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REPORT

TO

SYDNEY ADVENTIST HOSPITAL LIMITED

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED STAGE 1 PHASE 1 EXPANSION WORKS

AT

THE SYDNEY ADVENTIST HOSPITAL
FOX VALLEY ROAD, WAHROONGA, NSW

8 July 2010

Ref: 22758Vrpt SAH Wahroonga

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TABLE OF CONTENTS

1 INTRODUCTION

Jeffery and Katauskas Pty Ltd have been commissioned by Mr Cameron Martin, Associate Director, of Morris Bray Architects, on behalf of the Sydney Adventist Hospital Limited, to carry out a geotechnical investigation for proposed Stage 1 Phase 1 expansion works at the Sydney Adventist Hospital, Wahroonga, NSW. The scope of the work was set out in proposal Ref.P32032V Wahroonga, dated 15 February 2010.

From the supplied Master Plan drawings (by Morris Bray Architects: Job No. 09006 Drwgs Nos. PT16-SK30 to SK45, dated March 2010) and the SCP Consulting Pty Ltd geotechnical brief (letter Ref. 2010-023-PT, dated 11 February 2010), we understand that Stage 1 Phase expansion works may comprise the following:

1. A new Clinical Services Building (CSB) expansion of up to eleven storeys, but with a potentially three additional storeys for future wards. A two stepped excavation into the site slope of about 3m deep for each step is currently proposed. This building is anticipated to have maximum column working load of 10,000kN.
2. New Fire Services and Oxygen Tank structures with loads of up to 2,000kN.
3. Two new temporary car parks.

The scope of the investigation was to obtain geotechnical information on subsurface conditions at 14 borehole locations nominated by SCP Consulting as a basis for comments and recommendations on the items specified in the above mentioned SCP briefing letter (with exception of salinity and acid sulphate which was to be covered by the environmental engineers either EIS or Coffey), including bearing capacity and shaft adhesion for footings, design parameters for retaining walls, batter slopes, site classification to AS2870, subgrade preparation, excavation conditions, excavation techniques, excavation support, and CBR for pavement thickness design. In preparation of this report we have also referred to previous work completed by

Jeffery and Katauskas at the Hospital, namely Report Ref. 22758Zrpt2 dated 22 April 2009.

A summary of the principal findings, issues and recommendations of the geotechnical investigation is provided in Section 4.1.

2 INVESTIGATION PROCEDURE

The investigation has included the following:

1. Drilling of four deep, cored boreholes (BHs) in the proposed area of the new CSB. BHs 101 to 104 were initially drilled using a spiral auger fitted with a tungsten carbide (TC) drill bit to depths between 2.75m and 4.3m below existing ground surface level. These boreholes were then extended using a NMLC triple tube core barrel with water flush to final borehole termination depths ranging between 5.79m and 7.24m.
2. Drilling of two deep, augered boreholes, BHs 107 and 111, to depths of 5m and 6m below existing ground surface, in the area of the proposed new Fire Services and Oxygen Tank structures.
3. Drilling of eight shallow, augered boreholes, BHs 105, 106 and 108 to 110, and 112 to 114, to depths in the range of 1.5m to 1.95m below existing ground surface levels, in the areas of the two proposed new temporary car parks.
4. All boreholes were drilled with our crawler mounted JK300 specialised geotechnical rig.
5. Prior to drilling, the proposed borehole locations were electromagnetically scanned for buried services, with reference to Dial Before you Dig Plans.
6. The nature and composition of the subsurface soils and rocks were assessed by logging the materials recovered during drilling.

7. The apparent strength/relative density of the subsoils were assessed from Standard Penetration Test (SPT) 'N' values, which were augmented, where possible, by hand penetrometer tests on cohesive samples recovered in the SPT split tube sampler. The strength of the augered sections of the 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 reference to the results of the moisture content tests. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

8. The strength of the cored bedrock was assessed by examination of the recovered core and subsequent correlation with laboratory Point Load Strength Index Tests (I_{s50}). Bedrock core samples were returned to Soil Test Services (STS), a NATA registered laboratory, where I_{s50} tests were completed on selected core samples and core photographs were taken. Copies of the photographs are presented with the borehole logs. Using established correlations, the approximate unconfined compressive strength (UCS) of the bedrock was interpreted from the I_{s50} results. The approximate UCS results and results of the I_{s50} tests are summarised in Table C and plotted on the relevant cored borehole logs.

9. Standard compaction and four day soaked CBR tests were also completed by STS on selected samples of the residual silty clays and the results are summarised in Table B. STS also completed moisture content, Atterberg Limits and Linear Shrinkage testing (for shrink/swell assessment); these results are summarised in Table A.

10. Groundwater observations were recorded during drilling and soon after completion of the augered stage of each of the boreholes. In the boreholes where the sandstone was core drilled, the use of water flush in the drilling process obscured further groundwater observations. No long term groundwater monitoring was carried out. However, we note that groundwater monitoring

well was installed into BH101, and the groundwater level was measured after a period of several days from completion drilling. We also completed a pump out test was completed on 7 June 2010 (refer to attached results sheet) to estimate the seepage rate for the borehole well geometry. In addition, we also measured the groundwater level in an active well installed in JK3 during our 2009 investigation.

11. Reference was made to relevant geotechnical data contained in our 2009 geotechnical investigation of adjoining areas (Report Ref. 22758Zrpt2 dated 22 April 2009).

The borehole locations, as shown on Figures 1A, 1B and 1C, were set out by taped measurements from existing surface features. The boreholes were drilled as close as practical to the SCP Consulting nominated locations, given the access constraints imposed by the existing site development. The surface reduced levels (RLs) at some of the borehole locations were estimated by interpolation between spot heights and ground contours indicated on the provided survey plans; however, there were a number of boreholes where the levels could not be approximated due to absence of survey data. The survey datum is the Australian Height Datum (AHD).

All fieldwork was carried out with our geotechnical engineers, Mr Li Yang and Mr Mark L Tsang, in full time attendance to set out the boreholes, direct the buried services scan, nominate the sampling and testing and compile logs of the substrata encountered. The borehole logs are attached to this report, together with a glossary of logging terms and symbols used.

For further details of the investigation procedures adopted, and their limitations, reference should be made to the Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The Seventh Adventist Hospital is located on the west side of Fox Valley Road, just to the north of the intersection with Comenarra Parkway, and lies within an undulating regional topographic setting. The majority of the hospital site is gently sloping with steeper slopes down towards Coups Creek beyond the carpark areas over the north.

The most of the site of the proposed Stage 1 Phase 1 expansion is located over the western portion of the hospital complex immediately to the west of the existing CSB and San Clinic and to the south-west of existing main Hospital building, as indicated on attached Figure 1A. However, there is also a temporary carpark proposed in the north-eastern corner of the site as shown on Figure 1A. All these existing buildings were multi storey frame structures built into the hillside slope; however, the community centre was a one and two storey building.

To the south of the existing main Hospital building, where it is propose to locate new Fire Services structures and new Oxygen Tanks, the ground surface was covered by grass and had a slope of about 5^0-6^0 towards Fox Valley Road. Further to the south-west in the area proposed to be covered by a temporary carpark, the ground surface was also partly covered by grass and partly by a playground, with a slope of about 5^0 to the south. There were single storey houses located further to the south.

Access to the site was via the main entrance off Fox Valley Road. A main asphaltic concrete (AC) paved internal road extended from the main entrance into the site in a north-west direction. A ring road branched off and up to the west at 2° past the Clinic Building, then south-east past the Tower Building and back down to the main entrance. A second road branched off to the west and down a driveway to the parking below the Clinic Building. The main internal road continued down at about 6° past the eastern end of the Clinic Building to a large open AC paved parking area to the west and to the east.

The western parking area was located beyond the CSB and Clinic Building and sloped down to the north-west and north-east at about 4° towards a central valley which in turn sloped down towards the north at 2°. This western parking had been terraced and the terraces separated by sloping islands with concrete kerbs along their perimeter. The eastern parking area was also AC paved and sloped down towards the north at approximately 3°.

Between the northern ends of the eastern and western parking areas was the Community Centre. A road extended in front of the Community Centre and provided access to a temporary AC surfaced carpark further to the east. Detention ponds were located to the east of the eastern parking area and to the south of the temporary parking area.

Beyond the northern extremities of the above parking areas and Community Building, the ground surface was grassed and sloped relatively steeply into densely vegetated bushland adjacent to Coups Creek. A detention pond was located over the upper zones of this slope at the western end.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney (Map 9130) indicates that the site is underlain by Hawkesbury Sandstone (which is described as typically consisting of medium to coarse grained quartz sandstone, very minor shale and laminate lenses) and close to the contact with the higher lying Ashfield Shale (which typically is described as consisting of black to dark grey shale and laminate). This subsurface profile does not take into account the soils derived from insitu weathering of the sandstone and shale or earthworks (eg. filling) that have previously been undertaken at the site.

The investigation has revealed a variable subsurface profile comprising fill over residual clays with sandstone and shale bedrock at relatively shallow depths, with sole exception of BH107 (new Fire Services area) where the shale had a deep cover of fill. For details of these variable subsurface conditions at specific locations reference should be made to the attached borehole logs. Graphical summaries of the borehole information are presented as Figures 2 and 3. Further comments on the subsurface profile are presented below:

1. **Fill** was encountered in ten of fourteen boreholes, generally extending to depths ranging from 0.2m to 0.8m, with the greater depth in the proposed north-eastern car park area; however, even deeper fill down to 2.1m was locally encountered in BH107 which was drilled in the area of the proposed Fire Services structure. The fill predominantly comprised silty clay of low to medium plasticity with gravel in places and roots near its surface. The fill was variably compacted but mostly poorly compacted, including the deep fill in BH107. This assessment is mostly based on SPT tests and our observations during drilling, which do not give a precise determination of in situ densities since they are affected by friction during driving/pushing (SPT only), the presence of gravel within the fill and the moisture content of the fill. Nonetheless, they provide a qualitative guide. In the four boreholes that did not encounter fill, instead a surficial layer of topsoil was encountered.
2. **Residual Silty Clay** of medium and high plasticity, and very stiff and hard strengths was encountered in the boreholes.
3. **Bedrock** was reached in almost all boreholes with exception for BH108 (proposed carpark area) at depths ranging from 0.4m to 1.5m in the boreholes with sole exception of BH107 (Fire Services area) where the shale was at 2.1m due to a deep cover of fill. Most of the bedrock was assessed to be

sandstone with exception of BHs 103, 107, 109 and 111, where shale was encountered. The upper zones of the bedrock was of poor quality (Class 5 or 4), being extremely to high weathered and of extremely low to low strengths. Since these rock strengths are approximate, mostly based on auger cuttings, we have only classified in the table below the bedrock (mostly sandstone) encountered in the deep cored boreholes drilled in the proposed CSB area. Using the methods proposed by Pells *et al* (1998), the following rock classifications and depths to have been assessed:

Cored BH No	Rock Type	Class 5 (m)	Class 4 (m)	Class 3 (m)	Class 2 (m)
101	Sandstone	0.4	0.8	2.3	n/a
102	Sandstone	1.5	1.5	2.0	n/a
103	Shale then Sandstone	1.2	4.5	5.1	5.1
104	Sandstone	1.1	3.1	3.8	4.2

4. **Groundwater** was not encountered during and after the auger drilling stage of the boreholes. In the boreholes where the sandstone was core drilled, the use of water flush in the drilling process obscured further groundwater observations. No long term groundwater monitoring was carried out. However, we note that groundwater monitoring well was installed into BH101, and the groundwater level was measured at depths of about 4.6m-4.7m (or around RL159.8) after a period of 5 days from completion drilling. We also completed a pump out test in BH101 was completed on 7 June 2010 to estimate the seepage rate for the borehole well geometry. The inflow rate was estimated at 0.6 litres per hour during the first half hour, reducing to 0.32 litres per hour after 1 hour (refer to attached results sheet). In addition, we also measured the groundwater level on 7 July in an active well in JK3, which had been installed during our 2009 investigation, at a depth of 5.2m or about RL156.4m.

3.3 Laboratory Test Results

The approximate UCS of the rock core, as estimated from the laboratory point load strength index test results, have correlated well with our field assessment of the rock strength. The estimated UCS varied from 4MPa to 50MPa (refer to Table C). The results of moisture content tests on selected auger (fragment) samples of the bedrock correlate reasonably well with the field strength assessments.

The two 4-day soaked CBR tests completed on silty clay subgrade samples from both carpark areas returned the same value of 2.5%. The samples were compacted to 98% of their respective Standard Maximum Dry Densities (SMDD) and to their respective Standard Optimum Moisture Contents (SOMC), prior to testing.

The results of the Atterberg Limit and Linear Shrinkage tests on the residual silty clays indicated these clays to have a moderate to high potential for shrink/swell movements with changes in moisture content.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings and Issues and Further Work

As discussed in more detail in Section 3.2, the boreholes penetrated fill down to significantly variable depths, underlain by very stiff to hard residual clays that grade into mostly sandstone and in some boreholes, into shale, at depths ranging from 0.4m to 1.5m in the boreholes with sole exception of BH107 (Fire Services area) where the shale was at 2.1m due to a deep cover of fill. Groundwater was measured in two borehole standpipe wells at depths of about 4.6m (or around RL159.8) and 5.2m or about RL156.4m.

The comments and recommendations which follow are based on the architectural drawings detailed in Section 1 above. Should there be any changes to these drawings or if additional drawings become available, this report should be reviewed and the comments and recommendations revised, if appropriate, to suit the specific site and updated development details.

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

1. The proposed building (CSB expansion) will have a two stepped excavation into the site slope of about 3m deep for each step and hence, we anticipate that substantial volumes of rock will be excavated in view that the bedrock is at relatively shallow depths. Good engineering design, construction and maintenance practices should be adopted to maintain stability to adjoining sites and structures during excavation and in the long term, as well as reducing the risk of vibration damage to adjoining structures during excavation.
2. Support of excavation sides must be provided by cutting at batter slopes or by installation of an anchored or propped retention system, installed prior to bulk excavation, with the shoring piles taken preferably down to below the base of the excavations.
3. Groundwater was measured in two borehole standpipe wells at depths of about 4.6m (or around RL159.8) and 5.2m or about RL156.4m. If the proposed stepped excavations are near to or below the latter levels then there will be groundwater seepage issues. The basement walls and floor slab will have to be designed to cater for groundwater pressures and seepage.
4. The proposed CSB expansion may have column loads of up to 10,000kN and hence, support of its footings on good quality sandstone (e.g. Class 3)

is imperative. Class 3 sandstone was assessed to occur at depths of 2m to 4.5m in the CSB area.

5. We are unaware of records that document the manner of placement, compaction specification and control of the variable fill of the site. Accordingly, we consider this existing material to be 'uncontrolled' fill. Because of this fill, the site is considered to be Class P in accordance with AS2870. The fill is deemed unsuitable as a bearing stratum for, at least, the footings and is considered a 'moderate to high risk' (of poor performance) as a supporting subgrade under ground floor slabs and pavements. Clearly, as a minimum, all upper fill that is root affected and of topsoil quality must be fully stripped and the remainder subjected to proof rolling.
6. The area of the proposed Fire Services, represented by BH107, has particularly poor foundation conditions comprising uncontrolled, poorly compacted fill to a depth of 2.1m. Hence, structures in this area should be fully suspended on piled footings taken into bedrock which was found at a depth of at least 2.1m into shale.
7. The residual silty clays have a moderate to high potential for shrink/swell with changes in moisture content; Class M to H in terms of AS2870. CBR testing of clay samples produced a low value of 2.5%, which essentially indicates the residual silty clay subgrades to be "poor" subgrades for the car park pavements. The use of thick pavements would be expected if the pavements are to be supported on the clay subgrade.

Although only a limited subsurface investigation was completed, we believe sufficient information has been gained to be reasonably confident as to subsurface conditions. However, it will be essential during excavation and construction works that regular geotechnical inspections be commissioned to check initial assumptions about excavation and foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for excavation, shoring and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections.

4.2 Proposed Clinical Services Building (CSB) Expansion

The proposed Clinical Services Building (CSB) expansion will be up to eleven storeys, but with a potentially three additional storeys for future wards. A two stepped excavation into the site slope of about 3m deep for each step is currently proposed. This building is anticipated to have maximum column working load of 10,000kN.

4.2.1 Excavation Conditions, including Groundwater

The excavation for the CSB lower levels will require excavations into the hillside to estimated depths of approximately 3m to 4m, in two stepped terrace fashion. Based on the borehole logs (101 to 104), the proposed bulk excavation will encounter the soil profile (both fill and residual clay), and will extend into the underlying mostly sandstone (Class 5 to Class 3) but with upper shale layer in BH103. Particular care is required during demolition and excavation to avoid undermining or removing support from the adjoining buildings to the east and south-east. Prior to any excavation commencing we recommend that reference be made to the WorkCover Authority of NSW's "Code of Practice – Excavation Work" dated 31 March 2000 (Cat. No. 312).

Prior to commencement of demolition and construction, we recommend that dilapidation survey reports be carried out on the neighbouring buildings and structures that falls within the zone of influence of the excavation, which is defined by a distance back from the excavation perimeter of twice the total depth of the excavation. The reports would provide a record of existing conditions prior to commencement of the work. Excavations and retention systems will need to be carefully planned and scheduled so as not to have any adverse effects on the buildings and structures adjoining or above the excavation.

Groundwater was measured in two borehole standpipe wells at depths of about 4.6m (or around RL159.8) and 5.2m or about RL156.4m. We also completed a pump out test in BH101 was completed on 7 June 2010 to estimate the seepage rate for the borehole well geometry. The inflow rate was estimated at 0.6 litres per hour during the first half hour, reducing to 0.32 litres per hour after 1 hour (refer to attached results sheet). If the proposed stepped excavations are near to or below the latter levels then there will be groundwater seepage issues. The basement walls and floor slab will have to be designed to cater for groundwater pressures and seepage. Further groundwater readings should be taken in the borehole standpipe wells.

Given the proposed depths of excavation, we expect that some groundwater seepage flows will occur from the base of the fill, at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, during excavation, is expected to be satisfactorily controlled by conventional sump pumping or gravity drainage systems. Groundwater seepage into the bulk excavation should be monitored (volume, source, location, etc) by site personnel and the results submitted to the hydraulic and geotechnical engineers so that unexpected conditions can be timely addressed. We recommend that a toe drain be formed at the base of all cut faces to collect groundwater seepage and direct it to the stormwater system.

4.2.2 Excavation Techniques, including Vibration Monitoring

An assessment of the excavation characteristics of the various strata is presented below. The excavatability of the shale and sandstone and the selection of appropriate excavation equipment have been assessed on the basis of the borehole information and strength test result tables (Table C). Assessment of excavation characteristics and productivity is not an exact science and contractors must make their own evaluation based on experience with specific equipment, and their own study of the borehole information and possibly inspection of the available rock cores, but we only store these for maximum three months after completion of drilling unless other arrangements are made; they might also request for further cored boreholes to be completed at the site). The ease with which excavation of rock is achieved depends upon the equipment used, the skill, and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.

The soil profile and extremely weathered bedrock (Class 5) may be readily excavated using conventional earthworks equipment, eg hydraulic excavators. Some of the underlying weathered bedrock of extremely or very low strength, if encountered, may also be excavated by bucket excavators, possibly with some ripping. However, we expect that low to medium (Class 4 and Class 3) and higher strength bedrock (Class 2), if encountered, will be most effectively excavated using hydraulic impact rock hammers or rotary grinders. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes and for detailed rock excavation, such as for footings and buried services. We expect that excavation of the low to medium strength or stronger rock will present hard ripping or “hard rock” excavation conditions. Ripping may only just be possible with a Caterpillar D10 or D11 dozer and a very generous allowance should be made for rock hammer assistance to the ripping, especially where rock defect spacing (bedding and cross bedding, joints, etc) is greater than about 0.5m and/or is heavily iron indurated. The use of an impact ripper is recommended.

We recommend that considerable caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to adjoining buildings and structures. If a hydraulic impact hammer is used, considerable caution should be taken during rock excavation, as there will likely be direct transmission of ground vibrations to adjoining structures and buildings.

Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. This limit of vibrations should be reviewed once more definite details of the excavation and development staging are known to confirm that they are still suitable. Given the close proximity of the neighbouring buildings, a vibration monitoring management plan should be prepared for the site. Low vibration excavation techniques which may be considered include the use of rotary grinders or grid sawing, in conjunction with ripping.

4.2.3 Excavation Support

Where space permits, excavations in the soil and extremely weathered rock profile may be temporarily battered to a side slope no steeper than 1V:1H. An intermediate bench about 1.5m wide should be incorporated into batter slopes deeper than about 5m. Flatter batters may be required where groundwater seepage is encountered. The stability of the batters in the weathered bedrock must be subject to confirmation by an inspection by a geotechnical engineer as the excavation progresses and the batters are formed say in intervals of 1.5m depth. Where space permits, the permanent walls would then be constructed at the toe of the temporary batters in the soils and poorer quality shale. Caution will be required during backfilling to prevent over-compaction adjacent to the walls and thereby causing excessive forces on the walls.

We expect that good quality sandstone of low or higher strength (if encountered) may be cut vertically. However, localised stabilisation measures may be necessary if adverse defects, such as inclined joints or bedding, are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc.

Clay seams occurring in permanently exposed sandstone slopes may also require 'dental' treatment. We therefore recommend that the rock face be progressively inspected by a geotechnical engineer/engineering geologist as excavation proceeds, to identify adverse defects and to propose appropriate stabilisation measures.

Where temporary batters cannot be accommodated, or where they are not preferred, a retention system will be required and should be installed prior to excavation commencing. Given the subsoil conditions encountered, a suitable retention system would comprise an anchored soldier pile wall with reinforced shotcrete infill. The anchors should be progressively installed as excavation proceeds. However, rock anchored contiguous pile walls are preferred along excavation perimeters where it is important to reduced excavation induced movements; contiguous walls would also assist in reducing and controlling seepage volumes, if these become an issue, into the excavation. Construction of the contiguous pile walls should be of high quality, taking the uttermost care to prevent soil loss through gaps that may occur between the piles as this would add to the possibility of settlement occurring outside the excavation. Such gaps should be rectified without delay, such as by mass concrete infill. Conventional bored piles would be suitable and should be socketed into the bedrock to sufficient depth below bulk excavation levels to satisfy both stability and founding considerations. Grout piles are preferred to avoid potential problems with groundwater.

4.2.4 Parameters for Retaining Wall Design

The major consideration in the selection of lateral pressures for the design of retaining walls is the need to limit the deformations occurring outside the excavation (in this case, along the eastern and south-eastern perimeters of the excavation that have adjoining pr close by buildings). The following characteristic lateral pressure coefficients and subsoil parameters may be adopted for the design of temporary and permanent retention systems:

1. The retaining walls should be uniformly founded below bulk excavation levels. For footing recommendations, refer to Section 4.2.5 below.

2. Free-standing cantilever walls supporting areas where movement is of little concern, (ie. where only garden or grassed areas are to be retained), should be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.35, for the soil and extremely weathered rock profile, assuming a horizontal retained surface.

3. A bulk unit weight of 20kN/m^3 should be adopted for the soil and extremely weathered bedrock profiles.

4. For progressively anchored or internally propped walls where minor movements can be tolerated, we recommend the use of a trapezoidal lateral earth pressure distribution of 6H kPa for the soil profile and extremely weathered bedrock, where 'H' is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.

5. Any surcharge affecting the walls (eg traffic and construction loading, nearby footings, etc) should be allowed in the design using the 'active' earth pressure coefficient for free-standing cantilever walls and an 'at rest' earth pressure coefficient, K_o, of 0.55, for anchored or internally propped walls. If inclined retained surfaces are proposed, then the earth pressure coefficient will have to be appropriately increased or the inclined surface treated as a surcharge.

6. The retaining walls should be designed as drained and measures taken to induce complete and permanent drainage of the ground behind the walls. Strip or core drains installed between adjacent soldier piles are considered to be an appropriate drainage measure.

7. For piles embedded in the underlying low or higher strength bedrock below bulk excavation level, an allowable lateral toe resistance of 200kPa may be adopted. This value assumes excavation is not carried out within the zone of influence of the wall toe and the rock does not contain unfavourable defects, etc.

The upper 0.3m depth of socket below bulk excavation level should not be taken into account to allow for disturbance and tolerance effects during excavation. The quality of the toe restrained rock should be progressively inspected by a geotechnical engineer on exposure to confirm unexpected conditions do not exist.

8. Rock anchors should have a minimum free length of 3m. Rock anchors bonded at least 3m into low or higher strength shale should be designed for an allowable bond stress of 100kPa. Rock anchors bonded at least 3m into low to medium or higher strength sandstone should be designed for an allowable bond stress of 300kPa. All anchors should be proof tested to 1.2 times the working load under the direction of an experienced engineer independent of the anchor contractor. The testing may allow an upgrading of the above bond stresses. We recommend that only experienced contractors be considered for anchor installation. For the permanent situation, we assume that the proposed building will provide the required lateral support to the cut face. If not, permanent anchors will be required, which should be designed for corrosion resistance and long-term durability.

4.2.5 Footings

We recommend that the proposed new CSB be uniformly founded onto sandstone bedrock. The boreholes which are relevant to the proposed CSB area are BHs 101 to 104 inclusive. Care is required with the founding depths so that the adjacent buildings are not surcharged and that the proposed footing excavations do not undermine the adjacent existing footings. The footings (e.g. piles and pads) may be designed for the following allowable bearing pressures:

Depth (m) to Recommended Allowable Bearing Pressures

Borehole	700kPa	1000kPa	2000kPa	3500kPa
101	0.4	0.4	2.3	2.3
102	1.5	1.5	1.5	2.0
103	1.2	4.5	4.5	5.1
104	1.1	1.8	3.1	3.8

NOTES:

1. All footings should have a nominal socket of 0.3m into the selected rock foundation unit.
2. An allowable shaft adhesion of about 10% of the safe bearing values in compression may be adopted for design of pile sockets through the bedrock. Special tools should be used to roughen the sides of load bearing pile sockets in the bedrock.
3. The above bearing pressures are based on serviceability considerations of settlement less than 1% of the minimum footing dimension.

If the designer wishes to adopt the limit state design methods, such as in the Piling Code, AS2159, then the ultimate values of end bearing pressure may be estimated by multiplying the above recommended allowable values by Factors of Safety of around 3. A Factor of Safety of about 2 should be applied to the shaft adhesion values. We recommend that the ultimate values be multiplied by a geotechnical strength reduction factor, Φ_g, in the order of 0.5 to 0.6. Higher reduction factors may be adopted but these will depend on the intensity and type of proving of the footings and their foundation. An appropriate load factor should also be applied to the proposed footing loadings. It must be understood that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate pile performance. It must be emphasised that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate footing performance.

If bored or augered grout piles are to be socketed into the sandstone then we recommend that heavy, high capacity drilling rigs with rock augers or coring buckets be used to drill the piles, especially when drilling through medium strength or stronger rock or through the iron indurated bands.

All footing excavations should be free from all loose or softened materials prior to placement of concrete. We recommend that footing excavations be checked and approved prior to concrete being poured. The initial stages of footing excavation should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit. We can assist with future geotechnical inspections if you wish to commission us at the appropriate time.

During installation of CFA piles it is recommended that the initial piles be installed as close as practical to our borehole locations to calibrate the equipment and operator to the subsurface conditions by direction comparison of the installation performance and readings to the borehole results. These initial readings can then be used to assist with installation of piles away from the borehole locations to assess that the appropriate foundation material has been reached.

4.2.6 On-Grade Floor Slabs

The on-grade floor slabs will probably directly overlie a combination of bedrock and soil. For uniformity of support, we recommend that the proposed on-grade floor slabs be supported on bedrock and that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single sized, washed aggregate (eg. 'blue metal' gravel) and should connect with the wall drains.

The drains should lead groundwater seepage to a sump for gravity or pumped discharge to the stormwater system. The underfloor drainage may comprise a blanket drain or a drain installed into slots which are formed in the subgrade.

4.3 Proposed Fire Services Structures and Oxygen Tanks

BH107 that was drilled in the Fire Services area disclosed a deep, uncontrolled fill of poor compaction, directly underlain by shale bedrock at a depth of 2.1m below existing ground surface. Because of this fill, this area is considered to be Class P in accordance with AS2870. The lateral extend of this deep, uncontrolled fill in the area is unknown, and further investigation would be required for this to be determined. The fill is deemed unsuitable as a bearing stratum for, at least, the footings and ground floor slabs. Excavation and replacement of the fill appears to be a poor option due to the significant depth of fill. Hence, we recommend the structures (details of which are unknown to us) be designed and constructed as fully suspended, i.e. including slab, on piles (e.g. bored piers or CFA piles) taken into the shale. The piles may be designed for an allowable end bearing pressure of 1000kPa when socketed into the shale of very low to low strength, assuming a minimum socket of 0.3m. An allowable shaft adhesion of about 10% of the safe bearing values in compression may be adopted for design of pile sockets through the bedrock. Special tools should be used to roughen the sides of load bearing pile sockets in the bedrock.

BH111 was drilled in the area of the proposed Oxygen Tanks. The borehole disclosed shallow depth to shale at 0.4m. Hence, strip or beam footings on shale are recommended for the tanks (details of which are also unknown to us). Similar bearing pressures may be adopted as provided above for the Fire Services area. Above the shale is a relative shallow depth of fill, which should be fully stripped and removed or fully stripped and replaced with well, compacted, controlled engineered fill.

All footing excavations should be free from all loose or softened materials prior to placement of concrete. We recommend that footing excavations be checked and approved prior to concrete being poured. The initial stages of footing excavation should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit. We can assist with future geotechnical inspections if you wish to commission us at the appropriate time.

4.4 Temporary Car Parking Areas

BHs 105 and 106 represent the south-western car park area, and BHs 108 to 110 and 112 to 114, represent the north-eastern car park. The design of carpark pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of new fill imported to the site, as well as vehicle loading and use. We note that significant earthworks and stormwater diversion works will be required, particularly for the lower new ring road extension and the temporary parking.

4.4.1 Subgrade Preparation

Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1. Strip the upper layer of fill that contains deleterious materials or organics, and stockpile this separately since these materials are not suitable for re-use as engineered fill.
2. Given that the car parks are only temporary then the remaining fill after the above stripping may be left in place if it passes under the proof rolling checks recommended below. Clearly, leaving the fill in place even if proof rolling is satisfactory still carries a risk of poor pavement performance. We would prefer

that the remaining existing fill is fully excavated down to surface of the residual clay or bedrock and replaced with engineered fill. The replacement should extend to at least 1m beyond the boundaries of the pavement area. Such an excavation might have to be completed with battered sides of not steeper than 1 Vertical to 1 Horizontal. The earthworks contractor must ensure that during the backfilling earthworks that the engineered fill is well 'keyed' into the side batters of the excavation.

3. The exposed soil subgrade at the base of the excavation should be proof rolled with at least 8 passes of a heavy (not less than 7 tonne) smooth drum roller used in static or non-vibratory mode of operation. Caution is required when proof rolling near the site buildings. The purpose of the proof rolling is to detect any soft or heaving areas.
4. The final pass should be undertaken in the presence of a geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction.
5. If dry conditions prevail at the time of construction then any exposed residual clay subgrade may become desiccated or have shrinkage cracks prior to pouring any concrete slabs. If this occurs then the subgrade must be watered and rolled until the cracks disappear.
6. Unstable subgrade detected during proof rolling should be locally excavated down to a stiff or sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill. From the borehole results we expect few, if any, unstable subgrade areas to occur provided good site drainage is maintained and the earthworks are carried out during good weather.

4.4.2 Engineered Fill

Engineered fill should preferably comprise well graded granular material (ripped or crushed sandstone), free of deleterious substances and having a maximum particle size of 75mm. The existing silty and sandy clays may also be used for engineered fill provided unsuitable ('over-wet' and 'over-sized') material and any organics or building rubble are excluded. The fill for backfilling earthworks platforms should be compacted in layers of not greater than 200mm loose thickness to a minimum density of 98% SMDD for the granular material, and to a density strictly between 98% and l02% SMDD and within 2% of SOMC for clayey material.

All compacted fill should be suitably retained, or alternatively battered to a permanent slope no steeper than 1V:2H. The exposed fill embankments should be protected from erosion using a quickly establishing grass cover or by structural means (eg. stone pitching or shotcrete). Cut-off surface drains at the crest and the toe of embankments may also be required, particularly for embankments in excess of 1.5m in height.

4.4.3 Density Testing

Density testing of engineered fill should be carried out at a frequency of at least one test per layer per 1,000m^2, or three tests per layer per visit, whichever requires the most tests to confirm the compaction specification has been achieved. At least Level 2 testing of earthworks should be carried out in accordance with AS3758. Preferably, the Geotechnical Testing Authority should be engaged directly on behalf of the client and not as part of the earthworks contract.

4.4.4 Site Drainage

The clay subgrade may soften with an increase in moisture content. Therefore, good and effective site drainage should be provided both during construction and for long-term site maintenance. Earthworks platforms should be graded to maintain cross falls during construction. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untraffickable when wet. We recommend that if soil softening occurs, the subgrade be over-excavated to below the depth of moisture softening and the excavated material be replaced with a clean, well graded fill, compacted as specified above. Where the exposed subgrade exhibits shrinkage cracking, the surface should be moistened and rolled until the shrinkage cracks are no longer evident.

Subsoil drains should be provided along the perimeter of pavements with inverts at not less than 0.2m below clay subgrade level. The drainage trench should be excavated with a longitudinal fall to appropriate discharge points to reduce water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

4.4.5 Pavements

Based on the soaked CBR test results, and provided the subgrade preparation is carried out as described above, we recommend that the design of flexible pavements on this site be based on a CBR of 2.5% for the compacted clay subgrade. For concrete or rigid pavement design, an equivalent modulus for subgrade reaction of 20kPa/mm (750mm plate) or a short term Young's Modulus of 20MPa may be adopted.

Concrete pavements should be supported on a sub-base layer of RTA 3051 specification unbound good quality crushed rock compacted to a density of at least 100% SMDD. The subbase material would provide more uniform slab support and reduce 'pumping' of subgrade 'fines' at joints. Concrete pavements should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements should be used in areas where heavy vehicles manoeuvre, such as garbage bin truck unloading areas.

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 Jeffery and Katauskas Pty Ltd 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.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

FERNANDO VEGA
Senior Associate.

Joseph Chaghouri
Geotechnical Engineer.

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PO Box 976
North Ryde, Bc 1670
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Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

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 %**
106	0.40-1.00	25.0	49	19	30	14.0
107	2.50-3.00	10.6				
107	4.00-4.50	14.8				
107	4.50-5.00	19.0				
108	0.20-1.00	28.7	61	23	38	15.0
111	1.00-1.50	18.0				
111	2.50-3.00	18.6				

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

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Ref No: 22758V
Table B: Page 1 of 1

TABLE B
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULTS

BOREHOLE NUMBER	106	108
DEPTH (m)	0.40 - 1.00	0.20 - 1.00
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.589 STD	1.502 STD
Optimum Moisture Content (%)	21.3	28.2
Moulded Dry Density (t/m^3)	1.55	1.47
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	101	99
Moisture Contents		
Insitu (%)	25.0	28.7
Moulded (%)	21.6	28.0
After soaking and		
After Test, Top 30mm(%)	26.1	30.2
Remaining Depth (%)	24.0	29.7
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	1.0	0.0
C.B.R. value: @5.0mm penetration	2.5	2.5

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

Approved Signatory
(A.Tatikonda)

Date: 1/7/10

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	3.10-3.13	0.5	10
	4.30-4.33	0.7	14
	5.43-5.46	0.2	4
102	4.07-4.10	0.3	6
	5.04-5.07	0.7	14
	6.12-6.15	0.3	6
103	4.87-4.89	2.5	50
	5.52-5.55	0.5	10
	6.53-6.56	2.2	44
	7.13-7.16	0.5	10
104	4.27-4.30	1.1	22
	5.46-5.49	0.9	18
	6.70-6.73	0.8	16

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

A.C.N. 003 550 801

BOREHOLE PUMP-OUT TEST RESULTS

Job No: 22758V

Client: Sydney Adventist Hospital Limited
Project: Stage 1, Phase 1 Development, Proposed Hospital
Location: San Hospital, 185 Fox Valley Road, Wahroonga, NSW

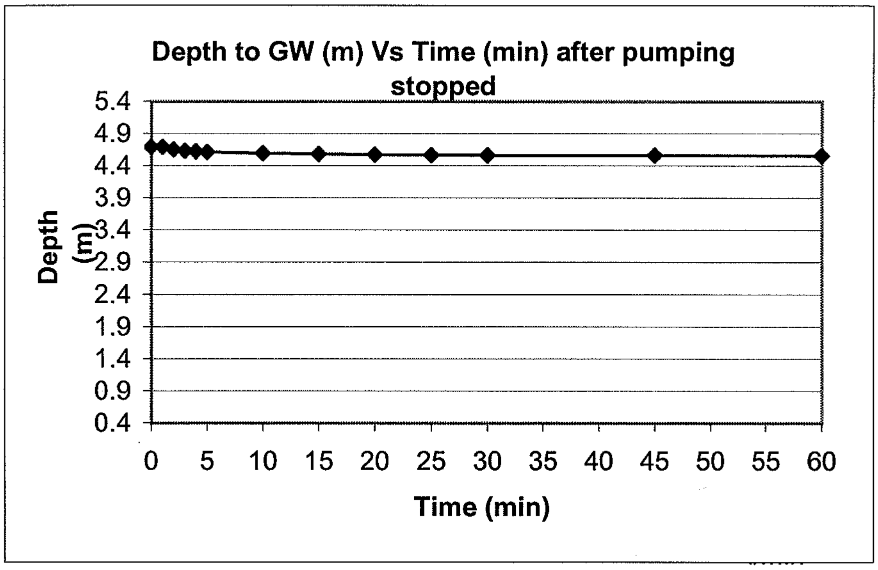
BOREHOLE: 101 Test Date: 7-Jun-10

Approximate Surface Level:
Subsurface Profile:

PVC Standpipe Diameter: 50mm
Screen Depth Below GL:

Weather: Fine
Recent: Fine

Water Depth Prior to Pumping:



Depth (m)	Time (min)
4.7	0
4.69	1
4.65	2
4.63	3
4.62	4
4.6	5
4.59	10
4.58	15
4.57	20
4.565	25
4.56	30
4.56	45
4.555	60

Water Inflow Rate First Half Hour 0.62 litres/hour

Water Inflow Rate Second Half Hour 0.02 litres/hour

Water Inflow Rate Full One Hour 0.32 litres/hour

File Name: 22758VpumpBH101

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BOREHOLE LOG

Borehole No. **101**

1/2

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Method:** SPIRAL AUGER JK300 | **R.L. Surface:** 164.4m

Date: 2-6-10 | **Datum:** AHD

Logged/Checked by: L.Y./[signature]

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 OF AUGERING			0		CL	TOPSOIL: Silty clay, low to medium plasticity, brown, with a trace of root fibres.	MC>PL	-	-	RESIDUAL
		N > 10 9,10/ 100mm REFUSAL			-	SILTY CLAY: medium plasticity, yellow brown.	XW	EL	-	
			1			SANDSTONE: fine to medium grained, light grey.	XW-DW	VL-L		VERY LOW 'TC' BIT RESISTANCE
			2			as above, but brown and light grey.	DW	M-H		MODERATE TO HIGH RESISTANCE
			3			REFER TO CORED BOREHOLE LOG				
			4							
			5							
			6							
			7							

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Borehole No. 101
2/2

CORED BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Core Size:** NMLC | **R.L. Surface:** 164.4m
Date: 2-6-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK300 | **Bearing:** - | **Logged/Checked by:** L.Y./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL VL L M H VH EH)	DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS – DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
		2		START CORING AT 2.75m						
FULL RET-URN		3		SANDSTONE: fine to medium grained, light grey and brown, with occasional cross bedded at 0-10°.	DW	M	×		- J, 70°, P, R	
				as above, but red brown and light grey.					- Be, 10° - Be, 0° - J, 70°, P, R - J, 60°, P, R	
AFTER 1 HR ON 7-6-10		4–5					×			
		5		as above, but light grey and yellow brown.		L	×		- CS, 10mm.t	
		6		END OF BOREHOLE AT 5.79m					50mm DIA. PVC PIPE INSTALLED TO 5.75m SLOTTED BETWEEN 2.75m AND 5.75m	
		7								
		8								
		9								


JEFFERY & KATAUSKAS PTY LTD
JOB No. 22758V BH101 Start Coring at: 2.75m
2
3
4
5
EOBH@5.79m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
102
1/2

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V
Date: 2-6-10
Method: SPIRAL AUGER JK300
R.L. Surface: 165.0m
Datum: AHD
Logged/Checked by: L.Y./

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 OF AUGERING			0			FILL: Silty clay, medium plasticity, brown mottled grey, with a trace of roots.				
	DS	N = 6 2,2,4			CH	SILTY CLAY: high plasticity, orange brown mottled yellow brown, with a trace of roots.	MC>PL	VSt-H	- 250 330 440	RESIDUAL
			1							
	DS				-	SANDSTONE: fine to medium grained, red brown and light grey.	DW	L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			2					M-H		MODERATE TO HIGH RESISTANCE WITH LOW BANDS
	DS		3							MODERATE TO HIGH RESISTANCE
						REFER TO CORED BOREHOLE LOG				
			4							
			5							
			6							
			7							

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CORED BOREHOLE LOG

Borehole No. 102
2/2

Client:	SYDNEY ADVENTIST HOSPITAL LIMITED
Project:	PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location:	SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V	**Core Size:** NMLC	**R.L. Surface:** 165.0m
Date: 2-6-10	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK300	**Bearing:** -	**Logged/Checked by:** L.Y./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		3		START CORING AT 3.5m						
FULL RET-URN		4 5 6		SANDSTONE: fine to medium grained, light grey and orange brown, with occasional cross bedding at 25-30°.	DW	L-M	× (L, ~4.1m) × (L-M, ~5.0m) × (L, ~6.1m)		- Be, 0° - Be, 0° - Be, 0° - J, 50° - J, 60° -J, 55°, P, R - J, 70°, P, R - CS, 20mm.t - Be, 0° - Be, 0°	
		7 8 9		END OF BOREHOLE AT 6.64m						


JEFFERY & KATAUSKAS PTY LTD
JOB No. 22758V
BH102
Start Coring at: 3.5 m
3
4
5
6
EOBH @ 6.64 m

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **103**
1/2

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V
Date: 2-6-10
Method: SPIRAL AUGER JK300
R.L. Surface: 167.7m
Datum: AHD
Logged/Checked by: L.Y./

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 OF AUGERING			0			FILL: Silty clay, low to medium plasticity, dark brown mottled brown and grey, with a trace of igneous gravel and roots.	MC>PL			
		N = 9 4,4,5								
			1		CH	SILTY CLAY: high plasticity, orange brown mottled red brown.	MC>PL	VSt -H	390 415 460	RESIDUAL
					-	SHALE: grey.	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
		N = 24 10,12,12	2							
						as above, but with ironstone bands.	XW-DW	EL-VL		
			3							
			4							
						REFER TO CORED BOREHOLE LOG				
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

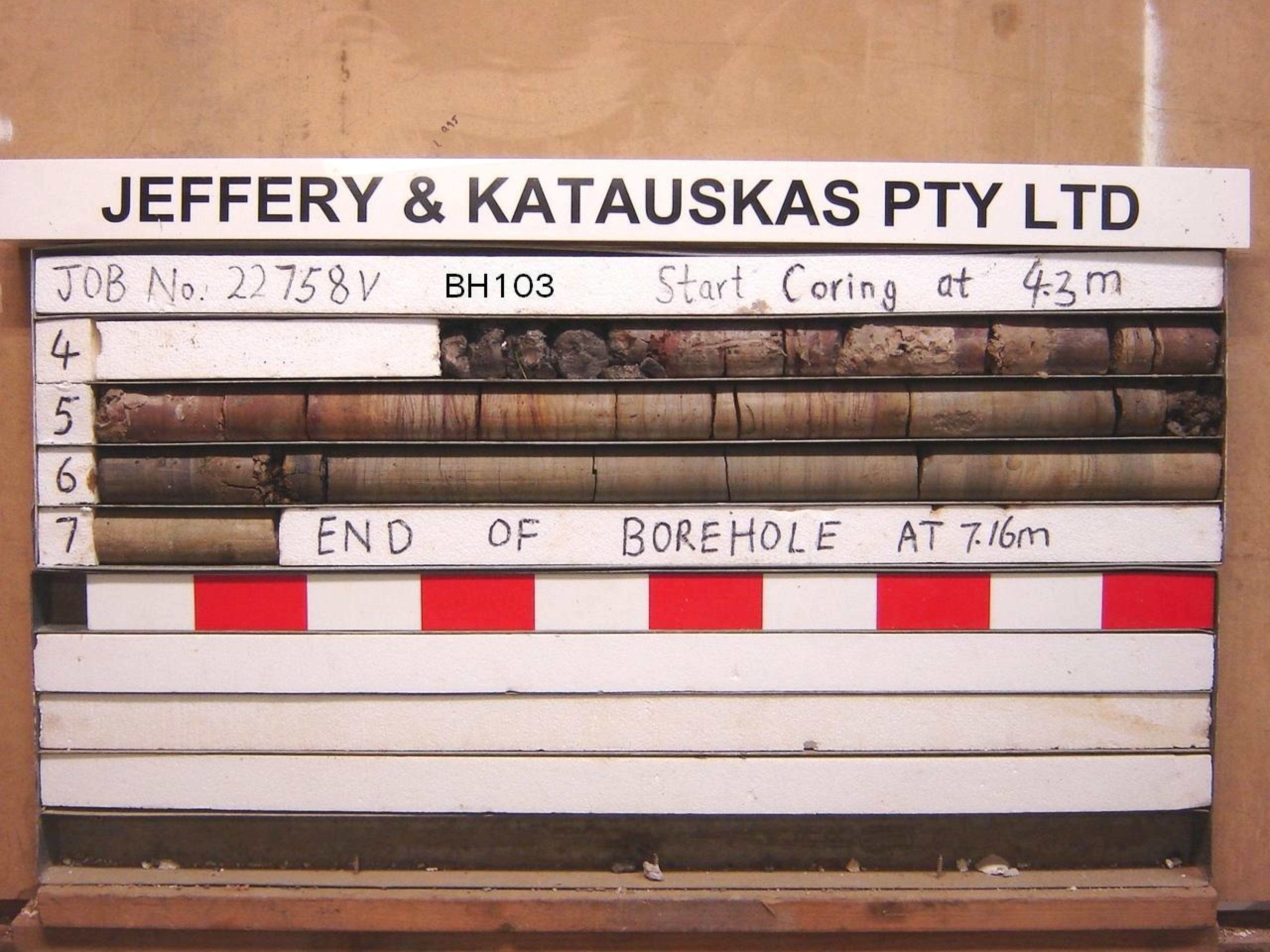
CORED BOREHOLE LOG

Borehole No. **103**
2/2

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Core Size:** NMLC | **R.L. Surface:** 167.7m
Date: 2-6-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK300 | **Bearing:** - | **Logged/Checked by:** L.Y./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		4		START CORING AT 4.3m						
FULL RETURN				SHALE: dark grey.	XW	EL			- Cr/CS, 150mm.t	
		5		SANDSTONE: fine grained, red brown and light grey, with cross bedding at 0-5°.	DW	H	×		- CS, 10mm.t - CS, 100mm.t - CS, 50mm.t - CS, 30mm.t	
						M-H	×			
		6		as above, but light grey, with cross bedding at 0-10°.			×		- Cr, WITH CLAY INFILL - CS, 15mm.t - Cr. 30mm.t	
		7					×			
		8		END OF BOREHOLE AT 7.16m						
		9								
		10								


JEFFERY & KATAUSKAS PTY LTD
JOB No. 22758V
BH103
Start Coring at 4.3m
4
5
6
7
END OF BOREHOLE AT 7.16m

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BOREHOLE LOG

Borehole No. **104**

1/2

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK500 **R.L. Surface:** 164.6m

Date: 3-6-10 **Datum:** AHD

Logged/Checked by: M.L.T./

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 OF AUGERING						0			FILL: Silty sandy clay, low to medium plasticity, brown and dark brown, fine to medium grained sand, with fine to coarse grained ironstone, shale and igneous gravel, and root fibres.	MC>PL			GRASS COVER
					N = 9 2,4,5			CL	SILTY SANDY CLAY: medium plasticity, orange brown and red brown, fine grained sand, with fine to coarse grained ironstone gravel.	MC<PL	H	- 420 410 420	RESIDUAL
						1		-	SANDSTONE: fine to medium grained, light grey stained orange brown and red.	XW-DW	VL-L	-	VERY LOW 'TC' BIT RESISTANCE
						2			as above, but orange brown and red brown, with iron indurated bands.	DW	L		LOW RESISTANCE
									as above, but light grey and orange brown.	XW-DW	VL-L		VERY LOW TO LOW RESISTANCE WITH MODERATE BANDS
						3			as above, but orange brown and red brown.	DW	L		LOW RESISTANCE
						4			as above, but light grey, without iron indurated band.	SW	M		MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

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Borehole No. 104 2/2

CORED BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Core Size:** NMLC | **R.L. Surface:** 164.6m
Date: 3-6-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** M.L.T./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		3								
		4		START CORING AT 4.14m						
FULL RETURN		5, 6, 7		SANDSTONE: fine to medium grained, light grey, bedded at 0-10°.	FR	H / M	× × ×			
		8, 9		END OF BOREHOLE AT 7.24m						


JEFFERY & KATAUSKAS PTY LTD
22758V
BH104
START CORING AT 4.14m
4
4.14m
5
6
7
END OF BOREHOLE AT 7.24m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **105**
1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A
Date: 3-6-10 **Datum:**
Logged/Checked by: L.Y./

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			FILL: Silty clay, low to medium plasticity, dark brown, with a trace of roots.	MC>PL			
								CL	SILTY CLAY: medium plasticity, yellow brown mottled red brown, with a trace of roots.	MC>PL	VSt	-	RESIDUAL
				DS	N = 6 2,2,4							200 210 230	
				DS		1		-	SANDSTONE: fine to medium grained, light grey, red brown and orange brown.	XW	EL-VL	-	VERY LOW 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

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Borehole No. 106
1/1

BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Method:** SPIRAL AUGER JK300 | **R.L. Surface:** N/A
Date: 3-6-10 | **Datum:**
Logged/Checked by: L.Y./

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			FILL: Silty clay, low to medium plasticity, grey and brown, with a trace of roots and sand.	MC>PL			
		N = 7 1,3,4	1		CL-CH	SILTY CLAY: medium to high plasticity, yellow brown and red brown, with a trace of roots.	MC>PL	VSt	- 370 470 340	RESIDUAL
					-	SANDSTONE: fine to medium grained, light grey and yellow brown	XW	EL	-	VERY LOW 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 1.5m				
			2							
			3							
			4							
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
107
1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A
Date: 3-6-10 **Datum:**
Logged/Checked by: L.Y./

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			FILL: Silty clay, low to medium plasticity, dark grey mottled brown, with a trace of shale fragments.	MC>PL			APPEARS POORLY COMPACTED HAND AUGERED TO 1.0m DEPTH
						1							
				■	N = 6 2,1,5								
						2		-	SHALE: dark grey.	DW	VL-L	-	LOW TO MODERATE 'TC' BIT RESISTANCE
				■		3							
				■		4							
						5			END OF BOREHOLE AT 5.0m				
						6							
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **108**
1/1

BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V
Date: 3-6-10
Method: SPIRAL AUGER JK300
R.L. Surface: N/A
Datum:
Logged/Checked by: L.Y./

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		TOPSOIL: Silty clay, low to medium plasticity, brown, with a trace of root fibres.				
	N = 5 1,2,3		CH	SILTY CLAY: high plasticity, orange brown mottled yellow brown.	MC>PL	VSt	- 230 250 235	RESIDUAL
		1						
	N = 15 7,7,8			as above, but light grey mottled red brown, with a trace of ironstone gravel.		H	>600 >600 >600	
		2		END OF BOREHOLE AT 1.95m				
		3						
		4						
		5						
		6						
		7						

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **109**
1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V
Method: SPIRAL AUGER JK300
R.L. Surface: N/A
Date: 3-6-10
Datum:
Logged/Checked by: L.Y./[illegible]

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			FILL: Silty clay, medium plasticity, dark brown, with a trace of roots and slag.	MC>PL			GRASS COVER
	DS	N = 9 1,4,5	1		CH	SILTY CLAY: high plasticity, orange brown, with a trace of slag.	MC>PL	H	470 560 475	RESIDUAL
	DS	N = 22 8,9,13			-	SHALE: light grey.	XW	EL	>600 >600 >600	
			2			END OF BOREHOLE AT 1.95m				
			3							
			4							
			5							
			6							
			7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **110**

1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A

Date: 4-6-10 **Datum:**

Logged/Checked by: L.Y./AS

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			FILL: Silty clay, low to medium plasticity, dark brown.	MC>PL			GRASS COVER APPEARS POORLY COMPACTED
					N = 6 1,2,4			CH	SILTY CLAY: high plasticity, orange brown mottled yellow brown.	MC>PL	H	230 350 390	RESIDUAL
						1		-	SANDSTONE: fine to medium grained, light grey and red brown.	DW	VL-L	>600 >600 >600	VERY LOW TO LOW 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **111**

1/1

BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK300 **R.L. Surface:** 164.3m

Date: 4-6-10 **Datum:** AHD

Logged/Checked by: L.Y./

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			FILL: Silty clay, low plasticity, grey and brown, with a trace of igneous gravel and roots.				APPEARS POORLY COMPACTED
					-	SHALE: grey.	DW	L		LOW TO MODERATE 'TC' BIT RESISTANCE
			1							LOW RESISTANCE
			2				XW-DW	VL		VERY LOW TO LOW RESISTANCE
			3							
			4							
			5							
			6			END OF BOREHOLE AT 6.0m				
			7							

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Borehole No. **112**
1/1

BOREHOLE LOG

Client: SYDNEY ADVENTIST HOSPITAL LIMITED
Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V
Date: 4-6-10
Method: SPIRAL AUGER JK300
R.L. Surface: N/A
Datum:
Logged/Checked by: L.Y./

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			FILL: Sandy silty clay, low to medium plasticity, brown and grey, with a trace of roots and fine to medium grained sandstone gravel.	MC≈PL			GRASS COVER APPEARS POORLY COMPACTED
								CH	SILTY CLAY: high plasticity, light grey and brown.	MC>PL	(VSt)		RESIDUAL
						1		-	SANDSTONE: fine to medium grained, light grey and brown.	DW	L-M	-	MODERATE 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **113**

1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V | **Method:** SPIRAL AUGER JK300 | **R.L. Surface:** N/A

Date: 4-6-10 | **Datum:**

Logged/Checked by: L.Y./

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			TOPSOIL: Silty clay, low to medium plasticity, brown, with a trace of root fibres.				
		N = 23 4,8,15			CH	SILTY CLAY: high plasticity, orange brown mottled yellow brown.	MC>PL	H	- >600 >600 >600	RESIDUAL
			1		-	SANDSTONE: fine to medium grained, brown and light grey.	DW	M		MODERATE 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 1.5m				
			2							
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
114
1/1

Client: SYDNEY ADVENTIST HOSPITAL LIMITED

Project: PROPOSED HOSPITAL DEVELOPMENT, STAGE 1, PHASE 1

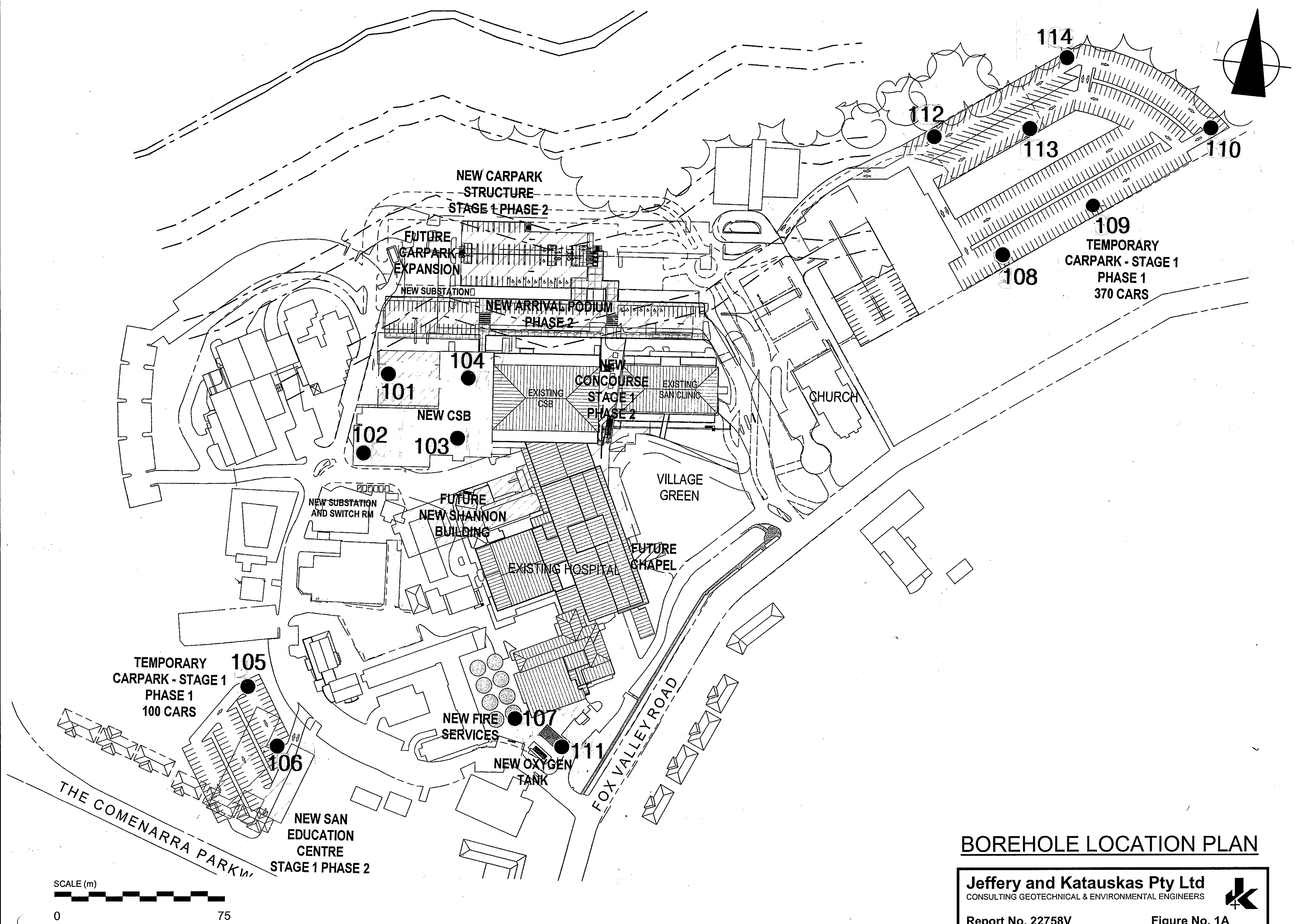
Location: SAN HOSPITAL, 185 FOX VALLEY ROAD, WAHROONGA, NSW

Job No. 22758V **Method:** SPIRAL AUGER JK300 **R.L. Surface:** N/A

Date: 4-6-10 **Datum:**

Logged/Checked by: L.Y./

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			TOPSOIL: Silty clay, low to medium plasticity, brown, with a trace of root fibres.				
					CL-CH	SILTY CLAY: medium to high plasticity, orange brown mottled brown.	MC>PL	VSt	-	RESIDUAL
		N > 4 1,4/10mm							250 280 300	
					-	SANDSTONE: fine to medium grained, brown and light grey.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
			1					L-M		MODERATE RESISTANCE
						END OF BOREHOLE AT 1.5m				
			2							
			3							
			4							
			5							
			6							
			7							



BOREHOLE LOCATION PLAN

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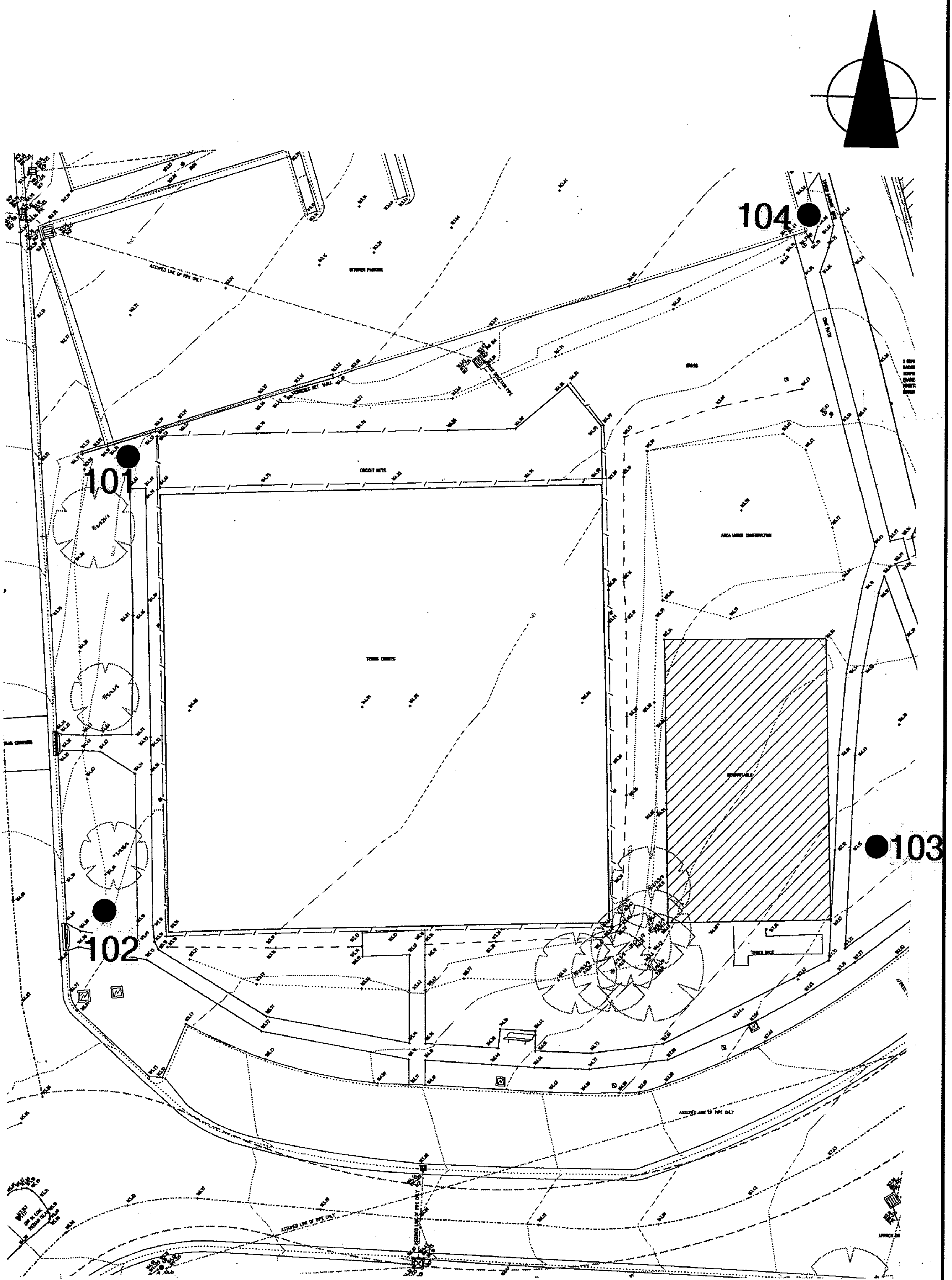
Report No. 22758V **Figure No. 1A**



BOREHOLE LOCATION PLAN

SCALE (m)

0 20

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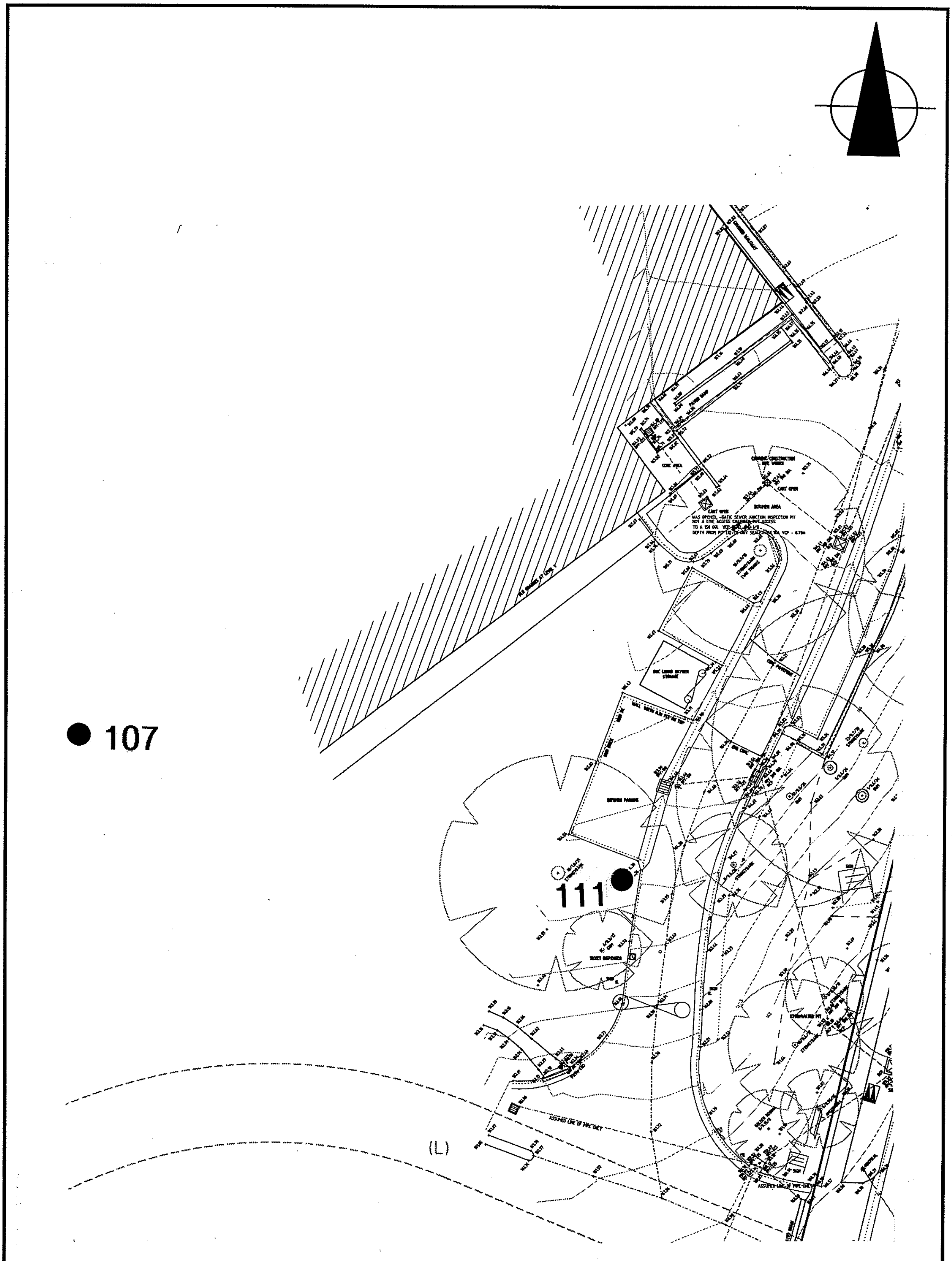
Report No. 22758V **Figure No. 1B**



BOREHOLE LOCATION PLAN

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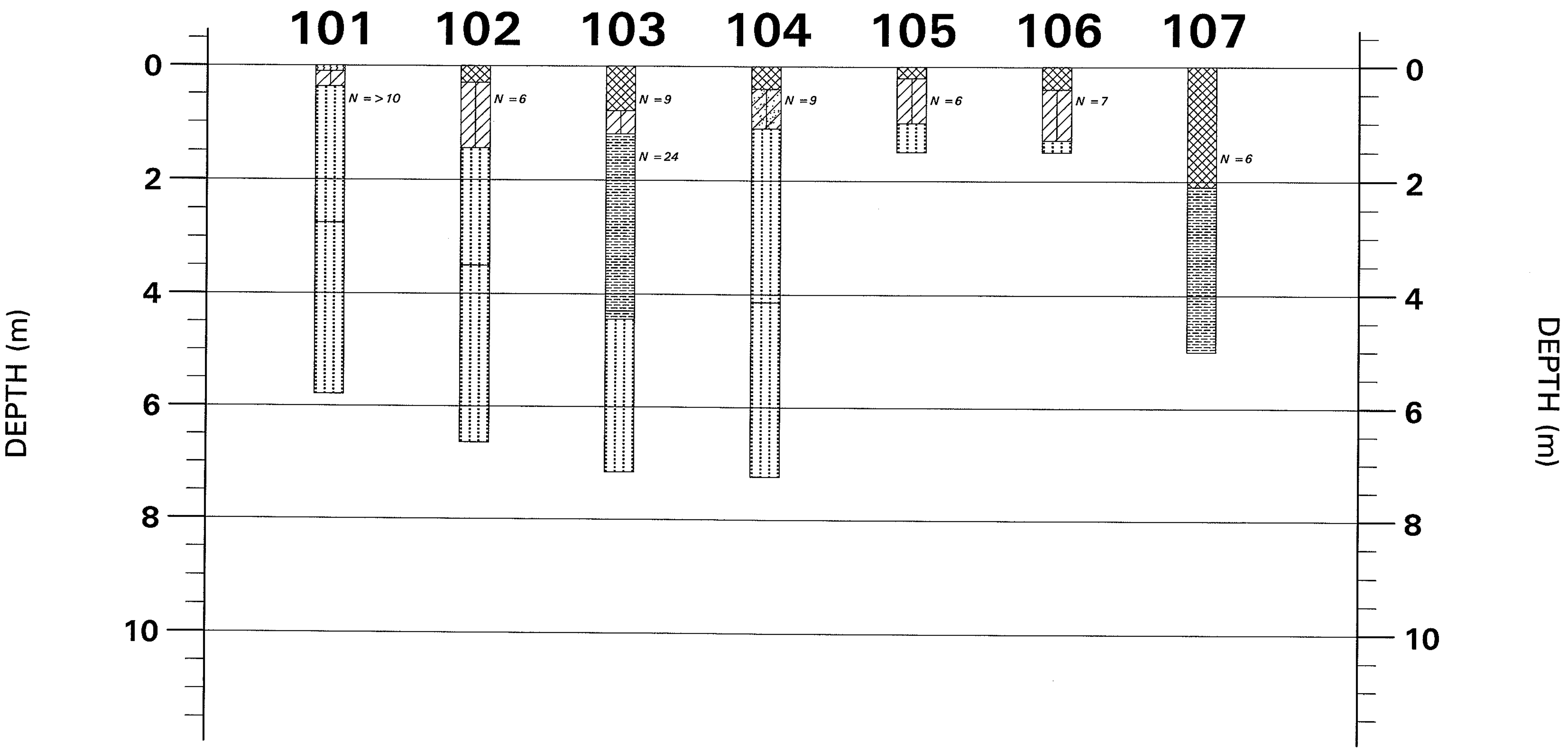
Report No. 22758V **Figure No. 1C**

SCALE (m)
0 20

GRAPHICAL BOREHOLE SUMMARY



Topsoil
Silty Clay
Sandstone/ Greywacke
Fill
Shale
Sandy Silty Clay

N — SPT "N" VALUE

Nc — SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

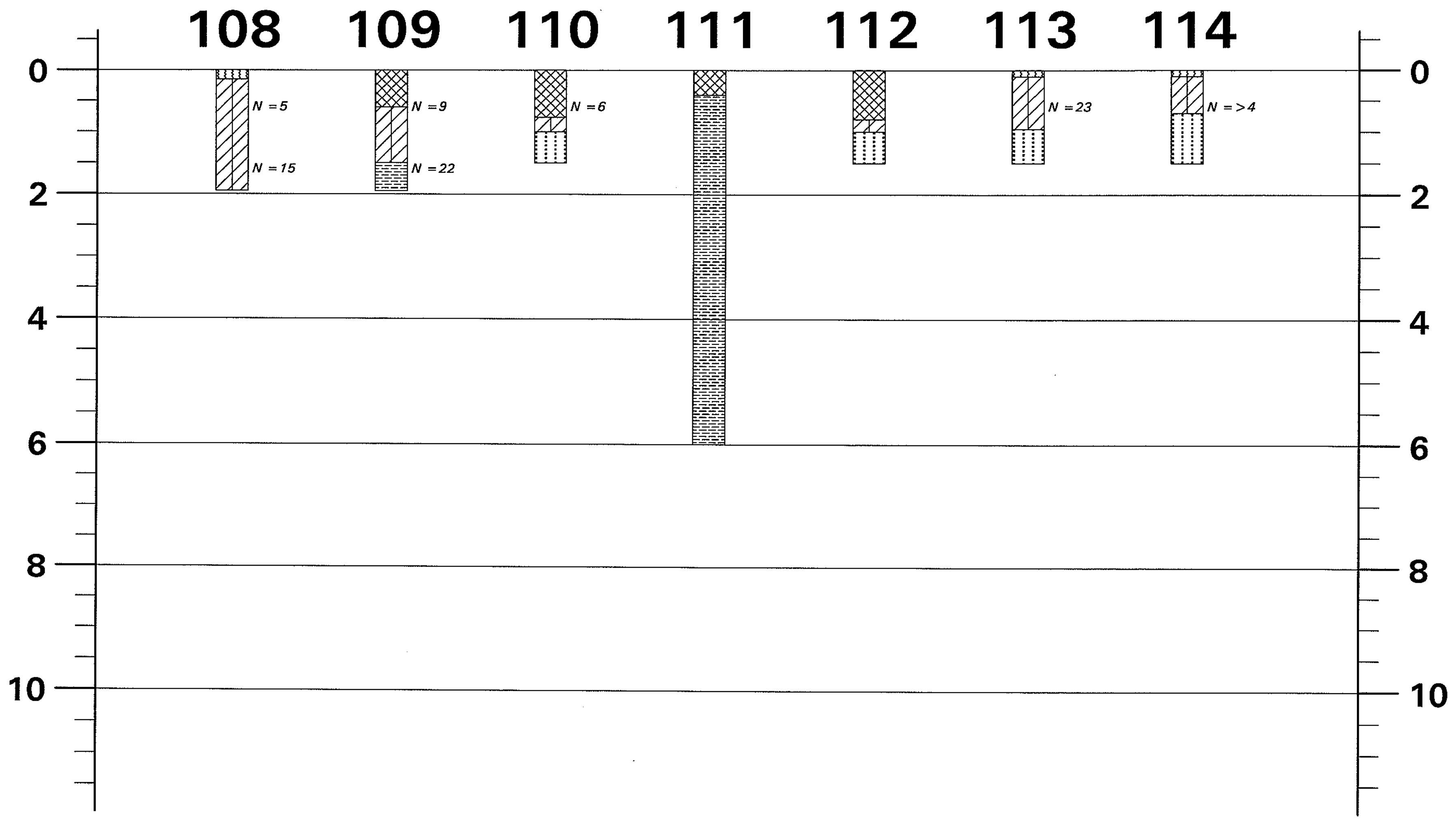
Scale: 1 : 100 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 22758V

Figure No.: 2

GRAPHICAL BOREHOLE SUMMARY



Topsoil	Shale	Nc SOLID CONE BLOW COUNTS PER 150mm
Silty Clay	Sandstone/ Greywacke	
Fill	N SPT "N" VALUE	

NOTE: REFER TO BOREHOLE LOGS

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

Jeffery and Katauskas Pty Ltd

Job No.: 22758V

Figure No.: 3

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A.C.N. 003 550 801

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1986 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

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

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

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

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

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level At a Frequency of			Plane of Floor of Uppermost Storey
		Less than 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	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 100 Hz, the higher values in the 50 Hz to 100 Hz column should be used.

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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 (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 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 <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 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.

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GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

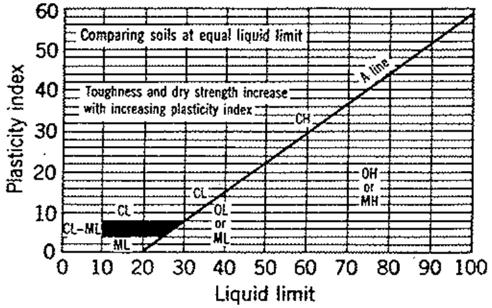
UNIFIED SOIL CLASSIFICATION TABLE

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	$C_U = \frac{D_{60}}{D_{10}}$ 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	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	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	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*)	$C_U = \frac{D_{60}}{D_{10}}$ 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

Laboratory classification criteria (coarse-grained soils): 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.

Use grain size curve in identifying the fractions as given under field identification

Fine-grained soils More than half of material is *smaller* than 75 μm sieve size (The 75 μm sieve size is about the smallest particle visible to naked eye)	Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
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
	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	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
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	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	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	



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. 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.

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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.

Ref: Standard Sheets/Log Symbols
November 2007

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LOG SYMBOLS

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	