REPORT

TO
GROUPGSA PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED POLICE STATION REDEVELOPMENT

AT
TAREE POLICE STATION
79 ALBERT STREET, TAREE, NSW

2 May 2018
Ref: 31340Prpt

dlcs ISO 9001 certified AS/NZS 4801 certified ISO 14001 certified

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Police Station Redevelopment at Taree Police Station, 79 Albert Street, Taree, NSW. A site location plan is presented as Figure 1. The investigation was commission by Mr Mark Bennett of GroupGSA Pty Ltd and was completed in accordance with our proposal (Ref: P46629P) dated 21 February 2018.

As part of our investigation we have been provided with architectural plans prepared by GroupGSA Pty Ltd (Project No. 180023, Drawing Nos. TAR-AR-0101, TAR-AR-0210, TAR-AR-0211, TAR-AR-0220, TAR-AR-0210 TAR-AR-0250, TAR-AR-0300, TAR-AR-0301, TAR-AR-0400 and TAR-AR-0401, all Phase 25%, print date 18/04/2018).

Based on these drawings we understand it is proposed to demolish the existing police station building and construct a new 1 and 2 level 'L' shaped building that wraps around the western and northern sides of the existing courthouse. The new building will have a finished floor level (FFL) at RL16.4m which will require excavation of up to about 2m towards the northern end of the site and will be constructed at or near the existing surface level towards the southern end of the site. The building will be set back about 19.0m and 8.9m from the northern and eastern boundaries, respectively, and about 1.7m to 3.1m from the western boundary. The new building will be setback from the existing courthouse between about 1.5m and 8.4m, with a connecting walkway located towards the centre of the site.

At the rear (northern) end of the site, there is a proposed concrete parking area with a secure vehicle examination bay in the north-eastern corner. The parking area will be constructed at RL16.25m and extend close to the northern, eastern and western boundaries, and will required excavations to about 2.5m depth.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for comments and recommendations on site preparation (including excavation, temporary batters, shoring and retaining wall design, and engineered fill placement), footing design, pavement design, groundwater considerations and soil aggression.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on the 9th and 10th April 2018, and included the auger drilling of five boreholes (BH1 to BH5) to refusal depths between 1.0m and 5.5m. The boreholes were drilled using our track mounted JK300 drill rig.

The borehole locations, as shown on Figure 2, were set out using taped measurements from existing surface features, and were electromagnetically scanned for buried services prior to drilling commencing. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plan prepared by Crux Survey Australia Pty Ltd (Ref. 121187-SU-DT-001, revision C, dated 23 March 2018) and so should be considered to be approximate. The datum of the levels is Australian Height Datum (AHD).

The nature and composition of the subsurface profile was assessed by logging the materials recovered during drilling. The relative compaction/strength of the subsoils were assessed from the Standard Penetration Test (SPT) 'N' value, augmented by hand penetrometer readings on cohesive samples recovered in the SPT sampler. The strength of the sandstone bedrock was assessed by observation of the auger penetration resistance using a tungsten carbide 'TC' drill bit, together with examination of the recovered rock cuttings, and from correlations with the results of subsequent moisture content tests completed on the rock chips. It should be noted that strengths assessed in this way are approximate, and variances of one strength order should not be unexpected.

Groundwater observations were made during and on completion of auger drilling. Slotted PVC standpipes were installed in BH3 and BH4 (which were completed on the first day of fieldwork) on completion of drilling to allow measurements of groundwater levels, and the groundwater levels were measured in these on the second day of fieldwork. No longer term groundwater monitoring has been carried out.

Our geotechnical engineer, Mr Michael Serra, set out the borehole locations, nominated sampling and testing locations and prepared logs of the strata encountered. The borehole logs are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected 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 and linear shrinkage, standard compaction properties, four day soaked CBR values, soil pH, sulphate contents, chloride contents and resistivity. The results of the laboratory testing are summarised in STS Tables A and B, and the Envirolab Certificate of Analysis No. 189349.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in a gently sloping terrain that consists of a number of small rounded hills on the north-western side of the Manning River. The general slopes of the topography are in the order of about 1^o to 3^o.

The site itself slopes down to the south-east at about 3^o and is occupied by the existing Taree Police Station. The police station comprises various 1 and 2 story brick buildings, as well as a single level demountable building towards the east, and open steel carports along the north-eastern corner of the site. The buildings are surrounded by concrete and asphaltic concrete (AC) pavements. The buildings appear in good external condition based on a cursory inspection, while some sections of the pavements had significant cracking and settlement. The onsite police personnel stated that groundwater seepage occurs in the rear (northern) parking area, with seepage appearing at the joints in the concrete pavement and ponding of water. They also commented that an underground petrol tank was located in the north-western corner of the site, in the vicinity of our BH3, and had been removed or filled in the early 1990's.

Neighbouring the site to the south-east of the police station is the existing Taree Court House. The court house consists of 2 and 3 level rendered buildings which are setback about 2m to 5m from the police station, connected by an enclosed walkway. The police station is accessed by one concrete driveway on either side of the existing combined Taree Police Station and Court House properties.

To the north of the site are single level weatherboard and demountable buildings beyond the eastern half of the site; the buildings are set back about 3m from the common boundary. A grassed area was present beyond the west portion of the northern boundary.

To the east is a two level brick building (St Mary's Hall) which is set back about 3m from the common boundary. A concrete driveway runs between the driveway and building to an AC carpark at the rear of that property. To the west is a single level weatherboard building (No. 89 Albert Street) located along the Albert Street frontage and set back about 1m from the common boundary with an open steel structure and weatherboard garage to its rear which abuts the boundary.

3.2 Subsurface Conditions

The boreholes have disclosed the site to be underlain by a variable profile. The conditions in the rear half of the site generally disclose a thin layer of fill and natural soil over shallow sandstone bedrock, with the exception of a deeper fill profile in BH3 which we were advised was drilled within an old backfilled tank pit. BH5, towards the front (southern) end of the site disclosed a deeper alluvial profile comprising a thin layer of fill over very soft to firm silty clay, which in turn overlies the sandstone bedrock. Some of the characteristic features of the conditions encountered are described below. Reference should be made to the attached borehole logs for detailed subsurface conditions at specific locations.

Pavements and Fill

In BH1, BH2 and BH4, a concrete pavement was encountered and was of 130mm thickness. BH3 was drilled through an asphaltic concrete (AC) surface of about 30mm thickness and BH4 was drilled through a bituminous surface of 16m thickness.

In BH1, BH2 BH4 and BH5, fill was encountered beneath the pavements to depths between 0.2m and 0.9m and comprised a mixture of igneous and sandstone gravel, sand and silty clay. In BH4 and BH5, the fill was assessed to be moderately to well compacted, based on the SPT ‘N’ values.

In BH3, fill was encountered beneath the AC pavement to a depth of 4.5m. The fill was quite variable, with layers of igneous and sandstone gravel, sand and silty clay to a depth of 1.6m. Igneous gravel (ballast) was encountered between 1.6m and 2.5m, underlain by sandy fill to 4.5m. The fill had a hydrocarbon odour from a depth of 1.2m to 4.5m. Based on the SPT ‘N’ value at 0.5m, the upper 1m of fill was assessed to be well compacted, however due to the continual collapsing of the gravel fill, no further SPT tests could be carried out. Based on our discussion with the police personnel onsite, we understand BH3 was completed within the vicinity of the old underground fuel tank.

Natural Silty Clay

Natural sandy and silty clays were encountered beneath the fill in all boreholes, except BH3. The clay was assessed to be of high plasticity. In BH1, BH2 and BH4, the clay was assessed to be of stiff to very stiff strength, while in BH5 (at the southern end of the site) the clay was assessed to be initially firm, becoming very soft to soft from a depth of about 2m.

Sandstone Bedrock

Sandstone bedrock was encountered below the fill and natural soilsfrom depths ranging between 0.45m (BH2) and 5.1m (BH5) and steps down significantly towards the south (towards the Manning River). In BH3 sandstone bedrock was encountered beneath the fill at a depth of 4.5m.

The sandstone was generally assessed to be of low to high strength on first contact, quickly becoming high strength, with auger refusal occurring in all boreholes at depths between 1.0m (BH1) and 5.5m (BH5).

Groundwater

Groundwater seepage was encountered during the drilling of BH5 at a depth of 1.4m and a depth of 1.0m upon completion of drilling. All other boreholes where ‘dry’ both during and on completion of drilling.

Standpipes were installed in BH3 and BH4 and measured 1 day after drilling. In BH4, groundwater was measured at a depth of 1.6m, while in BH3 the standpipe was ‘dry’ to a depth of 3.8m where it had become blocked.

Based on the groundwater level in BH4, BH5, and the discussion with police personnel onsite, it appears that the seepage may occur along or near the top of the sandstone bedrock towards the northern end of the site with the southern end of the site located over a deeper alluvial profile with a relatively high water table.

3.3 Laboratory Test Results

The results of moisture content tests on selected samples of the bedrock correlate reasonably well with the field strength assessment; the test result on a sample from BH5 had a higher than expected moisture content, which is likely due to seepage affecting the sample.

The results of the Atterberg Limits tests confirmed the natural clays to be of high plasticity, with the linear shrinkage tests indicating these soils to generally have a high potential for shrink-swell reactive movements with changes to moisture content.

Four day soaked CBR tests on samples of the natural clay from BH3 and BH4 compacted to 98% of its Standard Maximum Dry Density (SMDD) gave CBR values of 4.5% and 4.0%, respectively.

The soil pH values ranged from 7.7 to 9.1. The chloride and sulphate contents were less than 50mg/kg. Resistivity ranged from 4,700ohm.cm to 14,000ohm.cm. Based on these results the soil would be classified as providing a "Non-aggressive" exposure classification for concrete and steel piles in accordance with Table 6.4.2(C) and Table 6.5.2 (C) of AS2159-2009 'Piling – Design and Installation'.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

All excavation recommendations should be complemented by reference to Safe Work Australia's 'Excavation Work Code of Practice', dated July 2015 and AS3798 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.1.1 Dilapidation Surveys

Prior to the commencement of demolition and excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the north, south, east and west. The dilapidation surveys should include detailed internal and external inspections of the neighbouring buildings, where all defects including defect location, type, length and width are rigorously described and photographed. The respective owners should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future damage claims. MS - Not really practical to make several trips to Taree for this.

4.1.2 Excavation Techniques

Following the demolition of the existing buildings and pavements on the site, any vegetation and topsoil within the development footprint, will be required, together with any material containing deleterious substances; this material must be treated and disposed of in accordance with the recommendations of the EIS site assessment.

Demolition of existing structures and subsequent excavation will need to be carried out with care, so as to not destabilise, undermine or remove lateral support from the surrounding road and neighbouring properties. All demolition and excavation work will need to be carried out by suitably experienced and insured contractors.

Based on the investigation results, excavation to a maximum depth of about 2.5m for the two-storey building and the rear car parking area will extend through the soil profile and into the underlying sandstone bedrock, of variable, but predominantly high strength which will represent “hard rock” excavation conditions. The exception is the area of deeper fill in the vicinity of BH3 where the existing fill will extend below the proposed bulk excavation level.

Excavation of the soil profile and extremely low or very low strength bedrock may be readily completed using a bucket fitted to a large tracked excavator. Bedrock of low and higher strength would be most effectively excavated using rock breaker attachments to the excavators, or by sawing the rock into blocks and then ripping these from the excavation using ripping tynes on the excavators. The rock breakers could also be used for trimming rock excavation side slopes, detached boulders etc. and for detailed rock excavations such as for footings, trenches, lift pits etc. We recommend that the perimeter of the excavation be saw cut to reduce over break and to reduce transmitted vibrations.

Dust suppression by spraying with water should be carried out whenever rock saws or grinders are being used.

Rock excavations using rock breakers will need to be strictly controlled as there will probably be direct transmission of ground vibrations to nearby buildings and buried services. We recommend that quantitative vibration monitoring be carried out whenever rock breakers are used on this site, as a precaution against possible vibration induced damage. By referencing German Standard DIN4150-3:1999-02, the vibrations on the neighbouring house to the north and east should be limited to a peak particle velocity of 5mm/s, subject to review of the dilapidation survey reports. If this vibration limit is exceeded, the vibrations should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable depending on the vibration frequency. If the vibration monitoring confirms that transmitted vibrations are excessive, then it would be necessary to change to a smaller rock hammer. Alternatively, geotechnical advice could be sought with respect to alternative excavation techniques.

The following procedures are recommended to reduce vibrations, if rock breakers are used:

- Rock saw the perimeter faces. This will effectively reduce ground borne vibrations, provided the base of the rock saw slot is maintained at a lower level than the adjacent excavation level at all times. Rock sawing would also improve the aesthetics of the completed rock cut faces.
- Maintain rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.

- Operate the rock hammer in short bursts only, to reduce the amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances.

We recommend that a copy of this report be provided to the excavation contractor so that they can make their own assessment of excavation conditions.

4.1.3 Seepage

We expect some groundwater inflow into the excavation will occur as local seepage flows within the fill, at the soil/rock interface, as well as through joints and bedding partings within the bedrock profile, particularly during and immediately following periods of heavy rainfall.

Seepage volumes into the excavation, are expected to be relatively low such that they would be controllable by gravity or conventional sump and pump methods. Notwithstanding, groundwater seepage monitoring should be carried out by site staff during excavation so that any unexpected conditions can be addressed.

A spoon drain should be provided at the base of all rock cuttings to collect groundwater seepage and lead it to a sump for appropriate disposal.

4.2 Batter Slopes and Retaining Walls

4.2.1 Batter Slopes

The following temporary batters should be accommodated for the soil and weathered rock;

- Temporary batter slopes no steeper than 1V in 1H through the soils and extremely weathered bedrock, or any bedrock of less than low strength.
- Temporary batter slopes through sandstone bedrock of at least low strength may be cut vertically subject to inspections by the geotechnical engineers at not greater than 1.5m depth intervals. Where adverse defects are encountered then these will need to be stabilised such as with rock bolts, and possibly with reinforced shotcrete panels restrained by rock bolts for particularly poor areas of bedrock. If vertical sandstone excavations are proposed within 1.5m of adjoining structures then an inspection must be undertaken by the geotechnical and structural engineers prior to excavating below the level of the adjoining footings. It is likely

that some staging of the excavation will be recommended, such as by excavating slots ahead of the proposed bulk excavation, having these slots inspected by a geotechnical engineer to confirm there are no adversely oriented defects, before bulk excavating between the slots.

We recommend all surcharge loads (such as construction traffic etc) are kept well clear of the crest of the temporary batter slopes (at least twice the height of the batter slope from the toe of the batter), unless geotechnical assessment and/or stabilisation of excavation faces is carried out and the geotechnical engineers confirm that surcharge loads can be placed closer than the above recommendations.

Where temporary batters cannot be accommodated (i.e. along the northern, eastern and western boundaries of the proposed car park), then any proposed vertical excavation would need to be supported by an engineered shoring system, which must be installed prior to excavation commencing.

4.2.2 Shoring and Retaining Wall Design

We consider a shoring system comprising soldier pile walls with reinforced shotcrete infill panels would be suitable for this site. The soldier piles should be terminated in the sandstone bedrock below bulk excavation level. In the vicinity of the backfilled tank pit near BH3, the piles will need to be socketed below the level of the base of the pit. We note however that the sandstone bedrock was usually of high strength, and so these shoring piles will be difficult to drill, requiring large piling rigs, and rock augers with rock teeth, and possibly coring buckets. As the height of the shoring will generally be less than 2.5m, it may be possible to cantilever the piles from the rock socket, though again these sockets may be hard to drill and so a combination of shorter rock sockets and an upper level of temporary anchors may be preferred. If temporary anchors are used, then the walls will require alternative long terms lateral support, such as by bracing from the structures.

Retaining walls constructed within battered excavations are expected to be of low height (no more than 2.5m) and may be designed as cantilevered walls based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.35 and a bulk unit weight of 20kN/m^3. This assumes that some minor movement of the walls will be tolerable. Where movements are to be kept low, such as where walls are restrained by other structural elements on front of the wall, an ‘at rest’ earth pressure coefficient, K_0, of 0.6 should be used.

The above lateral earth pressure coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients should be appropriately increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the wall.

Where the toes of the soldier piles socketed within the bedrock are used for lateral restraint, the design may be based upon an allowable lateral bearing pressure of 300kPa for rock of at least medium strength, providing the upper 0.5m of socket into rock below any adjacent excavation (including detailed excavation for the construction of footings or buried services) is ignored in the design.

If temporary anchors are used for the design, they should have free lengths and bond lengths of at least 3m each. The bond length of the anchors into rock of at least low strength may be designed for an allowable bond of 100kPa. The anchors should be proof loaded to at least 130% of their working load prior to locking-off, and list-off tests must be completed on at least 25% of anchors about four days following lock-off to confirm the anchors are continuing to hold their load.

4.3 Subgrade Preparation and Pavement Design

4.3.1 Subgrade Preparation

Where clay or fill is exposed at the proposed subgrade level, it should be proof rolled with at least 8 passes of a non-vibratory smooth drum roller of about 12 tonne size. The final pass of the proof rolling should be carried out within the presence of a geotechnical engineer to detect any weak subgrade areas.

Any weak areas detected during proof rolling or where the clay subgrade is exposed to periods of rainfall and ‘softening’; the subgrade should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during the proof rolling inspection.

Where weathered sandstone is exposed at the subgrade level no proof rolling would be required.

4.3.2 Engineered Fill

Engineered fill should preferably comprise well graded granular materials, such as ripped or crushed sandstone, free of 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 natural clay and weathered bedrock may be used as engineered fill provided it is free of deleterious materials and crushed to be well graded with no particles greater than 75mm in size. Any clay fill should be compacted in 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of their Standard Optimum Moisture Content (SOMC).

Backfill behind retaining walls and for service trenches should also comprise engineered fill. Due to limited access for machinery, compaction of backfill immediately behind retaining walls and in service trenches may need to be completed using smaller compaction equipment (e.g. upright rammer compactors, sled compactors or small rollers). Due to the reduced energy output of such equipment, fill in such areas must be placed in maximum 100mm loose thickness layers, and have a maximum particle size not exceeding 40mm. The compaction of engineered fill induces higher stresses on the retaining walls, and so consideration could be given to backfilling the retaining walls with a hard and durable, free-draining granular fill, such as crushed igneous rock with less than 5% fines (clay and silt). Such material could be placed in 0.5m thick layers and tamped into place using the back of an excavator bucket. A non-woven geotextile filter fabric should be placed between the gravel and the batter behind/below, and should extend across the top of the backfill as well. The upper 0.4m of backfill should comprise a more clayey soil to prevent surface water entering the backfill.

Density tests should carried out on the engineered fill to confirm the above specifications are achieved at the frequency below. All density testing must be completed over the full thickness of each compacted fill layer.

- The frequency of density testing for engineered fill should be at least one test per layer per $500m^2$, one test per $100m^3$ distributed reasonably evenly throughout the full depth and area, and at least three tests per layer per earthworks lot, whichever requires the most tests.
- The frequency of density testing for engineered backfill behind retaining walls and trenches should be at least one test per two layers per 50 linear meters.

4.3.3 Pavement Design

Where the subgrade comprise residual clays, provided the subgrade is prepared as detailed above, it may be designed based on a soaked CBR of 4.0%, or an estimated modulus of subgrade reaction of 30kPa/mm for a 750mm plate. Where sandstone bedrock of at least low strength forms the subgrade for pavements, they should be designed for a CBR of 8% (or an estimated modulus of subgrade reaction of 50kPa/mm for a 750mm plate) as some weathering or disturbance from the removal of services is likely to occur.

Surface and subsoil drainage should be provided on the high side of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 200mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain.

Concrete pavements should have a subbase layer of at least 100mm thickness of DGB20 (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.

4.4 Footing Design

The excavation for the northern/two-story building will likely expose sandstone bedrock over a large portion of the building. As some footings for the building will be founded on the bedrock, the building should be supported entirely on footings founded within the sandstone bedrock to provide uniform support and reduce the risk of differential settlements. Towards the northern end of the site, we expect that sandstone will be exposed at or within 1m of the bulk excavation level and the building may be supported on pad or strip footings. Towards the southern end of the site, where the depth to the sandstone is more than about 1m, piling will be required.

In the southern single storey building, there is fill and then natural soils of as low as very soft strength. Such materials are not suitable for use as a bearing stratum, and this building should also be founded on piled footings founded within the sandstone bedrock.

As the depth to the sandstone bedrock varies considerably across the site, additional geotechnical investigation completed following demolition of the existing buildings would be advantageous to allow better targeting of pile depths and optimisation of the bearing pressures for the footing/piling design. The investigation should include a grid of boreholes probing down to bedrock. If possible, it may be advantageous if the client could obtain the geotechnical report and/or as constructed piling/footing records for the existing courthouse to provide further information on the depth to, and quality of, the sandstone bedrock.

Footings founded within the sandstone of at least low strength may be designed based on allowable bearing pressure of 1,000kPa. Where piles are adopted, an allowable shaft adhesion of 10% of these allowable bearing pressures for compression loads and 5% for uplift loads may be used. The skin friction should be ignored within the initial 0.3m of socket length and are based on adequate socket cleanliness and roughness being achieved.

Higher bearing pressures are probably feasible, but a more comprehensive geotechnical investigation involving cored boreholes would be required to confirm the presence and depth of such rock.

Where piling is required, we recommend that CFA (grout injected augered) piles be adopted due to the presence of a high groundwater table in places, and the presence of collapsing granular materials in the tank pit backfill near BH3.

We note that sandstone bedrock of high strength was encountered, and the potential piling contractors must ensure that pile rigs capable of drilling into this material be used,. When CFA piles are installed, the rock being drilled cannot be inspected, and therefore the piling contractor should be require to self-certify their piles are suitable for the adopted bearing pressures.

At least the initial stages of footing excavation or pile drilling should be inspected by a geotechnical engineer to confirm that the appropriate foundation material has been encountered. The need for additional inspections should be assessed following the initial inspection depending on the bearing pressure adopted and any variations between the borehole locations.

4.5 On-grade Floor Slabs

We expect that the proposed on-grade floor slab will be cast on weathered sandstone bedrock at the north end of the site and on alluvial clays, or possibly even engineered fill at the southern end. The subgrade should be prepared in accordance with the recommendations provided in the sections above.

The on-ground floor slabs should be cast independent of the footing system and structure above to allow for the minor movements that can occur where the structure is supported on rock and the slabs on the soil. If such movements cannot be tolerated, then the slabs should be designed as being suspended from the footings. Subsoil drainage should be installed around the perimeter of the building to intercept seepage and direct it to gravity fed drains or sumps for pumped disposal. The subsoil drainage should comprise a slotted pipe embedded in durable, single sized washed aggregate (eg.'blue metal' gravel) which is itself wrapped in a non-woven geotextile filter fabric.

Where the buildings are underlain by clays of firm strength or less, such as in the southern building, it will not be possible to improve the compaction of the moderately compacted fill. Further, the application of the shallow fill and floor slab could cause further consolidation of the very soft and soft soils below, resulting in settlement of the floor slabs in relation to the structure which would be founded on the bedrock. In this situation, the ground floor slab should also be suspended from the piled footings. Following the demolition of the existing structures, it will be important to drill additional boreholes along the southern edge of the proposed northern building to check that these soft soil conditions do not extend below any of that structure.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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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SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Police Station Redevelopment
Location: Taree Police Station, 79 Albert Street, Taree, NSW

Ref No: 31340P
Report: A
Report Date: 1/05/2018
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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	0.60-1.00	3.0				
2	0.90-1.00	3.8				
4	1.50-1.80	21.2	56	20	36	15.0
4	2.30-2.50	3.8				
5	1.50-1.95	35.5	58	25	33	15.0
5	5.10-5.40	22.0				

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: 17/4/18

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Authorised Signature / Date
(A. Tatikonda) 1/5/18

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:**	31340P
Project:	Proposed Police Station	**Report:**	B
Location:	Taree Police Station, 79 Albert Street, Taree, NSW	**Report Date:**	1/05/2018
		Page 1 of 1	

BOREHOLE NUMBER	3	4
DEPTH (m)	1.00 - 1.40	1.00 - 1.50
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.89 STD	1.82 STD
Optimum Moisture Content (%)	14.7	17.0
Moulded Dry Density (t/m^3)	1.85	1.79
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	102	100
Moisture Contents		
Insitu (%)	12.7	19.2
Moulded (%)	15.0	17.1
After soaking and		
After Test, Top 30mm(%)	18.5	21.5
Remaining Depth (%)	16.0	20.8
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	1.5	1.0
C.B.R. value: @5.0mm penetration	4.5	4.0

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample:17/4/18

NATA
NATA Accredited Laboratory
Number:1327

Authorised Signature / Date
(A. Tatikonda) 1/5/18

Envirolab Services Pty Ltd
ABN 37 112 535 645
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 189349

Client Details

Client	JK Geotechnics
Attention	Michael Serra, P Wright
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31340P , Taree**
Number of Samples	3 Soil
Date samples received	12/04/2018
Date completed instructions received	12/04/2018

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	19/04/2018
Date of Issue	16/04/2018

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil				
Our Reference		189349-1	189349-2	189349-3
Your Reference	UNITS	BH3	BH4	BH5
Depth		0.5-0.95	1.5-1.8	3.0-3.45
Date Sampled		09/04/2018	09/04/2018	10/04/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	15/04/2018	15/04/2018	15/04/2018
Date analysed	-	15/04/2018	15/04/2018	15/04/2018
pH 1:5 soil:water	pH Units	9.1	8.4	7.7
Chloride, Cl 1:5 soil:water	mg/kg	<10	20	<10
Sulphate, SO4 1:5 soil:water	mg/kg	23	32	41
Resistivity in soil*	ohm m	90	47	140

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	-			15/04/2018	[NT]	[NT]	[NT]	[NT]	15/04/2018	[NT]
Date analysed	-			15/04/2018	[NT]	[NT]	[NT]	[NT]	15/04/2018	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	103	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	91	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	105	[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: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-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: GROUPGSA
Project: PROPOSED TAREE POLICE STATION REDEVELOPMENT
Location: 79 ALBERT STREET, TAREE, NSW

Job No. 31340P
Method: SPIRAL AUGER JK300
R.L. Surface: ~ 16.9m
Date: 9/4/18
Datum: AHD
Logged/Checked by: M.S./P.W.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			CONCRETE: 130mm.t				100mm TOP COVER 5mm DIA. REINFORCEMENT
								-	FILL: Clayey sand, fine to medium grained, dark grey brown, with fine to medium grained igneous gravel.	M			
								CH	Sandy CLAY: high plasticity, dark grey and green grey.	w>PL	(St-VSt)		ALLUVIAL
					N = SPT 4/20mm REFUSAL			-	SANDSTONE: fine to medium grained, light yellow brown, with brown bands.	DW	H		HIGH 'TC' BIT RESISTANCE
						1			END OF BOREHOLE AT 1.0m				'TC' BIT REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
2

Client: GROUPGSA
Project: PROPOSED TAREE POLICE STATION REDEVELOPMENT
Location: 79 ALBERT STREET, TAREE, NSW

Job No. 31340P
Date: 9/4/18
Method: SPIRAL AUGER JK300
R.L. Surface: ~ 18.2m
Datum: AHD
Logged/Checked by: M.S./P.W.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 130mm.t				
			-	FILL: Gravel, fine to medium grained, dark grey, igneous, sub angular.	M			
			CH	Silty CLAY: high plasticity, dark grey, trace of sand.	w>PL	(St)		ALLUVIAL
	N = SPT 3/20mm REFUSAL		-	SANDSTONE: fine to medium grained, light yellow brown, and dark red brown.	DW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE
		1				H		HIGH RESISTANCE
				END OF BOREHOLE AT 1.1m				'TC' BIT REFUSAL
		2						
		3						
		4						
		5						
		6						
		7						

SAMPLES: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **3** 1/1

Client: GROUPGSA
Project: PROPOSED TAREE POLICE STATION REDEVELOPMENT
Location: 79 ALBERT STREET, TAREE, NSW

Job No. 31340P **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ~ 18.6m
Date: 9/4/18 **Datum:** AHD
Logged/Checked by: M.S./P.W.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND ON 10/4/18		0	-	ASPHALTIC CONCRETE:30mm.t	M			APPEARS WELL COMPACTED
				FILL: Sandy gravel, fine to medium grained, dark grey and green grey igneous, sub angular.				
	N = 26 10,15,11			FILL: Clayey gravelly sand, fine to medium grained, dark grey and dark yellow brown, fine to medium grained igneous gravel.				
		1		FILL: Silty sandy clay, low to medium plasticity, dark grey and dark red brown, trace of fine grained igneous gravel.	w<PL			HYDROCARBON ODOUR
					M			BANDED LOW TO HIGH 'TC' BIT RESISTANCE
				FILL: Gravelly sand, fine to medium grained, orange brown, fine to medium grained sandstone gravel.				
		2		FILL: Sandy gravel, medium to coarse grained dark grey and dark brown igneous, sub angular, with cobbles.				BALLAST TYPE MATERIAL
				FILL: Silty clayey sand, fine to medium grained, green grey and dark brown.				HYDROCARBON ODOUR
		3		as above, but trace of fine grained igneous gravel.				
		4						STANDPIPE BLOCKED AT 3.8m
			-	SANDSTONE: fine to medium grained, dark orange brown.	DW	L-M		LOW TO MODERATE RESISTANCE
						H		HIGH RESISTANCE
		5		END OF BOREHOLE AT 4.9m				'TC' BIT REFUSAL
		6						GROUNDWATER MONITORING WELL INSTALLED TO 4.9m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.4m TO 4.9m. CASING 0m TO 2.4m. 2mm SAND FILTER PACK 2.0m TO 4.9m. BENTONITE SEAL 0.5m TO 2.0m.
		7						

SAMPLES: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: GROUPGSA
Project: PROPOSED TAREE POLICE STATION REDEVELOPMENT
Location: 79 ALBERT STREET, TAREE, NSW

Job No. 31340P **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ~ 18.0m
Date: 9/4/18 **Datum:** AHD
Logged/Checked by: M.S./P.W.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	-	BITUMINOUS SURFACE: 16mm.t				
	N > 25 13, 25/150mm REFUSAL			FILL: Sandy gravel, fine to coarse grained, dark grey and dark brown igneous and sandstone, sub angular.	M			APPEARS WELL COMPACTED
ON 10/4/18	N > 16 5,16/ 150mm REFUSAL	1	CH	Silty CLAY: high plasticity, dark grey mottled light yellow brown, trace of fine grained sand.	w>PL	St-VSt	180 220 220	ALLUVIAL
		2	-	SANDSTONE: fine to medium grained, dark orange brown mottled red brown.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
						H		HIGH STRENGTH
		3 4 5 6 7		END OF BOREHOLE AT 2.5m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 2.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.5m TO 2.5m. CASING 0m TO 1.5m. 2mm SAND FILTER PACK 1.0m TO 2.5m. BENTONITE SEAL 0m TO 1.0m. COMPLETED WITH A CONCRETE GATIC COVER.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **5**

1/1

Client: GROUPGSA

Project: PROPOSED TAREE POLICE STATION REDEVELOPMENT

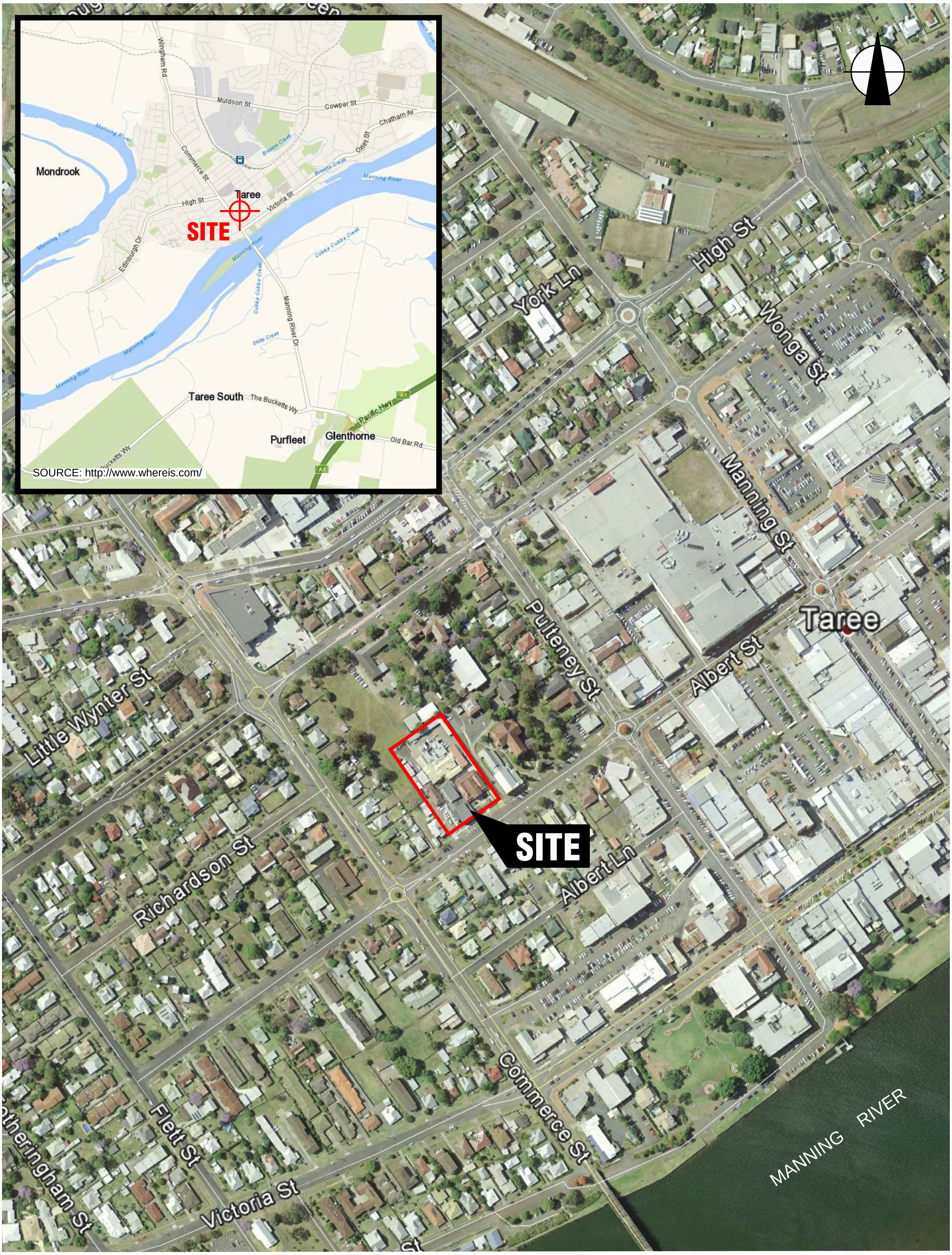
Location: 79 ALBERT STREET, TAREE, NSW

Job No. 31340P | **Method:** SPIRAL AUGER JK300 | **R.L. Surface:** ~ 15.9m

Date: 9/4/18 | **Datum:** AHD

Logged/Checked by: M.S./P.W.

Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	0		CONCRETE: 130mm.t				90mm TOP COVER 6mm DIA. REINFORCEMENT
		-	FILL: Clayey gravel, fine grained, dark grey and brown igneous, sub angular.	M			
N = 7 5,2,5			FILL: Sandy clay, medium to high plasticity, dark grey and grey brown, trace of fine to medium grained igneous and sandstone gravel, and ash.	w>PL			APPEARS MODERATELY COMPACTED
	1		FILL: Sandy gravel, fine to medium grained, orange brown sandstone, sub rounded.	M			
N = 3 2,2,1		CH	Silty CLAY: high plasticity, grey mottled light yellow brown.	w>PL	F	90 110 100	ALLUVIAL
	2				VS-S		
N = 1 0,0,1	3					30 40 50	
	4						
N = 0 0,0,0						30 30 20	SPT SUNK UNDER WEIGHT OF HAMMER
	5	-	SANDSTONE: fine to medium grained, grey and light brown mottled dark grey.	DW	H		HIGH 'TC' BIT RESISTANCE
			END OF BOREHOLE AT 5.5m				'TC' BIT REFUSAL
	6						
	7						

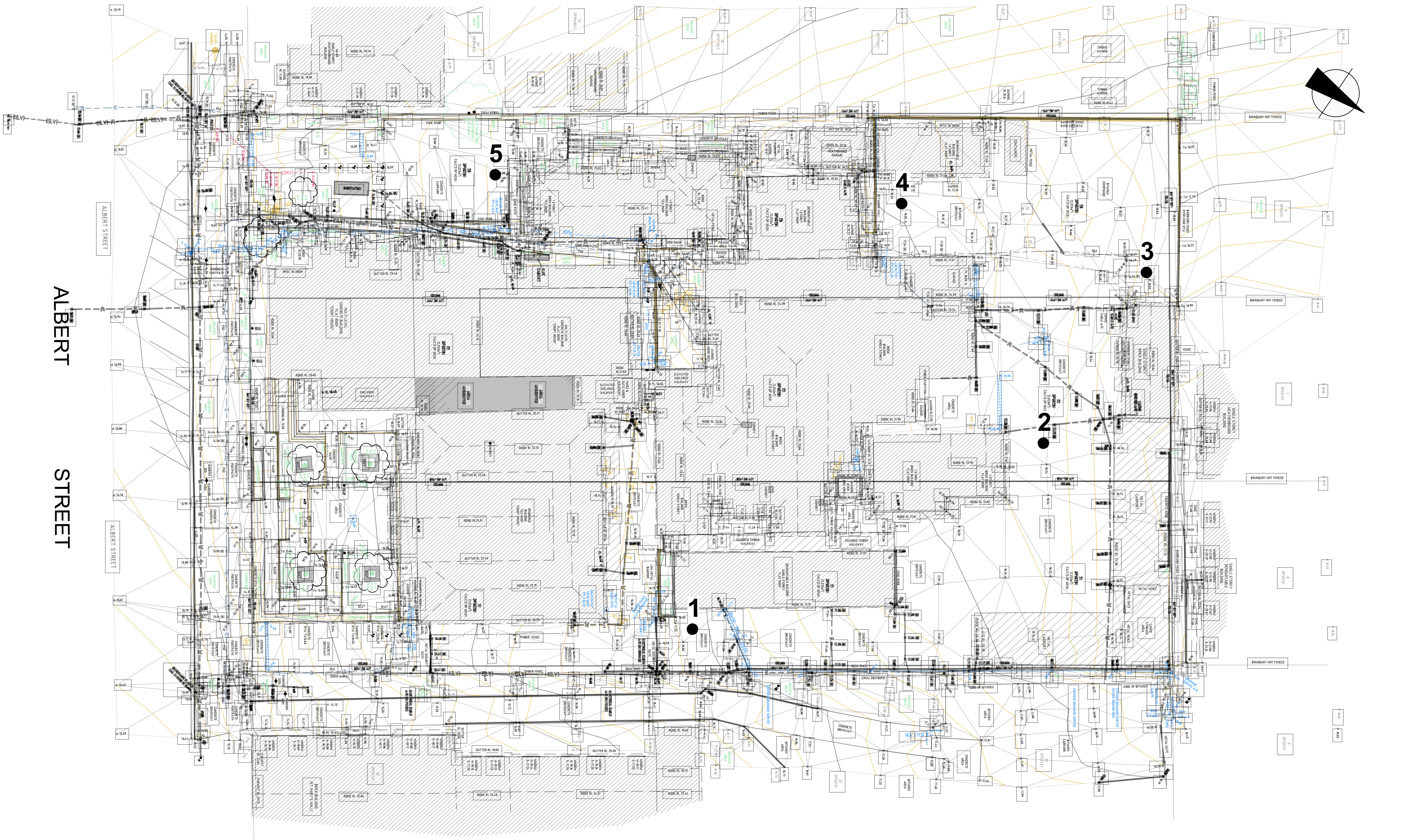


AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	TAREE POLICE STATION 79 ALBERT STREET, TAREE, NSW
Report No: 31340P	Figure No: 1
	JK **Geotechnics**

PLOT DATE: 16/04/2018 2:45:41 PM DWG FILE: S:\16 GEOTECHNICAL\JK GEOTECHNICAL JOBS\31000'S\31340P TAREE\CAD\31340P.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:	TAREE POLICE STATION 79 ALBERT STREET, TAREE, NSW
Report No: 31340P	Figure No: 2

JK **Geotechnics**

PLOT DATE: 30/04/2018 12:11:22 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31340P TAREE\CAD\31340P.DWG

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:

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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

NOTES:

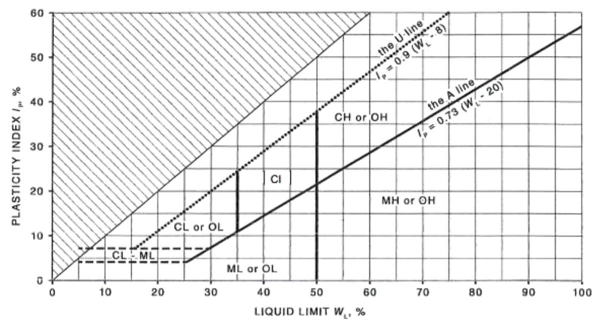
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

			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