REPORT

TO
IGLU No. 210 PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED REDEVELOPMENT

AT
SUMMER HILL AMBULANCE STATION
74 AND 75 CARLTON CRESCENT, SUMMER HILL, NSW

30 November 2018
Ref: 28412Lrpt2

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed redevelopment of Summer Hill Ambulance Station at 74 and 75 Carlton Crescent, Summer Hill, NSW. The original investigation was commissioned by NSW Government Health Infrastructure by Purchase Order No. 22987469 dated 29 May 2015. We understand that Iglu No.210 Pty Ltd have since purchased the site, and as such this report has now been prepared for their purposes. The geotechnical investigation was carried out in accordance with our fee proposal (Ref: P40483L) dated 11 May 2015. No additional investigations have been carried out as a result of issuing this report to Iglu No.210 Pty Ltd.

At the time of initial reporting in July 2015, the proposed redevelopment was currently at the conceptual stage and limited details were available. Based on an email from Michael Stern of Savills Australia, dated 29 May 2015, we understood the proposed redevelopment would comprise the demolition of the existing buildings and construction of a new ambulance station. We assumed that the new ambulance station would comprise lightly loaded one to two storey buildings. Excavation, if any was assumed to be limited to depths of 3m. We assumed that existing ground surface levels would not be raised.

We have not been provided with any proposed development drawings by Iglu No.210 Pty Ltd. On that basis this report assumes that the proposed development will be of a similar nature to that originally proposed in July 2015. Once specific development details are provided, this geotechnical report will need to be updated to reflect the actual proposed development.

The scope of the geotechnical investigation was to assess the subsurface conditions at five borehole locations and, based on the information obtained, to present our preliminary comments and recommendations on geotechnical issues relevant to the proposed development such as excavation conditions, retention parameters, groundwater, footing design and earthworks.

The geotechnical investigation was carried out in conjunction with a hazardous building materials assessment and a preliminary waste classification assessment by our environmental division, Environmental Investigation Services (EIS). Reference should be made to the separate reports by EIS (Ref: E28412K) for the results and recommendations.

2 INVESTIGATION PROCEDURE

Prior to commencement of the fieldwork a 'Dial Before You Dig' services search was carried out and all borehole locations were electromagnetically scanned for buried services by a specialist subcontractor.

The fieldwork for the investigation was carried out on 26 May 2015 and comprised the drilling of five boreholes (BH1 to BH5 inclusive) using our track mounted JK205 drill rig. Boreholes BH1 to BH4 were drilled to 6m depth. BH5 was drilled through what turned out to be a suspended concrete slab over an under croft void and was terminated at 0.2m upon penetration of the slab. The boreholes were initially diatube cored through the existing concrete pavements. After diatube coring, the boreholes were continued using spiral augers fitted with a Tungsten Carbide ('TC') bit.

The borehole locations are shown on the attached Figure 1 and these were set out by taped measurements from features shown on the supplied survey plan by RPS Australia East Pty Ltd (Project No PR127602-4, Drawing No PR127602-DET-A.dwg, dated 28 May 2015). The reduced levels of the ground at each borehole location were interpolated from spot levels shown on this plan. The survey datum is Australian Height Datum (AHD). A graphical borehole summary is provided in the attached Figure 2.

The strength of the subsurface soils and compaction of the fill were assessed from Standard Penetration Test (SPT) 'N' values and from hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. The strength of the underlying bedrock was assessed from the penetration resistance when auger drilling using a ('TC') bit, by examination of the recovered rock cuttings, and from correlations with subsequent moisture content tests of recovered rock chips. The moisture content test results are presented in the attached Table A. We note that rock strengths estimated in this way are indicative only and variations of at least one strength order should not be unexpected.

Groundwater observations were made in the boreholes during auger drilling and on completion. Boreholes were left open for up to 3 hours after completion to allow for short term groundwater monitoring. No longer term monitoring of groundwater levels was carried out.

Our Geotechnical Engineer (Rachael Price) was present on a full-time basis during the fieldwork to set out borehole locations, direct the electromagnetic scanning, nominate the testing and sampling, and prepare the borehole logs of the strata encountered. The borehole logs are attached to this report together with a set of explanatory notes, which describe the investigation techniques, their limitations and define the logging terms and symbols used.

Selected soil and rock chip samples were returned to a NATA registered laboratory, Soil Test Services Pty Ltd (STS), for moisture content, Atterberg Limits and linear shrinkage testing. The results are summarised in the attached STS Table A.

Selected soil and rock samples were also sent to Envirolab Services Pty Ltd, an external analytical laboratory, for soil pH, chloride content, sulphate content and resistivity testing. The results are presented in the attached Envirolab Certificate of Analysis (Ref: 128641).

Preliminary waste classification testing of the site soils was carried out by Environmental Investigation Services (EIS) in conjunction with this geotechnical investigation. Reference should be made to the EIS report (Ref: E28412K).

3 RESULTS OF INVESTIGATION

3.1 Site Description

This site description was prepared at the time of our July 2015 report. We have not carried out any additional site inspections and have assumed that the site is similar to that encountered in July 2015.

The site lies within an area of gently undulating regional topography. The site has an overall fall down to the south, with a maximum elevation of about RL23.6m at the northern end and about RL19.3m at the south-eastern corner. Average slopes across the site are about 3° down to the south/south-east. The site is accessible from Carlton Crescent which bounds the site to the north, and driveway access from Hardie Street in the south-east corner. Both roads are paved with asphaltic concrete and are kerb and guttered; they showed no evidence of significant cracking or distress.

The site comprises two adjoining two-storey rendered brick and concrete warehouses. At the time of fieldwork, both warehouses were operational and were used as a storage facility for ambulance vehicles. The buildings are stepped, with the upper slab level at the Carlton Crescent level. It appears that at least some cut to fill earthworks were carried out to construct the existing buildings.

The eastern most warehouse has a lower slab level which does not extend the full length of the site. At its northern end there is a concrete block wall and between this wall and the northern property boundary there is an undercroft void below the upper level slab. The ground level in this void slopes up toward the northern site boundary.

The western most warehouse has offices in the north-western corner at the upper slab level. Over the eastern side of this western most warehouse there is a concrete ramp which slopes down to the lower slab level.

A concrete driveway runs along the western site boundary and provides access from Carlton Crescent to the rear of the property where a small staff carpark area is located. A number of fuel storage tanks appear to be buried below this carpark with bowsers located nearby. All pavements on site are concrete and appeared to be in moderate to good condition with some lateral cracking (2 to 3 mm wide) observed. The concrete footpath adjacent to Carlton Crescent contained one small area of settlement and some lateral cracks. Vegetation on the site comprised one large tree (about 15m height) on the western site boundary and a small manicured garden to the north of the building.

There are three neighbouring properties which surrounded the site as follows, the existing ground surface level between the subject site and the adjacent properties appeared to be similar.

- The property to the west contains a skate park and tennis court which abut the subject site boundary, both of which appeared to be in good condition.
- The property to the south contained a single storey rendered café which appeared in good condition based on a cursory inspection from the subject site. This café is about 4m from the subject site boundary.
- The property to the east contains a two to three storey brick and rendered warehouse extending up to the common boundary. This property also appeared to be cut into the hillside with the first floor accessible from Carlton Crescent. This adjoining warehouse appeared to be in moderate to poor condition with large horizontal and vertical cracks observed in the rendering and spalling above the window frames.

A railway line is located about 20m to the north of the site, running parallel to Carlton Crescent. The railway tracks appear to be approximately three metres above the road height and are retained by a brick retaining wall. This wall appeared to be in good condition with no observed cracking.

3.2 Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 indicates that the site is underlain by Ashfield Shale which comprises 'black to dark grey shale and laminite'. The boreholes generally revealed a subsurface profile comprising gravelly and clayey fill layers overlying residual silty clay which graded into weathered shale bedrock. A summary of the encountered subsurface features are presented below. For a more detailed description of materials encountered, reference should be made to the attached borehole logs.

Pavements

Concrete pavements were encountered in all boreholes and were measured to range from 130mm to 170mm thick in BH1 to BH4. In BH5 a 200mm suspended concrete slab was encountered, at which point diatube coring was terminated to avoid penetrating further into the under croft void. Steel reinforcement was observed in the recovered cores and is noted on the borehole logs. The concrete pavements were underlain by a gravelly sub-base in BH1 and BH2 and a silty sand sub-base in BH4. The subbase layers extended to a depth of 0.3m at each of these locations. In BH3, the slab was directly overlying a silty clay fill.

Fill

Fill was encountered in BH2, BH3 and BH4 to a maximum depth of 1.5m (BH3 and BH4). The fill generally comprised silty clay ranging from low to high plasticity and was assessed as poorly compacted in BH2 and BH4 and well compacted in BH3. We note that the fill in BH3 was possibly a natural soil, but it was difficult to determine in the small diameter borehole. The fill contained varying amounts of sandstone, ironstone and igneous gravel, ash and glass fragments. In BH1, no fill was encountered (apart from the subbase layer). We note that localised deeper areas of fill may be encountered within the southern area of the site due to the possible presence of buried fuel tanks.

Residual Silty Clay

Residual silty clay was encountered below the fill in all boreholes, and extended to depths ranging from 1.0m (BH1) to 4.2m (BH2). The silty clay was assessed to be of high plasticity and stiff to hard strength. The silty clay contained minor amounts of fine to course grained igneous gravel.

Weathered Shale Bedrock

Weathered shale bedrock was encountered below the residual silty clay at depths ranging from 1.0m (BH1) to 4.2m (BH2). In BH1 and BH2, the shale was found to be extremely low to very low in strength on initial contact, before improving to at least medium strength at a depth of 1.7m

(BH1) and 4.9m (BH2). In BH1 the shale was then assessed to reduce to low strength at a depth of 3.5m before increasing to low/medium strength at 3.9m depth.

In BH3 and BH4, the shale was initially of medium to high strength on first contact, and continued to be at least medium strength for the full depth of the borehole. A 200mm thick very low to low strength band was encountered in BH3 at 4.3m depth.

Groundwater

Groundwater seepage and free-standing groundwater was encountered in all boreholes (except BH5) at varying depths during auger drilling. The measured seepage and groundwater levels below existing ground surface level are presented in the table below.

Measured Seepage and Groundwater Levels Below Existing Ground Surface

Borehole	Seepage	Ground Water Levels
BH1	5.8m	4.2m on completion of drilling. 1.2m after 3 hours
BH2	5.8m	3.8m on completion of drilling. 3.0m after 1.5 hours
BH3	5.0m	5.0m on completion of drilling. 3.4m after 1 hour
BH4	Not observed	4.6m on completion of drilling. 3.8m after 1.5 hours

We note that the groundwater levels may not have stabilised within the limited observation period. No long-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The moisture content test results on selected rock samples showed a reasonable correlation with our field assessments of rock strength. The Atterberg Limits and Linear Shrinkage testing on residual silty clay samples indicate the silty clay is of high plasticity and therefore has a high potential for shrink-swell movement with changes in moisture content. The results of the moisture content, Atterberg Limits and Linear Shrinkage testing are presented in the attached STS Table A.

Selected samples (1No. weathered shale and 2No. residual silty clay) were returned to Envirolab Services Pty Ltd for soil pH, chloride content, sulphate content and resistivity testing. The testing results showed that the selected samples had pH values between 5.9 to 7.3 indicating slightly to moderately acidic conditions, and low sulphate and chloride contents. The resistivity tests returned results ranging from 49 to 110 ohm.m which is also low.

4 PRELMINARY COMMENTS AND RECOMMENDATIONS

The comments and recommendations provided below are generalised and of a preliminary nature, they will need to be reviewed and most likely supplemented once the architectural and structural designs have been finalised.

4.1 Site Classification

We consider that the site will classify as Class ‘P’ in accordance with AS2870-2011 ‘Residential Slabs and Footings’ due to the following factors:

- The presence of relatively deep uncontrolled (poorly compacted) fill in BH2, BH3 and BH4 to depths of 1.2m to 1.5m.
- Abnormal soil moisture conditions due to the presence of pavements, and existing buildings.

Where footings are to be founded within the underlying natural residual silty clays, we expect characteristic surface movements to be in the range usually expected for a Class ‘H1’ site. This classification must be confirmed following detailing of the proposed development.

4.2 Dilapidation Report

Prior to the commencement of site works, including demolition, we recommend that a dilapidation report be completed on the neighbouring property to the east. The dilapidation report provides a record of existing conditions prior to commencement of the works. The adjoining property owner to the east should be asked to confirm that the report presents a fair record of existing conditions. These reports can also be used for assessment of potential damage claims, but must be carried out thoroughly with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc. The dilapidation report should be carefully reviewed by the geotechnical and structural engineers prior to commencement of works; in particular the size/energy of rock hammers, if they are to be used, should be reassessed. The preparation of such reports will help guard against opportunistic claims for damage that was present prior to the start of excavations.

4.3 Demolition

It is possible that existing structural elements (such as boundary walls) may be acting as retaining walls to the adjoining site to the east and the Carlton Crescent footpath to the north. Therefore all demolition works should be carried out with care, so as to not destabilise or undermine any adjoining structures or footpaths. This work will need to be carried out by suitably experienced (and insured) contractors.

Demolition of concrete slabs, footings and paved surfaces will be required. We recommend that saw cut slots be provided near adjoining buildings and use be made of buckets fitted to hydraulic excavators to lift out pieces, so as to reduce the risk of demolition vibrations being transferred to adjoining buildings.

We consider that during demolition works it would be prudent to undertake at least some quantitative vibration monitoring to confirm that vibrations from demolition activities are within tolerable limits.

It may also be necessary to remove subsurface structures such as fuel storage tanks, and these should be removed in accordance with the excavation recommendations in Section 4.4 below.

4.4 Excavation Conditions

Excavation recommendations provided in this report should be complemented by reference to the Safe Work Australia 'Excavation Work Code of Practice' dated March 2015. We have assumed that excavations for the proposed new ambulance station will extend to a maximum depth of 3m. This would allow for about 1 level of excavation below Carlton Crescent.

Excavation through the fill, clay and extremely low to very low strength shale should be readily achievable using conventional earth moving equipment, such as the buckets of hydraulic excavators. We note that deeper localised areas of fill may be encountered within the southern area of the site due to the possible presence of buried fuel tanks. Removal of the buried fuel tanks may be required during demolition or bulk excavation works, and therefore only suitably qualified contractors should be used for that specialised service.

It is possible that deeper areas of excavation may encounter the underlying medium to high or high strength shale bedrock. Excavation of the medium to high strength shale bedrock will require 'hard rock' excavation techniques, such as percussive techniques (comprising the use of rock hammers) or non-percussive techniques (comprising rotary grinders, rock saws, ripping tynes etc).

Where percussive excavation techniques are adopted there is the risk that transmitted vibrations may damage nearby structures. Therefore at the commencement of the use of percussive excavation some quantitative vibration monitoring should be carried out by the geotechnical engineers to provide guidance to the excavation contractor on the suitability of their adopted equipment and techniques. Where vibrations are found to be exceeding tolerable levels then it will be necessary to either reduce the size of the equipment being used or to adopt non percussive methods.

Where non percussive excavation techniques such as rock saws, ripping tynes, rotary grinders etc are to be used, then quantitative vibration monitoring would not be required, however care should still be taken, particularly when using ripping tynes not to dislodge wedges or blocks of rock from beyond the site boundaries as this could lead to damage to adjoining structures or services.

Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Groundwater seepage was encountered in the boreholes during drilling and at completion of drilling. Therefore seepage may be encountered within any bulk excavations across the site. Seepage is expected to occur across the surface of the underlying bedrock and through and joints or defects within the rock, particularly during and following periods of wet weather. We expect that any seepage will be able to be controlled using conventional sump and pump techniques.

4.4.1 Temporary and Permanent Batters

We recommend the following temporary batter slopes be adopted through the clayey fill, residual silty clays and any upper extremely low or very low strength shales:

- 1 Vertical in 1.5 Horizontal (1V:1.5H) through the clayey fill.
- 1V in 1H through residual silty clays of at least stiff strength and weathered shale bedrock.

We consider that the above temporary batters will remain stable in the short term provided all surcharge loads (such as traffic loads, construction loads etc) are kept well clear of the crest of the batters (at least twice the height of the batter slope from the crest).

All stormwater runoff should be directed away from all temporary batters to reduce the risk of erosion.

Excavation of any medium to high strength shale, can be provisionally cut vertically subject to geotechnical inspection following not more than each 1.5m vertical lift of excavation. The inspection should be carried out by an experienced geotechnical engineer to assess whether any stabilisation measures (eg. rockbolts, dowels, shotcrete, etc.) are required.

Permanent batters in the soils and weathered shale should be no steeper than 1V in 2H, and these should be protected against erosion such as by the planting of a deep rooted runner grass, or by the application of a shotcrete facing. The shale is likely to be suitable to stand vertically subject to geotechnical inspection, but may require a shotcrete facing to protect against long term deterioration and fretting of the face.

Where these batters cannot be accommodated within the site boundary, or are not preferred, then a shoring system should be installed prior to the commencement of excavation.

4.4.2 Excavation Adjacent the Eastern Site Boundary

If excavation is required up to the eastern site boundary, then it may be to a level which is below adjacent building footings. We recommend that the details of the adjoining building footings be sourced and investigated prior to any bulk excavation adjacent to this eastern boundary. Investigation of the adjoining building footings could comprise a series of test pits to expose the nature of the adjoining footings, including their type, founding depth and founding stratum. The test pits should be inspected by the geotechnical and structural engineers so that they can assess the impact that the excavation will have on the stability of the adjoining building footings. Adjoining footings may need to be underpinned prior to bulk excavation or alternatively a properly designed insitu shoring system installed along the boundary to support the adjoining ground and footing loads

4.5 <u>Shoring and Retaining Wall Design</u>

4.5.1 Cantilevered Retaining Walls

Where retaining walls are formed in front of temporary batters, it would be feasible to construct simple cantilevered walls, with these walls being backfilled with engineered fill.

Cantilevered walls of up to 3m high may be designed based on the following geotechnical parameters:

- Adoption of a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.35 for soils and extremely weathered shale and a K_a of 0.15 for the medium to high strength shale.
- Where the walls are restrained from lateral movement, such as by other structural elements in front of the wall, or where ground movements are to be reduced, an 'at rest' earth pressure coefficient, K_o, of 0.6 should be used.
- A bulk unit weight of 19kN/m^3 for the clayey fill and residual silty clays.
- A bulk unit weight of 22kN/m^3 for the shales.
- All surcharge loads, i.e. traffic loads, adjacent structures, etc. should be allowed for in the design.
- Full hydrostatic pressures should be allowed for in the design unless measures are taken to provide complete and permanent drainage behind the walls.
- We recommend that behind wall drainage be incorporated in the design of the walls.

Where retaining walls are constructed within temporary batter slopes, there must be careful attention to backfill material and procedures in order to reduce post construction settlements. Compaction of engineered fill behind retaining walls is difficult and time consuming to carry out effectively, and must be done with light plant and thin layers to avoid excessive compaction stresses on the retaining wall. Where the backfill is to support hard landscaping such as paving, the backfill should be compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD), and at a moisture content within 2% of its Standard Optimum Moisture Content (SOMC). Where there will only be soft landscaping above, and minor settlements can be tolerated, the compaction specification may be reduced to at least 95% of SMDD.

It is likely to be preferable to use a single-sized, durable, free-draining gravel, such as crushed igneous rock (blue metal), or crushed concrete as backfill material behind new walls. These materials do not require significant compactive effort, and provide good long term performance with regard to settlement. A slotted agricultural pipe should be positioned to collect any seepage at the base of the backfill and dispose it to the stormwater system. Where free-draining backfill is adopted, the upper 0.3m of fill should comprise compacted clayey soil to limit the flow of surface water into the permeable wall backfill. A layer of non-woven filter geotextile should be placed between the clay and free-draining material to prevent migration of the fines which could block the drainage medium.

4.5.2 Soldier Pile Walls

Where temporary batters cannot be accommodated within the site boundaries, or where they are not preferred, a soldier pile wall should be installed prior to the commencement of excavation, with reinforced shotcrete infill panels sprayed between the soldier panels progressively during the excavation following each 1.5m vertical lift of excavation. Where the excavation is less than 3m depth, it is likely to be possible to use cantilevered soldier piles with a deep toe socket into rock. These walls should be designed using the triangular lateral earth pressure and coefficients provided above with a passive lateral earth pressure coefficient (K_p) of 3.0 in the residual clays or weathered shale of extremely or very low strength, with a factor of safety of at least 2. Where soldier piles are spaced more than 3 diameters apart, the passive pressure adjacent to the toe must be appropriately reduced (such that the passive pressure per metre run does not exceed $3K_pd/S$ where d is the pile diameter and S is the pile spacing). Where the piles are socketed into shale of at least medium strength, they may be designed for an allowable lateral resistance of 300kPa. We note that socketing into the shale bedrock of medium or high strength will require large piling rigs with rock drilling equipment and even then productivity may be slow and bit wear may be high.

We note however that a cantilevered pile wall can be quite flexible, and if there are any movement sensitive services or infrastructure behind the walls, it may be necessary to use anchored pile walls to reduce these movements. Anchored pile walls would also reduce the embedment length at the toes of the piles. Where anchored pile walls are adopted, they should be designed for a trapezoidal lateral earth pressure distribution of 6H kPa, where H is the height of retained material in metres. Where there are movement sensitive services or infrastructure behind these shoring walls, the pressure magnitude should be increased to 8H kPa. These maximum pressures should apply over the central 60% of the height of the shoring, tapering to zero at the crest and toe. Where temporary anchors extend beyond the site boundary, it will be necessary to obtain permission from the owners of the adjacent properties prior to the installation of these anchors. The anchors should be designed with minimum free and bond lengths of at least 3m each, and an allowable bond of 60kPa may be used in the extremely and very low strength shale, with an increased bond of 200kPa in the shale of at least medium strength.

Where soldier pile walls are adopted, it is likely that bored piles will need to be either tremie poured or installed as grout injected (CFA) piles due to the seepage encountered in the investigation boreholes.

4.5.3 Soil Nail Walls

A third option for supporting a proposed excavation would be to use soil nail walls. Further geotechnical design would be required to provide a suitable soil nail design, though it is likely that the design would require soil nails (rock bolts) of about 4.5m length on a grid with 1.5m spacing both vertically and laterally. The construction procedure would be to excavate down about 1.0m, install the upper row of soil nails and reinforced shotcrete, and then undertake subsequent lifts of excavation, soil nail installation and shotcrete spraying one lift at a time to the bulk excavation level.

4.6 Footings

Depending on the details of the proposed structures, suitable footings may comprise either shallow (pad or strip) or bored piled footings. The existing fill is not considered to be a suitable bearing stratum, and therefore the proposed structures will need to be founded within either the residual silty clays or underlying shale bedrock. Where any part of the structure is founded within the bedrock, then for uniformity of support, and to reduce the risk of adverse differential movements, all footings should be founded within the bedrock.

Footings founded within the residual silty clays of at least stiff strength may be designed based on an Allowable Bearing Pressure (ABP) of 100kPa. Reference should also be made to Section 4.1 for details of the site classification for this site.

Where higher bearing pressures are required, footings may be founded on the underlying shale bedrock. Footings founded on and with a minimum embedment of 0.3m into shale bedrock of at least extremely low strength may be designed based on an ABP of 600kPa, while footings founded on and embedded at least 0.3m into at least medium strength shale may be designed for an ABP of 1000kPa. A combination of shallow and bored piled footings may be required to achieve the shale bedrock, as the bedrock appears to be dipping down to the south.

We note that in some places the initial shale bedrock will be of medium to high strength. Therefore, allowance should be made for large capacity piling rigs to penetrate the medium shale bedrock. If loads are such that long sockets in the medium to high strength shale are required, we recommend further investigations using cored boreholes to optimise bearing pressures and reduce socket lengths.

Groundwater seepage was encountered in the augered boreholes, therefore depending on pile depths bored piles may need to be either tremie poured or installed with auger grout injected (CFA) piling techniques.

4.6.1 Geotechnical Inspection of Footings

Footing excavations and pile drilling should be inspected by a geotechnical engineer or engineering geologist to assess that a competent bearing stratum has been achieved. The base of footings and piles should be free of loose or softened material and any free-standing water must be removed prior to concrete pouring. Where water has been allowed to pond at the base of footings or piles, the water should be removed and the footing re-excavated to remove any softened material. Where water cannot be removed from the base of pile excavations, concrete should be poured using tremie techniques. We note that the clays and extremely weathered shale are susceptible to softening upon exposure to water, and therefore pouring should be carried out without delay. Where delays are expected, a concrete blinding layer should be provided to protect the base of shallow footings from softening, while piles will need to be redrilled to remove softened material.

4.7 Earthworks

Earthworks recommendations below should be complemented by reference to AS 3798-2007 ‘Guidelines on earthworks for commercial and residential development’.

4.7.1 Subgrade Preparation

Depending on the detailed design of the proposed development, the subgrade at bulk excavation level could comprise a combination of clayey fill, natural silty clays or weathered shale bedrock.

Following excavation to the required subgrade levels, the subgrade should be proof rolled with at least eight passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes dead weight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer. The purpose of proof rolling is to improve the near surface density of the soils, and to identify any soft or heaving areas.

Any soft or heaving areas should be excavated to a sound base and replaced with engineered fill. If the subgrade is left exposed and the clays exhibit shrinkage cracking, then the surface should be lightly watered and rolled until the shrinkage cracks are no longer evident. If the clayey subsoil is exposed to prolonged periods of rainfall, softening will result and site trafficability will be poor.

The fill encountered over the southern end of the site (BH2 and BH4) was found to be poorly compacted and therefore following excavation, if the existing fill is exposed at subgrade level then it will require either subgrade stabilisation works or removal of the poorly compacted fill in order to provide suitable support to floor slabs and/or pavements. Subgrade stabilisation works may include over-excavation and placement of a granular bridging layer.

4.7.2 Engineered Fill

Engineered fill should preferably comprise well graded granular materials, such as crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). While not ideal the existing clayey soils on site could be used as engineered fill, however where adopted, it must be compacted between 98% and 102% of SMDD and within 2% of Standard Optimum Moisture Content (SOMC). This may require drying of the clayey soils prior to them being able to be re-used. 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.

Density tests should be carried out at frequencies of not less than 1 test/500m^2/layer or 3 tests per visit of the geotechnician, whichever is the greater. Where backfilling is completed in confined conditions, such as behind retaining walls and service trenches, the frequency of testing should be increased to 1 test/50m^2/2layers. As a minimum Level 2 testing of earthworks should be carried out in accordance with AS3798, though if the fill is to be supporting footings, the testing should be in accordance with Level 1 testing. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.8 <u>Soil Aggression</u>

The soils have been found to have very low sulphate and chloride contents, low resistivity and to be slightly to moderately acidic. In accordance with the criteria for concrete and steel piling exposure classification given in Table 6.4.2(C) and Table 6.5.2(C) of AS2159-2009 ‘Piling Design and Installation’, buried concrete and steel structures should be designed based on ‘non-aggressive’ exposure conditions. In accordance with the AS 2159-2009 ‘Piling – Design and

Installation', these values are indicative of a 'non-aggressive' exposure classification for buried concrete structures and steel structures.

4.9 Further Geotechnical Input

The following further geotechnical input has been recommended in the above sections:

- Review of this report and provision of additional development specific advice once the architectural and structural designs have been finalised.
- Quantitative vibration monitoring where percussive excavation techniques are adopted.
- Excavation of test pits adjacent the neighbouring building to the east to expose the footings.
- Inspection of vertically cut batters through the shale bedrock
- Geotechnical inspection of at least the initial stages of footing excavation by a geotechnical engineer to confirm that the design founding stratum is being achieved.
- Proof rolling of the clayey subgrade in the presence of an experienced geotechnical engineer or geotechnician.
- Density testing of all engineered fill at a frequency of 1 test/500m^2/layer or 3 tests/visit or 1 test/50m^2/2 layers where fill is placed in confined conditions.

5 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.

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

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Redevelopment
Location: 74 & 75 Carlton Crescent, Summer Hill, NSW

Ref No: 28412L
Report: A
Report Date: 1/06/2015
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	0.50-0.95	24.6	58.0	20.0	38.0	15.0
1	2.80-3.00	6.5				
1	4.20-4.50	7.2				
1	5.70-6.00	10.8				
2	4.50-4.80	13.6				
2	5.70-6.00	11.0				
3	4.20-4.50	16.0				
3	5.70-6.00	9.5				
4	1.50-1.95	20.8	60.0	22.0	38.0	15.5
4	2.70-3.00	11.4				
4	4.20-4.50	9.3				
4	5.70-6.00	10.8				

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: 27/05/2015

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

CERTIFICATE OF ANALYSIS 128641

Client:

Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: Rachael Price

Sample log in details:

Your Reference: **284112L, Summer Hill**
No. of samples: 3 Soils
Date samples received / completed instructions received 27/05/15 / 27/05/15

Analysis Details:

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

Report Details:

Date results requested by: / Issue Date: 3/06/15 / 1/06/15
Date of Preliminary Report:
NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Jacinta Hurst
Laboratory Manager

Misc Inorg - Soil				
Our Reference:	UNITS	128641-1	128641-2	128641-3
Your Reference	-------------	BH1	BH2	BH3
Depth	------------	1.5-1.62	3.0-3.45	1.5-1.95
Date Sampled		26/05/2015	27/05/2015	28/05/2015
Type of sample		Soil	Soil	Soil
Date prepared	-	28/05/2015	28/05/2015	28/05/2015
Date analysed	-	28/05/2015	28/05/2015	28/05/2015
pH 1:5 soil:water	pH Units	5.9	7.3	6.1
Chloride, Cl 1:5 soil:water	mg/kg	20	10	130
Sulphate, SO4 1:5 soil:water	mg/kg	110	47	180
Resistivity in soil*	ohm m	110	120	49

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-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B.
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.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			28/05/2015	[NT]	[NT]	LCS-1	28/05/2015
Date analysed	-			28/05/2015	[NT]	[NT]	LCS-1	28/05/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	112%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	108%
Resistivity in soil*	ohm m	1	Inorg-002	<1.0	[NT]	[NT]	LCS-1	97%

Report Comments:

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

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

Quality Control Definitions

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

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-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.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: IGLU No. 210 PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: 74 & 75 CARLTON CRESCENT, SUMMER HILL, NSW

Job No. 28412L
Date: 26-5-15
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 21.2m
Datum: AHD
Logged/Checked by: R.A.P./D.W.

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
						0			CONCRETE: 170mm.t				5mm DIA. REINFORCEMENT, 30mm,60mm,130mm & 140mm TOP COVER
	■							-	FILL: Silty gravel, medium to coarse grained sub angular to angular igneous, dark grey.	M			
	■			■	N = 14 3,7,7			CH	SILTY CLAY: high plasticity, light grey mottled red brown.	MC>PL	H	550 570 550	RESIDUAL
▼ AFTER 3 HRS				■	N > 19 13,6/20mm REFUSAL	1		-	SHALE: light grey.	XW	EL		VERY LOW TO LOW 'TC' BIT RESISANCE
				■ ■		2 3			SHALE: dark grey, with light grey, fine grained sandstone laminae.	DW	M-H		MODERATE RESISTANCE
									SHALE: dark grey, with occasional M strength iron indurated bands.		L		LOW RESISTANCE
▼ ON COMPLET-ION ▶				■ ■		4 5					L-M		LOW TO MODERATE RESISTANCE
						6 7			END OF BOREHOLE AT 6.0m				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: IGLU No. 210 PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: 74 & 75 CARLTON CRESCENT, SUMMER HILL, NSW

Job No. 28412L
Date: 26-5-15
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 20.6m
Datum: AHD
Logged/Checked by: R.A.P./D.W.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 130mm.t				7mm DIA. REINFORCEMENT, 70mm TOP COVER
			-	FILL: Silty clayey gravel, dark grey, medium to coarse grained sub angular.	M			
	N = 4 2,2,2	1		FILL: Silty clay, low plasticity, dark brown, trace of fine to medium grained sandstone and igneous gravel and slag.	MC>PL			APPEARS POORLY COMPACTED
	N = 7 2,3,4	2	CH	SILTY CLAY: high plasticity, light grey mottled yellow brown and red brown, trace of of fine to coarse grained ironstone gravel.	MC>PL	St	150 160 140	RESIDUAL
AFTER 1.5 HRS	N = 16 7,7,9	3				St-VSt	160 200 280	
ON COMPLETION		4						
			-	SHALE: light grey, with dark grey laminae.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
	N > 22 7,15/0mm REFUSAL			SHALE: dark grey.	DW	VL		
		5				M-H		MODERATE TO HIGH RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: IGLU No. 210 PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: 74 & 75 CARLTON CRESCENT, SUMMER HILL, NSW

Job No. 28412L
Date: 26-5-15
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 20.7m
Datum: AHD
Logged/Checked by: R.A.P./D.W.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 140mm.t				5mm DIA. REINFORCEMENT, 80mm TOP COVER
			-	FILL: Silty clay, low plasticity, brown, with fine to medium grained igneous gravel.	MC>PL			APPEARS WELL COMPACTED
	N = 15 6,6,9	1		FILL: Silty clay, high plasticity, brown, trace of fine grained ironstone gravel.	MC<PL		>600 >600 >600	POSSIBLY NATURAL
	N = 12 7,6,6	2	CH	SILTY CLAY: high plasticity, light grey mottled orange brown, trace of fine to coarse grained ironstone gravel.	MC<PL	H	>600 >600 510	RESIDUAL
AFTER 1 HR		3 4	-	SHALE: light grey, with fine grained sandstone laminae.	DW	M-H		MODERATE 'TC' BIT RESISTANCE
				as above, but light grey, and dark grey.		VL-L		LOW RESISTANCE
ON COMPLETION		5	-	SHALE: dark grey.		M		MODERATE RESISTANCE
		6 7		END OF BOREHOLE AT 6.0m				

SAMPLES: ES, U50, DB, DS

Graphic Log

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BOREHOLE LOG

Borehole No.
4
1/1

Client: IGLU No. 210 PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: 74 & 75 CARLTON CRESCENT, SUMMER HILL, NSW

Job No. 28412L
Date: 26-5-15
Method: SPIRAL AUGER JK205
R.L. Surface: ≈ 20.4m
Datum: AHD
Logged/Checked by: R.A.P./D.W.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
						0			CONCRETE: 170mm.t				10mm DIA. REINFORCEMENT 70mm TOP COVER, 5mm DIA. AT 70mm & 130mm TOP COVER
								-	FILL: Silty sand, fine to medium grained, with igneous gravel, trace of glass fragments.	M			
					N = 2 0,0,2	1			FILL: Silty clay, low plasticity, brown, trace of fine to coarse grained ironstone and shale gravel and ash.	MC>PL			APPEARS POORLY COMPACTED
					N = 5 2,2,3	2		CH	SILTY CLAY: high plasticity, light grey mottled orange brown and red brown, trace of fine to coarse grained ironstone gravel.	MC<PL	VSt	280 220 210	RESIDUAL
						3		-	SHALE: light grey, with occasional iron indurated bands.	DW	M-H		MODERATE 'TC' BIT RESISTANCE
AFTER 1.5 HRS ON COMPLETION						4 5			as above, but dark grey, and light grey.				
									SHALE: dark grey, with occasional iron indurated bands.				
						6 7			END OF BOREHOLE AT 6.0m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: IGLU No. 210 PTY LTD
Project: PROPOSED REDEVELOPMENT
Location: 74 & 75 CARLTON CRESCENT, SUMMER HILL, NSW

Job No. 28412L
Date: 26-5-15
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 23.6m
Datum: AHD
Logged/Checked by: R.A.P./D.W.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
						0			CONCRETE: 200mm.t				10mm DIA. REINFORCENENT, 40mm TOP COVER
									END OF BOREHOLE AT 0.2m				BOREHOLE TERMINATED DUE TO SUSPECTED VOID
						1							
						2							
						3							
						4							
						5							
						6							
						7							

Scale (m):

0 20

JK Geotechnics

GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

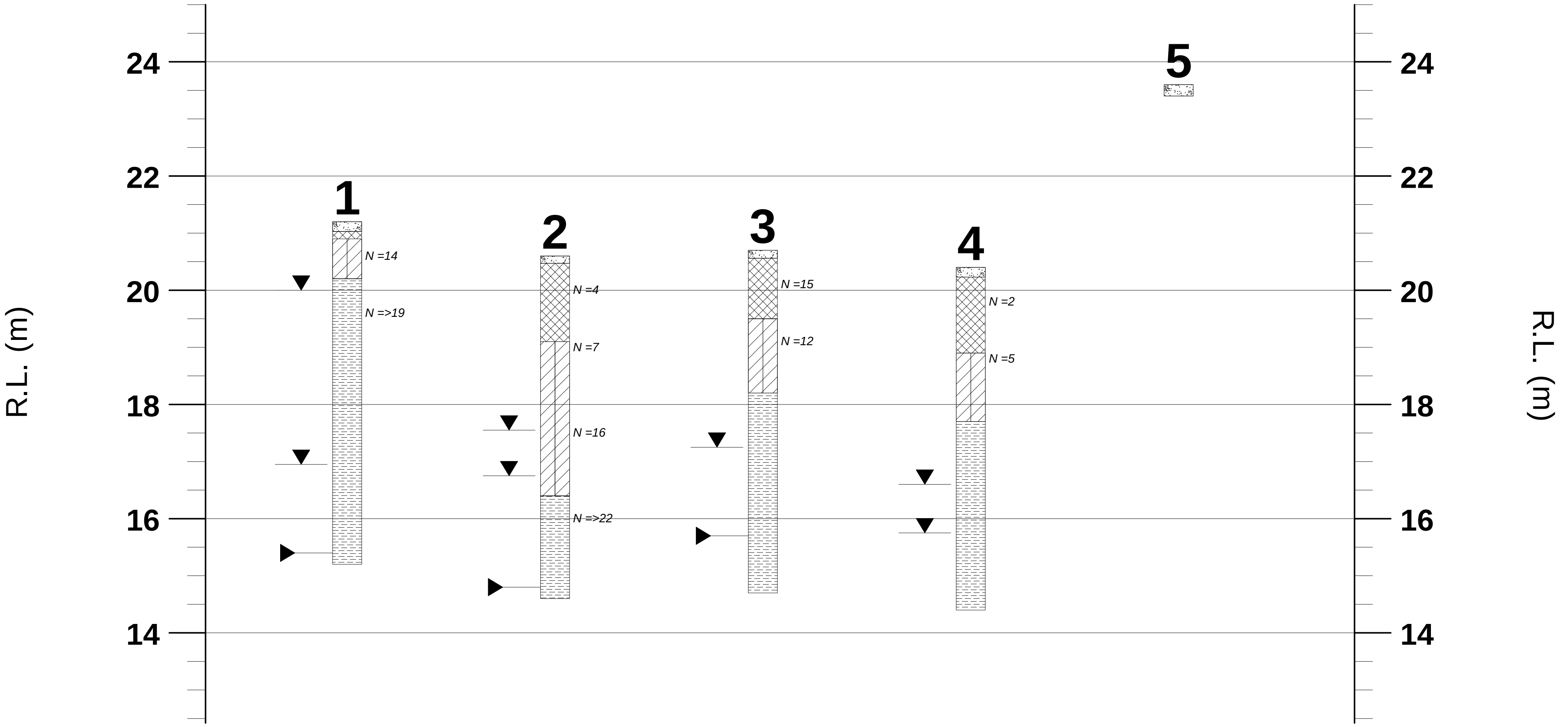
Title:

BOREHOLE LOCATION PLAN

Report Number: 28412L

Figure Number: 1

GRAPHICAL BOREHOLE SUMMARY



▨	Concrete	▤	Shale	N	SPT "N" VALUE
▩	Fill	▼	Observed water level	Nc	SOLID CONE BLOW COUNTS PER 150mm
▧	Silty Clay	▶	Groundwater seepage level		

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 28412L

Figure No.: 2

VIBRATION EMISSION DESIGN GOALS

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

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

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

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, “Methods of Testing Soils for Engineering Purposes” – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'* , published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

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

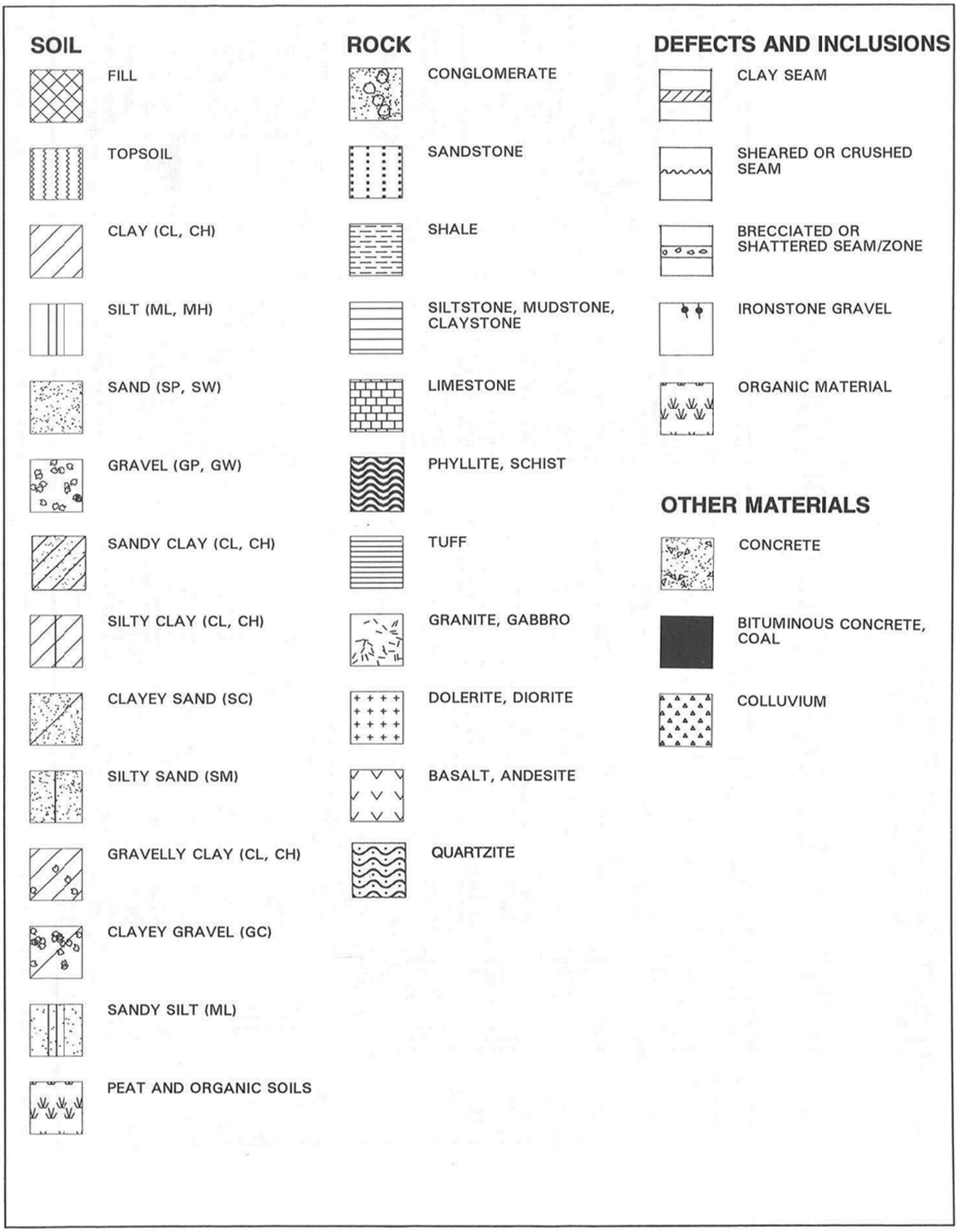
SITE INSPECTION

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

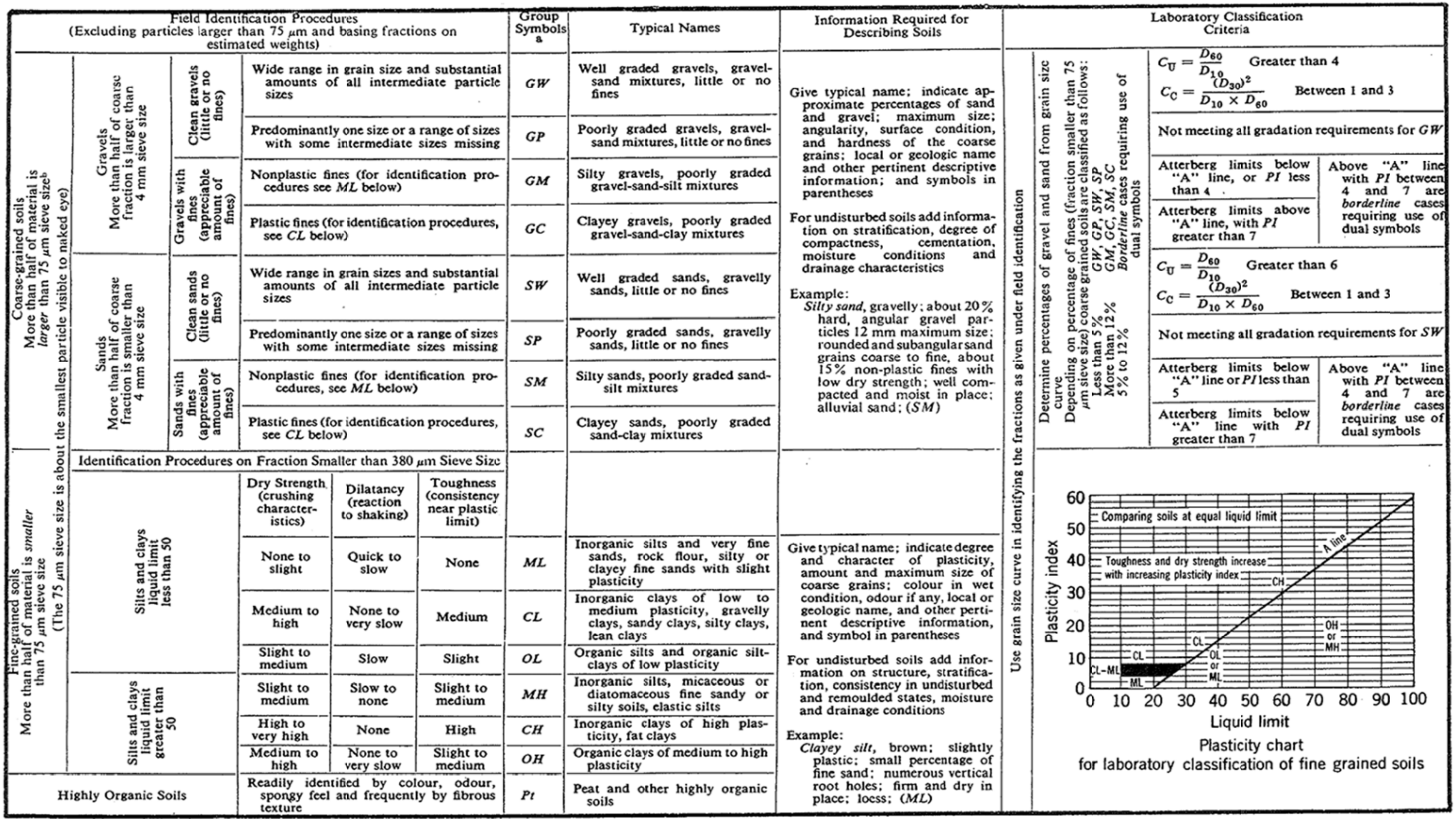
Requirements could range from:

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



UNIFIED SOIL CLASSIFICATION TABLE



Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirements for *GW*	
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures				Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures				Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines				$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirements for *SW*	
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line with *PI* greater than 7	
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size											
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		Plasticity chart for laboratory classification of fine grained soils		
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays					
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity					
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts					
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays					
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity					
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils					

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).

2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	—▼—	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶——	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil 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 screening.
	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. 'R' as noted below.
	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 degree 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 (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless 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 less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense >85 >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
----------------------	--------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
----------------------	--------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
----------------------	--------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
----------------------	--------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
----------------------	---------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
----------------------	---------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	