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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT (WATERBROOK) AT 35 WATER STREET & 64 BILLYARD STREET, WAHROONGA

Prepared for
WATERBROOK AT WAHROONGA PTY LTD

Project 43905
June 2006

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GEOTECHNICAL REPORT
PROPOSED RESIDENTIAL DEVELOPMENT (WATERBROOK) AT 35 WATER STREET AND 64 BILLYARD STREET WAHROONGA

1. INTRODUCTION

This report presents the results of a geotechnical investigation for a complex of five low to medium rise residential buildings at 35 Water Street and 64 Billyard Street, Wahroonga. All buildings will have 1 to 3 levels of basement carpark and 3 to 5 residential levels. The investigation was carried out for Waterbrook at Wahroonga Pty Ltd, the owner of the development.

The investigation comprised the drilling of three auger/diamond drill holes to depths of 2 – 3 m below the bulk excavation level for buildings B and C. These two buildings will have three and two levels of basement carpark respectively. The investigation was carried out to provide information on subsurface conditions for the purpose of assessing excavation and foundation conditions and determining appropriate parameters for the design of foundations, temporary batter slopes and retention and shoring systems for both the short and long term.

2. REGIONAL GEOLOGY AND TOPOGRAPHY

The site lies at the crest of a broad, flat lying ridge which falls gently to the east and south.

From the Sydney 1:100 000 Geological sheet, the area is underlain by shale and laminite (finely interbedded fine grained sandstone and siltstone/shale) of the Ashfield Shale formation of Triassic age. Weathering products of the Ashfield shale are generally medium to high plasticity clay which overlies very low, low and medium strength weathered rock to medium and high strength fresh shale at depth.

Two main, steeply dipping joint sets are generally present within the rock, trending north-northeast and east-southeast. Discontinuous more randomly orientated joints and minor compaction faults, dipping at 30° to 50°, are common. These are often clean, polished or slickensided. More continuous major faults, also dipping at 40° to 50° are more widely spaced. These tend to have associated crushed or clayey zones, 50 to 200 mm thick.

3. SITE DESCRIPTION

The site is an amalgamation of 35 Water Street (Rippon Grange) and 64 Billyard Street, Wahroonga, and is pan-handled in shape. It has an approximate 155 m frontage along Water Street to the north, a 205 m frontage along Young Street to the east, and a 23 m wide frontage to Billyard Street to the south at the start of the "pan-handle". Six residential properties are present to the west of the pan-handle. The western boundary abuts a further three residences.

The northwestern part of the site, occupied by Rippon Grange, is virtually flat, lying between RLs 197 and 199. From this area the ground slopes gently at about 10° to the west and south.

4. SITE INVESTIGATION

4.1 Methods

The site investigation comprised the drilling of three boreholes, BH 1 – 3. Borehole BH 1 was drilled towards the western end of Building C, while Boreholes BH 2 and 3 were drilled approximately centrally in Buildings B1 and B2 respectively (see Drawing 1 for the borehole locations).

The boreholes were drilled using a Bobcat mounted auger/rotary drilling rig. They were augered through the clay soil and upper extremely weathered rock with a 100 mm diameter spiral flight auger to a depth of 2.5 m (SPT refusal). Minor rotary drilling was then carried out to seat the casing. Below depths of 2.8 to 3 m the holes were extended to depths of 10.38 to 13.0 m using NMLC diamond core drilling techniques.

The core was geotechnically logged and point load strength tested. The logs of each borehole are summarised on the section (Drawing 2) while the detailed logs are given in Appendix A, together with Notes Relating to this Report.

4.2 Results

The three bores intersected 1.6 to 2.5 m of topsoil and stiff to hard, orange-brown residual silty clay with SPT values of 19 and 31, overlying extremely weathered, extremely low strength shale. The profiles in each of the three boreholes are summarised in Table 1 below and on the long section (Drawing 2).

Table 1 – Summary of Ground Conditions at Borehole Locations

Stratum	**Depth to Top of Layer (RL)**		
	1	**2**	**3**
Filling/Pavement	0 (189.5)	0 (195.6)	0 (193.4)
Residual Clay	0.3 (189.2)	0.05 (195.5)	0.05 (193.3)
Extremely weathered, generally extremely low strength shale	2.5 (187.0)	1.6 (194.0)	1.7 (191.7)
Highly or highly to moderately weathered, generally very low to low strength shale	4.4 (185.1)	6.0 (189.6)	7.2 (186.2)
Fresh, medium strength shale	6.8 (182.7)	10.7 (184.9) with low strength layer below 12.4	8.73 (184.6)
Fresh, medium strength laminite	7.6 (181.9))	-	12.10 (181.3)

In broad terms the site appears to be underlain by variably weathered shale down to approximately RL 182 with laminite (finely interlayered fine grained sandstone and siltstone) below.

The shale is weathered and of very low to low strength to depths of approximately 9 m (BH 3) to 11 m (BH 2) in the upper part of the site (Buildings A & B).

The weathered shale has numerous bedding plane partings, often clay coated. The fresh shale is more massive except in Borehole 2 which intersected a series of clayey bedding planes below a depth of 12.7 m (RL 182.9). These planes may represent a near horizontal sheared or faulted zone beneath the site.

The borehole also intersected a series of joints within the shale with dips of 30° - 45°, 65° - 70° and 85° - 90°. It is likely that the steeper joints belong to the N – S and E – W set of joints mentioned in Section 2, while the flatter lying joints are likely to be more randomly orientated (see Section 2).

Fresh, medium strength shale is present below these depths though strengths are still at the low end of medium strength (Point Load Strength Index values generally of 0.4 to 0.5 MPa which are approximately equivalent to unconfined compressive strength values of 6 to 8 MPa).

Below the shale, at approximately RL 182 stronger laminite is present.

Due to the use of water during drilling groundwater levels could not be observed. However, from excavations in similar conditions, it is anticipated that free groundwater will be present below depths of approximately 6 m.

5. COMMENTS

5.1 The Proposed Development

The proposed development will entail the construction of a series of low to medium rise (2 to 5 levels) buildings (A to F) across the site, each building having basement car parking.

From the architectural drawings the lowest basement levels for each building will be:

Building No.	Lowest Basement RL	Maximum Depth of Excavation
A1	190.5	3.0
B1	186.5	10.5
B2	186.5	8.5
C	181.5	8.5
D	182.5	3.5
E1	179	2.0
E2	179.2	4.0
F	189	4.5

The depths of excavation are approximate only as the site slopes across all building sites. Column loadings have not yet been determined but are anticipated to range up to 2000 – 2500 kN for the taller buildings (B and C).

5.2 Excavation Conditions

Based on the boreholes it is anticipated that the basements for buildings A, D, E & F will be entirely excavated in clay then extremely low to low strength weathered shale with minor medium strength, iron cemented layers or lenses.

Excavation for the other building basements will also encounter extremely low to low strength weathered shale. However, the lower portions of these excavations are likely to encounter low to medium strength laminite.

The extremely low to low strength rock should be readily excavated by hydraulic excavator with some light ripping. Any medium strength rock will either require heavier ripping or the use of hydraulic rock hammers.

It should be noted that off-site disposal of excavated materials will require assessment for re-use or classification of the soil in accordance with *Environmental Guidelines: Assessment*

Classification and Management of Non-Liquid Wastes (NSW EPA, 1997) prior to disposal at an appropriately licensed landfill.

5.3 Vibration

Although much of the proposed excavation works will be some distance from adjacent buildings and roadways, based on recent experience with a similar development in weathered shale in Killara, it is recommended that prior to excavation starting detailed dilapidation reports be compiled on all surrounding buildings and any other neighbouring structures, particularly those located within 20 m of the excavation. The respective owners of these buildings and the Council should be asked to confirm that the dilapidation reports represent a fair record of actual conditions.

Even though it is envisaged that much of the excavation will be carried out without the need for rock hammers they will probably be locally required.

Vibration and noise associated with the excavation, particularly the use of rock hammers may result in complaints from neighbours and restrictions to equipment operating on site. To prevent structural damage the recommended maximum peak particle velocity from AS 2187 Explosives Code for various structures subject to vibration is 10 mm/sec for houses. However it is recommended that a peak particle velocity of 5 mm/sec at foundation level of adjacent buildings be employed at this site for human comfort considerations.

Vibration monitoring carried out by Douglas Partners at many excavation sites around Sydney has indicated a close relationship between peak particle velocity versus distance for various hammer types, milling heads and rock saw attachments. Table 1 presents a summary of the results.

Table 2 - Measured Relationship Between Hammer Weight/Rock Saw/Milling Head and Distance from Monitor

Hammer Type	Distance (m)	Peak Particle Velocity (mm/s)
Krupp 300 or equivalent	1.0 1.5 2.0	15 10 7
Krupp 600 or equivalent	3.5 6 9	10 4 – 5 2
Krupp 900 or equivalent	6 9 20	20 10 – 11 4
3m diameter saw on excavator	0.5 - 1 1 – 2	5 3
Milling head of excavator	1 1 - 2	5 3

Neighbours will probably find vibration levels above about 3 mm/s as being strongly perceptible to disturbing, hence complaints from neighbours are possible and some reassurance, possibly by vibration monitoring, may be necessary. Further it is recommended that neighbouring residents and occupiers be notified of the proposed timing of the excavation so that any vibration sensitive items can be secured.

Table 2 may be used for initial plant selection, however monitoring may be necessary to establish site-specific relationships if rock breaking equipment is used for extensive excavation close to the boundaries.

5.4 Excavation Support

5.4.1 General

It is understood that excavation is proposed up to or close to the eastern and western site boundaries for Building E. Temporary retaining structures will be required along these boundaries during the construction phase while permanent retaining will be required as part of the final structure.

Elsewhere the excavations will be set back from the boundaries, possibly sufficiently to allow temporary battering of the slopes.

Temporary shoring may be designed to be incorporated within the permanent excavation support. It is expected that all temporary shoring greater than about 2 m will be anchored. The final structure will need to prop or brace the retaining wall system in the long term, so that temporary anchors may be detensioned. 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.

Permanent shoring for this site will need to be designed for lateral earth pressures for the materials above consistently medium strength shale or laminite or for backfill. Where unsupported batters have been excavated it will be necessary to provide drainage at the back of the wall prior to backfilling the void between the outer wall and the batter slopes. Backfilling behind free-standing retaining walls should be carried out using granular fill and compacted using a portable hand operated vibrating plate (sled) compactor, such as a "Wacker Packer".

5.4.2 Unsupported Excavation Batters

Where space permits, battering of the clayey soil and extremely weathered rock may be adopted. For temporary slopes, where groundwater is controlled below excavation level, it is recommended that the sides of the excavation be battered at slopes no steeper than the maximum recommended temporary batters slopes provided in Table 3. For batter heights greater than 5 m it is recommended that an intermediate bench be excavated at mid height, the bench width to be at least 2 m. Where there is insufficient room for battering of the excavation sides, vertical excavation will generally require a full-depth retention system to be installed prior to excavation commencing, as outlined in the following sections. Excavation within the medium to high strength rock may be cut vertically.

Table 3 - Recommended Maximum Batter Slopes for Exposed Material

Material	Temporary Batter Slope (H:V)
Filling or natural clay soils and extremely weathered rock	1:1
Very low to low strength shale	0.5H:1V[1]
Medium strength siltstone (or better)	Vertical(1)

(1) Subject to jointing assessment by experienced Engineering Geologist or Geotechnical Engineer

Unsupported excavated faces in the laminite and siltstone should be inspected by a suitably experienced geotechnical professional at the time of excavation, to assess whether localised dowelling, rock bolting or anchoring is required to control any adversely oriented jointing or faulting within the face and to identify localised areas that may require protection by shotcreting. These inspections should be carried out every 1.5 m increment in depth of excavation.

5.4.3 Shoring above Medium Strength Laminite

5.4.3.1 Shoring

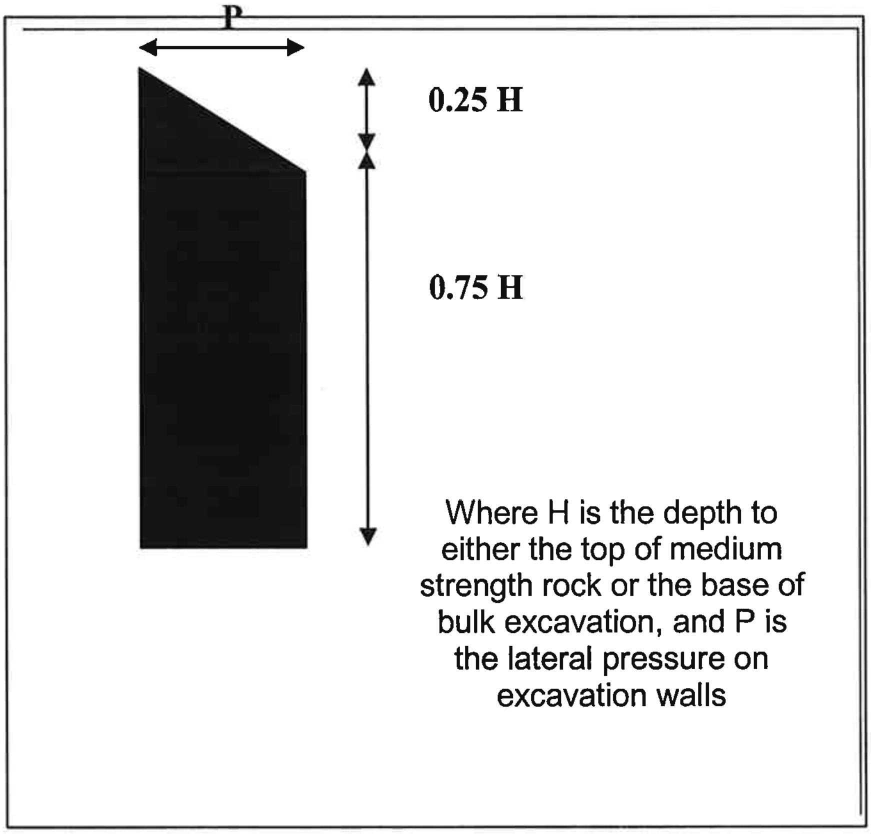
The use of a soldier pile and infill panel wall system is recommended for this site, comprising piles socketed below the bulk excavation level and installed on or behind the excavation line at typical intervals of 2 – 3 m centres in advance of excavation. As excavation proceeds, structurally reinforced shotcrete infill panels, or similar, will need to be constructed between the piles. Provision of 150 mm wide drainage strips or similar behind each panel will be necessary to prevent the build up of hydrostatic pressure behind the wall. Two or more rows of ground anchors drilled through the soldier piles will be required with at least one row located at the pile toe if the piles are not taken to below the level of bulk excavation.

Some of the shoring piles could be adopted for use as structural load bearing members, see Section 5.6, but only if they are taken to below bulk excavation level.

The exposed soil and rock profile in between the soldier piles is expected to be temporarily self-supporting for panel depths of up to 1.5 m, until the ground anchors are installed and the reinforced infill panels are constructed. Adversely oriented jointing may result in localised instability in the exposed material (e.g. unstable wedges) which may require remedial measures prior to shotcreting. It is suggested that regular inspection of the excavated faces be carried out by an experienced Engineering Geologist or Geotechnical Engineer during the course of excavation works to advise further on such stabilisation measures as well as to record the orientation of the joints and faults. This latter information can be used to refine the final support design.

5.4.3.2 Earth Pressure Design

The shoring wall will be subject to earth pressures above the top of medium strength rock (refer to Drawing 2). Horizontal earth pressures for the wall can be calculated using the trapezoidal earth pressure shown below:



Where H is the depth to either the top of medium strength rock or the base of bulk excavation, and P is the lateral pressure on excavation walls

Where: P = 4H (kPa) where some lateral movement is allowable
P = 6H (kPa) where reduced lateral movement is required

Additional pressures should be considered where surcharging of the wall system results from adjacent structures, including that due to adjacent pavement areas behind the wall. In particular, additional pressure due to the surcharge loading of construction traffic adjacent to the site (eg 10 kPa) should be considered.

The earth pressure formulation provided above does not include either earthquake loads or hydrostatic pressure due to the build up of groundwater behind any retaining wall. If drainage behind the wall is not provided full hydrostatic pressure should be allowed for.

5.4.4 Support of Medium and High Strength Rock

Support requirements for medium and high strength material will be dependent on the presence or otherwise of adversely oriented discontinuities in the rock mass.

In jointed laminite and shale, such as exists on this site, the possibility of adversely oriented rock wedges bound by joints or faults "daylighting" on the face or at the base of the cut must be considered. Such an occurrence could result in a major block failure, which may not be adequately resisted by a retaining wall system designed to support earth pressures alone. "Failure" generally refers to the mobilisation of a rock wedge that is associated with a resultant thrust or pressure on the back of the wall.

Experience with deep excavations in Ashfield Shale is that laterally and vertically extensive faults dipping at both 45° and near the horizontal are not uncommon and a number of slope failures along such faults have occurred at other sites. Since the drilling carried out to date has identified a number of 25° to 50° dipping joints and crush zones, the possibility of continuous sheared zones or other weak interfaces influencing excavation stability cannot be discounted.

Excavation will require careful inspection by an experienced engineering geologist or geotechnical engineer at every 1.5 m drop. Support by anchors and/or shotcrete can then be introduced as required, alternately and more prudently a light pattern of anchoring may be extended below the shoring with additional anchors added when required.

From experience with deep excavations in shale, laminite and siltstone, due to the common occurrence of 30 to 60° dipping joints it is strongly recommended that re-entrant cuts are not made within excavated faces as these can often lead to the requirement for excessive additional rock support measures.

5.4.5 Ground Anchors

Anchoring of soldier piles and rock faces may be accomplished by pre-stressed ground anchors. Ground anchors should be inclined to the horizontal no steeper than 30° below the horizontal, to allow anchoring in better quality shale and laminite. The free lengths of all anchors should be extended to behind a 45° plane rising from the base of the excavation. The allowable bond strength for anchor design required for the retaining wall can be determined from Table 4.

Table 4 - Bond Strengths for Retaining Wall Anchors

Material	Allowable bond strength (kPa)	Ultimate bond strength (kPa)
Extremely low strength shale	100	200
Low strength shale	350	700
Medium strength shale / laminite	700	1400
High strength laminite	1000	2000

The parameters given above assume that the anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with good engineering practice.

After anchors have been installed, it is suggested that they be proof loaded to 125% of the nominal design working load and then locked-off at no higher than 80% of the working load until the anchors are decommissioned. Periodic checks should be made throughout the construction period to ensure that the load is maintained and is not lost due to creep effects or other causes.

5.5 Seepage and Groundwater

Minor groundwater inflows into the excavations are anticipated and may occur as local seepage flows along relict joints within the residual clays, at the clay/weathered shale interface and through joints and bedding planes within the laminite and shale bedrock, particularly after heavy rain. The groundwater levels may rise following protracted periods of wet weather.

Monitoring of groundwater seepage into the initial excavation should be undertaken to ensure that the proposed drainage system is adequate for the flows encountered. It is recommended that drainage be installed behind all retaining walls as well as around and below basement floor slabs. The need for the later should be reviewed following inspection of the completed excavation.

Appropriate waterproofing will be required for permanent walls in contact with the excavated areas. The retaining wall drains should be connected into the underfloor drainage system. Groundwater seepage monitoring should be carried out during basement excavation prior to finalising the design of the pump out facility. The sump(s) should have an automatic level control pump to avoid flooding of the basement level. Outlets into the stormwater system will require Council approval.

Note groundwater is likely to have dissolved iron in it. This will flocculate and precipitate on coming into contact with oxygen (forming a red-brown gelatinous mass), particularly if the pH of the water is raised such as by contact with concrete. The above should be taken into account when designing the drainage system.

5.6 Foundations

Footing selection is dependent on foundation loading, the depth of a suitable bearing stratum, the design bearing pressure and the serviceability (settlement) criterion. For Building C pad footings are recommended as a foundation system. A pad footing system could be complimented by using some of the perimeter soldier piles as structural, load bearing columns. All other buildings are likely to be founded on extremely to very low strength shale or laminite; requiring either large pad footings, a raft or possibly pile footings drilled to medium strength rock

5.6.1 Pad Footings

For Building C, pad footings founded below the basement level could be designed for an allowable bearing pressure of 3 500 kPa on medium strength shale/laminite or 6 000 kPa on high strength laminite. These values assume that the foundation excavations are clean and free of loose debris. For the remaining buildings pad footings founded on extremely low strength shale/laminite could be designed for an allowable bearing pressure of 700 kPa.

Total settlement under these working loads should be less than 1% of the minimum footing width.

In general, it is noted that the Ashfield Shale material, when exposed to the atmosphere in an open excavation, is susceptible to breakdown and a loss of strength following drying out and wetting. For this reason, it is recommended that footings are poured within a few hours of final excavation, or alternatively a blinding layer of concrete may be poured to protect the exposed material.

At the time of construction, the adequacy of the founding material in each pad footing should be confirmed by Geotechnical Engineer or Engineering Geologist prior to placement of steel and concrete, including the blinding layer. "Spoon" testing should be undertaken in at least one-third of footings designed for bearing pressures of 3500 kPa or greater.

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.

5.6.2 Piles

Piles founded in at least medium strength shale laminite may be designed for an allowable end bearing pressure of 3500 kPa and a shaft adhesion of 350 kPa, while piles founded within high strength laminite may be designed for an allowable end bearing pressure of 6000 kPa and a shaft adhesion of 600 kPa. Total settlement under these working loads, for footing proportion thus, should be less than 1% of the footing's dimension.

5.7 Basement and Floor Slabs

Weathered bedrock is expected to be exposed beneath the basement floor slab for all buildings. Under-floor drainage will be required and a boundary subsoil drain will be required for seepage collection as mentioned earlier in Section 5.5.

Slabs that will act as a garage pavement should have a subbase layer of at least 100mm thickness of ripped sandstone or crushed concrete compacted to at least 100% of SMDD.

5.8 Design for Earthquake Loads

In accordance with the Earthquake Loading Standard, AS1170.4, 1993, and based on the subsurface conditions encountered, the site has an acceleration coefficient (a) of 0.08 and a site factor of 0.67, assuming that all major structural loads are carried to rock of at least medium strength rock.

DOUGLAS PARTNERS PTY LTD

Reviewed by

J C Braybrooke
Principal Engineering Geologist

G S Young
Principal

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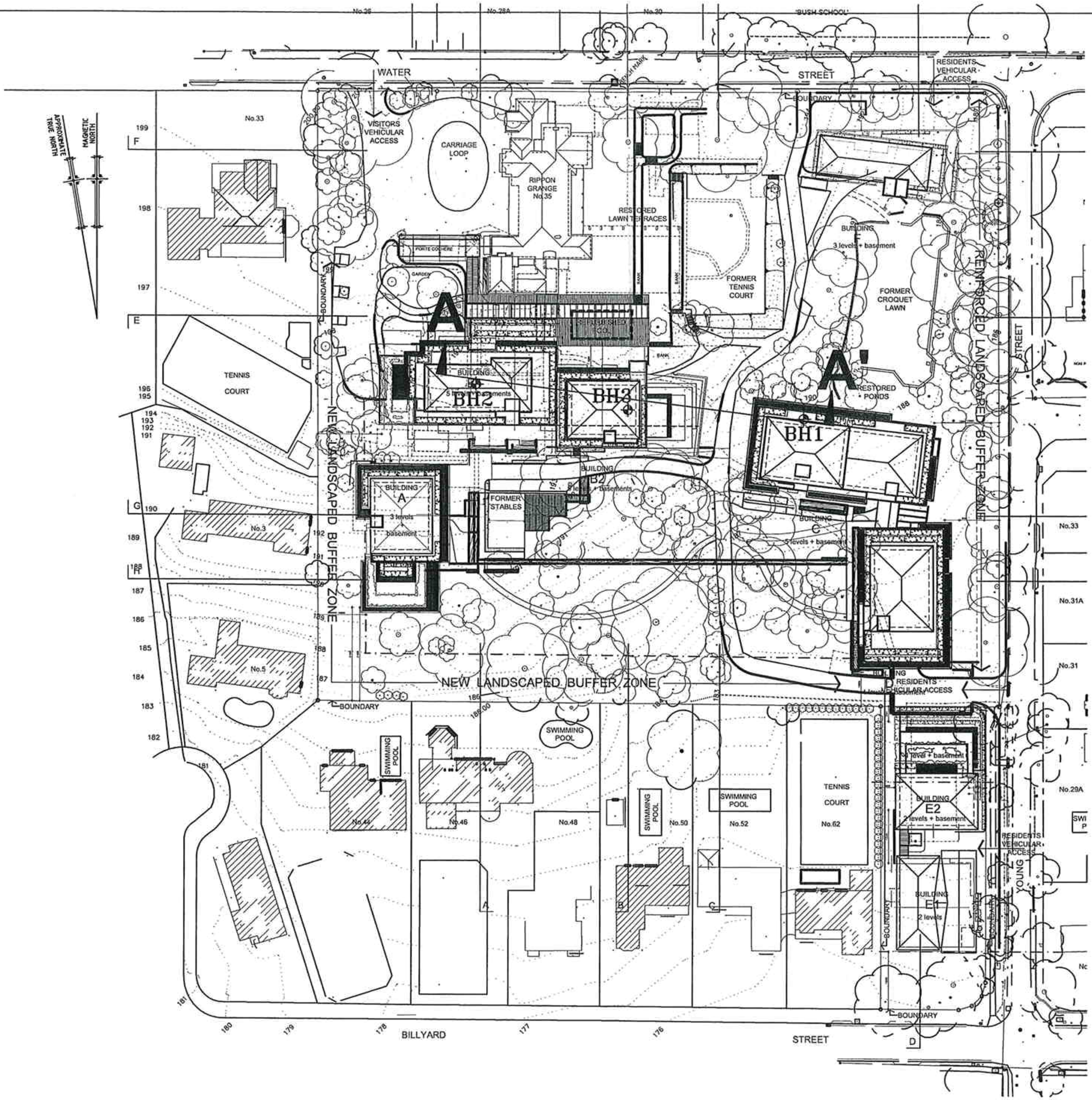

LEGEND

⌖ TEST BORE LOCATION

SCALE
0 10 20 30 40 50m
1:1000

NOTE: See Drawing 2 for Geological Section A–A'

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Geotechnics, Environment, Groundwater

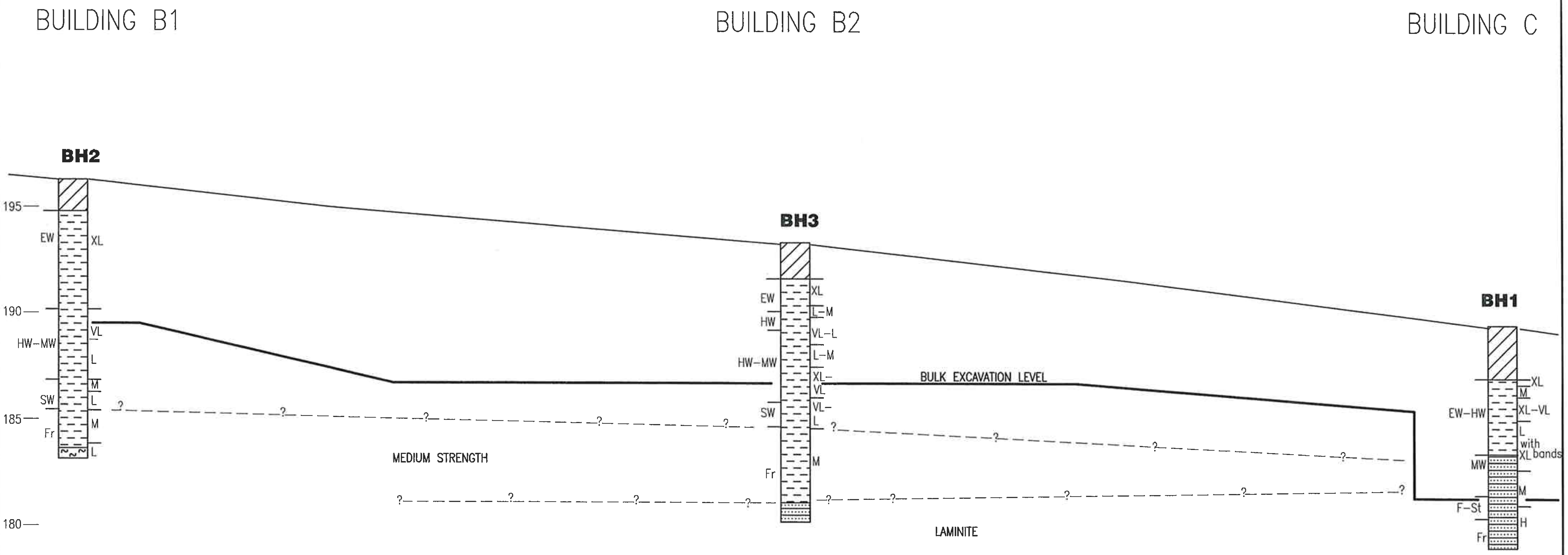
Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville Cairns, Wollongong, Darwin

TITLE:

Geological Section A–A'
35 Water Street
WAHROONGA

CLIENT: Waterbrook Wahroonga Pty Ltd

DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 43905	OFFICE: SYDNEY
APPROVED BY: JBraybrooke		DATE: 30.5.2006	DRAWING No: 1



LEGEND

▨ CLAY
SHALE
LAMINITE
HORIZONTAL SHEARED ZONES

WEATHERING		STRENGTH	
EW	EXTREMELY WEATHERED	XL	EXTREMELY LOW
HW	HIGHLY WEATHERED	VL	VERY LOW
MW	MODERATELY WEATHERED	L	LOW
SW	SLIGHTLY WEATHERED	M	MEDIUM STRENGTH
FSt	FRESH STAINED	H	HIGH STRENGTH
Fr	FRACTURED		

NOTE: See Drawing 1 for location of Geological Section

SCALE
0 5 10 15 20m
1:200

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Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville Cairns, Wollongong, Darwin

TITLE:

Geological Section A-A'
35 Water Street
WAHROONGA

CLIENT: Waterbrook Wahroonga Pty Ltd
DRAWN BY: PSCH | SCALE: As shown | PROJECT No: 43905 | OFFICE: SYDNEY
APPROVED BY: | DATE: 30.5.2006 | DRAWING No: 2

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APPENDIX A
Notes Relating to this Report
Results of Field Work

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and Classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

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

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

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

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

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

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

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

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

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

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

  as 15, 30/40 mm.

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

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

Cone Penetrometer Testing and Interpretation

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

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

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

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

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

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

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

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

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

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

Hand Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Bore Logs

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

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

Ground Water

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

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

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

Engineering Reports

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

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

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

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

Site Inspection

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

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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

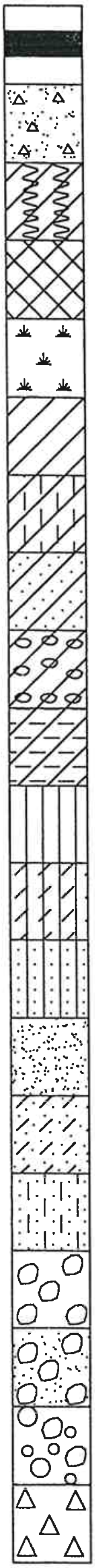
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

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

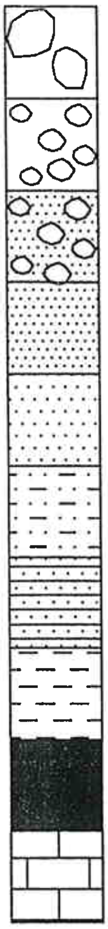


- BITUMINOUS CONCRETE
- CONCRETE
- TOPSOIL
- FILLING
- PEAT
- CLAY
- SILTY CLAY
- SANDY CLAY
- GRAVELLY CLAY
- SHALY CLAY
- SILT
- CLAYEY SILT
- SANDY SILT
- SAND
- CLAYEY SAND
- SILTY SAND
- GRAVEL
- SANDY GRAVEL
- COBBLES/BOULDERS
- TALUS

SEAMS

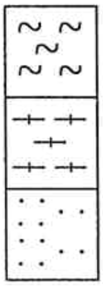
- SEAM >10mm
- SEAM <10mm

SEDIMENTARY ROCK



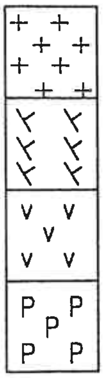
- BOULDER CONGLOMERATE
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE FINE GRAINED
- SANDSTONE COARSE GRAINED
- SILTSTONE
- LAMINITE
- MUDSTONE, CLAYSTONE, SHALE
- COAL
- LIMESTONE

METAMORPHIC ROCK



- SLATE, PHYLLITE, SCHIST
- GNEISS
- QUARTZITE

IGNEOUS ROCK



- GRANITE
- DOLERITE, BASALT
- TUFF
- PORPHYRY

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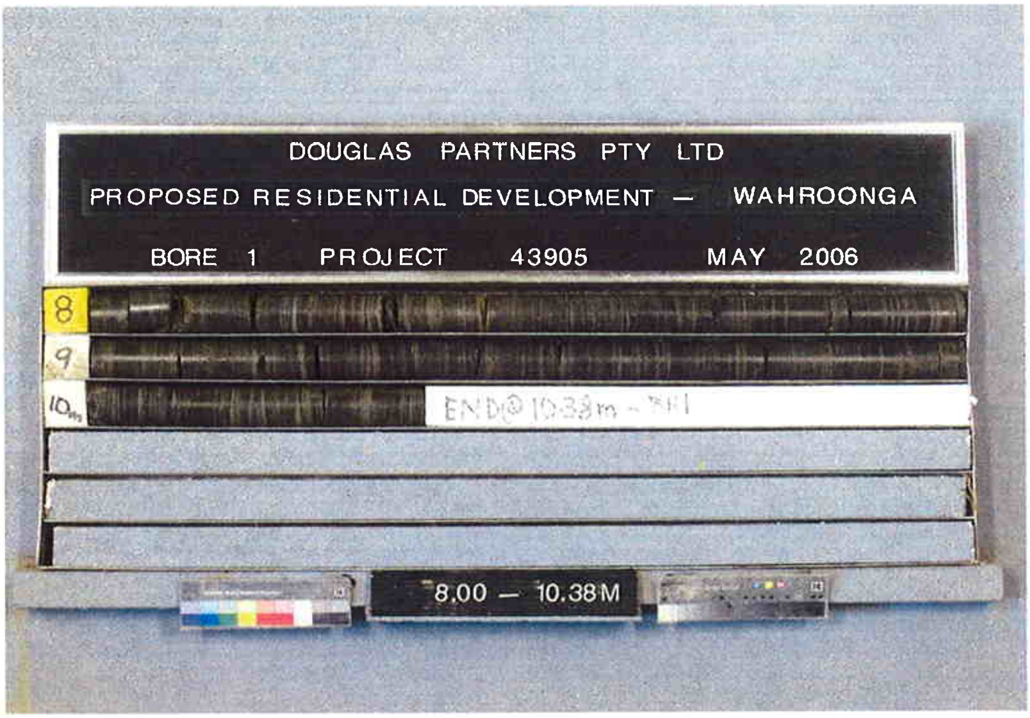

35 Water Street and 64 Billyard Street, Wahroonga	**Project 43905**	**June 2006**	

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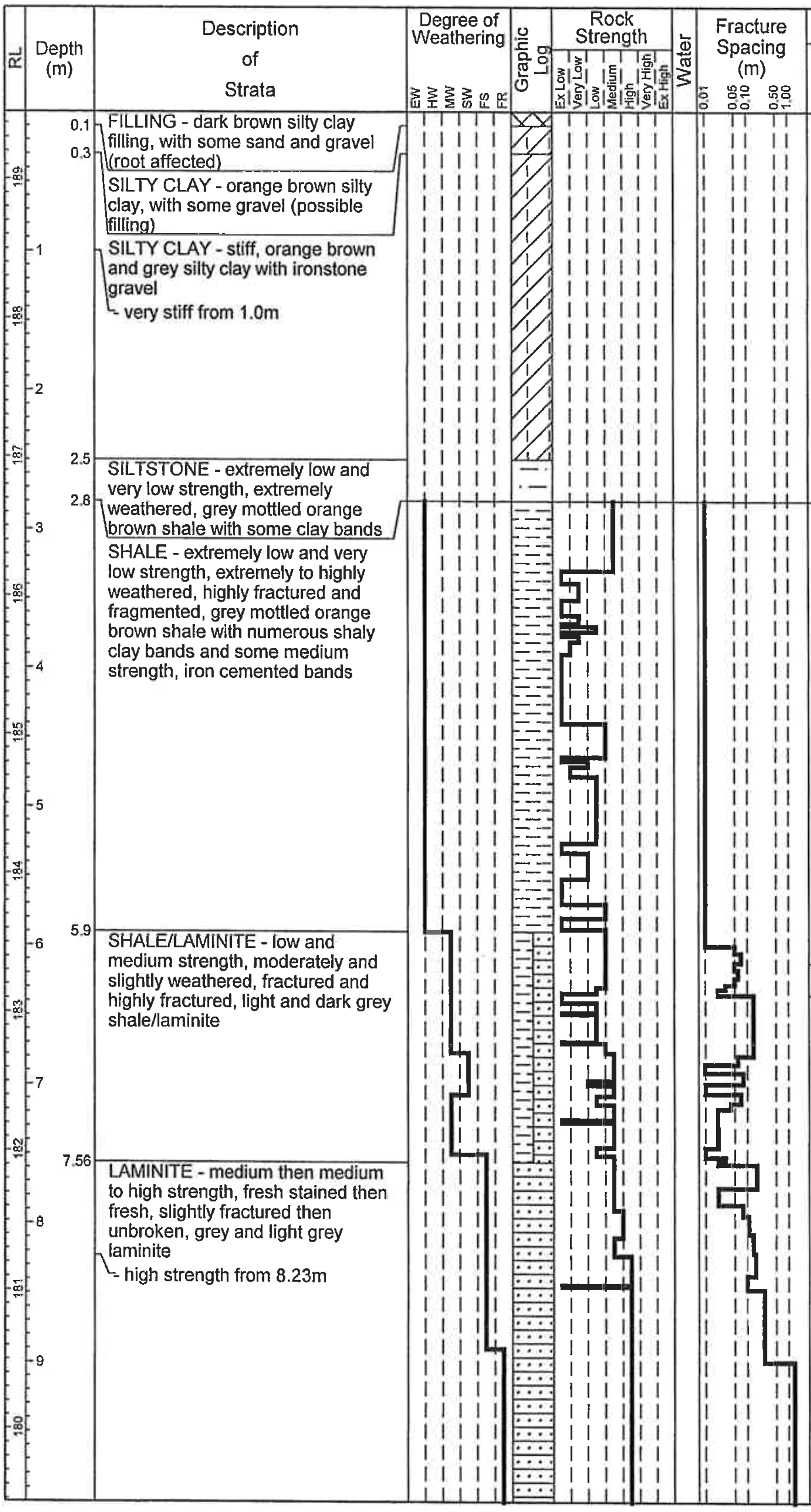
BOREHOLE LOG

CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 189.48
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 43905
DATE: 04 May 06
SHEET 1 **OF** 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	FILLING - dark brown silty clay filling, with some sand and gravel (root affected)		A			
0.3	SILTY CLAY - orange brown silty clay, with some gravel (possible filling)		A			
	SILTY CLAY - stiff, orange brown and grey silty clay with ironstone gravel - very stiff from 1.0m		A			
			S			6,8,11 N = 19
2.5	SILTSTONE - extremely low and very low strength, extremely weathered, grey mottled orange brown shale with some clay bands		S			14,25/100mm refusal
2.8	SHALE - extremely low and very low strength, extremely to highly weathered, highly fractured and fragmented, grey mottled orange brown shale with numerous shaly clay bands and some medium strength, iron cemented bands	- alternate bands of medium strength shale/clay from 2.8-3.3m - highly fractured & fragmented from 2.8-6.0m	C	100	0	
			C	100	0	
			C	100	0	
5.9	SHALE/LAMINITE - low and medium strength, moderately and slightly weathered, fractured and highly fractured, light and dark grey shale/laminite	6m: J85°- 90° irregular 6.12m: 20°- 25° undulating 6.35m: B0°- 5° with up to 70mm clay seam	C	100	0	PL(A) = 0.4MPa
		7.17m: J85°- 90°				PL(A) = 0.7MPa
7.56	LAMINITE - medium then medium to high strength, fresh stained then fresh, slightly fractured then unbroken, grey and light grey laminite - high strength from 8.23m	7.49m: J40°- 45° irregular, stepped				PL(A) = 0.6MPa
		8.07m: J35°- 40°	C	100	68	
		8.44m: B0°- 5° with 4-5mm clay 8.73m: B0°- 5° with clay smear				PL(A) = 1.7MPa
			C	100	100	

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** MMK **CASING:** GL to 2.5m
TYPE OF BORING: Solid flight auger (TC bit) to 2.5m; Rotary to 2.8m; NMLC-Coring to 10.38m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: 5-10% water loss between 6.0 and 7.0m

SAMPLING & IN SITU TESTING LEGEND

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

CHECKED
Initials: MS
Date: 8/6/06

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BOREHOLE LOG

CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 189.48
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 43905
DATE: 04 May 06
SHEET 2 **OF** 2

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
									C	100	100	PL(A) = 1.3MPa
	10.38	Bore discontinued at 10.38m										
179												
	11											
178												
	12											
177												
	13											
176												
	14											
175												
	15											
174												
	16											
173												
	17											
172												
	18											
171												
	19											
170												

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** MMK **CASING:** GL to 2.5m
TYPE OF BORING: Solid flight auger (TC bit) to 2.5m; Rotary to 2.8m; NMLC-Coring to 10.38m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: 5-10% water loss between 6.0 and 7.0m

SAMPLING & IN SITU TESTING LEGEND

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

CHECKED
Initials:
Date: 8/6/06

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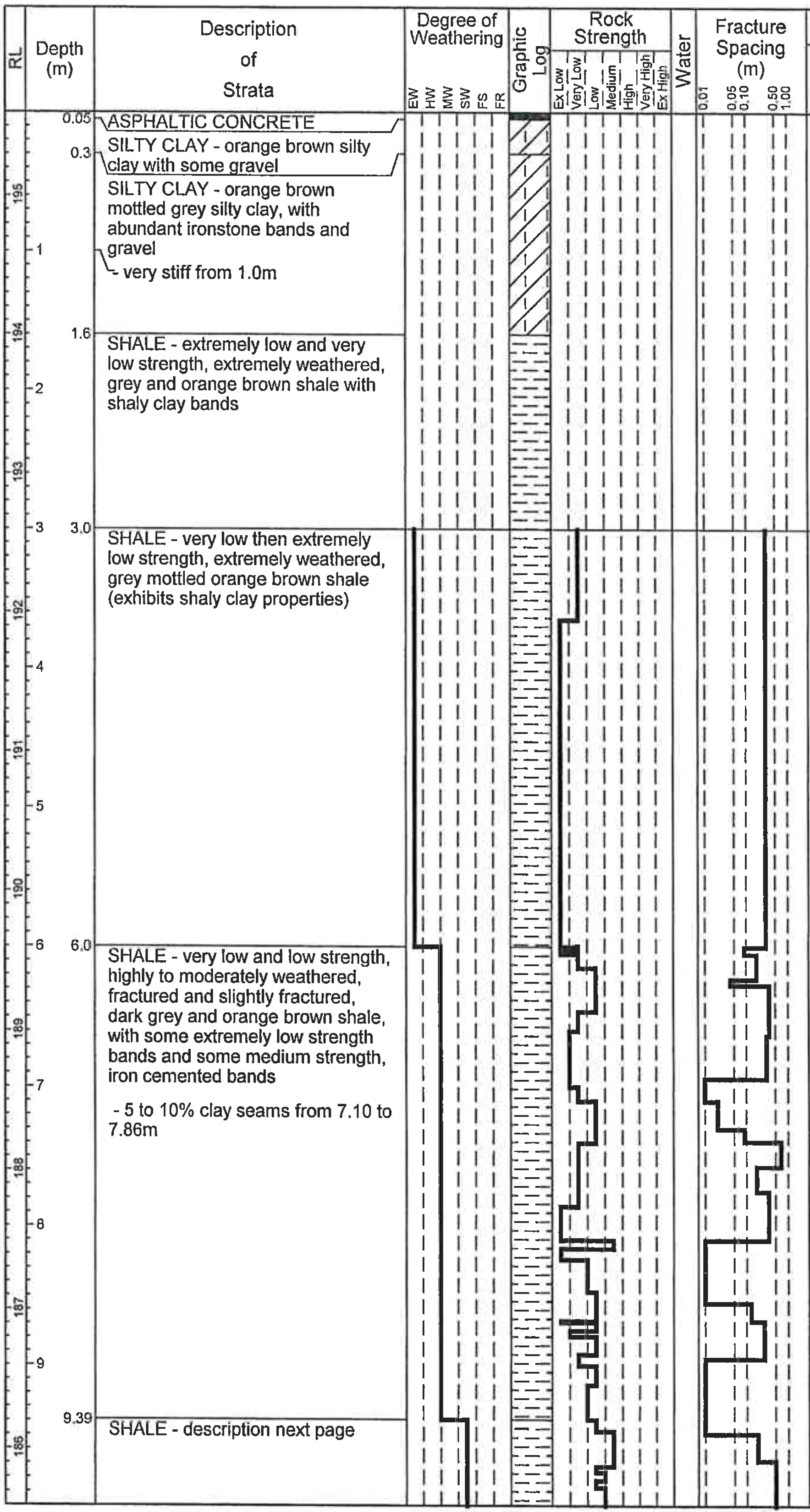
DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT — WAHROONGA
BORE 2 PROJECT 43905 MAY 2006
3
4
5
6
7
8
3.00 — 9.00M

BOREHOLE LOG

CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 195.6
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 43905
DATE: 05 May 06
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE					
0.3	SILTY CLAY - orange brown silty clay with some gravel		A			
	SILTY CLAY - orange brown mottled grey silty clay, with abundant ironstone bands and gravel - very stiff from 1.0m		A A S			6,13,18 N = 31
1.6	SHALE - extremely low and very low strength, extremely weathered, grey and orange brown shale with shaly clay bands	Note: Unless otherwise stated, rock is fractured along rough planar ironstained bedding planes & joints dipping at 0°- 10°	S			25/110mm refusal
3.0	SHALE - very low then extremely low strength, extremely weathered, grey mottled orange brown shale (exhibits shaly clay properties)		C C	100 100	0 0	
6.0	SHALE - very low and low strength, highly to moderately weathered, fractured and slightly fractured, dark grey and orange brown shale, with some extremely low strength bands and some medium strength, iron cemented bands - 5 to 10% clay seams from 7.10 to 7.86m	6.27m: J30°- 35° 6.94m: J45° 7.1m: J85°- 90° undulating 7.3m: J40°- 45° 8.1m: J40°- 45° -fragmented from 8.10-8.55m along numerous healed & partially healed joints dipping at 45°- 90° 8.68m: B0°- 5° with 4-5mm clay - fragmented from 8.95-9.49m due to numerous bedding planes & very low strength of rock	C C	100 100	15 66	PL(A) = 0.2MPa PL(A) = 0.2MPa PL(A) = 0.5MPa
9.39	SHALE - description next page					

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** MMK **CASING:** GL to 2.5m
TYPE OF BORING: Solid flight auger (TC bit) to 2.5m; Rotary to 3.0m; NMLC-Coring to 12.95m
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: 8/6/06

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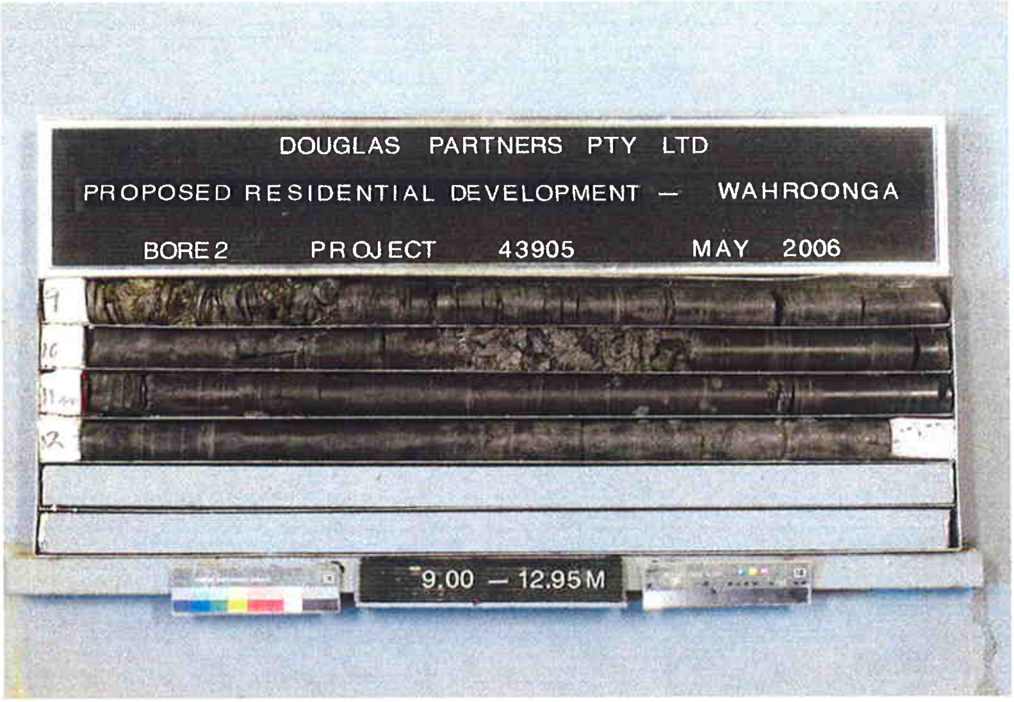
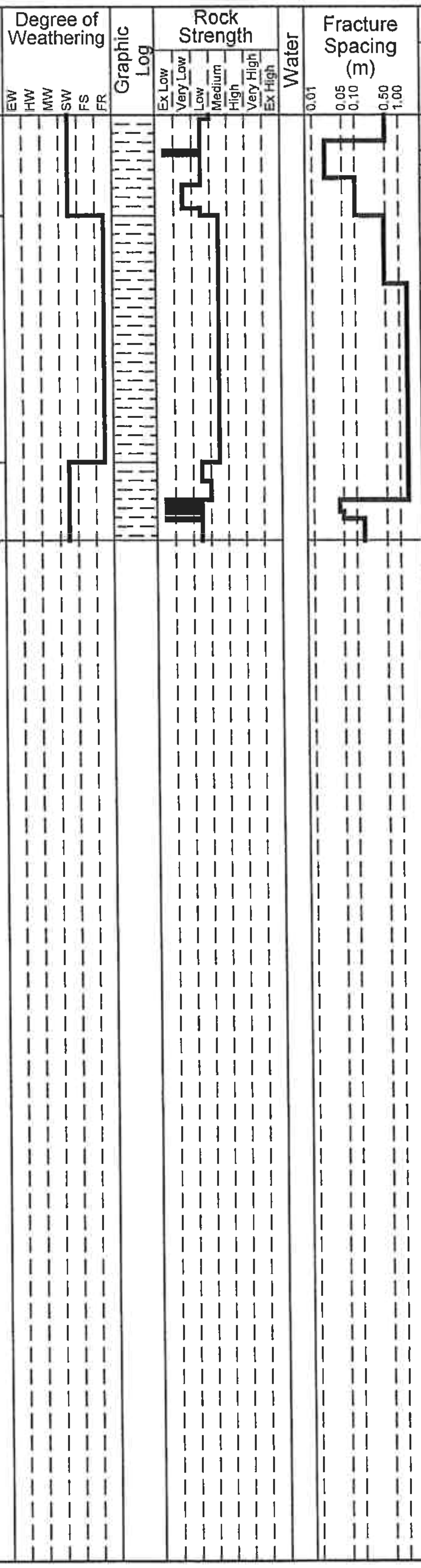

DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT — WAHROONGA
BORE 2 PROJECT 43905 MAY 2006
9
10
11
12
9.00 — 12.95 M

BOREHOLE LOG

CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 195.6
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 43905
DATE: 05 May 06
SHEET 2 **OF** 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
10.0	SHALE - low and medium strength, slightly weathered, fractured and slightly fractured, dark grey/black shale, with some extremely low strength bands	10.18m: J75°- 80° smooth 10.25m: J65°- 70° irregular clay coating 10.35m: J55°- 90° undulating 10.44m: fragmented from 10.44-10.70m (possible crush zone)	C	100	66	PL(A) = 0.4MPa
10.7	SHALE - medium strength, fresh, slightly fractured and unbroken, dark grey/black shale with some siltstone laminations	11.17m: J75°- 80° clean				PL(A) = 0.5MPa
			C	100	92	PL(A) = 0.5MPa
12.41	SHALE - low strength, slightly weathered, fractured and slightly fractured, dark grey black shale with 10-15% extremely low strength, clayey bands	12.67m: B0° with 5-7mm clay 12.71m: B0° with 5-7mm clay 12.75m: B0° with 5-7mm clay 12.8m: B0° with 5-7mm clay				PL(A) = 0.3MPa
12.95	Bore discontinued at 12.95m					

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** MMK **CASING:** GL to 2.5m
TYPE OF BORING: Solid flight auger (TC bit) to 2.5m; Rotary to 3.0m; NMLC-Coring to 12.95m
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: JB
Date: 8/6/06

Douglas Partners
Geotechnics · Environment · Groundwater

35 Water Street and 64 Billyard Street, Wahroonga	**Project** **43905**	**June** **2006**	

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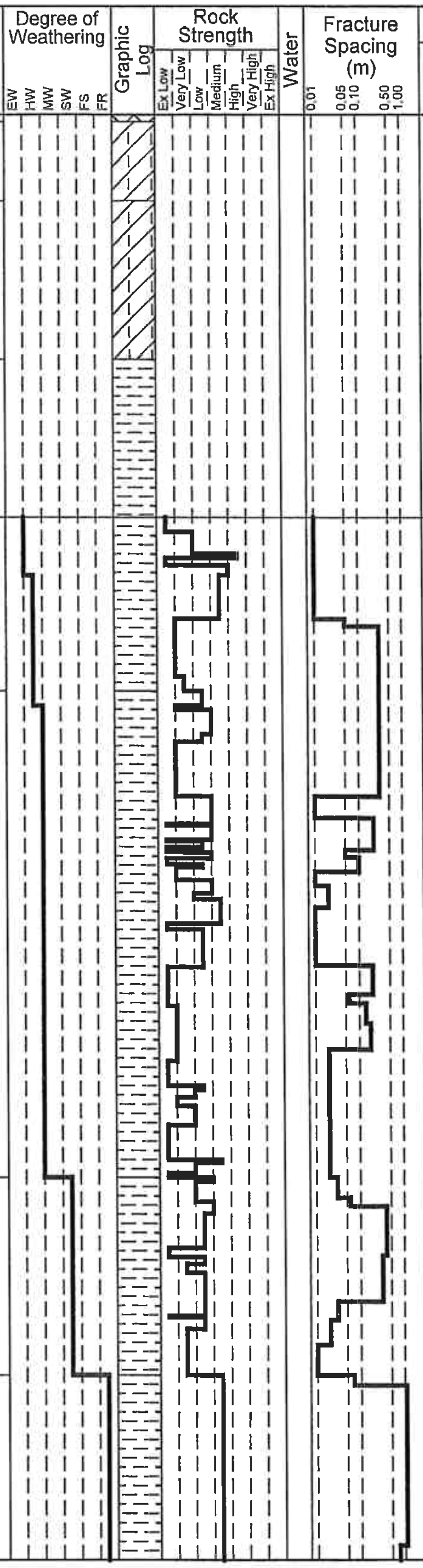
BOREHOLE LOG

CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 193.38
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 43905
DATE: 05 May 06
SHEET 1 OF 2

RL	Depth (m)	Description of Strata
193	0.05	FILLING - dark brown silty clay topsoil filling (root affected)
		SILTY CLAY - orange brown silty clay with ironstone gravel (possible filling)
192	0.6	SILTY CLAY - stiff, orange brown mottled grey silty clay with ironstone gravel - very stiff from 1.0m - grading to shaly clay from 1.3m
191	1.7	SHALE - extremely low to very low strength, grey and orange brown shale with shaly clay bands
190	2.8	SHALE - extremely low and very low strength, highly fractured and fragmented, extremely to highly weathered, grey and orange brown shale with some iron cemented bands
189	4.0	SHALE - very low and low strength, highly to moderately weathered, fractured, light and dark grey shale, with numerous extremely low strength clayey bands - clay band from 5.91 to 6.18m
186	7.36	SHALE - low strength, slightly weathered, slightly fractured and unbroken, dark grey/black and light grey shale with 10%-15% fine grained sandstone laminations
185	8.73	SHALE - medium strength, fresh, slightly fractured and unbroken, dark grey/black shale



Discontinuities

B - Bedding J - Joint
S - Shear D - Drill Break

Note: Unless otherwise stated, rock is fractured along rough, planar, ironstained bedding planes & joints dipping at 0°- 10°

3.55m: J75°- 80°

4.73m: J85°- 90° undulating with clay coating

7.56m: B0°- 5° with 2-3mm clay

7.9m: B0°- 5° with 2-3mm clay

8.22m: J70°

8.35m: J70°

- fragmented from 8.52-8.73m due to highly fractured and very low strength nature of rock

8.8m: B0°- 5° with clay smear

9.9m: J70°

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			5,8,11 N = 19
A			
S			25/100mm refusal
C	100	0	
C	100	0	PL(A) = 0.2MPa
C	100	10	PL(A) = 0.3MPa
C	100	61	PL(A) = 0.2MPa PL(A) = 0.3MPa PL(A) = 0.4MPa

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** MMK **CASING:** GL to 2.5m

TYPE OF BORING: Solid flight auger (TC bit) to 2.5m; Rotary to 2.8m; NMLC-Coring to 13.0m

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:
Date: 8/6/06

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DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT — WAHROONGA
BORE 3 PROJECT 43905 MAY 2006
7m
8m
9m
10
11m
12m
7.00 — 13.00 M

BOREHOLE LOG

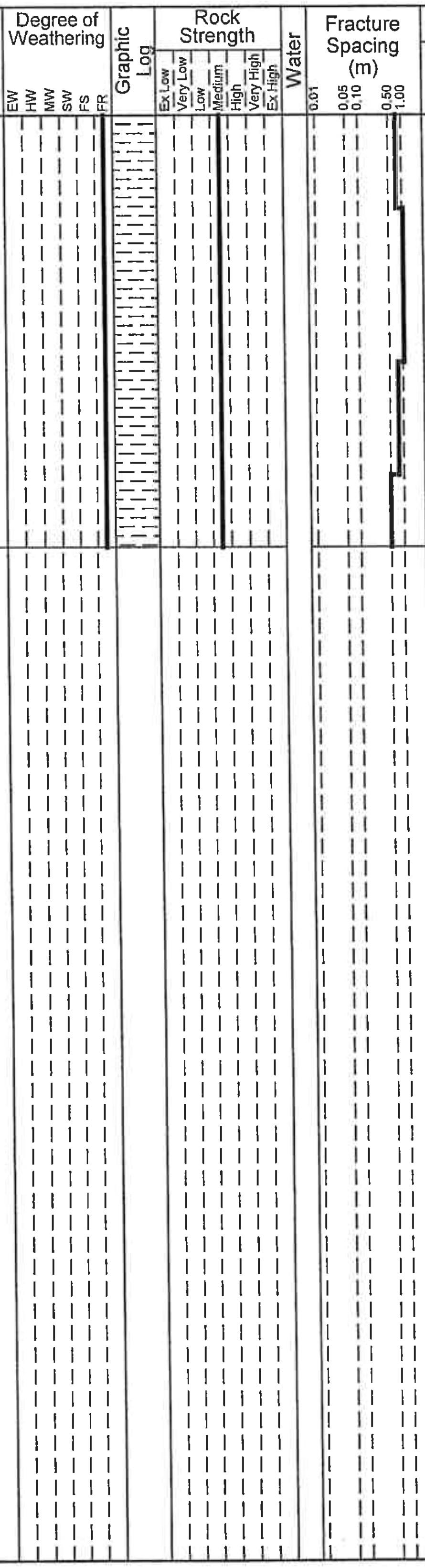
CLIENT: Murlan Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 35 Water Street, Wahroonga

SURFACE LEVEL: 193.38
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 43905
DATE: 05 May 06
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
183		SHALE - medium strength, fresh, slightly fractured and unbroken, dark grey/black shale *(continued)*		C	100	61	
182	11			C	100	99	PL(A) = 0.8MPa
181	12	- grading to laminite from 12.10m					PL(A) = 0.7MPa
180	13 13.0	Bore discontinued at 13.0m					
179	14						
178	15						
177	16						
176	17						
175	18						
174	19						

Discontinuities: B - Bedding J - Joint S - Shear D - Drill Break



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CHECKED
Initials:
Date: 8/6/06

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