REPORT

TO

PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

ON

GEOTECHNICAL INVESTIGATION

FOR

STAGE 2, NEUROSCIENCE RESEARCH PRECINCT

AT

PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

BARKER STREET, RANDWICK, NSW

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Stage 2 expansion works for the Prince of Wales Medical Research Institute (POWMRI). The investigation was commissioned by Mr Rod Winton of Winton Associates Pty Ltd on behalf of POWMRI by letter dated 30 October 2008. The commission was on the basis of our proposals (Ref: P15792Zemail3) dated 24 October 2008, and (Ref: 22529Z BIT prop) dated 4 November 2008.

Based on the Taylor Thomson Whitting (TTW) brief (Ref: 061652) dated 2 July 2008 and the attached unreferenced architectural concept plans, we understand that it is proposed to construct a new medical research building with towers of two to 11 storeys above ground with two levels of basement carparking across the entire site. The proposed development is to take place in a number of stages with only the Stage 2 works currently being undertaken. The proposed lower carpark level is to have a finished floor reduced level (RL) at 38.5m with some local excavation to RL 35.5m and is anticipated will require excavation to depths between 7m and 16m below existing surface levels. Column working loads of between 8,000kN and 12,000kN have been indicated for the proposed buildings.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at 11 nominated locations as a basis for comments and recommendations on excavation conditions and methodology, vibration issues, retention options/design, footing options/design, earthquake design parameters, basement slab on grade, stormwater disposal using absorption trenches, and other geotechnical issues associated with the proposed works.

An environmental investigation was carried out concurrently with the geotechnical investigation by Environmental Investigation Services (EIS), our environmental division. The environmental report (Ref: E22529K) should be read in conjunction with the geotechnical report.

We note that Jeffery and Katauskas have previously carried out several geotechnical investigations within the Prince of Wales Hospital. A summary of the results of relevant investigations was presented in our report (Ref: 21902Zrpt) dated 14 February 2008.

2 INVESTIGATION PROCEDURE

Nine boreholes (BH2 to BH9 and BH16) were drilled to depths between 4.33m and 8.12m using spiral augering techniques with our truck mounted JK500 drill rig and track mounted JK250 and JK300 drill rigs. All of the above boreholes were subsequently extended to final depths between 11.85m and 16.36m using rotary diamond coring techniques with water flush. Two additional boreholes (BH3A and BH17A) were hand augered to a depth of 2m below existing surface levels. The latter boreholes were augmented by two Dynamic Cone Penetration (DCP) tests (DCP3A and DCP17A). Prior to drilling commencing, the borehole locations were electromagnetically scanned for buried services.

The borehole locations, as shown on the attached Investigation Location Plan (Figure 1) were set out by taped measurements from existing surface features. The surface reduced levels (RLs) of the boreholes were estimated by interpolation between the spot heights and ground levels shown on the supplied survey plan (Job Ref: 30744, dated 15/11/07), the originator of which was not clear. The survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsurface soils and rocks were assessed by logging the materials recovered during drilling. The relative density of the sandy subsoils was assessed from the Standard Penetration Test (SPT) 'N' values and the DCP tests. In the augered portion of the boreholes, the strength of the sandstone bedrock was assessed by observation of the drilling resistance of a tungsten carbide (TC) bit attached to the auger, together with examination of the recovered rock chip

samples and subsequent correlation with laboratory moisture content testing. The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory point load strength index tests.

Groundwater observations were made during augering, on completion of augering and on completion of coring individual boreholes. PVC standpipe piezometers were installed in BH4 and BH16 on completion of coring to allow for long term groundwater monitoring, if required. No longer term groundwater monitoring was undertaken. Temporary PVC standpipes were installed in BH3A and BH17A and constant head infiltration testing was carried out. The test results are presented on the attached Constant Head Permeability Test sheets.

Our geotechnical engineer, Adrian Ivor, was present full time during the fieldwork and set out the borehole locations, directed the electromagnetic scanning, nominated the sampling and testing, logged the subsurface profile and carried out the infiltration tests. The borehole logs and DCP test results are attached to this report together with the Report Explanation Notes, which describe the investigation techniques adopted and define the logging terms and symbols used.

Selected soil and rock chip samples were submitted to Soil Test Services Pty Ltd (STS), a NATA registered laboratory, for moisture content and particle size distribution determination. The test results are summarised in Tables A and B. The recovered rock core was photographed and select sections of core were subjected by STS to Point Load Strength Index testing. The core photographs are presented opposite each borehole log and the test results are plotted on the relevant log and summarised in Table C. Selected soil samples were also submitted to LabPoint, a NATA registered laboratory, for soil pH and soil sulfate content determinations. The test results are summarised in Table D.

Contamination testing of the site soils has been completed by EIS and the results are presented in a separate report.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located over the south-western corner of the Prince of Wales Hospital grounds and is bounded by Barker Street along the south and Hospital Road along the west. The ground slopes down to the south-west at about 3°.

At the time of the fieldwork, the site was occupied by the existing POWMRI facility comprising a one and two storey brick and rendered building and the existing Randwick Ambulance Station comprising one single storey brick building and a two storey brick building. Based on a cursory inspection, all of the buildings on site appeared to be in good external condition. Towards the northern end of the existing POWMRI building, works were currently under way for the Stage 1 expansion.

A concrete and sandstone block retaining wall up to 4.5m high was located towards the north eastern corner of the site and supported the area to the north east. The retaining wall appeared to be in good condition.

To the north of the site were the Black Dog Institute, a one and two storey clad building and the Kiloh Centre a two storey brick building. To the east of the site was an internal access road to Prince of Wales Hospital beyond which was a one and two storey brick and rendered building. The above buildings appeared to be in good condition based on a cursory inspection from within the subject site.

3.2 Subsurface Conditions

The 1:100,000 geological map indicates the site is underlain by Hawkesbury Sandstone.

The boreholes disclosed a generalised subsurface profile below the pavements consisting of fill over natural sands then sandstone bedrock at relatively shallow to moderate depth. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions at specific locations. Geotechnical cross-sections are presented in Figures 2 to 5 and a summary of the encountered subsurface profile is presented below:

Pavements

A pavement comprising asphaltic concrete (AC) between about 50mm and 100mm thick over a 100mm to 150mm thick roadbase was encountered from the surface of BH4, BH5, BH8, BH9 and BH16.

Fill

Fill was encountered from beneath the pavement or from the surface in all of the boreholes except BH3A. The fill was encountered to depths between 0.3m (BH6 and BH9) and 1.5m (BH7) and comprised silty sand, silty gravelly sand and silty sandy gravel.

Natural Sandy Soils

Natural sandy soils were encountered from beneath the fill in all of the boreholes except BH3A where it was encountered from the surface. The sands were generally medium dense with the exception of the upper natural sands in BH7, BH16, BH3A and BH7A which were assessed as being loose.

Sandstone Bedrock

Sandstone bedrock was encountered in all of the rig drilled boreholes at depths between 3.5m (BH3) and 6.8m (BH7). The sandstone bedrock was generally of extremely low or very low strength on first contact and increased in strength with depth to high strength. Based on the DCP3A and DCP17A refusals, inferred sandstone bedrock was encountered at depths of 2.3m and 2.8m respectively. We note, however, that the DCP test can refuse on buried obstructions, 'floaters', other hard layers, etc, and not necessarily on bedrock.

The rock encountered within the cored portions of the boreholes was classified in accordance with *Pells et al (1998)* as follows:

Location (BH)	Depth (m) of Different Rock Classifications					Rock Classification @ RL38m
	V	IV	III	II	I	
2			6.2 - 8.3		8.3 - 12	III @ 6.5m depth
3				4.9 - 7.2	7.2 - 11.8	III @ 6.6m depth
4		4.6 - 5.5	5.5 - 7.2	7.2 - 13.5		II @ 7.1m depth
5	4.6 - 5.4			5.4 - 13.4		II @ 7.3m depth
6				8.1 - 15.06		II @ 10.8m depth
7		7.2 - 8.0	12.5 - 13.5	8.0 - 12.5 13.5 - 16.0		III @ 12.5m depth
8		4.5 - 6.7			6.7 - 14.6	I @ 8.3m depth
9		7.2 - 8.4	4.6 - 5.8	5.8 - 7.2	8.4 - 15.8	I @ 8.6m depth
16			5.3 - 7.0	7.0 - 14.4		II @ 7.4m depth

Groundwater

Groundwater seepage was not encountered in any of the boreholes during or on completion of augering. On completion of coring, groundwater was measured at depths between 2.8m (BH5) and 5.2m (BH7). It should be noted that water is injected into the boreholes during coring and the water levels may not have stabilised prior to these measurements being taken.

BH4 and BH16 have piezometer standpipes installed to allow future water levels to be monitored. On 13 November 2008, during the fieldwork period, and three and two days after drilling BH4 and BH16, the groundwater was measured at depths of 4.2m and 6.0m respectively.

3.3 Laboratory Test Results

The moisture content tests on the recovered rock chip samples generally correlated well with our field assessment of the in-situ rock strength.

The Point Load Index Strength tests on the recovered rock core samples generally correlated well with our field assessment of the in-situ rock strength. The estimated Unconfined Compressive Strength (UCS) of the sandstone bedrock varied between <1MPa and 56MPa with an average estimated UCS of about 26MPa.

The particle size distribution tests on the selected sand samples from BH3A and BH17A showed the sands to be generally poorly graded, and for the sample from BH17A to contain a trace of silt fines.

The soil pH tests on the recovered soil samples from BH2, BH5, BH7 and BH16 returned pH values of 8.0, 9.5, 7.6 and 7.6 respectively. The soil sulphate tests on the same samples all returned results of less than 50mg/kg.

3.4 Borehole Infiltration Testing

The constant head borehole infiltration tests in BH3A and BH17A resulted in insitu permeability values of $3.1x10^{-5}$m/s and $7.7x10^{-5}$m/s respectively when analysed using established formulae. Reference should be made to the attached Constant Head Permeability Test Results sheets for details of the testing.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

Based on the investigation results, the principle geotechnical issues associated with the proposed development at the subject site relate to:

- The proposed excavation through the soil profile will require to be supported by a full depth engineered retention system.
- Ground vibrations associated with rock excavation will need to be controlled due to the sensitivity of the activities within the surrounding operating research buildings.

These issues are discussed in further detail in the sections that follow.

4.2 Excavation Conditions

4.2.1 Excavation Methods

The proposed bulk excavation to RL38m and deeper, will encounter the soil profile and extend into the underlying sandstone bedrock of variable but relatively high strength.

The soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators, loaders, dozers, etc). Some of the underlying weathered sandstone of extremely or very low strength, if encountered, should also be excavated by bucket excavator, possibly with some ripping. However, we expect that Class III sandstone would be most effectively excavated using a Caterpillar D9L dozer or equivalent, whilst Class II sandstone will require a Caterpillar D10 dozer or equivalent. Hydraulic impact rock hammers may also be used for effective excavation. This latter equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations, such as for footings or buried services.

The above comments are made so that an appreciation of the rock strength and quality in terms of typical excavatability can be gained. However, the equipment to be used will be dictated by vibration limits which have been set on the surrounding medical research buildings. Further advice in this regard is presented in Section 4.2.2 below.

4.2.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on this site and there will likely be direct transmission of ground vibrations to adjoining operating research buildings and structures where vibration limits (measured as peak particle velocity) of no higher than 0.28mm/sec are specified. Prior to excavation commencing, detailed dilapidation reports should be compiled on the neighbouring buildings and structures to the north, east and west. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the works. We also recommend that continuous vibration monitoring be carried out during rock excavations. The selection of monitoring points will be critical and we recommend that advice from an acoustic consultant be sought in this regard.

Rock excavations should commence after initially providing a vertical saw cut slot along the perimeter of the excavation and maintaining the base of the slot at a lower level than the adjoining rock excavation at all times. The use of a rock saw would also provide a 'smooth' cut rock face. Rock excavation with hydraulic rock hammers, if used, should preferably commence away from likely critical areas using a moderately sized excavator fitted with a moderate energy hydraulic hammer no larger than a Krupp 900 size or equivalent. If it is found that transmitted vibrations are excessive, then it would be necessary to change to a smaller rock hammer or to use alternative excavation techniques. Preferred excavation techniques to significantly reduce vibrations include a rotary grinder or grid sawing in conjunction with ripping and/or hammering. 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 rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate hammers one at a time and 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.

4.2.3 Stress Relief

The site is underlain by sub-horizontally bedded Hawkesbury Sandstone. A characteristic feature of these rocks is that the major and intermediate principal stresses act horizontally and within the depth of interest, are significantly higher than the overburden pressures. The combination of the above leads to lateral displacement of the sides of rock excavations due to stress relief.

Monitoring of several deep excavations within the sandstone in Sydney indicates that the lateral displacement at the top of the excavation is generally 0.5mm to 2mm per metre depth of excavation. Due to the topographic setting of the site, being over the side of a hillslope, we anticipate that the lateral deflection will be in the range 5mm to 1mm per metre of excavation depth. For example, for the maximum depth of excavation into sandstone of 6m, a lateral deflection at the crest of the excavation up to 6mm can be expected. The lateral deflection is expected to reduce in stepped fashion to zero at bulk excavation level and should be minimal (ie. significantly less than 6mm) at a horizontal distance of 12m (ie. twice the excavation depth) from the perimeter of the excavation.

4.2.4 Seepage

Although groundwater seepage within the soil profile was not encountered during the investigation, we would expect some slow groundwater seepage flows may occur at the soil-rock interface and through joints and bedding planes within the completed cut faces. The seepage volumes may well increase after periods of heavy or prolonged rain. Seepage, if any, during excavation is expected to be satisfactorily controlled by conventional sump pumping systems. However, we recommend that the piezometer standpipes which were installed in BH4 and BH16 be regularly monitored during the period up to excavation commencing to confirm the above and to determine the groundwater response to rainfall.

We recommend that a toe drain be formed at the base of all rock cuttings to collect groundwater seepage and lead it to a sump for pumped disposal to the stormwater system.

4.3 Excavation Support

4.3.1 Soil Profile Support

Where space permits, excavations in the soil profile may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1.5 Horizontal (H) for the soil profile and 1V in 1H for the Class V rock. Subject to review of the piezometer standpipe monitoring results, we expect that seepage at the soil-rock interface may cause localised instability at the toe of soil batters and allowance should be made for sandbagging. We also recommend that a minimum 1m wide berm be provided between the toe of soil batters and the crest of vertical rock faces.

Where temporary batters cannot be accommodated or where they are not preferred, an engineered retention system will be required, which should be installed prior to excavation commencing. Given the subsurface conditions encountered, a suitable retention system includes a temporary anchored or internally propped contiguous pile wall using grout injected auger/CFA piles. The use of conventional bored piles is not considered suitable even when using temporary liners, as the sandy soils are likely to collapse into the pile bore prior to the lining being installed. Conventional driven sheet pile walls would not be suitable due to the noise and vibrations which occur during installation. Sheet pile walls using vibratory techniques for installation are unlikely to penetrate the extremely weathered sandstone bedrock which was encountered, without pre-drilling.

Care is required during the drilling of CFA piles in order not to cause a drawdown of soil into the drill hole, particularly when drilling below the groundwater or bedrock level. The ground surface surrounding the pile hole being drilled must be monitored

by site personnel and if signs of settlement are detected, drilling must stop and further geotechnical advice sought. There are no subsurface conditions requiring a specific grout mix for use in CFA piles. The mix design will be dictated by strength, durability, workability and operational requirements.

Lateral restraint in the form of rock anchors or internal props will be required and must be installed progressively as the propping point has been uncovered. The possibility of neighbouring basements must be confirmed prior to anchor design or installation.

Construction of the contiguous pile walls should be of high quality, taking care to prevent soil loss through gaps that will most likely occur between the piles, as this will add to the possibility of settlement occurring outside the excavation. Such gaps should be rectified progressively during the excavation, such as by mass concrete infill or shotcrete.

4.3.2 Bedrock Support

We expect that Class IV sandstone may be cut vertically and will be self-supporting over the short term. However, over the longer term, a permanent retention system will be required. Permanent retention can be provided if the perimeter contiguous pile wall extends to the base of the Class IV sandstone. Alternatively, the contiguous pile wall may be founded at least 0.5m into Class IV sandstone and the Class IV sandstone below founding level subsequently supported using shotcrete and rock anchors or shotcrete which is restrained by, and designed to span between, the proposed floor slabs.

We expect that Class III or better sandstone may be cut vertically (ie will be self-supporting over the long term). However, localised stabilisation measures may be necessary if adverse defects (such as inclined joints or bedding) are found.

Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay seams occurring in permanently exposed sandstone slopes may require 'dental' treatment. We therefore recommend that the rock face be progressively inspected by an experienced geotechnical engineer/engineering geologist as excavation proceeds (at no more than 1.5m vertical intervals) to identify adverse defects and to propose appropriate stabilisation measures. We note that the extent of stabilisation measures which may be required can only be finalised during the above inspections. However, based on the investigation results, it would appear that the stabilisation measures which will be required within the Class III or better sandstone, will be of limited extent.

4.4 Retaining Walls

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 temporary or permanent systems to retain the sandy soil profile:

- Conventional free-standing cantilever walls supporting areas where movement is of little concern (ie. where only garden or grassed areas are to be retained), may be designed for a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.35, for the soil profile and Class V rock, assuming a horizontal retained surface.
- Free-standing cantilever walls, the tops of which are restrained by the ground floor slab of the permanent structure or where movements need to be reduced, should be designed for a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, K_o, of 0.55, for the soil profile and Class V rock, assuming a horizontal retained surface.

- Alternatively, the earth pressure coefficients may be determined based on the sand having an effective angle of friction, ϕ', of 30°.
- For progressively anchored or propped walls where some minor movements can be tolerated (eg. along the street frontages, provided there are no movement sensitive buried services), we recommend the use of a uniform rectangular earth pressure distribution of 6H kPa for the soil profile and Class V rock, where 'H' is the retained height in metres.
- For progressively anchored or propped walls which are supporting areas which are sensitive to lateral movement (such as adjacent to existing buildings), we recommend the use of a uniform rectangular earth pressure distribution of 8H kPa for the soil profile and Class V rock, where 'H' is the retained height in metres.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile and Class V rock.
- Any uniformly distributed surcharge affecting the walls (eg. nearby footings, traffic, construction loads, etc) should be allowed 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. Point loads and line loads affecting the walls will result in non-uniform lateral pressure distributions. Further geotechnical advice should be sought for the specific loading conditions being considered.
- The retaining walls should be designed as drained and measures taken to provide permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric (such as Bidim A34), to act as a filter against subsoil erosion. Also refer to Section 4.8 below.

- Lateral toe restraint may be achieved by keying the piles into bedrock below bulk excavation level. An allowable lateral stress of 500kPa may be adopted for embedment design. The upper 0.5m below bulk excavation level, however, should be ignored in the analysis to take excavation tolerances and disturbances into account.
- Alternatively, where the piles are terminated in bedrock above bulk excavation level, lateral toe restraint should be achieved using a lower row of anchors which are installed prior to excavation of the ground in front of the toe.
- If rock anchors are to be installed beyond the site boundaries, then the permission of the owners (this may include the road authorities) must be obtained before installation. Anchors bonded into Class III and better sandstone should be designed for an allowable bond stress of 350kPa. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installation. We have assumed that permanent lateral support for the perimeter piled walls will be provided by the new structure. If not, permanent anchors will be required which must be designed for corrosion resistance and for long term durability.

It is likely that the excavation within the soil profile and Class V rock will induce some movements of the adjacent ground that falls within the zone of influence of the excavation. This is separate to, and may be in addition to, movements due to stress relief as outlined in Section 4.2.3 above. The zone of influence can be defined as being located above an imaginary line extending upwards from the top of the bedrock at 30° to the horizontal. In sands, lateral movements of even relatively stiff cantilever walls, can possibly be of the order of 1% of the excavated depth. Precedence suggests that for propped or anchored walls which are designed on the

basis of an 8H uniform lateral earth pressure, lateral in adjacent vertical movements of the retention system would probably be less than 0.1% of the excavation depth.

4.5 Footings

4.5.1 Contiguous Pile Walls

Contiguous pile walls using CFA piles which are founded in Class III or better sandstone bedrock below bulk level should be designed for an allowable end bearing pressure of 6MPa. An allowable shaft adhesion value of 600kPa may be applied over that length of rock socket below 0.5m beneath bulk excavation level, provided a socket roughness of at least R2 is achieved (ie grooves of depth 1mm to 4mm, width greater than 2mm, at spacing of 50mm or 200mm). Refer to Section 4.3.1 above for advice on CFA pile installation.

The allowable stresses given above are based on serviceability criteria of deflections at the pile toe equal to 1% or less of the pile diameter.

The designer may wish to adopt the limit state design methods, such as in the Piling Code, AS2159-1995. Ultimate end bearing values of 40MPa and ultimate shaft adhesion values of 1,000kPa should be used for the Class III sandstone bedrock. A geotechnical strength reduction factor, ϕ_g of 0.75 and 0.6 should be adopted for end bearing and shaft adhesion respectively. A Young's Modulus value of 1000MPa is applicable.

Perimeter pile walls which are founded in Class IV or better sandstone above bulk excavation level should be designed in end bearing only with the design based on an allowable end bearing pressure of 1.5MPa or an ultimate end bearing pressure of 6MPa. For this option, particular care is required during bulk excavation to avoid undermining the pile toes.

We recommend that the initial pile be drilled adjacent to one of the boreholes and that a geotechnical engineer be in attendance to assist in calibrating the rig by comparing the drilling data with the borehole log.

4.5.2 Internal Footings

Based on the investigation results, internal pad or strip footings at bulk excavation level of RL38m or lower will be founded in Class III or better sandstone. The footings should be designed on the basis of an allowable bearing pressure of 6MPa.

The allowable stress given above is based on serviceability criteria of deflections less than 1% of footing dimension.

The designer may wish to adopt the limit state design methods. An ultimate bearing value of 40MPa should be used for the Class III sandstone. A geotechnical strength reduction factor, ϕ_g of 0.6 should be adopted. A Young's Modulus value of 1000MPa is applicable.

We recommend that say, 30% of footings be subjected to spoon testing and that the footing excavations be inspected by a geotechnical engineer prior to concrete placement.

4.5.3 External Footings

Relatively lightly loaded buildings or structural elements which are located outside the footprint of the proposed basement may be supported using pad, strip or stiffened raft footings founded in the loose or medium dense sands using indicative allowable bearing pressures of 100kPa and 200kPa respectively.

The above should limit settlements of footings with at least 0.5m embedment and up to 1.5m width to less than 10mm, based on a Young's Modulus of 10MPa and 15MPa respectively. The above settlements are expected to occur almost instantaneously as the load is applied (ie. during the construction period).

We note that bearing pressures in sand depend on the footing geometry in addition to the sand density. Therefore, once the structural loads are finalised, bearing pressure and settlement analyses should be undertaken to assess the suitability of a high level footing option.

We recommend that DCP testing of high level footing excavations in sand be carried out by a geotechnical engineer to confirm that the design foundation material has been encountered.

Footings in sands should be founded below the zone of influence of the excavation or, alternatively, the effect of surcharge on the perimeter contiguous pile wall must be taken into account in the wall design (refer Section 4.4 above).

More heavily loaded buildings or structures which are located outside the basement footprint should be supported using pile footings founded in sandstone bedrock. As outlined in Section 4.3.1 above, our preference is to use CFA piles. The following parameters are applicable for the design of piles founded in the different rock classifications:

Rock Classifications	Allowable		Ultimate		Young's Modulus (MPa)
	End Bearing Pressure (MPa)	Side Adhesion (kPa)	End Bearing Pressure (MPa)	Side Adhesion (kPa)	
V	1	100	3	150	80
IV	3.5	350	15	800	600
III	6	600	40	1,000	1,000
II	10	1,000	100	2,750	1,800

We note that it will be difficult to prove the founding achieved in CFA piles, particularly if drilled prior to completion of bulk excavation for the basement. Additional cored boreholes may be necessary depending on the pile locations and the sandstone classification required.

The above allowable end bearing pressures will result in settlement at the pile toe of less than 1% of pile diameter. The above side adhesion values apply to sockets which have a roughness category of R2 or better (refer Section 4.5.1 above).

4.6 Basement On-Grade Floor Slab

The basement on-grade floor slab will directly overlie bedrock. We therefore recommend that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate (eg. 'blue metal' gravel) and should connect with the wall drains and lead groundwater seepage to a sump for pumped discharge to the stormwater system. The underfloor drainage may comprise a blanket drain or a drain should be installed into slots which are cut into the rock subgrade.

Joints in the concrete on-grade floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.7 External Pavements

4.7.1 Pavement Design Parameters

The design of car park and driveway pavements over a soil subgrade will depend on subgrade preparation, the nature and composition of new fill imported to the site, as well as vehicle loadings and use.

Provided satisfactory subgrade preparation is carried out, we recommend that the design of flexible pavements for car park and driveway areas be based on a CBR value of 10%, as estimated from the DCP test results. For concrete or rigid pavement design, a short term Young's Modulus of 35MPa should be adopted.

4.7.2 Subgrade Preparation

Subgrade preparation for proposed pavement areas may require removal of a number of trees. Stripping of root-affected soil will also be required. Following this, any obvious deleterious or contaminated existing fill should be removed. These stripped materials should be taken off site as they are not suitable for re-use as engineered fill. The soils should then be excavated to suit the design subgrade levels for the pavement.

The exposed subgrade should then be proof-rolled with a 5 tonne minimum deadweight smooth drum vibratory roller. Proof-rolling should be carried out under the direction of an experienced earthworks superintendent or geotechnical engineer to assist in the detection of unstable areas which were not disclosed by this investigation and to densify the upper sand profile.

Any unstable areas identified during proof-rolling should be locally excavated down to a competent base and replaced with engineered fill.

The excavated sands and crushed sandstone may be re-used as engineered fill, provided deleterious materials and any building rubble is excluded. The fill for backfilling earthworks platforms should be compacted in layers of not greater than 200mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density.

All compacted fill should either be retained or battered to a slope no steeper than 1 Vertical (V) in 2 Horizontal (H). All exposed fill embankments should be protected from erosion by quickly establishing a grass cover or by structural means.

Density testing should be carried out at a frequency of at least one test per fill layer per 1000m^2 or three tests per layer per visit, whichever requires the most tests. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not as part of the earthworks contract.

The earthworks recommendations provided here should be complemented by reference to AS3798.

4.7.3 Concrete Pavements

Concrete pavements overlying a soil subgrade should be supported on a sub-base layer of RTA3051 specification unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD. The subbase material would provide more uniform slab support and would reduce 'pumping' of sub-grade 'fines' at joints.

Concrete pavements should be provided with effective shear connection at points by using dowels or keys. Concrete pavements or specially thickened asphaltic concrete should be used in areas where heavy vehicles manoeuvre such as garbage bin and truck unloading areas.

4.8 Infiltration System

Based on the borehole infiltration testing, the subsurface sands have a permeability between 3.1 and $7.7x10^{-5}$m/sec. This is consistent with the sandy subsoils as indicated by borehole logs and the particle size distribution curves presented in attached Table B.

Based on the above, an approximate infiltration rate of 0.1H litre/sec/m^2 has been estimated where 'H' is the head of water in the system in metres. The above estimation has also assumed that infiltration only occurs from the base of the system. Consideration should be given to the effects of possible siltation over the longer term, and the reduction in capacity if groundwater levels rise during prolonged rainfall periods.

The water from the infiltration system is expected to flow vertically through the soil profile until it intersects groundwater or bedrock, after which the water is expected to flow downslope (ie. towards the south-west). The effects of this additional groundwater flow and its possible impact on downhill properties must be given due consideration.

4.9 Earthquake Design Parameters

On the basis of 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.

4.10 Hydrogeological Issues

We anticipate that surface water will infiltrate into the ground over the higher lying catchment to the north-east. The infiltrated water will mostly flow vertically through the soil profile and once bedrock is encountered, the groundwater will flow downslope over the rock surface (ie. towards the south-west).

Based on the above, the proposed basement will intersect the natural groundwater flows. However, provided the recommendations with respect to drainage are adopted, groundwater through-flow will occur with minimal build-up of uphill groundwater levels. Further, the proposed drainage will not lower the groundwater below historic levels within the soil profile and, therefore, settlement of surrounding ground due to groundwater drawdown is not expected to occur. The above also applies to groundwater originating from the proposed infiltration systems (refer Section 4.8 above). Based on the above, the proposed basement is unlikely to significantly affect the natural groundwater flows within the site and surrounding areas. However, we recommend that groundwater seepage into the bulk excavation be monitored, so that unexpected conditions can be timeously addressed.

4.11 Soil Aggression

Based on the investigation results, a 'non-aggressive' exposure classification is applicable to piles in soil in accordance with AS2159-1995.

However, good engineering practices will be necessary to protect concrete in contact with the moderately alkaline soils of the site (soil pH values of between 7.6 and 9.5). The designer is referred to AS2159-1995 for further guidance.

4.12 Further Geotechnical Input

The following summarises the further geotechnical input which is required and which has been previously discussed in the preceding sections of this report:

- Monitoring of piezometer standpipes.
- Dilapidation surveys of surrounding buildings and structures.
- Continuous vibration monitoring during rock excavation.
- Geotechnical witnessing of initial CFA pile drilling.
- Proof-testing of anchors, if appropriate.
- Geotechnical footing inspections and spoon testing, if appropriate.
- Groundwater monitoring into bulk excavation.
- Proof-rolling of pavement subgrade.
- Density testing of engineered fill and subbase.
- Further geotechnical investigations, if appropriate.

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.

A ZENON
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

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PO Box 976
North Ryde, Bc 1670
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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
SUMMARY OF MOISTURE CONTENT TEST RESULTS

AS 1289 **TEST METHOD** **2.1.1**

BOREHOLE NUMBER	DEPTH	MOISTURE CONTENT
	m	%
2	5.50-6.00	11.9
4	4.00-4.30	6.0
5	3.80-4.00	6.9
5	4.50-4.90	5.8
6	7.00-7.30	5.5
6	7.90-8.20	3.2
8	3.80-4.20	4.9
16	5.10-5.30	5.6

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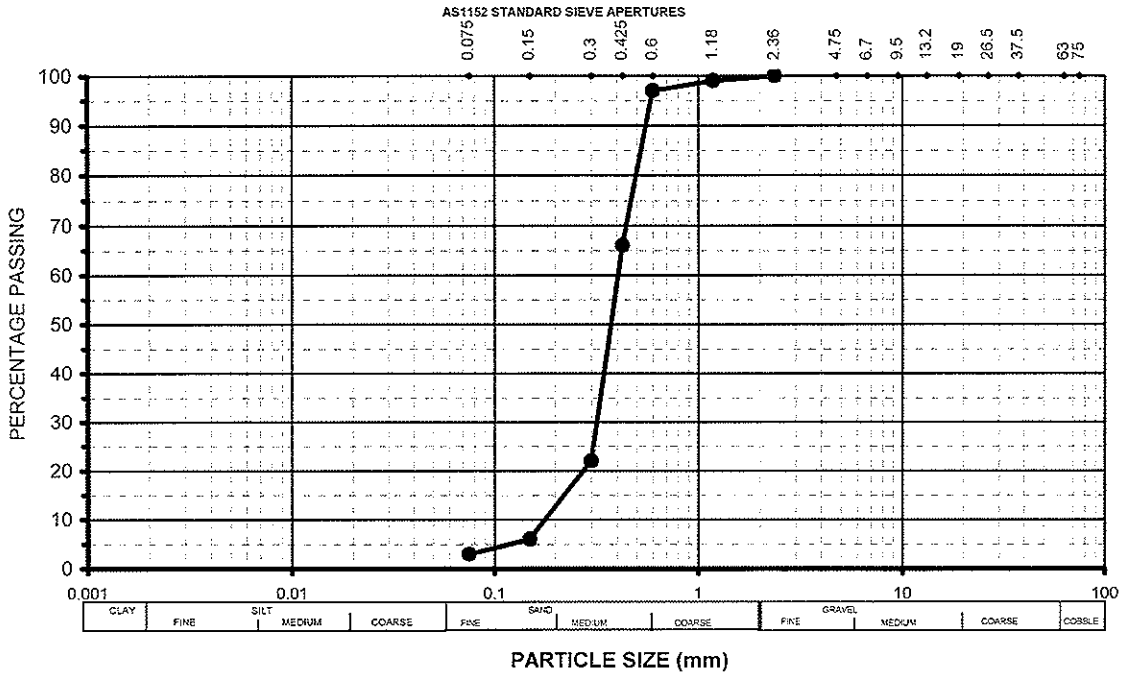
SOIL TEST SERVICES
ABN 43 002 145 173



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 um	97
425 um	66
300 um	22
150 um	6
75 um	3

Test Method: AS1289.3.6.1 Dry Sieve(Washed)

Notes:

•**SAND**:fine to coarse grained,light grey,trace of fine gravel

Job No: 22529Z
Borehole Number: 3A
Depth(m):1.00-2.00
TABLE: B, Page 1 of 2

PARTICLE SIZE DISTRIBUTION CURVE

NATA Accredited Laboratory Number:1327

Approved Signatory / Authorised Signature / Date
(A. Tatikonda) 1/12/08

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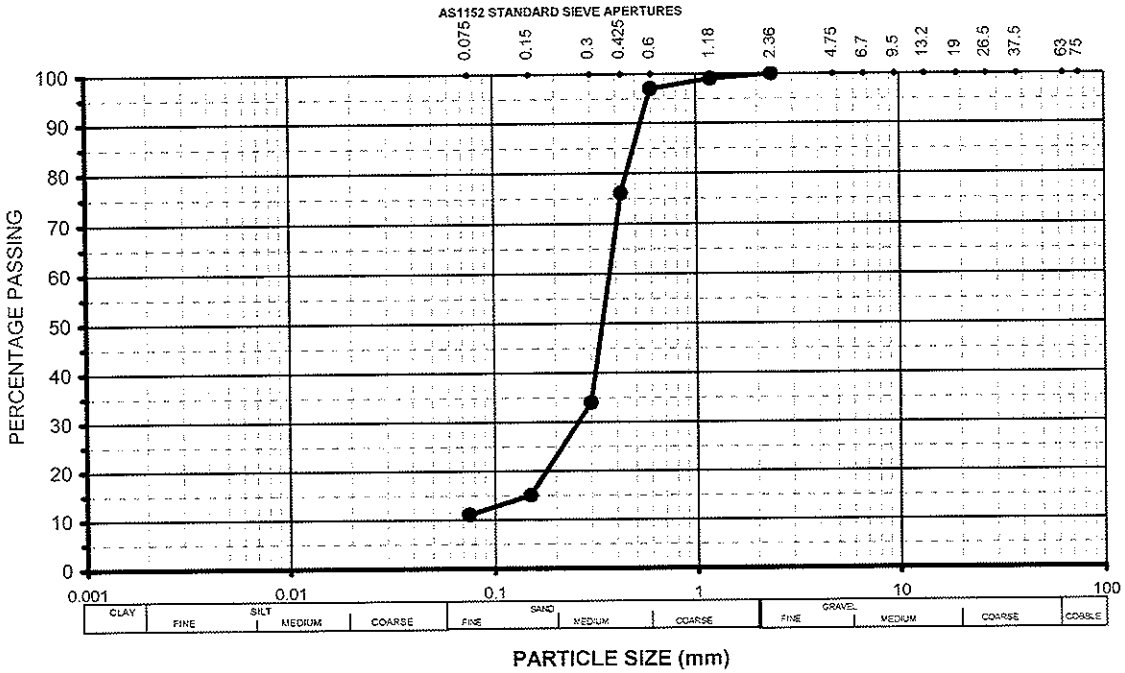
SOIL TEST SERVICES
ABN 43 002 145 173



SIEVE ANALYSIS RESULTS

SIEVE SIZE	% PASSING
2.36 mm	100
1.18 mm	99
600 um	97
425 um	76
300 um	34
150 um	15
75 um	11

Test Method: AS1289.3.6.1 Dry Sieve(Washed)

Notes:

•**SAND**:fine to coarse grained,brown,trace of fine gravel

Job No: 22529Z
Borehole Number: 17A
Depth(m):1.00-1.60
TABLE: B, Page 2 of 2

PARTICLE SIZE DISTRIBUTION CURVE

NATA Accredited Laboratory Number:1327

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(A. Tatikonda) 1/12/08

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH	$I_{S(50)}$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
	m	MPa	(MPa)
16	5.40-5.42	0.9	18
	5.79-5.83	1.2	24
	6.20-6.23	1.3	26
	6.80-6.82	1.7	34
	7.20-7.24	1.3	26
	7.69-7.72	1.2	24
	8.33-8.36	1.1	22
	8.71-8.73	1.4	28
	9.20-9.23	1.5	30
	9.74-9.77	1.3	26
	10.19-10.23	1.1	22
	10.85-10.88	1.4	28
	11.20-11.23	1.4	28
	11.71-11.75	1.2	24
	12.31-12.34	1.1	22
	12.71-12.74	1.1	22
	13.18-13.22	1.3	26
	13.65-13.69	1.6	32

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 $I_{S(50)}$

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	6.25-6.27	0.7	14
	6.77-6.80	1.5	30
	7.16-7.19	0.5	10
	7.75-7.79	1.1	22
	8.33-8.36	0.7	14
	8.73-8.76	1.6	32
	9.12-9.16	1.5	30
	9.70-9.73	1.3	26
	10.19-10.22	1.2	24
	10.81-10.84	2.8	56
	11.18-11.21	1.7	34
	11.83-11.86	1.8	36
3	4.93-4.96	0.9	18
	5.66-5.70	1.2	24
	6.07-6.11	1.0	20
	6.62-6.66	1.2	24
	7.07-7.10	1.1	22
	7.72-7.75	2.2	44
	8.20-8.23	2.0	40
	8.66-8.70	1.8	36
	9.25-9.28	1.3	26
	9.70-9.73	1.3	26
	10.19-10.23	2.0	40
	10.69-10.73	2.0	40
	11.26-11.28	1.7	34
	11.66-11.69	1.9	38

NOTES:SEE PAGE 7 OF 7

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TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
4	4.73-4.76	0.6	12
	5.14-5.17	0.6	12
	5.71-5.74	1.2	24
	6.26-6.29	0.9	18
	6.74-6.77	0.9	18
	7.27-7.30	1.1	22
	7.75-7.79	2.3	46
	8.36-8.39	1.7	34
	8.79-8.83	1.8	36
	9.26-9.31	0.9	18
	9.79-9.84	1.6	32
	10.27-10.31	1.4	28
	10.74-10.77	1.5	30
	11.22-11.26	1.5	30
	11.73-11.77	1.4	28
	12.20-12.24	1.3	26
	12.72-12.76	1.6	32

NOTES:SEE PAGE 7 OF 7

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ABN 43 002 145 173

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
5	4.73-4.75	0.2	4
	5.28-5.31	0.2	4
	5.73-5.76	1.4	28
	6.17-6.20	1.0	20
	6.67-6.69	1.2	24
	7.28-7.32	0.7	14
	7.72-7.76	1.2	24
	8.21-8.24	0.8	16
	8.71-8.75	1.1	22
	9.24-9.26	1.6	32
	9.77-9.82	2.1	42
	10.20-10.24	1.8	36
	10.77-10.80	1.4	28
	11.19-11.24	1.4	28
	11.74-11.77	1.4	28
	12.20-12.24	1.5	30
	12.77-12.81	1.5	30
	13.16-13.20	1.2	24
6	8.25-8.28	1.3	26
	8.65-8.68	1.0	20
	9.17-9.20	1.2	24
	9.68-6.72	1.4	28
	10.21-10.25	1.1	22
	10.59-10.63	1.7	34
	11.21-11.25	1.6	32
	12.25-12.28	2.0	40

NOTES:SEE PAGE 7 OF 7

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TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
6	12.77-12.80	1.8	36
	13.23-13.27	1.4	28
	13.78-13.82	1.0	20
	14.27-14-31	1.3	26
	14.77-14.81	1.1	22
	15.28-15.32	1.2	24
7	7.35-7.38	0.2	4
	7.82-7.85	0.4	8
	8.27-8.30	0.8	16
	8.79-8.83	0.6	12
	9.36-9.39	1.3	26
	9.78-9.82	1.7	34
	10.20-10.24	1.5	30
	10.65-10.68	2.4	48
	11.26-11.30	1.6	32
	11.82-11.85	1.4	28
	12.30-12.33	1.1	22
	12.86-12.90	0.4	8
	13.11-13.13	0.04	<1
	13.62-13.65	1.3	26
	14.14-14.17	1.3	26
	14.89-14.93	1.1	22
	15.22-15.25	1.2	24
	15.76-15.80	1.1	22
	16.24-16.26	1.3	26

NOTES:SEE PAGE 7 OF 7

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TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
8	4.82-4.85	0.1	2
	5.26-5.30	0.1	2
	5.80-5.82	2.0	40
	6.19-6.22	1.1	22
	6.75-6.78	1.0	20
	7.23-7.26	1.6	32
	7.74-7.78	0.9	18
	8.20-8.23	1.2	24
	8.73-8.77	1.7	34
	9.20-9.24	2.4	48
	9.76-9.80	2.1	42
	10.22-10.75	1.9	38
	10.49-10.52	1.3	26
	10.74-10.78	2.1	42
	11.28-11.33	1.2	24
	11.73-11.76	1.1	22
	12.20-12.23	1.3	26
	12.70-12.74	1.4	28
	13.21-13.24	1.1	22
	13.79-13.81	0.8	16
	14.15-14.20	1.1	22
9	4.74-4.77	0.9	18
	5.23-5.26	2.1	42
	5.89-5.92	1.4	28
	6.22-6.26	2.0	40
	6.72-6.75	1.1	22

NOTES:SEE PAGE 7 OF 7

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TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
9	7.29-7.33	1.4	28
	7.72-7.75	1.2	24
	8.20-8.23	1.4	28
	8.76-8.79	1.9	38
	9.28-9.32	1.5	30
	9.74-9.76	2.2	44
	10.28-10.32	1.5	30
	10.77-10.80	1.4	28
	11.32-11.35	1.3	26
	11.75-11.78	1.3	26
	12.30-12.35	1.4	28
	12.65-12.69	1.6	32
	13.18-13.22	1.6	32
	13.71-13.74	1.3	26
	14.18-14.22	1.6	32
	14.73-14.76	1.7	34
	15.17-15.20	1.4	28
	15.69-15.72	1.2	24

NOTES:SEE PAGE 7 OF 7

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AT

TABLE D
Page 1 of 1

LabPoint
LabPoint Pty Ltd
ABN 82 096 903 749
Phone: (02) 9624 5588
Fax: (02) 9624 2266
E-Mail: labpoint@bigpond.net.au
Unit 31, 35 Foundry Road,
Seven Hills NSW 2147
P.O. Box 177
Kings Langley NSW 2147

Report No: NAA08-2883

Date Received: 26/11/2008

Order No: COC dated 25/11/08

Attention: Mr. Ashwin Tatikonda

Soil Test Services Pty Ltd
115 Wicks Road
Macquarie Park NSW 2113

Type of Samples: Four soil samples - project 22529 Z. Analysed 'as received'

Tests	BH 2 4.0-4.5m	BH 5 0.5-1.0m	BH 7 1.5-1.95m	BH 16 2.2-3.0m	Methods
pH	8.0	9.5	7.6	7.6	AS 1289 4.3.1 - 1997
Sulphate	<50	<50	<50	<50	AS 1289 D2.1 1997 & APHA 4500 SO_4^{2-} - E

Note: Units: mg/kg dryweight for soils except pH. Analysed "as received".

Samples will be disposed of seven days after issue of this report unless otherwise notified.

The above soil samples have been prepared by customer as follows:

(a) Oven dried at 50 C

(b) Sieved over 2.36 mm sieve

Dr Rama Bhat
Manager Environmental Services
Date Issued: 1/12/2008

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Accreditation for compliance with ISO/IEC 17025
Laboratory No. 11111

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 2 1/2

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 44.5m

Date: 14-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DAMP ON COMPLETION OF AUGERING						0			FILL/TOPSOIL: Silty sand, fine to coarse grained, dark grey, with occasional fine to coarse grained sandstone and igneous gravel and charcoal fragments.				
								SP	SAND: fine to coarse grained, dark grey.	D	MD		AEOLIAN
					N = 10 3,4,6	1			SAND: fine to coarse grained, light yellow.				
						2							
					N = 15 4,7,8	3							
						4							
▼ ON COMPLETION OF CORING						5		-	SANDSTONE: fine to coarse grained, light grey and orange brown.	XW-DW	EL-VL		LOW TO MODERATE 'TC' BIT RESISTANCE
										DW	VL-L		HIGH RESISTANCE
						6				DW-SW	L		MODERATE TO HIGH RESISTANCE 'TC' BIT REFUSAL
									REFER TO CORED BOREHOLE LOG				
						7							

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22529Z BH2 START CORING AT 6.20m
6
7
8
VOID/CORE LOSS
0.26m
9
10
11
12
END OF BH2 AT 12.05m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **2** 2/2

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 44.5m
Date: 14-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		6		START CORING AT 6.20m					
60% RET-URN		7		SANDSTONE: fine to coarse grained, light grey to grey, red brown and orange brown laminae, bedded at 0-15°.	SW	M-H	×		- Be, P, R, 2mm
				SANDSTONE: fine to coarse grained, light grey to grey, with dark grey laminae, bedded at 0-5°.	SW-FR		×		- Be, P, R - J, 35°, P, R
		8		CORE LOSS 0.26m			×		- Be, P, R
30% RET-URN		9		SANDSTONE: fine to medium grained, grey.	FR	M	×		
				SANDSTONE: fine to coarse grained, light grey.		H	×		- XWS, 10mm.t
		10		as above, but with dark grey laminae, bedded at 0-15°.			×		
		11		as above, but light grey.			×		- XWS, 30mm.t
		12		END OF BOREHOLE AT 12.05m			×		

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Borehole No. **3** 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 44.6m

Date: 17-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			FILL/TOPSOIL: Silty sand, fine to coarse grained, dark brown, with roots.				
								SP	SAND: fine to coarse grained, dark grey.	D	MD		AEOLIAN
						1			as above, but mid grey.				
					N = 13 4,5,8				as above, but dark brown.				
						2			as above, but light yellow and light brown.				
					N = 18 6,8,10	3		-	SANDSTONE: fine to coarse grained, light grey, red and orange brown.	DW	VL-L	-	LOW 'TC' BIT RESISTANCE
ON COMPLETION OF CORING ▼						4				DW-SW	L		MODERATE TO HIGH RESISTANCE
						5			REFER TO CORED BOREHOLE LOG				
						6							
						7							

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JOB NO. 22529Z BH3 START CORING AT 4.87m

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Borehole No. **3** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 44.6m
Date: 17-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		4		START CORING AT 4.87m					
50% RET URN		5		SANDSTONE: fine to coarse grained, red brown, with grey and orange brown laminae, bedded at 0-15°.	SW-FR	M-H	×		- Be, 3°, P, R
		6		as above, but light grey and red brown.			×		
				SANDSTONE: fine to coarse grained, light grey, bedded at 0-15°.			×		
		7		SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-5°.	SW	L	×		- XWS, 90mm.t
				SANDSTONE: fine to medium grained, light grey, with grey and light brown laminae, bedded at 0-5°.	FR	H	×		- XWS, 20mm.t
		8					×		- Be, 15°, P, R - XWS, 20mm.t - Be, 20°, P, R
		9					×		
							×		- Be, 10°, P, R
		10					×		- Be, 15°, P, R
							×		- Be, 15°, P, R

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Borehole No. **3** 3/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 44.6m
Date: 17-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK250 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
				SANDSTONE: fine to medium grained, light grey, with grey and light brown laminae, bedded at 0-5°.	FR	H	× ×		- Be, 5°, P, R
		12		END OF BOREHOLE AT 11.85m					
		13							
		14							
		15							
		16							
		17							

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 4 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 45.1m

Date: 11-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			ASPHALTIC CONCRETE: 50mm.t over Roadbase: 100mm.t	D			
								SM	FILL: Silty sandy gravel, fine to coarse grained, igneous and sandstone gravel grey brown.	D	MD		AEOLIAN
						1			SILTY SAND: fine to coarse grained, dark grey.				
									as above, but light grey.				
					N = 12 4,6,6								
						2			as above, but orange brown.				
									as above, but light yellow and light brown.				
					N = 19 6,8,11	3							
ON COMPLETION OF CORING ▼ ON 13-11-08						4		-	SANDSTONE: fine to coarse grained, light grey, red and orange brown.	DW	VL-L	-	VERY LOW 'TC' BIT RESISTANCE
										DW-SW	L-M		MODERATE TO HIGH RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

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Borehole No. **4** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 45.1m
Date: 11-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components.)	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_S(50)$ (EL VL L M H VH EH)	DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS: DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating.) Specific / General
		4		START CORING AT 4.58m					
80% RET-URN		5		SANDSTONE: fine to coarse grained, light grey, red and orange brown, with occasional dark grey laminae bedded at 0-5°	DW-SW	M-H			- XWS, 100mm.t - XWS, 10mm.t
					SW-FR				
		6		SANDSTONE: fine to medium grained, light grey, with occasional dark grey laminae, bedded at 0-5°.	FR				- XWS, 40mm.t - XWS, 40mm.t
		7							- CS, 5mm.t - XWS, 30mm.t
60% RET-URN		8		SANDSTONE: fine to coarse grained, light grey, with occasional light brown laminae, bedded at 0-10°,		H			- XWS, 30mm.t - XWS, 30mm.t
		9				M-H			- XWS, 5mm.t - XWS, 5mm.t
		10				H			
80% RET-URN									

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Borehole No. **4** 3/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 45.1m
Date: 11-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
80% RET-URN		12		SANDSTONE: fine to coarse grained, light grey, with occasional light brown laminae, bedded at 0-10°.	FR	H	× × × ×		
		13							
				END OF BOREHOLE AT 13.54m					PVC STANDPIPE PIEZOMETER INSTALLED TO BASE OF BOREHOLE. SLOTTED FROM 2m TO 13.54m DEPTH. CAST IRON GATIC COVER AT SURFACE.
		14							
		15							
		16							
		17							

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Borehole No. **5** 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 45.3m

Date: 11-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			ASPHALTIC CONCRETE: 50mm.t over Roadbase, 100mm.t	D			
								SM	FILL: Silty gravelly sand, fine to coarse grained, dark grey, with fine to coarse grained igneous gravel.	D	MD	-	AEOLIAN
						1			SILTY SAND: fine to medium grained, light grey.				
					N = 11 6,5,6				SILTY SAND: fine to coarse grained, dark brown.				
						2			SILTY SAND: fine to coarse grained, light brown yellow.				
▼ ON COMPLETION OF CORING					N = 21 7,10,11	3			as above, but light yellow.				
						4		-	SANDSTONE: fine to coarse grained, dark red brown.	DW	VL	-	VERY LOW 'TC' BIT RESISTANCE
									SANDSTONE: fine to coarse grained, light yellow brown, orange brown, red and light grey.	DW	VL-L		LOW RESISTANCE LOW TO MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

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Borehole No. **5** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 45.3m
Date: 11-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** A.I./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components.)	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_S(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating.) Specific / General
		4		START CORING AT 4.61m					
FULL RETURN		5		SANDSTONE: fine to medium grained, light grey, with occasional red, orange brown and brown laminae, bedded at 0-5°.	DW-SW	L	×		- CS, 6mm.t
							×		- XWS, 60mm.t
		6		SANDSTONE: fine to coarse grained, light grey, with occasional red, orange brown and dark grey laminae, bedded at 0-5°.	SW-FR	M-H	×		- CS/XWS SEAM. 70mm.t
				SANDSTONE: fine to coarse grained, light grey, with occasional dark grey laminae, bedded at 0-10°.			×		
60% RETURN		7			FR		×		- CS, 5mm.t
10% RETURN		8		SANDSTONE: fine to coarse grained, light grey.			×		
		9		SANDSTONE: fine to coarse grained, light grey, with occasional light brown laminae, bedded at 0-10°.		H	×		
		10					×		
				SANDSTONE: fine to coarse grained, light grey, with occasional dark grey laminae, bedded at 0-5°.			×		

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CORED BOREHOLE LOG

Borehole No. 5 3/3

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 45.3m
Date: 11-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
50% RET-URN		12		SANDSTONE: fine to coarse grained, light grey, with occasional dark grey laminae, bedded at 0-5°.	FR	H	×		
		13					×		
				END OF BOREHOLE AT 13.37m			×		
		14							
		15							
		16							
		17							

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Borehole No. 6 1/4

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 48.8m

Date: 12-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DAMP ON COMPLETION OF AUGERING						0			TOPSOIL: Silty sand, fine to coarse grained, brown, with occasional root fibres.				GRASS COVER
								SM	SILTY SAND: fine to coarse grained, light brown.	D	MD	-	AEOLIAN
					N = 23 9,10,13	1							
						2		SM	SAND: fine to coarse grained, yellow and light brown.				
					N = 19 6,9,10	3							
						4							
					N = 30 8,14,16	5							
						6							
					N = 23 6,11,12			-	SANDSTONE: fine to coarse grained, orange brown, yellow and red.	XW-DW	VL	-	VERY LOW 'TC' BIT RESISTANCE
						7							LOW TO MOD.

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Borehole No. 6 2/4

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 48.8m

Date: 12-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SANDSTONE: fine to coarse grained, orange brown, yellow and red.	XW-DW	VL-L		RESISTANCE LOW TO MODERATE RESISTANCE LOW TO MODERATE RESISTANCE
						8			SANDSTONE: fine to coarse grained, mid grey.	SW	M		LOW RESISTANCE MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						9							
						10							
						11							
						12							
						13							
						14							

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Job No: 22529Z BH6 START CORING AT 8.12m
8
9
10
11
12
13
14
15
16
END OF BH6 AT 16.06m

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Borehole No. **6** 3/4

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 48.8m
Date: 12-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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
		8		START CORING AT 8.12m					
				SANDSTONE: fine to medium grained, grey brown, with dark grey laminae, bedded at 0-5°.	SW-FR	M-H			
		9							
				as above, but grey.					
80% RET-URN		10		SANDSTONE: fine to coarse grained, light grey, with occasional light brown and dark grey laminae, bedded at 0-5°.					- XWS, 20mm.t
				SHALE: dark grey.	SW	L			- XWS, 20mm.t
		11		SANDSTONE: fine to coarse grained, light grey, with occasional light brown and dark grey laminae, bedded at 0-5°.	SW-FR	H			
		12							
70% RET URN		13							- XWS, 50mm.t
		14		SANDSTONE: fine to coarse grained, light grey.					
60%									

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Borehole No. **6** 4/4

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 48.8m
Date: 12-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
RET-URN				SANDSTONE: fine to coarse grained, light grey.	SW-FR	H	×		- XWS, 10mm.t
		16		END OF BOREHOLE AT 15.06m					
		17							
		18							
		19							
		20							
		21							

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BOREHOLE LOG

Borehole No. 7
1/4

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z
Date: 13-11-08
Method: SPIRAL AUGER JK500
Logged/Checked by: A.I./
R.L. Surface: ≈ 50.5m
Datum: AHD

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DAMP ON COMPLETION OF AUGERING	■			■		0			FILL/TOPSOIL: Silty sand, fine to coarse grained, brown, with occasional root fibres.				
									FILL: Silty sand, fine to coarse grained, dark brown, with a trace of medium to coarse grained sandstone gravel.				
				■		1							
	■			■	N = 7 1,2,5			SP	SAND: fine to coarse grained, dark brown.	D	L		AEOLIAN
						2			as above, but light yellow and light brown.		MD		
				■	N = 18 6,8,10	3							
						4							
				■	N = 19 5,7,12	5							
▼ ON COMPLETION OF CORING													LOW 'TC' BIT RESISTANCE
						6			SANDSTONE: fine to coarse grained, light grey and red brown.	DW-SW	VL-L		VERY LOW TO LOW RESISTANCE / LOW TO MODERATE RESISTANCE
						7							

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BOREHOLE LOG

Borehole No. 7 2/4

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 50.5m

Date: 13-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SANDSTONE: fine to coarse grained, light grey and red brown.	DW-SW	VL-L		
									REFER TO CORED BOREHOLE LOG				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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JOB NO. 22529Z BH7 START CORING AT 7.23
7
8
9
10
11
12
13
14
15
16
END OF BH7 AT 16.37m

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Borehole No. **7** 3/4

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 50.5m
Date: 13-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS: DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS: DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating.) Specific / General
		7		START CORING AT 7.23m					
				SANDSTONE: fine to medium grained, red brown, light grey and orange brown.	DW-SW	L			- XWS, 20mm.t - XWS, 50mm.t - J, 30°, P, R, IS
		8				M			- Be, 5°, IS, P, S - Be, 5°, IS, P, S - Be, 0°, IS, P, S
				SANDSTONE: fine to medium grained, light grey brown, massive.	SW				
		9							
				as above, but with dark grey laminae, bedded at 0-5°.	FR	H			- XWS, 20mm.t - CS, 40mm.t
		10		SANDSTONE: fine to coarse grained, light grey, with a trace of grey laminae, bedded at 0-10°.					
		11							
		12							
				SANDSTONE: fine to medium grained, light grey brown, massive.	SW-FR	M			- XWS, 20mm.t - XWS, 70mm.t
		13				H			

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Borehole No. **7** 4/4

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 50.5m
Date: 13-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		15		SANDSTONE: fine to medium grained, light grey brown, massive.	SW-FR	H	×		- XWS, 30mm.t
				as above, but with shale inclusions.	DW-SW		×		- CS. 140mm.t
		16		SANDSTONE: fine to coarse grained, light grey, with a trace of dark grey laminae.	FR		×		- Be, 5°, P, S, 5mm CLAY INFILL
							×		
				END OF BOREHOLE AT 16.36m					
		17							
		18							
		19							
		20							

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Borehole No. **8** 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 46.3m

Date: 10-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DAMP ON COMPLETION OF AUGERING						0			ASPHALTIC CONCRETE: 50mm.t over Roadbase 150mm.t.				
								SM	FILL: Silty gravelly sand, fine to coarse grained, grey, fine to coarse grained igneous gravel.	D	MD		AEOLIAN
						1			SILTY SAND: fine to coarse grained, grey.				
									SILTY SAND: fine to coarse grained, light yellow brown, with occasional cemented bands.				
					N = 13 5,11,12	2			SILTY SAND: fine to medium grained, dark grey brown.				
									SILTY SAND: fine to coarse grained, light yellow.				
					N = 23 10,10,13	3							
						4		-	SANDSTONE: fine to medium grained, light grey and orange brown.	XW-DW	VL-L		LOW 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

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JOB No 22529Z BH8 START CORING AT 4.33m
4
CORE LOSS 0.14m
5
6
7
8
9
10
11
12
13
14
END OF BH8 AT 14.57m

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Borehole No. **8** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 46.3m
Date: 10-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** A.I./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_S(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		4		START CORING AT 4.22m					
				CORE LOSS 0.14m					
FULL RET-URN		5		SANDSTONE: fine to coarse grained, red brown, orange brown and grey.	DW-SW	VL-L			- XWS, 40mm.t - XWS, 5mm.t - XWS, 20mm.t - XWS,10mm.t - CS, 15mm.t - XWS, 10mm.t
		6		SANDSTONE: fine to medium grained, grey, with occasional dark grey laminae, bedded at 0-5°.	SW F	L M-H			- XWS, 20mm.t
		7							
80% RET-URN		8				H			- XWS, 20mm.t - XWS, 10mm.t
		9		as above, but light grey.					- XWS, 10mm.t
		10							
				SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0°.	SW FR	L-M H			

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Borehole No. **8** 3/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 46.3m
Date: 10-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** A.I./8

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
80% RET URN		12		SANDSTONE: fine to coarse grained, light grey.	FR	H	×		- XWS, 30mm.t	
				as above, but fine to medium grained, with occasional dark grey laminae, bedded at 0-10°.			×			
		13				M-H	×			
							×		- XWS, 3mm.t	
		14					×			
				END OF BOREHOLE AT 14.57m						
		15								
		16								
		17								

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Borehole No. 9 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 46.6m

Date: 14-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			ASPHALTIC CONCRETE: 100mm.t				
								-	FILL: Sandy gravel, fine to coarse grained igneous and sandstone, dark grey.	-	-	-	
								SM		D	MD		AEOLIAN
								SP	SILTY SAND: fine to coarse grained, brown.				
						1			SAND: fine to coarse grained, yellow light brown.				
									as above, but dark brown.				
					N = 11 5,5,6								
						2			as above, but light brown yellow.				
					N = 23 6,11,12	3							
								-	SANDSTONE: fine to coarse grained, light grey and red brown.	DW	VL-L	-	LOW 'TC' BIT RESISTANCE
						4					L		LOW TO MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						5							
						6							
						7							

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JOB NO. 22529Z, BH9 START CORING AT 4.55

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Borehole No. **9** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 46.6m
Date: 14-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK300 **Bearing:** - **Logged/Checked by:** A.I./

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 DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific / General
		4		START CORING AT 4.55m					
		5		SANDSTONE: fine to coarse grained, grey and orange brown.	DW-SW	M-H			- XWS, 10mm.t - XWS, 30mm.t
				SANDSTONE: fine to medium grained, grey.	SW				
				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-5°.					- CS, 20mm.t
		6		SANDSTONE: fine to coarse grained, light grey, with dark grey laminae, bedded at 0-10°.	SW-FR	H			
		7							- XWS, 8mm.t
				SANDSTONE: fine to medium grained, light grey.					- XWS, 10mm.t - XWS. 5mm.t
		8							- XWS, 25mm.t
				SANDSTONE: fine to coarse grained, light grey, with dark grey laminae, bedded at 0-10°.	FR				- Be, 1°, P, R
		9		SANDSTONE: fine to coarse grained, light grey.					- Be, P, R
90% RET-URN		10							- Be, P, R
				SANDSTONE: fine to coarse grained, light grey, with dark					

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Borehole No. **9** 3/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 46.6m
Date: 14-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK300 **Bearing:** - **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
				\grey laminae, bedded at 0-5°.	FR	H			
				SANDSTONE fine to coarse grained, light grey, with dark grey laminae, bedded at 0-5°.					- XWS, 5mm.t
		12							
		13							
		14							
		15							
				END OF BOREHOLE AT 15.80m					
		16							
		17							

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Borehole No. **16** 1/3

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 45.4m

Date: 10-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			ASPHALTIC CONCRETE: 50mm.t over Roadbase: 100mm.t				
									FILL: Sandy gravel, fine to coarse grained, igneous and sandstone, dark grey.				
						1		SP	FILL: Silty sandy gravel, igneous and sandstone, dark grey. SAND: fine to medium grained, grey, with silt.	D	L		AEOLIAN
					N = 7 2,2,5	2							
								SM	SILTY SAND: fine to medium grained, light yellow brown.		MD		
					N = 11 4,5,6	3							
						4		SC	CLAYEY SAND: fine to medium grained, light grey mottled orange brown and red.				
					N > 10 16,10/ 80mm REFUSAL	5		-	SANDSTONE: fine grained, red brown.	DW	VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
									SANDSTONE: fine to medium grained, light grey.	DW-SW	L		LOW TO MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						6							
						7							

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Borehole No. **16** 2/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Core Size:** NMLC **R.L. Surface:** ≈ 45.4m
Date: 10-11-08 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** A.I./

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
		5		START CORING AT 5.34m					
▼ ON 13-11-08 90% RET-URN		6		SANDSTONE: fine to medium grained, light grey, with occasional dark grey laminae, bedded at 0-5°.	DW-SW	M			- Be, 5°, P, S - XWS, 20mm.t
				SANDSTONE: fine to coarse grained, light grey, with light brown laminae, bedded at 0-10°	SW-FR	H			- XWS, 20mm.t - XWS, 90mm.t
		7							- CS, 5mm.t
		8							- XWS, 20mm.t
60% RET-URN				SANDSTONE: fine to coarse grained, light grey, with occasional dark grey laminae, bedded at 0-5°.	FR				
		9							- Be, 5°, P, S, CLAY SMEAR
		10							- XWS, 3mm.t - XWS, 10mm.t
NO RET-URN		11							

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Borehole No. **16** 3/3

CORED BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE
Project: STAGE 2 POWMRI
Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z | **Core Size:** NMLC | **R.L. Surface:** ≈ 45.4m
Date: 10-11-08 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** A.I./

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
		13		SANDSTONE: fine to coarse grained, light grey, with occasional dark grey laminae, bedded at 0-5°.	FR	H	× × × ×		
10% RET-URN		14							
				END OF BOREHOLE AT 14.4m					50mm DIAMETER PVC STANDPIPE PIEZOMETER INSTALLED TO BASE OF BOREHOLE. SLOTTED FROM 2m TO 14.4m DEPTH. CAST IRON GATIC COVER AT SURFACE
		15							
		16							
		17							
		18							

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Borehole No. **3A** 1/1

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** HAND AUGER **R.L. Surface:** ≈ 44.6m

Date: 19-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			TOPSOIL: Silty sand, fine to coarse grained, dark brown, with a trace of root fibres.	D			
								SP	SAND: fine to medium grained, brown.	D	L	-	AEOLIAN
						1			as above, but grey.		MD		
									as above, but light grey.				
						2			END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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Borehole No. **17A** 1/1

BOREHOLE LOG

Client: PRINCE OF WALES MEDICAL RESEARCH INSTITUTE

Project: STAGE 2 POWMRI

Location: BARKER STREET, RANDWICK, NSW

Job No. 22529Z **Method:** HAND AUGER **R.L. Surface:** ≈ 44.6m

Date: 19-11-08 **Datum:** AHD

Logged/Checked by: A.I./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			TOPSOIL/FILL: Silty sand, fine to coarse grained, dark brown, with a trace of root fibres.	D			APPEARS POORLY COMPACTED
									FILL: Silty sand, fine to coarse grained, grey, with a trace of fine to coarse grained igneous and sandstone gravel and glass fragments.				
						1		SP	SAND: fine to coarse grained, dark brown, with a trace of silt fines.	D	MD		AEOLIAN
									as above, but brown.				
									as above, but light grey.				
						2			END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	PRINCE OF WALES MEDICAL RESEARCH INSTITUTE		
Project:	STAGE 2 POWMRI		
Location:	PRINCE OF WALES HOSPITAL, RANDWICK, NSW		
Job No.	22529Z	Hammer Weight & Drop:	9kg/510mm
Date:	19-11-08	Rod Diameter:	16mm
Tested By:	A.I.	Point Diameter:	20mm

	Number of Blows per 100mm Penetration						
Test Location	RL ~44.6m	RL ~44.6m					
Depth (mm)	**3A**	**17A**					
0 - 100	1	2					
100 - 200	1	3					
200 - 300	2	2					
300 - 400	3	2					
400 - 500	4	2					
500 - 600	5	1					
600 - 700	3	2					
700 - 800	4	2					
800 - 900	8	4					
900 - 1000	6	5					
1000 - 1100	7	6					
1100 - 1200	7	6					
1200 - 1300	5	8					
1300 - 1400	6	7					
1400 - 1500	5	6					
1500 - 1600	5	9					
1600 - 1700	6	7					
1700 - 1800	6	6					
1800 - 1900	8	8					
1900 - 2000	13	11					
2000 - 2100	10	12					
2100 - 2200	16	10					
2200 - 2300	22	9					
2300 - 2400	30/50mm	11					
2400 - 2500	REFUSAL	12					
2500 - 2600		14					
2600 - 2700		11					
2700 - 2800		20					
2800 - 2900		25/20mm					
2900 - 3000		REFUSAL					

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.

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A.B.N. 17 003 550 801 A.C.N. 003 550 801

CONSTANT HEAD PERMEABILITY TEST

Job No: 22529Z

Client:	POWMRI	
Project:	Stage 2 POWMRI	
Location:	Barker Street, Randwick, NSW	
BOREHOLE:	3A	
Depth:	2.00 m	
Soil Profile:	**0.0-0.2m**	TOPSOIL/FILL: Silty sand, fine to coarse grained, dark brown with occasional root fibres
	0.2-0.4m	SILTY SAND: Fine to coarse grained, brown.
	0.4-1.0m	SILTY SAND: Fine to coarse grained, grey
	1.0-2.0m	SILTY SAND: Fine to corase grained, light grey
Casing Type:	Slotted PVC	**Outside diameter:** 0.06m
Diameter of Bore (2r):	0.06 m	
DATE:	19-Nov-08	
Weather:	Occasional showers	
Recent:	Occasional showers	

Infiltration for Steady State:	8.40 l/min	
	1.40E-04 m3/s	
Height of Water in Test Borehole:	2.00 m	
Depth to Water from Top of Borehole:	0.00 m	
Radius of Test Borehole:	0.03 m	
Unsaturated Distance Between Water Surface in Borehole and Water Table or Impervious Layer:	3.50 m	(assumed groundwater level)

INTERPRETATION OF TEST RESULTS

Based on: CONDITION II; The water table or impervious layer lies at or below the base of the test borehole and is no further than three times the test depth from the base of the borehole.

Calculated Permeability	k =	3.1E-05 m/s
	=	3.1E-03 cm/s

Assumes water temperature is 20 degrees during testing.

Method of analysis has been adapted from the Earth Manual, US Department of the Interior Water and Power Resource Service, 1974.

File Name: 18285VTperm

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CONSTANT HEAD PERMEABILITY TEST

Job No: 22529Z

Client:	POWMRI	
Project:	Stage 2 POWMRI	
Location:	Barker Street, Randwick, NSW	
BOREHOLE:	17A	
Depth:	2.00 m	
Soil Profile:	**0.0-0.1m**	TOPSOIL/FILL: Silty sand, fine to coarse grained, dark brown with occasional root fibres
	0.1-0.8m	FILL: Silty sand, fine to coarse grained grey, with a trace of fine to coarse grained igneous and sandstone gravel and glass fragments
	0.8-1.0m	SILTY SAND: Fine to corase grained, light grey, dark brown
	1.0-1.6m	SILTY SAND: Fine to corase grained, light grey, brown
	1.6-2.0m	SILTY SAND: Fine to coarse grained, grey to light grey

Casing Type: Slotted PVC **Outside diameter:** 0.06m
Diameter of Bore (2r): 0.06 m

DATE: 19-Nov-08
Weather: Occasional showers
Recent: Occasional showers

Infiltration for Steady State:	25.40 l/min	
	4.23E-04 m3/s	
Height of Water in Test Borehole:	2.00 m	
Depth to Water from Top of Borehole:	0.00 m	
Radius of Test Borehole:	0.03 m	
Unsaturated Distance Between Water Surface in Borehole and Water Table or Impervious Layer:	4.50 m	(assumed groundwater level)

INTERPRETATION OF TEST RESULTS

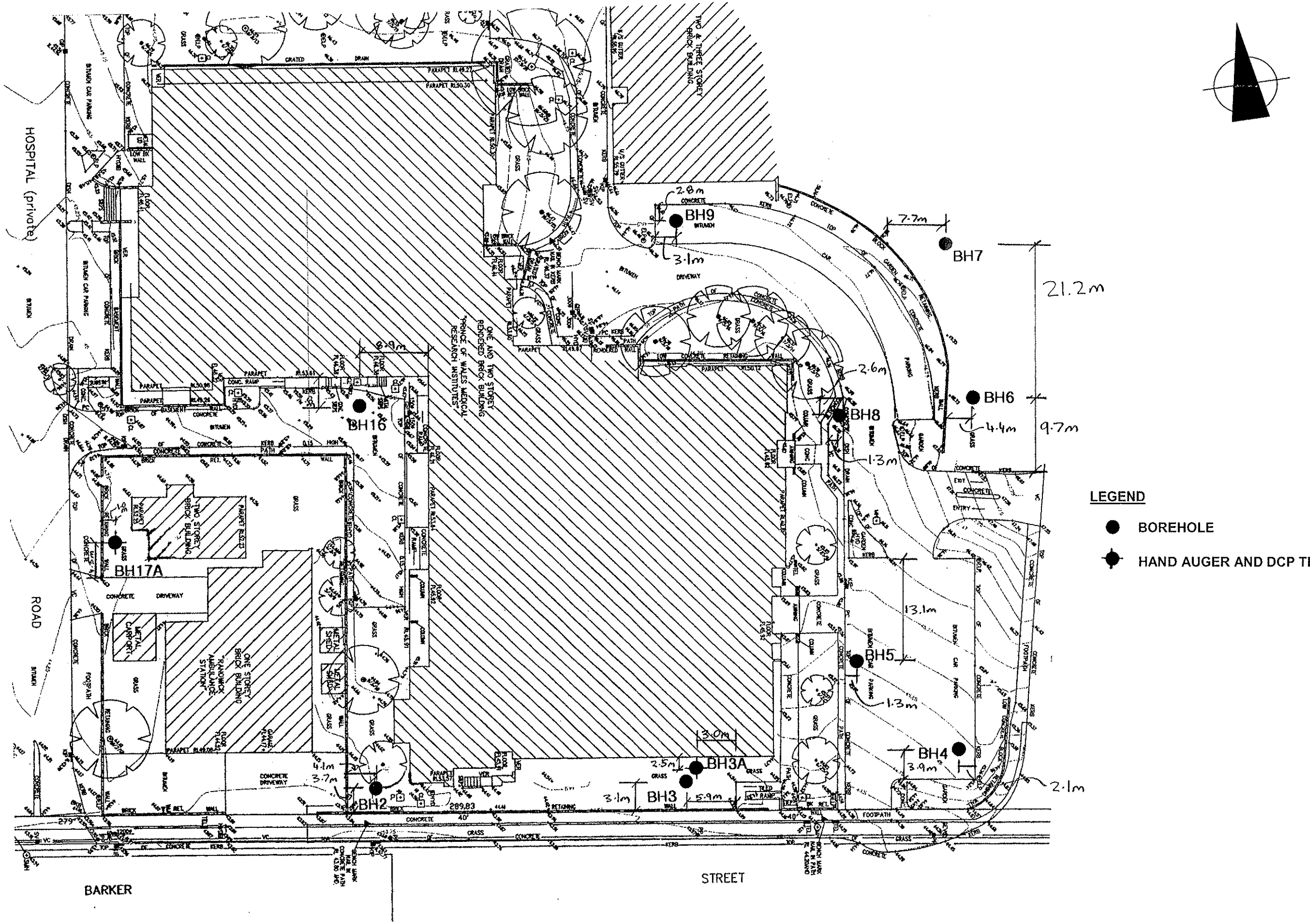
Based on: CONDITION II; The water table or impervious layer lies at or below the base of the test borehole and is no further than three times the test depth from the base of the borehole.

Calculated Permeability	k =	7.7E-05 m/s
	=	7.7E-03 cm/s

Assumes water temperature is 20 degrees during testing.

Method of analysis has been adapted from the Earth Manual, US Department of the Interior Water and Power Resource Service, 1974.

File Name: 18285VTperm



INVESTIGATION LOCATION PLAN

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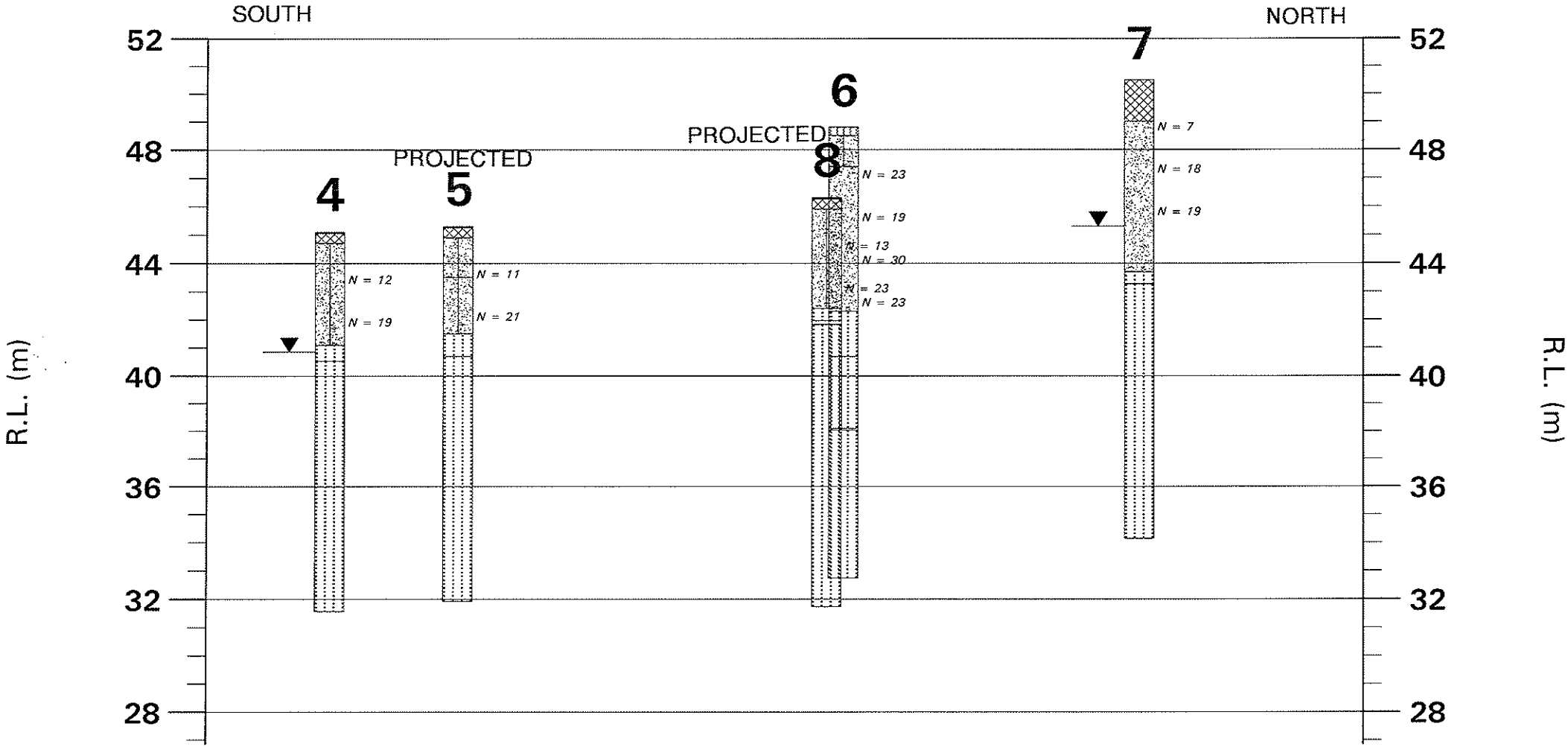
Report No. 22529Z Figure No. 1

GEOTECHNICAL CROSS SECTION

EAST FACE



Sandstone/ Greywacke
Asphaltic/ Bituminous Paving or Coal
Fill
Silty Sand
Core Loss/ Empty
Topsoil
Sand
Shale
Observed water level
N SPT "N" VALUE
Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

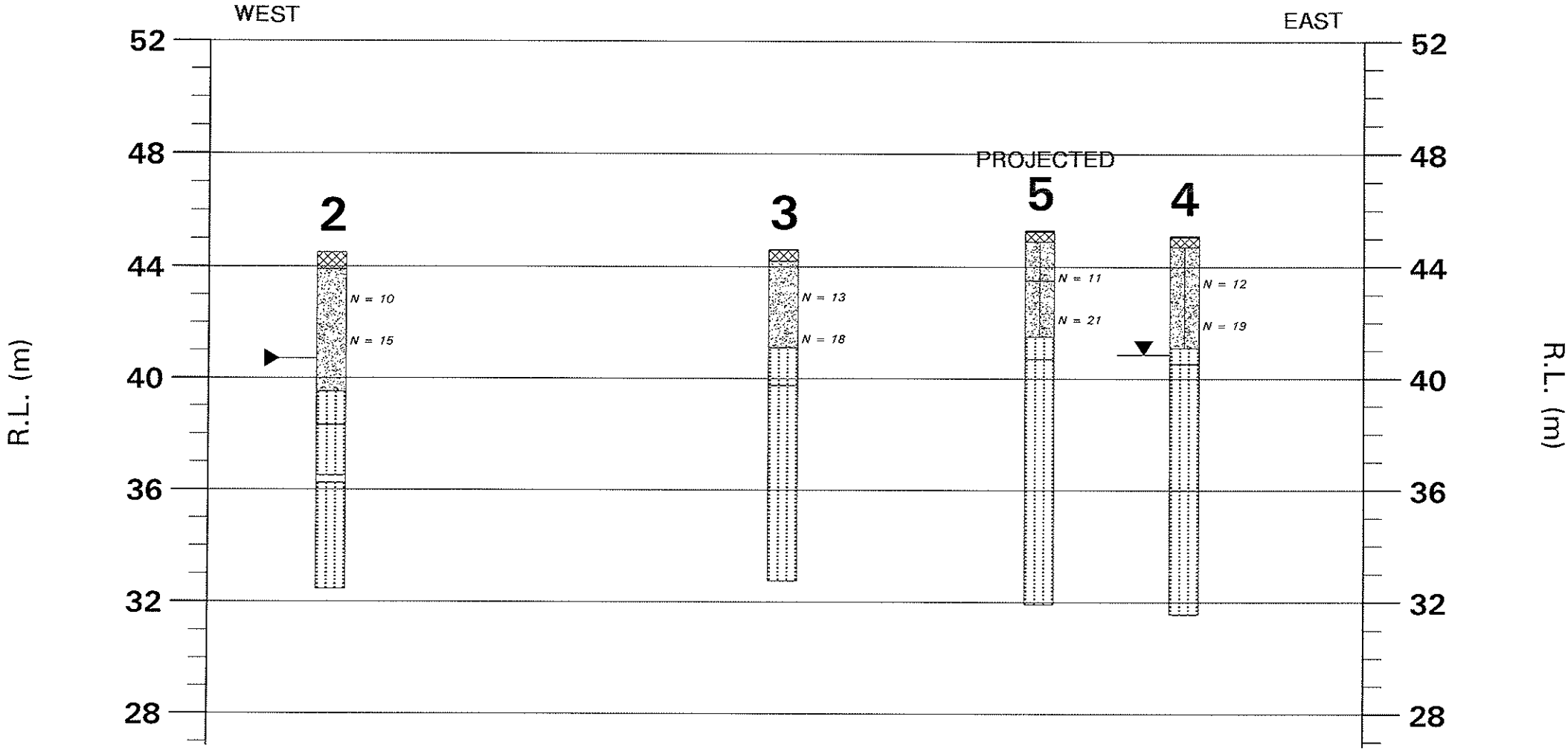
Scale: 1 : 200 (vert) ; 1 : 500 (hori)

Jeffery and Katauskas Pty Ltd

Job No.: 22529Z Figure No.: 2

GEOTECHNICAL CROSS SECTION

SOUTH FACE



Fill
Sand
Sandstone/ Greywacke
Core Loss/ Empty
Asphaltic/ Bituminous Paving or Coal
Silty Sand
Groundwater seepage level
Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; 1 : 500 (hori)

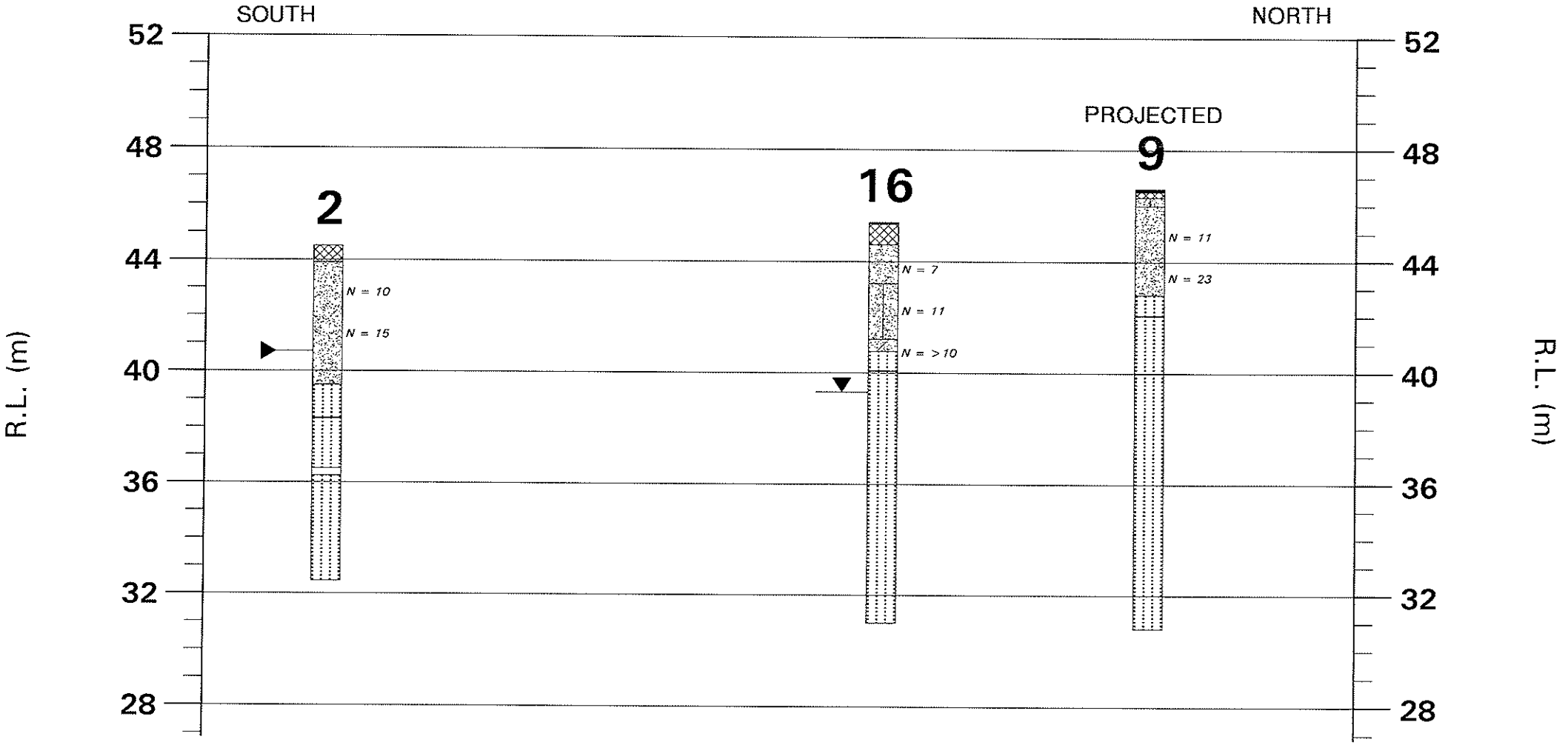
Jeffery and Katauskas Pty Ltd

Job No.: 22529Z

Figure No.: 3

GEOTECHNICAL CROSS SECTION

WEST FACE



Sandstone/ Greywacke
Core Loss/ Empty
Fill
Sand
Asphaltic/ Bituminous Paving or Coal
Silty Sand
Clayey Sand
Groundwater seepage level
Observed water level
N SPT "N" VALUE
Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; 1 : 500 (hori)

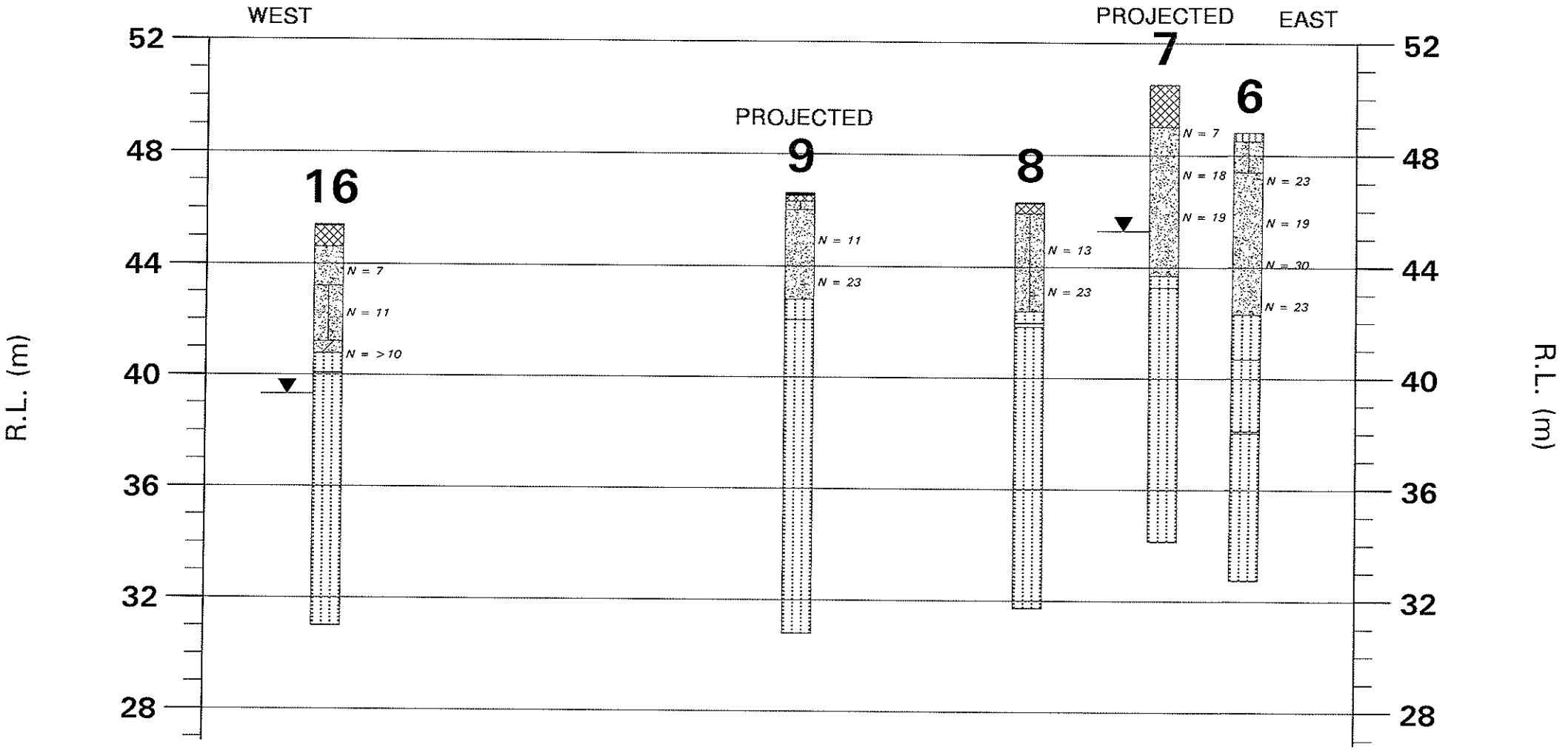
Jeffery and Katauskas Pty Ltd

Job No.: 22529Z

Figure No.: 4

GEOTECHNICAL CROSS SECTION

NORTH FACE



Sandstone/ Greywacke

Asphaltic/ Bituminous Paving or Coal

Fill

Sand

Silty Sand

Clayey Sand

Core Loss/ Empty

Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; 1 : 500 (hori)

Jeffery and Katauskas Pty Ltd

Job No.: 22529Z

Figure No.: 5

APPENDIX A

Rock Bolt

Specification

SPECIFICATION FOR ROCK BOLTS

1 Drilling Holes, including Water Testing and Cleaning Holes

a) Rock bolt holes are to be drilled at spacings and/or locations nominated on the drawings or as directed on site.

b) Required hole lengths vary as shown on the drawings or as directed on site. Drill holes should be overdrilled by an additional 500mm such that incomplete cleaning does not affect bond length of anchor.

c) The minimum acceptable hole diameter shall be as nominated on the drawing, or 75mm.

d) Rock bolt holes are to be drilled at not less than 10° below the horizontal, or as shown on the drawings and directed on site.

e) Each of the holes for permanent rock bolts is to be water tested to ensure that the water loss is not greater than 0.5 litres per minute. If water loss is found to be in excess of this criterion, the hole is to be initially grouted and then redrilled and retested until a satisfactory test result is obtained. Supervision of this procedure may be carried out to assess the need for grouting and redrilling. All holes with an unsatisfactory water loss are to be identified to the superintendent within 24 hours of the initial water test.

f) Prior to installation, all holes are to be flush cleaned by clean water passing through a hose or delivery pipe inserted to the base of the hole. The hole will be pronounced clean once clear or almost clear water is being returned out of the hole opening. This procedure shall be supervised to ensure it is being carried out correctly.

g) On completion of drilling and flushing, all holes should be plugged or otherwise protected to prevent entry of foreign matter.

h) The contractor is to record for each hole, date drilled, length drilled, orientation of hole, time of water test, water test result, details of grouting and redrilling if required. The details are to be provided to the superintendent prior to installation of the rock bolt.

2 Rock Anchors

a) Rock bolts are to consist of a hot-dipped galvanised N24 rod, threaded at one end (head) unless nominated otherwise on the drawing. Bolts are to be locked off against a head assembly (refer 3(f) below)

b) Where used in conjunction with shotcrete, rock bolts may have a cogged 400mm minimum length dome tied to the mesh reinforcement bolts, as directed by the superintendent.

c) Total in hole length of rock bolts to be as shown on the drawings or as directed on site by the superintendent.

d) The safe working load of the rock bolts shall be adopted as 45kN unless nominated otherwise on the drawings. The selected grade of N24 bars shall not fail under these loads.

e) Care should be taken to prevent damage, kinking or bending of bolts. Any anchors sustaining damage shall not be used.

f) Bolts shall be kept free from oil, grease, mud or any other deleterious substances. The steel should not be visibly pitted or rusted.

3 Installation and Grouting

a) Spacers or spiders shall be provided along the length of the rock bolts to maintain them centrally within the drill hole.

b) Grout mix to surround rock bolt is to have a target water/cement ratio of 0.45. A target laboratory test criterion would be an average grout strength of 25MPa at seven days. (No single test shall be less than 20MPa.)

c) Grout is to be pumped to the base of the hole through hoses or grout tubes until the consistency of the grout mix escaping at the hole openings is the same as that being pumped in. Once this is the case, the grout tube may be withdrawn slowly such that the rate of grout exiting the hole is virtually maintained. Only when the tube is completely removed from the hole should the pumping mechanism be switched off.

d) If grout level drops below drill hole opening whilst still wet, it should be topped up until loss of grout is negligible. If the grout level cannot be maintained, then the rock bolt or dowel must be withdrawn and the hole treated as per Item 1(e).

e) Once grout is dry or almost dry, a thick, non-shrink topping grout should be packed into the hole until the grout completely covers the bolt/dowel up to the drill hole opening. The grout shall be finished flush with the surrounding rock face.

f) The rock bolt head assembly shall comprise base plate resting on top of the rock face, or shotcrete reinforcement, or a mortar seating pad and a hemispherical seating washer and nut tightened against the plate. In certain locations, a double plate system will be required and the threaded length of the bolts will be sufficient to permit this arrangement where directed by the engineer.

g) Where mortar pads are required, the mortar shall be non-shrink of a strength at least equal to the grout. The mortar pad shall be formed to the required size and the bearing plate seated to provide uniform bearing.

h) The end plates may be fitted and nuts nipped tight no sooner than four days after grouting.

4 Load Testing of Rock Bolts

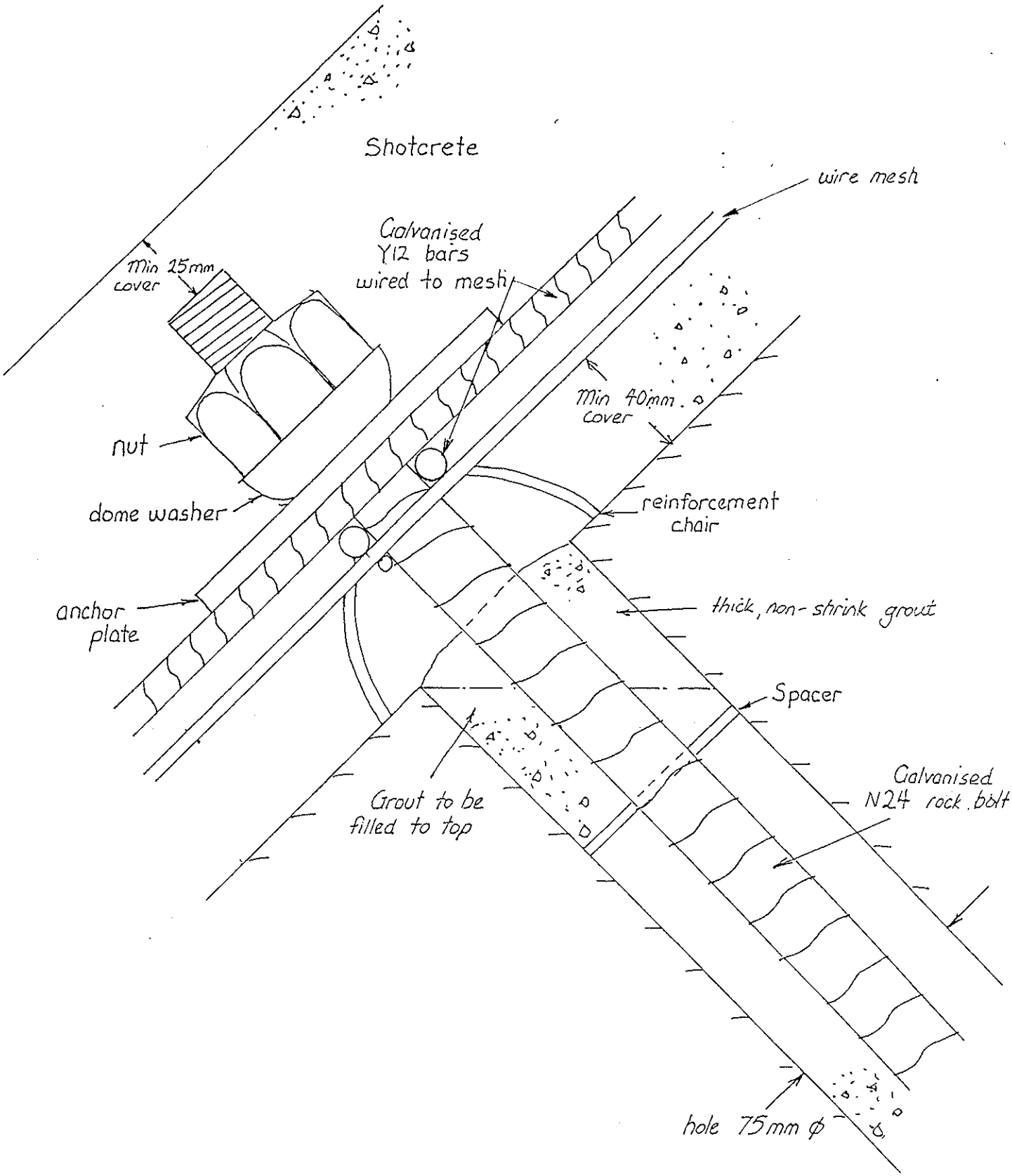
Load testing of rock bolts shall be as directed by the superintendent. Temporary rock bolts will not normally be load-tested.

5 Australian Standards

Wherever Australian Standards exist with regard to the materials and workmanship referred to in this Specification, then they shall be deemed to apply.

6 Reference Drawings

The above Specification to be read in conjunction with attached 22529Z Sketch 1.



TYPICAL ROCK BOLT HEAD DETAIL

Jeffery & Katauskas Pty Ltd

Report No. 22529Z Sketch 1

APPENDIX B

Report Explanation

Notes

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 subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

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

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

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

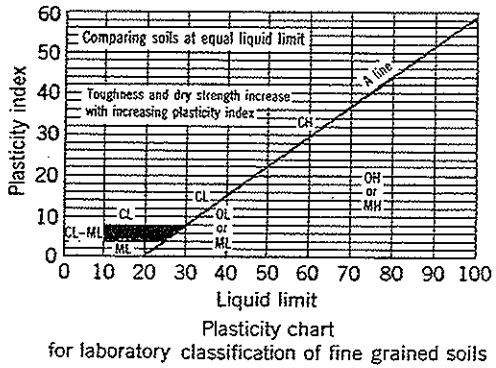

BITUMINOUS CONCRETE, COAL


COLLUVIUM

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

UNIFIED SOIL CLASSIFICATION TABLE

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



Plasticity chart
for laboratory classification of fine grained soils

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

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

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LOG SYMBOLS

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

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL	0.03	Easily remoulded by hand to a material with soil properties.
Very Low:	VL	0.1	May be crumbled in the hand. Sandstone is "sugary" and friable.
Low:	L	0.3	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.
Medium Strength:	M	1	A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
High:	H	3	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.
Very High:	VH	10	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.
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	