REPORT

TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MENTAL HEALTH INTENSIVE CARE UNIT

AT

PRINCE OF WALES HOSPITAL,
AVOCA STREET, RANDWICK, NSW

28 October 2010

Ref: 24288WHrpt

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
Postal Address: PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 • Fax: 02 9888 5003 • Email: engineers@jkgroup.net.au • ABN 17 003 550 801

TABLE OF CONTENTS

1	INTRODUCTION	1
2	INVESTIGATION PROCEDURE	3
3	RESULTS OF INVESTIGATION	5
3.1	Site Description	5
3.2	Subsurface Conditions	6
3.3	Laboratory Test Results	9
4	COMMENTS AND RECOMMENDATIONS	9
4.1	Geotechnical Constraints	10
4.2	Dilapidation Survey	10
4.3	Subgrade Preparation	11
4.4	Excavation Conditions	11
4.4.1	Excavation Methods	11
4.4.2	Potential Vibration Risks	12
4.4.3	Seepage	13
4.5	Excavation Support	14
4.5.1	Support Methods	14
4.5.2	Retaining Wall Design Parameters	15
4.6	Engineered Fill	17
4.7	Footings	18
4.7.1	Proposed Building Layout (October 2009 Concept Design)	18
4.7.2	Proposed Building Layout (October 2010 Concept Design)	18
4.7.3	General	19
4.8	On-Grade Floor Slabs	21
4.9	Soil Aggression	21
4.10	Earthquake Design Parameters	22
4.11	Further Geotechnical Input	22
5	GENERAL COMMENTS	22

TABLE A: SUMMARY OF POINT LOAD STRENGTH INDEX TEST RSEULTS

TABLE B: SUMMARY OF LABORATORY CHEMICAL TEST RESULTS

BOREHOLE LOGS BH1 TO BH6 (INCLUDING COLOUR ROCK CORE PHOTOGRAPHS)

DYNAMIC CONE PENETRATION TEST RESULTS

FIGURE 1: BOREHOLE LOCATION PLAN

FIGURE 2: GRAPHICAL BOREHOLE SUMMARY

REPORT EXPLANATION NOTES

APPENDIX A: ENVIROLAB TEST REPORT (CERTIFICATE OF ANALYSIS 46129)

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Prince of Wales Hospital (POWH) Mental Health Intensive Care Unit development, located off Avoca Street, Randwick, NSW. The investigation was completed in accordance with the services outlined in the contract document, Contract Number HIC10026, between Jeffery and Katauskas Pty Ltd and the client, Health Infrastructure.

We understand from our discussions with the project structural engineer, Taylor Thomson Whitting (TTW) on 22 October 2010, that the proposed development is at a concept stage, with the final location, layout and size of the building yet to be finalised. An additional geotechnical investigation may be required depending on the final location of the building.

We note that the boreholes drilled as part of the investigation were nominated by TTW based on a concept design prepared in October 2009. The approximate building outline for the October 2009 concept design was presented on the supplied architectural drawings prepared by Leighton Irwin (Project No. 6684, Drawing Nos. SK-972, SK-974, SK-975, SK-976 and SK-977, Revision 2, dated 15 October 2009). The approximate outline of the October 2009 concept design is shown approximately on Figure 1. There were no finished floor levels (FFL) indicated on these supplied drawings. This option may include demolition of the existing single storey brick building at the north-western corner of the site.

We have also been supplied with revised concept architectural drawings prepared by BVN Architecture (Project No. S1009002, Drawing Nos. AR_MHICU_A_01, AR_MHICU_D00 and AR_MHICU_D01, all Issue 2, dated 19 October 2010). We have also been supplied with an unreferenced proposed basement layout drawing prepared by BVN Architecture, dated 25 October 2010. Based on the October 2010 concept design drawings prepared by BVN Architecture, we understand that a one and two storey building is proposed at the south-western corner of the site, as shown on Figure 1. The proposed ground floor level (Level 00) will be constructed at a FFL at reduced level (RL) 52.1m, which is about 2.5m above existing grade. A basement (Level B1) is proposed and will be constructed at a FFL at RL49.5m, which will be at or close to existing grade.

In the supplied *'Geotechnical and Environmental Investigation – Brief and Offer of Service'* prepared by TTW, dated 19 August 2010, column loads between 500kN and 2,000kN, have been indicated for the proposed building. There is also to be provision for construction of five additional levels in which case column loads between 1,500kN and 6,000kN could be expected.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at six borehole locations, and based on the results obtained, to present our comments and recommendations on subgrade preparation, retaining wall design parameters, footings, earthquake design parameters and slab-on-grade.

We were also commissioned to carry out a Stage 1 Preliminary Environmental Site Assessment. This work was carried out by Environmental Investigation Services (EIS) [the environmental consulting division of the Jeffery and Katauskas Group] who prepared a report, Ref: E24288KBrpt, dated November 2010. This geotechnical report must be read in conjunction with the above EIS report.

This report confirms and amplifies the preliminary geotechnical information and advice outlined in our email, dated 11 October 2010, sent to Mr Darren Jeffree of TTW. The borehole logs presented in this report supersede the 'draft' borehole logs and borehole location plan attached to the email.

2 INVESTIGATION PROCEDURE

Prior to the commencement of the fieldwork, the borehole locations were electromagnetically scanned by a specialist sub-contractor for buried services.

The fieldwork for the geotechnical investigation was carried out on 16 and 17 September 2010 and comprised the following scope of work:

- Five boreholes (BH1 to BH5) were auger drilled to depths between 2.58m (BH3) and 3.5m (BH5) using our track mounted JK250 drill rig. BH1 to BH4 were extended into the underlying sandstone bedrock using diamond coring techniques to final depths between 5.50m (BH3) and 6.50m (BH2) below existing grade;
- A sixth borehole (BH6) was drilled to a refusal depth of 0.55m, using a hand auger;
- A Dynamic Cone Penetrometer (DCP) test was carried out to a refusal depth of 1.55m adjacent to BH6; and
- A temporary 50mm diameter slotted PVC standpipe was installed into BH2 for groundwater monitoring during the fieldwork period. Additional groundwater observations were made in the other boreholes, both during and on completion of the fieldwork.

We note that refusal of the DCP equipment often indicates the depth to the underlying bedrock, however, due to the equipment's limitations, it may also refuse on obstructions within fill, tree roots, ironstone gravel bands or other "hard" layers within the soil profile, and not necessarily on bedrock.

The borehole locations were set out by tape measurements from existing surface features and apparent site boundaries and are shown on Figure 1. The approximate surface reduced levels (RLs) indicated on the attached borehole logs and DCP test results, were interpolated between spot levels and contour lines shown on the supplied survey plan prepared by Craig and Rhodes (Reference No. 096-10, dated 23 August 2010). The survey datum is Australian Height Datum (AHD). Figure 1 is based on one of the supplied architectural drawings prepared by BVN Architecture (Drawing No. AR_MHICU_A_01).

The relative compaction and density of the subsoil profile was assessed from the Standard Penetration Test (SPT) 'N' values and DCP test results. The strength of the upper weathered bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of recovered rock cuttings. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{s(50)}$) tests. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineer (Adrian Callus) was present on a full-time basis during the fieldwork to set out the borehole locations, direct the electromagnetic scanning, nominate the testing and sampling, direct the standpipe installation and to prepare the attached borehole logs and DCP test results, all under the direction of our Associate (Adrian Hulskamp). The Report Explanation Notes define the logging terms and symbols used.

The recovered rock cores were photographed and returned to a NATA registered laboratory [Soil Test Services Pty Ltd (STS)] for Point Load Strength Index testing. The photographs are enclosed facing the relevant cored borehole log. The Point Load Strength Index test results are summarised in the attached Table A. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table A.

Selected disturbed soil samples were recovered from site and submitted to Envirolab Services Pty Ltd for soil pH, sulphate and chloride tests. The results are summarised in the attached Table B. The Envirolab *'Certificate of Analysis'* is presented in Appendix A.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within slightly undulating topography, partway down a south facing hillside which grades at about 3° to 4°. The majority of the site, however, is relatively flat. The site is bound by Avoca Street to the east and Nurses Road to the west.

At time the time of the fieldwork, the southern side of the site was occupied by a 12 storey brick building (Vera Adderley Building) which was in the early stages of being demolished. An in-ground swimming pool was located on the northern side of the building. The ground surface surrounding the building and swimming pool exposed natural sands and sandy fill soils in areas which were not covered by crushed concrete to improve trafficability across the demolition site. Several scattered medium to tall trees were also located within the site.

An exposure of sandstone outcropped near the northern site boundary, as shown on Figure 1. The sandstone exposure was between about 1.5m and 3.7m high and exposed 'massive' distinctly weathered sandstone bedrock of at least medium rock strength, based on sounding with a geological hammer. There was evidence of staining from past groundwater seepage over the cliff face. Several sub-vertical planar joints and sub-horizontal bedding partings spaced at least 0.7m apart were observed in the rock face. We did not observe any signs of cliff face instability, however, our observations were limited in many areas by vegetation cover, particularly over the upper reaches of the cliff face. The sandstone outcrop may have been created, at least at its south-eastern end, by previous quarrying, as there was a single drill and blast hole observed in the face.

A single storey brick building was located at the north-western corner of the site, above the sandstone cliff face, and appeared to be in good condition based on a cursory inspection from within the site.

An open air asphaltic concrete (AC) surfaced carpark was located to the south of the site. A three storey above ground carpark was located along the western side of Nurses Road. An AC surfaced internal access road was located along the northern site boundary.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates that the site is underlain by medium to fine grained 'marine' sand, with podsols, close to the contact with the underlying Hawkesbury Sandstone. Our site observations and investigation results have confirmed the presence of the Hawkesbury Sandstone at the site.

In summary, the boreholes encountered shallow sandy fill overlying aeolian (dune) sands/silty sands with sandstone bedrock encountered at shallow to moderate depth, where the sandstone does not outcrop. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. A graphical borehole summary is presented as Figure 2.

Fill

Fill comprising silty sand was encountered from the ground surface in all boreholes and extended down to depths between 0.4m (BH1, BH2 and BH3) and 1.1m (BH4) below existing grade. The depth of fill in BH6 was not determined due to hand auger refusal on an obstruction. Inclusions of sandstone and igneous gravel, concrete and metal fragments, roots and root fibres slag and root fibres were encountered within the fill.

Due to the absence of inclusions other than root fibres within the fill in BH1, BH2 and BH5, these soils may be disturbed natural sands as a result of vehicular movements or other disturbances within the demolition site.

Based on the SPT 'N' values and DCP test results, the fill was assessed to be poorly and moderately compacted.

Natural Sands

Natural (aeolian) sands or silty sands were encountered below the fill in all boreholes, with the exception of BH6, and extended down to depths between 2.3m (BH3) and 3.05m (BH5). In BH5, a thin layer of clayey sand of possible residual origin was encountered just above the bedrock surface at 2.6m depth. Based on the SPT 'N' values, the sands were assessed to be predominantly very loose and loose.

Assuming natural sands are present just below the depth of fill proved in BH6, then the DCP test results infer loose sands down to the DCP refusal depth at 1.55m.

Sandstone Bedrock

Sandstone bedrock was encountered in all rig boreholes at depths between 2.3m (BH3) and 3.05m (BH5) below existing grade and extended down to the borehole termination depths. Sandstone bedrock was inferred at the DCP refusal depth of 1.55m in DCP6.

The sandstone bedrock in the rig boreholes was sub-horizontally bedded, fresh and of at least medium rock strength from first contact and down to borehole termination depths. The sandstone cliff line was also sub-horizontally bedded, but distinctly weathered and of at least medium rock strength and with widely spaced defects and is therefore quite competent.

Within the rock core samples, few defects were encountered but included inclined joints, bedding partings and clay seams, most of which were widely spaced. However, core loss zones were encountered in all rig boreholes and ranged in thickness between 0.10m and 0.42m. The core loss zones are inferred to be clay bands or extremely weathered bands, which have "washed" away during the coring process.

A preliminary engineering classification of the bedrock (in accordance with Pells et al. 1998) has been carried out and is tabulated below. The classification is based on the borehole results. We note that the engineering classification has not taken into account specific footing sizes, pile types, pile diameters and founding levels and are therefore only indicative. These preliminary classifications should be reviewed once footing sizes/pile types, pile diameters and founding levels have been selected to confirm the applicability within the zone of influence of such footings/piles.

BH	Depth (m)/Reduced Level (mAHD) at Top of Sandstone Class			
	Class V	Class IV	Class III	Class II
BH1	-	2.6/47.1	-	-
BH2	-	-	3.2/46.6	-
BH3	-	-	2.3/47.3	-
BH4	2.5/47.4	-	-	4.6/45.3
BH5*	-	3.05/46.55		

* Based on augered borehole only.

Groundwater

Groundwater seepage was encountered in all rig boreholes at depths between 1.0m (BH5) and 1.9m (BH2) below existing grade. Groundwater was measured at a depth of 1.4m on completion of augering in BH3. In BH1, BH2, BH4 and BH5, the sides of the boreholes collapsed to depths between 0.5m (BH5) and 1.8m (BH1) on completion of augering, which in sands is indicative of the ground water level. BH5 was 'dry' during and on completion of drilling.

In BH2 and BH3, groundwater was measured at a depth of 1.4m and 1.2m, respectively, on completion of coring. These levels are most likely influenced by the introduced drill flush water during rock coring. In all cored boreholes, there was full or near full return of the drill flush water, which indicates a relatively impermeable rock mass.

In the temporary PVC standpipe installed into BH2, groundwater was measured at 1.3m depth at 8:30am and 3:00pm on the 17 September 2010. No long term groundwater monitoring was carried out.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores correlated well with our field assessment of bedrock strength. The estimated UCSs generally ranged from 10MPa to 34MPa. However, a UCS of less than 1MPa was recorded for one sample tested from BH4 at 4.28m depth.

The results of the soil chemistry testing show the samples tested to be slightly alkaline (pH 7.1 to 7.6) and to have very low sulphate and chloride contents.

4 COMMENTS AND RECOMMENDATIONS

We understand that the proposed development is at a concept stage and that the location, layout and size of the building has yet to be finalised. Therefore, the comments and recommendations below are of a general nature and should be reviewed prior to finalising the structural design.

We recommend that once the architectural details have been finalised, particularly the location of the proposed building, Jeffery and Katauskas be contacted to assess the need or otherwise for an additional geotechnical investigation. The purpose of the additional investigation would be to further assess the subsurface conditions, if the building is located as per the current concept design or away from areas where there is currently no subsurface information. The additional investigation may not be warranted if the proposed layout is similar to the October 2009 layout, as shown on Figure 1. Further geotechnical advice in relation to additional investigations is discussed in Section 4.7.2.

4.1 Geotechnical Constraints

Depending on the location of the proposed building, the primary geotechnical constraint associated with the proposed development will be to reduce the risk of vibration induced damage to adjacent buildings during excavation and construction, particularly if rock excavation is required. This constraint could arise if the October 2009 concept layout is adopted, as the proposed building may extend beyond and into the existing sandstone cliff face.

Furthermore, there will be a need to maintain stability to the adjacent single storey building, during excavation and construction and in the long term.

The above constraints are addressed in the following sections of this report.

4.2 Dilapidation Survey

We recommend that prior to bulk excavation, if required, a detailed dilapidation report be completed on the adjacent single storey brick building at the north-western corner of the site. The owner of the building should be asked to sign a copy of the dilapidation report, to confirm that the reports present a fair assessment of existing conditions. The dilapidation report can then be used as a benchmark in assessing potential future damage claims, arising from the works.

4.3 Subgrade Preparation

Construction of the proposed building will require demolition of the existing Vera Alderley building and swimming pool shell. There will also be a requirement to strip all grass and vegetation and remove any existing trees (including their root balls) which are present within the proposed building footprint.

Following the above, any topsoil (if present), root affected soils and any deleterious or contaminated existing fill (if present) should be stripped and disposed appropriately offsite. Stripped topsoil and root affected soils should be stockpiled separately as they are not suitable for reuse as engineered fill but may be reused for landscaping purposes. For guidance on the offsite disposal of soil, reference should be made to the EIS report.

Following demolition of the swimming pool shell, the swimming pool excavation should be backfilled using engineered fill as discussed in Section 4.6, if used as a structural supporting roll.

We note that if the lowest floor level of the building is designed as a suspended slab, then there would only be a need to backfill the swimming pool excavation and then nominally compact the soils using a bucket fitted to a large hydraulic excavator.

4.4 Excavation Conditions

4.4.1 Excavation Methods

Excavation to achieve design levels, if required, through the underlying sandy soils can be readily excavated using buckets fitted to a hydraulic excavator.

However, if excavation of any low or higher strength sandstone bedrock is required, then the bedrock would be most effectively excavated using hydraulic impact rock hammers, after initially forming a perimeter saw cut slot, fitted to a hydraulic excavator. The hydraulic impact rock hammers would also be required for breaking up of boulders or blocks or for trimming rock excavation side slopes.

Care will be required during excavation of the bedrock, if required, to avoid undermining or removing support from the single storey brick building located at the north-western corner of the site.

We have assumed that excavation will not be required below the groundwater table at this site. Further geotechnical advice must be obtained should this occur.

4.4.2 Potential Vibration Risks

We recommend that considerable caution be taken during rock excavation as there will likely be direct transmission of ground vibrations to the single storey brick building at the north-western corner of the site and other nearby buildings inside the hospital grounds.

Excavation procedures and the dilapidation report should be carefully reviewed prior to the commencement of excavation so that appropriate equipment is used.

The excavation with hydraulic impact rock hammers should employ a moderately sized excavator fitted with a relatively low energy hydraulic rock hammer no larger than say, a Krupp 600 size or equivalent. To reduce the transmission of vibrations, we recommend that a perimeter vertical saw cut slot be provided, and the base of the slot maintained at a lower level than the adjoining rock excavation at all times. We further recommend that continuous vibration monitoring be carried out during rock excavations. Subject to review of the dilapidation report, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec on the adjoining single storey. This limit takes both human comfort and potential structural damage into account. If it is found that transmitted vibrations are excessive, then it would be necessary to use a smaller rock hammer or alternative excavation techniques. The use of a rotary grinder or grid sawing in conjunction with ripping and hammering present alternative low vibration excavation techniques.

When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

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

- Maintain the rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate hammer in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

4.4.3 Seepage

Based on the investigation results, we do not expect significant groundwater seepage flows into the excavation cuts through the sandstone cliff face. However, during or after periods of prolonged wet weather, groundwater seepage may occur at the soil/bedrock interface and through joints and bedding planes within the completed cut faces.

Seepage, if any, during excavation through the bedrock above the groundwater table is expected to be satisfactorily controlled by conventional sump and pump techniques or gravity drainage.

We recommend that groundwater seepage into the excavation be monitored by site personnel and the results (quantity, location, source, etc) reported to the geotechnical and hydraulic engineers so that any unexpected conditions can be promptly addressed.

Elsewhere across the site, the expected permeability of the underlying sands is such that any seepage into any localised excavations above the groundwater table or any water ingress from rainfall at ground surface level should drain naturally by infiltration.

4.5 Excavation Support

4.5.1 Support Methods

For shallow cuts through the sandy soils at the site (less than 1m high) we recommend that temporary batter slopes through the sands be cut at not steeper than 1 Vertical (V) on 1.5 Horizontal (H). Retaining walls can then be constructed along the toe of the temporary batters and subsequently backfilled. If deeper cuts through the soils are proposed, then further geotechnical advice must be sought.

All surcharge loads must be kept well clear of the batter slope crests i.e. at least the height of the soil cut away from the crest.

If seepage occurs at the soil/bedrock interface then localised instability at the toe of the soil batters may occur and therefore an allowance should be made for sandbag support at the toe of the batters.

We expect that sandstone bedrock of low or higher strength can be cut vertically. However, localised stabilisation measures may be necessary if adverse defects such as inclined joints are found. However, based on the good quality of the sandstone exposed in the cliff face, we do not expect significant stabilisation measures will be required. However, 'dental' treatment may be required for clay seams, extremely weathered bands, etc. Nevertheless, we recommend that the rock faces be progressively inspected by a geotechnical engineer at 1.5m depth intervals, to identify adverse defects and to assess appropriate stabilisation measures, such as rock bolting, shotcreting etc. A small provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures.

A toe drain should be provided at the base of all rock cuttings to collect groundwater seepage and lead it to a sump for pumping or gravity discharge to the stormwater system.

4.5.2 Retaining Wall Design Parameters

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of the retaining walls, if required:

- For allowable bearing pressure recommendations, reference should be made to Section 4.7 below.
- For free-standing cantilever walls which are retaining areas where movement is of little concern (such as landscaped areas), a triangular lateral earth pressure distribution may be adopted with an 'active' earth pressure coefficient, K_a, of 0.35, assuming a horizontal retained surface.

- For cantilever walls where the tops are restrained by the permanent structure or which retain areas where movement is of some concern or for propped walls, a triangular lateral earth pressure distribution should be adopted with an 'at rest' earth pressure coefficient, K_o, of 0.55, assuming a horizontal retained surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil profile.
- All surcharge loads affecting the walls (eg. adjacent footings, construction loads, live loads etc) should be taken into account in the design, using the appropriate earth pressure coefficient from above. If inclined retained surfaces are proposed, then they should be treated as a surcharge.
- The passive toe resistance for retaining walls founded in soil may be estimated using a 'passive' earth pressure coefficient (K_p) of 2.8 for loose sands (but with a Factor of Safety of at least 2.0 to limit deformations), assuming horizontal ground in front of the wall. The passive pressure due to the upper 0.3m below bulk excavation level should be ignored in the analyses to take excavation disturbance and tolerances into account.
- At the northern end of the site where the rock outcrops, lateral toe restraint in that area may be achieved by fixing the walls to sandstone bedrock above bulk excavation level using rock dowels or keying the retaining walls into sandstone bedrock below bulk excavation level, if appropriate. An allowable bond/lateral stress of 250kPa may be adopted for dowel/key design, provided the rock is at least low strength and no excavation is carried out in the passive zone. The dowels must be of sufficient length to engage a volume of rock to give global stability against sliding and overturning. The embedded portion of the dowels should be grouted into holes which have been drilled back into the cut face below a line at 30° to the vertical.

4.6 Engineered Fill

Engineered fill would be required where ground surface levels need to be raised.

Materials preferred for use as engineered fill are well graded granular materials such as ripped or crushed sandstone, which are 'clean', free of organic matter and free of particle sizes greater than 75mm. The benefit of granular materials is that their compaction specification and controls are less stringent than for clayey soils.

Any site-won excavated sand or sandstone bedrock may be reused as engineered fill, on condition the materials are free of organic matter and free of particle sizes greater than 75mm. The excavated sandstone would most likely require crushing to conform to the maximum particle size requirement.

Engineered fill comprising ripped or crushed sandstone or sand, should be compacted in maximum 200mm loose layers using a large static roller to a minimum density ratio of 98% of Standard Maximum Dry Density (SMDD) or density index (I_D) of 70%. The density ratio could be relaxed to 95% of SMDD, if only landscaped gardens are proposed above backfilled swimming pool excavation.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. The frequency of density testing for engineered fill should be at least one test per layer per $250m^2$, or three tests per visit, whichever requires the most tests. We recommend that at least a Level 2 control of fill compaction be adhered to on this site. The geotechnical testing and inspection authority (GTA) should be directly engaged by the client and not by the earthworks contractor.

4.7 Footings

4.7.1 Proposed Building Layout (October 2009 Concept Design)

Based on the moderate to high column loads and the shallow to moderate depth to the underlying sandstone bedrock (where the bedrock does not outcrop), we recommend that the proposed new building be uniformly founded within the underlying sandstone bedrock.

Based on the subsurface conditions, continuous flight auger (CFA) piles, otherwise known as grout injected auger piles, would be the preferred pile type. Conventional bored piles would not be suitable due to the collapsing nature of the sands and shallow groundwater level. Pad or strip footings would be suitable at the northern end of the proposed building where the rock outcrops.

For CFA piles socketted a nominal 0.3m into Class IV or better quality sandstone bedrock or for pad or strip footings, a maximum allowable end bearing pressure of 3MPa would be applicable, based on serviceability.

Where high level pad or strip footings are located directly behind the crest of a sandstone face, the sandstone exposed below the toe of the footing must be inspected by a geotechnical engineer to identify any adverse defects or presence of poor quality bedrock, so that the bearing pressure for each particular footing can be assessed. For design purposes a reduced maximum allowable end bearing pressure of 1MPa would apply, provided the bedrock is of at least low strength.

4.7.2 Proposed Building Layout (October 2010 Concept Design)

At this stage, there is only limited subsurface information (BH5) in the area of the proposed October 2010 building footprint and is limited to the north-eastern corner of the proposed building where the courtyard is proposed. Nevertheless, we would expect the subsurface conditions to be similar to that encountered within the remaining boreholes, although the bedrock levels in particular and quality could vary from what was encountered in the boreholes.

For preliminary design purposes, we recommend that CFA piles socketted a nominal 0.3m into the underlying sandstone bedrock be designed for a maximum allowable end bearing pressure of 3MPa, based on serviceability and subject to completion of an additional geotechnical investigation which should comprise at least four cored boreholes. We would be happy to prepare a fee proposal for this additional work, if requested.

4.7.3 General

The above maximum allowable bearing pressures are based on serviceability which results in settlement of less than 1% of the minimum footing dimension.

Deeper sockets may be designed using an allowable shaft adhesion value of 10% in compression and 5% for tension of the above maximum allowable end bearing pressures, respectively, for piled footings.

Due to the presence of medium or higher strength sandstone bedrock, relatively slow penetration rates should be expected during the drilling of rock sockets. The prospective piling contractors should be provided with a full copy of this report so that an appropriate high torque drilling rig and CFA drill bits are brought to site, especially if deeper rock sockets are required.

The initial stages of piling should be compared to the borehole information by a geotechnical engineer to confirm that founding depths are consistent with the borehole data.

High level pad or strip footings should be excavated, inspected and poured within minimal delay.

Footings on the sandstone bedrock may also be designed using "Limit State Design" principles as detailed by Pells et al. (1998), subject to completion of the additional investigation, if the proposed building is to be located as per the October 2010 concept design. An ultimate bearing capacity of 20MPa and an ultimate shaft adhesion value of 800kPa (compression) could be adopted for the Class III or better quality sandstone bedrock, where piles are used. Similarly, ultimate bearing capacities of 10MPa and 3MPa and ultimate shaft adhesion values of 500kPa and 100kPa (compression) could be adopted for the Class IV and Class V sandstone bedrock, respectively, where piles are used.

Settlement limitations to the structure will need to be satisfied and can be estimated using elastic modulus values of 350MPa, 200MPa and 100MPa for the Class III or better quality sandstone, Class IV and Class V sandstone bedrock, respectively.

It should be noted that such ultimate bearing pressures must be used in conjunction with an appropriate *"Geotechnical Strength Reduction Factor"* (ϕ_g). Provided there is good workmanship and quality control during footing construction, we recommend that a ϕ_g value of not greater than 0.5 be adopted for end bearing and shaft adhesion.

We note the above design recommendations for bearing and shaft adhesion are contingent on achieving good construction practice and an appropriate inspection and test plan.

4.8 On-Grade Floor Slabs

Based on the investigation results, slab-on-ground construction is feasible, as we expect predominantly natural sands or a shallow sandy fill to be exposed.

The exposed subgrade should be proof rolled with at least six passes of a large sized (preferably at least ten tonnes by dead-weight) smooth drum roller. The last two passes should be under the direction of an experienced earthworks superintendent or geotechnical engineer. The objective of the proof rolling is improve the near surface compaction of the sands and to assist in the detection of unstable areas. If any unstable areas are exposed during proof rolling, these areas should be removed down to a sound base and replaced with engineered fill.

We note that if suspended floor slabs are adopted, then there is no necessity for proof rolling.

The on-grade floor slabs should be constructed independently of building footings, perimeter walls, columns etc (i.e. designed as a 'floating' slab). Slab joints should be capable of resisting shear forces but not bending moments by providing dowelled or keyed joints.

Underfloor drainage would not be required due to the free draining nature of the underlying sandy soils.

4.9 Soil Aggression

Based on the soil chemistry test results, a "Non-aggressive" exposure classification results in accordance with AS2159-2009.

4.10 Earthquake Design Parameters

Based on the investigation results and in accordance with AS1170.4–2007, a Hazard Factor (Z) of 0.8 is applicable for the site, together with a subsoil Class B_e for structures founded on, or in, rock and a subsoil Class C_e for structures founded in the soil profile.

4.11 Further Geotechnical Input

We summarise below the recommended additional geotechnical input that needs to be carried out:

- Following finalisation of the architectural details and after the location of the proposed building has been confirmed, Jeffery and Katauskas should be contacted to assess the need or otherwise for an additional geotechnical investigation at the site.
- Dilapidation survey of the existing single storey brick building at the north-western corner of the site.
- Geotechnical footing inspections, if appropriate.
- Proof rolling inspections, if appropriate.
- Density testing of engineered fill, if required.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and 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.

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

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

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of 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.

Adrian Hulskamp
Associate

Bruce Walker
Principal
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

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

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	3.07-3.10	0.7	14
	3.82-3.85	0.8	16
	4.11-4.15	1.3	26
	4.84-4.87	0.9	18
	5.62-5.65	1.6	32
2	3.83-3.86	0.8	16
	4.24-4.27	0.8	16
	4.89-4.92	1.7	34
	5.40-5.43	0.9	18
	6.20-6.23	1.1	22
3	3.13-3.16	0.8	16
	3.94-3.97	0.5	10
	4.33-4.36	0.9	18
	5.28-5.32	1.5	30
4	3.16-3.20	0.9	18
	3.85-3.88	0.6	12
	4.28-4.32	0.05	<1
	5.15-5.18	1.1	22
	5.86-5.90	0.9	18

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. = $20\ I_{S\,(50)}$

TABLE B SUMMARY OF LABORATORY CHEMICAL TEST RESULTS SOIL pH, SULPHATE AND CHLORIDE CONTENTS					
Borehole Number	**Sample Depth (m)**	**Sample Description**	**pH Units**	**Sulphate (mg/kg)**	**Chloride (mg/kg)**
BH1	0.5 - 0.95	Sand	7.3	<2.0	4.7
BH1	1.5 - 1.95	Sand	7.1	<2.0	4.5
BH4	1.5 - 1.95	Sand	7.6	4.7	4.0

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1/2

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH
Date: 17-9-10
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 49.7m
Datum: AHD
Logged/Checked by: A.P.C./ASH

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES		0			FILL: Silty sand, fine to coarse grained, dark brown, with root fibres.	M			GRAVEL COVER
	ES, DS	N = 8 4,3,5			SP	SAND: fine to coarse grained, yellow brown, with a trace of root fibres and silt fines.	M	L	-	AEOLIAN
			1							
	DS	N = 2 1,1,1						VL		
(groundwater level)			2				W			
					-	SANDSTONE: fine to medium grained, light grey.	FR	M	-	MODERATE TO HIGH 'TC' BIT RESISTANCE
			3			REFER TO CORED BOREHOLE LOG				
			4							
			5							
			6							
			7							


JEFFERY & KATAUSKAS PTY LTD
Job No: 24288WH
BH1
Start Coring at 2.89m
2
3
CL 0.1m
4
5
CL 0.1m
End of Borehole at 5.97m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. 1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH | **Core Size:** NMLC | **R.L. Surface:** ≈ 49.7m
Date: 17-9-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.P.C./ASH

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 2.89m						
FULL RETURN		3		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-5°.	FR	M	× ×			
				CORE LOSS 0.10m						
50% RETURN		4		SANDSTONE: fine to medium grained, light grey, with occasional grey laminae, bedded at 0-5°.	FR	M-H	× ×		- Be, 10°, P, S - J, 60°, P, R	
		5		CORE LOSS 0.10m						
				SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-15°.	FR	M-H	×		- Be, 10°, P, S, CLAY VENEER - CS, 10mm.t	
		6		END OF BOREHOLE AT 5.97m						
		7								
		8								
		9								

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **2**

1/2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT

Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH

Date: 16-9-10

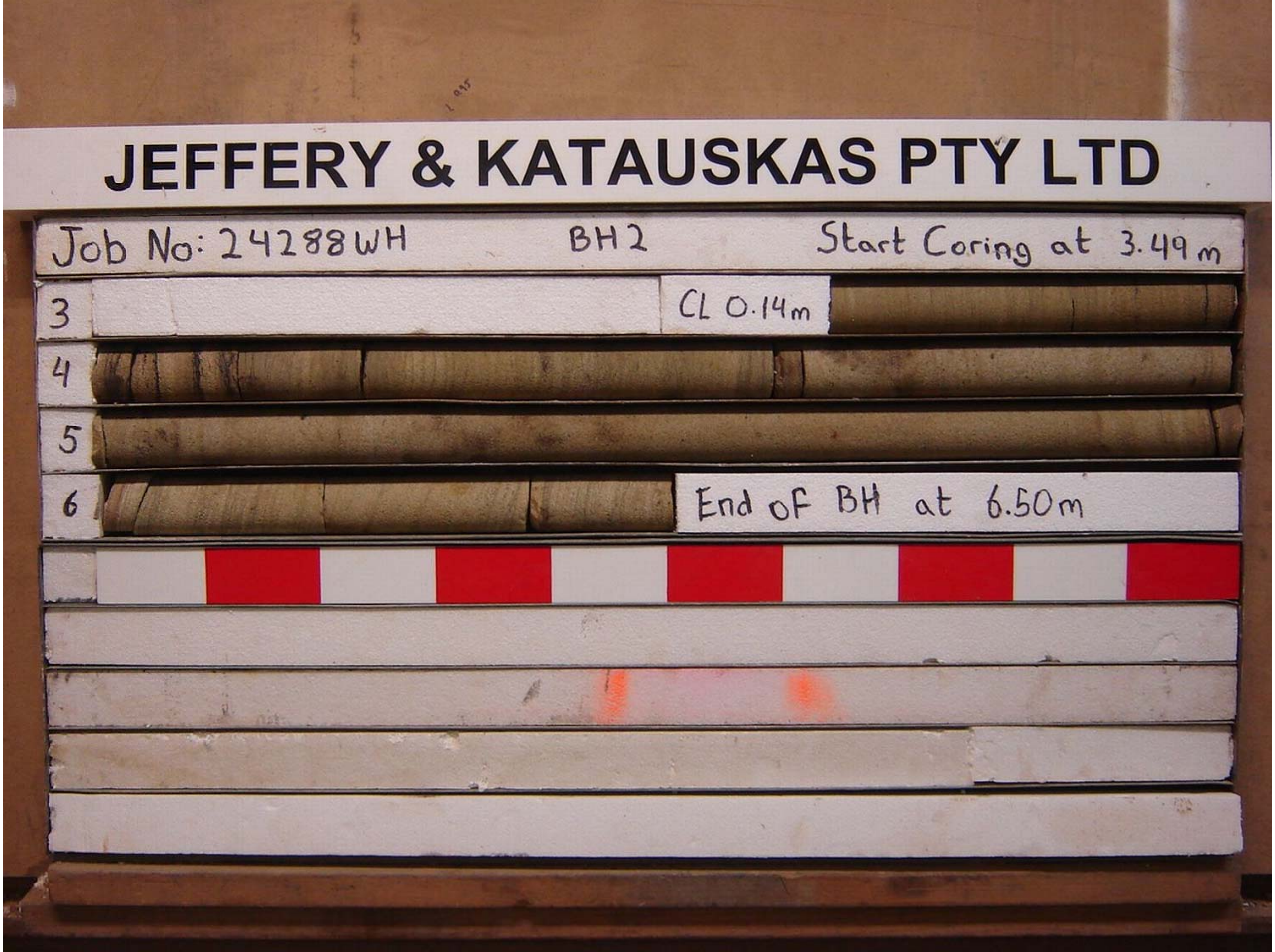
Method: SPIRAL AUGER JK250

R.L. Surface: ≈ 49.8m

Datum: AHD

Logged/Checked by: A.P.C./ AJH

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty sand, fine to medium grained, dark brown.	M			GRAVEL COVER
ON COMPLETION OF CORING	N = 15 6,8,7	1	SP	SAND: fine to coarse grained, yellow brown, with a trace of silt fines.	M	MD	-	AEOLIAN
	N = 5 3,2,3	2			W	L		
	N > 5 9,5/50mm REFUSAL	3	-	SANDSTONE: fine to medium grained, light grey.	FR	M	-	MODERATE 'TC' BIT RESISTANCE
		4		REFER TO CORED BOREHOLE LOG				50MM DIA. PVC STANDPIPE INSTALLED TO 6. 5M DEPTH. SLOTTED BETWEEN 0.9M AND 6.5M. BENTONITE PLUG TO 0.5M DEPTH THEN BACKFILL TO SURFACE. REFER TO TEXT OF REPORT FOR ADDITIONAL GROUNDWATER MEASUREMENTS DURING THE FIELDWORK
		5						
		6						
		7						


JEFFERY & KATAUSKAS PTY LTD
Job No: 24288WH
BH2
Start Coring at 3.49m
3
CL 0.14m
4
5
6
End of BH at 6.50m

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2

CORED BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT

Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH | **Core Size:** NMLC | **R.L. Surface:** ≈ 49.8m

Date: 16-9-10 | **Inclination:** VERTICAL | **Datum:** AHD

Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.P.C./MH

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 3.49m						
				CORE LOSS 0.14m						
50% RETURN		4		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-5°.	FR	M	× ×			
		5		SANDSTONE: fine to medium grained, light grey, massive.		M-H	× ×			
		6		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°.			×			
		7, 8, 9		END OF BOREHOLE AT 6.50m						

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **3**
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH
Date: 17-9-10
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 49.6m
Datum: AHD
Logged/Checked by: A.P.C./AJH

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES		0			FILL: Silty sand, fine to coarse grained, dark brown, with coarse grained igneous gravel, concrete and metal fragments, roots and root fibres.	M			GRAVEL COVER
ON COMPLETION OF CORING	DS	N = 8 4,4,4	1		SP	SAND: fine to coarse grained, orange brown and yellow brown.	M	L	-	AEOLIAN
ON COMPLETION OF AUGERING	DS	N = 9 4,4,5	2		SM	SILTY SAND: fine to medium grained, light grey and orange brown, with a trace of clay fines.	W			
	DS				-	SANDSTONE: fine to medium grained, light grey.	FR	M		MODERATE 'TC' BIT RESISTANCE
						REFER TO CORED BOREHOLE LOG				
			3							
			4							
			5							
			6							
			7							


JEFFERY & KATAUSKAS PTY LTD
Job No 24288WH
BH3
Start Coring at 2.58m
2
CL 0.13m
3
4
5
End of BH at 5.55m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **3**
2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH | **Core Size:** NMLC | **R.L. Surface:** ≈ 49.6m
Date: 17-9-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.P.C./ASH

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 2.58m						
				CORE LOSS 0.13m						
FULL RET-URN		3 4 5		SANDSTONE: fine to medium grained, light grey, with a trace of dark grey laminae, bedded at 0-10°.	FR	M H	× × × ×		- Be, 5°, Un, R - CS, 10mm.t - CS, 2mm.t - Be, 20°, Un, R - Be, 20°, Un, R, IS - J, 80°, P, R	
		6 7 8 9		END OF BOREHOLE AT 5.50m						

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **4**

1/2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT

Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH

Date: 17-9-10

Method: SPIRAL AUGER JK250

Logged/Checked by: A.P.C./AJH

R.L. Surface: ≈ 49.9m

Datum: AHD

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES		0			FILL: Silty sand, fine to coarse grained, dark brown, with coarse grained sandstone and igneous gravel, roots and root fibres.	M			GRAVEL COVER APPEARS MODERATELY COMPACTED
	DS	N = 8 3,5,3								
			1		SP	SAND: fine to coarse grained, yellow brown.	M	L	-	AEOLIAN
▼	DS	N = 9 3,4,5					W			
			2							
	DS				-	SANDSTONE: fine to medium grained, grey and orange brown.	SW-FR	M	-	LOW TO MODERATE 'TC' BIT RESISTANCE
			3			REFER TO CORED BOREHOLE LOG				
			4							
			5							
			6							
			7							


JEFFERY & KATAUSKAS PTY LTD
Job No 24288WH
BH4
Start Coring at 3.00m
3
Core loss 0.42m
4
5
6
End of Hole at 6.12m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **4**
2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH **Core Size:** NMLC **R.L. Surface:** ≈ 49.9m
Date: 17-9-10 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK250 **Bearing:** - **Logged/Checked by:** A.P.C./AJH

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 3.00m						
50% RETURN		3		SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	FR	M	×		- J, 20°, Un, R - J, 50°, Un, R	
				CORE LOSS 0.42m						
		4		SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	FR	M	×		- IRON INDURATED SEAM, 20mm.t	
				SANDSTONE: fine to medium grained, mid grey, with dark grey laminae, bedded at 0-5°.	DW	VL	×		- CS, 2mm.t	
		5		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.	FR	M-H	× ×			
		6		END OF BOREHOLE AT 6.12m						
		7								
		8								
		9								

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH
Date: 16-9-10
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 49.6m
Datum: AHD
Logged/Checked by: A.P.C./MW

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
			0			FILL: Silty sand, fine to coarse grained, dark grey brown, with root fibres.	M			
		N = 10 4,5,5			SP	SAND: fine to medium grained, yellow brown.	M	L	-	AEOLIAN
			1				W			
		N = 2 2,1,1						VL		
			2							
		SPT 5/50mm REFUSAL			SC	CLAYEY SAND: fine to medium grained, dark grey.				POSSIBLY RESIDUAL ?
			3		-	SANDSTONE: fine to medium grained, light grey.	SW-FR	M	-	MODERATE 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 3.5m				
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **6**
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED POWH MENTAL HEALTH INTENSIVE CARE UNIT
Location: PRINCE OF WALES HOSPITAL, AVOCA ST, RANDWICK, NSW

Job No. 24288WH **Method:** HAND AUGER **R.L. Surface:** ≈ 55.1m
Date: 17-9-10 **Datum:** AHD
Logged/Checked by: A.P.C./AJH

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					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to coarse grained, dark brown, with fine to coarse grained sandstone and igneous gravel, brick fragments and roots.	M			WEED COVER APPEARS POORLY COMPACTED
									END OF BOREHOLE AT 0.55m				HAND AUGER REFUSAL ON OBSTRUCTION IN FILL
						1							
						2							
						3							
						4							
						5							
						6							
						7							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	HEALTH INFRASTRUCTURE	
Project:	PROPOSED NEW BUILDING	
Location:	PRINCE OF WALES HOSPITAL, MENTAL HEALTH INTENSIVE CARE UNIT, RANDWICK	
Job No.	24288WH	Hammer Weight & Drop: 9kg/510mm
Date:	17-9-10	Rod Diameter: 16mm
Tested By:	A.D.C.	Point Diameter: 20mm

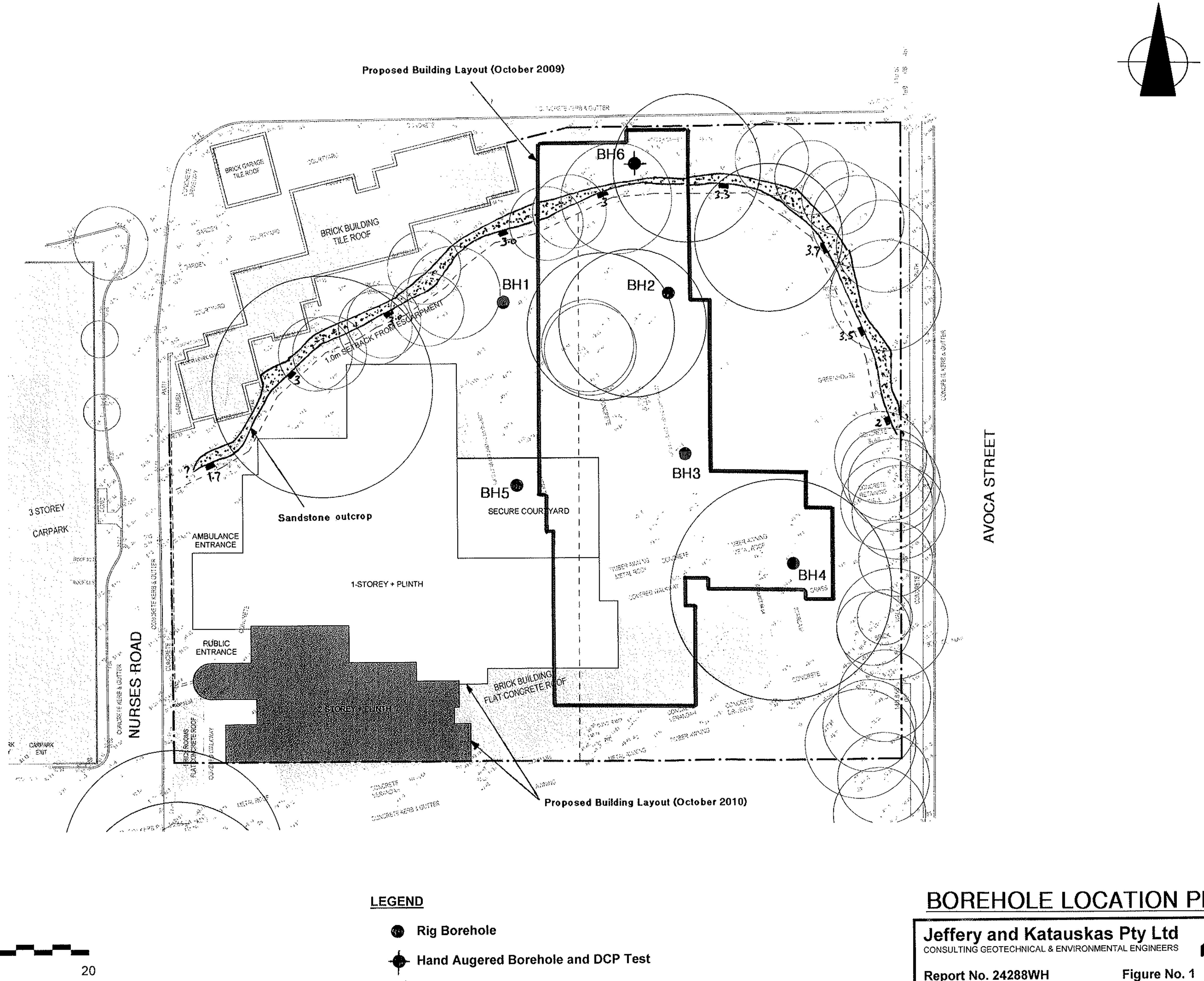
Number of Blows per 100mm Penetration

Test Location	RL ~55.1m						
Depth (mm)	**6**						
0 - 100	1						
100 - 200							
200 - 300							
300 - 400	1						
400 - 500							
500 - 600							
600 - 700	2						
700 - 800	2						
800 - 900	2						
900 - 1000	2						
1000 - 1100	3						
1100 - 1200	4						
1200 - 1300	4						
1300 - 1400	4						
1400 - 1500	5						
1500 - 1600	8/50mm						
1600 - 1700	REFUSAL						
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:

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

Ref: Scala3.xls April 99



LEGEND

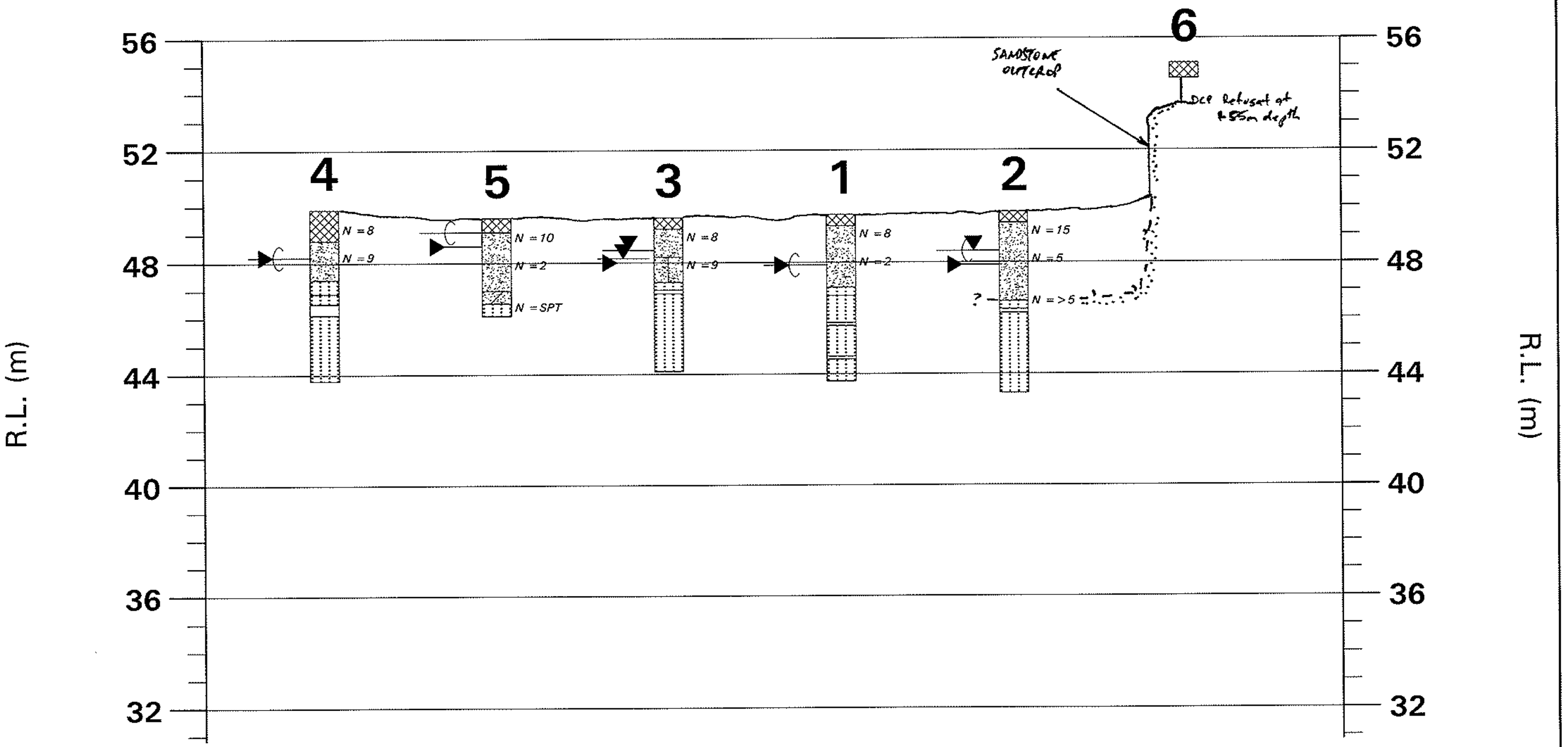
- Rig Borehole
- Hand Augered Borehole and DCP Test
- 3.0 Denotes approximate height of sandstone outcrop

BOREHOLE LOCATION PLAN

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 24288WH **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY



Sandstone/ Greywacke	Sand	Borehole Collapse Depth	Observed water level	Nc	SOLID CONE BLOW COUNTS PER 150mm		
Core Loss/ Empty	Clayey Sand	Groundwater seepage level		N	SPT "N" VALUE		
Fill	Silty Sand						

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 24288WH **Figure No.:** 2

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

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 sub-surface 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.

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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

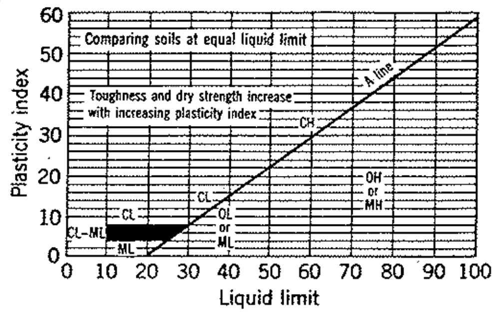
- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

Major divisions			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	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	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): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows: Less than 5% — *GW, GP, SW, SP*; More than 12% — *GM, GC, SM, SC*; 5% to 12% — *Borderline* cases requiring use of dual symbols.

Major divisions		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
Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	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.

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		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

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

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	

APPENDIX A

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

CERTIFICATE OF ANALYSIS 46129

Client:
Environmental Investigation Services
PO Box 976
North Ryde BC
NSW 1670

Attention: Adrian Callus

Sample log in details:

Your Reference:	**24288WH, Randwick**
No. of samples:	3 Soils
Date samples received:	21/09/10
Date completed instructions received:	21/09/10

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

Report Details:

Date results requested by:	28/09/10
Date of Preliminary Report:	Not issued
Issue Date:	28/09/10

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

Results Approved By:

Nick Sarlamis
Inorganics Supervisor

Miscellaneous Inorg - soil Our Reference: Your Reference Depth Date Sampled Type of sample	UNITS ------------- ------------	46129-1 BH1 0.5-0.95 17/09/2010 Soil	46129-2 BH1 1.5-1.95 17/09/2010 Soil	46129-3 BH4 1.5-1.95 17/09/2010 Soil
Date prepared	-	22/9/2010	22/9/2010	22/9/2010
Date analysed	-	22/9/2010	22/9/2010	22/9/2010
pH 1:5 soil:water	pH Units	7.3	7.1	7.6
Chloride, Cl 1:5 soil:water	mg/kg	4.7	4.5	4.0
Sulphate, SO4 1:5 soil:water	mg/kg	<2.0	<2.0	4.7

Method ID	Methodology Summary
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
LAB.81	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 21st ED, 4110-B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			22/9/2010	[NT]	[NT]	LCS-1	22/9/2010
Date analysed	-			22/9/2010	[NT]	[NT]	LCS-1	22/9/2010
pH 1:5 soil:water	pH Units		LAB.1	[NT]	[NT]	[NT]	LCS-1	99%
Chloride, Cl 1:5 soil:water	mg/kg	2	LAB.81	<2.0	[NT]	[NT]	LCS-1	106%
Sulphate, SO4 1:5 soil:water	mg/kg	2	LAB.81	<2.0	[NT]	[NT]	LCS-1	91%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job
Asbestos counting was analysed by Approved Counter:	@ERROR
Asbestos counting was authorised by Approved Signatory:	@ERROR

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

Quality Control Definitions

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

Laboratory Acceptance Criteria

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

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.