REPORT TO

THE YARD 120c PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Date: 20 March 2020
Ref: 33019SFrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical assessment for a proposed residential development at 120c Old Canterbury Road, Summer Hill, NSW. The investigation was commissioned by Mr Jamie Howieson of The Yard 120c Pty Ltd by email dated 24 February 2020, and was carried out in accordance with Section 2 of our fee proposal, Ref: P51289S, dated 24 February 2020.

Based upon the provided architectural drawings prepared by Fox Johnston (Dwg. A-200-001 and 002, Rev DA02 undated), we understand that it is proposed to construct a multi-storey residential development over three basement levels. The lowest basement Finished Floor Level (FFL) is RL0.5m, which will generally results in excavations about 9.3m deep to achieve Bulk Excavation Level (BEL), except at the southern end of the site where excavations will extend approximately 16.7m below existing street level. The basement will be set back approximately 1m from the eastern, southern and western boundaries to allow for a deep soil area. The Light Rail corridor is adjacent to the site along the western boundary and the Sydney Water Hawthorne Canal wraps around the site along the eastern boundary. Old Canterbury Road and the associated bridge abutments spanning over the Light Rail are present along the southern boundary.

JK Geotechnics carried out a geotechnical investigation at the subject site in 2013 for a different proposed development; the available information has been used to assist with the preparation of this report. The results of the previous investigation have been provided as attachments to this report.

The purpose of this report is to review geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention systems, hydrogeological considerations, footings, basement slab, earthquake design factors and pile exposure classification.

2 ASSESSMENT PROCEDURE

2.1 Current Assessment

The assessment comprised:

- A site walkover on 3 March 2019.
- Review of the latest architectural drawings.
- Review of the previous geotechnical results, the results of which have been summarised in Section 2.2 below.
- Review of aerial photography and regional geology map.

2.2 Previous Geotechnical Investigation

The investigation was carried out on 31 May 2013 and comprised the drilling of five boreholes using our JK300 track mounted rig and completion of two Dynamic Cone Penetrometer (DCP) tests. The boreholes were auger drilled using a tungsten carbide (TC) bit to depths between of 3.6m and 4.8m below existing surface levels, where TC bit refusal occurred.

The apparent compaction of the fill and strength of the subsurface soils were assessed from the Standard Penetration Test (SPT) 'N' values, augmented by the results of hand penetrometer readings on cohesive soil samples recovered in the SPT split tube sampler. The strength of the weathered bedrock was assessed from observations of drilling resistance of the TC auger bit, examination of the recovered rock cuttings and correlation with subsequent laboratory moisture content tests. It should be noted that rock strengths assessed in this way are only estimates, and variances of one strength order should not be unexpected.

The purpose of the DCP tests was to interpret the degree of compaction/relative density of the fill embankment, on which Old Canterbury Road is located, and attempt to probe the depth of the underlying sandstone bedrock. It should be noted that the DCP may refuse on hard layers or obstructions in the fill, ironstone bands within the residual clays or sandstone 'floaters'.

Groundwater observations were made during and on completion of drilling. No longer term groundwater monitoring was undertaken as part of the investigation.

The fieldwork was carried out under the direction of our engineering geologist who was present on a full-time basis to set out the exploratory hole locations, direct the buried service scan, log the encountered subsurface profile and nominate sampling and in-situ testing.

Selected soil and rock chip samples were submitted to NATA registered laboratories for testing to determine moisture content, liquid limits, linear shrinkage, pH values, chloride and sulphate content. The results of the laboratory testing are summarised in the attached Soil Test Service Pty Ltd (STS) Table A and Envirolab Services Pty Ltd Certificate of Analysis No 91775.

The investigation locations, as shown on Figure 1, were set out by taped measurements from existing surface features and apparent site boundaries. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the general site arrangement plan prepared by SRB Consulting Civil Engineers. The survey datum is Australian Height Datum (AHD). The borehole logs and DCP test results are included with this report, together with Explanatory Notes, which describe the methods and procedures employed in the investigation, their limitations and define the logging terms and symbols used.

3 RESULTS OF ASSESSMENT

3.1 Site Description

The site is situated on the lower reaches of a west facing valley side, which slopes down at approximately 5°. The majority of the site is relatively level and flat, with the site surface comprising asphaltic concrete, assessed to be in a poor condition. The southern end of the site comprised a heavily vegetated north facing fill embankment, which sloped at between 25° and 45°.

At the time of the assessment, the site contained construction materials, such as scaffolding, pallets, etc, and shipping containers, with vegetation and mature trees (up to 9m high) located around the perimeter of the site. The site is bound to the west by a railway line, which is up to 1.8m higher than the subject site at the southern end reducing in height to about 0.4m at the northern end. The site is bound to the north and east by the Hawthorne Canal, which runs within a concrete lined channel. The canal flows from south to north, with the invert of the culvert measured to be approximately 2.5m below site level at the southern end and up to 4.3m at the northern end. Cracking was noted in the concrete side walls, when viewed from the site. The upper banks of the canal were heavily vegetated with shrubs and small trees. The site is accessed via a concrete vehicle bridge which spans the canal, and appeared to be in good condition when viewed from the site. Old Canterbury Road situated along the top of the fill embankment bounds the site to the south and is approximately 7.0m higher than the main part of the site.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Map of the Sydney Region indicates that the site is situated close to the boundary of the Ashfield Shale and an old river valley, which has possibly been dredged and/or filled Hawkesbury Sandstone outcrops further to the north and would be expected to occur at relatively shallow depth below the valley in which the site is located. Please note, the geological map does not take into account previous earthworks cut and fill at the site.

The boreholes revealed a subsurface profile comprising fill over residual clays overlying sandstone bedrock. A summary of the encountered subsurface features are presented below. For a more detailed description of materials encountered, reference should be made to the attached borehole logs.

Pavements/Fill

A 20mm thick asphaltic concrete pavement was penetrated at four of the five borehole locations, with BH1 encountering a gravelly sand fill at the surface. Fill was encountered to depths of 0.3m (BH4) and 2.1m (BH2) and typically comprised gravelly sand and sandy gravelly clay, with sandstone, igneous, brick and ash fragments. The fill was assessed to be poorly compacted from the SPT results.

Residual Silty Clay

Silty clay was encountered beneath the fill in BH3, BH4 and BH5 to depths of 3.0m, 2.1m and 3.0m, respectively, and was assessed to be of high plasticity and stiff to hard strength. These silty clays are likely the result of weathering of the shale, as these clays were encountered within the southernmost boreholes which are located close to the boundary with the Ashfield Shale.

Residual Sandy Clay

Sandy clay was encountered beneath the fill in BH1 and BH2, and beneath the residual silty clay in BH3, and was assessed to be of low plasticity and stiff to very stiff strength. These sandy clays are the result of weathering of the underlying sandstone bedrock.

Weathered Sandstone
Sandstone bedrock was encountered in all the boreholes at depths ranging from 1.85m (BH1) to 3.4m (BH3), or between about RL7.7m to RL8.1m. The sandstone was assessed to be low to medium strength on initial contact improving to medium (to high) strength with depth. All five boreholes were terminated due to TC bit refusal within the sandstone. It appears that the sandstone steps down or grades down to the east, towards the Hawthorne Canal.

Groundwater
Three of the five boreholes were noted to be dry on completion of drilling; BH2 encountered water seepage within the fill at 2.0m depth (≈RL 7.75m) and BH4 encountered water seepage within the sandstone bedrock at 2.5m depth (≈RL 7.3m).

The boreholes were left open for the duration of the fieldwork with groundwater levels measured up to 4 hours after drilling:

Borehole	Time after Completion	Groundwater Depth (m)	Approximate Groundwater Reduced Level (m AHD)
BH1	1 hour	2.5	RL 7.30m
BH1	4 hours	2.1	RL 7.70m
BH2	0 hours	2.0	RL 7.75m
BH2	3 hours	2.0	RL 7.75m
BH3	1 hour	2.7	RL 7.05m
BH3	2 hours	1.7	RL 8.05m
BH4	0 hours	2.1	RL 7.70m
BH5	0 hours	Dry*	-

* Borehole backfilled on completion

3.3 Laboratory Test Results

Based on the liquid limits and linear shrinkage test results, the residual silty clays are of high plasticity and are assessed to have high shrink/swell potential with changes in moisture content.

The samples of the fill returned pH values of 8.3, indicating slightly alkaline conditions, while the sample of the residual silty clay returned a pH value of 5.7, indicating moderately acidic conditions. The sulphate contents of the fill and residual clay ranged from 54mg/kg to 120mg/kg and the chloride contents ranged from 15mg/kg to 35mg/kg, indicating low sulphate and chloride contents.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Issues

The main issues for the site are the proximity of the Sydney Light Rail, Old Canterbury Road and Hawthorne Canal, as well as the presence of the steep fill embankment at the southern end of the site. The development will need to satisfy the relevant authorities that it will not adversely affect these assets. The constructability of the retention system along the southern boundary due to the existing fill embankment is difficult and will need to be adequately assessed once further details of the retention system are known. The proposed basement excavation will also likely extend below the groundwater table and consequently a tanked basement may be required, unless it can be proven that a drained basement is acceptable by satisfying WaterNSW.

Based on the results of the boreholes and our understanding of the proposed development (refer to Section 1), we have summarised the principal geotechnical findings, issues and recommendations to be considered in the planning, design, and construction of the development.

1. Prior to demolition or excavation, we recommend detailed dilapidation surveys be completed on any neighbouring structures that fall within the zone of influence, taken as twice the excavation depth from the common boundary. A reduction in the detail of the dilapidation reports may be considered given the large zone of influence due to the deep excavations.
2. Excavation for the proposed basement will be through pavement, fill, natural clays and then predominantly through sandstone bedrock of medium to high strength. Excavation of the sandstone will require the use of "hard rock" excavation equipment for effective excavation, which may transmit vibrations through the rock mass that could affect adjoining infrastructure.
3. The retention systems may comprise a soldier pile wall with shotcrete infill panels that may be terminated above BEL by socketing the piles a minimum 1m into at least medium strength sandstone. Depending on the results of further groundwater investigations, a contiguous or secant pile wall may be necessary to limit groundwater inflows into the basement excavation. Lateral support for the piles may comprise temporary ground anchors or internal propping.
4. The proposed basements will extend below the measured groundwater level. Given the relatively low permeability of the soils and bedrock, we expect seepage rates to be manageable using conventional sump and pump methods. However seepage rates are likely to be greater than allowed by the relevant authorities and therefore the basement will probably need to be designed as a tanked structure. The retention system should be reviewed following further analysis of expected groundwater seepage into the excavation and the proposed basement construction techniques.
5. Pad footings founded within the sandstone bedrock at the base of the excavation can be used to support the proposed development. Depending on the allowable bearing pressure adopted for the footings, suitable geotechnical inspections and testing of the footing excavations will have to be scheduled.
6. The site is bounded by the Sydney Trains Light Rail, the Sydney Water Hawthorne Canal and Old Canterbury Road. The relevant authorities will need to be satisfied that the proposed development does not adversely impact their assets, which will likely require computer modelling.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report.

4.2 Further Geotechnical Work/Input

The following summarises the further geotechnical input which may be required and which has been previously discussed in the preceding sections of this report:

- Drilling of at least five cored boreholes within the level area of the site extending below the proposed basement bulk excavation level.
- Drilling of one or two boreholes on the footpath/roadway of Old Canterbury Road to provide details for the retention system along this boundary.
- Groundwater monitoring and, if required, permeability testing and seepage analysis o assess the potential groundwater inflow volumes into the proposed basement.
- Advice, and likely computer modelling, to assess the impact of the proposed development on the adjacent (Sydney Trains) Light Rail corridor, (Sydney Water) stormwater canal and (RMS) Old Canterbury Road.
- Inspections during pile drilling for the retaining walls.
- Witness anchor tests.
- At least initial vibration monitoring during bulk excavation.
- Progressive inspection of excavated cut faces to confirm if additional support or treatment is required.
- Footing inspections and testing to confirm founding conditions.

Given no structural drawings have been issued and only preliminary architectural drawings are available, we recommend a review by a geotechnical engineer after initial structural design has been completed to confirm that our recommendations have been correctly interpreted. It is possible that further advice/input will be required during the structural design to address issues that may not have been addressed in this report. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

4.3 Excavation

All excavation recommendations should be complemented by reference to the latest edition of Safe Work Australia's 'Excavation Work Code of Practice'.

4.3.1 Dilapidation Surveys

Prior to the commencement of excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the east, including the Hawthorne Canal, Old Canterbury Road and the road bridge spanning the Sydney Light Rail, and any other buildings within the zone of influence. The zone of influence is taken as a distance from the common boundary of at least twice the excavation depth and as the excavation will typically be 9.3m, except for the southern boundary, this zone of influence extends about 20m.

The dilapidation surveys should include internal and external inspection of the buildings, where all defects including defect location, type, length and width are described and photographed. The respective owners of the buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.3.2 Excavation Methods

For the proposed basement excavation, we generally expect excavations approximately 9.3m deep, except at the southern end of the site, where excavation of the existing fill embankment results in excavations approximately 16.7m deep below Old Canterbury Road. Further geotechnical investigations, in the form of cored boreholes, will be required to confirm the excavation conditions, however the following is based upon the limited investigation previously carried out on the subject site.

The excavations will encounter pavements, fill, natural clays and predominantly sandstone bedrock up to high strength. Excavation of the soils and up to very low strength sandstone will be achievable using conventional excavation equipment, such as the buckets of large hydraulic excavators, possibly with some light ripping from a dozer or ripping hook fitted to the excavator. Excavation of the fill embankment is expected to be achieved using conventional earthmoving equipment, such as buckets of hydraulic excavators. However it should be noted that the composition of the fill embankment is not known and large inclusion within the fill should not be unexpected.

Excavation of sandstone bedrock of low strength or higher will represent 'hard rock' excavation conditions and will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping hooks and rock saws. The excavator contractor should be made aware of this by being supplied with all geotechnical information, particularly the borehole logs and point load strength test results. Low productivity and increased equipment wear should be expected due to the rock strength. We recommend that a copy of this report be provided to the excavation contractor so that they can make their own assessment of excavation conditions.

The excavated material and groundwater will need to be disposed off-site and therefore will need to be suitably classified for waste disposal purposes.

4.3.3 Vibration Monitoring

Subject to review of the dilapidation reports on adjacent structures, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec whenever hydraulic rock impact hammers are used. If required, to reduce the transmission of vibrations, a perimeter vertical saw cut slot may be provided through the sandstone bedrock and the base of the slot maintained at a lower level than the adjoining rock excavation at all times.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there will be direct transmission of ground vibrations to nearby structures and buried services. We recommend that initial quantitative vibration monitoring be carried out when using hydraulic rock hammers to determine if the transmitted vibrations are within an acceptable limit for the nearby structures and services. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations. Where the transmitted vibrations are excessive, it would be necessary to change to alternative excavation methods, such as smaller rock hammers, rotary grinders, ripping hooks or rock saws. Full time vibration monitoring may be required.

The following procedures are recommended to reduce vibrations if rock hammers are used and transmitted vibrations are found to be excessive:

- Maintain rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate the rock hammer in short bursts only, to reduce amplification of vibrations.

Alternatively, rock excavations using low vibration emitting equipment, such as rock saws and rock grinders fitted to a hydraulic excavator may be used. If rock saws or rock grinders are used, the resulting dust should be suppressed with water. Use of this low vibration emitting equipment would reduce the likelihood of vibration induced damage to the neighbouring structures and services. With the use of the low vibration equipment we do not consider that it will be necessary to carry out any quantitative vibration monitoring, although we recommend that at least an initial site visit at the commencement of rock excavation be carried out by a geotechnical engineer to inspect the excavation methods and procedures being adopted.

The use of excavation contractors with appropriate experience and with a competent supervisor who is aware of vibration damage risks, is also recommended. The contractor should have all appropriate statutory and public liability insurances.

4.4 Retention

Excavations within the soils and sandstone bedrock up to very low strength will not be self-supporting and retention systems will need to be installed prior to the start of excavation. The medium to high strength sandstone is generally self-supporting, provided it is free of adverse defects, and therefore consideration can be given to terminating the retention system once good quality sandstone bedrock is encountered and then vertically excavating the sandstone bedrock below unsupported. However, this method of construction will make groundwater inflows hard to control and will need to be considered further when the detailed investigation is completed.

The fill profile generally comprises of gravelly sand, with some sandy gravelly clay encountered in BH2. The natural soil profile has been assessed to be residual and predominantly comprising of silty clay, however thin bands of sandy clay were also logged. Groundwater was encountered between 1.7m and 2.1m depth generally within the natural profile. These more gravelly/sandy layers may be problematic due to high seepage inflows and/or collapsing soils if exposed within an excavation (such as if a soldier pile wall is utilised). Therefore we consider that the retention system should comprise a contiguous or secant pile wall socketed at least 1m into medium to high strength sandstone bedrock. If the retention system is proposed to be load bearing, then the piles will likely need to extend below BEL. If the piles are only very lightly loaded, then it may be feasible to terminate above BEL provided they are founded on a suitable founding stratum, such as high strength sandstone bedrock, but further assessment by a geotechnical engineer is required. It also assume that no adverse defects are present below the pile toe otherwise appropriate stabilisation measures will be necessary.

The good quality sandstone below the pile toes may then be excavated vertically unsupported, provided the sandstone cut faces are progressively inspected by a geotechnical engineer at no more than 1.5m depth increments to assess the presence of any potentially unstable rock wedges and the need for further stabilisation works, e.g. rock bolts, dowels, etc. A provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures.

Bored piles are likely feasible, however the presence of the granular fill and groundwater may result in collapse of the pile holes. Therefore, if bored piles are proposed, then we recommend some trial piles be drilled to assess whether the method is feasible. Alternatively, Continuous Flight Auger (CFA) piles may be adopted which will avoid these issues that bored piles encounter. We envisage that powerful rigs may be required in order to penetrate through the medium to high strength sandstone bedrock. CFA piles may not be able to penetrate to great depths in high strength sandstone.

Installation of the retention system along the southern boundary where the steep fill embankment is present will be challenging. The shoring piles along this boundary will be very deep, at least 17m to extend to BEL and then likely deeper to satisfy stability, and therefore only large piling rigs will be capable of drilling these piles. Consequently, a thick piling rig working platform will be necessary which will be difficult to construct given the steep slope present. We envisage that relatively extensive earthworks will be required to establish a suitable stable working platform for the piling rig comprising of a benched batter slope which should be assessed by a geotechnical engineer for slope stability. Otherwise, consideration could be given to drilling from outside the site boundary on the existing footpath but we assume obtaining permission is unlikely and furthermore there are overhead power lines which may restrict the piling rig.

4.5 Retaining Wall Design Parameters

Propped or anchored walls may be designed based on a trapezoidal earth pressure distribution of 6H kPa to 8H kPa, where H is the retained height of soils and weathered sandstone up to and including low strength. The 8H should be used adjacent to movement sensitive buildings and services, while the 6H may be able to be used where some movement of the shoring system can be tolerated. We imagine that 8H will need to be adopted for the majority of shoring walls given the proximity of the sensitive assets, such as Sydney Light Rail, Old Canterbury Road and the Hawthorne Canal.

Where the shoring system supports medium and high strength weathered sandstone below the toe of the contiguous piles, then the wall may be designed for a uniform pressure of 10kPa to support small local wedges of rock. Appropriate surcharge loads (such as adjoining buildings, traffic, sloping backfill, footing loads etc) are additional to the above earth pressures and should be allowed for in the design. The additional earth pressures from surcharge loads may be calculated using an 'at rest' earth pressure coefficient of 0.5.

Full hydrostatic pressures should be allowed unless measures are taken (and are permissible) to provide complete and permanent drainage behind the walls.

If piles extend below BEL, a passive toe resistance of the retention system may be estimated based on a preliminary allowable lateral resistance of 300kPa for sandstone of medium or higher strength. The passive resistance should be ignored to at least 0.5m below the base of the excavation, including footing, lift pit and service excavations due to the potential for fracturing of the upper sandstone during bulk excavation.

Anchors should have their bond length formed within sandstone of at least medium strength and may be provisionally designed based on a preliminary allowable bond stress of 300kPa. The anchor bond should be formed outside a line drawn up at 45° from the bulk excavation level, with a minimum free length of 4m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load. We have assumed that the final lateral support will be provided by the floor slabs for the proposed structure.

Temporary anchors will be required to provide lateral support to the proposed retention system. Permission will likely need to be obtained from the Local Council or RMS, whoever owns the road, and possibly other asset owners, if anchors are to be installed beneath the road or installed beyond the subject site boundaries. Sydney Trains do not normally allow anchors within the rail corridor. Consideration also needs to be given to the presence of existing basement levels to the east of the site and to ensure no anchors encounter these basements. If deep basements in these buildings are present, then internal propping may also be required along this boundary. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping.

We expect that the Sydney Light Rail will require specific shoring wall analysis, including an assessment of the likely ground movements beyond the shoring walls. This may also be necessary for Council/RMS for Old Canterbury Road and Sydney Water for Hawthorne Canal. The shoring wall design engineers should then be requested to provide comment on whether such movements will be problematic to any adjoining structures or services.

4.6 Pile Exposure Classification

Based upon the laboratory test values and in accordance with the AS2159–2009 *'Piling-Design and installation'*, a 'Mild' exposure classification should be adopted for concrete piles and a 'Non Aggressive' exposure classification for steel piles

4.7 Hydrogeological Considerations

Groundwater was encountered during drilling in a number of boreholes and upon completion at depths between 1.7m and 2.5m below existing surface level, or between about RL7.0m and RL8.1m. As such, we expect continuous groundwater flow into the basement excavation during construction and in the long term. Given the assumed relatively low permeability of the natural clays and sandstone bedrock, we expect that during construction the seepage would be able to be controlled using conventional sump and pump techniques. Increased groundwater seepage should be expected to occur through the sandy clay layer encountered in some of the boreholes.

We note that no long-term groundwater monitoring was carried out as part of the previous investigation and therefore we recommend that at least three groundwater monitoring wells are installed. Data loggers should then be installed to monitor groundwater levels for a minimum of 1 month. If groundwater levels are measured to be above BEL as expected, then we recommend that permeability testing be carried out to assess the permeability of the subsurface conditions. Once the permeability values are known, an estimate of the potential groundwater inflow volumes into the basement can be carried out by undertaking a seepage analysis using computer software. Where proposed basements extend below groundwater levels, WaterNSW specify a tanked basement is required, unless sufficient evidence can be provided to warrant a drained basement.

It is likely, based on the size of the basement excavations, and the level of groundwater encountered during these investigations that a dewatering licence will need to be obtained for temporary dewatering during construction. However, where a drained basement is not permitted, then the basement will need to be a tanked structure designed for hydrostatic uplift pressures. If a tanked basement is required, then further consideration must be given to the type and depth of any shoring system, including drainage behind the shoring walls. It is very difficult to form a cut-off where shoring piles are founded on the surface of the bedrock and if this construction method is used in the short-term, then an internal waterproof wall will be needed in the long-term.

4.8 Footings

Based on the results of the investigation, we expect that sandstone bedrock will be uniformly exposed across BEL. We therefore recommend the proposed development is uniformly founded on the underlying sandstone bedrock in the form of pad and strip footings. Assuming the medium to high strength sandstone bedrock encountered in the boreholes extends to BEL, we recommend that footings be designed based upon a preliminary Allowable Bearing Pressure (ABP) of 1,200kPa. This bearing pressure is based on serviceability criteria for deflections at the base of the footing of less than 1% of the minimum footing width.

Higher bearing pressures, in the range of 3,500kPa to 6,000kPa are likely feasible on the medium to high strength sandstone bedrock but further geotechnical investigations in the form of cored boreholes are required to assess the founding conditions below the proposed footing level. If higher bearing pressures are required, then we recommend, as a minimum, five cored boreholes within the site and one or two on Old

Canterbury Road, that extend at least 3m below BEL, i.e. to RL-2.5m, will be required to provide sufficient information.

Inspections of the footings by a geotechnical engineer will be required to confirm that suitable founding conditions have been achieved. The level of detail for these inspections will be dependent upon the ABP adopted for the footings. We envisage that all footings will need to be inspected by a geotechnical engineer, including spoon testing carried out on select footings to assess the presence of any adverse defects, such as extremely weathered seams, are present below the footings. All footings should be cleaned, inspected and poured as soon as possible. If water is allowed to pond at the base of the hole, any water softened material must be removed prior to pouring.

4.9 Basement Slab

Based on the investigation results, we expect sandstone bedrock to be uniformly exposed at BEL. Therefore, the basement slab should be underlain by a layer of durable igneous granular material such as DGB20 or other approved material to act as a separation layer between the rock and the basement slab.

Depending on the results of further groundwater investigation, the basement may need to be a tanked basement designed for hydrostatic uplift pressures. If a drained basement is permitted then drainage should be provided around the basement perimeter and below the lowest basement slab to direct seepage into sumps with permanent and fail safe automatic pumps to remove water from the basement. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage is sufficient for the actual seepage flows. A full drainage blanket may be necessary. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel.

4.10 Earthquake Design Factors

Based upon AS1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia", the following design parameters may be adopted:

- Hazard Factor (Z) = 0.08;
- Class C_e – Shallow soil site.

Due allowance for the effect of the fill embankment at the southern end of the site is required.

5 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client:	JK Geotechnics	**Ref No:**	26583SD
Project:	Proposed Industrial Building	**Report:**	A
Location:	120c Old Canterbury Road, Summer Hill, NSW	**Report Date:**	6/06/2013
		Page 1 of 1	

AS 1289	**TEST METHOD**	**2.1.1**	**3.1.2**	**3.4.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**LINEAR SHRINKAGE %**
1	1.90-2.30	5.0		
1	2.50-2.60	7.4		
1	3.00-3.60	5.7		
2	4.00-4.30	9.0		
3	0.50-0.95	19.0	52	15.0
3	3.60-4.00	5.1		
3	4.50-4.80	4.7		
4	2.40-2.60	5.3		
5	1.50-1.95	19.2	53	15.5
5	3.00-3.40	5.4		
5	3.80-4.30	5.5		

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 04/06/2013

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

CERTIFICATE OF ANALYSIS **91775**

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Adam Mitchell

Sample log in details:

Your Reference:	**265835D, Summer HIll**	
No. of samples:	3 soils	
Date samples received / completed instructions received	04/06/13	/ 04/06/13

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	12/06/13	/ 11/06/13
Date of Preliminary Report:	Not issued	

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Jacinta Hurst
Laboratory Manager

ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil				
Our Reference:	UNITS	91775-1	91775-2	91775-3
Your Reference	-------------	BH1	BH2	BH3
Depth	------------	0.5-0.95	0.5-0.95	1.5-1.95
Date Sampled		31/05/2013	31/05/2013	31/05/2013
Type of sample		Soil	Soil	Soil
Date prepared	-	06/06/2013	06/06/2013	06/06/2013
Date analysed	-	06/06/2013	06/06/2013	06/06/2013
pH 1:5 soil:water	pH Units	8.3	8.3	5.7
Chloride, Cl 1:5 soil:water	mg/kg	15	14	35
Sulphate, SO4 1:5 soil:water	mg/kg	120	120	54

Document Set ID: 33989839
Version: 1, Version Date: 25/09/2020

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			06/06/2013	[NT]	[NT]	LCS-1	06/06/2013
Date analysed	-			06/06/2013	[NT]	[NT]	LCS-1	06/06/2013
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	94%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	101%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.

Document Set ID: 33989839
Version: 1, Version Date: 25/09/2020

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: STONE MASON AND ARTIST PTY LTD
Project: PROPOSED INDUSTRIAL BUILDING
Location: 120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Job No. 26583SD
Date: 31-5-13
Method: SPIRAL AUGER JK300
R.L. Surface: ≅ 9.80m
Datum: AHD
Logged/Checked by: I.S./A.M.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Gravelly sand, fine to coarse grained, dark grey, fine to coarse subangular ash and sandstone gravel.	M	-	-	
	N = 5 3,2,3			FILL: Gravelly sand, fine to coarse grained, dark grey, fine to coarse aubangualr sandstone and igneous gravel, with clay bands				APPEARS POORLY COMPACTED
AFTER 4 HRS	N > 4 2,2,2/		CL	SANDY CLAY: low plasticity, grey brown mottled orange brown.	MC≈PL	St		RESIDUAL
AFTER 1 HR		2	-	SANDSTONE: medium grained, red brown and orange brown.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
								MODERATE RESISTANCE
		3				M-H		HIGH RESISTANCE
				END OF BOREHOLE AT 3.6m				'TC' BIT REFUSAL

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: STONE MASON AND ARTIST PTY LTD
Project: PROPOSED INDUSTRIAL BUILDING
Location: 120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Job No. 26583SD
Date: 31-5-13
Method: SPIRAL AUGER JK300
R.L. Surface: ≅ 9.75m
Datum: AHD
Logged/Checked by: I.S./A.M.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES		0			ASPHALTIC CONCRETE: 20mm.t				
						FILL: Gravelly sand, fine to coarse grained, dark grey and brown, fine to coarse grained igneous gravel	M	-	-	
	ES, DS	N = 7 5,4,3				FILL: Sandy gravelly clay, low plasticity, dark grey and brown, fine to medium grained subrounded sandstone gravel,with brick and ash fragments.	MC>PL			APPEARS POORLY COMPACTED
	DS	N = 1 1,1,0	1							
ON COMPLETION			2		CL	SANDY CLAY: low plasticity, grey brown mottled orange brown, with iron indurated gravel bands.	MC≈PL	-		RESIDUAL
	ES		3		-	SANDSTONE: medium grained, red brown, with iron indurated bands.	DW	M-H L		MODERATE TO HIGH RESISTANCE LOW RESISTANCE
	DS		4			SANDSTONE: medium grained, light grey and brown.		M-H		HIGH RESISTANCE
						END OF BOREHOLE AT 4.3m				'TC' BIT REFUSAL
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: STONE MASON AND ARTIST PTY LTD
Project: PROPOSED INDUSTRIAL BUILDING
Location: 120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Job No. 26583SD
Method: SPIRAL AUGER JK300
R.L. Surface: ≅ 9.75m
Date: 31-5-13
Datum: AHD
Logged/Checked by: I.S./A.M.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			ASPHALTIC CONCRETE: 20mm.t				
						FILL: Gravelly sand, fine to medium grained, dark grey, fine to medium grained subangular sandstone gravel and ash, with clay pockets	M	-	-	APPEARS POORLY COMPACTED
		N = 9 2,3,6			CH	SILTY CLAY: high plasticity, light grey, with fine to medium grained sand partings, trace of roots.	MC≈PL	VSt	210 200 210	RESIDUAL
			1			SILTY CLAY: high plasticity, red brown mottled grey.				
AFTER 2 HRS		N = 12 5,5,7				SILTY CLAY: high plasticity, grey mottled red brown and orange brown, with ironstone gravel bands.			300 310 310	
			2							
AFTER 1 HR										
		N > 13 3,3, 10/100mm REFUSAL	3		CL	SANDY CLAY: low plasticity, grey and light grey.			200 200 210	
					-	SANDSTONE: medium to coarse grained, light grey.	DW	L		LOW 'TC' BIT RESISTANCE
						SANDSTONE: medium grained, brown.		M-H		HIGH RESISTANCE
			4							
						END OF BOREHOLE AT 4.8m				'TC' BIT REFUSAL
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: STONE MASON AND ARTIST PTY LTD
Project: PROPOSED INDUSTRIAL BUILDING
Location: 120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Job No. 26583SD
Date: 31-5-13
Method: SPIRAL AUGER JK300
R.L. Surface: ≅ 9.80m
Datum: AHD
Logged/Checked by: I.S./A.M.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		ASPHALTIC CONCRETE: 20mm.t				
				FILL: Gravelly sand, fine to medium grained, brown, fine to medium sunangular igneous and sandstone gravel.	M	-	-	APPEARS POORLY COMPACTED
	N = 11 4,4,7		CH	SILTY CLAY: high plasticity, red brown mottled grey, trace of root fibres and ironstone gravel.	MC≈PL	VSt	300 350 380	RESIDUAL
ON COMPLETION	N = 20 8,9,11			as above, but grey mottled red brown.		VSt- H	450 350 410	
			-	SANDSTONE: fine to medium grained, red brown and light grey.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
								LOW TO MODERATE RESISTANCE WITH HIGH BANDS
						M-H		HIGH RESISTANCE
				END OF BOREHOLE AT 4.4m				'TC' BIT REFUSAL

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5

Client: STONE MASON AND ARTIST PTY LTD
Project: PROPOSED INDUSTRIAL BUILDING
Location: 120C OLD CANTERBURY ROAD, SUMMER HILL, NSW

Job No. 26583SD **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≅ 9.75m
Date: 31-5-13 **Datum:** AHD
Logged/Checked by: I.S./A.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			ASPHALTIC CONCRETE: 20mm.t	M	-	-	APPEARS POORLY COMPACTED
									FILL: Gravelly sand, fine to medium grained, dark brown, fine to coarse subangular igneous and concrete gravel and brick fragments, trace of clayey pockets.				
					N = 13 5,6,7			CH	SILTY CLAY: high plasticity, red brown mottled grey, with ironstone gravel bands.	MC<PL	St	150 180 180	RESIDUAL
					N = 15 7,7,8	1–2			SILTY CLAY: high plasticity, grey, trace of sand and root fibres.		H	410 420 420	
						3		-	SANDSTONE: medium to coarse grained, red brown and light grey.	DW	M-H		HIGH 'TC' BIT RESISTANCE
						4			SANDSTONE: medium to coarse grained, orange brown and brown.				
						4.4			END OF BOREHOLE AT 4.4m				'TC' BIT REFUSAL
						5							
						6							
						7							

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

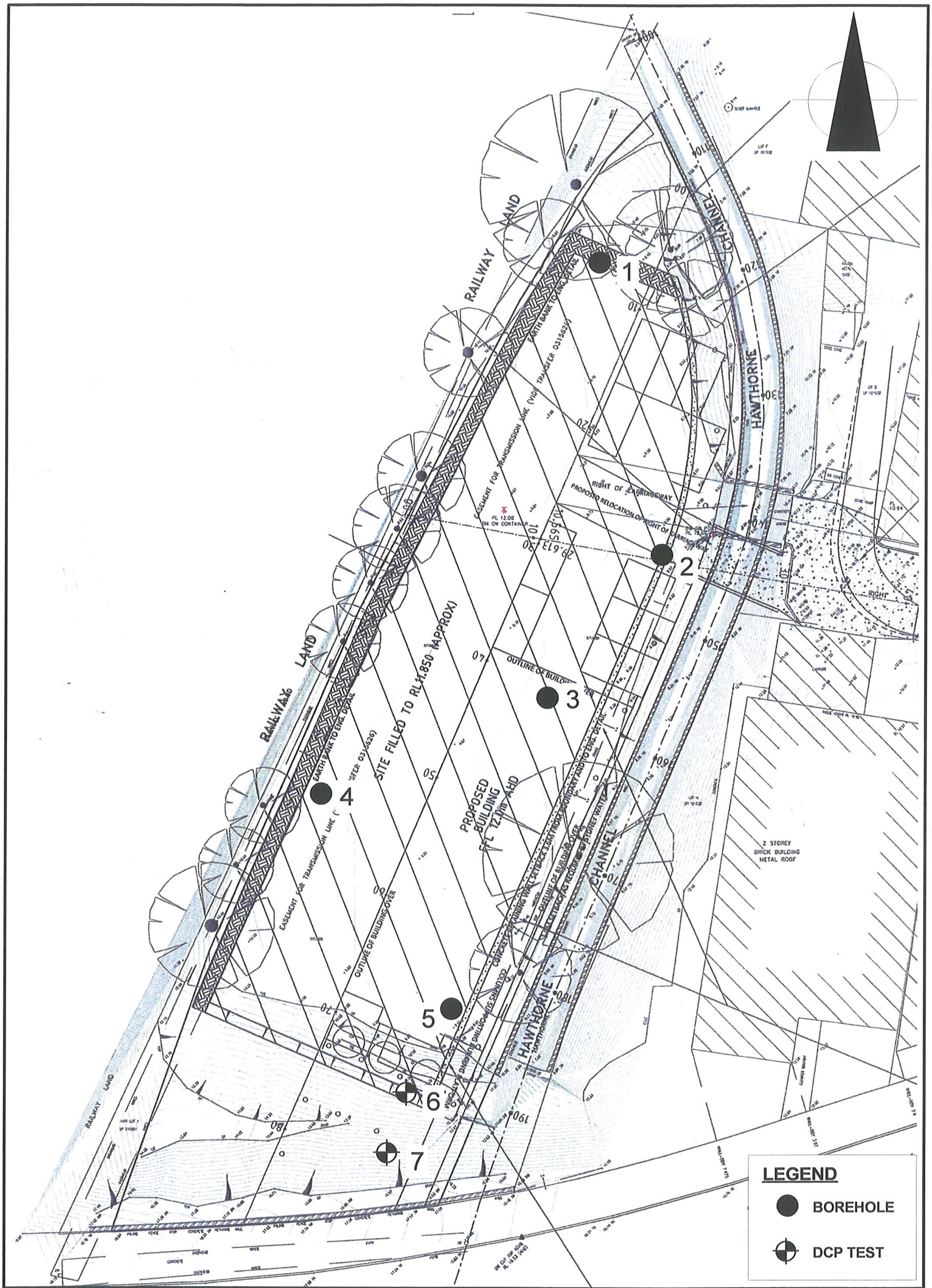
Client:	STONE MASON & ARTIST PTY LTD		
Project:	PROPOSED INDUSTRIAL BUILDING		
Location:	120C OLD CANTERBURY ROAD, SUMMER HILL, NSW		
Job No.	26583SD	Hammer Weight & Drop: 9kg/510mm	
Date:	31-5-13	Rod Diameter: 16mm	
Tested By:	I.S.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration

Test Location				Test Location			
Depth (mm)	**6**	**7**		Depth (mm)	**6**		
0 - 100	↓	SUNK		3000-3100	16		
100 - 200	1	↓		3100-3200	17		
200 - 300	↓	1		3200-3300	30/90mm		
300 - 400		4		3300-3400	REFUSAL		
400 - 500	1	3		3400-3500			
500 - 600	2	5		3500-3600			
600 - 700	2	12		3600-3700			
700 - 800	4	10		3700-3800			
800 - 900	3	11		3800-3900			
900 - 1000	4	37		3900-4000			
1000 - 1100	4	2/0mm		4000-4100			
1100 - 1200	4	REFUSAL		4100-4200			
1200 - 1300	5			4200-4300			
1300 - 1400	7			4300-4400			
1400 - 1500	6			4400-4500			
1500 - 1600	5			4500-4600			
1600 - 1700	7			4600-4700			
1700 - 1800	5			4700-4800			
1800 - 1900	4			4800-4900			
1900 - 2000	6			4900-5000			
2000 - 2100	4			5000-5100			
2100 - 2200	5			5100-5200			
2200 - 2300	10			5200-5300			
2300 - 2400	20			5300-5400			
2400 - 2500	34			5400-5500			
2500 - 2600	15			5500-5600			
2600 - 2700	11			5600-5700			
2700 - 2800	10			5700-5800			
2800 - 2900	11			5800-5900			
2900 - 3000	11			5900-6000			

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal

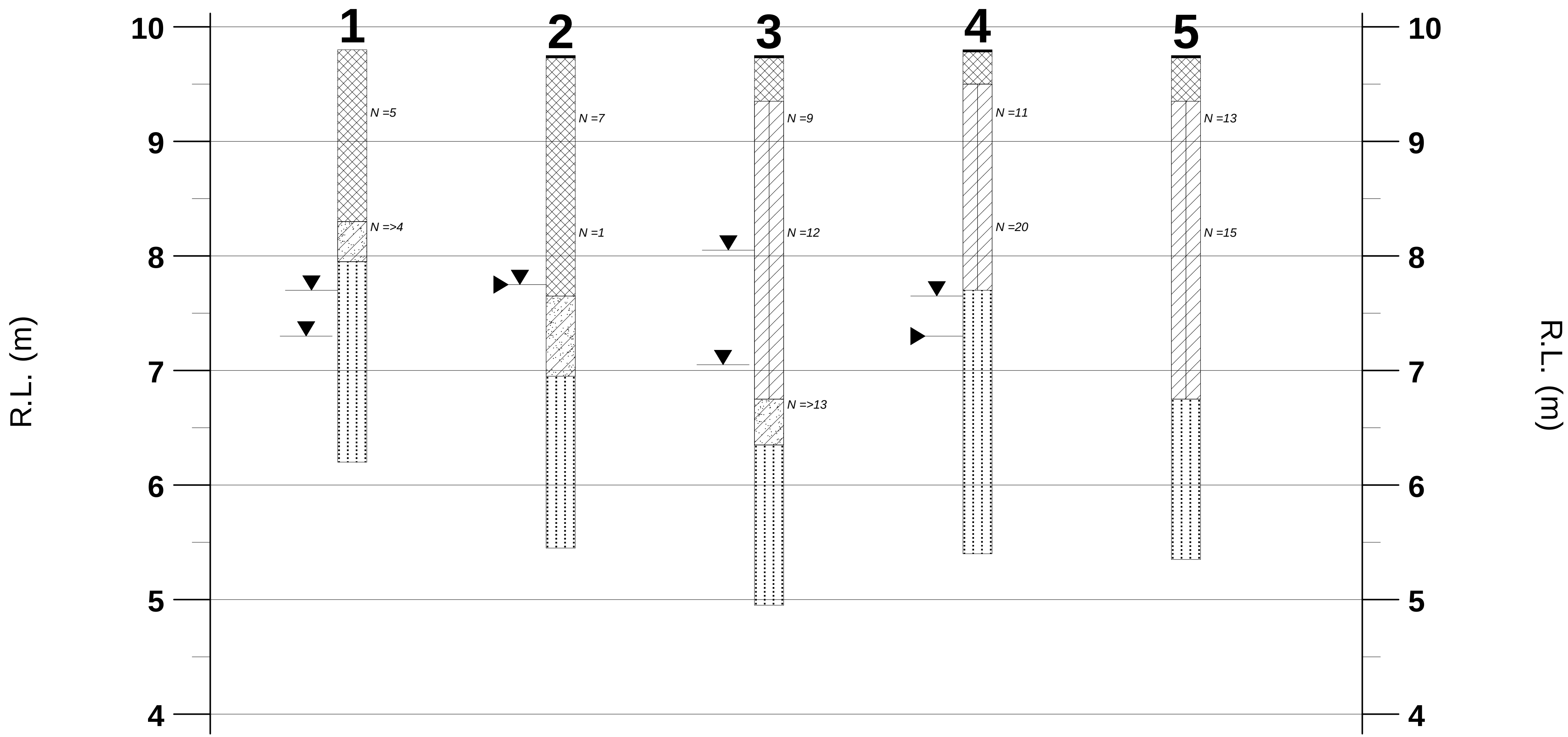
Ref: JK Geotechnics DCP 0-6m July 2012



JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Title:	Report Number:	Figure Number:
INVESTIGATION LOCATION PLAN	26583SD	1

GRAPHICAL BOREHOLE SUMMARY



Fill
Sandy Clay
Sandstone/ Greywacke
Asphaltic/ Bituminous Paving or Coal
Silty Clay
Observed water level
Groundwater seepage level
N SPT "N" VALUE
Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 50 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 26583SD **Figure No.:** 2

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

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

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

NOTES:

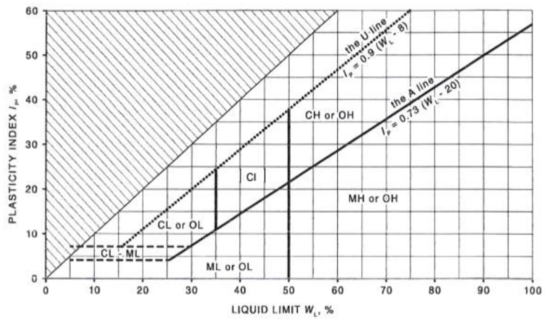
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



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LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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