness – Specific	Formation
ON COMPLETION OF CORING		21							
			4	START CORING AT 4.00m					
100% RETURN		20	4.00	SANDSTONE: fine to medium grained, light grey and orange brown, sub-horizontally bedded.	HW	M - H	1.2	(4.04m) CS, 0°, 40 mm.t (4.24m) CS, 0°, 50 mm.t	Hawkesbury Sandstone
				NO CORE 0.15m					
				SANDSTONE: fine to medium grained, light grey and red brown, sub-horizontally bedded.	HW	M VL		(4.49m) CS, 0°, 80 mm.t (4.70m) XWS, 40 mm.t (4.81m) CS, 0°, 80 mm.t	
		19	5	SILTSTONE: grey, bedded sub-horizontally.	MW	M	1.0 1.2	(4.92m) Be, 0°, P, R, Fe Sn	
				SANDSTONE: fine to medium grained, light grey and orange brown, bedded sub-horizontally.	HW	VL		(5.15m) Be, 0°, P, R, Clay Vn (5.20m) CS, 0°, 30 mm.t (5.48m) XWS, 60 mm.t	
				LAMINITE: SILTSTONE: grey, interbedded with SANDSTONE: fine to medium grained, light grey and orange brown, with medium strength bands, bedded sub-horizontally.	MW	M	0.80		
		18	6	SANDSTONE: fine to medium grained, light grey and orange brown, bedded sub-horizontally.		L		(6.17m) CS, 10 mm.t (6.29m) CS, 20 mm.t	
				SILTSTONE: dark grey, with fine grained, grey sandstone laminae.	SW	M	1.0	(6.59m) Be, 0°, P, R, Clay Vn	
				SANDSTONE: fine to medium grained, light grey and orange brown, bedded sub-horizontally.			0.60	(6.79m) Be, 5°, P, R, Clay Vn	
100% RETURN		17	7	as above, but medium grained and bedded at 5-10°.		H	1.4 1.9	(7.74m) Be, 10°, P, R, Fe Sn (7.92m) Be, 5°, R, Clay Vn	
		16	8	END OF BOREHOLE AT 8.00 m					
		15	9						

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 35073LT
Borehole No: BH2
Depth: 4.0 – 8.00m

JOB NO. 35073LT BH2 START CORING AT 4.0m

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~24.4 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	ASPHALTIC CONCRETE: 40mm.t				
						24			CH	FILL: Sandy gravel, fine to medium grained, igneous, grey, fine to coarse grained sand.	w>PL	Hd		RESIDUAL
					N = 8 4,4,4		1			Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.			440 420 410	
					N > 25 7,12,13/ 120mm REFUSAL	23				as above, but light grey and red brown.	w~PL		>600 >600 >600	
							2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey, trace of fine grained sand, and occasional ironstone bands.	XW	Hd		HAWKESBURY SANDSTONE
ON COMPLETION						22 21	3 4			SANDSTONE: fine to medium grained, light grey.	DW			VERY LOW 'TC' BIT RESISTANCE WITH BANDS OF LOW TO MODERATE RESISTANCE MODERATE TO HIGH RESISTANCE WITH BANDS OF VERY LOW TO LOW RESISTANCE
						20			-	LAMINITE: SANDSTONE: fine to medium grained, grey, interbedded with SILTSTONE: grey .		L		LOW TO MODERATE RESISTANCE
						19 18	5 6		-	SANDSTONE: fine to medium grained, light grey.		M H		MODERATE RESISTANCE HIGH RESISTANCE
										END OF BOREHOLE AT 6.50 m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL

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BOREHOLE LOG

Borehole No.
3
2 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~24.4 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
						17 16 15 14 13 12 11	8 9 10 11 12 13							INSTALLED TO 6.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.1m TO 3.1m. CASING 3.0m TO 0.1m. 2mm SAND FILTER PACK 6.1m TO 2.8m. BENTONITE SEAL 2.8m TO 1.8m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

BOREHOLE LOG

Borehole No.
4
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~23.8 m
Date: 12/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION									-	ASPHALTIC CONCRETE: 40mm.t	D			
										FILL: Gravelly sand, fine to medium grained, grey brown, fine to coarse grained igneous gravel.				
					N = 11 4,5,6	23	1		CH	Silty CLAY: high plasticity, orange brown and red brown, trace of fine to medium grained ironstone gravel.	w<PL	Hd	>600 470 540	RESIDUAL
										Silty CLAY: high plasticity, red brown and light grey.				
					N > 16 10,10,6/ 20mm REFUSAL	22	2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey and red brown, trace of fine grained sand, with occasional ironstone gravel bands.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE WITH BANDS OF LOW TO MODERATE RESISTANCE
						21	3			SANDSTONE: fine to medium grained, light grey.	DW	L - M		LOW TO MODERATE 'TC' BIT RESISTANCE
												VL - L		
						20	4					L - M		MODERATE TO HIGH RESISTANCE WITH BANDS TO VERY LOW TO LOW RESISTANCE
						19	5			LAMINITE: SILTSTONE: light grey, interbedded with SANDSTONE: fine grained, grey and brown.	DW	L - M		MODERATE RESISTANCE
						18	6			SANDSTONE: fine to medium grained, light grey.	DW	M		MODERATE TO HIGH RESISTANCE
												H		HIGH RESISTANCE
						17				END OF BOREHOLE AT 6.50 m				'TC' BIT REFUSAL

BOREHOLE LOG

Borehole No.
5
1 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~22.4 m
Date: 13/7/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						-	ASPHALTIC CONCRETE: 40mm.t				
							FILL: Gravelly sand, fine to coarse grained, grey brown, fine to medium grained igneous gravel.				
		N = 16 4,5,11	22			CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.	w<PL	Hd	570 540 >600	RESIDUAL
				1			as above, with with ironstone band.				
			21				as above, but light grey and red brown.				
		N > 6 7,6/ 50mm REFUSAL	20	2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey and red brown, trace of fine grained sand and occasional bands of ironstone.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE BANDED VERY LOW TO LOW 'TC' BIT RESISTANCE
ON COMPLETION OF CORING			19	3		-	LAMINITE: SANDSTONE: fine grained, light grey and red brown, interbedded with SILTSTONE: dark grey.	DW	VL - L		LOW TO MODERATE RESISTANCE
			18 17 16	4 5 6			REFER TO CORED BOREHOLE LOG				

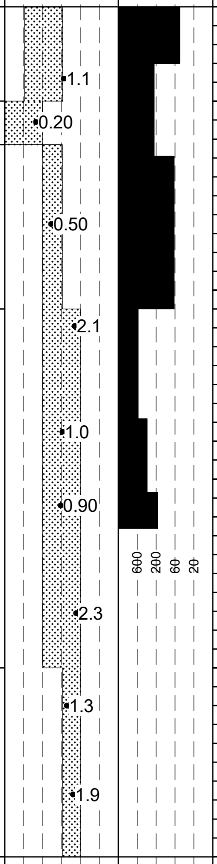
CORED BOREHOLE LOG

Borehole No.
5
2 / 2

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT
Core Size: NMLC
R.L. Surface: ~22.4 m
Date: 13/7/22
Inclination: VERTICAL
Datum: AHD
Plant Type: JK309
Bearing: N/A
Logged/Checked By: T.F./A.B.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		19			START CORING AT 3.50m						
100% RETURN					SANDSTONE: fine to medium grained, red brown and light grey, bedded at 5-15°.	MW	L - M	1.1	(3.54m) CS, 30 mm.t (3.57m) XWS, 10 mm.t (3.58m) CS, 70 mm.t (3.65m) XWS, 20 mm.t (3.67m) CS, 30 mm.t (3.80m) Be, 15°, P, R, Fe Sn		Hawkesbury Sandstone
			4		LAMINITE: SILTSTONE: dark grey, interbedded with SANDSTONE: fine grained, grey and orange brown, bedded at 0-5°.	HW	VL - L	0.20	(4.00m) XWS, 50 mm.t		
		18			SANDSTONE: fine to medium grained, light grey and red brown, bedded at 0-5°.	MW	M	0.50	(4.29m) CS, 10 mm.t (4.30m) XWS, 30 mm.t (4.33m) CS, 30 mm.t (4.41m) CS, 20 mm.t (4.56m) Be, 2°, P, R, Fe Sn (4.60m) CS, 20 mm.t (4.69m) Be, 0°, P, R, Fe Sn (4.72m) CS, 10 mm.t (4.78m) Be, P, R, Fe Sn (4.81m) Be, P, R, Fe Sn (4.84m) XWS, 30 mm.t (4.92m) Be, 0°, P, R, Fe Sn		
		17	5		as above, but light grey, with occasional grey laminae.	SW	M - H	2.1 1.0 0.90 2.3	(5.10m) Be, 0°, P, R, Fe Sn (5.68m) Be, 5°, P, R, Fe Sn (6.07m) CS, 5 mm.t (6.26m) Be, 0°, P, R, Fe Sn		
100% RETURN		16 15	6 7			FR	H	1.3 1.9	(7.15m) CS, 2 mm.t		
		14 13	8 9		END OF BOREHOLE AT 8.00 m						



JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 35073LT PARRAMATTA.GPJ <<DrawingFile>> 26/09/2022 12:55 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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Job No: 35073LT

Borehole No: BH5

Depth: 3.50 – 8.00m

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BOREHOLE LOG

Borehole No.
6
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~20.9 m
Date: 15/7/22 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION										FILL: Silty sand, fine to medium grained, grey brown, fine to coarse grained igneous gravel, and root fibres.	M			GRASS COVER
					N = 12 4,5,7	20			CH	Silty CLAY: high plasticity, orange brown, trace of fine to coarse grained ironstone gravel.	w<PL	Hd	450 470 410	RESIDUAL
					N > 16 8,12,4/ 10mm REFUSAL	19	1 2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, light grey, trace of fine to medium grained sand, with fine to medium grained sandstone bands and occasional ironstone gravel bands.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT RESISTANCE
						18	3			SANDSTONE: fine to medium grained, light grey and red brown.	DW	L - M		MODERATE TO HIGH RESISTANCE
										as above, but with interbedded siltstone, grey and dark grey.		M		BANDS OF LOW RESISTANCE
										SANDSTONE: fine to medium grained, light grey and grey.		H		MODERATE TO HIGH RESISTANCE HIGH RESISTANCE
						17 16 15 14	4 5 6			END OF BOREHOLE AT 3.80 m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 3.7m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.7m TO 2.2m. CASING 2.3m TO 0m. 2mm SAND FILTER PACK 3.7m TO 2.2m. BENTONITE SEAL 2.2m TO 1.1m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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BOREHOLE LOG

Borehole No.
7
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~21.7 m
Date: 15/7/22 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■ ■			■	N = 2 1,0,2	21				FILL: Silty clay, low plasticity, brown and grey, trace of fine grained sand, fine to medium grained igneous gravel, and root fibres.	w>PL			APPEARS POORLY COMPACTED
	■			■			1		CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.	w<PL	St - VSt	270 180 310	RESIDUAL
				■	N = 20 4,8,12	20	2			as above, but light grey and red brown.		Hd	>600 570 610	
	■			■	N=SPT 12/ 150mm REFUSAL	19	3		-	Extremely Weathered sandstone: silty CLAY, low to medium plasticity, light grey and red brown, trace of fine grained sand, with occasional ironstone bands.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
	■			■						SANDSTONE: fine to medium grained, grey and red brown.	DW	M - H		HIGH RESISTANCE
						18 17 16 15	4 5 6			END OF BOREHOLE AT 3.60 m				'TC' BIT REFUSAL

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 35073LT PARRAMATTA.GPJ <<DrawingFile>> 26/09/2022 13:01 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

BOREHOLE LOG

Borehole No.
8
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL UPGRADE
Location: PARRAMATTA EAST PUBLIC SCHOOL, NORTH PARRAMATTA, NSW

Job No.: 35073LT **Method:** SPIRAL AUGER **R.L. Surface:** ~22.9 m
Date: 15/7/22 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./A.B.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION										FILL: Silty clay, low plasticity, grey brown, trace of fine grained sand, fine to medium grained igneous gravel, and root fibres.	w>PL			GRASS COVER
					N = 8 4,4,4				CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained ironstone gravel.	w>PL	St	150 110 120	RESIDUAL
						22	1				w<PL	Hd	540 520	
					N = 20 5,9,11	21	2			as above, but light grey and red brown.			>600 >600 >600	
									-	Extremely Weathered sandstone: silty CLAY, low plasticity, red brown and light grey, with fine to coarse grained ironstone gravel bands.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
						20	3			SANDSTONE: fine to medium grained, light grey and red brown.	DW	L - M		HIGH RESISTANCE
												H		
						19 18 17 16	4 5 6			END OF BOREHOLE AT 3.70 m				'TC' BIT REFUSAL

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AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the report.

Title:

SITE LOCATION PLAN

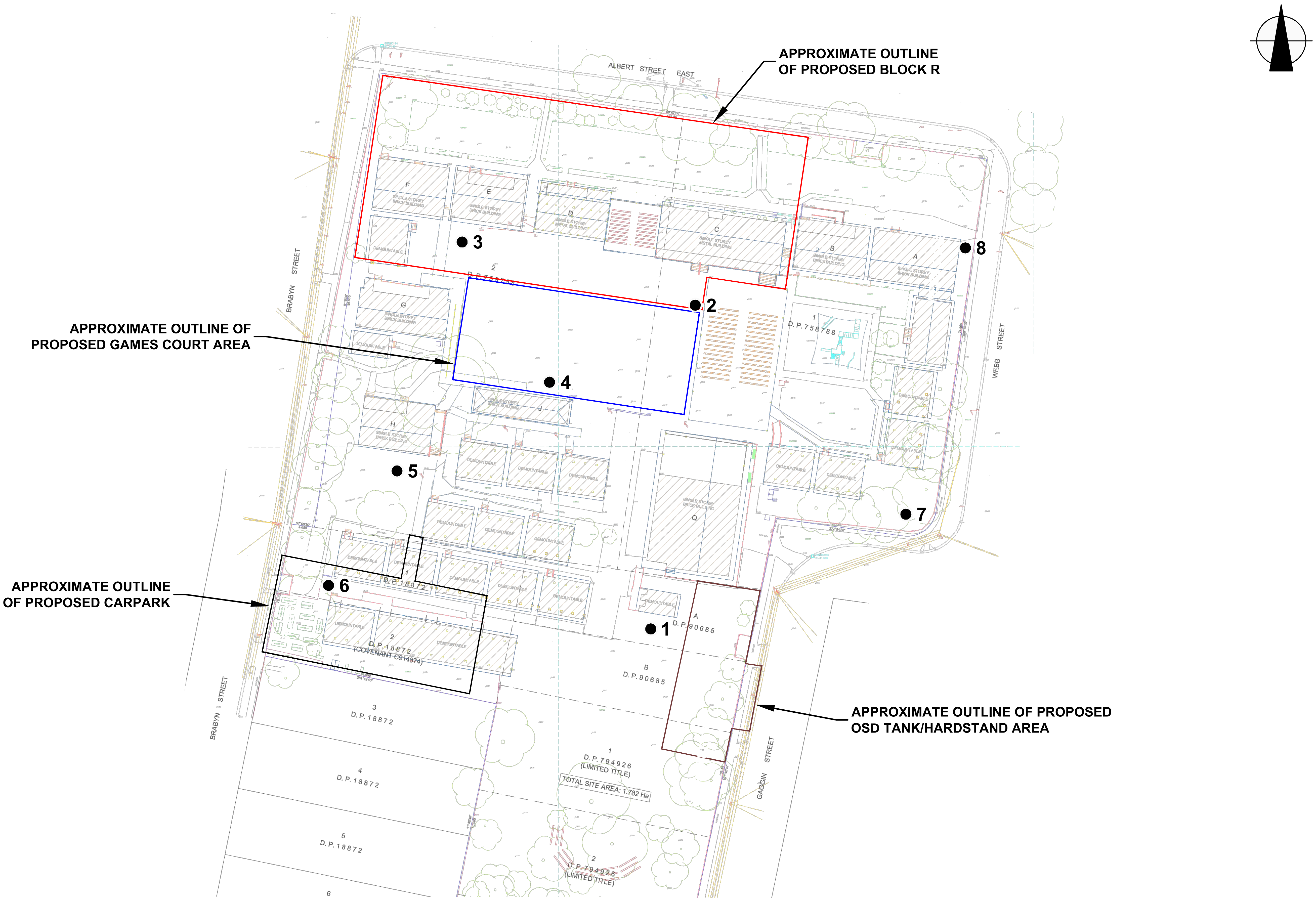
Location: PARRAMATTA EAST PUBLIC SCHOOL, BRABYN STREET,
NORTH PARRAMATTA, NSW

Report No: 35073LT

Figure No: 1

JKGeotechnics

PLOT DATE: 17/02/2025 1:10:45 PM   DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35073LT PARRAMATTA\CAD\35073LT.DWG



APPROXIMATE OUTLINE OF PROPOSED BLOCK R

APPROXIMATE OUTLINE OF PROPOSED GAMES COURT AREA

APPROXIMATE OUTLINE OF PROPOSED CARPARK

APPROXIMATE OUTLINE OF PROPOSED OSD TANK/HARDSTAND AREA

0 8 16 24 32 40

SCALE 1:800 @A3 METRES

This plan should be read in conjunction with the report.

Title:

BOREHOLE LOCATION PLAN

Location: PARRAMATTA EAST PUBLIC SCHOOL, BRABYN STREET, NORTH PARRAMATTA, NSW

Report No: 35073LT

Figure No: 2

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PLOT DATE: 17/02/2025 1:10:55 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35073LT PARRAMATTA\CAD\35073LT.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed 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:

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

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:

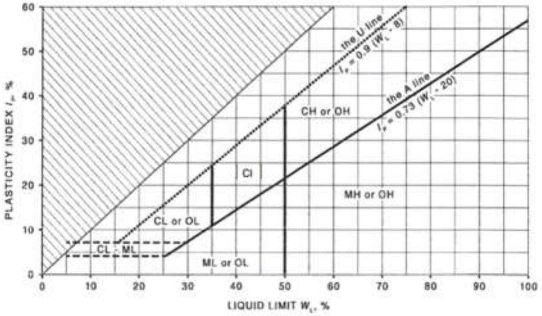
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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	Sample taken over depth indicated, for environmental analysis.		
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.		
	DB	Bulk disturbed sample taken over depth indicated.		
	DS	Small disturbed bag sample taken over depth indicated.		
	ASB	Soil sample taken over depth indicated, for asbestos analysis.		
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.		
	SAL	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	Vane shear reading in kPa of undrained shear strength.		
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).		
Moisture Condition (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained Soils)	D	DRY – runs freely through fingers.		
	M	MOIST – does not run freely but no free water visible on soil surface.		
	W	WET – free water visible on soil surface.		
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	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

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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