JC GEOTECHNICS

3RD DECEMBER 2020

Freeburnville Pty Ltd

GEOTECHNICAL INVESTIGATION REPORT

Proposed Residential Development

96-98 Lethbridge Street & 42-46 Evan Street

Penrith, NSW

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1 INTRODUCTION

JC Geotechnics Pty Ltd (**JCG**) was commissioned by Freeburnville Pty Ltd (**Freenburnville**) via returned Proposal Acceptance Record dated 9 November 2020, to carry out a geotechnical investigation at 96-98 Lethbridge Street & 42-46 Evan Street, Penrith, NSW for a proposed residential development. The commission was based on our proposal (Ref: GP2020-251) dated 30 September 2020.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at four (4) cored borehole locations to assist with the planning and design of the proposed development.

This report presents the factual results of the geotechnical investigation, interpretation and assessment of the existing geotechnical conditions at the site as a basis for comments and recommendations regarding dilapidation surveys, excavation, vibration considerations, excavation support, hydrogeological considerations, and footing design.

To assist in reading this report, reference should be made to “About Your Report” enclosed after the body of the text (**Appendix A**).

The following documents were supplied to assist in the report preparation.

- Architectural plans for Project No. 19107, Drawing Nos. DA-101, DA-102, DA-103, DA-104 and DA-105, All Revision A, dated 19 August 2020, prepared by Urban Link.

Based on the above information, we understand that the proposed development comprises demolition of the existing site structures and construction of two, five-storey apartment blocks over two levels of basement car parking. The finished floor level of the lowest proposed basement (which is common to the two apartment blocks) is not shown on the provided drawings, however we infer that excavation to a maximum depth of approximately 7m below existing surface levels may be required.

2 SCOPE OF WORK

Prior to work commencing, a Safe Work Method Statement (SWMS) for the work was prepared by JCG. All safety measures and precautions were followed in accordance with the SWMS.

The proposed borehole locations (which were dictated by access constraints at the time of the fieldwork) were assessed with reference to ‘Dial Before You Dig’ (DBYD) plans and scanned using electromagnetic detection techniques for the presence of buried services by a specialist subcontractor.

A site walkover inspection was carried out by our geotechnical engineer in order to assess the overall surface conditions and to identify relevant site features.

The fieldwork comprised the drilling of four boreholes (BH1 to BH4) using spiral auger drilling techniques, to depths between 2.4m (or RL of about 32.9m) and 8.2m (or RL of about 28.6xm), below existing surface levels.

The boreholes were then extended using NMLC sized rotary coring methods with water flush, to depths between 8.55m (or RL of about 27.15m) and 10.30m (or RL of about 26.5m), below existing surface levels.

Groundwater observations were made in the boreholes during and on completion of auger drilling. Introduction of water to the boreholes for rotary coring precluded further measurement of groundwater levels during the fieldwork, however Class 18 PVC piezometers (groundwater monitoring wells) were installed in two of the boreholes (BH1 and BH4) for the purpose of further groundwater monitoring.

The strength of the soils was assessed from the results of Standard Penetration Testing (SPT) 'N values' and hand penetrometer testing on cohesive soil samples recovered from the SPT split tube sampler.

The strength of the shale bedrock encountered during auger drilling was assessed by observation of the resistance to auger penetration while drilling with a tungsten carbide drilling bit ('TC bit').

The strength of the recovered rock core was assessed from tactile assessment on site during the fieldwork and compared with the results of subsequent Point Load Strength Index (PLSI) testing.

A geotechnical engineer from **JCG** was present full-time on site for the duration of the fieldwork to set out the test locations, log the encountered subsurface profile and nominate in-situ testing and sampling.

The borehole locations are shown on **Figure 1**, which is attached to **Appendix B**. The borehole logs together with coloured photographs of the rock cores are attached to this report as **Appendix C**). Reduced Levels (RLs) of the existing ground surface at each borehole location were interpolated between spot level heights shown on the provided Stormwater Concept Plan and are considered approximate.

Selected samples were returned to Macquarie Geotech, a NATA registered laboratory, for Atterberg Limits, Linear Shrinkage and Point Load Strength Index testing. Selected samples were also returned to ALS Environmental, a NATA registered laboratory, for testing to determine soil pH, sulfate content, chloride content and electrical conductivity. The results of the laboratory testing are attached to this report as **Appendix D**.

Explanatory notes, relating to abbreviations used in the presentation of borehole logs, etc. are included in **Appendix E**.

3 INVESTIGATION RESULTS

3.1 Site Description

The site spans the base of a shallow gully among gently undulating topography. It is 'L shaped' and bound by Evan Street to the west and Lethbridge Street to the north, isolating four single residential properties adjacent to the intersection of Evan and Lethbridge Streets.

To the east of the site are single residential dwellings and townhouses which are typically single storey brick or brick and weatherboard dwellings with tiled roofs, offset about 4m to 5m from the common boundaries. To the south of the site and to the north of the site in the vicinity of Evan Street, there were single residential dwellings and sheds which were offset approximately 1m from the common boundaries.

At the time of the investigation, the site contained six residential dwellings which were typically brick and weatherboard structures with tiled roofs and appeared to be in fair condition based on a cursory inspection. The central portion of the site also contained a disused tennis court, two disused in-ground swimming pools and a heavily vegetated park/easement which was grass covered and contained numerous large sized trees. We infer that a stormwater drainage pipe likely connects a concrete lined open channel (to the east of the site) with further drainage infrastructure beneath and beyond Evan Street (to the west of the site). the open concrete lined channel runs in an East-West direction and is about 1m in depth. The concrete lining appeared to be in good condition based on a cursory inspection from within the site. The channel was dry at the time of our presence on site.

‘Dial Before You Dig’ plans also indicate that two sewer pipes pass through the central portion of the site, each oriented approximately east to west. We infer that the sewer pipes and the stormwater pipe described above likely follow the path of an old concrete lined open channel, and possibly an old creek line that would have predated the open channel.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Series Sheet 9030 Edition 1 (dated 1991), Penrith by the Geological Survey of New South Wales, Department of Mineral Resources, indicates that the site is underlain by ‘Black to dark-grey shale and laminite’ of the Wianamatta Group, near the boundary of an area characterised by Quaternery aged soil deposits, ‘Gravel, sand, silt, clay’.

This geological profile does not take into account the residual soils derived from in-situ weathering of the bedrock or earthworks (filling) that have previously been undertaken at the site. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface profile at the borehole locations.

A general discussion of the encountered subsurface conditions is presented below:

Fill was encountered in all boreholes from the existing ground surface to depths between 0.1m (BH4) and 0.4m (BH2). The fill comprised silty clay of low plasticity with traces of roots (topsoil).

Residual silty clay of medium to high or high plasticity was encountered below the fill in all boreholes, extending to depths between 1.7m (BH2) and 6.2m (BH4), below existing surface levels. The residual soil was assessed to be of stiff to very stiff strength, generally increasing to ‘hard’ near the surface of the underlying shale bedrock, however at the location of BH4, the near surface alluvial soils were initially ‘hard’, decreasing in strength to ‘firm’ or ‘stiff’ approximately 1.1m below existing surface levels. The latter is likely the result of the presence of an old creek.

Weathered shale bedrock was encountered at depths of about 1.85m (BH1), 1.7m (BH2), 4.0m (BH3) and 6.2m (BH4), below existing surface levels. On first contact, the shale was assessed to be distinctly weathered and of very low strength. Distinctly weathered shale bedrock of at least low strength was encountered from depths of about 3.4m (or RL of about 32.3m), 4.35m, 7.2m (or RL of about 29.6m) and 7.3m (or RL of about 27.6m) at the locations of BH1, BH2, BH3 and BH4, respectively. The bedrock has been classified according to P.J.N. Pells et al (1998) as per **Table 1** below.

BH \ Rock Class	Approximate Depth/RL to Top of Class V (m)	Approximate Depth/RL to Top of Class IV (m)	Approximate Depth/RL to Top of Class III (m)
BH1	1.85/33.8	3.4*/32.3	5.2*/30.5
BH2	1.7/33.6	4.51/30.8	4.91/30.4
BH3	4.0/32.8	7.2*/29.6	9.10/27.7
BH4	6.2/28.7	Not encountered	8.19/26.7

*Inferred.

^require the drilling of additional cored boreholes with spoon testing.

Groundwater

Groundwater was not observed in BH1, BH2 or BH3 during auger drilling. Groundwater seepage was encountered in BH4 during auger drilling at about 3m depth, and we anticipate that the ground water level at the location of BH4 will be less than 3m below existing surface levels. We note that groundwater levels wouldn’t have been stabilised within the limited time of observation. Water flush used during the coring process precluded further monitoring of the groundwater levels. However, the groundwater levels were measured during our site visit on 3rd December 2020 at a depth of about 2m (or RL of about 33.7m) in BH1 and 1.7m (or RL of about 33.2m) in BH4, below existing surface levels.

3.3 Laboratory Testing

The four samples submitted to ALS for soil chemistry testing returned pH values between 5.4 and 8.4, indicating moderately acidic to mildly alkaline conditions. The sulphate contents ranged between 100ppm and 220ppm, and the chloride concentrations ranged between 970ppm and 1640ppm. The electrical conductivity of the samples tested ranged between 687 to 944 microsiemens/centimetre.

In accordance with Table 6.4.2(C) of AS2159-2009, we recommend that all buried concrete elements be designed for at least ‘mildly-aggressive’ conditions based on the results of the aggressivity laboratory testing.

The results of the Atterberg Limits and linear shrinkage testing undertaken by Macquarie Geotech indicated that the sample tested was of high plasticity, with a high potential for reactivity to changes in moisture content.

The results of Point Load Strength Index testing on selected rock core samples correlated well with our tactile field assessment of rock strengths. The estimated Unconfined Compressive Strengths (UCSs) of the rock core ranged from approximately 2MPa to 20MPa.

The results of all laboratory testing are attached as **Appendix D**.

4 COMMENTS AND RECOMMENDATIONS

We recommend that a meeting be held after initial structural design has been completed to confirm that our recommendations have been correctly interpreted. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

4.1 Dilapidation

Prior to demolition and excavation, we recommend that detailed dilapidation surveys be carried out on all structures, buried services and infrastructure (including the open concrete lined channel) present within the zone of influence of the proposed excavation. The zone of influence of the excavation is defined by a distance back from the excavation perimeter of twice the total depth of the excavation. The dilapidation reports would provide a record of existing conditions prior to commencement of the work. A copy of each report should be provided to the respective property owner who should be asked to confirm that it represents a fair assessment of existing conditions. The reports should be carefully reviewed prior to demolition and excavation.

4.2 Excavation

Prior to any excavation commencing, we recommend that reference be made to the WorkCover Excavation Work Code of Practice current at the time of construction.

Based on boreholes information, it is expected that the proposed basement excavation will extend through fill, residual soils and into shale bedrock. As batters generally do not appear to be feasible within the site boundaries, an engineered retention system must be installed for the full height of the proposed excavation, extending below bulk excavation level for lateral toe restraint, prior to excavation commencing.

The fill materials, residual and alluvial soils, as well as any very low strength shale bedrock could be excavated using buckets of large hydraulic conventional earthmoving equipment particularly when fitted with ‘tiger teeth’. Some assistance from ripping tynes may be required for iron indurated bands within the soils and extremely weathered shale.

Excavation of low and higher strength shale will require the use of rock excavation equipment. Rock excavation equipment could include hydraulic excavators with ripping tynes, impact hammers, rock saws and rock grinder attachments. Care will be required during excavation to control the transmission of ground vibrations where rock hammers are employed. We recommend that the boundary faces of the excavation be saw cut to minimise overbreak and instability. The saw cuts should extend below the level at which rock breakers are used to reduce transmitted vibration.

Groundwater seepage into the excavation is likely to occur given the depth of the proposed excavation and the presence of Class V shale, as well as alluvial soils and shallow groundwater encountered near the inferred old creek line. Groundwater seepage may occur at the soil/rock interface or through joints and defects within any rock, particularly during or immediately following periods of wet weather. We expect that any seepage encountered will be able to be controlled using conventional sump and pump techniques. However, the seepage towards the central portion of the site (i.e. around the channel) is likely to be of high order and this should be allowed for during construction.

Monitoring of the actual seepage rates should be carried out during excavation and basement construction works prior to finalising the design of the permanent dewatering system. Outlets into the stormwater system for both the short and / or long term will require Council approval.

Existing buried services which run below the site will require diversion prior to the commencement of excavation. We understand that a large existing sewer (400mm vitreous clay pipe) and a smaller sewer (150mm vitreous clay pipe) are present, passing through the footprint of the proposed excavation at approximate depths of 2m and 1.5m, respectively, below existing levels as shown on 'Dial Before You Dig' drawings by Sydney Water. Diversion of the existing pipes (and all sewer inlets) will need to be completed prior to commencement of the basement excavation and subject to approval by Sydney Water. Enquiries should be made for further information and details on the buried services, such as invert levels.

We also note that Sydney Water may require a finite element analysis be carried to assess the potential impact of the proposed development on the existing 200 DICL water main than runs along Evans and Lethbridge Streets. We can complete the analysis and reporting, if commissioned to do so.

4.3 Vibration Control

We recommend that quantitative vibration monitoring be completed to confirm that peak particle velocities (PPV) fall within acceptable limits. Other movement sensitive infrastructure within the road reserve and site e.g. the concrete channel may also require vibration monitoring to reduce the risk of damage to infrastructure. We note that vibration limits will reduce the risk of vibration damage to the neighbouring buildings and structures, however vibrations may still be perceptible to occupants of neighbouring buildings. If excessive vibrations are identified by the monitoring then it will be necessary to use lower energy equipment such as smaller rock hammers and/or using rock saws to cut gridlines within the shale, maintaining the base of the slots below the level at which the rock hammer is being used. Full time monitoring should be used at this site if rock breakers are to be used to protect all parties from inadvertent exceedances of tolerable vibration limits.

Where rock hammers are used, to reduce vibrations we recommend that the rock hammer be continually orientated towards the face, edges and points of chisels/moils be maintained and hammers to be operated one at a time and in short bursts only to reduce potential amplification of vibrations.

We recommend that only the services of excavation contractors with suitable experience and importantly with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. be engaged. The contractor should be provided with a copy of this report (and any subsequent reports) and have all appropriate statutory and public liability insurances.

4.4 Excavation Support

If space allows, for temporary stability, we recommend that the fill, residual clayey soils and weathered shale may be battered or benched at an angle of no steeper than 1Vertical (V) to 1 Horizontal (H). flatter batters will be required below groundwater levels. We do not recommend the use of temporary batters for the central portion of the site due to the presence of alluvial soils and high groundwater levels. Where battering of the excavation sides is not feasible or not preferred, the excavation must be supported by full depth engineered retaining walls installed prior to the commencement of excavation.

We forewarn that if a battered excavation is adopted, then backfilling of the permanent basement walls must be completed using appropriate compacted engineered fill, or future consolidation of the fill profile could result in extensive remediation works being required. Engineered fill comprising site won clays or ripped shale should be compacted to a density ratio of between 98% and 102% of Standard Maximum Dry Density (SMDD) at a moisture content within 2% of Standard Optimum Moisture Content (SOMC). Due to the limited size of equipment that will be able to operate behind the basement retaining walls, the maximum particle size of the engineered fill must not exceed 40mm and the fill must be compacted in maximum 100mm loose thickness layers.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. All density testing must be completed over the full thickness of each compacted fill layer. The frequency of density testing for engineered backfill behind retaining walls should be at least one test per two layers per 50 linear m of wall.

Compaction of engineered fill behind retaining walls can be problematic and the use of a single sized durable gravel, such as “blue metal” or crushed concrete gravel (free of fines), which do not require significant compactive effort could be considered if good performance is a priority. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in 200mm thick loose layers. Free draining backfill material must be separated from the in-situ soils by a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against 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 0.3m thick compacted layer of clayey engineered fill.

We anticipate that an appropriate retention system for this project may comprise a propped or anchored cut-off shoring pile walls.

The following parameters may be used for static design of temporary and permanent retaining walls at the subject site:

For progressively anchored or propped walls where minor movements can be tolerated (provided there are no buried movement sensitive services), we recommend the use of a trapezoidal earth pressure distribution of 4HkPa for fill materials, soil and shale of less than low strength, where H is the retained height in meters. This pressure should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom.

For progressively anchored or propped walls which support areas that are highly sensitive to movement (such as areas where movement sensitive structures or infrastructures or buried services are located in close proximity) we recommend the use of a trapezoidal earth pressure distribution of 8HkPa for fill materials, soil and weathered shale bedrock of less than low strength, where 'H' is the retained height in meters. This pressure should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom.

All surcharge loading affecting the walls (temporary batters, construction equipment, construction loads, adjacent high-level footings, inclined surfaces, etc.) should be also be incorporated in the retaining wall design.

The above earth pressures for anchored and propped walls assume horizontal backfill surfaces behind the shoring.

We also note that compaction of the backfill material will impose additional stresses on retaining walls which must be considered in the retaining wall design. A rectangular lateral earth pressure distribution of 15kPa should be adopted from ground surface level down to the point where such a distribution meets the appropriate earth pressure distribution from above, based on the use of small vibratory plate 'sled' compactors or upright rammer compactors to compact the retaining wall backfill. If larger compaction equipment is adopted for the compaction of the retaining wall backfill, higher compaction induced stresses on the retaining wall could result.

The retaining walls should be designed for full hydrostatic pressure.

For piles embedded into shale bedrock of at least low strength (Class IV or better) below bulk excavation level, an allowable lateral toe resistance value of 150kPa may be adopted. For piles embedded into Class III (medium strength or better) shale bedrock below bulk excavation level, an allowable lateral toe resistance value of 300kPa may be adopted. This value assumes excavation is not carried out within the zone of influence of the wall toe and the rock does not contain adverse defects etc. The upper 0.3m depth of the toe socket should not be taken into account in the design, to allow for tolerance and disturbance effects during excavation.

We expect that bored pile holes for the shoring system in the vicinity of the stormwater and sewer pipes (old creek line) will encounter difficulties. However, the use of temporary/sacrificial liners together with tremie techniques may assist in overcoming such difficulties. If bored piers are adopted, we recommend that trial piers be drilled to assess their suitability. Alternatively, grout injected (CFA) piles should be used. The concrete should also be a tremie mix.

We recommend that at least two additional cored boreholes, with groundwater monitoring wells, be drilled near the south eastern corner of the site (following demolition and removal of vegetation to facilitate access) and near the street frontage at No. 40 Evan Street. The additional boreholes would help to optimise the design of the shoring system and provide additional groundwater information. We note that variation in the subsurface conditions encountered during drilling for this investigation was observed particularly the depth and strength of bedrock across the site. Such variation is, in our opinion, due to the presence of the old creek line that runs through the middle of the site in an East-West direction.

Any required anchors must have their bond length within shale of low strength or greater. For the design of anchors bonded into low strength (or greater) shale, an allowable bond stress value of 200kPa may be used, subject to the following conditions:

- Anchor bond lengths of at least 3m behind the 'active' zone of the excavation (taken as a 45-degree zone above the base of the excavation) are provided;
- Overall stability, including anchor group interaction, is satisfied;
- All anchors should be proof loaded to at least 1.33 times the design working load before being locked off at working load. Such proof loading is to be witnessed by an engineer independent of the anchoring contractor. We recommend that only experienced contractors be considered for anchor installation with appropriate insurances;
- If permanent anchors are to be used, these must have appropriate corrosion provisions for longevity.
- Permission to install anchors which extend beyond the property boundaries must be granted by the owners of the neighbouring properties prior to finalisation of the anchor design.
- Anchors/props must be installed progressively as excavation proceeds.

4.5 Hydrogeological Considerations

Groundwater was observed at relatively shallow depths in the monitoring wells previously installed in BH1 and BH4. It should also be noted that the presence of the open concrete lined channel indicates the presence of an old creek and high groundwater levels as indicated by the measurements taken in the monitoring wells on 3 December 2020.

Notwithstanding, all the above observed groundwater levels are well above the proposed lowest basement level. Therefore, if full drainage is allowed through the basement retention system, high inflow rate into the basement, particularly towards the central portion of the site should be expected and drawdown will result outside the basement excavation, particularly around BH4, where deep alluvial soils were encountered. Such drawdown may result in settlements and differential settlements below existing infrastructures (concrete channel) and adjoining buildings and buried services. Consequently, we consider it would be more prudent to adopt a perimeter cut-off wall socketed into at least medium strength shale bedrock to limit possible drawdown and control groundwater inflow. We anticipate that due to the relatively low permeability of the underlying bedrock, groundwater inflows into the excavation should not have an adverse impact on the proposed development or exiting infrastructures and neighbouring sites. We expect that groundwater inflows through the base of the excavation may be controlled by a conventional sump and pump system during construction. In the long term, drainage will need to be provided under the basement floor slab and around the perimeter. We recommend that seepage modelling be completed to confirm that drainage beneath the basement slab is possible. In this regard, we recommend that two additional monitoring wells be installed at the site for monitoring of groundwater levels across the site.

It is possible that further detailed groundwater monitoring including pump out tests and subsequent numerical seepage modelling may demonstrate that a drainage through the basement perimeter can be tolerated. However, in view of the uncertainties for this approach, we consider the perimeter cut-off would be appropriate.

4.6 Foundation Design

It is expected that shale bedrock will be present at bulk excavation level, however the shale bedrock at bulk excavation level is likely to be of at least medium strength (Class III) at the northern end of the site and of very low strength (Class V) within the central and southern portions of the site. considering the size of the proposed buildings and the expected high column loads, it is recommended that all footings for the buildings be founded within shale bedrock of similar strength to provide uniform support and to reduce the potential for differential settlements. At the northern end of the site, we anticipate that pad footings will be feasible, and within the central and southern portions of the site, we anticipate that piles will be required if footings are designed to be founded into medium strength bedrock or better.

Piles, pad and strip footings founded within shale of at least 'medium' strength below bulk excavation level may be designed for an allowable bearing capacity of 3500kPa and an allowable shaft adhesion of 350kPa (in compression) and 175kPa (in tension), provided that the sockets are satisfactorily cleaned and roughened. All footings must be inspected by the geotechnical engineer.

The allowable design parameters given above are based on serviceability criteria of settlements at the footing base of less than or equal to 1% of the minimum footing dimension.

Inspection of foundations at the commencement of footing construction (prior to pouring concrete or placing steel reinforcement) should be carried out by a geotechnical engineer to determine that the required socket and founding material has been achieved and provide guidance regarding variations that may occur between the inferences made and conditions observed at the time of construction.

We recommend that following demolition of the existing structures and prior to finalisation of the proposed shoring elevations, at least two further cored boreholes, with groundwater monitoring wells, be drilled (as described in Section 4.4, above) to confirm the subsurface conditions and groundwater levels across the site.

5 LIMITATIONS

The geotechnical assessment of the subsurface profile and geotechnical conditions within the proposed development area and the conclusions and recommendations presented in this report have been based on available information obtained during the work carried out by JC Geotechnics and in the provided documents listed in Section 1 of this report. Inferences about the nature and continuity of ground conditions away from and beyond the locations of field exploratory tests are made but cannot be guaranteed.

It is recommended that should ground conditions including subsurface and groundwater conditions, encountered during construction and excavation vary substantially from those presented within this report, JC Geotechnics Pty Ltd be contacted immediately for further advice and any necessary review of recommendations. JC Geotechnics does not accept any liability for site conditions not observed or accessible during the time of the inspection.

This report and associated documentation and the information herein have been prepared solely for the use of Freeburnville Pty Ltd and any reliance assumed by third parties on this report shall be at such parties' own risk. Any ensuing liability resulting from use of the report by third parties cannot be transferred to JC Geotechnics Pty Ltd, directors or employees.

For and on behalf of
JC Geotechnics Pty Ltd

Jithendhar R Marikanti
Geotechnical Engineer

Reviewed By

Joseph Chaghouri
Principal Geotechnical Engineer

APPENDIX A

About Your Report

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical- related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include the general nature of the structure involved, its size and configuration, the location of the structure on the site and its orientation, physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program.

To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should NOT be used:*

- when the nature of the proposed structure is changed: for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an un-refrigerated one,
- when the size or configuration of the proposed structure is altered.
- when the location or orientation of the proposed structure is modified.
- when there is a change of ownership, or for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

Geotechnical reports present the results of investigations carried out for a specific project and usually for a specific phase of the project. The report may not be relevant for other phases of the project, or where project details change.

The advice herein relates only to this project and the scope of works provided by the Client.

Soil and Rock Descriptions are based on AS1726- 1993, using visual and tactile assessment except at discrete locations where field and/or laboratory tests have been carried out. Refer to the attached terms and symbols sheets for definitions.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing is extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact. For this reason, most experienced owners retain their geotechnical consultants through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems that encountered on site.

SUB SURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions, and thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

Subsurface conditions can change with time and can vary between test locations. Construction activities at or adjacent to the site and natural events such as flood, earthquake or groundwater fluctuations can also affect the subsurface conditions.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems.

No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professional develop their plans based on mis-interpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

The interpretation of the discussion and recommendations contained in this report are based on extrapolation/ interpretation from data obtained at discrete locations. Actual conditions in areas not sampled or investigated may differ from those predicted.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, give contractors ready access in the complete geotechnical engineering report prepared or authorized for their use. Those who do not provide such access may proceed under mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are not exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publication's directory.

FURTHER GENERAL NOTES

Groundwater levels indicated on the logs are taken at the time of measurement and may not reflect the actual groundwater levels at those specific locations. It should be noted that groundwater levels can fluctuate due to seasonal and tidal activities.

This report is subject to copyright and shall not be reproduced either totally or in part without the express permission of the Company. Where information from this report is to be included in contract documents or engineering specifications for the project, the entire report should be included in order to minimize the likelihood of misinterpretation.

APPENDIX B

Borehole Location Plan

Source: Project 19107 Drawing DA-103 prepared by Urbanlink Pty Ltd

Drawn	AM
Checked	JC
Date	3/12/2020
Scale	NTS

Client: Freeburnville Pty Ltd
Proposed Residential Development
96-98 Lethbridge Street &42-46 Evan Street, Penrith, NSW

JC Geotechnics

Figure	1
Title	Approximate Borehole Location Plan
Job No.	GR1189.1J

APPENDIX C

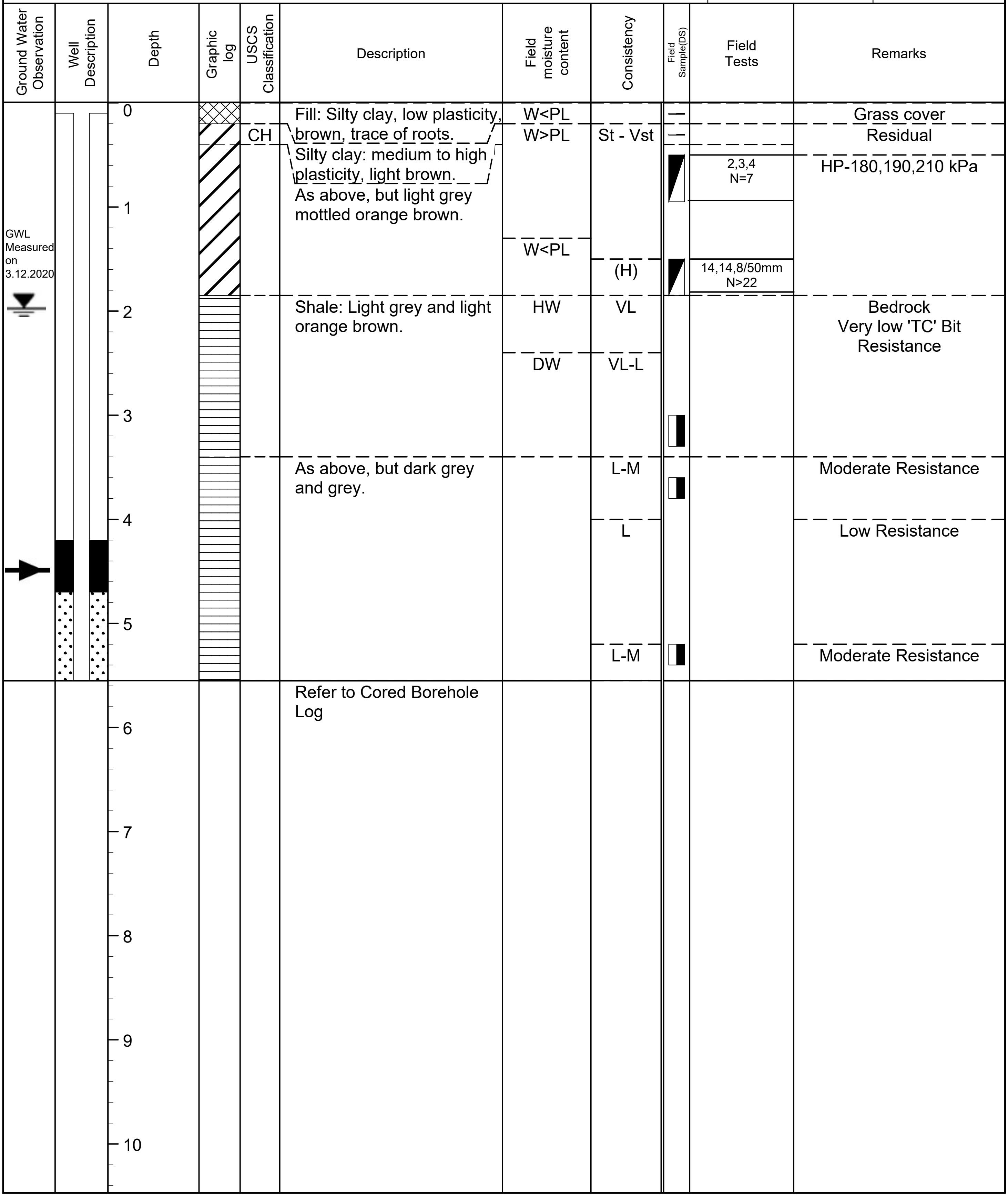
Borehole (BH1 to BH4) Logs and Core Photographs

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Client: Freeburnville Pty Ltd
Project: Proposed Residential Development

Borehole No: 1
Project No: GR1189.1J

Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW		Elevation: ≈35.7m	Datum: AHD
Drilling Contractor: Site Drilling and Investigation		Date Drilled: 17/11/2020	Logged By: DF
Drill Rig: EDSON 100		Date Completed: 17/11/2020	Checked By: JC



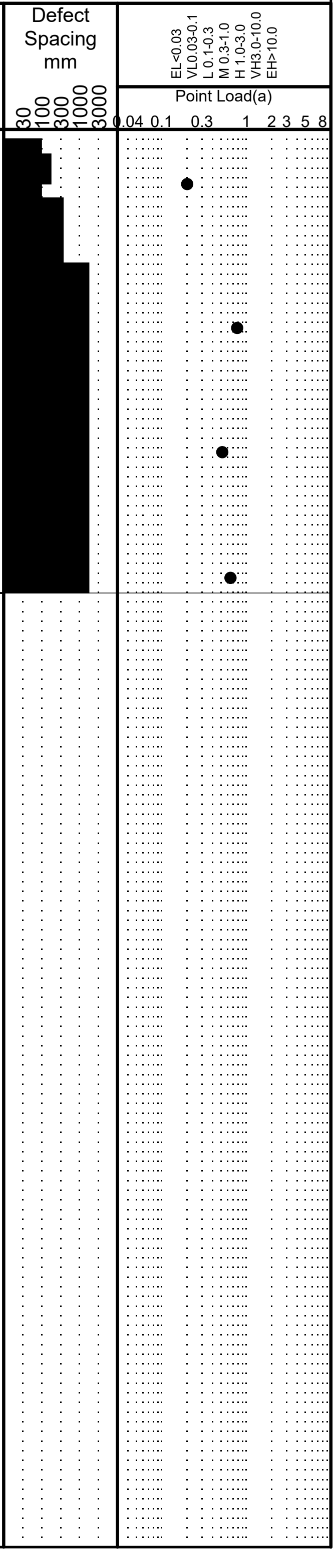
Ground Water Observation	Well Description	Depth	Graphic log	USCS Classification	Description	Field moisture content	Consistency	Field Sample(DS)	Field Tests	Remarks
					Fill: Silty clay, low plasticity, brown, trace of roots.	W<PL				Grass cover
				CH	Silty clay: medium to high plasticity, light brown.	W>PL	St - Vst			Residual
					As above, but light grey mottled orange brown.				2,3,4 N=7	HP-180,190,210 kPa
GWL Measured on 3.12.2020						W<PL	(H)		14,14,8/50mm N>22	
					Shale: Light grey and light orange brown.	HW	VL			Bedrock Very low 'TC' Bit Resistance
						DW	VL-L			
					As above, but dark grey and grey.		L-M			Moderate Resistance
							L			Low Resistance
							L-M			Moderate Resistance
					Refer to Cored Borehole Log					

JC Geotechnics Pty Ltd

CORING LOG OF BOREHOLE NO. 1

Client: Freeburnville Pty Ltd	**Project No.:** GR1189.1J
Project: Proposed Residential Development	**Logged By:** DF
Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW	**Checked By:** JC
Drilling Equipment/Method: EDSON100/NMLC Coring	**Elevation:** ≈35.7m
Date Drilled: 17/11/2020 **Completed:** 17/11/2020	**Datum:** AHD
Depth To Water:	**Total Depth:** 8.55 m

GW Observation	Depth (m)	Graphic Log	Material Descrption	Weathering Condition	Strength	DEFECT DESCRIPTION
	6 7 8		Start coring at 5.55m Shale: grey and dark grey bedded at 0° 10° with claystone lenses.	DW	L-M	5.65,JT,70°,P,SM 5.83,JT,80°,P,SM
					M	6.37,CS,60mm
	9 10 11 12 13 14		End of Borehole at 8.55m			




GR1189.1J BH1 START CORING AT 5.55m
5
6
7
8
END AT 8.55m

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Client: Freeburnville Pty Ltd
Project: Proposed Residential Development

Borehole No: 2
Project No: GR1189.1J

Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW		Elevation: ≈35.3m	Datum: AHD
Drilling Contractor: Site Drilling and Investigation		Date Drilled: 17/11/2020	Logged By: DF
Drill Rig: EDSON 100		Date Completed: 17/11/2020	Checked By: JC

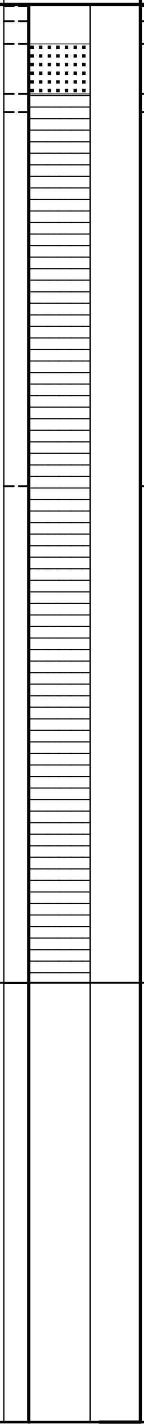
Ground Water Observation	Well Description	Depth	Graphic log	USCS Classification	Description	Field moisture content	Consistency	Field Sample(DS)	Field Tests	Remarks
Dry on completion of Augering		0			Fill: Silty clay, low plasticity, brown with roots.	W<PL				Appears Poorly Compacted
		1		CH	Silty clay: medium to high plasticity, light brown.	W>PL	VSt		3,3,5 N=8	HP 240,240,280kPa HP 300,300,280kPa
					Shale: grey	DW	VL-L		5,8/50mm N>8	Bedrock
		2			As above, but light grey.	SW	M			Moderate Resistance
		3 4 5 6 7 8 9 10			Refer to cored Borehole Log.					

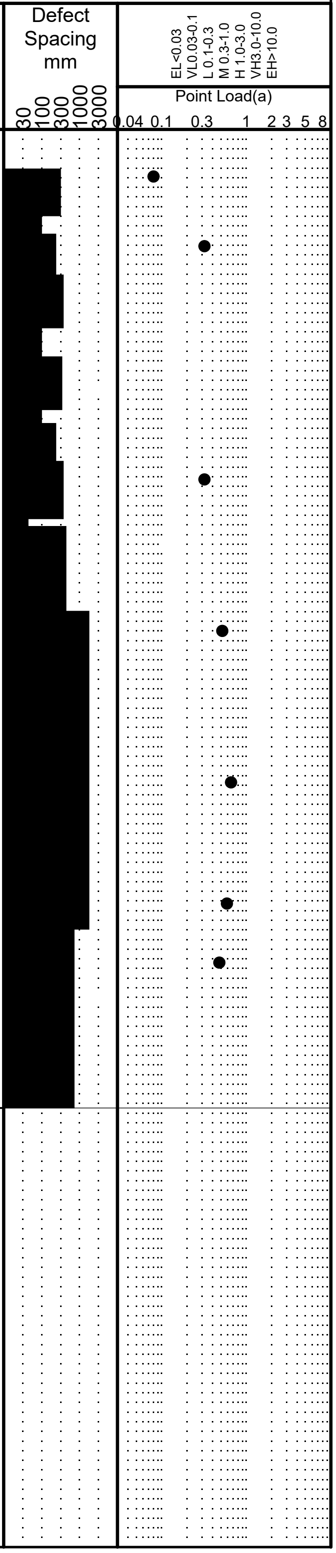
CORING LOG OF BOREHOLE NO. 2

Client: Freeburnville Pty Ltd	**Project No.:** GR1189.1J
Project: Proposed Residential Development	**Logged By:** DF
Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW	**Checked By:** JC
Drilling Equipment/Method: EDSON100/NMLC Coring	**Elevation:** ≈35.3m
Date Drilled: 17/11/2020 **Completed:** 17/11/2020	**Datum:** AHD
Depth To Water:	**Total Depth:** 8.75 m

GW Observation	Depth (m)	Material Descrption	Weathering Condition	Strength	DEFECT DESCRIPTION
		Start coring at 2.40m	-	-	
		Core Loss, 150mm			
		Sandstone: fine grained, orange brown and light brown, bedded at 0°-10°.	DW	L-M	
					2.86,Cr,25mm
	3				2.98,JT,60°,P,SM
		Shale: orange brown and brown.			3.25,JT,70°,P,SM
		As above, but grey, light grey, red brown and orange brown.		VL	3.66,CS,20mm
					3.79,CS,100mm
	4				4.14,CS,20mm
					4.23,CS,60mm
100 % Return				L	4.48,CS,50mm
					4.86,Cr,10mm
	5			L-M	4.91,Be,5°,P,SM
		Shale: dark grey and grey.	SW	M	5.46,CS,20mm
	6				
	7				
					7.57,Be,5°,P,SM
			DW		
	8				
					8.42,XWS,30mm,0°
		End of Borehole at 8.75m.			
	9				
	10				
	11				

Graphic Log





This information pertains only to this boring and should not be interpreted as being indicative of the site


GR1189.1J BH2 START CORING AT 2.40m
2
CORE LOSS 0.15m
3
4
5
6
7
8
END AT 8.75m

Client: Freeburnville Pty Ltd
Project: Proposed Residential Development

Borehole No: 3
Project No: GR1189.1J

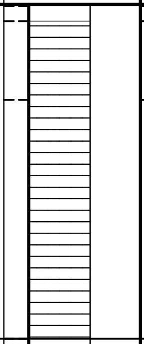
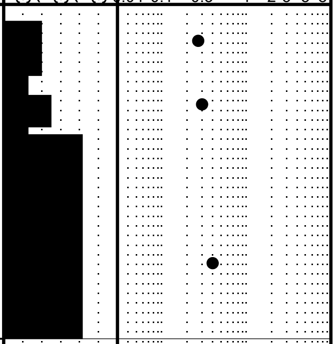
Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW		Elevation: ≈36.8m	Datum: AHD
Drilling Contractor: Site Drilling and Investigation		Date Drilled: 17/11/2020	Logged By: DF
Drill Rig: EDSON 100		Date Completed: 17/11/2020	Checked By: JC

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Ground Water Observation	Well Description	Depth	Graphic log	USCS Classification	Description	Field moisture content	Consistency	Field Sample(DS)	Field Tests	Remarks
Dry on completion of Augering		0	☒		Sandstone Pavement: 50mm thickness	W<PL				Pavement
			▨		Fill: Silty clay, low plasticity, brown, trace of fine to coarse grained sandstone gravel.	W>PL	St			Residual
			▨					◪	3,2,3 N=5	HP=140,170,180kPa
		1	▨		Silty clay: High plasticity, light grey, light brown, orange brown and red brown with fine to medium grained ironstone gravel.					
		2	▨			W≈PL	H	◪	6,9,10 N=19	HP=450,530,550kPa
		3	▨			W<PL		◪	10,13/150mm N>13	
			▤		Extremely Weathered shale, light grey and red brown.	XW	H			(Soil Strength)
		4 5 6 7	▤		Shale: grey with low to medium strength iron indurated bands.	DW	VL	◨		Very Low 'TC' Bit Resistance
		8	▤		Shale: dark grey	DW	L	◨		Low 'TC' Bit Resistance
		9 10			Refer to Cored Borehole Log					

CORING LOG OF BOREHOLE NO. 3

Client: Freeburnville Pty Ltd		**Project No.:** GR1189.1J	
Project: Proposed Residential Development		**Logged By:** DF	
Location: 96-98 Lethbridge St and 40-46 Evan St Penrith,		**Checked By:** JC	
Drilling Equipment/Method: EDSON100/NMLC Coring		**Elevation:** ≈35.3m	
Date Drilled: 18/11/2020	**Completed:** 18/11/2020	**Datum:** AHD	
Depth To Water:		**Total Depth:** 10.3 m	

GW Observation	Depth (m)	Graphic Log	Material Descrption	Weathering Condition	Strength	DEFECT DESCRIPTION	Defect Spacing mm 30 100 300 1000 3000	EL<0.03 VL0.03-0.1 L 0.1-0.3 M 0.3-1.0 H 1.0-3.0 VH3.0-10.0 EH>10.0 Point Load(a) 0.04 0.1 0.3 1 2 3 5 8
100 % Return			Start Coring at 8.20m					
			Shale: dark grey	DW	L-M	8.3,Cr,20mm 8.43,JT,40°,P,SM 8.45,JT,40°,P,SM 8.56,XWS,5mm 8.63,Cr,10mm 8.65,Be,0°,P,SM 8.69,Be,0°,P,SM 8.9,Be,0°,P,SM 8.95,Be,0°,P,SM		
	9		Shale: grey, bedded at 0°-10°.					
	10			SW	M			
	11 12 13 14 15 16 17		End of Borehole at 10.30m					


GR1189.1J BH3 START CORING AT 8.20m
8
9
10
END AT 10.30m

Client: Freeburnville Pty Ltd

Project: Proposed Residential Development

Borehole No: 4

Project No: GR1189.1J

Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW		Elevation: ≈34.9m	Datum: AHD
Drilling Contractor: Site Drilling and Investigation		Date Drilled: 17/11/2020	Logged By: DF
Drill Rig: EDSON 100		Date Completed: 17/11/2020	Checked By: JC

Ground Water Observation	Well Description	Depth	Graphic log	USCS Classification	Description	Field moisture content	Consistency	Field Sample(DS)	Field Tests	Remarks
		0			Fill: Silty clay, low plasticity, brown, trace of roots.	W<PL				Grass cover
					Silty Clay/Clayey silt: low plasticity, light orange brown, light brown and red brown.	W<PL	H			Alluvial
								◪	5,6,6 N=12	HP>600, >600
GWL Measured on 3.12.2020		1				W>PL	(F)	◧		
							St	◪	3,5,5 N=10	HP=140,130kPa
		2		CI	Silty clay: medium plasticity, orange brown and red brown with fine to medium grained ironstone gravel.	W>PL	St			
		3						◪	5,5,5 N=10	HP=140,180,190kPa
		4 5 6						◪	8,19/150 N>19	
		7			Shale: light grey with extremely weathered bands.	DW	VL			Bedrock Very Low 'TC' Bit Resistance.
					Sandstone: fine to medium grained, grey and orange brown.		L			Low Resistance
		8 9 10			Refer to Cored Borehole Log					

CORING LOG OF BOREHOLE NO. 4

Client: Freeburnville Pty Ltd

Project No.: GR1189.1J

Project: Proposed Residential Development

Logged By: DF

Location: 96-98 Lethbridge St and 40-46 Evan St Penrith, NSW

Checked By: JC

Drilling Equipment/Method: EDSON100/NMLC Coring

Elevation: ≈34.9m

Date Drilled: 18/11/2020

Completed: 18/11/2020

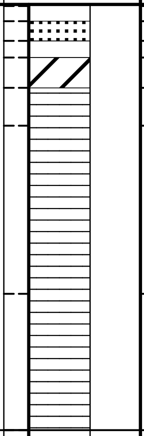
Datum: AHD

Depth To Water:

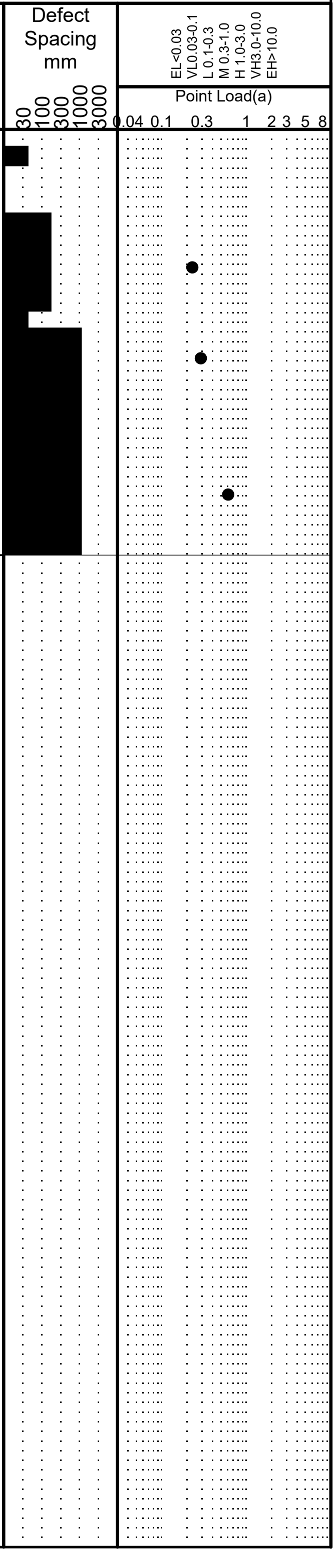
Total Depth: 10.2 m

GW Observation	Depth (m)	Material Descrption	Weathering Condition	Strength	DEFECT DESCRIPTION
		Start coring at 7.5m			
		Sandstone: fine to medium grained, grey and orange brown.	DW	L	7.6, CS,50mm,0°
		Core loss, 0.11m	-	-	
		Silty clay: medium plasticity, dark grey.	RS	(St)	
	8	Shale: grey and brown, bedded at 0°-10°.	DW	VL L-M	8.59,Be,5° 8.66,Cr,15mm
	9	As above, but dark grey and grey.			8.7,Be, 5°
		Shale: grey	SW	M	9.3,JT,25°,P,R
	10				9.97, JT, 45°,P,S
		End of Borehole at 10.2m			
	11				
	12				
	13				
	14				
	15				
	16				

Graphic Log



Defect Spacing mm / Point Load(a)



This information pertains only to this boring and should not be interpreted as being indicative of the site

JC Geotechnics Pty Ltd

Sheet 2 of 2


GR1189.1J BH4 START CORING AT 7.50m
7
CORE LOSS 0.11m
8
9
10
END AT 10.20m

APPENDIX D

Laboratory Test Results

MOISTURE CONTENT TEST REPORT

Client	JC Geotechnics Pty Ltd	**Job #**	S20509-1
Address	Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137	**Report #**	S64590-MC
Project	Proposed New Development (GR1189 1J)		

Test Procedure

☑ AS 1289 2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method).
☐ AS4133 1.1.1 Determination of the moisture content of rock - Oven drying method (standard method)
☐ RMS T120 Moisture content of road construction materials (Standard method)
☐ RMS T262 Determination of moisture content of aggregates (Standard method)

Sampling	Sampled by Client - results apply to the sample as received	**Date Sampled**	17/11/2020
Preparation	Prepared in accordance with the test method	**Date Tested**	19/11/2020

Sample #	Source	Sample Description	Moisture Content %
S64590	BH3 0.5-0.95m	Gravelly Silty CLAY	**23.3**

Notes

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards. This document shall not be reproduced, except in full. Results relate only to the samples tested.

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

25/11/2020

Date:

Macquarie Geotechnical
U7/8 10 Bradford Street
Alexandria NSW 2015

SOIL CLASSIFICATION REPORT

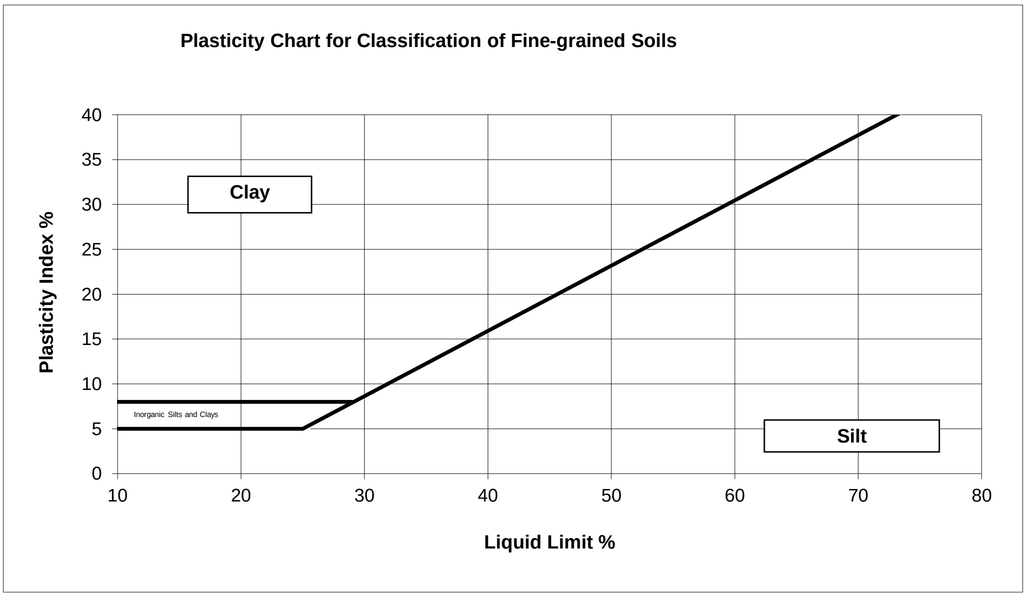
Client	JC Geotechnics Pty Ltd	**Source**	BH3 0.5-0.95m
Address	Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137	**Sample Description**	Gravelly Silty CLAY
Project	Proposed New Development (GR1189 1J)	**Report No**	S64590-PI
Job No	S20509-1	**Lab No**	S64590

Test Procedure

- ☐ AS1289 2.1.1 Soil moisture content tests (Oven drying method)
- ☑ AS1289 3.1.1 Soil classification tests - Determination of the liquid limit of a soil - Four point casagrande method
- ☐ AS1289 3.1.2 Soil classification tests - Determination of the liquid limit if a soil - One point Casagrande method (subsidiary method)
- ☑ AS1289 3.2.1 Soil classification tests - Determination of the plastic limit of a soil - Standard method
- ☑ AS1289 3.3.1 Soil classification tests - Calculation of the plasticity Index of a soil
- ☑ AS1289 3.4.1 Soil classification tests - Determination of the linear shrinkage of a soil - Standard method

Sampling	Sampled by Client - results apply to the sample as received	**Date Sampled**	17/11/2020
Preparation	Prepared in accordance with the test method	**Date Tested**	23/11/2020

Liquid Limit (%)	71	**Linear Shrinkage (%)**	17.5
Plastic Limit (%)	22	**Plasticity Index**	49



Soil Preparation Method: Dry Sieved
Soil History: Oven Dried
Soil Condition: Curling Occuring

Notes

Accredited for compliance with ISO/IEC 17025 - Testing.

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NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

25/11/2020

Date:

Macquarie Geotechnical
U7/8 10 Bradford Street
Alexandria NSW 2015

POINT LOAD STRENGTH INDEX REPORT

Client	JC Geotechnics Pty Ltd	**Moisture Content Condition**	As received
Address	Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137	**Storage History**	Core boxes
Project	Proposed New Development (GR1189 1J)	**Report #**	S64591-PL
Job #	S20509-1	**Test Date**	

Test Procedure ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling Sampled by Client - results apply to the sample as received **Date Sampled** 17/11/2020

Preparation Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Failure Mode
S64591	BH1 5.85-5.95m	Shale	Axial	51.3	44.0	0.56	0.19	0.20	1
S64592	BH1 6.80-6.90m	Shale	Axial	51.9	34.0	1.80	0.80	0.78	1
S64593	BH1 7.62-7.72m	Shale	Axial	51.8	35.0	1.23	0.53	0.52	1
S64594	BH1 8.45-8.55m	Shale	Axial	51.8	29.0	1.32	0.69	0.65	1
S64595	BH2 2.60-2.70m	Sandstone	Axial	51.5	38.0	0.20	0.08	0.08	3
S64596	BH2 3.06-3.15m	Shale	Axial	52.0	32.0	0.70	0.33	0.32	1
S64597	BH2 4.60-4.70m	Shale	Axial	51.6	31.0	0.69	0.34	0.32	1
S64598	BH2 5.60-5.70m	Shale	Axial	51.7	40.0	1.35	0.51	0.52	1
S64599	BH2 6.60-6.70m	Shale	Axial	52.4	35.0	1.57	0.67	0.66	1
S64600	BH2 7.40-7.50m	Shale	Axial	51.7	35.0	1.38	0.60	0.59	1

Failure Modes

1 - Fracture through fabric of specimen oblique to bedding, not influenced by weak planes.

2 - Fracture along bedding.

3 - Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.

4 - Chip or partial fracture.

Notes

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

19/11/2020

Date

Macquarie Geotech
U7/8 10 Bradford Street
Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client	JC Geotechnics Pty Ltd	**Moisture Content Condition**	As received
Address	Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137	**Storage History**	Core boxes
Project	Proposed New Development (GR1189 1J)	**Report #**	S64601-PL
Job #	S20509-1	**Test Date**	

Test Procedure ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling Sampled by Client - results apply to the sample as received **Date Sampled** 17/11/2020

Preparation Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Failure Mode
S64601	BH2 8.30-8.40m	Shale	Axial	51.6	44.0	1.35	0.47	0.48	1
S64602	BH3 8.33-8.40m	Shale	Axial	52.1	35.0	0.63	0.27	0.27	1
S64603	BH3 8.75-8.85m	Shale	Axial	51.7	32.0	0.66	0.31	0.30	1
S64604	BH3 9.80-9.90m	Shale	Axial	51.5	42.0	1.08	0.39	0.40	1
S64605	BH4 8.30-8.40m	Shale	Axial	51.8	38.0	0.57	0.23	0.23	1
S64606	BH4 8.9-9m	Shale	Axial	51.6	35.0	0.68	0.30	0.29	1
S64607	BH4 9.8-9.9m	Shale	Axial	51.7	42.0	1.65	0.60	0.61	1

Failure Modes

1 - Fracture through fabric of specimen oblique to bedding, not influenced by weak planes.

2 - Fracture along bedding.

3 - Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.

4 - Chip or partial fracture.

Notes

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
This document shall not be reproduced, except in full.
Results relate only to the samples tested.

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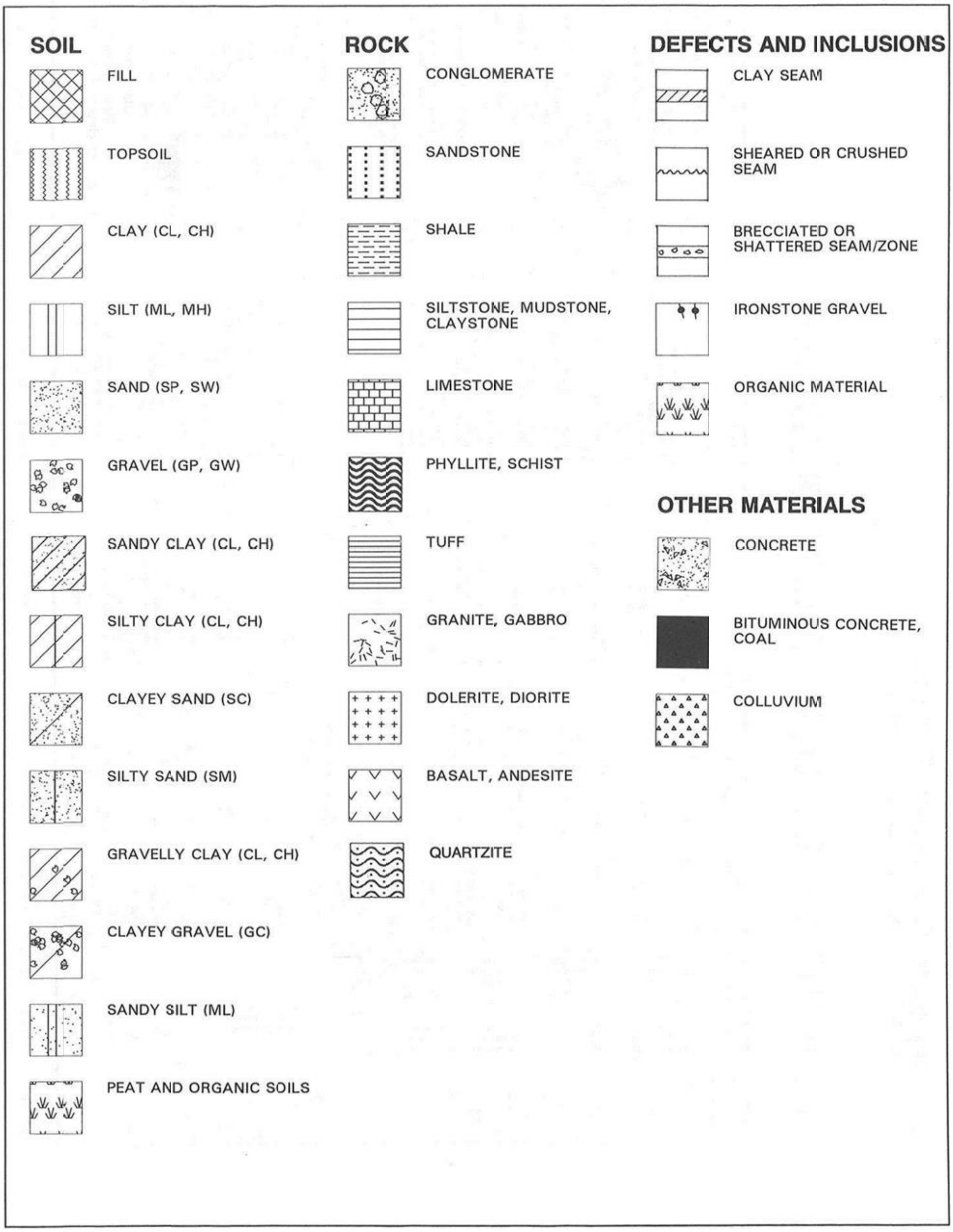
APPENDIX E

Explanatory Notes

JC GEOTECHNICS

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

The following information is intended to assist in the interpretation of terms and symbols used in geotechnical borehole logs, test pit logs and reports issued by or for the JC Geotechnics Pty Ltd. More detailed information relating to specific test methods is available in the relevant Australian Standards AS1726-2017.

Soil Descriptions

Description and Classification of Soils for Geotechnical Purposes: Refer to AS1726-2017 (Clause 6.1.6)

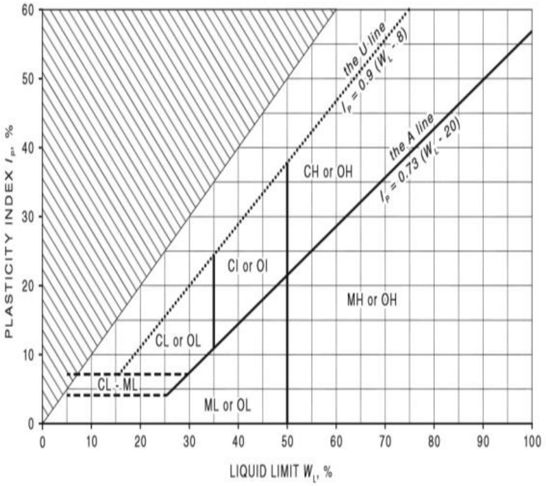
The following chart (adapted from AS1726-2017, Clause 6.1.6, Table A1) is based on the Unified Soil Classification System (USCS).

Table 1

Major Divisions		Particle size mm	USCS Group Symbol	Typical Names	Field classification of sand and gravel	Laboratory Classification					
COARSE GRAINED SOILS (more than 65% of soil excluding oversize fraction is greater than 0.075 mm)	BOULDERS	200				% < 0.075 mm		Plasticity of fine fraction	$C_u = \frac{D_{60}}{D_{10}}$	$C_u = \frac{(D_{30})^2}{(D_{10})(D_{60})}$	NOTES
	COBBLES	63									
	GRAVELS (more than half of coarse fraction is larger than 2.36 mm)		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	Use the gradation curve of material passing 63 mm for classification of fractions	≤ 5% fines	—	>4	Between 1 and 3	(1) Identify fines by the method given for fine-grained soils.
		coarse	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		
		20 medium	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	Below 'A' line or PI<4	—	Fines behave as silt	(2) Borderline classifications occur when the percentage of fines (fraction smaller than 0.075 mm size) is greater than 5% and less than 12%. Borderline classifications require the use of SP-SM, GW-GC.
		6 fine	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	Above 'A' line and PI>7	—	Fines behave as clay	
	SANDS (more than half of coarse fraction is smaller than 2.36 mm)	2.36	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	—	>6	Between 1 and 3	
		coarse 0.6	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		
		medium 0.2	SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength		≥ 12% fines, fines are silty	Below 'A' line or PI<4	—	—	
		fine 0.075	SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength		≥ 12% fines, fines are clayey	Above 'A' line and PI>7		—	

Classification of fine-grained soils

JC GEOTECHNICS

Major Divisions		**USCS Group Symbol**	**Typical Names**	**Field classification of sand and gravel**			**Laboratory classification**	
				Dry Strength	**Dilatancy**	**Toughness**	**% < 0.075 mm**	
FINE GRAINED SOILS (more than 35% of soil excluding oversize fractions is less than 0.075 mm)	SILT and CLAY (low to medium plasticity, %) (Liquid Limit ≤50%)	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 silts and clays of low plasticity	Low to medium	Slow	Low	Below A line	
	SILT and CLAY (high plasticity) (Liquid Limit >50%)	MH	Inorganic silts, mic- aceous or diato-maceous fine sands or silts, elastic silts	Low to medium	None to slow	Low to medium	Below A line	
		CH	Inorganic clays of high plasticity, fat clays	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 SOILS	PT	Peat and other highly organic soils	-	-	-	-	

Soil Colour: Is described in the moist condition using black, white, grey, red, brown, orange, yellow, green or blue. Borderline cases can be described as a combination of two colours, with the weaker followed by the stronger. Modifiers such as pale, dark or mottled, can be used as necessary. Where colour consists of a primary colour with secondary mottling, it should be described as follows: (Primary) mottled (Secondary). Refer to AS 1726-2017, Clause 6.1.5

Soil Moisture Condition: Is based on the appearance and feel of soil. Refer to AS 1726-2017, Clause 6.1.7

Term	**Description**
Dry (D)	Cohesive soils; hard and friable or powdery, well dry of plastic limit. Granular soils; cohesionless and free-running.
Moist	Soil feels cool, darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.
Wet	Soil feels cool, darkened in colour. Cohesive soils usually weakened and free water forms on hands when handling. Granular soils tend to cohere and free water forms on hands when handling.

Consistency of Cohesive Soils: May be estimated using simple field tests, or described in terms of a strength scale. In the field, the undrained shear strength (s_u) can be assessed using a simple field tool appropriate for cohesive soils, in conjunction with the relevant calibration. Refer to AS 1726-2017, Table 11.

Consistency - Essentially Cohesive Soils					
Term	**Field Guide**	**Symbol**	**SPT "N" Value**	**Undrained Shear Strength s_u (kPa)**	**Unconfined Compressive Strength q_u (kPa)**
Very soft	Exudes between the fingers when squeezed in hand	VS	0-2	<12	<25
Soft	Can be moulded by light finger pressure	S	2-4	12-25	25-50
Firm	Can be moulded by strong finger pressure	F	4-8	25-50	50-100
Stiff	Cannot be moulded by fingers	St	8-15	50-100	100-200
Very stiff	Can be indented by thumb nail	VSt	15-30	100-200	200-400
Hard	Can be indented with difficulty by thumb nail.	H	>30	>200	>400
Friable (Fr)	Can be easily crumbled or broken into small pieces by hand	Fr	-	-	-

Soil Particle Sizes	
Term	**Size Range**
BOULDERS	>200 mm
COBBLES	63-200 mm
Coarse GRAVEL	20-63 mm
Medium GRAVEL	6-20 mm
Fine GRAVEL	2.36-6 mm
Coarse SAND	0.6-2.36 mm
Medium SAND	0.2-0.6 mm
Fine SAND	0.075-0.2 mm
SILT	0.002-0.075 mm
CLAY	<0.002 mm

Note: SPT - N to q_u correlation from Terzaghi and Peck, 1967. (General guide only).

Consistency of Non-Cohesive Soils: Is described in terms of the density index, as defined in AS 1289.0-2014. This can be assessed using a field tool appropriate for non-cohesive soils, in conjunction with the relevant calibration. Refer to AS 1726-2017, Table 12

Consistency - Essentially Non-Cohesive Soils				
Term	**Symbol**	**SPT N Value**	**Field Guide**	**Density Index (%)**
Very loose	VL	0-4	Foot imprints readily	0-15
Loose	L	4-10	Shovels Easily	15-35
Medium dense	MD	10-30	Shoveling difficult	35-65
Dense	D	30-50	Pick required	65-85
Very dense	VD	>50	Picking difficult	85-100

Standard Penetration Test (SPT): Refer to. AS 1289.6.3.1-2004 (R2016). Example report formats for SPT results are shown below:

Test Report	**Penetration Resistance (N)**	**Explanation / Comment**
4, 7, 11	N=18	Full penetration; N is reported on engineering borehole log
18, 27, 32	N=59	Full penetration; N is reported on engineering borehole log
4, 18, 30/15 mm	N is not reported	30 blows causes less than 100 mm penetration (3rd interval) – test discontinued
30/80 mm	N is not reported	30 blows causes less than 100 mm penetration (1st interval) – test discontinued
rw	N<1	Rod weight only causes full penetration
hw	N<1	Hammer and rod weight only causes full penetration

hb	N is not reported	Hammer bouncing for 5 consecutive blows with no measurable penetration – test discontinued

Rock Descriptions

Refer to AS 1726-2017 Clause 6.2.3 for the description and classification of rock material composition, including:

(a) Rock name (Table 15, 16, 17, 18)
(b) Grain size
(c) Texture and fabric
(d) Colour (describe as per soil)
(e) Features, inclusion and minor components.
(f) Moisture content
(g) Durability

The condition of a rock material refers to its weathering characteristics, strength characteristics and rock mass properties. Refer to AS 1726-2017 (Clause 6.2.4 Tables 19, 20 and 21).

Weathering Condition (Degree of Weathering):

The degree of weathering is a continuum from fresh rock to soil. Boundaries between weathering grades may be abrupt or gradational.

Rock Material Weathering Classification				
Weathering Grade		**Symbol**		**Definition**
Residual Soil (Note 1)		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 Rock (Note 2)		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 Rock (Note 2)	Distinctly Weathered (Note 2)	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 recognizable. 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
Moderately Weathered Rock (Note 2)		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 recognizable, but shows little or no change of strength from fresh rock.
Slightly Weathered Rock		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock
Fresh Rock		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

Notes:

1. Minor variations within broader weathering grade zones will be noted on the engineering borehole logs.
2. Extremely weathered rock is described in terms of soil engineering properties.
3. Weathering may be pervasive throughout the rock mass, or may penetrate inwards from discontinuities to some extent.
4. Where it is not practicable to distinguish between ‘Highly Weathered’ and ‘Moderately Weathered’ rock the term ‘Distinctly Weathered’ may be used. ‘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.

Strength Condition (Intact Rock Strength):

Strength of Rock Material

(Based on Point Load Strength Index, corrected to 50 mm diameter – $I_{s(50)}$. Field guide used if no tests available. Refer to AS 4133.4.1-2007 (R2016).

Term	**Sym**	**Point Load Index (MPa)** $I_{s(50)}$	**Field Guide to Strength**
Extremely Low	EL	≤0.03	Easily remoulded by hand to a material with soil properties.
Very Low	VL	>0.0 ≤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 3 cm thick can be broken by finger pressure.

Low	L	>0.1	≤0.3	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long by 50 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	M	>0.3	≤1.0	Readily scored with a knife; broken by hand with difficult a piece of core 150 mm long by 50 mm diameter can be y.
High	H	>1	≤3	A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	VH	>3	≤10	pick after more than one blow; rock rings under hammer.
Extremely High	EH	>10		Specimen requires many blow rock ring with geological pick to break through intact material; under hammer

Notes:

1. These terms refer to the strength of the rock material and not to the strength of the rock mass which may be considerably weaker due to the effect of rock defects.
2. Anisotropy of rock material samples may affect the field assessment of strength.

Discontinuity Description: Refer to AS 1726-2017, Table 22.

Anisotropic Fabric	
BED	Bedding
FOL	Foliation
LIN	Mineral lineation
Defect Type	
LP	Lamination Parting
BP	Bedding Parting
FP	Cleavage / Foliation Parting
J, Js	Joint, Joints
SZ	Sheared Zone
CZ	Crushed Zone
BZ	Broken Zone
HFZ	Highly Fractured Zone
AZ	Alteration Zone
VN	Vein

Roughness (e.g. Planar, Smooth is abbreviated Pl / Sm)		Class
Stepped (Stp)	Rough or irregular (Ro)	I
	Smooth (Sm)	II
	Slickensided (Sl)	III
Undulating (Un)	Rough (Ro)	IV
	Smooth (Sm)	V
	Slickensided (Sl)	VI
Planar (Pl)	Rough (Ro)	VII
	Smooth (Sm)	VIII
	Slickensided (Sl)	IX

Aperture		**Infilling**		
Closed	CD	No visible coating or infill	Clean	Cn
Open	OP	Surfaces discoloured by mineral/s	Stain	St
Filled	FL	Visible mineral or soil infill <1mm	Veneer	Vr
Tight	TI	Visible mineral or soil infill >1mm	Coating	Ct

Other	
Cly	Clay
Fe	Iron
Co	Coal
Carb	Carbonaceous
Sinf	Soil Infill Zone
Qz	Quartz
CA	Calcite
Chl	Chlorite
Py	Pyrite
Int	Intersecting
Inc	Incipient
DI	Drilling Induced
H	Horizontal
V	Vertical

Note: Describe 'Zones' and 'Coatings' in terms of composition and thickness (mm).

Discontinuity Spacing: On the geotechnical borehole log, a graphical representation of defect spacing vs depth is shown. This representation takes into account all the natural rock defects occurring within a given depth interval, excluding breaks induced by the drilling / handling of core. Refer to AS 1726-2017, BS5930-2015.

Defect Spacing			**Bedding Thickness (Sedimentary Rock**	
Spacing/Width (mm)	**Descriptor**	**Symbol**	**Descriptor**	**Spacing/Width (mm)**
			Thinly Laminated	< 6
<20	Extremely Close	EC	Thickly Laminated	6 – 20
20 – 60	Very Close	VC	Very Thinly Bedded	20 – 60
60 – 200	Close	C	Thinly Bedded	60 – 200
200 – 600	Medium	M	Medium Bedded	200 – 600
600 – 2000	Wide	W	Thickly Bedded	600 – 2000
2000 – 6000	Very Wide	VW	Very Thickly Bedded	> 2000
>6000	Extremely Wide	EW		

Defect Spacing in 3D	
Term	**Description**
Blocky	Equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section

Defect Persistence (areal extent)
Trace length of defect given in metres

Symbols

The list below provides an explanation of terms and symbols used on the geotechnical borehole, test pit and penetrometer logs.

Test Results			
PI	Plasticity Index	c'	Effective Cohesion
LL	Liquid Limit	c_u	Undrained Cohesion
LI	Liquidity Index	c'_R	Residual Cohesion
DD	Dry Density	ϕ'	Effective Angle of Internal Friction
WD	Wet Density	ϕ_u	Undrained Angle of Internal Friction
LS	Linear Shrinkage	ϕ'_R	Residual Angle of Internal Friction
MC	Moisture Content	c_v	Coefficient of Consolidation
OC	Organic Content	m_v	Coefficient of Volume Compressibility
WPI	Weighted Plasticity Index	$c_{\alpha\varepsilon}$	Coefficient of Secondary Compression

Test Symbols	
DCP	Dynamic Cone Penetrometer
SPT	Standard Penetration Test
CPTu	Cone Penetrometer (Piezocone) Test
PANDA	Variable Energy DCP
PP	Pocket Penetrometer Test
U50	Undisturbed Sample 50 mm (nominal diameter)
U100	Undisturbed Sample 100mm (nominal diameter)
UCS	Uniaxial Compressive Strength
Pm	Pressuremeter

Test Results			
WLS	Weighted Linear Shrinkage	e	Voids Ratio
DoS	Degree of Saturation	ϕ'_{cv}	Constant Volume Friction Angle
APD	Apparent Particle Density	q_t / q_c	Piezocone Tip Resistance (corrected / uncorrected)
s_u	Undrained Shear Strength	q_d	PANDA Cone Resistance
q_u	Unconfined Compressive Strength	$I_{s(50)}$	Point Load Strength Index
R	Total Core Recovery	*RQD*	Rock Quality Designation

Test Symbols	
FSV	Field Shear Vane
DST	Direct Shear Test
PR	Penetration Rate
A	Point Load Test (axial)
D	Point Load Test (diametral)
L	Point Load Test (irregular lump)

▽ 28/11/13	Groundwater level on the date shown	▶	Water Inflow	◁	Water Outflow