JC GEOTECHNICS

NOVEMBER, 2019

BCL2 Limited

GEOTECHNICAL INVESTIGATION REPORT

1-7 Anderson Avenue and
12 El Alamein Avenue,
Liverpool NSW 2170

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Document Control Record

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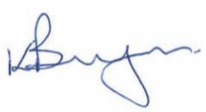
Author Signature		**Reviewer Signature**	
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APPENDICES

1. INTRODUCTION

JC Geotechnics Pty Ltd have been commissioned by BCL2 Limited to carry out a geotechnical site investigation at Nos. 1-7 Anderson Avenue and 12 El Alamein Avenue, Liverpool, NSW. The site investigation was carried out on the 28th August 2019 in accordance with our Proposal Ref. GP2019-008 dated 9th July 2019.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at four borehole location as a basis for comments and recommendations on the following:

- Detailed logs of the boreholes and groundwater observations;
- Excavation conditions;
- Support systems including design parameters;
- Hydrogeological considerations;
- Footings and bearing pressures;
- Basement floor slab; and
- Earthquake design parameters.

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

2. AVAILABLE INFORMATION

Prior to preparation of this report, the following documents were made available to JC Geotechnics:

- Survey drawing titled "Plan Showing Details and Levels" Site Layout Job BGR/001/01 Type S by YSCO Geomatics dated 14/12/2016.

The following architectural drawings for Job No. 08555, prepared by Kennedy Associates Architects.

- Drawing titled "Proposed Level 02", Drawing No. 1846-DA07 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 01", Drawing No. 1846-DA08 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 00", Drawing No. 1846-DA09 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 01", Drawing No. 1846-DA10 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 02", Drawing No. 1846-DA11 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 03", Drawing No. 1846-DA12 P2 and dated 13/08/2019.
- Drawing titled "Proposed Level 04", Drawing No. 1846-DA13 P2 and dated 13/08/2019.

- Drawing titled "Summary of GFA Calculations", Drawing No. 1846-DA28 P2 and dated 13/08/2019.

The provided architectural drawings indicate that the proposed development includes demolition of the existing dwellings and construction of a five storey residential building, with two underground basement levels. Vehicular access to the carpark levels will be via Anderson Avenue.

As the Finished Floor Level (FFL) of Basement 2 is at RL 9.450m AHD, the maximum depth of excavation required is expected to be approximately 7.8m below existing natural ground levels.

3. SCOPE OF WORK

In accordance with our proposal, the fieldwork for the geotechnical site investigation was carried out by a Geotechnical Engineer from JC Geotechnics Pty Ltd following in general the guidelines provided in Australian Standard AS 1726-2017 and comprised the following:

- A site walk-over inspection was carried out by a Geotechnical Engineer in order to determine the overall surface conditions and to identify relevant site features.
- Reviewed DBYD plans and engaged an external locator to carry out underground service locating using electromagnetic detection equipment at each borehole location.
- Four Boreholes (BH1 to BH4) were drilled using a track mounted drilling rig with a Tungsten Carbide 'TC' bit attached to the augers. The boreholes were augered to depths of approximately 9.0m (6.7m AHD) in BH1, 6.5m (9.4m AHD) in BH2, 6.5m (10m AHD) in BH3 and 6.15m (10.75m AHD) in BH4.
- The strength of the subsurface soils in the boreholes were assessed from Standard Penetration Testing (SPT) was carried out during augering.
- Groundwater levels were recorded during drilling and on shortly after completion of the boreholes.
- The approximate locations of the four boreholes performed during the geotechnical site investigation are shown on "Figure 1 – Borehole Location Plan" attached in Appendix B.

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. The borehole logs together with explanatory notes are attached in Appendix C.

4. SITE DESCRIPTION

The site is located within the City of Liverpool Council area, approximately 580m east of Cabramatta Creek and 40m west of the Liverpool Paramatta Transitway. The site is relatively flat approximately and rectangular in shape with a total area of approximately 3347m^2, consisting of the property Nos. 1-9 Anderson Avenue and 12 Alamein Avenue, Liverpool NSW.

At the time of our presence, the site comprised four single storey homes and associated garages. The houses appeared to be in reasonable external condition based on a cursory inspection. The site is covered is mainly grassed with a number of medium to large trees scattered around the property particularly in No.3 Anderson Avenue which is a vacant lot.

The site is bounded by the following:

- Anderson Avenue to the south;
- Hillier Road to the east;
- Alamein Avenue to the west; and
- Residential homes to the north comprising relatively recent double storey homes and an older single storey home.

5. SUBSURFACE CONDITIONS

Reference to the Penrith 1:100,000 Geological Series Sheet 9030 Edition 1, dated 1991, by the Geological Survey of New South Wales, Department of Mineral Resources, indicates the site is located within a geological area underlain by Triassic Age Bringelly Shale (Rwb) of the Wianamatta Group. The Bringelly Shale is described as "shale, carbonaceous claystone, laminite, fine to medium grained lithic sandstone, rare coal". The site is located close to the boundary of the geological area underlain by Quarternary Age unconsolidated sediments (Qpn) comprising "medium grained sand, silt and 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.

The subsurface profile encountered in the boreholes was generally consistent in that it comprises fill and residual clay up to a depth of 9m in BH1 and shale encountered only in one boreholes (BH4). Reference should be made to the attached borehole logs (Appendix C) for detailed descriptions of the subsurface profile at each specific location.

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

Fill was encountered in all boreholes extending to depths ranging between 1.0m and 1.7m, below existing surface levels. the deepest fill was found in BH4 to depth of about 1.7m. Other areas of fill, perhaps of greater depths, might be present within the site. Generally, the fill comprised a relatively thin layer of silty sand overlying sandy clay of low plasticity with occasional trace of organic fibres and sandstone gravel. The fill appeared to be poorly compacted. This assessment is mostly based on our observations during drilling and on SPT tests, which do not give precise determination of in situ densities since they are affected by friction, the presence of gravel and the moisture content of the fill. Nonetheless, they provide a qualitative guide.

Residual Silty Clays were found below the fill to depths of 9.0m at the base of BH1 and 6.5m at the base of BH2 and BH3. The residual clay was present to a depth of 5.8m in BH4. The silty clays were generally of high plasticity and of stiff consistency.

Shale bedrock was encountered in BH4 only, at a depth of 5.8m to the base of the borehole at 6.15m. The shale was assessed to be extremely weathered and of extremely low strength.

Groundwater levels were measured six hours after completion of drilling of BH1 at a depth of 3.6m (12.1m AHD) and one hour after drilling in BH3 at a depth of 5.0m (11.5m AHD). In BH2, groundwater was measured three hours after drilling at a depth of 4.8m (11.1m AHD). In BH4, groundwater was observed at the base of the borehole (6.15m - 10.75m

AHD) after drilling. However, these groundwater levels may not be a true representation of site groundwater levels due to the limited time of observation.

6. COMMENTS AND RECOMMENDATIONS

6.1 Dilapidation Reports

Prior to the commencement of demolition and excavation, we recommend that detailed dilapidation reports be compiled on the neighbouring structures and infrastructures which falls within the 2H zone of influence of the excavation, where H is the height of the excavation, and the respective owners asked to confirm that the reports present a fair record of existing conditions The report may then be used to as a benchmark against which to assess possible future claims arising from the works.

During the excavation works, utmost care must be taken to not undermine or render unstable, the footings of the adjoining structures, retaining walls, etc. that falls within the zone of influence of the excavation (including service trenches, etc.).

6.2 Excavation Conditions

Prior to any excavation commencing, we recommend that reference be made to the WorkCover Excavation Work Code of Practice – July 2015.

Based on the borehole results, the proposed excavation is mainly expected to be through fill and residual clay soil with some extremely low strength shale also likely (particularly at the north western end of the site).

Excavation within any soils and extremely low to very low strength shale is expected to be readily achieved using a large hydraulic excavator, particularly if fitted with ‘tiger teeth’, down to the level of low or stronger bedrock. However, localised use of rock breaking equipment or ripping may be required where high strength bands are encountered.

For low or greater strength rock (if encountered), excavation will require the use of heavy ripping and/or hydraulic rock hammers such as a D9 dozer or similar for effective production. Excavation for foundations or trenches will require the use of hydraulic hammers and possibly a rock saw. The use of a smaller size bulldozer may result in lower productivity and higher wear and tear, and this should be allowed for. Grid sawing techniques with ripping or hammering will also facilitate the excavation. Both noise and vibration will be generated by the proposed excavation work within these bedrock materials.

Should rock hammers be used for hard rock excavation, vibration monitoring will be required, and further advice should be sought from JC Geotechnics. 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.

6.3 Stability of Basement Excavation

As excavation of the proposed basement levels will extend to approximately 7.8m depth temporary shoring should be provided to support the full excavation depth. Shoring design should consider both short term (construction) and permanent conditions as well as the presence of adjacent buildings and roads. Consideration may be given to batter slope and shoring system where the shoring supports a 1.5H:1V batter slope with the toe of the slope located at least 1m above the highest groundwater level.

As the observed groundwater level is above the proposed lowest basement level, consideration may given to either full drainage through the basement retention system or a cut-off piled wall if inflows are too high. For a drained basement, some drawdown of the groundwater could result in differential settlement beneath the adjacent existing buildings, infrastructure and utilities. Care should therefore be taken to ensure that any drawdown of the groundwater does not impact on adjacent infrastructure. This can be done by carrying out pump out tests and subsequent numerical seepage modelling to assess settlement of adjacent footings and to confirm inflow rates are acceptable for control by sump and pump methods and that therefore that a cut-off wall is not required. For a drained basement, consideration should be given to a contiguous pile wall arrangement with shotcreting between piles to provide a more robust system and to minimise any slumping of concrete into the excavation. Weep holes should be installed to provide drainage behind the shoring wall. If a cut-off wall is required, then a secant piled arrangement should be provided to effectively provide an near impermeable barrier to any groundwater inflow and therefore minimise risk of impact from groundwater drawdown on adjacent infrastructure.

For the maximum retained height being considered, a permanent anchorage system is likely to be required to provide the required lateral support during construction. As two or more rows of anchors are required to support the shoring or where significant lateral movements cannot be tolerated (e.g. due to adjacent infrastructure), the shoring/basement wall should be designed as a braced structure. Anchor designs should be based on allowing effective bonding to be developed behind an ‘active zone’ determined by drawing a line at 45° from the base of the wall to intersect the ground surface behind the excavated face. It is considered that basement floor slabs will provide permanent restraint to the retaining walls where these are incorporated into the permanent structure. Anchors are therefore considered to be temporary but depending on the sensitivity of the adjacent infrastructure and the height of the retaining walls, it may be necessary to incorporate the temporary anchors into the permanent works to control deflections.

Anchor installation beyond the property boundaries will be subject to approval by owners of adjoining properties, roads and infrastructure. Where an anchorage system is shown to be impractical (e.g. due to low bond stress in the clay), consideration of other temporary support options would be necessary. These options include the following:

- Temporary solutions such as installation of props associated with staged excavation; and
- Staged excavations and temporary partial berms in front of walls.
- Top-down construction where floor slabs and beams are constructed at the top of shoring wall and at floor levels of the upper basement levels prior to excavation within the basement level underneath the floor slabs.

The shoring wall and anchors can be designed using the recommended parameters provided in Section 6.7 below.

Detailed design of anchored or propped retaining walls should utilise commercial software packages such as ROCSCIENCE or PLAXIS that can model the sequence of anchor installation and excavation to ensure deflections are within tolerable limits. The design of retaining structures should to take into account horizontal pressures due to surcharge loads from any adjacent infrastructure.

Detailed construction supervision, monitoring and inspections will be required during piling and subsequent bulk excavation and should be carried out by an experienced Geotechnical Engineer, in addition to inspection of the structural elements by the Project Structural Engineer. The inspections should constitute as "Hold Points".

6.4 Recommended Design Earth Pressures

The parameters in Table 1 may be used for design of temporary and permanent retaining walls at the subject site:

Table 1: Preliminary Geotechnical Design Parameters for Retaining Walls

Units	Unit Weight (kN/m^3)	Effective Cohesion c' (kPa)	Angle of Friction ϕ' (°)	Modulus of Elasticity E_{sh} (MPa)
Fill	19	0	26	5
Residual Soils	20	5	24	10
Shale	22	25	30	65

Table 2 below provides preliminary coefficients of lateral earth pressure for the soils and rocks, based on horizontal ground surface and fully drained conditions.

Table 2: Preliminary Coefficients of Lateral Earth Pressure

Units	Coefficient of Active Lateral Earth Pressure Ka	Coefficient of Active Lateral Earth Pressure at Rest Ko	Coefficient of Passive Lateral Earth Pressure Kp
Fill	0.39	0.53	2.55
Residual Soils	0.42	0.47	2.38
Shale	0.3	0.5	3.0

The following recommendations are provided for the design and construction for the shoring wall;

- 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 4H kPa for soil and shale bedrock, 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 8H kPa for soil and shale bedrock, 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 a drained basement, strip drains protected with a non-woven geotextile fabric should be 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;
- The values in Table 2 assume 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 any 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 any neighbouring basements and/or services and their levels must be confirmed prior to finalising anchor design.
- Anchor design and installation should 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 1.3 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.

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.

Assessment of the potential impact of the proposed development on Sydney Water assets (if any) may also be required.

6.5 Hydrogeological Considerations

Groundwater was encountered in all boreholes with the highest standing groundwater level measured (six hours after completion of drilling of BH1) at a depth of 3.6m (12.1m AHD) which is approximately 2.65m above the proposed basement level and possibly 3m above bulk excavation level. However, these groundwater levels may not be a true representation of site groundwater levels due to the limited time of observation. It should be noted that groundwater levels may be subject to seasonal and daily fluctuations influenced by factors such as rainfall and future development of the surrounding lands.

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.

A dewatering management plan including the installation of three monitoring wells, groundwater sampling, groundwater quality testing and groundwater modelling may be required for submission to WaterNSW should an application for Approval for Water Supply Works and/or Water Use be required.

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

We recommend detailed groundwater monitoring including the installation of at least three groundwater monitoring wells for monitoring of groundwater levels and pump out tests within the monitoring wells for estimation of seepage volumes into bulk excavation.

6.6 Foundations

Ground conditions comprise poorly compacted fill up to 1.7m depth overlying residual clay soil to at least 9m depth and extremely low strength shale bedrock at 5.8m (11.1m AHD) depth at the northwest end of the site. Consequently, it may be inferred that rock could be dipping towards the east. Suitable foundations are therefore likely to comprise footings supported by piles socketed into the shale bedrock at a minimum depth of at least 7.0m in the north east corner and deeper piles required for the rest of the site. An allowable bearing capacity of 700kPa may be adopted for the extremely low strength shale. However, it is recommended that additional boreholes be carried out to assess rock depth across the site, with cored boreholes potentially being required if an allowable bearing pressure in excess of 700kPa is required. Shaft friction should be ignored for fill and residual clay soils. However for piles socketed into bedrock, an allowable shaft friction of 25kPa may be adopted for the rock socket.

Bored piles socketed into bedrock, may be considered for support of foundations but may be unsuitable due to the need for sacrificial liners to support the overburden soils and will require dewatering during pile installation. Consideration should therefore also be given to Continuous Flight Auger (CFA) piles which can be installed relatively quickly and generally generate lower noise and vibration than bored and driven piles. However, construction of CFA piles usually is associated with deviation of piles vertically with potential for pile necking and honey-combing and requires strict quality control procedures.

The actual pile capacity will vary subject to method of installation, pile dimensions and material present at the pile toe. The material strength required at pile toe level should be present over a length of at least 3 x diameter below pile toe.

Design of pile foundations should be carried out in accordance with Australian Standards AS 2159-2009 Piling – Design and Installation.

It is recommended that an experienced Geotechnical Engineer should inspect foundations to confirm bedrock has been reached and compliance with the recommendations in the geotechnical report has been achieved. Foundation inspections should only be undertaken under conditions satisfying WHS requirements.

6.7 Floor Slab

Following bulk excavation for the proposed basement, residual clay is mainly expected to be exposed at the base. Following the removal of all loose and softened materials, we recommend that perimeter and underfloor drainage be provided and should comprise a strong, durable, single sized washed aggregate such as 'blue metal gravel'. Joints in the concrete floor slab should be designed to accommodate shear forces but not bending moments by using dowelled and keyed joints. The basement floor slab should be isolated from columns. The completed excavation should be inspected by the hydraulic engineer to confirm the extent of the drainage required.

Permission may need to be obtained from WaterNSW and possibly Council for any permanent discharge of seepage into the drainage system. However, if permission for discharge is not obtained, the basement may need to be designed as a tanked basement.

6.8 Preliminary Site Earthquake Classification

The results of the site investigation indicate the presence of fill and residual soil and underlain by shale bedrock. In accordance with Australian Standard AS 1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia" the site may be classified as a "Shallow soil site" (Class C_e) for design of foundations and retaining walls embedded in the underlying weathered Shale. The Hazard Factor (Z) for Sydney in accordance with AS 1170.4-2007 is considered to be 0.08.

6.9 Summary of Further Geotechnical Work

It is recommended that further geotechnical work be carried out as follows:

- Additional geotechnical investigation comprising cored boreholes to assess depth to rock across the site.
- Installation of piezometers for groundwater monitoring and carry out pump out tests.
- Geotechnical modelling to assess groundwater inflow and impact on adjacent buildings.
- Dewatering Management Plan to satisfy Department of Primary Industries (DPI) requirements.

7. LIMITATIONS

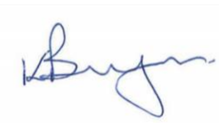
The conclusions and recommendations presented in this report are indicative of the subsurface conditions at the site only at the specific sampling or testing locations and then only to the depths investigated and at the time the work was carried out. Subsurface conditions can change abruptly due to variable geological processes and as a result of human influences. Such changes may occur subsequent to JC Geotechnics geotechnical investigation.

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

This report and the information herein has been prepared solely for the use of **BCL2 Limited** for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report, then all recommendations should be reviewed. Copyright of this report is the property of JC Geotechnics Pty Ltd. We have used a degree of care, skill and diligence normally exercised by Consulting Engineers in similar circumstances and locality. 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



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APPENDIX A

About Your Report

JC GEOTECHNICS

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

Investigation Location Plan

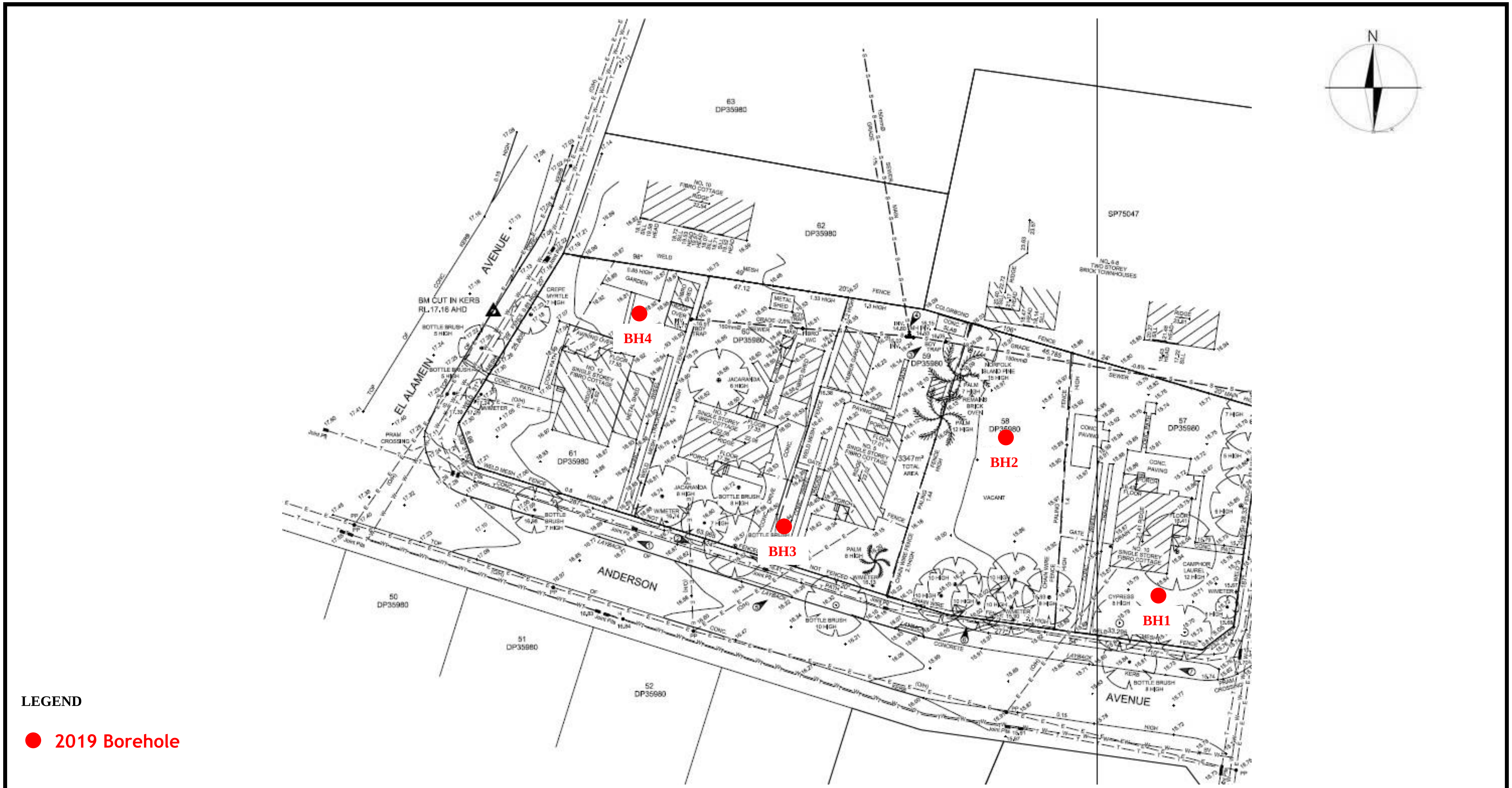

Image Source: Drawing Title "Site Layout", Prepared by Blue CHP Limited, Project No. BGR64, Job No. 01, Drawing No. 001, Issue No. S, dated on 13 August 19.

Drawn by	RKC	**BCL2 Limited** **Geotechnical Investigation Report** **Proposed New Dwelling** **1-7 Anderson Avenue and 12 El Alamein Avenue, Liverpool, NSW**	Figure: 1
Checked by	JC		Title: **Borehole Location Plan**
Date	06/09/2019		
Scale	NTS	JC Geotechnics	Job No: **GR1036.1J**

APPENDIX C

Borehole Logs and Explanatory Notes

JC Geotechnics Pty Ltd
Shop 2-4, 143-147 Parramatta Road, Concord, NSW
T:02 8066 0665
E: admin@jcgeotechnics.com.au

BOREHOLE NUMBER 1

CLIENT BCL2 Limited
PROJECT NAME Proposed New Dwelling
PROJECT NUMBER GR1036.1J
PROJECT LOCATION 1-7 Anderson Avenue and 12 El Alamein Avenue

DATE STARTED 28/8/19 **COMPLETED** 28/8/19
R.L. SURFACE 15.7 m approx **DATUM** AHD
DRILLING CONTRACTOR Hagstrom Drilling
SLOPE 90° **BEARING** N.A.
EQUIPMENT Hanjin D & B DR022
HOLE LOCATION See Figure 1 - Borehole Location Plan
HOLE DIAMETER 100 mm
LOGGED BY RKC **CHECKED BY**
NOTES Depths are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Moisture	Con. / Den.	Additional Observations
Augered Drilling Tungsten Carbide bits			0.0 – 0.3			FILL: Silty SAND, fine to medium grained, brown, trace of roots.		M		APPEARS MODERATELY COMPACTED
			0.3 – 1.5			FILL: Sandy CLAY, low plasticity, grey mottled, orange-brown and red-brown, trace of fibres.		MC < PL	F	
	After 6 hours of drilling (approx. 3.7 m)		1.5 – 6.8		CH	Silty CLAY, high plasticity, grey mottled orange-brown and red-brown.	SPT 1, 1, 5 N=6 SPT 3, 8, 11 N=19 SPT 6, 7, 10 N=17 SPT 3, 4, 8 N=12	MC > PL	F Vst St	RESIDUAL
			6.8 – 8.95			Silty CLAY, high plasticity, grey mottled orange-brown and red-brown, trace of fine to medium sandstone gravel with iron indurated bands.	SPT 2, 5, 9 N=14 SPT 2, 3, 4 N=7		St F	
			8.95			Borehole 1 terminated at 8.95m				

BOREHOLE NUMBER 2

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CLIENT BCL2 Limited
PROJECT NAME Proposed New Dwelling
PROJECT NUMBER GR1036.1J
PROJECT LOCATION 1-7 Anderson Avenue and 12 El Alamein Avenue

DATE STARTED 28/8/19 **COMPLETED** 28/8/19
R.L. SURFACE 15.9 m approx **DATUM** AHD
DRILLING CONTRACTOR Hagstrom Drilling
SLOPE 90° **BEARING** N.A.
EQUIPMENT Hanjin D & B DR022
HOLE LOCATION See Figure 1 - Borehole Location Plan
HOLE SIZE 100 mm
LOGGED BY RKC **CHECKED BY**
NOTES Depths are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Moisture	Con. / Den.	Additional Observations
Augered Drilling Tungsten Carbide bits			0.0			FILL: Silty SAND, fine to medium grained, brown, trace of organic fibres.		M		APPEARS MODERATELY COMPACTED
			0.4			FILL: Sandy CLAY, low plasticity, grey mottled orange-brown and red-brown.		MC < PL	F	
			1.6		CH	Silty CLAY, high plasticity, grey mottled orange-brown and red-brown.	SPT 4	MC > PL	Vst	RESIDUAL
							SPT 7, 10 N=17			
							SPT 3, 6, 11 N=17			
						4.0 m, colour changed to grey.				
	After 3 hours of drilling		4.5			Silty CLAY, high plasticity, grey mottled orange-brown and red-brown.	SPT 3, 8, 14 N=22			
						5.50 m, trace of fine to medium grained sandstone gravel with iron indurated bands.				
							SPT 3, 6, 8 N=14		St	
			6.5			Borehole 2 terminated at 6.5m				

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BOREHOLE NUMBER 3

CLIENT BCL2 Limited
PROJECT NAME Proposed New Dwelling
PROJECT NUMBER GR1036.1J
PROJECT LOCATION 1-7 Anderson Avenue and 12 El Alamein Avenue

DATE STARTED 28/8/19 **COMPLETED** 28/8/19 **R.L. SURFACE** 16.5 m approx **DATUM** AHD
DRILLING CONTRACTOR Hagstrom Drilling **SLOPE** 90° **BEARING** N.A.
EQUIPMENT Hanjin D & B DR022 **HOLE LOCATION** See Figure 1 - Borehole Location Plan
HOLE SIZE 100 mm **LOGGED BY** RKC **CHECKED BY**
NOTES Depths are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Moisture	Con. / Den.	Additional Observations
Augered Drilling Tungsten Carbide bits			0.0			FILL: Silty SAND, fine to medium grained, brown, trace of organic fibres.		M		APPEARS POORLY TO MODERATELY COMPACTED
						FILL: Sandy CLAY, low plasticity, brown mottled orange-red, trace of organic fibres.		MC < PL	F	
			1		CH	Silty CLAY, high plasticity, grey mottled orange-brown and red-brown, trace of organic fibres.		MC > PL	St	RESIDUAL
						Silty CLAY, high plasticity, orange-brown and red-brown mottled grey, trace of organic fibres.	SPT 3, 4, 9 N=13			
			3			3.0 m, trace of fine grained sandstone gravel with iron indurated bands.	SPT 4, 8, 13 N=21		Vst	
						Silty CLAY, high plasticity, orange-brown and red-brown, mottled grey, trace of fine to medium grained sandstone gravel with iron indurated bands.				
	After 1 hour of drilling (5.0 m)					4.60 m, change in colour to grey mottled orange-brown and red-brown.	SPT 5, 9, 13 N=22			
						Silty CLAY, high plasticity, orange-brown and red-brown, with fine grained sandstone gravel with iron indurated bands.	SPT 6, 9, 10 N=19			
			6.5			Borehole 3 terminated at 6.5m				

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BOREHOLE NUMBER 4

CLIENT BCL2 Limited
PROJECT NAME Proposed New Dwelling
PROJECT NUMBER GR1036.1J
PROJECT LOCATION 1-9 Anderson Avenue and 12 El Alamein Avenue

DATE STARTED 28/8/19 **COMPLETED** 28/8/19
R.L. SURFACE 16.9 m approx **DATUM** AHD
DRILLING CONTRACTOR Hagstrom Drilling
SLOPE 90° **BEARING** N.A.
EQUIPMENT Hanjin D & B DR022
HOLE LOCATION See Figure 1 - Borehole Location Plan
HOLE SIZE 100 mm
LOGGED BY RKC **CHECKED BY**
NOTES Depths are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Moisture	Con. / Den.	Additional Observations
Augered Drilling Tungsten Carbide bits	Trace of water at bottom after drilling					FILL: Silty SAND, fine to medium grained, brown, trace of organic rootlets.		M		APPEARS POORLY COMPACTED
			1			FILL: Sandy CLAY, low plasticity, grey mottled, orange-brown and red-brown, fine to medium grained sandstone gravel with iron indurated bands.	SPT 3	MC < PL	F	
			2 3		CH	Silty CLAY, high plasticity, orange-brown and red-brown mottled grey, brown, trace of fine to medium grained sandstone with iron indurated bands and charcoal.	SPT 6, 9 N=15	MC > PL	St	RESIDUAL
			4			Silty CLAY, high plasticity, grey mottled orange-brown and red-brown, trace of organic fibres.	SPT 4, 4, 8 N=12			
			5			Silty CLAY, high plasticity, orange-brown and red-brown mottled grey, trace of fine to medium grained sandstone with iron indurated bands.	SPT 4, 6, 10 N=16		Vst	
			6			SHALE, dark brown, orange-brown and red-brown, extremely weathered.	SPT 50mm/30/R	EL		WEATHERED MATERIAL
			7 8 9 10			Borehole 4 terminated at 6.15m				

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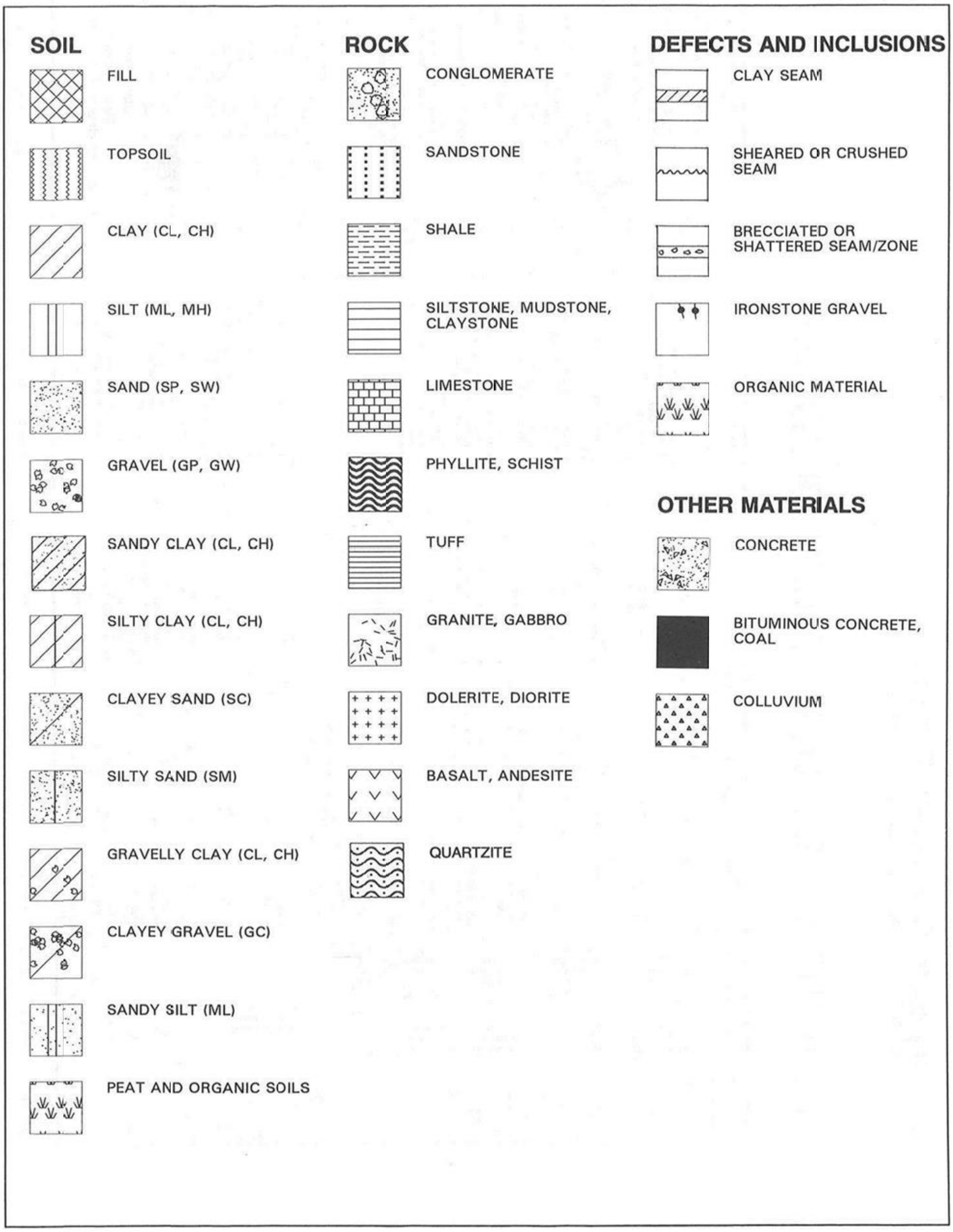
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



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.

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Soil Descriptions

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

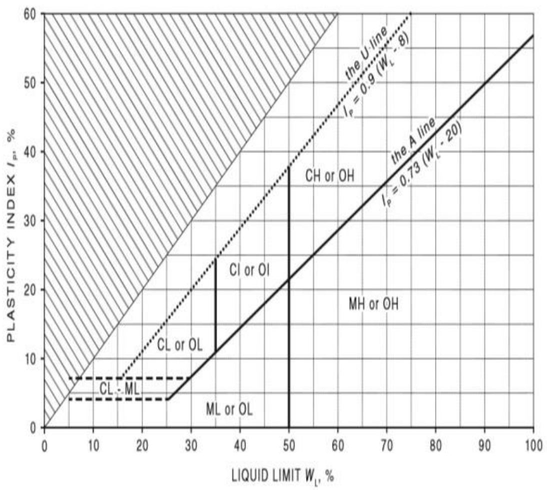
The following chart (adapted from AS1726-2017, Clause 6.1.6, Table A1) is based on the Unified Soil Classification System (USCS).

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 20	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		
		medium 6	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.
		fine 2.36	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)		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

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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>
<tr><td colspan="5">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.</td></tr>
</table>

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

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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

Geotechnics