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Geotechnics • Environment • Groundwater

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REPORT
on
GEOTECHNICAL INVESTIGATION

JUSTINIAN HOUSE REDEVELOPMENT
18-22 SINCLAIR STREET, WOLLSTONECRAFT

Prepared for
ST VINCENTS & MATER HEALTH

Project 45046
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1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Douglas Partners Pty Ltd (DP) on behalf of St Vincents & Mater Health for a proposed redevelopment of Justinian House. The work was carried out in general accordance with the DP proposal dated 9 July 2007. The investigation was commissioned by Savills (Aust) Pty Ltd, project managers for the project.

The construction of a four-storey medical research facility, with ground and basement level car parking is proposed for the site. The geotechnical investigation was carried out to provide information for detailed design purposes.

The purpose of the investigation was to identify and assess:

- The soil and rock profile in the vicinity of the proposed works;
- Likely excavation conditions and excavation support requirements;
- Foundation types, founding levels and allowable bearing pressures;
- Groundwater issues; and
- Relevant other geotechnical issues such as slope stability, soil and groundwater aggressivity, and potential impacts on adjacent properties.

The investigation comprised drilling of exploratory boreholes and in-situ sampling and testing of soils, followed by laboratory testing of soil, rock and groundwater samples. Details of the field and laboratory work are given in the report, together with comments addressing the objectives of the investigation.

2. SITE DESCRIPTION

The site comprises an L-shaped area of about 6710 m^2, bounded to the west, south and east by existing public roads. Site levels fall generally in the south easterly direction with an approximate overall grade of 8.5H:1V and an overall difference in level of about 8.5 m. The presence of numerous existing retaining structures give the site a terraced profile.

The majority of the site, being its southern part, is occupied by an existing multi-level brick building named Justinian House. The northern part of the site is occupied by two, existing one- and two- storey brick cottages. A small, existing, bitumen-paved carpark is located at the rear of the existing cottages along Sinclair Street. An existing, in-ground swimming pool is situated on the western side of the site adjacent to Gillies Street. The existing structures are skirted by concrete driveways and footpaths, both suspended and at-grade, as well as landscaped gardens that include a sparse cover of trees.

3. GEOLOGY

The Geological Survey of NSW 1:100,000 Geological Series Sheet 9130 (Sydney) indicates that the site is close to a boundary of Ashfield Shale with the underlying Hawkesbury Sandstone. Hawkesbury Sandstone is generally a medium to course grained quartz sandstone, with minor shale and laminate layers, while Ashfield Shale typically comprises black to dark grey shales and laminites. No major geological structures, such as dykes or faults, are indicated at the site.

No rock outcrops, natural exposures or cuttings were observed at or near the site.

4. FIELD WORK

4.1 Field Work Methods

The field investigation comprised three boreholes drilled with a limited access, track-mounted drilling rig. Diatube coring of existing concrete pavements was initially required for two of the three boreholes, before the boreholes were drilled to depths of 1.3 – 4.6 m with 110 mm diameter solid flight augers, and thereafter advanced through rock to depths of 9.0 – 12.9 m using diamond coring techniques to obtain NMLC-sized rock cores.

Standard penetration tests (SPTs) were carried out at 1.0 – 1.5 m depth intervals in soil, with disturbed sampling of soils taken from the auger tip and using the SPT split-spoon sampler.

Drilling was attended by an experienced geotechnical engineer, who located the boreholes and logged the material encountered. The boreholes were backfilled with excavated spoil on completion, except where piezometers were installed. Ground elevations at borehole locations were obtained by levelling from known benchmarks.

Following installation of two piezometers, the wells were purged, groundwater levels measured and groundwater samples taken nearly two weeks after completion of the drilling program.

Note that borehole BH3 could not be drilled due to safety concerns relating to the proximity of existing overhead powerlines.

4.2 Field Work Results

Appendix A contains Drawing 1, a site plan showing the location of the exploratory boreholes and cross-sections. Drawings 2 and 3 are also contained in Appendix A, and comprise two summary cross-sections of the sub-surface profile across the site. Borehole logs are given in Appendix B, together with notes defining classification methods and descriptive terms.

Boreholes and our interpretation of site observations confirm the information contained on the Sydney geology map, that the site is near the boundary between Ashfield Shale overlying

Hawkesbury Sandstone. The boundary between Ashfield Shale and Hawkesbury Sandstone falls somewhere between the up-slope Sinclair Street and the down-slope Gillies Street. Ashfield Shale capping was absent from the down-slope end of the site. The sub-surface conditions encountered in the boreholes, up-slope and down-slope, are summarised in Tables 1a and 1b, respectively.

Table 1a Summary of Up-Slope Ground Profile

Depth to top of stratum (m)	Stratum	Description
0.0	1	Discontinuous CONCRETE pavement over loose to medium dense sandy RESIDUAL SOILS and FILLING.
0.7 – 0.8	2	Sandy Silty CLAY, stiff, medium to high plasticity, with a minor proportion of gravel. RESIDUAL SOIL.
1.0 – 1.5	3	Shaley CLAY/Clayey SHALE, extremely low strength, and SHALE, extremely low to medium strength, moderately to slightly weathered, fragmented to fractured. Assessed as Class V SHALE*.
4.6 – 5.3	4	SILTSTONE/SANDSTONE, medium to high strength. Assessed as Class III SANDSTONE*. A discontinuous, medium strength laminite layer about 1 m thick was encountered at the base of this stratum in the northern part of the site. The laminate layer was assessed as Class III Shale*.
8.0 – 8.3	5	SANDSTONE, mainly high strength, fresh, slightly fractured to unbroken. Assessed as Class I SANDSTONE*.

*Assessed in accordance with the classification system of P.J.N. Pells et al in their paper entitled *"Foundations on Sandstone and Shale in the Sydney Region"*, Australian Geomechanics, December 1998.

Table 1b Summary of Down-Slope Ground Profile

Depth to top of stratum (m)	Stratum	Description
0.0	1	CONCRETE pavement over Clayey GRAVEL of sandstone fragments. Probably extremely weathered SANDSTONE.
0.4	2	SANDSTONE, very low to medium strength, slightly to moderately weathered, slightly fractured. Assessed as Class IV SANDSTONE*.
3.2	3	SANDSTONE, mainly high strength, fresh, slightly fractured to unbroken. Assessed as Class I SANDSTONE*.

*Assessed in accordance with the classification system of P.J.N. Pells et al in their paper entitled *"Foundations on Sandstone and Shale in the Sydney Region"*, Australian Geomechanics, December 1998.

4.3 Groundwater

Free groundwater was measured at reduced levels between 74.7 - 82.3 m AHD during a return visit to the site to collect groundwater samples from the earlier installed piezometers. Comparison with the borehole records indicates that the phreatic surface was located within the sandstone strata at the date of measurement.

5. LABORATORY TESTING

NATA-registered laboratories were used to carry out the following laboratory tests on samples obtained during the fieldwork:

- 1 No. Corrosion Assessment (Sulphate + Chloride + pH) of a soil sample;
- 2 No. Corrosion Assessment (Sulphate + Chloride + pH) of groundwater samples; and
- 24 No. Point Load Tests of rock core samples.

The results of laboratory testing for Corrosion Assessment, reported in Appendix C, are summarised in Table 2. For comment on these results refer to Section 7.6.

Table 2 Corrosion Assessment Results

Sample	Sample type	Soluble Sulphate as SO_4(mg/kg)	Choride (mg/kg)	pH units
BH4, 0.2 m	Soil	<25	<100	8.2
BH1, 5.10 m	Groundwater	78	83	6.2
BH4, 5.73 m	Groundwater	99	70	5.5

The results of Point Load Testing of rock core samples are reported on the borehole logs contained in Appendix B.

6. PROPOSED DEVELOPMENT

It is understood, based on a telephone conversation with Lawson Katiza of Savills (Australia) Pty Ltd on 6 July 2007, that the proposed redevelopment of the site will comprise the demolition of existing structures and the construction of a new four-storey medical reseach facility with ground and basement level carparking. It is further understood that the design elevation of the lowest proposed basement slab is RL 78.1 m AHD, about 9.5 m below the highest point of the site, based on the survey drawing 04100-2 by Brunskill McClelahan & Associates Pty Ltd dated 22 June 2007, and DP levelling of borehole locations. The Geotechnical Brief for the redevelopment of Justinian House, provided by SCP Consulting Pty Ltd through Savills (Australia) Pty Limited and dated 26 June 2007, states that the estimated maximum column loads will be up to 3000 kN working and 4200 kN ultimate.

7. COMMENTS

7.1 Excavations

7.1.1 Excavation Methods

Depths of excavation will vary across the site due to variations in existing surface levels. The proposed basement slab level is approximately 2.5 to 9.5 m below the present ground surface level at the site. As shown in the cross sections, Drawings 2 and 3, of Appendix A, the upper zones of the excavation are expected to include filling, residual sandy, gravelly and clayey soils, and extremely low to medium strength shale. These materials should readily be excavated using conventional earthmoving equipment or hydraulic excavators, probably with some medium ripping assistance through the shale. The lower zones of excavation are expected to include extremely low to high strength sandstone, siltstone and shale, which are likely to require heavy ripping in conjunction with hydraulic excavators fitted with rock breakers or milling heads.

Excavation for footings and trenches will also probably require the use of hydraulic excavators fitted with rock breakers. At the proposed basement slab level, high strength sandstone is intersected and will be particularly difficult based on the unbroken nature of the rock core samples possibly requiring the use of a rotary rock saw or milling head.

7.1.2 Disposal of Excavated Materials

The materials that will be derived from the excavation works may include significant amounts of filling and natural soil overburden from within the proposed bulk excavation footprint (assumed to correspond with the site footprint). It should be noted that any off-site disposal will require assessment for re-use or classification of the excavated material in accordance with "Environmental Guidelines: Assessment, Classification and Management of Non-Liquid Wastes (NSW EPA, 1997)" prior to disposal at an appropriately licenced landfill.

7.1.3 Vibration

Noise and vibration will be caused by excavation work on the site, and precautions will therefore be required when excavating close to adjacent buildings. The level of acceptable vibration is dependent on various factors including the type of building structure (e.g., reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium.

The Australian Standard AS 2187.2 1993 (Explosives Code) recommends the maximum peak particle velocity (PPV) of 25 mm/s for commercial and industrial structures of reinforced concrete or steel construction subjected to vibration. A lower PPV limit of 10 mm/s is prescribed for houses and low-rise residential or commercial buildings. Ground vibration arising from excavation plant is of a continuous, rather than transient, nature, unlike blasting events. Thus, more stringent vibration limits than those given for blasting should generally apply. It is likely that the neighbouring brick buildings are founded on the underlying bedrock surface, and it is therefore suggested that PPV be generally limited to 8 mm/s at the adjacent building line.

It is noted that vibration levels above 3 mm/s may be disturbing to the adjacent property owners and some complaints from neighbours are probable. Some reassurance, possibly via vibration monitoring, may be necessary.

Vibration monitoring carried out by Douglas Partners at various excavation sites around Sydney has indicated that to limit vibrations (PPV) to 5 mm/s, a Krupp 600 kg or 900 kg (or equivalent) hydraulic hammer should not be used within 6 m or 15 m, respectively, from the building or structure in question.

If vibrations from excavation of sectionsof the site are a potential problem, then consideration could be given to rock sawing or rock milling methods of rock excavation.

To respond to potential claims resulting from construction activities, it is suggested that dilapidation surveys be conducted on adjoining buildings prior to the commencement of work on site. Buildings supported on shallow foundations can be particularly susceptible to the detrimental effects of settlement and vibration.

7.1.4 Slope Stability to Open Excavations

In the absence of detailed information with respect to the geometry of the proposed new basement, it is assumed that the basement excavation footprint corresponds with the site footprint. Battered excavations are therefore considered to be unsuitable for the proposed works, as insufficient space will be available.

7.2 Excavation Support

7.2.1 General

Based on the assumption that the basement excavation footprint corresponds with the site footprint, retaining structures will be required to support the perimeter boundary excavations, both during the basement construction process and as part of the final structure.

Some forms of shoring and/or underpinning may be designed to be incorporated in the permanent excavation support. Alternatively, the final structure may be used to prop or brace the retaining wall system in the longer term, allowing temporary anchors to be released (i.e. destressed). Shoring support methods and possibly underpinning systems will generally require tie-back anchors for stability, particularly where limited ground movements behind the wall are essential. The legal implications of the use of rock anchors extending onto neighbouring properties and public land will need to be considered. Approval should be sought from Council and adjacent property owners.

The following shoring options may be considered for the support of the proposed excavations:

- Contiguous Pile Wall – consisting of closely spaced, or touching, small diameter bored (or continuous flight auger (CFA)) and socketed reinforced concrete piles. The wall may form part of the final structure, sealed by a shotcrete panel facing that is constructed as the bulk excavation progresses, or simply by mortar filling the gaps in between the piles (with appropriate drainage incorporated). One or more rows of ground anchors tied into waling beams are generally required.
- Soldier Pile/Infill Panel Wall System – consisting of bored or CFA rock socketed piles installed at typical intervals of 2-3 m centres in advance of excavation. Then, as excavation proceeds, structurally reinforced infill panels, or similar, are constructed in between the piles. The piles are often designed to also provide foundation support for the perimeter of the structure. Piles are normally drilled with minimum “toe in” design to provide lateral restraint at the base of the excavation based on the passive resistance of the rock in which the pile is socketed. Again, one or more rows of ground anchors tied into waling beams are generally required.

Soldier piles in conjunction with reinforced shotcrete panels are commonly used in Sydney for excavation support in cohesive soils overlying weak rocks. At the Sinclair Street end of this site, the upper 1 m of the ground profile consists of loose sandy soils and filling, and will require that the type of excavation support be varied such that the top 1 m of the ground profile is supported by the provision of continous support to the face in the form of horizontal laggings or sheeting behind the soldier piles in advance of excavation. Below this level, the exposed soil and rock profile in between the soldier piles is expected to be temporarily self-supporting for panel depths up to about 2 m, until the ground anchors are installed and the reinforced infill panels constructed.

At no stage should progressive vertical excavation exceed 2.5 m without infill panel support being constructed. A maximum depth increment of 1.0 m to 1.5 m is recommended for excavation over the upper 3 m of the profile at this site. It is possible that adverse jointing may cause localised instability in the exposed material (e.g. unstable wedges) which may require remedial measures prior to shotcreting. It is suggested that regular inspection of the excavated spaces between soldier piles be carried out by an experienced engineering geologist or geotechnical engineer during the course of excavation works to advise further on such stabilisation measures (e.g. rock bolting).

It is likely, in the ground conditions revealed by this investigation, that at some locations soldier piles can only be taken part way down the excavation due to the presence of medium to high strength sandstone and siltstone which may prevent further drilling. In medium strength Hawkesbury Sandstone, support is generally not required other than from rock anchors for general stability if joint conditions so dictate. The need for rock anchors is likely where the excavation intersects a discontinuous laminite stratum within the sandstone, due to the presence of a significant amount of moderate to high angle jointing, as found in borehole BH1.

Nevertheless, it is recommended that soldier piles, or other permanent wall piles, are socketed a minimum of two pile diameters into at least medium strength sandstone or siltstone prior to termination, except where they are required to carry structural loads from the proposed structure, where longer rock sockets may be required. Soldier piles may be designed on the basis of the allowable foundation pressures given in Section 7.4, to carry structural compression loads associated with the proposed structure.

Drainage is normally provided behind soldier pile/infill panel wall systems using one of a number of proprietary strip drains combining a filter fabric and a cellular plastic matrix. A width of between 100 mm and 300 mm is usually adequate for strip drains, with one or two strips installed against the face of each panel.

Where the shoring system is required to support the footings for an adjacent structure, a soldier pile/infill panel wall system is not recommended due to the risk of a loss of material occurring beneath the existing foundations. A temporarily anchored contiguous pile wall, for example, would generally reduce the risk of this situation occurring.

Prospective drilling/piling contractors should be required to inspect the rock core obtained during the investigation, to determine the feasibility of their machines drilling sockets into the medium and high strength rock.

Based on the occurrence of a significant amount of moderate to high angle jointing in the laminate stratum within the Hawkesbury Sandstone encountered in borehole BH1, mass instability could also develop due to sliding along the joint planes.

7.2.2 Design

Excavations braced/anchored either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of the Class III Sandstone (refer to Drawings 2 and 3). Table 4 contains active earth pressures and bulk unit weights that are recommended for the design of gravity, cantilever or single propped/anchored walls, assuming a level surface behind the wall.

Table 4 Recommended Active Earth Pressure Coefficients and Bulk Unit Weights

Material	K_a		γ_b (kN/m^3)
	Short term/Temporary	Long term/Permanent	
Loose sandy soils and filling	0.3	0.4	19
Stiff to very stiff clay	0.25	0.3	19
Class V Shale	0.2	0.25	22
Class III Shale	0.2	0.25	24
Class IV/III Sandstone	0	0	24
Class I Sandstone	0	0	24

Due to the proposed maximum depth of excavation of approximately 9.5 m, it will be necessary to install several rows of temporary anchors to support the retaining wall system. Careful planning will be required to ensure that ground anchors do not intersect the alignment of existing or planned piled foundations.

Preliminary design for lateral earth pressures for a multi-anchored wall system may be based on a uniform rectangular earth pressure distribution of 4H, where H is the depth to the top of the Class IV/III Sandstone. For situations where only minor lateral movements are acceptable, such as the support of adjacent building footings, an increased (uniform) pressure of between 6H and 8H should be adopted, depending on the level of restraint required. For detailed wall design a computer modelling package such as WALLAP or FLAC is recommended capable of estimating internal wall movements as well as stresses.

In order to reduce the risk of damage occurring to adjacent structures the additional lateral pressures acting on the retaining structure due to surcharging of the ground behind the wall should be considered. Similarly, the additional lateral pressures arising from adjacent pavement areas behind the wall, particularly due to construction traffic surcharge loading (e.g. 5-10 kPa), should also be considered. To increase the wall stiffness and thereby reduce lateral (inward) wall deflections in these situations, the active earth pressure coefficients shown in Table 4 should generally be increased by 50 % for design purposes.

The pressure distribution given above does not include hydrostatic pressures due to the build-up of groundwater behind any retaining wall. The full hydrostatic head should be considered in design if positive drainage measures are not incorporated, to prevent groundwater pressure build-up behind the wall. Where hydrostatic pressures apply, the buoyant unit weight of soil can be adopted for calculation of lateral earth pressures.

Where appropriate, lateral restraint may also be developed by embedding the piles below the base of the excavation and developing passive pressure. The ultimate passive resistance available by embedding the piles into the Class IV/III and Class I sandstone intersected at the bulk excavation level and thus the required minimum "toe in" can be estimated using the value of 6000 kPa. This value may be adopted below one pile diameter beneath the bulk excavation level. It is noted that this is an ultimate value and should incorporate a factor of safety to limit wall movement. Jointing and other defects may be a controlling factor for passive pressure in rock and therefore embedded piles will require geotechnical inspection and confirmation during excavation.

Where piles are terminated above the basement excavation level, however, it will be important to assess the stability of the rock directly beneath each pile. Generally no passive pressure will be available and as such it may be necessary to restrain the toe of each pile with temporary or permanent rock bolts, as appropriate.

The design of the temporary shoring system and possibly the long-term basement must also cater for a possible mobilised wedge that would give rise to a total anchor force of $4.2*h^2$ (kN/m) where h is the full height of the proposed excavation (m). This is based on an anchor inclination of 10° below horizontal and the following assumed material and strength parameters:

- Planar failure on a joint/fault dipping at 45°, striking parallel to and "daylighting" at the base of the excavation;
- Shear strength along joint interface: $\phi = 25°$, $c' = 0$ kPa; and
- Bulk unit weight of sandstone and shale wedge: $\gamma_b = 22$ kN/m^3.

A factor of safety of unity (1.0) may be adopted for this design approach given that it assumes an unlikely combination of adverse factors likely to be encountered on the site. If adverse conditions are observed by geotechnical inspection during excavation then additional anchors should be installed to increase the factor of safety to at least 1.5. The anchor inclination is considered to be the flattest angle that can realistically be used which will allow relatively easy anchor installation and grouting. If there is a requirement to increase the angle of installation of the anchors then, to keep a similar factor of safety to that designed for, the anchor capacity would need to be increased as shown in Table 5.

Table 5 Increased Capacity Requirement for Steeper Anchors

Angle of Installation (degrees below horizontal)	Required Increase in Capacity (%)
10	0
15	5
20	14
25	22

Inspection of the cut faces during the excavation phase should be carried out by an experienced geotechnical professional to ensure the adequacy of design. The mapping of all actual joints and faults will also allow the recalculation of the horizontal force required to restrain the actual joint wedges present for final support design. It is unlikely that the final basement structure (e.g. floor slabs, etc.) will need to be designed to restrain the full ($4.2h^2$) mobilised wedge load. In most cases it is generally adequate for the permanent basement walls to be designed to support lateral earth pressures. It is noted that this approach to permanent support design will however require interaction between the Structural and Geotechnical Engineer.

The design of temporary shoring systems and the final basement structure should be based on the more severe of the two mechanisms defined previously, viz. lateral earth pressures and possible mobilised wedge loading.

7.2.3 Ground Anchors

Where necessary the use of inclined pre-stressed tie-back (ground) anchors is suggested for the lateral restraint of perimeter piled wall systems. Such ground anchors should be inclined below the horizontal, as steeply as possible, to allow anchorage into the stronger bedrock materials at depth. The design of temporary and permanent ground anchors for the support of piled wall systems may be carried out on the basis of the maximum allowable average bond stresses given in Table 6.

Table 6 Bond Stresses for Anchor Design

Material Description	Maximum Allowable Average Bond Stress (kPa)
Class V Shale	100
Class III Shale	200
Class IV/III Sandstone	200
Class I Sandstone	700

Ground anchors should be designed to have a free length equal to their height above the base of the excavation, a minimum 3 m bond length and after installation they should be proof loaded to 125% of the design working load and locked-off at no higher than 60% of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes.

The parameters given above can only safely be adopted on the condition that anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with normal good anchoring practice.

In normal circumstances the building will restrain the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors would generally require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.
It may be necessary to obtain permission from North Sydney Council for installing temporary or permanent anchors around the perimeter of the site as installation may encroach into council property. In addition, care should be taken to avoid damaging buried services or pipes during anchor installation.

7.2.4 Ground Movements

For a relatively major excavation such as is proposed, there is a possibility that there will be some horizontal movement due to stress relief effects. Release of these stresses due to the excavation will generally cause horizontal movements along the rock bedding surfaces and partings.

Based on monitoring experience for excavations in the Sydney region, excavations of over 70m length may give rise to lateral stress relief movements in the order of 1 to 2 mm/m of the excavated height on the adjoining ground surface (i.e. behind the top of the excavation). Empirical data suggest that most of the movement occurs during or shortly after the bulk excavation phase.

Movements related to stress relief could cause damage to the surrounding buildings if they are founded at shallow depth, behind the proposed excavation. It is recommended that appropriate allowance be made for the repair of pavements and public utilities, where excavation is carried out close to such structures. Also, with respect to adjacent buildings it is recommended that a dilapidation survey be carried out prior to excavation works so that an appropriate response may be made to damage claims.

7.3 Groundwater

The basement excavation is proposed to RL 78.1 m (AHD) and therefore some seepage will occur into the basement. The rate of groundwater seepage into the basement is not expected to be great as the groundwater table is within the Class IV/III Sandstone. However, it is not possible to quantify the amount of inflow expected on the basis of the piezometer measurements. This would require large scale groundwater pumping tests over a period of several weeks. A more usual approach is to monitor the early phases of excavation below the groundwater table to assess pumping requirements over the longer term.

Pumping from open sumps is considered likely to be sufficient for controlling groundwater inflow to the excavation during construction. It is suggested that to relieve any long-term post-construction seepage accumulating below the basement floor, appropriate sub-floor drainage should be provided for the final structure. In addition, adequate cross-fall of such drains to one or more permanent sumps should be incorporated. It is anticipated that periodic pumping of this sump would generally be required using an activated pumping system.

Groundwater entering excavations and post-construction accumulation of groundwater below the basement floor will need to be disposed of in accordance with the *Protection of the Environment Operations Act 1997* (POEO Act). Ultimately, this requires that any water discharged into the natural environment should comply with the *Australian and New Zealand Guidelines for Fresh and Marine Water Quality*, Australian and New Zealand Environment and Conservation Council (ANZECC) and Agricultural and Resource Management Council of Australia and New Zealand, October 2000.

The above water quality guideline criteria include trigger criteria values for pH, turbidity, nutrients, dissolved oxygen and faecal coliforms (unlikely to affect excavation water). An appropriate strategy would be to carry out initial testing of groundwater samples from the developed piezometers in boreholes BH1 and BH4 to assess compliance with the ANZECC water quality guidelines. Further monitoring would also be needed during construction. If the tested water quality complies with the guidelines, it may be pumped directly into the stormwater system. Alternatively, the pumped groundwater would require on-site treatment such as sedimentation and dosing to improve the quality of water to a sufficient level to comply with the ANZECC requirements before disposal into stormwater.

7.4 Foundations

7.4.1 General

The floor of the basement excavation will be to RL 78.1 m AHD as shown on Sections A-A′ and B-B′ (Drawings 2 and 3, respectively). Basement excavation level intersects medium and high strength sandstone, and foundations may be supported on either of these strata. Suitable foundation types include spread footings such as pads or strip footings, and if sizing is unsuitable, piles.

On the basis of the conditions in the bores it is expected that piles could be constructed as uncased bored piers. The excavation of the deep basement is expected to effectively dewater most of the site down to this level. Therefore only minor perched pockets of groundwater are expected to impact on the foundation construction. Some provision for temporary casing support over the upper metre and the use of submersible pumps to dewater pile holes immediately prior to concrete placement is recommended. If groundwater seepage into a pier is such that there is more than 100 mm over the base at the time of concreting, tremie concrete placement methods should be employed to achieve a clean pile base. Pile concrete should be placed within 24 hours of drilling to avoid softening of the rock over the socket length. Pile cleaning and roughening should be carried out as per the general guidelines presented in Reference 3.

With regard to proving of high level foundations, attention is drawn to the suggested minimum requirements set out in References 1 and 2. In particular, "spoon" testing (or proof core drilling) should be undertaken in at least one-third of high level footings proportioned on the basis of an allowable bearing pressure of between 3500 kPa and 6000 kPa. For spread footings proportioned on the basis of an allowable bearing pressure of greater than 6000 kPa, "spoon" testing should be undertaken at all footing locations.

The purpose of "spoon" testing is to check that no significant weak seams exist within a depth of 1.5 times the least footing dimension below the foundation level.

7.4.2 Design

Recommended maximum allowable pressures for the range of possible foundation materials encountered in the boreholes and intersecting the proposed basement level at the site are presented in Table 7. These parameters apply to the design of spread footings, such as pads or strip footings, and for socketed bored piles.

Table 7 Recommended Foundation Design Parameters

Foundation Stratum	Classification[1]	Maximum Allowable Pressure	
		End Bearing (MPa)	Shaft Adhesion[2] (kPa)
Sandstone	IV/III	3.5	350
	I	12	1000

1. Classification based on References 1 & 2.
2. Shaft adhesion applicable to the design of bored piles, uncased over rock socket length, where adequate sidewall cleanliness and roughness is achieved.

The foundation design parameters given in the above table can be adopted on the assumption that the foundation excavations (e.g. pads or piles) are clean and free of loose debris, with pile sockets free of smear and adequately roughened immediately prior to concrete placement. An experienced geotechnical professional should inspect all pile excavations and spread footings (e.g. pads) prior to the placement of steel and concrete

Foundations proportioned on the basis of the above parameters would be expected to experience total settlements of less than 1 % of the minimum footing dimension under the applied working (i.e. serviceability) load, with differential settlements between adjacent columns expected to be less than half this value

By way of example, a 1.5 m long piled foundation of 600 mm diameter founded within Class III Sandstone should safely support a working load of up to 3000 kN, with an estimated pile settlement of less than 6 mm.

7.5 Seismic Design

In accordance with the Earthquake Loading Standard, AS 1170.4 – 1993, the site has an acceleration coefficient (a) of 0.08 and a site factor of 0.67, assuming that all major structural loads are carried to rock of at least medium strength.

7.6 Reinforced Concrete Durability

The results of pH, chloride and sulphate analyses indicate that the concentrations within the soil and groundwater analysed are non-aggressive (Table 6.1, AS 2159 – 1995).

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Reviewed by

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Principal

REFERENCES:

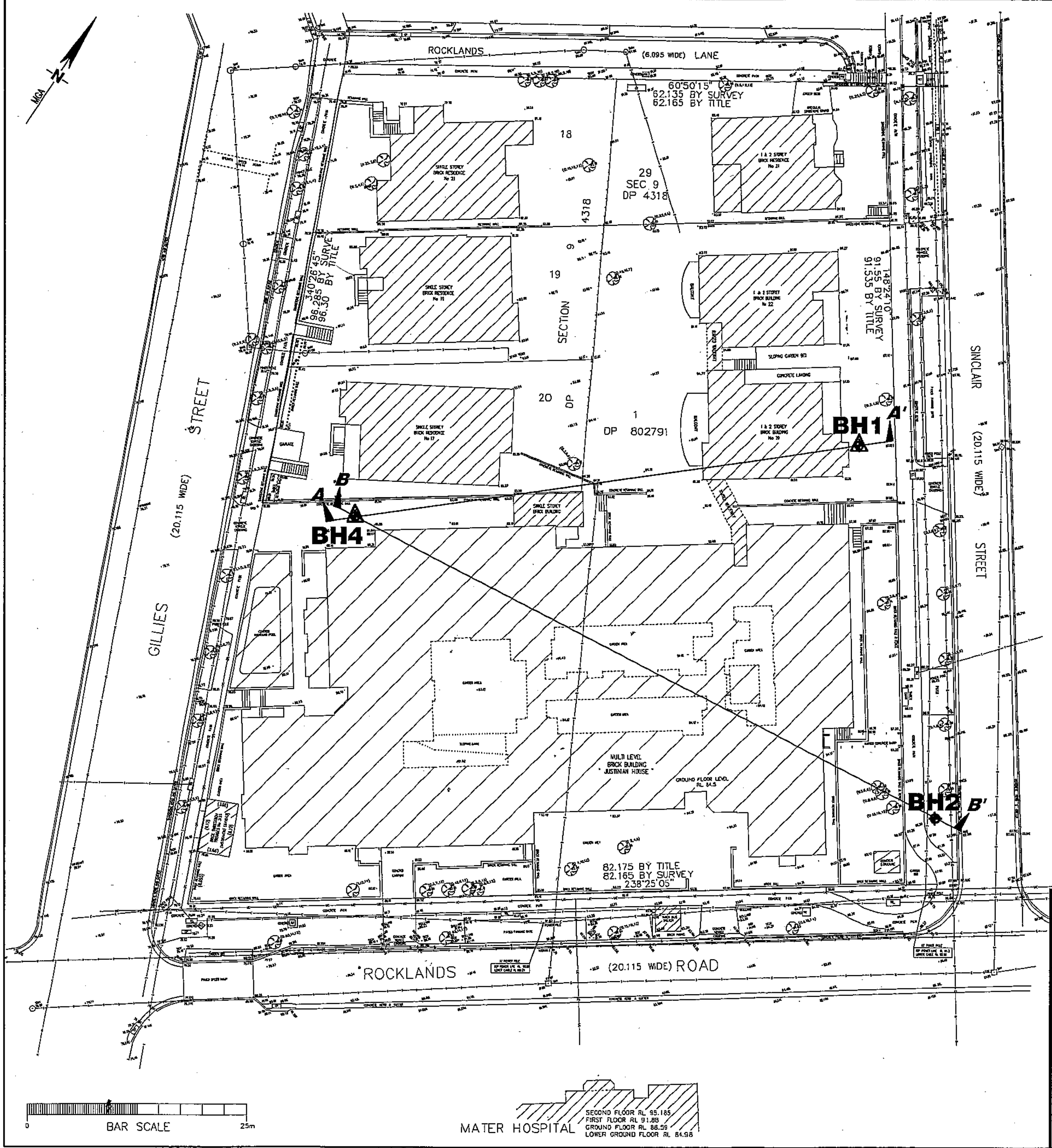
1. Pells, P.J., Mostyn, G. and Walker, B.F. "Foundations on Sandstone and Shale in the Sydney Region". Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

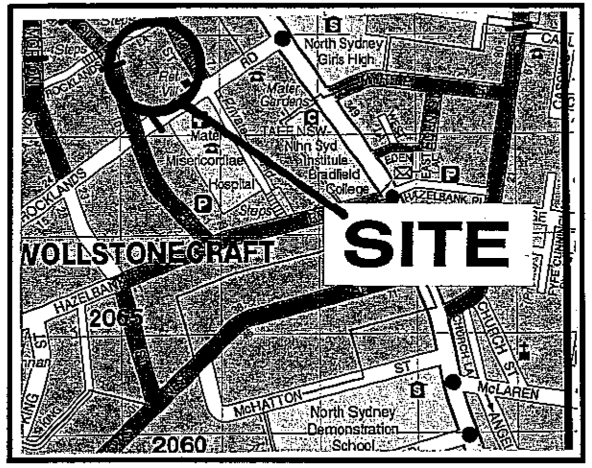
2. Pells, P.J., Douglas, D.J., B., Thorne, C. and McMahon, B.K. "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region". Australian Geomechanics Journal, Vol. 3, 1978.

3. Walker, B.F. and Pells, P.J.N. "The Construction of Bored Piles Socketed into Shale and Sandstone". Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

APPENDIX A

Drawings





LOCALITY PLAN

LEGEND

◆ TEST BORE LOCATION

▲ TEST BORE & STAND-PIPE PIEZOMETER

Douglas Partners
Geotechnics, Environment, Groundwater

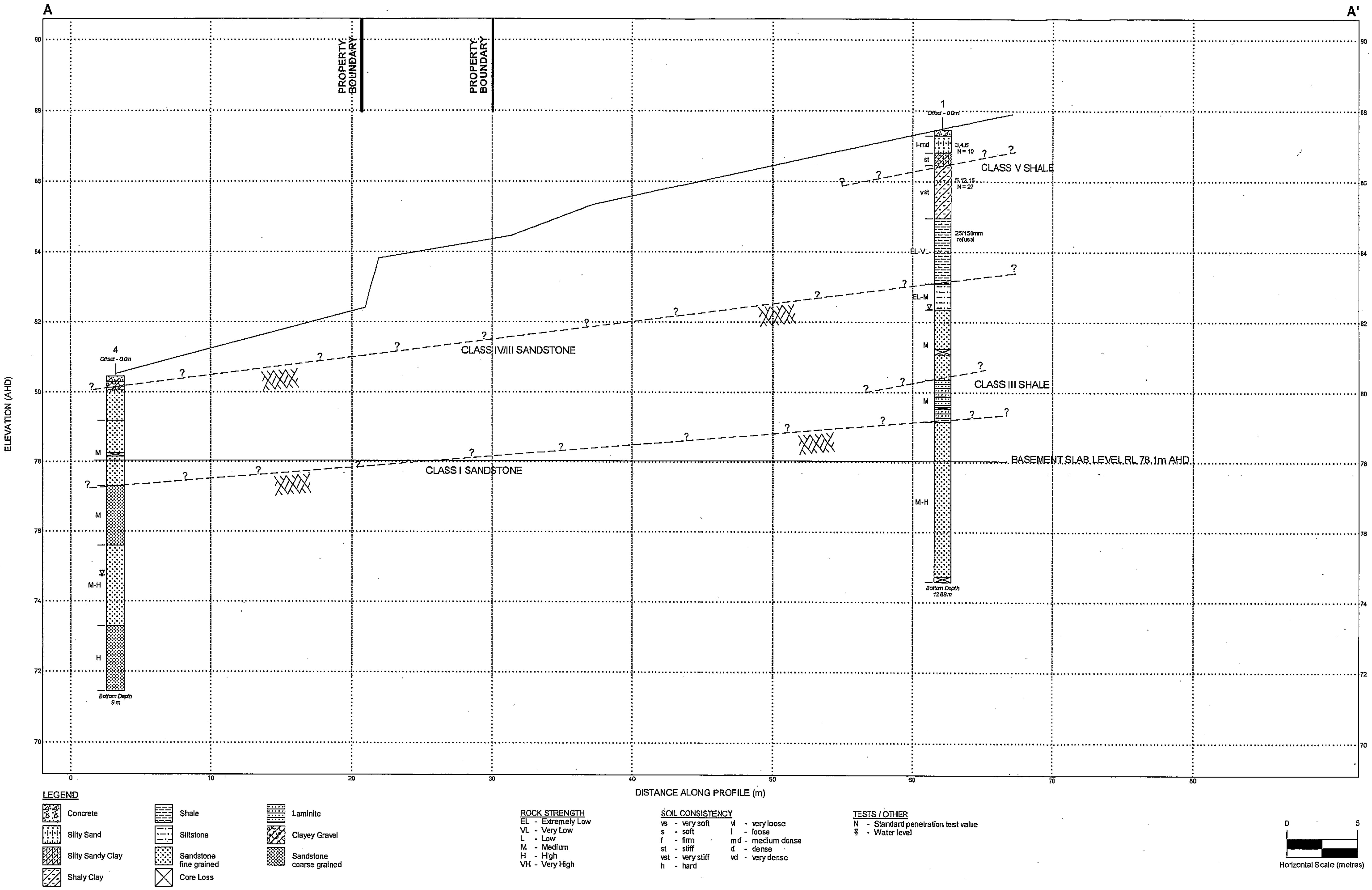
Sydney, Newcastle, Brisbane, Melbourne, Wyong, Canberra, Campbelltown, Townsville, Perth, Cairns, Wollongong, Darwin, Gold Coast, Sunshine Coast

TITLE:

Location of Exploratory Boreholes
Justinian House Redevelopment
18-22 Sinclair Street
WOLLSTONECRAFT

CLIENT: St Vincents & Mater Health			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 45046	OFFICE: SYDNEY
APPROVED BY: GSY		DATE: 29.8.2007	DRAWING No: 1

P:\45046 WOLLSTONECRAFT, Justinian House Redevelopment GSY\Drawings\45046-1.dwg, 8/29/2007 10:51:11 AM



Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: St Vincents & Mater Health

PROJECT: Justinian House Redevelopment

TITLE: Section A-A'
18-22 Sinclair Street, Wollstonecraft

APPROVED DP: GSY

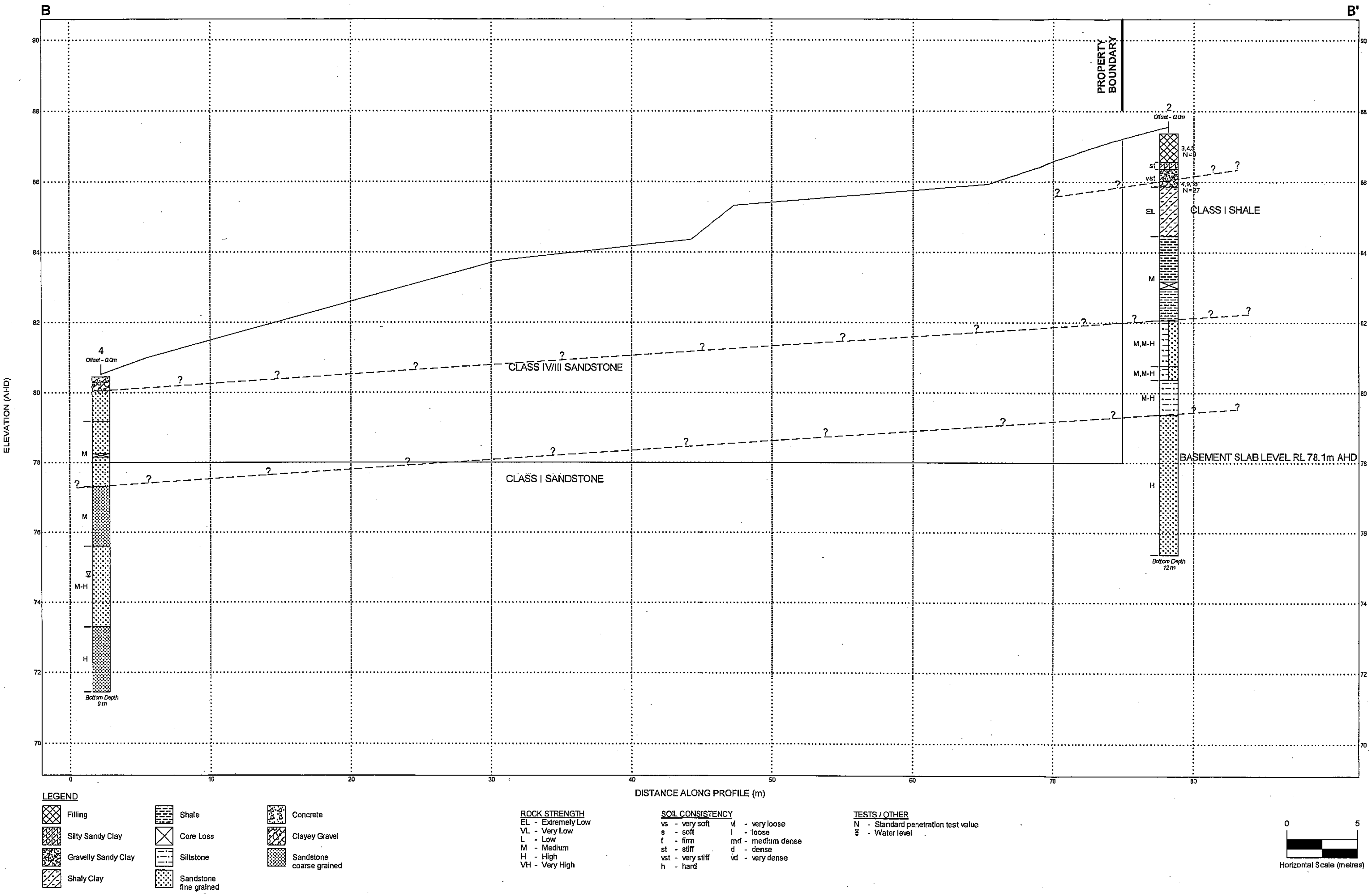
DATE: 31 August 07

REVISION:

SCALE: 1: 250 (H)

PROJECT No: 45046

DRAWING No: 2



ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

SOIL CONSISTENCY
vs - very soft
s - soft
f - firm
st - stiff
vst - very stiff
h - hard
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
▽ - Water level

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: St Vincents & Mater Health			TITLE: Section B-B' 18-22 Sinclair Street, Wollstonecraft		
PROJECT: Justinian House Redevelopment					
APPROVED DP: GSY	DATE: 31 August 07	REVISION: 0	SCALE: 1: 250 (H)	PROJECT No: 45046	DRAWING No: 3

DP A3 CROSS SECTION 45046.GPJ DOUGLAS PARTNERS 2005 CPT.GDT 31/08/07

APPENDIX B
Notes Relating to this Report
Results of Field Work

Douglas Partners
Geotechnics • Environment • Groundwater

NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT “N” Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil 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 are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is 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.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining 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 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end 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 friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical 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 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- 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 in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is 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.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, 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. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

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 condition, 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 depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

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 than 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.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

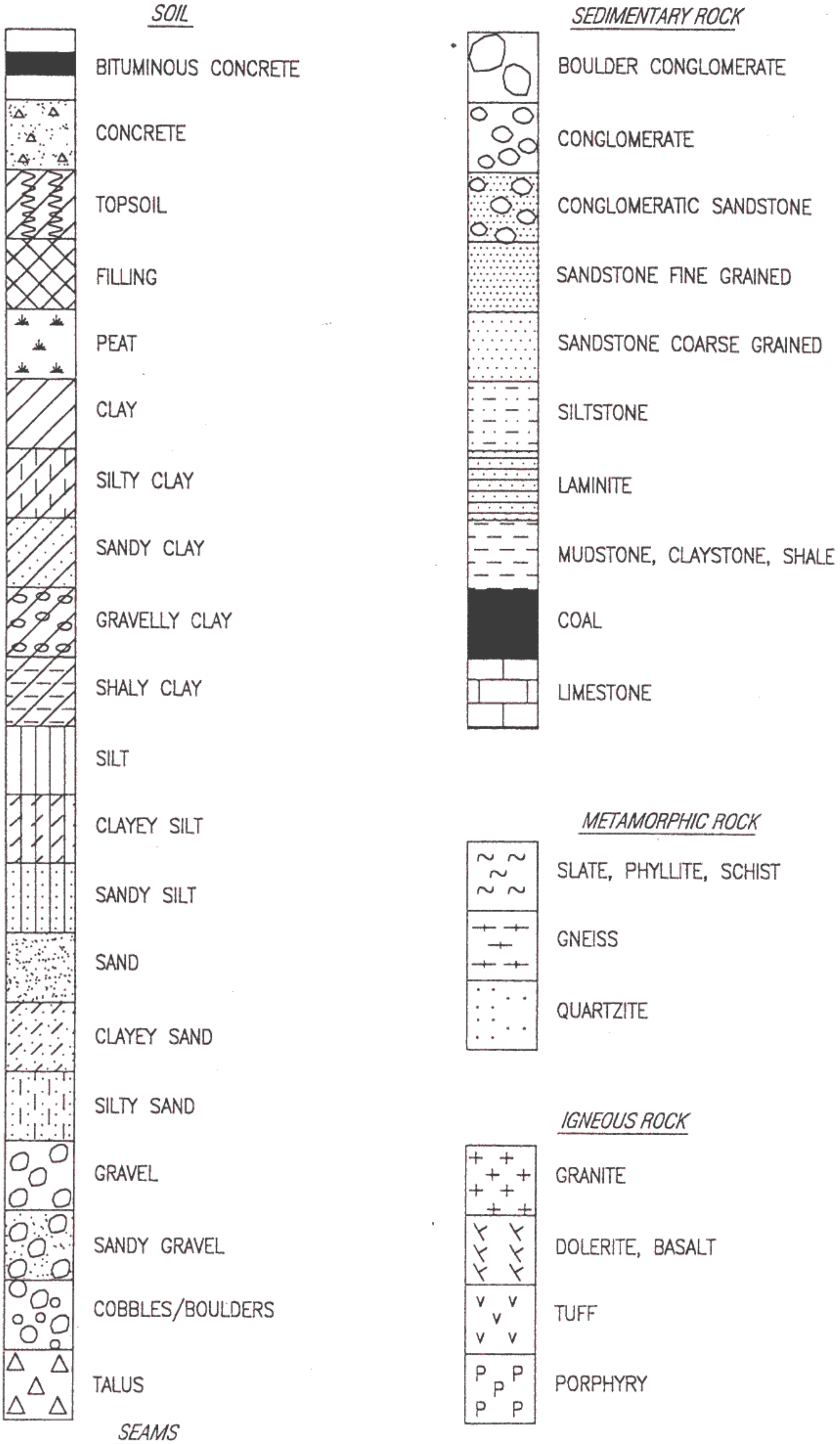
SEDIMENTARY ROCK TYPES

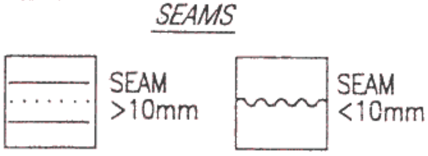
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.

GRAPHIC SYMBOLS FOR SOIL & ROCK





Douglas Partners
Geotechnics, Environment, Groundwater

DP, 45046, 13/08/07, BH1, Start 4.56m

PHOTO 1: BH1, Core Box 1 of 2

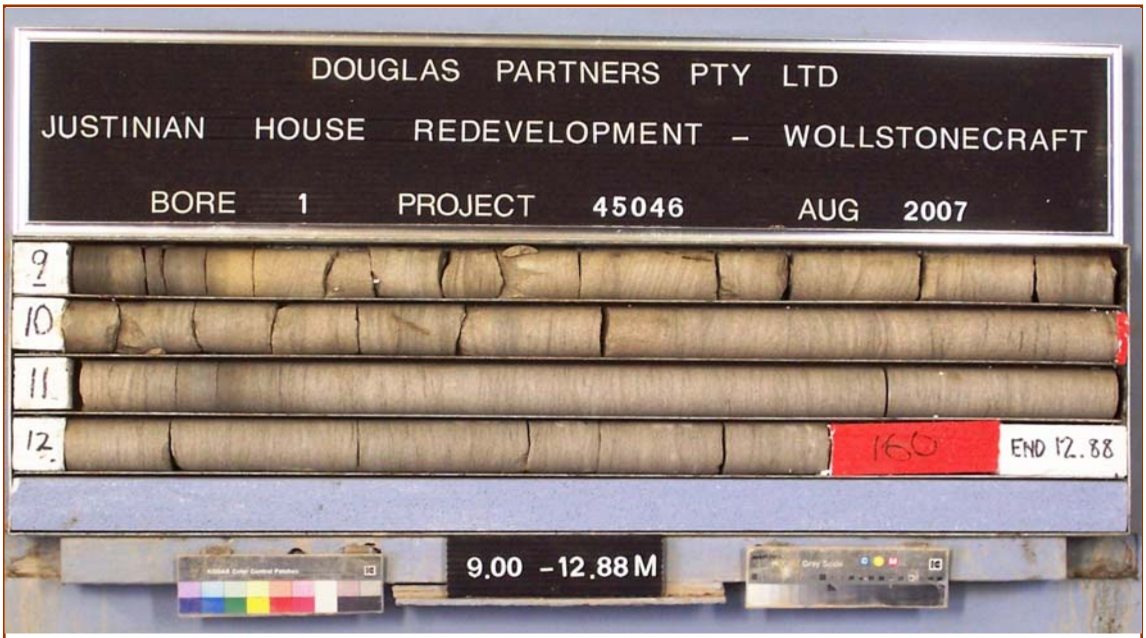


PHOTO 2: BH1, Core Box 2 of 2

TITLE	**PROJECT**	
Core Photographs for borehole BH1	45046	Douglas Partners Geotechnics • Environment • Groundwater

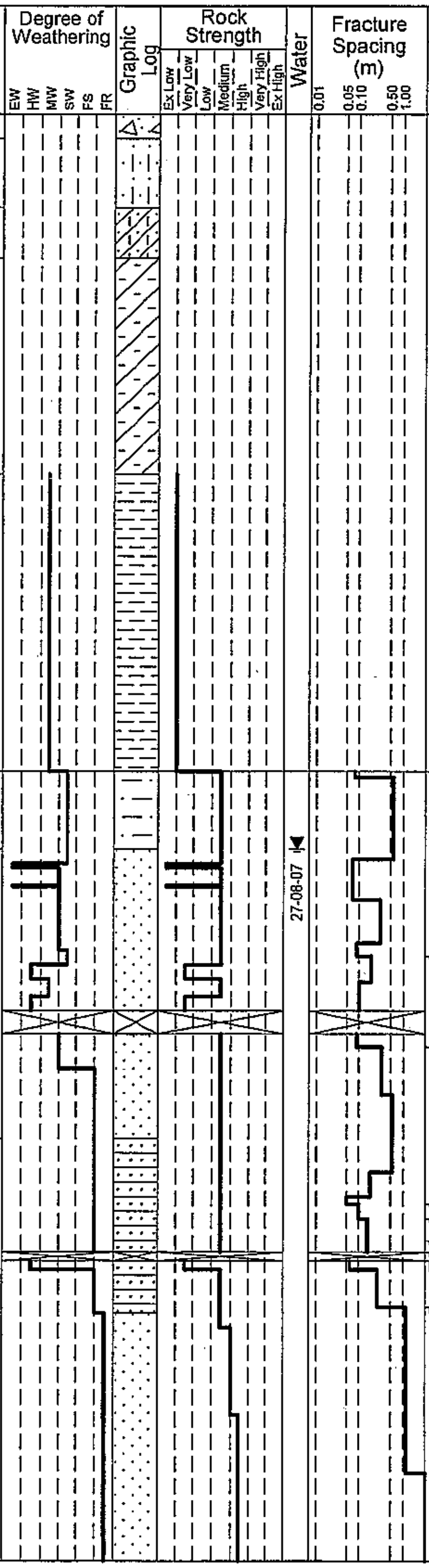
BOREHOLE LOG

CLIENT: St Vincents & Mater Health
PROJECT: Justinian House Redevelopment
LOCATION: 18-22 Sinclair Street, Wollstonecraft

SURFACE LEVEL: 87.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 45046
DATE: 13 Aug 07
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
		CONCRETE					
87	0.17	SILTY SAND - loose to medium dense, brown, fine grained silty sand, non-plastic fines, moist					
	0.65 1.0	SANDY SILTY CLAY (CI) - stiff, orange brown, medium plasticity silty sandy clay, fine sand with some fine gravel and shale fragments, moist		S			3,4,6 N = 10
86 85		SHALY CLAY (CI) - very stiff, light and dark grey mottled red-brown and orange-brown, medium plasticity shaly clay. Some grey, shale fragments of gravel size, dry to moist		S			5,12,15 N = 27
84 83	2.5	SHALE - extremely low to very low strength, moderately weathered, grey shale, dry	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°	S			25/150mm refusal
	4.56	SILTSTONE - medium strength, slightly weathered, slightly fractured, grey siltstone	4.6m: J85° ironstained				PL(A) = 0.5MPa
82 81	5.1 6.22	SANDSTONE - medium strength, moderately to slightly weathered then fresh stained, slightly fractured, grey brown, fine grained sandstone. Some siltstone laminations	5.17-5.45m: J85°- 90° rough, irregular, ironstained 5.75m: B0°- 5° ironstained 5.84m: J70° 6.22m: CORE LOSS: 160mm 6.47m: J25° ironstained 6.8m: J35° healed	C	100	63	PL(A) = 0.5MPa PL(A) = 0.5MPa PL(A) = 0.5MPa
80	7.1 7.88	LAMINITE - medium strength, fresh stained, fractured to slightly fractured, grey laminite (approximately 20% fine sandstone laminae)	7.33m: J60° ironstained 7.5m: J70° ironstained 7.55m: J45° ironstained 7.65m: J55° ironstained 7.8m: J85° ironstained 7.88m: CORE LOSS: 60mm 7.94-8.0m: crushed rock 8.26m: microfault?	C	88	73	PL(A) = 0.7MPa
79 78	8.3	SANDSTONE - medium to high then high strength, fresh, slightly fractured and unbroken, light grey, fine to medium grained sandstone	9.4m: J90°	C	100	100	PL(A) = 1MPa PL(A) = 1.3MPa



RIG: Multi drill **DRILLER:** Traccess **LOGGED:** AK/SI **CASING:** HW to 4.5m

TYPE OF BORING: Diatube to 0.17m; Solid flight auger to 4.56m; NMLC-Coring to 12.88m

WATER OBSERVATIONS: Free groundwater observed at 5.10m on 27/8/07

REMARKS: Standpipe installed to 12.72m; Screen from 9.72m to 12.72m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ≚ Water level

CHECKED
Initials: GSY
Date: 21.9.07

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: St Vincents & Mater Health
PROJECT: Justinian House Redevelopment
LOCATION: 18-22 Sinclair Street, Wollstonecraft

SURFACE LEVEL: 87.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 45046
DATE: 13 Aug 07
SHEET 2 **OF** 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
10.0	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, fine to medium grained sandstone		C	100	100	PL(A) = 1.6MPa
			C	91	91	PL(A) = 1.3MPa PL(A) = 1.2MPa
12.72		12.72m: CORE LOSS: 160mm - left down hole				
12.88	Bore discontinued at 12.88m					

RIG: Multi drill **DRILLER:** Traccess **LOGGED:** AK/SI **CASING:** HW to 4.5m

TYPE OF BORING: Diatube to 0.17m; Solid flight auger to 4.56m; NMLC-Coring to 12.88m

WATER OBSERVATIONS: Free groundwater observed at 5.10m on 27/8/07

REMARKS: Standpipe installed to 12.72m; Screen from 9.72m to 12.72m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▽ Water level

CHECKED
Initials: GSY
Date: 21·9·07

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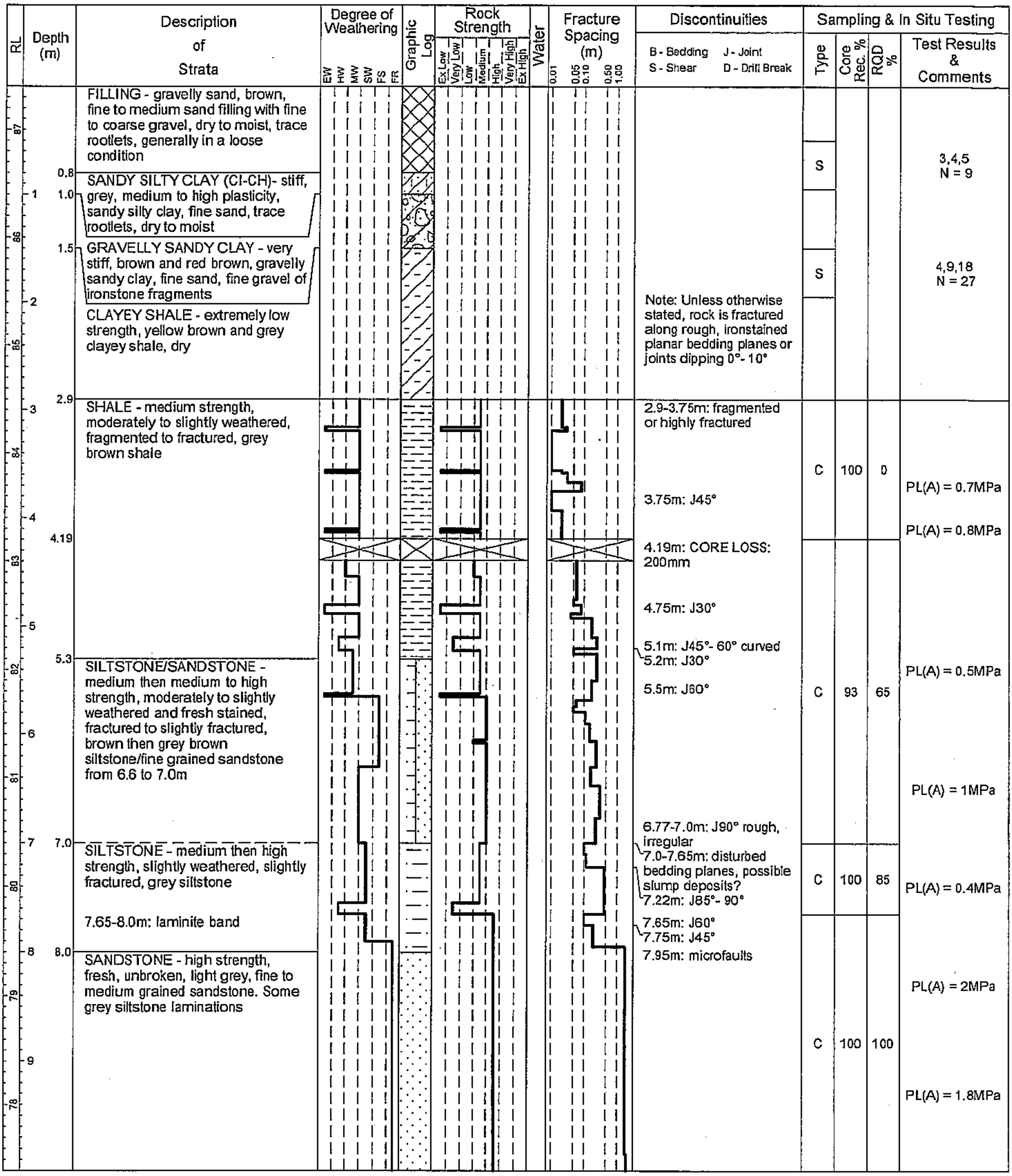
BOREHOLE LOG

CLIENT: St Vincents & Mater Health
PROJECT: Justinian House Redevelopment
LOCATION: 18-22 Sinclair Street, Wollstonecraft

SURFACE LEVEL: 87.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 45046
DATE: 13 Aug 07
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
	FILLING - gravelly sand, brown, fine to medium sand filling with fine to coarse gravel, dry to moist, trace rootlets, generally in a loose condition		S			3,4,5 N = 9
0.8 1.0	SANDY SILTY CLAY (CI-CH)- stiff, grey, medium to high plasticity, sandy silty clay, fine sand, trace rootlets, dry to moist					
1.5	GRAVELLY SANDY CLAY - very stiff, brown and red brown, gravelly sandy clay, fine sand, fine gravel of ironstone fragments		S			4,9,18 N = 27
	CLAYEY SHALE - extremely low strength, yellow brown and grey clayey shale, dry	Note: Unless otherwise stated, rock is fractured along rough, ironstained planar bedding planes or joints dipping 0°- 10°				
2.9	SHALE - medium strength, moderately to slightly weathered, fragmented to fractured, grey brown shale	2.9-3.75m: fragmented or highly fractured 3.75m: J45°	C	100	0	PL(A) = 0.7MPa PL(A) = 0.8MPa
4.19		4.19m: CORE LOSS: 200mm 4.75m: J30° 5.1m: J45°- 60° curved 5.2m: J30°	C	93	65	
5.3	SILTSTONE/SANDSTONE - medium then medium to high strength, moderately to slightly weathered and fresh stained, fractured to slightly fractured, brown then grey brown siltstone/fine grained sandstone from 6.6 to 7.0m	5.5m: J60° 6.77-7.0m: J90° rough, irregular				PL(A) = 0.5MPa PL(A) = 1MPa
7.0	SILTSTONE - medium then high strength, slightly weathered, slightly fractured, grey siltstone 7.65-8.0m: laminite band	7.0-7.65m: disturbed bedding planes, possible slump deposits? 7.22m: J85°- 90° 7.65m: J60° 7.75m: J45° 7.95m: microfaults	C	100	85	PL(A) = 0.4MPa
8.0	SANDSTONE - high strength, fresh, unbroken, light grey, fine to medium grained sandstone. Some grey siltstone laminations		C	100	100	PL(A) = 2MPa PL(A) = 1.8MPa

RIG: Multi drill **DRILLER:** Traccess **LOGGED:** AK/SI **CASING:** HW to 2.4m

TYPE OF BORING: Solid flight auger to 2.4m; Rotary to 2.9m; NMLC-Coring to 12.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: GSY
Date: 21-9-07

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PHOTO 3: BH2, Core Box 1of 2

PHOTO 4: BH2, Box 2 of 2

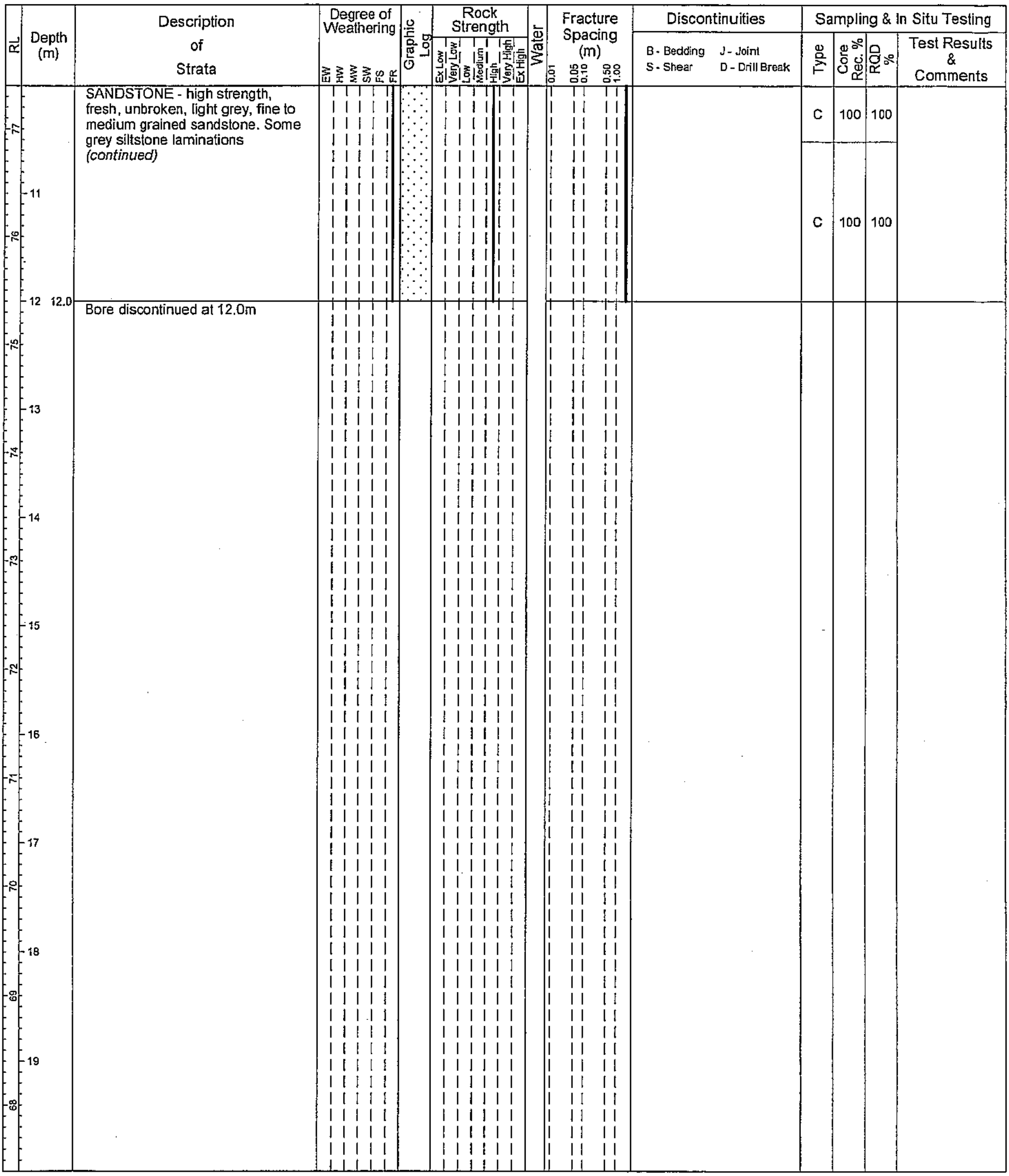
TITLE Core Photographs for Borehole BH2	**PROJECT** 45046	Douglas Partners Geotechnics • Environment • Groundwater

BOREHOLE LOG

CLIENT: St Vincents & Mater Health
PROJECT: Justinian House Redevelopment
LOCATION: 18-22 Sinclair Street, Wollstonecraft

SURFACE LEVEL: 87.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 45046
DATE: 13 Aug 07
SHEET 2 **OF** 2



Depth (m)	Description of Strata	Type	Core Rec. %	RQD %	Test Results & Comments
	SANDSTONE - high strength, fresh, unbroken, light grey, fine to medium grained sandstone. Some grey siltstone laminations *(continued)*	C	100	100	
		C	100	100	
12.0	Bore discontinued at 12.0m				

RIG: Multi drill **DRILLER:** Traccess **LOGGED:** AK/SI **CASING:** HW to 2.4m

TYPE OF BORING: Solid flight auger to 2.4m; Rotary to 2.9m; NMLC-Coring to 12.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ≡ Water level

CHECKED
Initials: GSY
Date: 21.9.07

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PHOTO 5: BH4, Box 1of 2

PHOTO 6: BH4, Box 2of 2

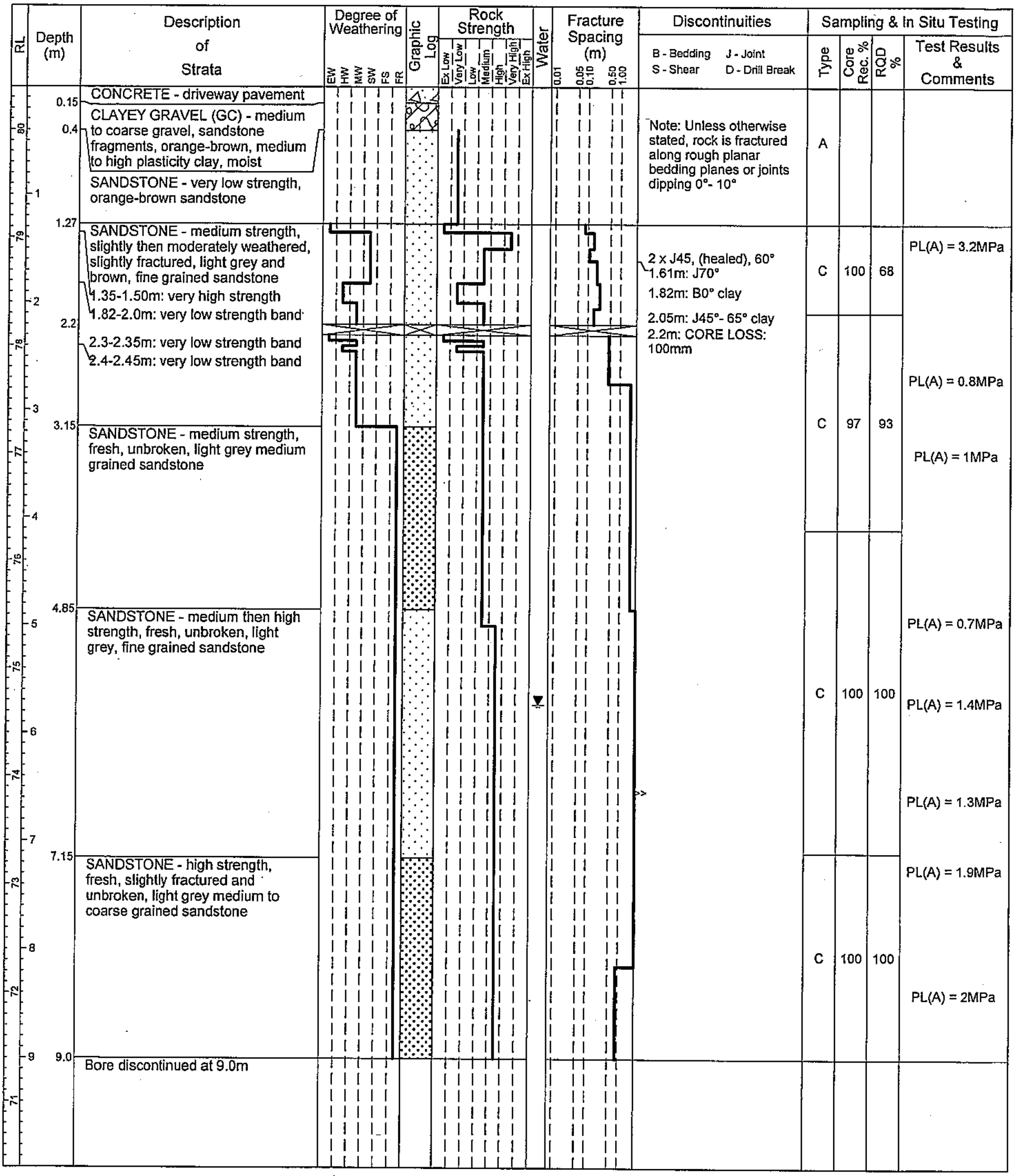
TITLE Core photographs for borehole BH4	**PROJECT** 45046	**Douglas Partners** Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: St Vincents & Mater Health
PROJECT: Justinian House Redevelopment
LOCATION: 18-22 Sinclair Street, Wollstonecraft

SURFACE LEVEL: 80.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 45046
DATE: 15 Aug 07
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
	CONCRETE - driveway pavement					
0.15	CLAYEY GRAVEL (GC) - medium to coarse gravel, sandstone fragments, orange-brown, medium to high plasticity clay, moist	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°	A			
0.4	SANDSTONE - very low strength, orange-brown sandstone					
1.27	SANDSTONE - medium strength, slightly then moderately weathered, slightly fractured, light grey and brown, fine grained sandstone 1.35-1.50m: very high strength 1.82-2.0m: very low strength band	2 x J45, (healed), 60° 1.61m: J70° 1.82m: B0° clay 2.05m: J45°- 65° clay 2.2m: CORE LOSS: 100mm	C	100	68	PL(A) = 3.2MPa
2.2	2.3-2.35m: very low strength band 2.4-2.45m: very low strength band		C	97	93	PL(A) = 0.8MPa
3.15	SANDSTONE - medium strength, fresh, unbroken, light grey medium grained sandstone					PL(A) = 1MPa
4.85	SANDSTONE - medium then high strength, fresh, unbroken, light grey, fine grained sandstone		C	100	100	PL(A) = 0.7MPa PL(A) = 1.4MPa PL(A) = 1.3MPa
7.15	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey medium to coarse grained sandstone		C	100	100	PL(A) = 1.9MPa PL(A) = 2MPa
9.0	Bore discontinued at 9.0m					

RIG: Multi rig **DRILLER:** Traccess **LOGGED:** AK/SI **CASING:** HW to 1.27m
TYPE OF BORING: Diatube to 0.15m; Solid flight auger to 1.27m; NMLC-Coring to 9.0m
WATER OBSERVATIONS: Free groundwater observed at 5.73m depth
REMARKS: Standpipe installed to 9.0m; Screened between 6.0m and 9.0m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: GSY
Date: 21.9.07

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APPENDIX C

Results of Laboratory Testing

Douglas Partners
Geotechnics · Environment · Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117

PO Box 472
West Ryde NSW 1685
Australia

96 Hermitage Road
West Ryde NSW 2114

***Phone** (02) 9809 0666*
Fax: (02) 9809 4095
sydney@douglaspartners.com.au

CHEMICAL TEST RESULTS

CLIENT: **ST VINCENTS AND MATER HEALTH**
LEVEL 7, AMP CENTRE, 50 BRIDGE STREET
SYDNEY NSW 2060

PROJECT NO: 45046

DATE: 04/09/07

DATE OF TESTING: 31/08/07

PROJECT: JUSTINIAN HOUSE REDEVELOPMENT
LOCATION: 18-22 SINCLAIR STREET, WOLLSTONECRAFT

PAGE: 1 of 1

	Units	13360-1 BH1/5.10 water	13360-2 BH4/5.73 water
pH	pH Units	6.2	5.5
Chloride Cl	mg/kg	83	70
Soluble Sulphate, as SO4	mg/kg	78	99

<u>Envirolab Services Pty Ltd:</u>
Report 13360

Laboratory: SYDNEY

N Weimann

Signed: Norman Weimann
Manager. Earthworks / Laboratory Testing Services

Envirolab Services Pty Ltd
ABN 37 112 535 645
54 Frenchs Rd Willoughby NSW 2068
ph 02 9958 5801 fax 02 9958 5803
email: tnotaras@envirolabservices.com.au

CERTIFICATE OF ANALYSIS 13360

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Penelope Ford

Sample log in details:

Your Reference:	**45046, 18-22 Sinclairs St Wollstonecraft**
No. of samples:	2 Waters
Date samples received:	28/08/07
Date completed instructions received:	28/08/07

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

Report Details:

Date results requested by:	4/09/07
Date of Preliminary Report:	Not Issued
Issue Date:	31/08/07

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

Results Approved By:

David Springer
Business Development & Quality Manager

Miscellaneous Inorganics Our Reference:	UNITS	13360-1	13360-2
Your Reference	-------------	BH1/5.10	BH4/5.73
Date Sampled	------------	27/08/07	27/08/07
Type of sample		Water	Water
pH	pH Units	6.2	5.5
Sulphate, SO4	mg/L	78	99
Chloride (titration) - water	mg/L	83	70

Method ID	Methodology Summary
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
LAB.9	Sulphate determined turbidimetrically.
LAB.11	Chloride determined by argentometric titration.

QUALITY CONTROL Miscellaneous Inorganics	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base || Duplicate || %RPD	Spike Sm#	Spike % Recovery
pH	pH Units	0.1	LAB.1	<0.10	13360-1	6.2 || 6.2 || RPD: 0	LCS-1	101%
Sulphate, SO4	mg/L	5	LAB.9	<5.0	13360-1	78 || [N/T]	LCS-1	106%
Chloride (titration) - water	mg/L	20	LAB.11	<20	13360-1	83 || [N/T]	LCS-1	104%

Report Comments:

Asbestos analysed by: Not applicable for this job

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

Quality Control Definitions

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

Laboratory Acceptance Criteria:

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

Douglas Partners
Geotechnics · Environment · Groundwater

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Australia

96 Hermitage Road
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Phone (02) 9809 0666
Fax: (02) 9809 4095
sydney@douglaspartners.com.au

CHEMICAL TEST RESULTS

CLIENT: **ST VINCENTS AND MATER HEALTH**
LEVEL 7, AMP CENTRE, 50 BRIDGE STREET
SYDNEY NSW 2060

PROJECT NO: 45046

DATE: 22/08/07

DATE OF TESTING: 15/08/07

PROJECT: JUSTINIAN HOUSE REDEVELOPMENT
LOCATION: 18-22 SINCLAIR STREET, WOLLSTONECRAFT

PAGE: 1 of 1

	Units	13132-1 BH4/0.2 soil
pH	pH Units	8.2
Chloride Cl	mg/kg	<100
Soluble Sulphate, as SO4	mg/kg	<25

Envirolab Services Pty Ltd:
Report 13132

Laboratory: SYDNEY

NWeimann

Signed: Norman Weimann
Manager. Earthworks / Laboratory Testing Services