REPORT TO

FRESH HOPE CARE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED INDEPENDENT LIVING UNITS

AT

154 TO 162 STAFFORD STREET, PENRITH, NSW

Date: 8 April 2020
Ref: 32041BXrtp Rev1

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Table of Contents

ATTACHMENTS

STS TABLE A: MOISTURE CONTENT, ATTERBERG LIMITS & LINEAR SHRINKAGE TEST REPORT
STS TABLE B: FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT
ENVIROLAB SERVICES CERTIFICATE OF ANALYSIS 207808
BOREHOLE LOGS 1 TO 5 INCLUSIVE
FIGURE 1: SITE LOCATION PLAN
FIGURE 2: BOREHOLE LOCATION PLAN
VIBRATION EMISSION DESIGN GOALS
REPORT EXPLANATION NOTES

1 INTRODUCTION

This revised report presents the results of a geotechnical investigation for the proposed independent living units at 154 to 162 Stafford Street, Penrith, NSW. The investigation was carried out in December 2018 and was commissioned by New Hope Care, in consultation with Mr David Arguelles of Linear Project Management, and was carried out in accordance with our proposal dated 15 August 2018, Ref P47847B. The results of our investigation were originally presented in our report dated 8 January 2019 (Ref: 32041BXrpt), but changes to the architectural drawings have been made and this report has been prepared based on the revised architectural drawings.

In order to prepare this revised report, we were provided with the following:

- Survey Plan by Project Surveyors, Drawing No B04360-1, dated 5 September 2018.
- Architectural Drawings prepared by Smith & Tzannes Pty Ltd (Project No. 19_086, Drawing Nos DD-A-100, DD-A-101, DD-A-102, DD-A-103 and DD-A-202, all revision A, dated 31 March 2020).

From this information, we understand that the proposed development will comprise:

- Three separate, two or three storey buildings (Building A, B and C) containing residential independent living units.
    - Building B and sections of Building C are to be constructed over a common single ground floor (Level 0) with Finished Floor Levels (FFL) proposed at RL46.18m and RL47.16m respectively. Excavation for this lowest level (Building B and C) will be required to depths ranging from about 2.5m to 3.5m. The excavation will generally be set back between 4.4m and 6m from the site boundaries, however, the proposed excavation will extend to the site boundary with the neighbouring property to the west (No. 68-70 Doonmore Street).
    - The FFL for the ground floor of Building A and the eastern portion of Building C is proposed at a higher elevation of RL47.4m. This will require Excavation below Building C to a maximum depth of about 0.6m and fill below Building A to a maximum of about 1m. these building will have minimum offsets of 1.5m from the site boundaries.
- An On Site Detention (OSD) tank adjacent to the central portion of the northern site boundary. This OSD tank will have an invert level at RL46.475m and will require excavation to a maximum depth of about 1.2m.
- An access driveway and ramp to the Building B and C ground floor level carpark within the central portion of the northern boundary of the site.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention, footings, engineered fill, on-grade floor slabs and pavements.

This geotechnical investigation was carried out in conjunction with a Preliminary Environmental Site Assessment by our specialist division, Environmental Investigation Services (EIS). The results of the preliminary environmental site assessment are provided in a separate report by EIS, Ref: E32041KTrpt.

2 INVESTIGATION PROCEDURE

Boreholes BH1 to BH5 were drilled to depths of 6.0m below the existing ground surface, using spiral augering techniques with our track mounted JK205 drill rig. The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The borehole locations were dictated by access limitations due to existing buildings on site. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between contours and spot heights shown on the supplied survey plan by Project Surveyors. The survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsurface soil and rock strata were assessed by logging the materials recovered during drilling. The strength of the soil profile was assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. The strength of the bedrock was assessed by observation of the drilling resistance of a tungsten carbide (TC) drill bit attached to the augers, tactile examination of recovered rock chips, and correlation with the results of subsequent laboratory moisture content testing.

Groundwater observations were made during, on completion and a short time after completion of drilling. No longer term groundwater monitoring was completed.

Our geotechnical engineer, Mr Kartik Singh, was on site full time during the fieldwork and set out the borehole locations, nominated the sampling and testing locations and prepared the borehole logs. The borehole logs are attached to this report along with our Report Explanation Notes which define the logging terms and symbols used and describe the investigation techniques adopted.

Selected soil and rock chip samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, Atterberg Limits, linear shrinkages, standard compaction properties, four day soaked CBR values, pH, sulphate content, chloride content and resistivity. The results of the testing are presented in the attached STS Tables A and B and Envirolab Certificate of Analysis 207808.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated in gently undulating terrain and on the side of a hill that slopes down to the north-east at approximately 3°. Stafford Street and Doonmore Streets form the northern and western site boundaries, respectively.

At the time of the investigation the site was occupied by a number of single storey brick and weatherboard buildings including the centrally located Church of Christ. The buildings appeared to be in fair to good external condition. An on grade asphaltic concrete carpark was located between the Church and No.154 Stafford Street, and the car park surface appeared to be in fair condition with evidence of patchwork repairs to previous cracking and potholes. The remainder of the site was grassed, together with scattered small to medium sized trees.

The site is bound to the south and east by one and two storey brick residential buildings set back between 1.5m and 5m from the common boundaries. Ground surface levels across the site boundaries were generally similar with those within the subject site, apart from the neighbouring properties beyond the south-western corner of the site that were a maximum of 0.5m lower than the subject site. A concrete block retaining wall was located on the boundary and appear to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Penrith indicates that the site is underlain by Bringelly Shale of the Wianamatta Group.

The boreholes disclosed a generalised subsurface profile of shallow fill overlying residual silty clay that graded into weathered claystone at relatively shallow depths. No groundwater was encountered during the investigation. For detailed subsurface conditions, reference should be made to the attached borehole logs. A summary of the encountered subsurface conditions is presented below:

Pavement

In BH2 and BH3, Asphaltic concrete (AC) of 30mm thick was initially penetrated, underlain by a 270mm thick layer of gravelly sand fill. This fill layer is likely to comprise the base layer as part of the pavement.

Fill

Fill comprising silty clay and silty sandy clay of medium to high plasticity was encountered from the surface in BH1, BH4 and BH5 and immediately beneath the pavement in BH3 and extended to depths ranging from 0.2m to 0.6m. In BH2, the only fill encountered was the pavement materials to a depth of 0.3m as described above.

Residual Soils

Residual silty clay was encountered below the fill in each borehole and was assessed to be of medium to high plasticity and generally of very stiff to hard strength. However, some stiff strength clays were encountered in BH4 and BH5.

Weathered Claystone

Weathered claystone was encountered in each borehole at depths ranging from 1.2m to 2.0m. The claystone was generally extremely weathered and of hard strength on first contact and improved to be highly weathered and of very low, low and medium strength with depth.

Groundwater

No groundwater was encountered within the boreholes during, on completion, or up to two hours following completion of BH1.

3.3 Laboratory Test Results

The results of the Atterberg limits testing on the recovered silty clay samples from BH3 and BH4 confirmed the samples to be of medium and high plasticity, respectively. The linear shrinkage test results on the same samples indicated the silty clays to have a moderate to high potential for shrink/swell movements with changes in moisture content.

The results of the moisture content tests carried out on selected rock chip samples correlated reasonably well with our field assessment of bedrock strength.

The four day soaked CBR test carried out on a silty clay sample from BH1 compacted to 98% of Standard Maximum Dry Density (SMDD) resulted in a CBR value of 1.0%.

The soil pH values ranged from 5.0 to 5.3, indicating acidic soil conditions. The sulphate contents ranged from 250mg/kg to 320mg/kg, the chloride contents ranged from 270mg/kg to 430mg/kg and the resistivity ranged from 2,800ohm.cm to 3,600ohm.cm. Based on these results the silty clay soils (BH2 and BH5) and extremely weathered claystone (BH1) would be classified as having a 'mild' exposure classification for concrete piles in accordance with Tables 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'. For steel piles, the clays and weathered claystone would be classified as 'non-aggressive' in accordance with Table 6.5.2(C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Prior to the start of demolition and excavation dilapidation surveys should be carried out on the adjoining properties to the south and east located within a horizontal distance from the excavation perimeter of at least twice the excavation depth. The dilapidation surveys should comprise detailed inspections of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

The proposed bulk excavation to a maximum depth of about 3.5m will encounter fill, residual silty clay and weathered claystone bedrock. The strength of the rock within the excavation depth will generally be of very low strength, but may contain low to medium strength bands locally.

The soil cover should be readily excavated using conventional earthworks equipment, such as the buckets of hydraulic excavators. Some of the underlying extremely weathered claystone and claystone of very low strength may also be excavated by the bucket of a large excavator, possibly with some ripping. Excavation of the claystone of low and medium strength will require the use of rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes and for detailed rock excavations, such as for footings or buried services.

Hydraulic rock hammers must be used with care as there will likely be direct transmission of ground vibrations to adjoining buildings, structures and infrastructure. If hydraulic rock hammers are used, excavation should commence within the central portion of the site (i.e. away from adjoining structures) using a moderately sized excavator fitted with a relatively low energy hydraulic hammer, no larger than a Krupp 900 size or equivalent. At least during the initial excavation using a rock hammer, the vibrations transmitted to the adjoining structures should be quantitatively monitored to assess how close the hammer can operate to the adjoining structures while maintaining the transmitted vibrations within acceptable limits. If the transmitted vibrations are of concern then we recommend that full time monitors be attached to the adjoining structures, with the monitors attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached so that excavation works can cease. 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 equipment, such as a smaller rock hammer, ripping hooks, rotary grinders, or rock saws. Alternative excavation techniques which can reduce transmitted vibrations include the provision of vertical saw cut slots along the perimeter of the excavation before breaking out the rock from between the saw cuts using a rock hammer, and maintaining the base of the slots at a lower level than the adjoining rock excavation at all times. When using the rock saw, the resulting dust must be suppressed by spraying with water.

The following procedures are also recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate hammer in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil on the hammer.

We also recommend use of excavation contractors with experience in such work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

4.2 Groundwater Seepage

Groundwater was not encountered within the boreholes during or on completion of drilling. However, localised inflows may occur after periods of heavy rain close to, or at, the contact between the soils and the underlying bedrock. In addition, concentrated inflows may be encountered where defects within the bedrock daylight into the excavation and where highly fractured sections of bedrock are exposed in the excavation face.

In general, we expect that inflows, if any, to be of small volume and will be able to be managed by conventional sump and pump techniques. Inspection and monitoring of groundwater seepage during excavations is recommended so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design. In the long term, drainage should be provided behind all retaining walls and possibly below the lowest floor level slab. The completed excavation should be inspected by the hydraulic consultant to assess if the designed drainage system is adequate for the actual seepage flows.

4.3 Earthworks and Filling

Some minor cut and fill earthworks will be required outside of the proposed excavation, mainly within the north-eastern portion of the site. We are unaware of any records of placement or compaction control of the existing fill and as such it must be considered uncontrolled and is not suitable to support floor slabs. Therefore, where the floor slabs are to be supported on the soils, all existing fill should be excavated and replaced by controlled, engineered fill where the slab subgrade is at or above the existing ground surface. Following demolition of the existing buildings there will areas that will require backfilling following removal of existing footings and these areas should also be filled with engineered fill.

The following subgrade preparation measures should be followed where the filled subgrade is to support floor slabs or pavements. If fully suspended floor slabs are adopted supported on the footings founded within the claystone, then no particular subgrade preparation would be required other than stripping of root affected soils.

- Strip all topsoil and root affected soils and any obvious deleterious materials that may be present. Soil deemed to be unsuitable due to contamination (if any) should be treated or disposed of in the appropriate manner. Any topsoil and root affected soils would not be suitable for reuse as engineered fill, but may be used within landscaped areas. Therefore, this material should be stockpiled separately or disposed offsite.
- Excavate any remaining existing fill to expose the residual silty clay.
- Proof roll the exposed subgrade with at least 7 passes of a minimum 8 tonne deadweight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and within the presence of a geotechnical engineer to detect any weak subgrade areas.
- Where access is restricted, a smaller roller or even a hand held vibrating plate compactor (wacker packer) may need to be used. This will limit any subgrade improvement that may be achieved during proof rolling and will require fill to be placed in thinner layers.
- Care will need to be exercised close to the neighbouring structures as ground borne vibrations caused by the proof rolling may cause damage. If there are any causes for concern during proof rolling, then further advice should be sought and/or the non-vibration (static) mode of the roller used.
- Treat any unstable areas detected during proof rolling by excavation to a sound base and replacement of the excavated material with engineered fill or as recommended by the geotechnical engineer during the proof rolling inspection.
- Place engineered fill as required in thin layers as recommended in Section 4.3.1 below.

The existing subgrade will mainly comprise natural silty clays. The clays may be found to be unstable if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening may result and site trafficability will be poor. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill.

4.3.1 Engineered Fill and Compaction Control

Any fill areas required to raise site levels and/or backfill poor areas of subgrade revealed during proof rolling should comprise engineered fill. Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of organic materials, other contaminants and deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated residual silty clay and weathered claystone may be reused as engineered fill, provided it is free of deleterious materials and particles greater than 75mm in size. Any clay fill should be compacted in maximum 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and to a moisture content within 2% of Standard Optimum Moisture Content (SOMC). We note that careful control of compaction and moisture content is required when using clay soils for engineered fill and may result in an increased overall cost than if granular materials were used.

To confirm the above specifications have been achieved, density tests should be carried out at a frequency of one test per layer per 500m^2 or three tests per visit, whichever requires the most tests, for bulk engineered fill and one test per 2 layers per 50m^2 for backfill of limited areas. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.4 Retention

Although it appears that temporary batters may be able to be accommodated within the site for most of the proposed excavation, they would not be appropriate for the deepest excavations of about 3.5m or adjacent to the boundary of neighbouring property at 68 – 70 Doonmore Street. In addition, the adoption of temporary batters would require the removal of material from site and then importation of backfill, which may not be cost effective when compared to shoring systems. Therefore, it is likely that the most practical option may be to install shoring systems at least around the southern portion, if not all, of the proposed excavation below Building B and C, prior to excavation. Where excavations are shallow, such as in the north-eastern corner, temporary batters may be economical.

Where space permits, temporary batters within the soils and weathered claystone of no more than 3m in height should be no steeper than 1 Vertical in 1 Horizontal (1V:1H). Such batters should remain stable in the short term, provided all surcharge loads, including construction loads, are kept well clear of the crest of the batter.

Permanent batters, if required, should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent batters to also reduce erosion.

Retention systems may comprise soldier pile walls with shotcrete infill panels. Such walls may be designed as cantilevered walls where the retained height is less than 3m, but additional support by external anchors or internal props may be required to support the deeper excavations. Shoring piles should be embedded to sufficient depth below bulk excavation level to satisfy stability considerations, but at least 1m below the bulk excavation level. Allowance will also need to be made for localised excavation below bulk excavation level such as for footings, lift pits etc. Piles extending below bulk excavation level may also be incorporated into the footing system.

Where anchors extend below adjacent properties permission will need to be obtained from the adjacent property owners before installation of the anchors below their properties. Such permission can take some time to obtain, which should be allowed for with the project program.

Particular care is required with sequencing and quality of construction of soldier pile walls. The shotcrete infill panels must be completed without delay to reduce the shrinkage of clay soils immediately outside the excavation and to limit potential rock wedge failures within the bedrock.

- Free standing cantilever walls retaining a maximum height of 3m may be designed based on a triangular lateral earth pressure distribution using an active earth pressure coefficient, K_a of 0.33 and a bulk unit weight of 20kN/m^3. This assumes that some resulting ground movement behind the walls is acceptable. Where movement of free standing walls is to be reduced an ‘at rest’ earth pressure coefficient, K_0, of 0.6 should be used.
- Anchored or propped walls where some movements can be tolerated, such as where adjacent structures or movement sensitive services are located beyond a horizontal distance of at least twice the wall height, may be designed using a trapezoidal earth pressure distribution of magnitude 6H kPa, where H is the retained height in metres. Where anchored or propped walls support areas which are sensitive to lateral movement, such as where adjacent structures or movement sensitive services are located within a horizontal distance of twice the wall height, they should be designed using a trapezoidal earth pressure distribution of 8H kPa. These pressures should be assumed to be uniform for the central 50% of the earth pressure distribution. In addition to these pressures, the retention wall design should be checked and designed to accommodate a wedge formed by a joint inclined at 45° intersecting the excavation face just above the bulk excavation level.

- The above coefficients and earth pressures assume horizontal backfill surfaces and any inclined backfill must be taken as a surcharge load. All surcharge loads affecting the walls (ie. traffic loading, construction loads, nearby high level footings etc) should be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the wall.
- Lateral toe restraint for piles socketed into claystone bedrock of at least very low strength below bulk excavation level may be designed based an allowable lateral resistance of 150kPa. The upper 0.3m depth of the socket below bulk excavation level, including any localised excavations such as for lift over run pits, footings, buried services, etc. should be ignored to allow for disturbance and tolerance effects during excavations.
- Anchors should be bonded at least 3m into very low strength (or better) claystone bedrock where an allowable bond stress of 150kPa may be adopted for design. The anchor bond length should commence beyond a line projected up from the base of the excavation at an angle of 45^o, with a minimum free length of 3m. All anchors should be proof-tested to at least 1.3 times their working load, under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor, before locking off at about 80% of their 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.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The excavated clay and claystone will be difficult to properly compact within the limited space available behind the walls and consideration should be given to the use of more readily compactable materials, such as ripped or crushed rock. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas a lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. If clay and claystone fill is to be used a greater control of fill compaction and moisture control will be required and further geotechnical advice on the use of such material should be obtained. An alternative for backfill would also be to use a uniform granular material, such as crushed concrete of 30mm to 70mm in size, surrounded in a geofabric.

4.5 Footings

Based on the results of the investigation, we expect that the proposed excavation will generally expose variable (generally extremely weathered or very low strength) claystone bedrock. For uniformity of support and to control potential differential movements, we recommend that all structures be supported on footings founded in the weathered claystone bedrock.

Pad or strip footings may be used where claystone is exposed or is at shallow depths, but where the depth to claystone is more than about 1m bored piers would be more practical. Where sections of the ground floor level are in proximity to and at a higher level than the Building B and C excavation (i.e. the eastern Lobby/Community sections of Building B), piles will be required to achieve uniform support within the claystone. Such piles should be founded below the zone of influence of the retaining walls so that additional surcharge loads are not placed on the walls. This zone of influence may be taken as a line drawn up at 45° from the base of the excavation.

Footings founded within extremely weathered claystone may be designed based on an allowable bearing pressure of 700kPa. Where footings are founded within claystone of very low strength or higher strength design may be based on allowable end bearing pressures of 1000kPa.

All piles should be drilled to achieve a nominal socket of at least 0.3m into the appropriate quality claystone. Allowable shaft adhesions equivalent to 10% of the above allowable bearing pressures for compression and 5% for tension may be applied to rock sockets in excess of the nominal 0.3m embedment and provided socket roughness and cleanliness is maintained.

We recommend that the bored pile drilling and pad footing excavations be inspected by a geotechnical engineer prior to pouring to confirm that an appropriate founding material has been exposed. All footings should be excavated, cleaned, inspected and poured with minimal delay to avoid deterioration. Water should be prevented from ponding in the base of footings as this will tend to soften the foundation material, resulting in further excavation and cleaning being required.

Based on the subsurface conditions encountered, the site would be classified for earthquake design as Class C_e in accordance with Section 4 of AS1170.4-2007.

Site classification in accordance with AS2870-2011 is not relevant for the proposed development as claystone will be encountered within the proposed excavations and all footings should be founded within the claystone. Nevertheless, due to the clay fill encountered to a maximum depth of 0.6m the site would be classified as Class P in accordance with AS2870-2011. However, if all footings were founded below the fill and within the residual silty clays we would expect shrink/swell movements similar to a Class H1 site in accordance with AS2870-2011. If any minor structures separate to the main buildings are proposed that will be founded within engineered fill or residual silty clay we recommend that specific advice be obtained for those structures on appropriate allowance for shrink/swell movements and bearing pressures. As a guide, an allowable bearing pressure of 100kPa would be appropriate for engineered fill or clays of stiff strength or 200kPa for clays of at least very stiff strength.

4.6 On-Grade Floor Slabs

We expect that the Building B and C lowest floor slab will be cast on weathered claystone and the higher level sections of the ground floor slabs for Buildings A and C will be cast on engineered fill or residual silty clay. The soil subgrade for the slabs should be prepared as recommended in Section 4.3 above. However, where claystone is exposed within the excavation proof rolling is unlikely to be required, but the subgrade should be inspected by a geotechnical engineer.

The Building B and C lowest floor slab should be underlain by a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2014) unbound base martial (or similar good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. This will help to provide a separation layer between the slab and the claystone subgrade and provide a uniform base for construction of the slabs. As recommended in Section 4.2 drainage may be required below the lowest slab and may comprise adoption of a high strength, durable, single sized washed aggregate, such as 'blue metal' gravel, as the subbase layer or a grid of subsoil drains. The under-floor drainage may connect with the wall drainage (where appropriate) and lead to the stormwater system for controlled disposal.

The proposed on grade floor slabs should be separated from all walls, footings etc (i.e. designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys. In addition, close to the interface between soil and bedrock subgrade conditions we recommend that additional dowels be provided.

4.7 External Pavements and Building B and C Entry Driveway

The only external pavement we are aware of is the entry driveway to the proposed Building B and C ground floor, which is likely to only carry cars and as such a nominal pavement thickness is likely to be sufficient. Where pavements are proposed the subgrade should be prepared as outlined in Section 4.3.

The soaked CBR test on a sample of the residual silty clay from BH1 gave a very low CBR value of 1.0%. The pavement thickness may be designed based on this CBR of 1.0%, or an estimated modulus of subgrade reaction of 12kPa/mm (750mm plate), but we recommend that some form of subgrade improvement be carried out in order to reduce the pavement thickness. Given the limited area of the pavement the most practical option would be to provide a select layer of good quality granular material with a CBR of at least 10%. The thickness of the select layer should be determined as part of the overall pavement thickness design, but we recommend a minimum thickness of 0.3m.

Alternatively, lime stabilisation could be considered, but it is unlikely to be practical for such a small area. Care would also be required to supress any lime during placement to prevent it becoming airborne and affecting adjoining properties. If lime stabilisation was adopted it should be carried out to a depth of 0.2m to 0.3m and the amount of lime required would need to be determined by laboratory testing.

Adequate drainage should be provided to prevent moisture ingress into the pavement and subgrade.

Concrete pavements should have a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2014) unbound base material (or similar good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

5 SALINITY

The site is located in an area where soil and groundwater salinity may occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. The local council has guidelines relating to salinity issues which should be checked for relevance to this project.

6 GENERAL COMMENTS

The 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

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.

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

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Independent Living Units
Location: 154-162 Stafford Street, Penrith, NSW

Ref No: 32041B
Report: A
Report Date: 14/12/2018
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	3.10 - 4.00	7.6	-	-	-	-
1	5.50 - 6.00	6.0	-	-	-	-
2	5.60 - 6.00	6.1	-	-	-	-
3	0.60 - 1.00	-	40	17	23	11.5
3	4.10 - 4.50	8.9	-	-	-	-
3	4.50 - 5.00	7.6	-	-	-	-
3	5.50 - 6.00	5.2	-	-	-	-
4	0.50 - 0.95	-	55	20	35	15.0 *
4	2.80 - 3.00	11.9	-	-	-	-
4	4.50 - 5.50	8.9	-	-	-	-
5	3.00 - 4.00	9.5	-	-	-	-
5	4.50 - 6.00	10.6	-	-	-	-

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: 06/12/2018.
- Sampled and supplied by client.
- * = Linear Shrinkage curled.

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Authorised Signature / Date
(D. Treweek)

14/12/18

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

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

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32041B
Project:	Proposed Independent Living Units	**Report:**	B
Location:	154-162 Stafford Street, Penrith, NSW	**Report Date:**	17/12/2018
		Page 1 of 1	

BOREHOLE NUMBER	BH 1
DEPTH (m)	0.50 - 1.00
Surcharge (kg)	4.5
Maximum Dry Density (t/m^3)	1.79 STD
Optimum Moisture Content (%)	17.0
Moulded Dry Density (t/m^3)	1.77
Sample Density Ratio (%)	99
Sample Moisture Ratio (%)	95
Moisture Contents	
Insitu (%)	18.0
Moulded (%)	16.2
After soaking and	
After Test, Top 30mm(%)	28.1
Remaining Depth (%)	17.0
Material Retained on 19mm Sieve (%)	0
Swell (%)	3.5
C.B.R. value: @2.5mm penetration	1.0

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 06/12/2018.
- Sampled and supplied by client.

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NATA Accredited Laboratory
Number:1327

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Authorised Signature / Date
(D. Treweek)

17/12/18

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

CERTIFICATE OF ANALYSIS 207808

Client Details

Client	JK Geotechnics
Attention	A Frost
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32041B, Penrith**
Number of Samples	3 Soil
Date samples received	11/12/2018
Date completed instructions received	11/12/2018

Analysis Details

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

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

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

Report Details

Date results requested by	18/12/2018
Date of Issue	13/12/2018

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil				
Our Reference		207808-1	207808-2	207808-3
Your Reference	UNITS	BH1	BH2	BH5
Depth		1.5-1.95	0.5-0.95	0.5-0.95
Date Sampled		06/12/2018	06/12/2018	06/12/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	12/12/2018	12/12/2018	12/12/2018
Date analysed	-	12/12/2018	12/12/2018	12/12/2018
pH 1:5 soil:water	pH Units	5.1	5.0	5.3
Chloride, Cl 1:5 soil:water	mg/kg	430	280	270
Sulphate, SO4 1:5 soil:water	mg/kg	250	310	320
Resistivity in soil*	ohm m	28	32	36

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			12/12/2018	[NT]	[NT]	[NT]	[NT]	12/12/2018	[NT]
Date analysed	-			12/12/2018	[NT]	[NT]	[NT]	[NT]	12/12/2018	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	110	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	113	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: FRESH HOPE CARE
Project: PROPOSED INDEPENDENT LIVING UNITS
Location: 154 TO 162 STAFFORD STREET, PENRITH, NSW

Job No. 32041B
Date: 6/12/18
Method: SPIRAL AUGER JK205
R.L. Surface: ≈ 46.3m
Datum: AHD
Logged/Checked by: K.K.S./ A.F.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL-ETION AND AFTER 2HRS	■					0	▦		FILL: Silty sandy clay, medium to high plasticity, brown, trace of fine to medium grained sub angular igneous gravel.	w≈PL			GRASS COVER
	■		■	■	N = 9 3,4,5	1	▨	CH	Silty CLAY: high plasticity, grey, trace of fine to coarse grained ironstone gravel.	w>PL		280 330 300	RESIDUAL
				■	N = 28 8,11,16	2 3	☰	-	Extremely Weathered Claystone: CLAY, high plasticity, grey with fine to coarse grained ironstone gravel.	XW	Hd		BRINGELLY SHALE
				■		4	☰		CLAYSTONE: grey brown.	HW	VL		VERY LOW 'TC' BIT RESISTANCE
				■		5	☰		CLAYSTONE: grey.		L-M		MODERATE RESISTANCE WITH BANDS OF HIGH RESISTANCE
						6 7			END OF BOREHOLE AT 6.0m				

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BOREHOLE LOG

Borehole No.
2
1/1

Client: FRESH HOPE CARE
Project: PROPOSED INDEPENDENT LIVING UNITS
Location: 154 TO 162 STAFFORD STREET, PENRITH, NSW

Job No. 32041B
Date: 6/12/18
Method: SPIRAL AUGER JK205
R.L. Surface: ≈ 48.1m
Datum: AHD
Logged/Checked by: K.K.S./A.F.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION	■					0		-	ASPHATIC CONCRETE 30mm.t.	M			
				■	N = 13 3,5,8	1		CI-CH	FILL: Gravelly sand, fine to coarse grained, sandstone gravel, light brown, fine to coarse grained sand. Silty CLAY: medium to high plasticity, grey mottled orange and brown, trace of fine to coarse grained ironstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
				■ ■	N = 32 11,14,18	2		-	Extremely Weathered Claystone: CLAY, high plasticity, grey, with fine to coarse grained ironstone gravel.	XW	Hd		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
						3 4			CLAYSTONE: grey with ironstone gravel bands.	HW	VL		VERY LOW TO LOW RESISTANCE
						5					L		LOW RESISTANCE
				■							L-M		MODERATE RESISTANCE
						6 7			END OF BOREHOLE AT 6.0m				

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: FRESH HOPE CARE

Project: PROPOSED INDEPENDENT LIVING UNITS

Location: 154 TO 162 STAFFORD STREET, PENRITH, NSW

Job No. 32041B

Date: 6/12/18

Method: SPIRAL AUGER
JK205

R.L. Surface: ≈ 47.9m

Datum: AHD

Logged/Checked by: K.K.S./A.F.

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 COMPL -ETION	■					0		-	ASPHALTIC CONCRETE: 30mm.t.	M			
									FILL: Gravelly sand, fine to coarse grained, sandstone gravel light brown, fine to coarse grained sand.	w>PL		170 180 240	
	■			■	N = 5 1,2,3			CI-CH	FILL: Silty clay, high plasticity, light grey mottled orange brown, trace of shale fragments.	w<PL	Vst		RESIDUAL
						1			Silty CLAY: medium to high plasticity, grey mottled orange brown, trace of fine to coarse grained ironstone gravel.				
				■	N = 15 4,6,9			CH	CLAY: high plasticity, grey mottled orange brown, trace of fine to medium grained ironstone gravel.	w<PL		300 350 360	
				■		2 3 4		-	Extremely Weathered claystone: CLAY, high plasticity, grey mottled orange brown, trace of fine to medium grained ironstone gravel.	XW	Hd		BRINGELLY SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
				■					CLAYSTONE: grey, with ironstone bands.	HW	VL		VERY LOW RESISTANCE
				■		5					L		LOW TO MODERATE RESISTANCE
				■							M		
						6			END OF BOREHOLE AT 6.0m				
						7							

BOREHOLE LOG

Borehole No.
4
1/1

Client: FRESH HOPE CARE

Project: PROPOSED INDEPENDENT LIVING UNITS

Location: 154 TO 162 STAFFORD STREET, PENRITH, NSW

Job No. 32041B

Date: 6/12/18

Method: SPIRAL AUGER
JK205

R.L. Surface: ≈ 48.3m

Datum: AHD

Logged/Checked by: K.K.S./ A.F.

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 COMPL -ETION	■					0	▩		FILL: Silty sandy clay, medium plasticity, brown.	w≈PL			GRASS COVER
	■			■	N = 6 2,2,4	1	▨	CH	Silty CLAY: high plasticity, grey mottled orange brown.	w>PL	St-Vs t	180 210 210	RESIDUAL
				■	N = 19 4,7,12		▨	CH	Silty CLAY: high plasticity, as above, but with fine to coarse grained ironstone gravel.		Hd	550 530 480	
				■		2	▤	-	Extremely Weathered Claystone: CLAY, high plasticity, grey mottled orange brown, trace of fine to coarse grained ironstone gravel.	XW	Hd		BRINGELLY SHALE
				■ ■		3 4 5	▤		CLAYSTONE: grey, with ironstone bands.	HW	VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
						6 7			END OF BOREHOLE AT 6.0m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: FRESH HOPE CARE
Project: PROPOSED INDEPENDENT LIVING UNITS
Location: 154 TO 162 STAFFORD STREET, PENRITH, NSW

Job No. 32041B
Date: 6/12/18
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 48.9m
Datum: AHD
Logged/Checked by: K.K.S./A.F.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION	■					0			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sub angular igneous gravel.	w≈PL			GRASS COVER
	■			■	N = 8 1,2,6	1		CI-CH	Silty CLAY: medium to high plasticity, grey mottled orange brown, trace of fine to coarse grained ironstone gravel.	w<PL	(St) Hd	 >600 >600 >600	RESIDUAL
				■	N > 35 20,15/ 50mm REFUSAL	2 3 4 5		-	CLAYSTONE: grey, with ironstone bands.	HW	VL L		BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE LOW RESISTANCE
						6 7			END OF BOREHOLE AT 6.0m				

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

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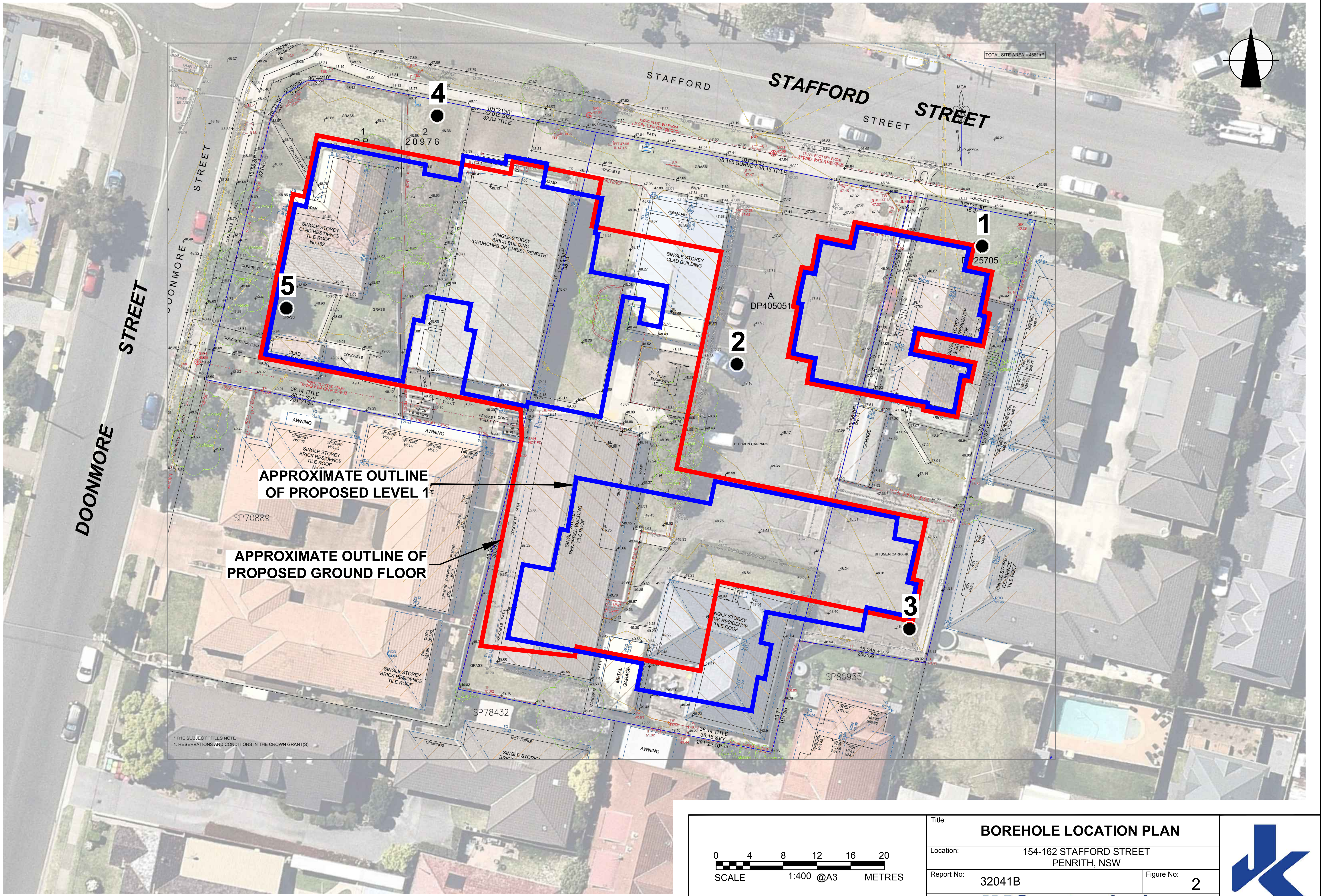
Title:	**SITE LOCATION PLAN**
Location:	154-162 STAFFORD STREET PENRITH, NSW
Report No: 32041B	Figure No: 1

JK Geotechnics

PLOT DATE: 3/01/2019 3:11:04 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32041B PENRITH\CAD\32041B.DWG



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 30 OCT 2018.

PLOT DATE: 8/04/2020 10:04:34 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32041B PENRITH\CAD\32041B.DWG

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:
BOREHOLE LOCATION PLAN

Location:
154-162 STAFFORD STREET
PENRITH, NSW

Report No: 32041B

Figure No: 2

JKGeotechnics

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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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:

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

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:

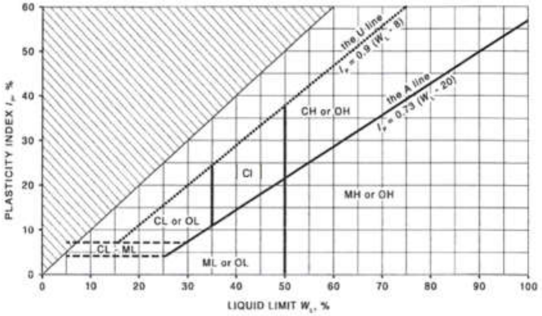
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



LOG SYMBOLS

<table>
<tr><th>Log Column</th><th colspan="2">Symbol</th><th>Definition</th></tr>
<tr><td>Groundwater Record</td><td colspan="2">▼
—C—
▶—</td><td>Standing water level. Time delay following completion of drilling/excavation may be shown.
Extent of borehole/test pit collapse shortly after drilling/excavation.
Groundwater seepage into borehole or test pit noted during drilling or excavation.</td></tr>
<tr><td>Samples</td><td colspan="2">ES
U50
DB
DS
ASB
ASS
SAL</td><td>Sample taken over depth indicated, for environmental analysis.
Undisturbed 50mm diameter tube sample taken over depth indicated.
Bulk disturbed sample taken over depth indicated.
Small disturbed bag sample taken over depth indicated.
Soil sample taken over depth indicated, for asbestos analysis.
Soil sample taken over depth indicated, for acid sulfate soil analysis.
Soil sample taken over depth indicated, for salinity analysis.</td></tr>
<tr><td rowspan="3">Field Tests</td><td colspan="2">N = 17
4, 7, 10</td><td>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.</td></tr>
<tr><td>N_c =</td><td>5
7
3R</td><td>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.</td></tr>
<tr><td colspan="2">VNS = 25
PID = 100</td><td>Vane shear reading in kPa of undrained shear strength.
Photoionisation detector reading in ppm (soil sample headspace test).</td></tr>
<tr><td>Moisture Condition
(Fine Grained Soils)

(Coarse Grained Soils)</td><td colspan="2">$w > PL$
$w \approx PL$
$w < PL$
$w \approx LL$
$w > LL$

D
M
W</td><td>Moisture content estimated to be greater than plastic limit.
Moisture content estimated to be approximately equal to plastic limit.
Moisture content estimated to be less than plastic limit.
Moisture content estimated to be near liquid limit.
Moisture content estimated to be wet of liquid limit.

DRY – runs freely through fingers.
MOIST – does not run freely but no free water visible on soil surface.
WET – free water visible on soil surface.</td></tr>
<tr><td>Strength (Consistency)
Cohesive Soils</td><td colspan="2">VS
S
F
St
VSt
Hd
Fr
( )</td><td>VERY SOFT – unconfined compressive strength ≤ 25kPa.
SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
HARD – unconfined compressive strength > 400kPa.
FRIABLE – strength not attainable, soil crumbles.
Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.</td></tr>
<tr><td>Density Index/
Relative Density
(Cohesionless Soils)</td><td colspan="2">
VL
L
MD
D
VD
( )</td><td><table>
<tr><th></th><th>Density Index (I_D) Range (%)</th><th>SPT ‘N’ Value Range (Blows/300mm)</th></tr>
<tr><td>VERY LOOSE</td><td>≤ 15</td><td>0 – 4</td></tr>
<tr><td>LOOSE</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr>
<tr><td>MEDIUM DENSE</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr>
<tr><td>DENSE</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr>
<tr><td>VERY DENSE</td><td>> 85</td><td>> 50</td></tr>
</table>Bracketed symbol indicates estimated density based on ease of drilling or other assessment.</td></tr>
<tr><td>Hand Penetrometer Readings</td><td colspan="2">300
250</td><td>Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.</td></tr>
</table>

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

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**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