Douglas Partners
Geotechnics • Environment • Groundwater

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REPORT
on
GEOTECHNICAL INVESTIGATION

GLENMORE PARK TOWN CENTRE
GLENMORE PARKWAY, GLENMORE PARK

Prepared for
AMP CAPITAL INVESTORS LIMITED

Project 71169
June 2009

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1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Douglas Partners Pty Ltd (DP) for proposed augmentation of an existing shopping centre development at Glenmore Park Town Centre. The work was carried out in general accordance with the DP proposal dated 2 April 2009 to AMP Capital Investors Limited, who commissioned the work on 8 May 2009.

A major expansion of the existing shopping centre, including additional 1 – 2 storey superstructures over a two level basement carpark and a separate 1 – 2 storey building are proposed for the site. The geotechnical investigation was carried out to provide information for planning and design purposes.

The purpose of the investigation was to provide information on:

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

The investigation comprised drilling of exploratory boreholes and in-situ strength 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 relevant geotechnical design and construction practice.

2. SITE DESCRIPTION AND GEOLOGY

The site is divided into two parts: the main part of the site refers to the proposed basement footprint, and the minor part of the site to a proposed detached building.

The main part of the site comprises an irregular-shaped area of about 1.5 ha, and is bounded to the north, south, east and west by existing public roads. Two single-storey, concrete-clad commercial buildings are located near the western and eastern site boundary, respectively, of the main part of the site. Site levels over the main part of the site fall generally in the north easterly direction with an approximate overall grade of 5 ° and an overall difference in level of about 10 m.

The minor part of the site comprises an irregular shaped area of about 700 m^2. It is bounded by public roads to the north, east and south, and the retained walkway of an existing supermarket building to the west. Site levels over the minor part of the site fall generally in the easterly direction with an approximate grade of 5 ° and an overall difference in level of about 1.5 - 2 m.

The existing ground surface over the northern half of the main part of the site was undulated and sparsely vegetated with grasses, weeds, and three copses of 15 – 20 m tall native trees. Bare ground was exposed regularly over the northern half of the main part of the site, revealing gravelly silty sand filling with some cobbles of silty sandstone fragments. An asphalt concrete-surfaced carpark, with a uniform and gentle slope, is located over the southern half of the main part of the site.

The existing ground surface over the minor part of the site has a terraced profile falling towards the east, with a 2 – 2.5 m high, mortared, sandstone block retaining wall supporting an existing walkway and outdoor eating area adjacent to a supermarket building on the western side. On the eastern side, a low retaining wall of similar construction supported an area entirely paved with masonry pavers, except for a circular, raised garden bed which occupied part of its eastern side, which also supported a single tree several metres in height. Beyond the toe of the lower retaining wall, the ground surface was covered with lawn.

Mulgoa Creek flows in a northerly direction 1.3 km west of the site towards the Nepean River.

The Geological Survey of NSW 1:100,000 Geological Series Sheet 9030 (Penrith) indicates that the site is underlain by Ashfield Shale. Ashfield Shale typically comprises black to dark grey shales and laminites. No major geological structures, such as dykes or faults, are indicated by this map for the site and nearby surrounds.

The corresponding Soil Conservation Service of NSW Soil Landscape Series Sheet indicates that the site is situated in an area of gently undulating rises on Wianamatta Group shales and that bedrock is overlain by shallow to moderately deep (<1 m), red and brown podzolic soils on crests, upper slopes and well drained areas. On lower slopes and in areas of poor drainage, the map indicates that bedrock is overlain by deep (1.5 – 3.0 m), yellow podzolic soils and soloths.

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

3. FIELD WORK

3.1 Methods

The field investigation comprised:

- seven boreholes (BH1 – BH7) drilled with a truck-mounted drilling rig to a maximum depth of 10.0 m. The boreholes were initially drilled to depths of 1.2 – 4.5 m with 110 mm diameter solid flight augers and rotary drilling techniques, and thereafter, four of the boreholes were advanced through rock to depths of 5.5 – 10.0 m using diamond coring techniques to obtain NMLC-sized (51 mm diameter) rock cores.

- Standard penetration tests (SPTs) carried out at 1.0 – 1.5 m depth intervals over the soil profile, with disturbed sampling of soils taken directly from the auger tip and the SPT split-spoon sampler.
- Installation of a stand-pipe piezometer in borehole BH5.
- The purging of the piezometer, groundwater level measurement and groundwater sampling taken after completion of the drilling program.

The boreholes were set out using tape measurement from existing surface features (e.g. fences and roads), and these locations are shown in Drawing 1 of Appendix A. Ground elevations at borehole locations were obtained by levelling from temporary benchmarks.

The boreholes were logged by an experienced engineering geologist. They were then backfilled with excavated spoil on completion, except where the piezometer was installed at borehole BH5.

3.2 Results

Detailed descriptions of the materials encountered at each borehole location are given on the engineering logs presented in Appendix B, together with notes defining classification methods and descriptive terms. Drawings 2 and 3 show geological sections through the site.

3.2.1 Sub-surface Profile

3.2.1.1 Main Part of the Site

In general, the boreholes drilled in the main part of the site (BH1 – BH6) indicate that it is underlain by a variable depth of filling overlying natural silty clay that is underlain by interbedded siltstone, sandstone, shale and laminite. The sub-surface conditions encountered in the boreholes are summarised as follows:

- **ASPHALTIC CONCRETE**: carpark pavement surface 0.05 – 0.1 m thick over the southern half of the main part of the site.
- **FILLING**: of varying composition across the main part of the site, the base and sub-base courses of the carpark pavement comprise of sandy clay, gravelly sand and silty clay filling layers to depths of 0.7 – 1.4 m. Over the remainder of the main part of the site, the filling

comprised of apparently engineered, moderately compacted, silty clay filling with minor proportions of gravels to depths of 1.0 – 2.3 m. The filling was overlying;

- **SILTY CLAY**: Typically very stiff and 0.2 – 1.7 m thick. The silty clay was overlying;
- **BEDROCK**: The bedrock generally comprises interbedded siltstone, sandstone, shale and laminite. Over the up-slope, south western half of the proposed basement footprint, thin beds of extremely low, very low and low strength siltstone and shale that are highly weathered overlie thick beds of mainly high and very high strength siltstones and sandstones that are moderately and slightly weathered grading to fresh with increasing depth, and slightly fractured and unbroken. Thick interbeds of high strength, fresh and slightly fractured laminite were encountered below RL 43 AHD. Over the down-slope, north eastern half of the proposed basement footprint, thin beds of very low strength and highly weathered sandstone and siltstone overlie generally very low to low strength, highly weathered and fractured, interbedded siltstone and sandstone.

3.2.1.2 Minor Part of the Site

Borehole BH7 indicates that the minor part of the site is underlain by filling, natural silty clay and overlying silty sandy clay. The sub-surface conditions encountered in the boreholes are summarised as follows:

- **PAVERS**: 50 m thick, masonry pavement surfacing. The pavers were overlying;
- **FILLING**: a 100 mm thick gravelly sand filling underlay to the pavers was overlying a 1.5 m thick layer of silty clay filling, with a minor proportion of gravel. The filling was overlying;
- **SILTY CLAY**: very stiff in consistency and 1.7 m thick. The silty clay was overlying;
- **SILTY SANDY CLAY**: hard in consistency.

3.2.2 Groundwater

Free groundwater was initially measured on 18 May 2009 at a reduced level (RL) of 40.9 m AHD (3.1 m depth) during a return visit to the site to collect a groundwater sample from the earlier installed piezometer at BH5 after the well was developed. During a later site visit on 9 June 2009, groundwater was measured at RL 38.8 m AHD (5.1 m depth). Comparison with the borehole records indicates that the phreatic surface intersected very low to low strength sandstone on the initial date of measurement, and had later fallen to a level that intersected low strength siltstone.

4. LABORATORY TESTING

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

- Four (4) moisture contents of soil samples;
- Two (2) Atterberg Limits of soil samples;
- Two (2) chemical analyses of aggressivity (Sulphate + Chloride + pH) of soil samples;
- One (1) chemical analysis of aggressivity (Sulphate + Chloride + pH) of a groundwater sample; and
- Twenty four (24) Point Load Index tests of rock core samples.

The results of laboratory testing for Atterberg Limits and Corrosion Assessment, reported in Appendix C, are summarised in Table 1.

Table 1 - Summary of Chemical Laboratory Test Results

Borehole	**Depth (m)**	**Sample type**	**Soluble Sulphate as SO_4(mg/kg)**	**Choride (mg/kg)**	**pH units**
BH2	2.5 – 2.95	Soil	<25	<100	5.2
BH5	2.5 – 2.95	Soil	140	940	5.7
BH5	3.0	Groundwater	≤16	440 - 540	7.5 – 7.7

Table 2 – Summary of Engineering Laboratory Test Results

Borehole	Depth (m)	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
BH3	1.0 – 1.45	19.6	-	-	-
BH4	1.0 – 1.45	15.6	42	18	24
BH4	2.5 – 2.95	9.02	-	-	-
BH7	2.5 – 2.95	15.6	42	19	23

The results of Point Load Index testing of rock core samples are reported on the borehole logs contained in Appendix B, at the relevant depths.

5. PROPOSED DEVELOPMENT

A major expansion of the existing shopping centre, including 1 – 2 storey buildings over a two level basement carpark and a separate 1 – 2 storey building are all proposed for the site. The finished level of the Shopping Centre slab is proposed at RL 49.5 AHD, that of Basement Level 1 at RL 46.25 AHD and the Lower Basement Level slab is proposed at RL 43.0 AHD. These levels are based on the concept design sketches by DesignInc, dated 23 May 2009, and emailed advice from the client dated 2 April 2009.

Column loadings were not available at the time of this report.

6. COMMENTS

6.1 Excavations

6.1.1 Excavation Methods

Depths of excavation will vary across the basement footprint due to variations in existing surface levels. The proposed Lower Basement slab level is approximately from 0 to 7 m below the present ground surface level at the site. As shown in Drawings 2 and 3 of Appendix A, excavations in the up-slope, south western half the proposed basement footprint are expected to be carried out in moderately well compacted sand and clay filling, very stiff silty clay, extremely low to medium and mainly high and very high strength shales, siltstones, sandstones and laminites. Excavations in the down-slope, north eastern half of the basement are expected to be carried out through moderately well compacted filling, very stiff silty clay, and very low strength sandstones and shales.

After breaking out any existing pavements, the filling and silty clay soils should readily be excavated using conventional earthmoving equipment such as dozers and hydraulic excavators. Extremely low to low strength rock should be excavatable by light to medium ripping using a Caterpillar D6 dozer or equivalent and a hydraulic excavator fitted with rock hammer attachment. Medium strength rock should be excavatable by heavy ripping using a Caterpillar D9 dozer or equivalent and a hydraulic excavator fitted with rock hammer attachment. High and very high strength siltstones, sandstones and laminites are likely to require heavy ripping using a minimum Caterpillar D10 track'dozer or equivalent, in conjunction with hydraulic excavators fitted with large rock hammers (eg. Krupp 900 kg) or milling heads. The low degree of fracturing of the high and very high strength rock will make excavation particularly difficult, with low productivity and high tyne/hammer wear expected.

Excavation for footings and trenches in the south western half of the proposed basement footprint will also require the use of hydraulic excavators fitted with large rock breakers. At the proposed basement slab level over the south western half of the proposed basement footprint, high and very high strength sandstone is intersected and will be particularly difficult based on the relatively unbroken nature of the rock core samples and may require the use of a rotary rock saw or milling head.

Prospective excavation contractors should be required to inspect the rock core obtained during the investigation, to make their own assessment of the feasibility of ripping the high strength rock using their machines.

6.1.2 Disposal of Excavated Materials

The materials that will be derived from the excavation works will generally include significant amounts of filling, natural soil and rock from within the proposed bulk excavation 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 the *"Protection of the Environment Operations Act,* *1997"* prior to disposal at an appropriately licenced landfill. Further, the burden of proof remains with the owner and transporter of the spoil materials. Waste classification assessment did not form part of the present scope of work.

6.1.3 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. The neighbouring commercial buildings are probably founded on residual soils and engineered filling rather than the underlying bedrock surface, and it is therefore suggested that PPV be generally limited to 5 – 8 mm/s at the building line.

It is noted that vibration levels above 5 mm/s may be disturbing to the adjacent property occupants 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 buildings or structures in question.

If vibrations are a potential problem, consideration could be given to rock sawing or rock milling methods of rock excavation. It is possible that this will be required along the eastern and southern boundaries of the proposed basement footprints.

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 are particularly prone to the detrimental effects of settlement and vibration. Vibration monitoring should also be considered to manage site work and provide a level of reassurance to adjacent property owners.

6.1.4 Slope Stability to Open Excavations

The proposed basement excavation footprint, is close to the site footprint. Battered excavation slopes are therefore considered to be unsuitable for the proposed works as insufficient space is available. Recommended maximum temporary batter slopes for the sub-surface materials present are given in Table 3, however, for completeness.

Table 3 – Temporary Batter Slope Ratios

Material	Temporary Batter Slope Ratio[1] (H:V)
Filling	1.5:1
Silty Clay	1.5:1
Extremely low strength rock	1:1
Very low strength rock	0.75:1[2]
Low strength fractured rock	0.5:1[2]
Medium strength or better, slightly fractured rock	Vertical[2]

1. *For cut heights no greater than 4 m.*
2. *Subject to inspection by an experienced geotechnical engineer or engineering* geologist.

6.2 Excavation Support

6.2.1 General

Retaining structures will be required to support the basement 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, enabling temporary anchors to be released (i.e. untensioned). Shoring support methods and possibly underpinning systems will generally require tie-back anchors for stability, particularly where ground movements behind the wall must be limited. 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 should be considered for the support of the basement 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 shotcrete 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. Around the perimeter of the proposed basement in the vicinity of the existing carpark, the upper 0.7 – 1.4 m of the ground profile consists of moderately well compacted, sandy clay, gravelly sand and silty clay filling layers. The type of excavation support will need to be varied such that the top 1.5 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 and around the perimeter of the remainder of the main part of the site, the exposed very stiff silty clay 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. However, a maximum depth increment of 1.0 m to 1.5 m is recommended for excavation over the upper 1.5 m of the profile around the perimeter of the existing carpark, given the presence of some granular filling. It is possible that adverse jointing may cause localised instability in the exposed rock (e.g. unstable wedges) which may require remedial measures prior to shotcreting. It is therefore recommended 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 on stabilisation measures (e.g. rock bolting).

Given the ground conditions revealed by this investigation, it is considered prudent for soldier piles to be socketed at least one pile diameter into high strength sandstone, due to the presence of some moderate angle jointing in the upper extremely low up to medium strength material, which could lead to possible wedge failures due to sliding along joint planes. Anchoring at regular intervals down the cut rock face (through the soldier piles socketed into the high strength sandstone and siltstone should provide an effective means of transferring anchor loads to the face of the excavation, thereby stabilising the cut face.

Soldier piles founded below basement level may be designed on the basis of the allowable foundation pressure given in Section 6.4, 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.

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

The designer and developer should note that some small degree of wall movement is unavoidable for conventional anchored pile wall systems. Further, the effects of stress relief, as described in the following, may cause slight lateral movement within the underlying rock. It is therefore recommended that provision is made for some minor remedial works (eg. repair of cracking) to the adjacent pavements.

6.2.2 Design

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 mobilised wedge loading.

6.2.2.1 Earth Pressures

Excavations braced/anchored either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of the high strength rock (refer to Drawings 2 and 3). Table 4 provides 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	
Filling	0.25	0.3	20
Silty Clay	0.25	0.3	20
Extremely low strength rock	0.2	0.25	22
Very low strength rock	0.2	0.25	22
Low strength fractured rock	0.2	0.25	22
Medium strength or better, slightly fractured rock	0	0	24

Due to the proposed maximum depth of excavation of up to approximately 7 m, it will be necessary to install several rows of temporary anchors to support the retaining wall system.

Preliminary design for lateral earth pressures for a multi-anchored wall system may be based on a uniform rectangular earth pressure distribution of 4H units (kPa), where H is the depth to the top of high strength rock in metres or retained height, whichever is less.

The additional lateral pressures arising from adjacent pavement areas behind the walls, particularly due to construction traffic surcharge loading (e.g. 5-10 kPa), should 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 hydrostatic head should also be considered in design if positive drainage measures are not incorporated to prevent groundwater pressure build-up behind the wall. Under these circumstances though, the buoyant unit weight of soil can be adopted below the design groundwater level used.

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 high strength rock 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 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 will generally be necessary to restrain the toe of each or alternate piles with temporary or permanent rock bolts, as appropriate.

6.2.2.2 Potential Mobilised Wedge Loading

The design of the temporary shoring system and possibly the long-term basement perimeter wall must also cater for a possible mobilised wedge that would give rise to a total anchor force of $4.3*h^2$ (kN/m) where h is the depth to the joint where failure may occur. 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 interface between the weaker rock and underlying high strength rock;

- Shear strength at interface: $\phi' = 25°$, $c' = 0$ kPa; and
- Bulk unit weight of rock 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. The anchor inclination is considered to be the flattest angle that can realistically be used which will allow relatively easy anchor installation and grouting. Should there be a requirement to increase the angle of installation of the anchors then, to keep a similar factor of safety to that designed for, then 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	6
20	13
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 shoring and anchoring 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 ($4.3h^2$) 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 considerable interaction between the Structural and Geotechnical Engineers.

6.2.3 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 practicable, 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)
Silty Clay	20
Extremely low strength rock	40
Very low strength rock	60
Low strength fractured rock	150
Medium strength, slightly fractured rock	300
High strength or better rock	600

Ground anchors should be designed to have a free length equal to their height above the base of the excavation (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 are indicative only and assume 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. Contractors should justify their own choice of values by proof testing and periodic checks as bond stress depends a lot on construction procedures and equipment.

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 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 including pipes during anchor installation.

6.2.4 Ground Movements

For a relatively major excavation such as is proposed, there is a possibility that there will be some additional horizontal gound 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 40 m 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.

As noted previously, 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 nearby buildings it is recommended that dilapidation surveys be carried out prior to excavation works so that an appropriate response may be made to damage claims.

6.3 Groundwater

The basement excavation is proposed to just below RL 43.0 AHD and therefore little seepage is expected to occur into the basement, based on the groundwater level noted in the piezometer installed at borehole BH5, which was measured below the proposed bulk excavation level. Any seepage that does occur will probably be along the soil-rock interface and through defects in the rock following periods of intense and/or prolonged rainfall.

Pumping from open sump pits is therefore likely to be a sufficient measure 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 sumps may 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 piezometer in borehole BH5 to assess its compliance with the ANZECC water quality guidelines. Further monitoring would also be needed during construction. If the tested water quality complies with the guidelines, then it may be pumped directly into the stormwater system, subject to the approval of the relevant government authorities. 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. In some circumstances, if groundwater is substantially contaminated, then it may be necessary to dispose of it off-site as liquid waste.

6.4 Foundations

6.4.1 Basement

The floor of the basement excavation will be just below RL 43.0 AHD as shown on Sections A-A′ and B-B′ (Drawings 2 and 3, respectively). The proposed basement excavation level intersects high and very high strength sandstone over the south western half of the basement footprint. Moderately compacted filling, however, is intersected at the proposed basement excavation level in the north east perimeter of the proposed basement footprint. Structures founded partly on rock and partly on soil should be avoided, to reduce the risk of adverse differential settlement across the basement substructure. All foundations should therefore be taken down into rock. Suitable foundation types over the south western half of the proposed basement footprint include piles and spread footings such as pads or strip footings. In the vicinity of the proposed basement's north east perimeter, however, piled foundations socketed into rock would be most suitable.

Footings founded in high and very high strength sandstone over the south western half of the proposed basement may be designed for a maximum allowable bearing pressure of 6000 kPa. Footings founded in extremely low and very low strength siltstone and sandstone over the north eastern half of the proposed basement, however, may be designed for a maximum allowable bearing pressure of only 700 kPa.

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.

With regard to proving of 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 footings proportioned on the basis of an allowable bearing pressure of 6000 kPa. If the maximum allowable bearing pressure were limited, however, to say, 3000 kPa, the foundation proving requirements could be limited to footing inspections by a geotechnical engineer.

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.

An experienced geotechnical professional should inspect all pile excavations and spread footings (e.g. pads) prior to the placement of steel and concrete.

6.4.2 Building in Minor Part of Site

Foundations for the proposed 1 – 2 storey building should be taken down below the filling to the very stiff silty clay, for which a maximum allowable bearing pressure of 100 kPa is recommended.

6.5 Seismic Design

Based on the sub-surface conditions encountered at the borehole locations, the site has been assessed in accordance with Section 4 of AS 1170.4 – 2007 (Structural Design Actions: Part 4 - Earthquake Actions in Australia) and has been assigned to the site sub-soil Class B_e (Rock). Based on Table 3.2 of AS1170, the Hazard Factor (Z) for the site is 0.08.

6.6 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). Reference should be made to Table 6.2 of AS 2159 – 1995 to determine minimum cover to reinforcement required, based on the exposure classification made in this section and minimum concrete strength to be used.

DOUGLAS PARTNERS PTY LTD

Reviewed by

Atha Kapitanof
Geotechnical Engineer

Dr Terry Wiesner
Principal

REFERENCES:

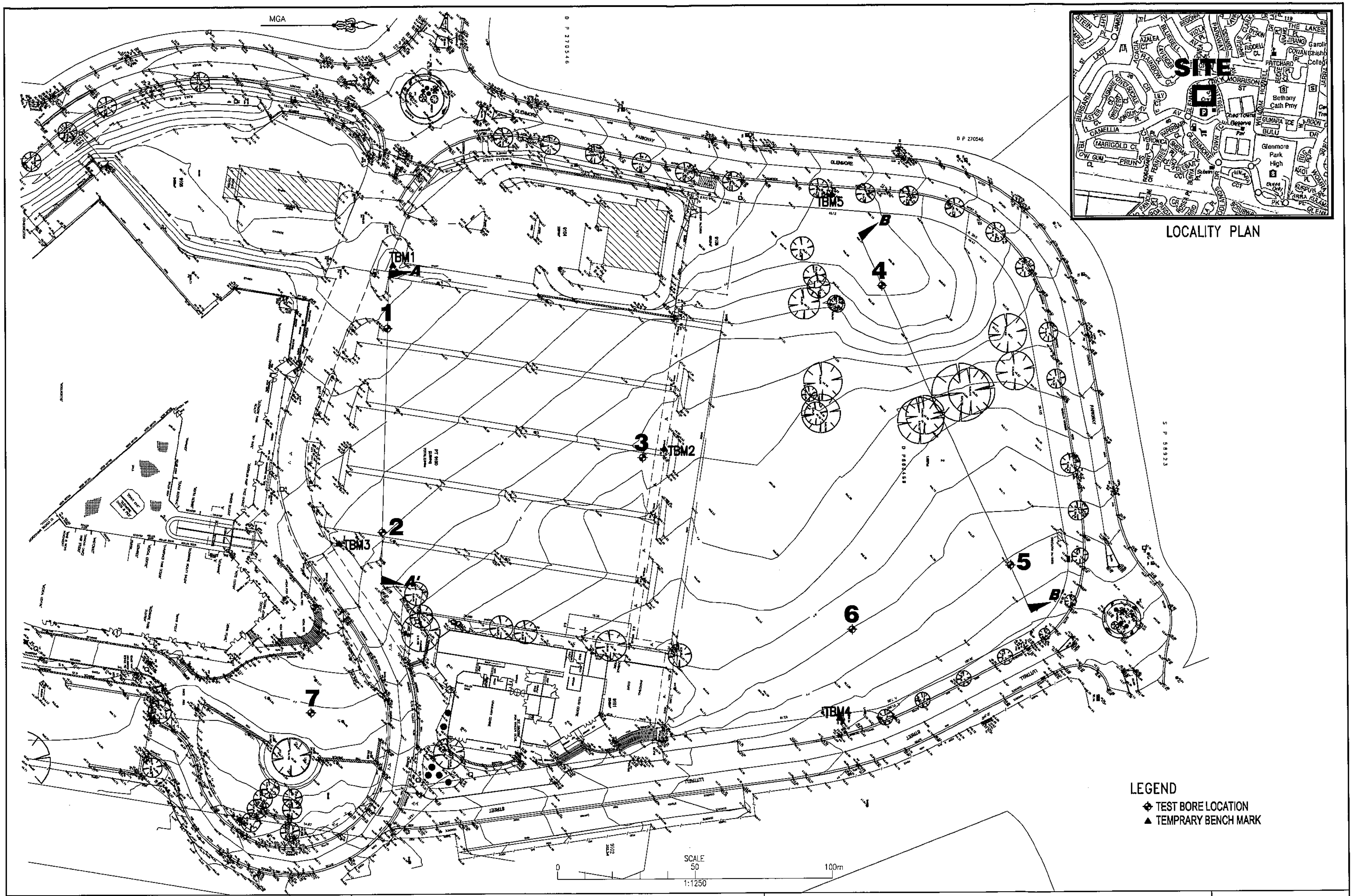
1. Pells, P.J., Mostyn, G. and Walker, B.F. "Foundations on Sandstone and Shale in the Sydney Region". Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

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



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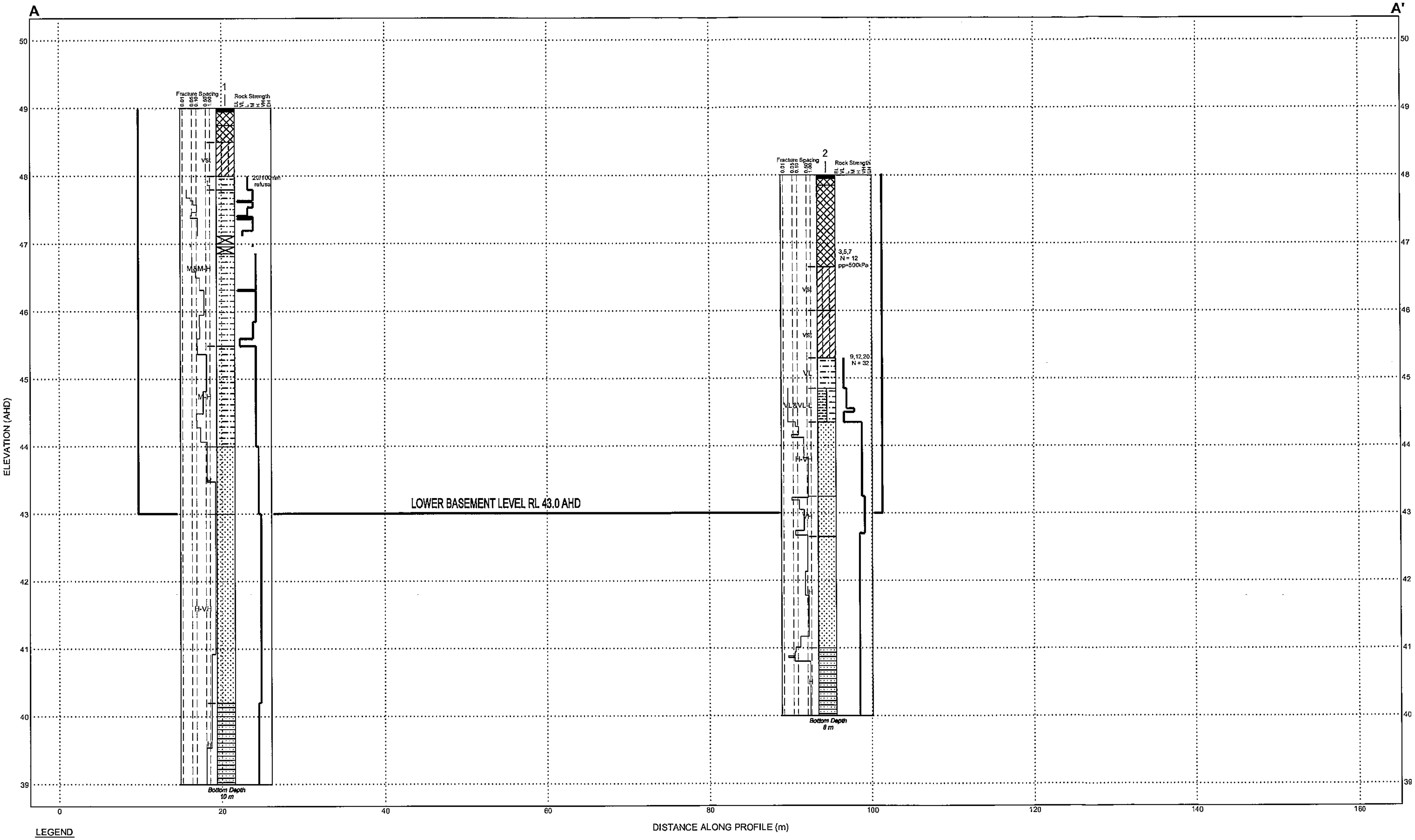
CLIENT: AMP Capital Shopping Centres

DRAWN BY: PSCH
SCALE: As shown
OFFICE: Sydney
APPROVED BY:
DATE: 15.6.2009

TITLE: **Location of Exploratory Boreholes**
Glenmore Park Town Centre
Glenmore Parkway, GLENMORE PARK

PROJECT No: 71169
DRAWING No: 1
REVISION: A

71169 GLENMORE PARK, Glenmore Park Town Centre GNJ\Drawings\71169-1.dwg, 6/15/2009 3:10:54 PM



LEGEND

- ■ Bituminous Concrete
- ▩ Filling
- ▨ Silty Clay
- ▤ Siltstone
- ☒ Core Loss
- ░ Sandstone fine grained
- ▤ Laminite
- ▤ Shale

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

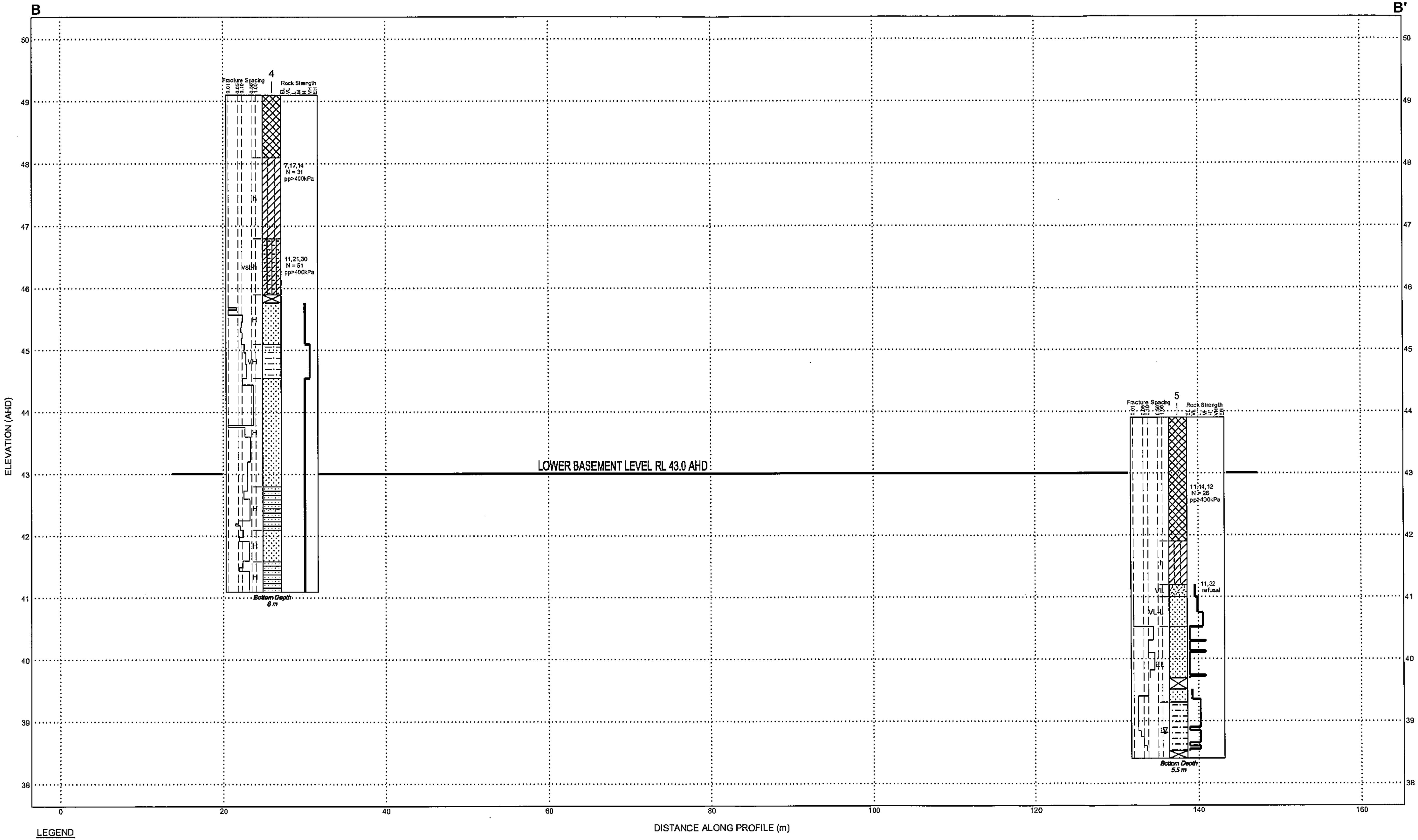
TESTS / OTHER
N - Standard penetration test value
▼ - Water level

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: AMP Capital Investors Ltd
DRAWN BY: SCP
SCALE: 1: 450 (H)
OFFICE:
APPROVED BY:
DATE: 15.06.2009

TITLE: **Geological Section**
Glenmore Park Town Centre
Glenmore Parkway, Glenmore Park

PROJECT No: 71169
DRAWING No: 2
REVISION: 0



LEGEND

- ⊠ Filling
- ▨ Silty Clay
- ▨ Shaly Clay
- ▥ Silt
- ☒ Core Loss
- ░ Sandstone fine grained
- ▤ Siltstone
- ▤ Laminite
- ░ Sand

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

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CLIENT: AMP Capital Investors Ltd
DRAWN BY: SCP
SCALE: 1: 450 (H)
OFFICE:
APPROVED BY:
DATE: 15.06.2009

TITLE: **Geological Section**
Glenmore Park Town Centre
Glenmore Parkway, Glenmore Park

PROJECT No: 71169
DRAWING No: 3
REVISION: 0

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APPENDIX B

Notes Relating to this Report
Results of Field Work

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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:—

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

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.

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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

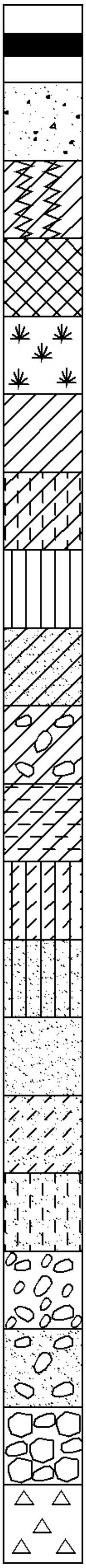
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

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.

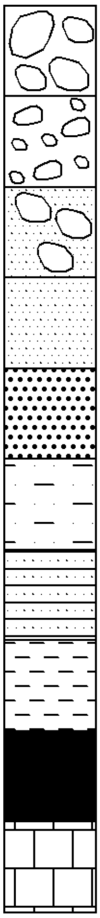
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



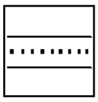
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SILT
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
COBBLES/BOULDER
TALUS

SEDIMENTARY ROCK

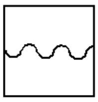


BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

SEAMS

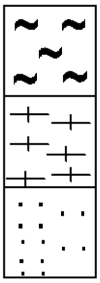


SEAM
>10mm



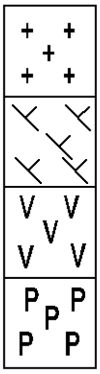
SEAM
<10mm

METAMORPHIC ROCK



SLATE, PHYLLITE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

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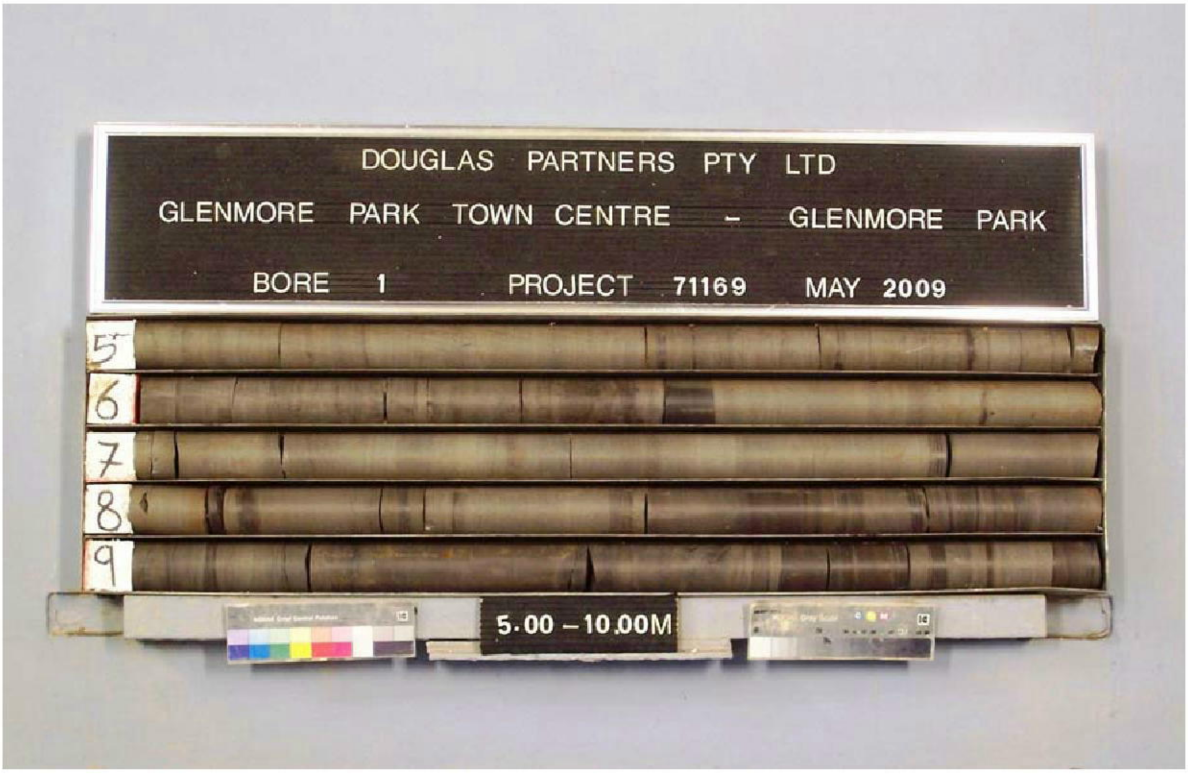
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 1 PROJECT 71169 MAY 2009
GLENMOOR PARK, JOB Nº 71169, BH.1. START CORE AT 1.20m
1.20m
C.L ~~0.12~~ 0.12m
2
CL 0.10m
3
4
1.20 – 5.00M

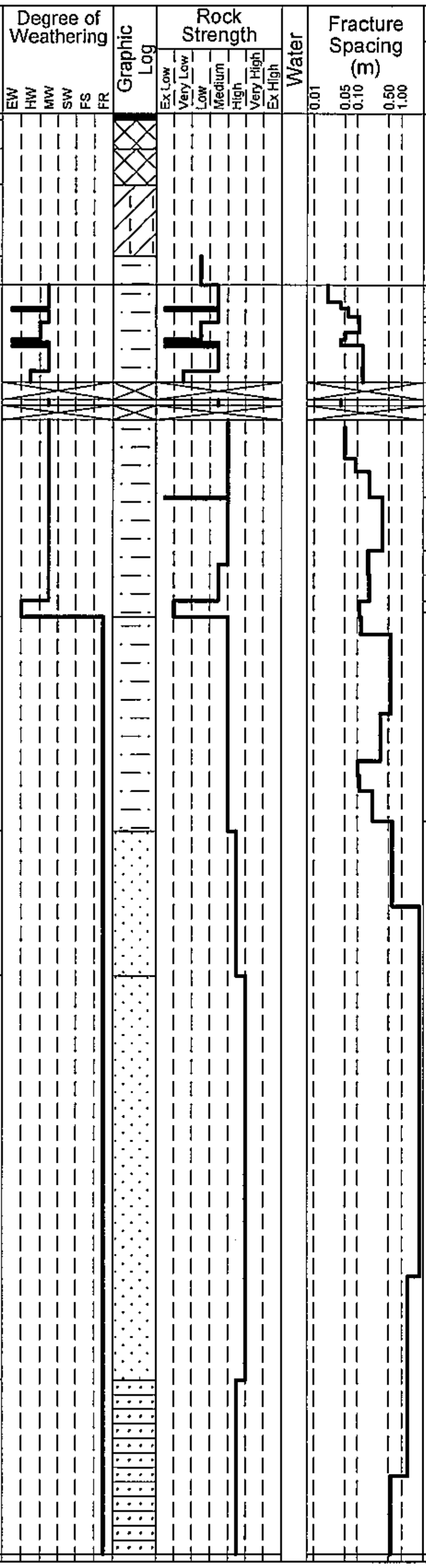

DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 1 PROJECT 71169 MAY 2009
5
6
7
8
9
5.00 – 10.00M

BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 49.0 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 71169
DATE: 13 May 09
SHEET 1 **OF** 1



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
49	0.05	ASPHALTIC CONCRETE		A			
	0.25	FILLING - orange sandy clay filling, with some angular gravel, humid	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10° or joints				
	0.5	FILLING - orange brown gravelly fine to coarse grained sand filling, humid		A			
48	1 / 1.0	SILTY CLAY - very stiff, grey mottled red silty clay, damp		A S			20/100mm refusal
	1.2	SILTSTONE - low strength, highly weathered, grey siltstone	1.2m: J85°- 90°, irregular 1.36m: B0°, 20mm clay 1.42m: B0°, 5mm clay 1.46-1.58m: B (x2) 0°, clay smear 1.58m: B0°, 10mm clay 1.62m: B0°, 5mm clay	C	77	37	PL(A) = 0.4MPa
47	1.88 2 / 2.04	SILTSTONE - medium and medium to high strength, moderately weathered, fractured to slightly fractured, light grey and brown siltstone with fine grained sandstone bands and laminations and some extremely low strength bands	1.75-1.88m: drilling breaks 1.88m: CORE LOSS: 120mm 2.04m: CORE LOSS: 100mm 2.1-2.5m: B (x3) 0°, clay veneer	C	100	84	PL(A) = 1.1MPa
46	3		2.68m: B0°, 10mm clay 3.05m: J35°, ironstained 3.22m: J45°- 50°, smooth, curved 3.4m: B0°, 20mm clay 3.48m: B0°, clay smear	C	100	91	
	3.51	SILTSTONE - medium to high strength, fresh, slightly fractured, grey siltstone					
45	4		4.5m: J45°, rough 4.72m: J45°- 50°, stepped, healed 4.93m: J30°				PL(A) = 1MPa
44	5 / 5.0	SANDSTONE - high strength, fresh, slightly fractured, light grey, fine grained sandstone with some siltstone laminations	5.52-8.08m: B (x2) 0°, ironstained	C	100	100	PL(A) = 2.3MPa
43	6 / 6.0	SANDSTONE - high to very high strength, fresh, unbroken, light grey, fine grained sandstone with some siltstone bands					PL(A) = 6.4MPa
42	7						
41	8			C	100	100	PL(A) = 2.5MPa
							PL(A) = 3MPa
	8.8	LAMINITE - high strength, fresh, slightly fractured and unbroken, light grey to grey laminite. Approximately 50% fine grained sandstone laminations and bands					
40	9			C	100	100	PL(A) = 1.7MPa
							PL(A) = 1.7MPa
	10 / 10.0						

Bore discontinued at 10.0m

RIG: DT 100 **DRILLER:** LC **LOGGED:** SI **CASING:** HQ to 1.0m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.2m; NMLC-Coring to 10.0m

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_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: LK
Date: 19/06/09

Douglas Partners
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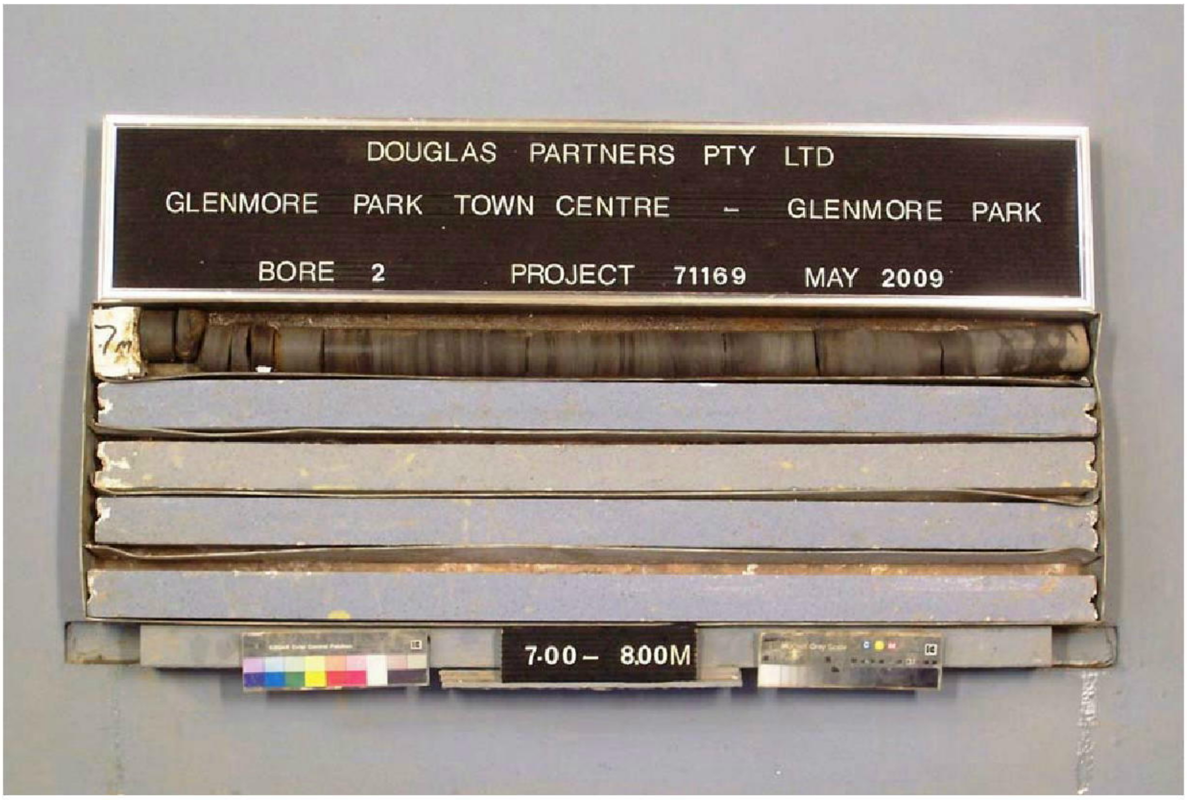
DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 2 PROJECT 71169 MAY 2009
GLENMOOR PARK. JOB № 71169. BH2. START CORE AT 3.15m
3.15m
4
5
6
3·15 – 7.00M


DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 2 PROJECT 71169 MAY 2009
7m
7·00 – 8.00M

BOREHOLE LOG

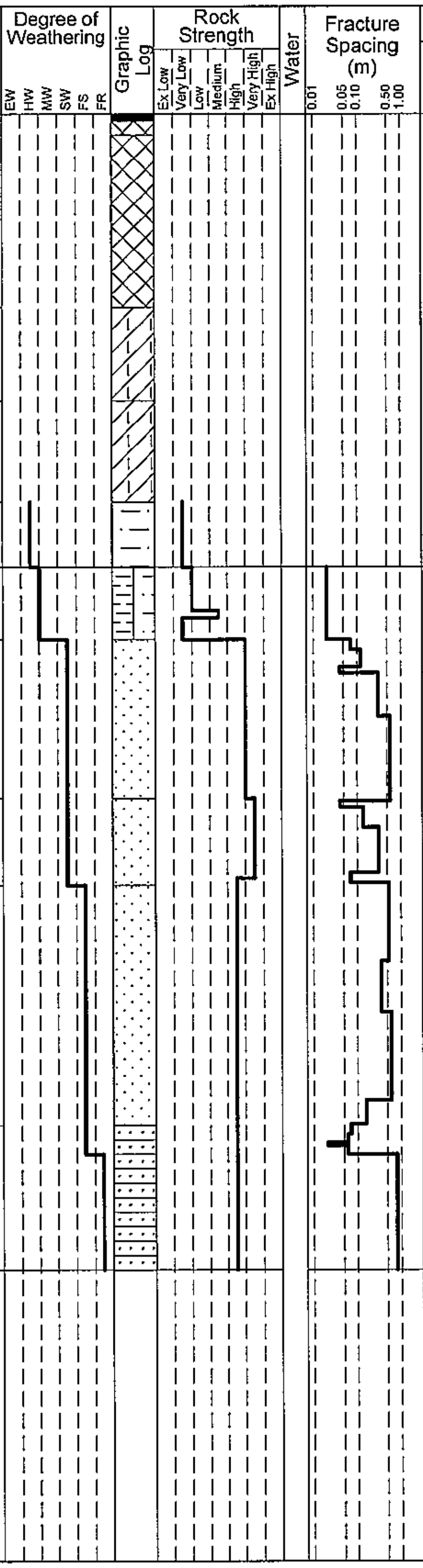
CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 48.0 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 71169
DATE: 14 May 09
SHEET 1 **OF** 1

RL	Depth (m)
48	
47	1
46	2
45	3
44	4
43	5
42	6
41	7
40	8
39	9

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE		A			
0.15	FILLING - orange brown and grey, gravelly sand filling, humid		A			
	FILLING - moderately compacted, orange brown and grey, silty clay filling with some gravel, humid (engineered filling)		A S			3,5,7 N = 12 pp=500kPa
1.35	SILTY CLAY - very stiff, orange brown and grey, silty clay with some fine sand, damp					
2.0	SILTY CLAY - very stiff, mottled red brown and grey, silty clay with some ironstone gravel bands, damp	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10° or joints				
2.7	SILTSTONE - very low strength, highly weathered, grey and brown siltstone with ironstone bands		S			9,12,20 N = 32
3.15	SHALE/SILTSTONE - very low and very low to low strength, highly to moderately weathered, highly fractured, dark grey shale/siltstone	3.15-3.65m: fractured along bedding planes at average 20mm intervals	C	100	28	
3.65	SANDSTONE - high to very high strength, slightly weathered, slightly fractured, light grey, fine grained sandstone	3.72-3.84m: B (x2) 0°, ironstained 3.88m: J85°, rough 4.18-4.77m: B (x2) 0°, ironstained				PL(A) = 2.2MPa PL(A) = 3.7MPa
4.75	SANDSTONE - very high strength, slightly weathered, slightly fractured, light grey and brown, fine grained sandstone	4.87m: J75°, ironstained 4.95m: J85°, ironstained	C	100	96	PL(A) = 7.8MPa
5.35	SANDSTONE - high strength, fresh stained, slightly fractured, light grey, fine grained sandstone	5.26-7.20m: B (x10) 0°-5°, ironstained 5.8m: B 0° ironstained 6.25m: B 0° ironstained				PL(A) = 1.3MPa PL(A) = 1.5MPa
7.0	LAMINITE - high strength, fresh, slightly fractured, grey laminite	7.2m: B 0° ironstained	C	100	82	PL(A) = 1.1MPa PL(A) = 2.6MPa
8.0	Bore discontinued at 8.0m					



RIG: DT 100 **DRILLER:** LC **LOGGED:** JB/SI **CASING:** HQ to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.15m; NMLC-Coring to 8.0m
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: uk
Date: 19/06/09

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BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 46.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 71169
DATE: 12 May 09
SHEET 1 **OF** 1

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01 0.05 0.10 0.50 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
	0.1	ASPHALTIC CONCRETE							A			
	0.2	FILLING - orange brown gravelly sand filling, humid							A			
	0.7	FILLING - orange brown and grey, silty clay filling, humid										
		SILTY CLAY - very stiff, grey mottled red, silty clay with some ironstone gravel, damp							A S			5,7,11 N = 18 pp>400kPa
		1.7m: becoming grey with ironstone bands										
	2.1	SHALE - very low strength, highly weathered, grey shale	HW		Very Low							
	2.51	Bore discontinued at 2.51m - refusal on very low strength, grey shale							A S			20/10mm refusal

RIG: DT 100 **DRILLER:** LC **LOGGED:** JB **CASING:** Uncased

TYPE OF BORING: Solid flight auger (TC-bit) to 2.51m

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_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:
Date: 17/6/09

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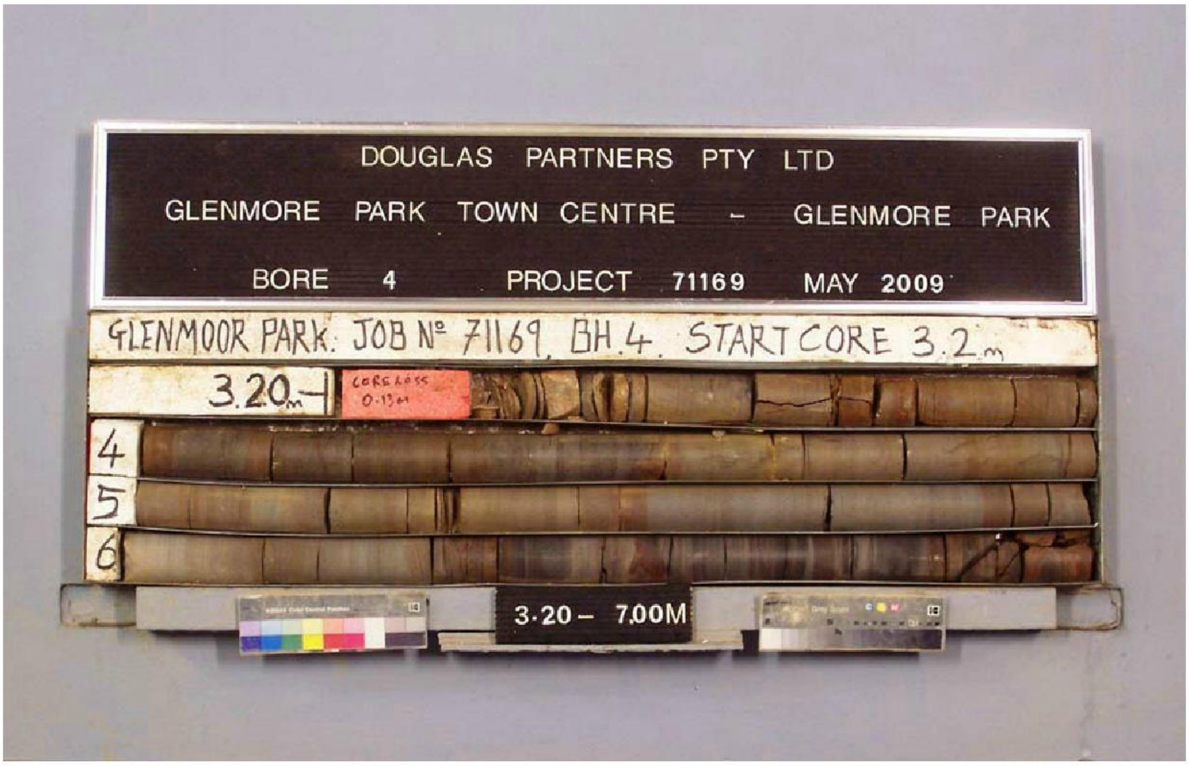
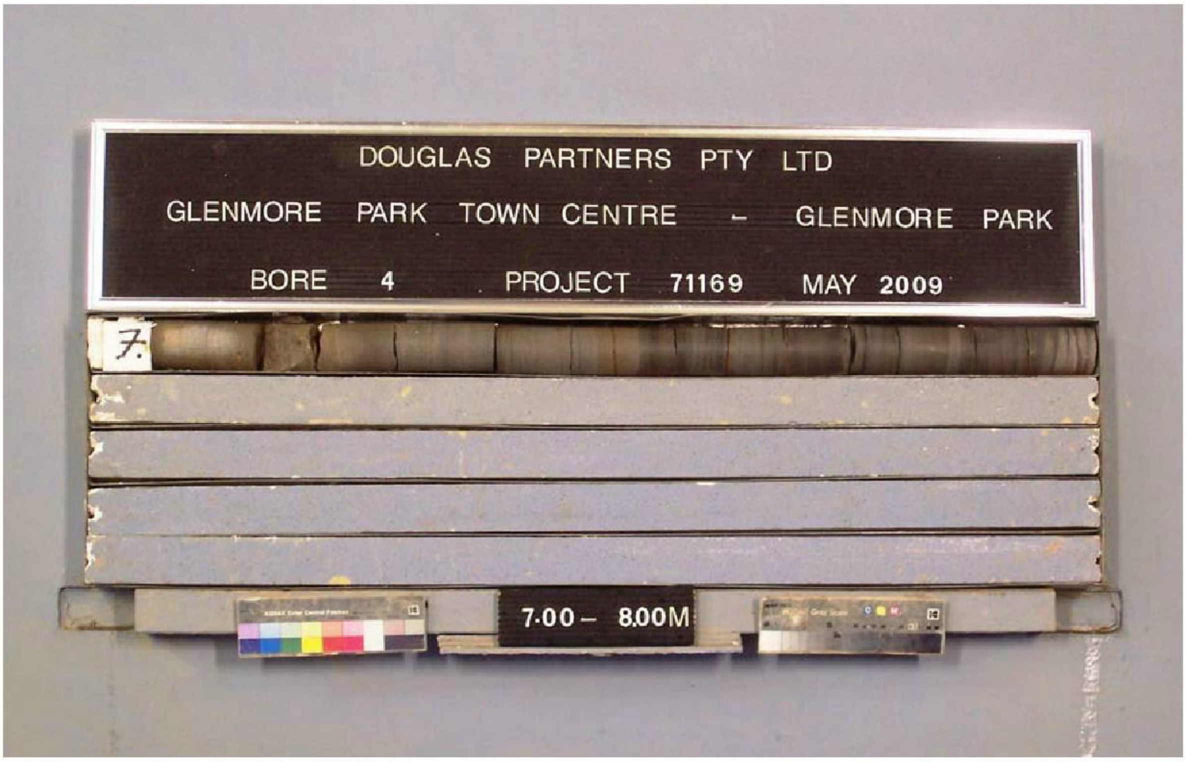

DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 4 PROJECT 71169 MAY 2009
GLENMOOR PARK. JOB Nº 71169, BH.4. START CORE 3.2m
3.20m
CORE LOSS 0.13m
4
5
6
3·20 – 7.00M

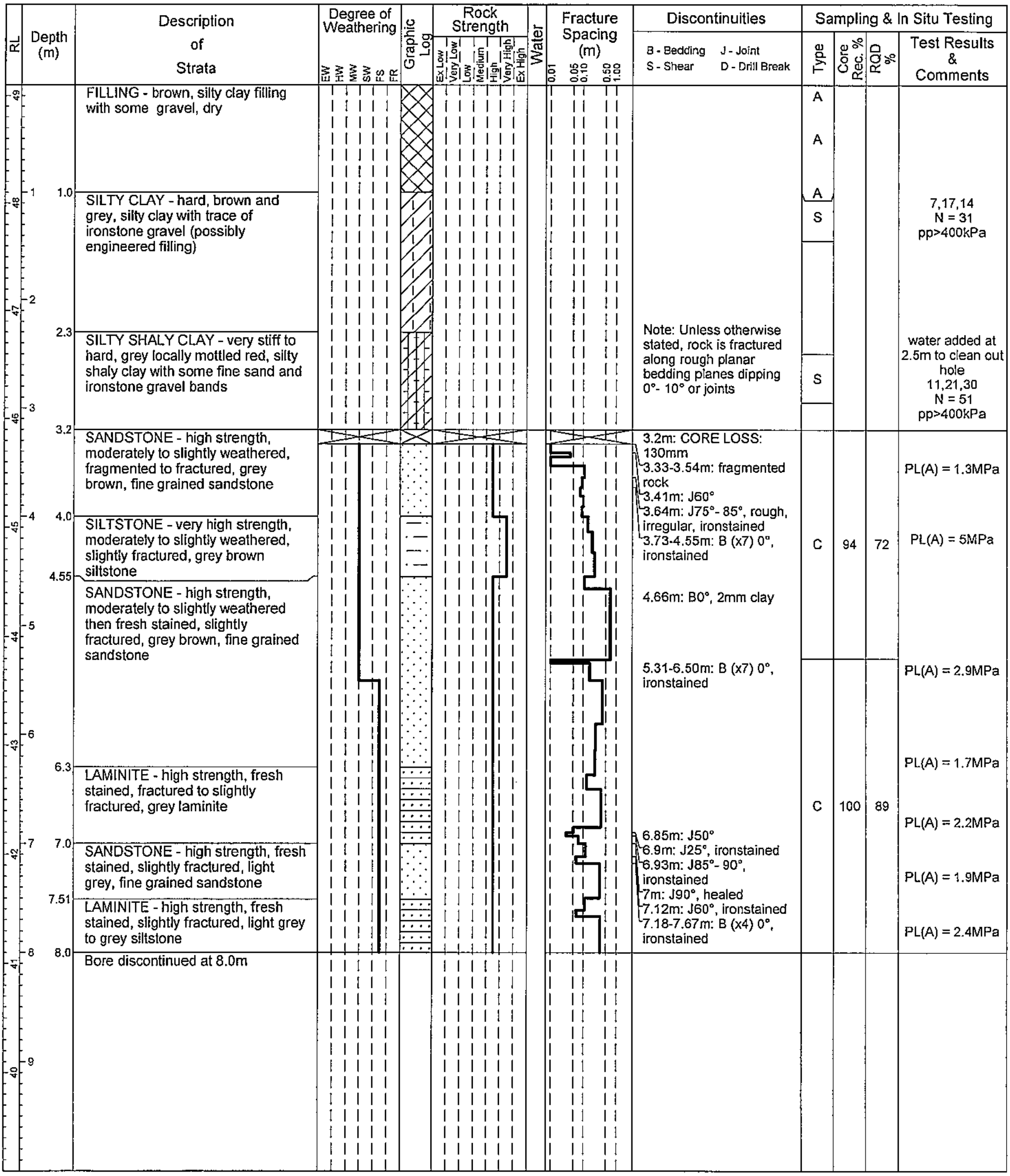

DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 4 PROJECT 71169 MAY 2009
7.
7·00 – 8.00M

BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 49.1 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 71169
DATE: 14 May 09
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
	FILLING - brown, silty clay filling with some gravel, dry		A A A			
1.0	SILTY CLAY - hard, brown and grey, silty clay with trace of ironstone gravel (possibly engineered filling)		S			7,17,14 N = 31 pp>400kPa
2.3	SILTY SHALY CLAY - very stiff to hard, grey locally mottled red, silty shaly clay with some fine sand and ironstone gravel bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10° or joints	S			water added at 2.5m to clean out hole 11,21,30 N = 51 pp>400kPa
3.2	SANDSTONE - high strength, moderately to slightly weathered, fragmented to fractured, grey brown, fine grained sandstone	3.2m: CORE LOSS: 130mm 3.33-3.54m: fragmented rock 3.41m: J60° 3.64m: J75°- 85°, rough, irregular, ironstained 3.73-4.55m: B (x7) 0°, ironstained	C	94	72	PL(A) = 1.3MPa
4.0	SILTSTONE - very high strength, moderately to slightly weathered, slightly fractured, grey brown siltstone					PL(A) = 5MPa
4.55	SANDSTONE - high strength, moderately to slightly weathered then fresh stained, slightly fractured, grey brown, fine grained sandstone	4.66m: B0°, 2mm clay 5.31-6.50m: B (x7) 0°, ironstained	C	100	89	PL(A) = 2.9MPa
6.3	LAMINITE - high strength, fresh stained, fractured to slightly fractured, grey laminite	6.85m: J50°				PL(A) = 1.7MPa PL(A) = 2.2MPa
7.0	SANDSTONE - high strength, fresh stained, slightly fractured, light grey, fine grained sandstone	6.9m: J25°, ironstained 6.93m: J85°- 90°, ironstained 7m: J90°, healed				PL(A) = 1.9MPa
7.51	LAMINITE - high strength, fresh stained, slightly fractured, light grey to grey siltstone	7.12m: J60°, ironstained 7.18-7.67m: B (x4) 0°, ironstained				PL(A) = 2.4MPa
8.0	Bore discontinued at 8.0m					

RIG: DT 100 **DRILLER:** LC **LOGGED:** JB/SI **CASING:** HQ to 2.5m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.2m; NMLC-Coring to 8.0m

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: LK
Date: 19/06/09

Douglas Partners
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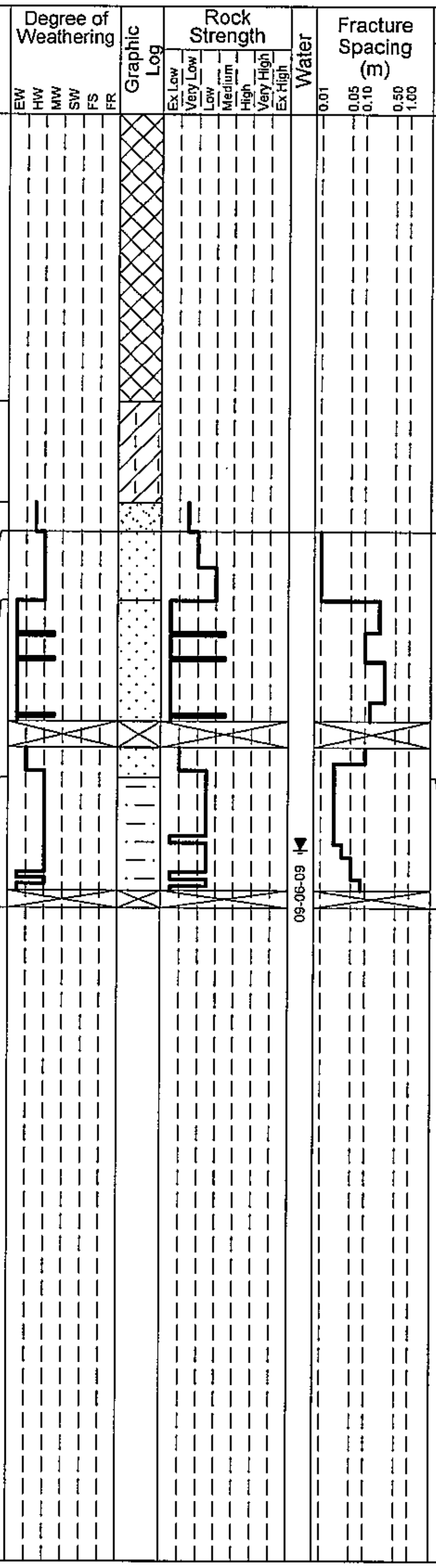
DOUGLAS PARTNERS PTY LTD
GLENMORE PARK TOWN CENTRE - GLENMORE PARK
BORE 5 PROJECT 71169 MAY 2009
GLENMORE PARK 71169 BH 5 15/5/09 START 2·9m
3
4
CORE LOSS
5
CORE LOSS
TB END @ 5·50 M
2·90 – 5.50M

BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 43.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 71169
DATE: 15 May 09
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
	FILLING - compacted, red brown and brown with grey mottling, silty clay filling (engineered filling)		A A A S			11,14,12 N = 26 pp>400kPa
2.0	SILTY CLAY - hard, red mottled grey, silty clay with ironstone gravel bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10° or joints	S			11,32 refusal
2.7	SANDSTONE - very low strength, highly weathered, grey and orange, fine grained sandstone					
2.9	SANDSTONE - very low to low then low to medium strength, highly to moderately weathered, fragmented, light grey brown, fine to medium grained sandstone	2.9-3.28m: fragmented rock				
3.38	SANDSTONE - extremely low strength, extremely weathered, light grey, fine grained sandstone with medium strength ironstone bands	3.38-4.5m: extremely low strength with ironstone bands	C	88	0	
4.21		4.21m: CORE LOSS: 180mm				
4.6	SILTSTONE - low strength, highly to moderately weathered, highly fractured to fractured, grey brown siltstone with some extremely low strength bands	4.5-5.38m: highly fractured to fractured 4.6-5.0m: J90° 5.15m: J85°, ironstained	C	89	0	PL(A) = 0.2MPa
5.38		5.38m: CORE LOSS: 120mm				
5.5	Bore discontinued at 5.5m					

RIG: DT 100 **DRILLER:** LC **LOGGED:** JB/SI **CASING:** HQ to 2.5m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 2.9m; NMLC-Coring to 5.5m

WATER OBSERVATIONS: No free groundwater observed whilst augering; 18/05/09 at 3.1m depth; 19/05/09 at 3.0m depth; 9/06/09 at 5.1m depth

REMARKS: Standpipe installed to 5.5m

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 lk
Date: 19/06/09

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 44.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 71169
DATE: 12 May 09
SHEET 1 **OF** 1

Depth (m)	Description of Strata	Degree of Weathering	Rock Strength	Sampling & In Situ Testing Type	Test Results & Comments
0.0	FILLING - hard, brown, silty clay filling with some angular to sub-angular, fine to medium gravel, dry (engineered filling)			A A A S	 3,4,5 N = 9 pp>400kPa
				A S	5,7,9 N = 16 pp>400kPa NB water added at 2.5m to clean out hole
2.3	SILTY CLAY - very stiff, red mottled brown and grey, silty clay, humid				
2.6	SILTY CLAY - very stiff, orange and grey, silty clay with some fine sand, damp				
4.0	SILTSTONE - very low strength, highly weathered, orange and grey, sandy siltstone	HW	Very Low	S	6,40/150mm refusal
4.3	Bore discontinued at 4.3m				

RIG: Scout **DRILLER:** LC **LOGGED:** JB **CASING:** Uncased

TYPE OF BORING: Solid flight auger (TC-bit) to 4.0m

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: LK
Date: 19/06/09

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: AMP Capital Investors Ltd
PROJECT: Glenmore Park Town Centre
LOCATION: Glenmore Parkway, Glenmore Park

SURFACE LEVEL: 46.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 71169
DATE: 12 May 09
SHEET 1 **OF** 1

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
	0.05	PAVERS							A			
	0.15	FILLING - orange brown, gravelly sand filling, humid										
46		FILLING - grey and brown, silty clay filling with some angular to sub-angular, fine to coarse gravel, humid							A			
	1								A			
									S			3,6,5 N = 11 pp=320kPa
45	1.65	SILTY CLAY - very stiff, grey and orange silty clay with some ironstone gravel (possible engineered filling)										
	2											
44									S			5,4,8 N = 12 pp=400kPa
	3											
43	3.3	SILTY SANDY CLAY - hard, orange and grey, silty sandy clay, (possibly highly weathered siltstone)										
	4								S			4,7,5 N = 12 pp>400kPa
	4.45	Bore discontinued at 4.45m										
42	5											
41	6											
40	7											
39	8											
38	9											
37												

RIG: Scout **DRILLER:** LC **LOGGED:** JB **CASING:** Uncased

TYPE OF BORING: Solid flight auger (TC-bit) to 4.45m

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:
Date: 19/06/09

Douglas Partners
Geotechnics · Environment · Groundwater

Document Set ID: 6025239
Version: 1, Version Date: 03/07/2014

APPENDIX C

Results of Laboratory Testing

Douglas Partners
Geotechnics · Environment · Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117

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Australia

PO Box 472
West Ryde NSW 1685

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

RESULTS OF MOISTURE CONTENT TEST

Client: AMP CAPITAL INVESTORS LTD
Project: GLENMORE PARK TOWN CENTRE
Location: GLENMORE PARKWAY, GLENMORE PARK

Project No: 71169
Report No: S09-119
Report Date: 26/05/09
Date Sampled: 21/05/09
Date of Test: 21/05/09
Page: 1 of 1

TEST LOCATION	DEPTH (m)	DESCRIPTION	MOISTURE CONTENT (%)
BH 3	1.0-1.45	CLAY - Light grey and brown mottled red brown clay with some sand	19.6
BH 4	1.0-1.45	SHALY CLAY - Light grey mottled brown shaly clay	15.6
BH 4	2.5-2.95	SHALY CLAY – Light grey mottled orange brown shaly clay	9.02
BH 7	2.5-2.95	SILTY CLAY - Brown mottled grey and orange brown silty clay w some sand and gravel	15.6

Test Method(s): AS 1289.2.1.1-2005
Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001
Remarks:

Approved Signatory:

Tested: LW
Checked: GSY

GEOFFREY S YOUNG
PRINCIPAL

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory Number: 828

This Document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025

Douglas Partners
Geotechnics · Environment · Groundwater

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RESULTS OF MOISTURE CONTENT, PLASTICITY AND LINEAR SHRINKAGE TESTS

Client: AMP CAPITAL INVESTORS LTD
Project: GLENMORE PARK TOWN CENTRE
Location: GLENMORE PARKWAY, GLENMORE PARK

Project No: 71169
Report No: S09-119
Report Date: 26/05/09

Date Sampled: 21/05/09
Date of Test: 21/05/09
Page: 1 of 1

TEST LOCATION	DEPTH (m)	DESCRIPTION	CODE	W_F %	W_L %	W_P %	PI %	*LS %
BH 4	1.0-1.45	SHALY CLAY - Light grey mottled brown shaly clay	2,5	-	42	18	24	-
BH 7	2.5-2.95	SILTY CLAY - Brown mottled grey and orange brown silty clay with some sand and gravel	2,5	-	42	19	23	-

Legend:

W_F	Field Moisture Content
W_L	Liquid limit
W_P	Plastic limit
PI	Plasticity index
LS	Linear shrinkage from liquid limit condition (Mould length 150mm)

Code
Sample history for plasticity tests
1. Air dried
2. Low temperature (<50°C) oven dried
3. Oven (105°C) dried
4. Unknown

Test Methods:

Moisture Content:	AS 1289 2.1.1 - 2005
Liquid Limit:	AS 1289 3.1.2 - 1995
Plastic Limit:	AS 1289 3.2.1 - 1995
Plasticity Index:	AS 1289 3.3.1 - 1995
Linear Shrinkage:	AS 1289 3.4.1 - 1995

Method of preparation for plasticity tests
5. Dry sieved
6. Wet sieved
7. Natural

*Specify if sample crumbled CR or curled CU

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001
Remarks:

Approved Signatory:

Tested: LW/FV
Checked: GSY

GEOFFREY S YOUNG
PRINCIPAL

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NATA Accredited Laboratory Number: 828

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CERTIFICATE OF ANALYSIS 29004

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Grant Jones

Sample log in details:

Your Reference:	**71169, Glenmore Park**
No. of samples:	3 Waters
Date samples received:	19/05/09
Date completed instructions received:	19/05/09

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:	26/05/09
Date of Preliminary Report:	Not Issued
Issue Date:	22/05/09

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Results Approved By:

Jacinta Hurst
Operations Manager

Envirolab Reference: 29004
Revision No: R 00

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Miscellaneous Inorganics				
Our Reference:	UNITS	29004-1	29004-2	29004-3
Your Reference	-------------	71169 pH	71169 Sulphate	71169 Chloride
Date Sampled	------------	19/05/2009	19/05/2009	19/05/2009
Type of sample		Water	Water	Water
Date prepared	-	20/05/2009	20/05/2009	20/05/2009
Date analysed	-	21/05/2009	21/05/2009	21/05/2009
pH	pH Units	7.7	7.5	7.6
Sulphate, SO4	mg/L	<5	16	<5
Chloride (titration) - water	mg/L	440	540	540

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 II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			20/05/09	29004-1	20/05/2009 || 20/05/2009	LCS-W1	20/05/09
Date analysed	-			21/05/2009	29004-1	21/05/2009 || 21/05/2009	LCS-W1	21/05/2009
pH	pH Units		LAB.1	[NT]	29004-1	7.7 || 7.8 || RPD: 1	LCS-W1	101%
Sulphate, SO4	mg/L	5	LAB.9	<5	29004-1	<5 || [N/T]	LCS-W1	97%
Chloride (titration) - water	mg/L	20	LAB.11	<20	29004-1	440 || 440 || RPD: 0	LCS-W1	102%

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Report Comments:

Asbestos was analysed by Approved Identifier: Not applicable for this job
INS: Insufficient sample for this test NT: Not tested PQL: Practical Quantitation Limit <: Less than >: Greater than
RPD: Relative Percent Difference NA: Test not required LCS: Laboratory Control Sample NR: Not requested

Quality Control Definitions

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

Laboratory Acceptance Criteria:

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the sample batch were within 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: 60-140% is acceptable for general organics and 10-140% for SVOC and speciated phenols.

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CERTIFICATE OF ANALYSIS 29050

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Grant Jones

Sample log in details:

Your Reference:	**71169, Glenmore Park Town Centre**
No. of samples:	2 Soils
Date samples received:	21/05/09
Date completed instructions received:	21/05/09

Analysis Details:

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

Report Details:

Date results requested by:	28/05/09
Date of Preliminary Report:	Not issued
Issue Date:	28/05/09

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Results Approved By:

Jacinta Hurst
Operations Manager

Envirolab Reference: 29050
Revision No: R 00

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Miscellaneous Inorg - soil			
Our Reference:	UNITS	29050-1	29050-2
Your Reference	-------------	BH2/2.5-2.95	BH5/2.5-2.95
Date Sampled	------------	13/05/2009	13/05/2009
Type of sample		Soil	Soil
Date prepared	-	25/05/2009	25/05/2009
Date analysed	-	27/05/2009	27/05/2009
pH 1:5 soil:water	pH Units	5.2	5.7
Sulphate, SO4 1:5 soil:water	mg/kg	<25	140
Chloride 1:5 soil:water	mg/kg	<100	940

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.

Client Reference: 71169, Glenmore Park Town Centre

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			25/5/09	[NT]	[NT]	LCS-1	25/5/09
Date analysed	-			27/05/09	[NT]	[NT]	LCS-1	27/05/09
pH 1:5 soil:water	pH Units		LAB.1	[NT]	[NT]	[NT]	LCS-1	101%
Sulphate, SO4 1:5 soil:water	mg/kg	25	LAB.9	<25	[NT]	[NT]	LCS-1	95%
Chloride 1:5 soil:water	mg/kg	100	LAB.11	<100	[NT]	[NT]	LCS-1	100%

Report Comments:

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