JC GEOTECHNICS

November 24, 2020

Blue CHP Limited

GEOTECHNICAL INVESTIGATION REPORT

20 – 22 Mindarie Street & 30 Pinaroo Place, Lane Cove North, NSW

JC GEOTECHNICS PTY LTD
Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137

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Document Control Record

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1. INTRODUCTION

JC Geotechnics Pty Ltd has been commissioned by **Blue CHP Limited** to carry out a geotechnical site investigation at 20 – 22 Mindarie St & 30 Pinaroo Pl, Lane Cove North, NSW. The site investigation was carried out on the 18th February 2020 in accordance with our Proposal Ref. GP2019-007 Lane Cove North, dated 18th February 2020.

The purpose of the investigation was to assess the subsurface conditions, at four borehole locations nominated by the Client on site, to assist with the planning and design of the proposed new dwelling.

This report presents the results of the geotechnical site investigation, interpretation, and assessment of the site's existing geotechnical conditions, as a basis to provide the following recommendations:

- Detailed logs of the boreholes with penetration test results and groundwater observations;
- Interpretation of subsurface profile;
- Dilapidation;
- Excavation conditions & support including geotechnical design parameters;
- Groundwater;
- Backfilling of proposed excavation

To assist in reading the report, reference should be made to "About Your Report" attached to **Appendix A**.

2. AVAILABLE INFORMATION

Prior to preparation of this report, the following information was made available to JC Geotechnics:

- Architectural Drawings for Residential Apartments titled "Cover Sheet & Location Plan", prepared by Blue CHP Pty Ltd, Project No. 2088.16, Drawing No. DA00, DA01, DA02, DA03, DA04, DA06, DA07, DA08, DA09, DA10, DA11, DA12, DA13, DA14, DA16, DA17, DA20, DA21, DA22, Revision. 01-WIP, dated 23 October 2020.
- Development Data

Based on the above, we understand that it is being proposed to demolish the existing site structures and construct a five-storey residential building over two levels of basement car parking in an area of approximately 1,750.8m^2. The lowest basement level is proposed to have a Finished Floor Level (FFL) of RL42.58m. In order to achieve the latter, excavation to depth ranging between 6.8m (Adjacent to Mindarie Street) and 8.3m (Adjacent to Pinaroo Pl), below existing surface levels will be required.

3. SCOPE OF WORK

The fieldwork for the geotechnical site investigation was carried out by an experienced Geotechnical Engineer from JC Geotechnics Pty Ltd broadly following the guidelines provided in Australian Standard AS 1726-2017 (Reference 1) and comprised the following:

- A site walk-over inspection was carried out by a Geotechnical Professional in order to determine the overall surface conditions and to identify relevant site features.
- Prior to commencement of the fieldwork, the proposed borehole locations were electromagnetically scanned by a specialist subcontractor with reference to Dial Before You Dig (**DBYD**) plans.
- Safe work measures and procedures were implemented during the course of the fieldwork.
- Four (4) Boreholes (numbered 1 to 4) were drilled using a truck mounted drilling rig. The boreholes were auger drilled to refusal to further penetration at depths of approximately 2.0m to 2.6m below existing surface levels (or RL of 41 to 47.2m).
- Standard Penetrometer (SPT) tests were carried out in the boreholes to assess the strength/relative density of the soils.
- The strength of the sandstone bedrock was assessed by observation of the auger penetration resistance using a Tungsten Carbide 'TC' drill bit attached to the augers together with examination of the recovered rock samples. It should be noted that strength assessment in this way are approximate and variances of one strength order should not be unexpected.
- Groundwater observations were recorded during drilling and on completion of the boreholes if present. No long-term monitoring of the groundwater levels was carried out as this was outside the scope of this investigation.
- The approximate locations of the four (4) boreholes during the geotechnical site investigation are shown on "**Figure 1** – Borehole Location Plan" attached to **Appendix D**.
- Selected samples were returned to Macquarie Geotech, a NATA registered laboratory, for Moisture Content testing.

A geotechnical engineer from JC Geotechnics was present full-time on site, to set out the test locations, log the encountered subsurface profile and nominate in-situ testing and sampling. The borehole logs, together with explanatory notes outlining the glossary of logging terms and symbols used are attached in Appendix C and Appendix E, respectively.

4. BRIEF SITE DESCRIPTION

The site generally slopes in two direction down from North to South at an angle of about 6° and down from West to East at an angle of about 6° as well.

The site is approximately rectangular in shape with a total area of approximately 1,530 m^2. The site is bounded by Mindarie Street to the North and Pinaroo Place to the East.

At the time of the investigation, the site was comprised of three residential properties, located at 20 Mindarie St, 22 Mindarie Street and 30 Pinaroo Place, respectively. The properties were constructed of bricks and appeared to be in reasonably good condition. A steel fencing bounded the properties extending from Pinaroo Place to Mindarie Street. Concrete pavement driveways extended from each of the properties to their respective streets. Minor cracks were observed on the concrete pavements. Each property had front yards with moderate vegetation (grass, plants, trees).

5. INVESTIGATION RESULTS

5.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 Edition 1, dated 1983, by the Geological Survey of New South Wales, Department of Mineral Resources, indicates the site is located within the geological boundary known as Hawkesbury Sandstone (Rh) of the Wianamatta Group of Middle Triassic age. Hawkesbury Sandstone is described as “medium to coarse-grained quartz sandstone, very minor shale and laminate lenses.”

It should be noted that the published geological profile does not take into account the residual soils derived from in-situ weathering of the bedrock or the presence of fill that may have been generated from previous earthworks.

5.2 Subsurface Conditions

The subsurface conditions encountered within the boreholes are summarised in below.

Fill

Fill material was encountered in all of the boreholes, BH1, BH2, BH3 and BH4 to a depth between 0.2m to 0.5m. The fill comprised of sand, sandy clay and clayey sand. Fine to medium grained sand was observed with various contents of rootlets and organic matter. The fill material was generally assessed to be poorly compacted.

Residual

Residual material was encountered in three of the boreholes, BH1, BH3 and BH4 below the fill and extends to a depth between 0.25m to 1.5m. The residual soil comprised of sand, clayey sand and sandy clay. Fine to medium grained sand that was generally assessed to be of loose density and low plasticity clays were encountered.

Bedrock

Sandstone bedrock was encountered in all the boreholes, BH1, BH2, BH3 and BH4 at depths between 0.25m to 1.5m. The sandstone encountered was extremely weathered and assessed to be of extremely low strength. The sandstone bedrock below the refusal depth of the TC bit is considered to be of at least low strength or better.

5.3 Groundwater

Groundwater was not encountered in boreholes during and shortly after completion of drilling. No long-term groundwater measurements were carried out as this was outside the scope of this investigation.

5.4 Laboratory Testing

The soil moisture content tested based on the recovered rock chips from the boreholes were 11.2% and 14.4% for BH1, 6.1% for BH2, 8.3% and 5.6% for BH3 and 15.1% for BH4. The laboratory test results are attached to **Appendix B.**

6. GEOTECHNICAL ASSESSMENT

6.1 Dilapidation

Prior to excavation and construction, we recommend that detailed dilapidation surveys be carried out on all structures, buried services and infrastructures surrounding the site that falls within the zone of influence of the 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 report would provide a record of existing conditions prior to commencement of the work. A copy of each report should be provided to the adjoining 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 construction.

6.2 Basement Excavation

Prior to any excavation commencing, we recommend that reference be made to the WorkCover Excavation Work Code of Practice – January 2020.

Based on the borehole logs, the proposed basement excavations will therefore extend through fill, residual soil and for the most part sandstone bedrock. A full depth engineered retention system must be installed prior to excavation commencing.

The soil and sandstone of less than low strength could be excavated using buckets of conventional earthmoving Hydraulic Excavators, particularly if fitted with ‘Tiger Teeth’ with some ripping.

Ripping of low strength sandstone or better bedrock would be required for most of the excavation and will present hard ripping or “hard rock” excavation conditions and therefore excavation productivity will be slow and higher than normal ‘wear and tear’ of excavation attachments is to be expected. The presence of defects will help facilitate excavation, but only marginal. Therefore, ripping would require a high capacity and heavy bulldozer of at least D10 or similar. The use of a smaller size bulldozer will result in lower productivity, and higher wear and tear, and this should be allowed for. Perimeter and Grid sawing techniques with ripping will also facilitate the excavation and assist in reducing vibration emissions.

Should rock hammers be used for this site, vibration monitoring must be carried out at all times and further advice must be sought from the geotechnical engineer.

Groundwater seepage monitoring should be carried out during bulk excavation prior to finalising the design of a pump out facility. Outlets into the stormwater system will require Council approval.

Furthermore, any existing buried services which run below the site will require diversion prior to the commencement of excavation or alternatively be temporarily supported during excavation, subject to permission or other instructions from the relevant service authorities. Enquiries should also be made for further information and details, such as invert levels, on the buried services.

6.3 Hydrogeological Considerations

Groundwater was not observed during the auger drilling of the boreholes. However, experience shows that due to the expected low permeability of the soil and bedrock profile, groundwater inflows into the excavation should not have an adverse impact on the neighbouring sites. We expect groundwater inflows into the excavation along the soil/rock interface and through any defects within the bedrock (such as jointing, and bending planes, etc.) particularly following a period of heavy rain. The initial flows into the excavation may be locally high but would be expected to decrease with time as the bedding seams/joints are drained. We recommend that monitoring of seepage be implemented during the excavation works to confirm the capacity of the drainage system.

We expect that any seepage that does occur should be able to be controlled by a conventional sump and pump system. We recommend that a sump-and-pump system be used both during construction and for permanent groundwater control below the basement floor slab.

In the long term, drainage should be provided behind all basement retaining walls, around the perimeter of the basement and below the basement slab. The completed excavation should be inspected by the hydraulic engineer to confirm that adequate drainage has been allowed for. Drainage should be connected to the sump-and-pump system and discharging into the stormwater system. The permanent groundwater control system should take into account any possible soluble substances in the groundwater which may dictate whether or not groundwater can be pumped into the stormwater system.

We recommend that pump-out tests be undertaken together with seepage analysis to estimate the predicted groundwater seepage volumes into the excavation.

The design of drainage and pump systems should take the above issues into account along with careful ongoing inspections and maintenance programs.

6.4 Basement Excavation Support

6.4.1 Retention System

From a geotechnical perspective, it is critical to maintain the stability of the adjacent structures, infrastructures and buried services during demolition, excavation and construction works.

Based on the provided architectural plans, the basement appears to extend to or close to the site boundaries and hence temporary batters are not feasible and not recommended for this site.

Unsupported vertical cuts of the soil and weathered rock profile are not recommended for this site as these carry the risk of potential slump failure especially after a period of wet weather. Slumping of the material may result in injury to personnel and/or damage to nearby structures/infrastructures and equipment.

A suitable retention system will be required for the support of the excavation. An anchored and/or propped soldier pile wall with concrete infill panels is recommended for this site. Anchors/props and shotcrete must be installed progressively as excavation proceeds. The use of a more rigid system (such as a contiguous or semi-contiguous pile wall) is recommended adjacent to neighbouring buildings/infrastructures, to reduce the lateral movements and the risk of potential damage.

Two options are presented for the shoring wall. The piles may extend the full depth of excavation and should be founded with enough embedment into sandstone bedrock below bulk excavation level to satisfy stability and founding considerations.

Alternatively, the piles may be terminated at least 0.5m into at least medium strength sandstone bedrock above bulk excavation level. A vertical face may possibly be cut below the toe of the pile wall and without support. Consideration may be given to extending, say every fourth pile, to below the bulk excavation level. These extended piles would carry vertical perimeter loads. Anchors should be installed at toe of any shoring pile that has been cut short (i.e. terminated 0.5m into at least medium strength sandstone and above the bulk excavation level) to provide lateral stability of the pile toe adjacent to the rock. If vertical cuts are adopted, a geotechnical engineer must inspect the excavations at regular intervals to check for any inclined joints or weak seams that require stabilisation, in particular for defects below the toe of piles founded above bulk excavation which may result in instability of the wall above. Such geotechnical inspections should be carried out a depth interval of no more than 1.5 m. If adverse defects are encountered, temporary stabilisation measures may comprise rock bolts, shotcrete and mesh, or dental treatment of thin weak seams using non-shrink grout, and this should be allowed for.

The choice of retention system to be used will depend on relative costs taken into account the time and difficulty associated with drilling into the medium strength and better sandstone which would be required for the former option. We assume that permanent support of the retention system will be provided by bracing from the proposed structure.

Bored piers may be used for this site. However, relatively large capacity piling rigs (minimum Soilmec SR-40 or larger) with rock augers and coring buckets will be required for drilling through the sandstone bedrock. The use of smaller drill rigs may result in less productivity and higher wear and tear, and this should be allowed for. It should be noted that the sandstone is considered to be fine to coarse grained which is likely to present difficult drilling conditions and high wear and tear, and this should be allowed for. The proposed pile locations should take into account the presence of any neighbouring anchors and/or the presence of buried services. Further advice should be sought from prospective piling contractor who should be provided with a copy of this report. Working platforms may also be required.

6.4.2 Design Parameters

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 soil and sandstone bedrock of less than medium strength, where H is the retained height in meters. These pressures 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 which 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 soil and sandstone of less than medium strength, where ‘H’ is the retained height in meters. These pressures 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 (including from construction equipment, construction loads, adjacent high-level footings, etc.) should be adopted in the retaining wall design as an additional surcharge using an ‘at rest’ earth pressure coefficient, k_o, of 0.55;

The retaining walls should be designed as drained and measures are to be taken to provide complete and permanent drainage behind the walls. Strip drains protected with a non-woven geotextile fabric should be used behind the shotcrete infill panels for soldier pile walls or inserted between gaps in contiguous piles. Alternatively, for the contiguous pile walls, weepholes comprising 20mm diameter PVC pipes grouted into holes or gaps between adjacent piles at 1.2m centres (horizontal and vertical), may be used. The embedded end of the pipes must, however, be wrapped with a non-woven geotextile fabric (such as Bidim A34) to act as a filter against subsoil erosion;

For piles embedded into medium to high strength sandstone bedrock and below bulk excavation level, an allowable lateral toe resistance value of 350kPa 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 socket should not be taken into account to allow for tolerance and disturbance effects during excavation.

If temporary anchors extend beyond the site boundaries, then permission from the neighbouring properties would need to be obtained prior to installation. Also, the presence of neighbouring basements (if any) or services and their levels must be confirmed prior to finalising anchor design.

Anchors should have their bond length within medium strength sandstone or better. For the design of anchors bonded into medium to high strength sandstone or better, an allowable bond stress value of 300kPa 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) is 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 locked off at working load. Such proof loading is to be witnessed by and 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.

6.5. Foundation Design

It is expected that sandstone of at least low strength or better to be exposed at bulk excavation level of RL40.725m. It is recommended that all footings for the building be founded within sandstone bedrock of similar strength to provide uniform support and reduce the potential for differential settlements.

Pad and strip footings founded within sandstone of at least low strength or better may be designed for an allowable bearing capacity of 1200kPa, based on serviceability and subject to the completion on an additional cored borehole. In addition, an allowable shaft adhesion of 10% of the recommended bearing pressure may be used for rock sockets in medium to high strength sandstone or better provided the socket is satisfactory cleaned and roughened. All footings must be visually inspected by the geotechnical engineer.

Higher Bearing Pressures (such as 3500kPa) may be able to be adopted should the sandstone bedrock is of at least medium or higher strength subject to the completion of at least four cored boreholes on the site.

The allowable bearing pressures 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.

Geotechnical inspections of foundations should be carried out by a geotechnical engineer to determine that the required socket and founding material has been achieved and determine any variations that may occur between the boreholes and inspected locations.

7. 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 Pty Ltd and in the provided documents listed in Section 2 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 Pty Ltd 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 **Blue CHP Limited** 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.

The conclusions and recommendations of this report should be read in conjunction with the entire report.

For and on behalf of

JC Geotechnics Pty Ltd

Kaiyu Xu

Kaiyu (Kail) Xu

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

Laboratory Testing Results

MOISTURE CONTENT TEST REPORT

Client:	JC Geotechnics Pty Ltd	**Job No:**	S20077-1
Address:	Shop 2-4, 143-147 Parramatta Road, Concord, NSW 2137	**Report No:**	S58064-MC
Project:	Proposed New Dwelling (GR1094 1J)		

Test Procedure:

- ☐ AS 1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (Standard method).
- ☑ AS4133 1.1.1 Rock moisture content tests - 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 **Date Sampled:** 18/02/2020

Preparation: Prepared in accordance with the test method

Sample No.	Source	Sample Description	Moisture Content %
S58064	BH1 0.5-m	Rock Chips	**11.2**
S58065	BH1 1.5-m	Rock Chips	**14.4**
S58066	BH2 0.5-m	Rock Chips	**6.1**
S58067	BH3 0.5-m	Rock Chips	**8.3**
S58068	BH3 1.8-m	Rock Chips	**5.6**
S58069	BH4 0.5-m	Rock Chips	**15.1**

Notes:

NATA

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.

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

25/02/2020

Date:

MACQUARIE GEOTECH

Macquarie Geotechnical
U7/8 10 Bradford Street
Alexandria NSW 2015

APPENDIX C

Engineering Borehole Logs

Client: **Blue CHP Limited**
Project: **Proposed New Dwelling**

Borehole No: 1
Project No: GR1094.1J

Location: 20 - 22 Mindarrie St & 30 Pinaroo Pl, Lane Cove North, NSW		Elevation: ≈49.5m	Datum: AHD
Drilling Contractor: TightSite		Date Drilled: 18/02/2020	Logged By: KX
Drill Rig: Truck mounted		Date Completed: 18/02/2020	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	Fill: Silty Sand, dark brown, fine to medium grained, traces of gravel, organic matter, rootlets.	M				Appears to be poorly compacted
		0.5 1	░	SP	Sand: orange mottled brown, fine to medium grained.	M	L-MD D	◤	4,7,33/130mm R	Residual
		1.5 2	▨		Silty clay/extremely weathered sandstone, light grey and red brown.	MC<PL	H	◤	12/90mm R	
			▨		Same as above, colour changes to light red brown.	MS	M			
		2.5 3 3.5 4 4.5 5			End of borehole at 2.3m.					'TC-Bit' refusal on inferred low strength bedrock.

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

Client: **Blue CHP Limited**
Project: **Proposed New Dwelling**

Borehole No: 2
Project No: GR1094.1J

Location: 20 - 22 Mindarie St & 30 Pinaroo Pl, Lane Cove North, NSW	Elevation: ≈48.7m	Datum: AHD
Drilling Contractor: TightSite	Date Drilled: 18/02/2020	Logged By: KX
Drill Rig: Truck Mounted	Date Completed: 18/02/2020	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	Fill: Silty Sand, dark brown, fine to medium grained.	M				Appears to be poorly compacted
		0.5 1	▨		Silty clay/extremely westhered sandstone, light grey.	MC<PL	H	◤	30/110mm R	
			▨		Same as above, colour changes to light orange and red brown.					
		1.5 2	▨		Same as above, colour changes to grey.					
		2.5 3 3.5 4 4.5 5			End of borehole at 2.2m					'TC-Bit' refusal on inferred low strength shale bedrock.

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

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

Client: Blue CHP Limited

Project: Proposed New Dwelling

Borehole No: 3

Project No: GR1094.1J

Location: 20 - 22 Mindarie St & 30 Pinaroo Pl, Lane Cove North, NSW	Elevation: ≈43.0m	Datum: AHD
Drilling Contractor: TightSite	Date Drilled: 18/02/2020	Logged By: KX
Drill Rig: Truck Mounted	Date Completed: 18/02/2020	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	Fill: Clayey sand, dark brown, fine to medium grained, trace organic matter					
				SC	Clayey sand: orange, fine to medium grained.	M	L			Residual
		0.5			Silty Clay/extremely weathered sandstone, light brown.	MC<PL	H		30 R	
		1								
		1.5							12/10mm R	
		2			End of borehole at 2.0m					'TC-Bit' Refusal on inferred low strength shale bedrock.
		2.5								
		3								
		3.5								
		4								
		4.5								
		5								

Client: Blue CHP Limited

Project: Proposed New Dwelling

Borehole No: 4

Project No: GR1094.1J

Location: 20 - 22 Mindarie St & 30 Pinaroo Pl, Lane Cove North, NSW

Elevation: ≈48.0m

Datum: AHD

Drilling Contractor: TightSite

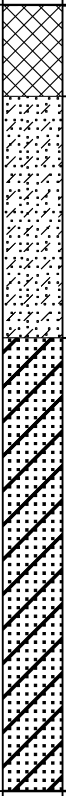
Date Drilled: 18/02/2020

Logged By: KX

Drill Rig: Truck Mounted

Date Completed: 18/02/2020

Checked By:



Ground Water Observation	Well Description	Depth	Graphic log	USCS Classification	Description	Field moisture content	Consistency	Field Sample(DS)	Field Tests	Remarks
		0		FILL	Fill: Sandy clay, dark brown, fine to medium grained, trace organic matter.	MC≈PL				Appears to be poorly compacted
		0.5 1		SP-SC	Sandy CLAY: low plasticity, orange mottled brown, fine to medium grained.	MC<PL	H	◩	22/60mm R	Residual
		1.5 2 2.5			Silty clay/extremely weathered sandstone, orange brown.	MC<PL	H			
		3 3.5 4 4.5 5			End of borehole at 2.6m.					'TC-Bit' refusal on inferred low strength shale bedrock

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

JC Geotechnics Pty Ltd

Sheet 1 of 1

APPENDIX D

Borehole Location Plan

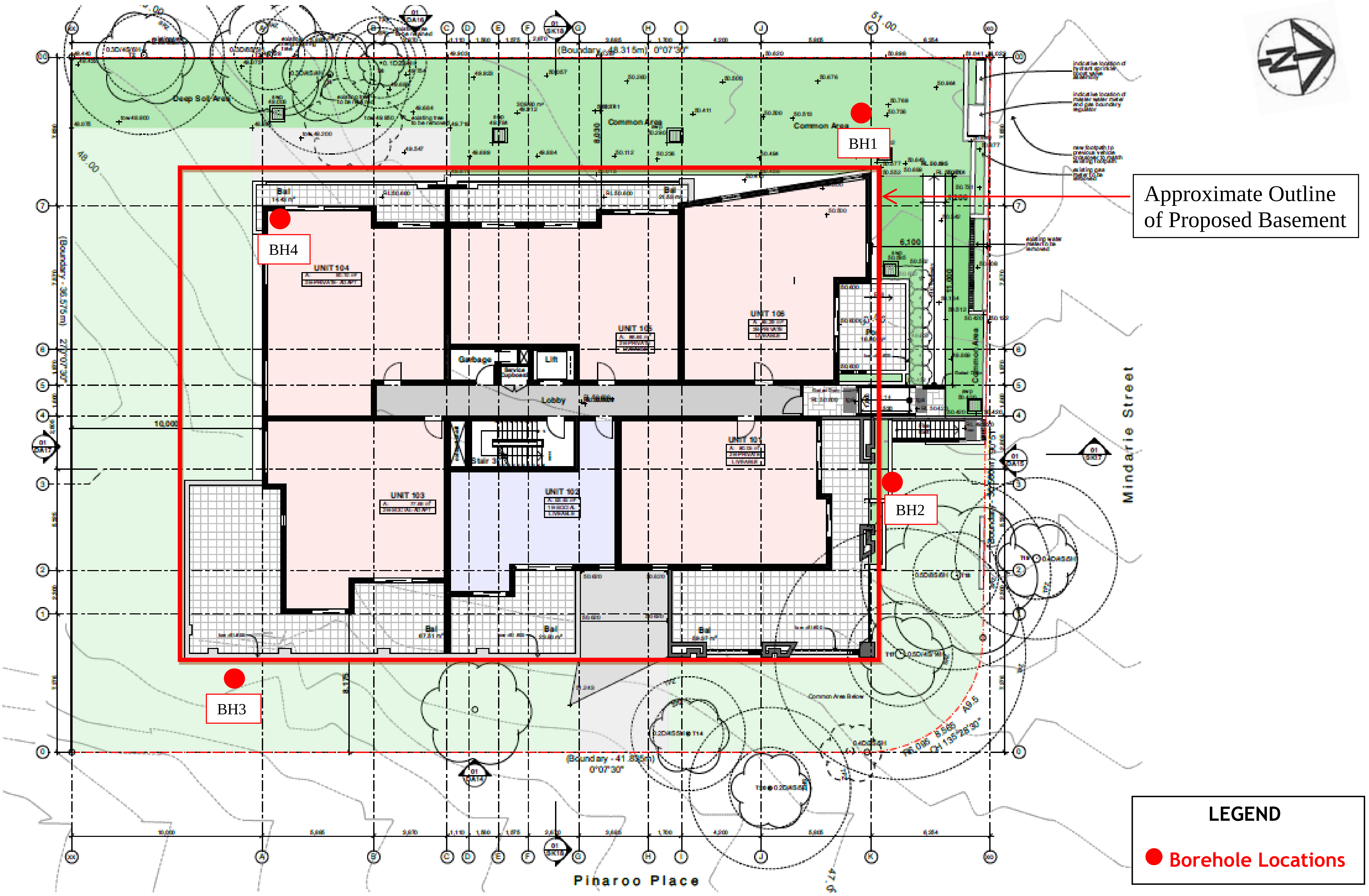


Image Source: Site & External Works Plan prepared by Stanton Dahl Architects, Project No. 2088.16, Drawing No. DA03, Revision 01, dated on 20/11/2020.

Drawn	DF
Checked by	JC
Date	25/11/2020
Scale	NTS

BlueCHP Limited
Geotechnical Investigation
20 & 22 Mindarie St & 30 Pinaroo Pl,
Lane Cove North, NSW

JC Geotechnics

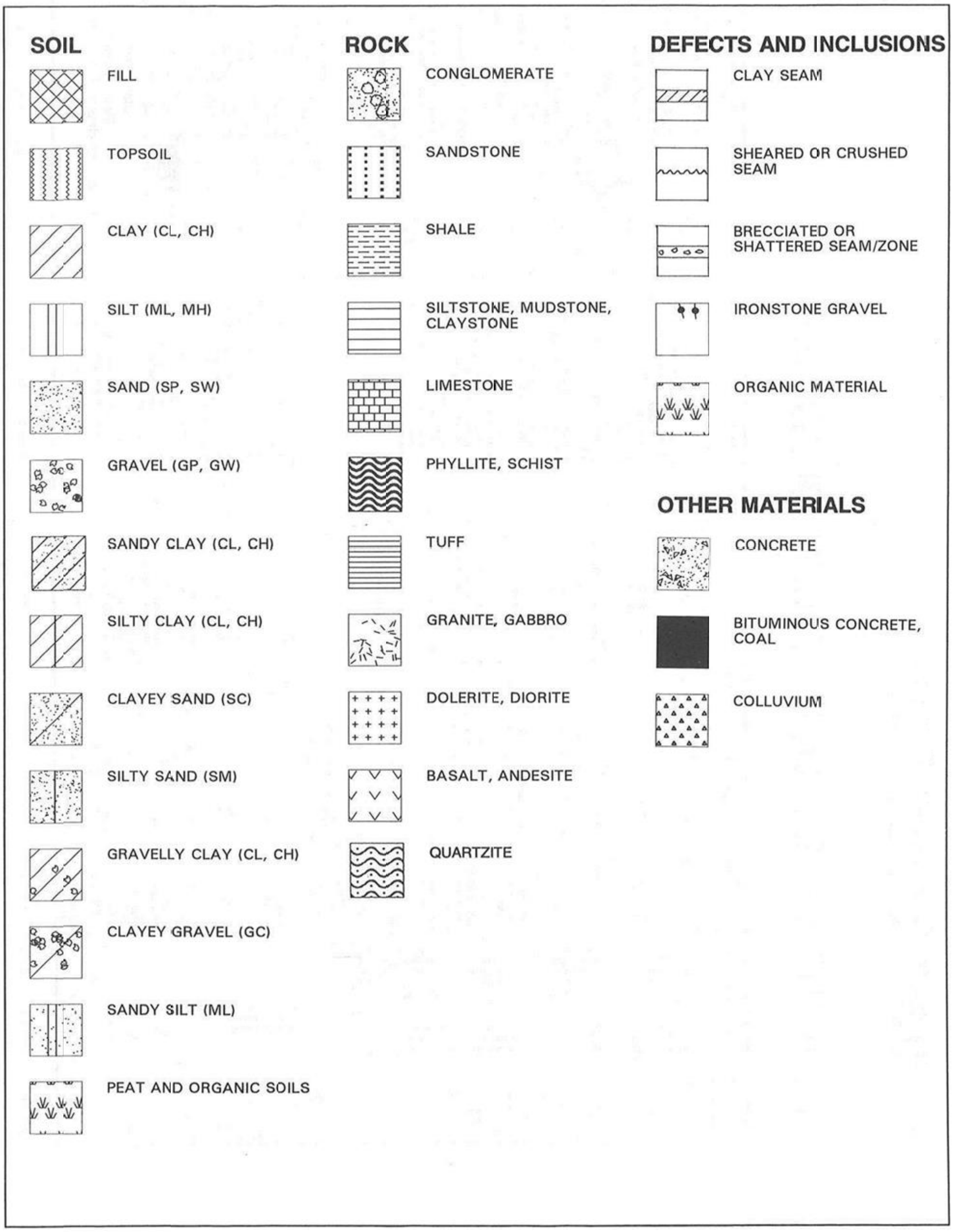
Figure	1
Title	**Approximate Borehole Location Plan**
Job No.	GR1094.1J

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.

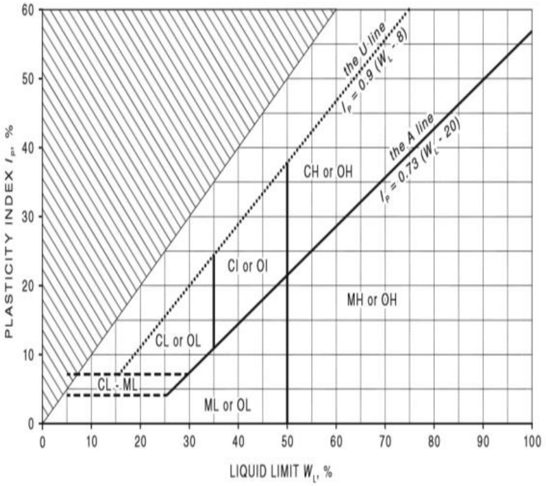
Soil Descriptions

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					
						% < 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
COARSE GRAINED SOILS (more than 65% of soil excluding oversize fraction is greater than 0.075 mm)	BOULDERS COBBLES	200									
	GRAVELS (more than half of coarse fraction is larger than 2.36 mm)	63	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. (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.
		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	
		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

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.

<table>
<tr><th colspan="5">Rock Material Weathering Classification</th></tr>
<tr><th colspan="2">Weathering Grade</th><th colspan="2">Symbol</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil (Note 1)</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported</td></tr>
<tr><td colspan="2">Extremely Weathered Rock (Note 2)</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible</td></tr>
<tr><td>Highly Weathered Rock (Note 2)</td><td rowspan="2">Distinctly Weathered (Note 2)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not 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</td></tr>
<tr><td>Moderately Weathered Rock (Note 2)</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognizable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered Rock</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock</td></tr>
<tr><td colspan="2">Fresh Rock</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

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