Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Residential Development
871 – 877 Pacific Highway
Chatswood

Prepared for
Megland Group Pty Ltd

Project 84722
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Integrated Practical Solutions

Douglas Partners
Geotechnics / Environment / Groundwater

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
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Reviewer		14 April 2015

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Report on Geotechnical Investigation
Proposed Residential Development
871 – 877 Pacific Highway, Chatswood

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed residential development at 871 – 877 Pacific Highway, Chatswood. The investigation was commissioned by Mr Tomy Chan of PBD Architects on behalf of Megland Group Pty Ltd, and was undertaken in general accordance with DP's proposal SYD150137 dated 9 February 2015.

It is understood that the development will include the construction of a six to seven storey residential unit building with a two level basement car park.

The purpose of the geotechnical investigation was to assess:

- the subsurface soil and rock profile in the vicinity of the proposed works;
- excavation conditions;
- excavation support requirements;
- groundwater levels; and
- suitable foundation types, founding levels and allowable bearing pressures.

The investigation included the drilling of two boreholes and laboratory testing of selected samples. The results of the field work are presented in this report, together with comments addressing the issues listed above.

A contamination assessment was undertaken at the same time as the geotechnical investigation and is reported separately.

2. Site Description and Regional Geology

The site is irregular in plan and covers an area of approximately 1400 m^2. It is bounded by a service station to the north, Wilson Street to the south, a rail corridor to the east and the Pacific Highway to the west. The site is relatively flat, with surface levels in the vicinity of RL 108 m relative to the Australian Height Datum (AHD). The eastern boundary of the site is supported by a retaining wall approximately 6 m high, above the rail tracks. At the time of investigation the site was occupied by a two storey mixed residential and office building.

The Geological Survey of NSW 1:100,000 Geological Series Sheet 9130 (Sydney) indicates that the site is underlain by Ashfield Shale, which typically comprises black to dark grey shale and laminite. The corresponding Soil Landscape Series Sheet, by the former NSW Department of Land and Water Conservation, indicates that bedrock at the site is overlain by erosional soils of the Glenorie soil association, typically comprising red and yellow, moderately reactive clay soils. The field work for this investigation confirmed the presence of clay and shale within the subsurface profile.

3. Field Work Methods

The field work was carried out over two days (18 and 19 February 2015) and included:

- Setting out and survey levelling of two boreholes (BH1 and BH2). The locations of the boreholes are shown on Drawing 1 in Appendix B;
- Electronic scanning for buried services at the borehole locations;
- Drilling of the two boreholes using a small truck-mounted drilling rig with solid flight augers and rotary (wash boring) methods through the soils, and NMLC diamond coring equipment in the bedrock. The bores were drilled to depths of 8.25 m and 7.8 m;
- Standard penetration tests (SPTs) at regular intervals within the soil;
- Discrete disturbed sampling of soil through the soil profile;
- Retrieval of continuous rock core samples through the rock profile; and
- Installation of a temporary standpipe well in both of the boreholes to allow for measurement of groundwater levels. The wells were installed with a Gatic cover made flush with the existing ground surface.

4. Field Work Results

The borehole logs are included in Appendix C, together with notes defining classification methods and terms used to describe the soil and rock.

Based on the results of the investigation, the subsurface profile can be summarised as follows:

- FILLING – grey and brown sand and gravel filling to depths of between 0.6 m and 0.7 m; overlying,
- CLAY – stiff, brown and red clay to depths of between 2.5 m and 2.6 m; overlying
- SHALE – extremely low to very low strength, extremely to highly weathered, fractured, light grey and red brown shale to depths of between 5.5 m and 6.0 m; overlying,
- SHALE – very low and low strength, highly to moderately weathered, fractured and slightly fractured, grey-brown shale, to a depth of 7.2 m in BH1 and to the base of BH2; overlying,
- SHALE – medium strength, fresh, slightly fractured grey shale, encountered in BH1 only.

Table 1 summarises the levels at which different materials were encountered in the boreholes.

Table 1: Summary of Material Strata Levels

Stratum	RL of Top of Stratum (m, AHD)	
	BH1	BH2
Ground Surface / Filling	107.9	107.9
Clay	107.2	107.3
Extremely Low to Very Low Strength Shale	105.3	105.4
Very Low and Low Strength Shale	101.9	102.4
Medium Strength Shale	100.7	NE
Base of Borehole	99.7	100.1

Notes: NE = Not encountered

An interpreted geotechnical cross-section of the site is presented in Drawing 2 in Appendix B. Anticipated bulk excavation levels are superimposed onto the cross-section for reference.

Groundwater was not encountered in either of the boreholes whilst auger drilling. The use of water as a drilling fluid prevented groundwater observations during rotary and core drilling operations. An attempt to measure the water levels in the standpipe wells was made on 17 March 2015, but only minor seepage was observed in the base of the wells.

5. Laboratory Testing

Seven samples selected from the better quality rock core were tested for axial point load strength index (Is_{50}). The results ranged from 0.2 MPa to 1.3 MPa which correspond to low strength and high strength rock, respectively.

6. Proposed Development

It is understood that the proposed development will include a six to seven storey residential unit building with a two level basement car park. Excavation of about 6 m depth is anticipated for the basement car park. Although foundation loadings are not known at this stage, column loads are anticipated to be in the order of 3000 - 4000 kN based on experience with similar developments.

7. Comments

7.1 Excavation Conditions

Excavation for the basement levels is expected to be required in filling, clay and shale of extremely low to very low strength. This should be readily achievable using conventional earthmoving equipment such as hydraulic excavators with bucket attachments. It should be noted that any off-site disposal of spoil will generally require assessment for re-use or classification in accordance with current EPA guidelines.

7.2 Excavation Support

7.2.1 Batter Slopes

Where space permits, temporary and permanent batter slopes may be used without the need for any retaining structures to support the side walls of the excavation. The recommended maximum temporary and permanent batter slopes for the expected subsurface materials on the site are given in Table 2. These may be adopted provided there is no surcharge from stockpiled materials, adjacent buildings, vehicular or other loads to a setback distance of at least the excavation depth behind the crest of the excavation. The suggested batter slopes assume 3 m excavation depth. Deeper excavations will need to incorporate intermediate benches.

Table 2: Recommended Maximum Batter Slopes for Excavations up to 3 m Deep

Material	Maximum Batter Slope Ratio (H:V)	
	Short Term (Temporary)	Long Term (Permanent)*
Filling and Stiff Clay	1:1	2:1
Extremely Low to Very Low Strength Shale	1:1	1.5:1

Note: * Will need to incorporate erosion protection

7.2.2 Retaining Structures

Where batter slopes are not considered feasible, temporary and permanent lateral support will be required during excavation and as part of the final construction.

Soldier piles with infill reinforced shotcrete panels are commonly used to support excavations in clay and shale, and could be adopted on this site. The soldier piles would generally be spaced at about 2 m to 3 m centres and should be founded at least two pile diameters below the lowest excavation level (both bulk and detailed) adjacent to the pile location. Shotcreting will be needed over the full excavation depth and should be undertaken in approximately 2.5 m drops as excavation proceeds in order to reduce the risk of local slippages. Temporary ground anchors may be required depending on the depth of excavation and the amount of lateral deformation that can be tolerated. Anchoring will not be possible along the railway boundary and internal propping/bracing should be used where necessary.

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the surrounding soil and rock. Table 3 outlines material parameters that could be used for the preliminary design of excavation support structures.

Table 3: Material and Strength Parameters for Excavation Support Structures

Material	Bulk Density (kN/m^3)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	Ultimate Passive Earth Pressure (kPa)
Filling	20	0.4	0.6	-
Stiff Clay	20	0.3	0.45	-
Extremely Low to Very Low Strength Shale	21	0.2[1]	0.3[1]	300[2]
Very Low to Low Strength Shale	22	0.15[1]	0.25[1]	1000[2]
Medium Strength Shale	23	0[1]	0[1]	3000[2]

Notes: [1] Unless unfavourably jointed; [2] Only below bulk/detailed excavation level and where jointing is favourable

Additional pressure should be allowed for where surcharging occurs, either from adjacent buildings, vehicular or other loads. Hydrostatic pressure acting on the shoring walls should also be included where adequate drainage is not provided behind the full height of the walls.

Where the depth of excavation exceeds 4 m, the shoring may need to incorporate more than one row of anchors. The lateral pressure distribution on a multi-anchored or braced wall is complex and for preliminary design purposes a uniform rectangular earth pressure of 6H (kPa) could be assumed, where H is the height of the retained material in metres. It is recommended that a software package such as WALLAP, FLAC or PLAXIS be used to analyse the shoring system to refine the preliminary design prior to commencement of construction.

7.2.3 Ground Anchors

Where necessary, the use of declined tie-back (ground) anchors is suggested for the lateral restraint of the perimeter piled walls. Such ground anchors should be declined below the horizontal to allow anchorage into the stronger bedrock materials at depth. The design of temporary ground anchors for the support of piled wall systems may be carried out using the ultimate bond stresses at the grout-rock interface given in Table 4.

Table 4: Ultimate Bond Stresses for Anchor Design

Material	Ultimate Bond Stress (kPa)
Stiff Clay	20
Extremely Low to Very Low Strength Shale	50
Very Low to Low Strength Shale	200
Medium Strength Shale	700

Based on a structural failure classification of Class B (refer Table 1 of AS 4678:2002), a factor of safety of at least 2.5 should be applied to the ultimate bond stresses given in Table 4, to obtain the allowable bond stresses.

Ground anchors should be designed to have a free length equal to their height above the base of the excavation and have a minimum 3 m bond length. After installation they should be proof loaded to 125% of the design working load and locked off at no higher than 80% 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 in Table 4 assume that the anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical engineer or engineering geologist may allow increased bond stresses to be adopted during construction, subject to testing.

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 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 will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services and other subsurface structures during anchor installation.

7.3 Groundwater

Based on the regional geology and topography of the site, the groundwater table is expected to be well below the bedrock surface and as such, significant groundwater inflows into the excavation are not anticipated. There may, however, be some flows along the top of bedrock, as well as through joints and bedding planes within the rock, particularly after rainfall. Any seepage into the excavation should be relatively minor and is expected to be controllable by periodic pumping from sumps in the excavation. The permanent drainage system for the site should allow for removal of seepage and low flows from joints in the basement area.

Based on previous experience with groundwater derived from the Ashfield Shale formation, iron oxides will tend to precipitate from the groundwater upon exposure to air as a red-brown gelatinous sludge. It is normally necessary to incorporate provision for regular flushing and cleaning in the maintenance and design of the sub-floor drainage system.

7.4 Foundations

7.4.1 Shallow Footings

The proposed bulk excavation works are expected to expose extremely low to very low strength shale, or possibly very low to low strength shale. Shallow pad or strip footings within the excavation could be designed on the basis of an allowable end bearing pressure of 700 kPa in extremely low to very low strength shale or 1000 kPa in the very low to low strength shale.

Settlement of a shallow footing is dependent on the loads applied to the footing and the foundation conditions below the footing. The total settlement of a spread footing designed using the above parameters is expected to be less than 1% of the footing width upon application of the design load. Differential settlements between footings would be approximately half of this value.

All spread footings should be inspected by an experienced geotechnical engineer to check the adequacy of the foundation material.

7.4.2 Piles

Where the allowable end bearing pressures for shallow footings provided in Section 7.4.1 are inadequate to support structural loads, bored piles could be drilled down to a stronger bearing stratum. Shoring piles could also be used to support vertical loads, provided that they are founded at an appropriate depth below the basement excavation and below the zone of influence of the retaining wall along the eastern boundary. Piles could be proportioned on the basis of the design parameters provided in Table 5.

Table 5: Pile Design Parameters

Material Description	Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion* (kPa)
Very Low to Low Strength Shale	1000	150
Medium Strength Shale	3500	350

Note: * Provided adequate socket roughness is achieved

The settlement of a pile is dependent on the loads applied to the pile and the foundation conditions in the socket zone and below the pile toe. The total settlement of a bored pile designed using the parameters in Table 5 should be less than a few millimetres upon application of the design load.

All bored piles should be inspected by an experienced geotechnical engineer during construction to check the adequacy of the foundation material and to check the socket cleanliness and roughness.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 871 - 877 Pacific Highway, Chatswood in accordance with DP's proposal SYD150137 dated 9 February 2015, and acceptance received from Mr Tomy Chan of PBD Architects on behalf of Megland Group Pty Ltd dated 13 February 2015. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Megland Group Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the subsurface conditions on the site only at the specific borehole locations, and then only to the depths investigated and at the time the work was carried out. Subsurface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the borehole locations. The advice may also be limited by site accessibility.

This report must be read in conjunction with all of the appended documentation and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

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Appendix A

About this Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations 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.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report 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 or excavation. 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 and test pits 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 or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations 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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report 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. DP 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 and environmental 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.

Sampling Methods

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Sampling

Sampling is carried out during drilling or test pitting 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 it to obtain 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.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally 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 samples.

Continuous Spiral Flight Augers

The borehole 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 sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils 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 before the full penetration depth, 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 SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. 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 types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

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Description and Classification Methods

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

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Rock Descriptions

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and loner sections
Unbroken	Core lengths mostly > 1000 mm

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

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

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core Drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

July 2010

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

■ Asphalt


Road base

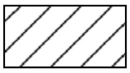

Concrete


Filling

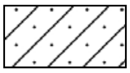
Soils


Topsoil

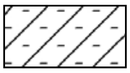

Peat


Clay

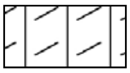

Silty clay

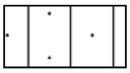

Sandy clay

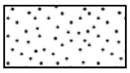

Gravelly clay

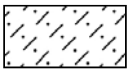

Shaly clay

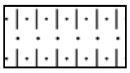

Silt


Clayey silt

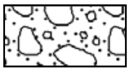

Sandy silt


Sand


Clayey sand


Silty sand


Gravel


Sandy gravel


Cobbles, boulders

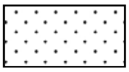

Talus

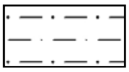
Sedimentary Rocks

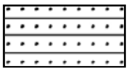

Boulder conglomerate

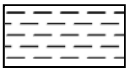

Conglomerate


Conglomeratic sandstone

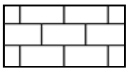

Sandstone


Siltstone

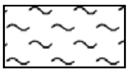

Laminite

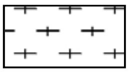

Mudstone, claystone, shale

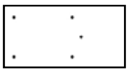
■ Coal


Limestone

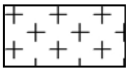
Metamorphic Rocks

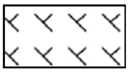

Slate, phyllite, schist

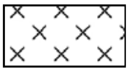

Gneiss


Quartzite

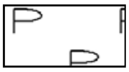
Igneous Rocks


Granite


Dolerite, basalt, andesite

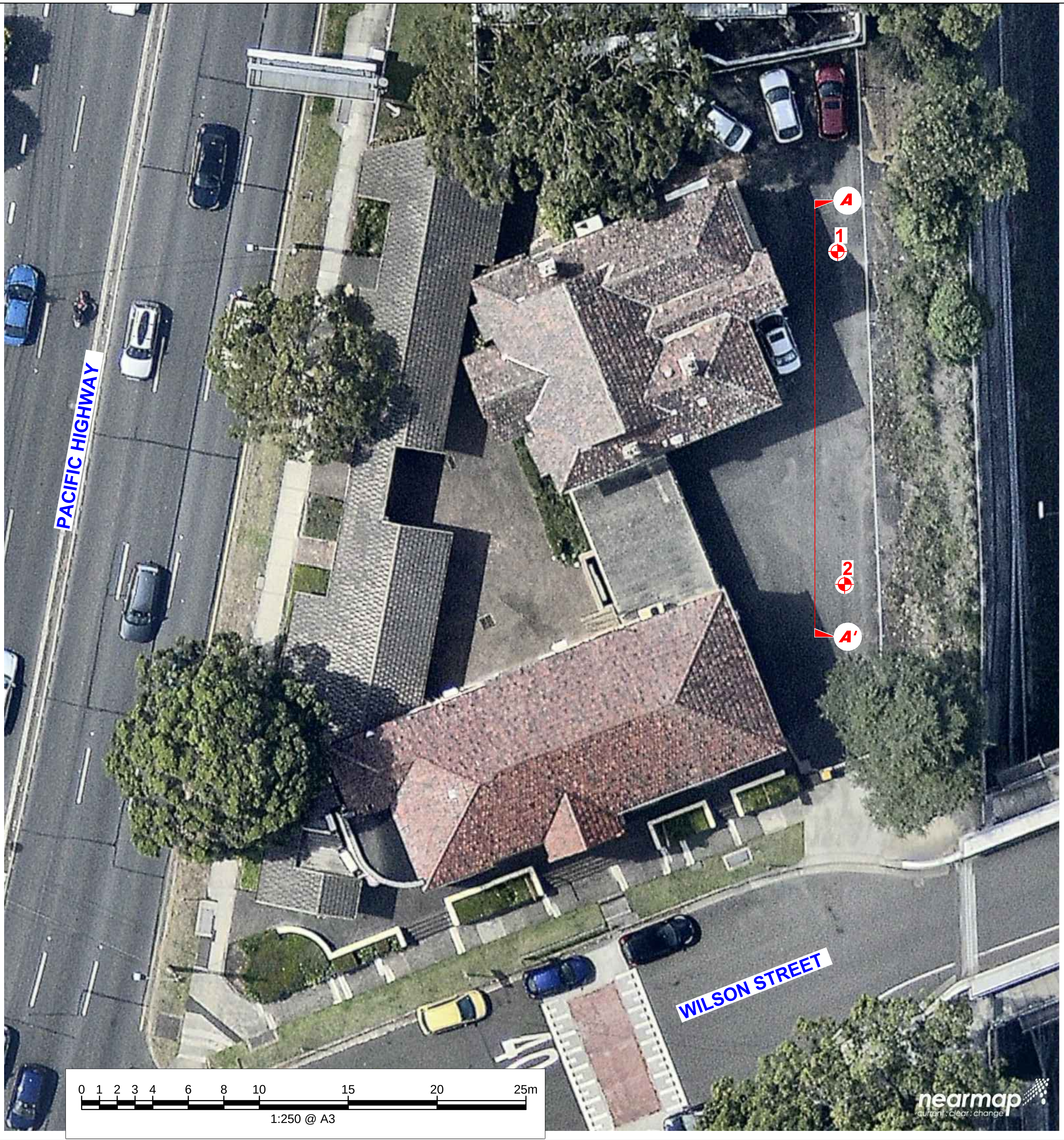

Dacite, epidote

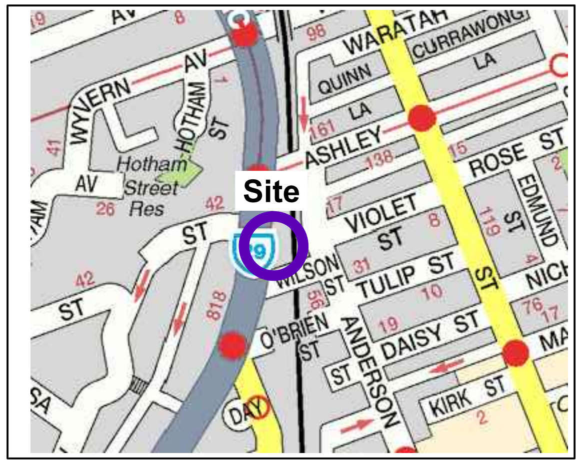

Tuff, breccia


Porphyry

Appendix B

Drawings





Locality Plan

NOTE:
1. Base drawing from Nearmap.com
2. Test locations are approximate only and are shown with reference to existing site features.

LEGEND

⊕ Borehole Location

A — A' Geotechnical Cross Section A-A'

P:\84722.00 CHATSWOOD, 871-877 Pacific Highway Investigation PMO\Drawings\84722-1.dwg, 19/03/2015 2:35:03 PM

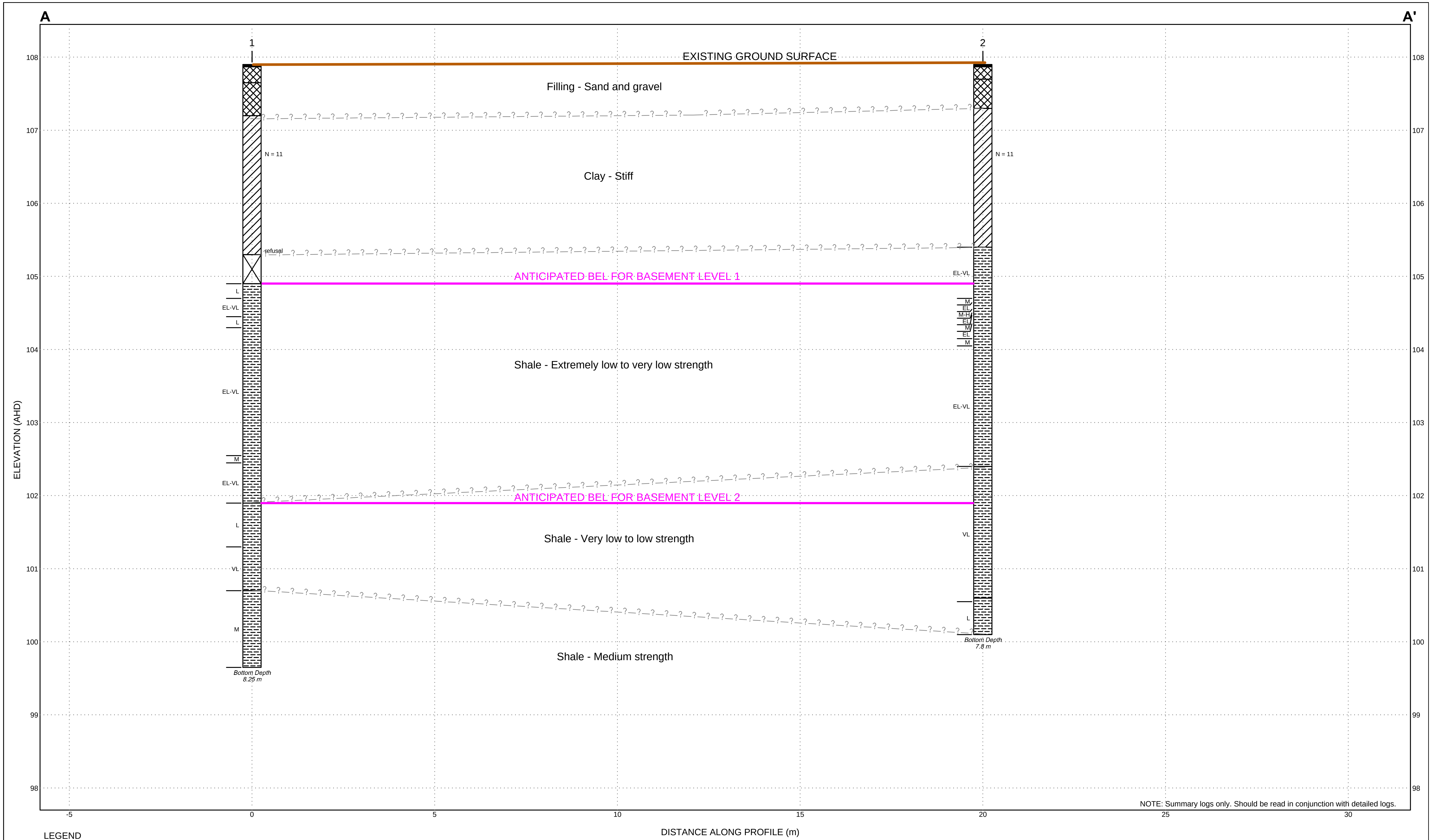
Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Megland Group Pty Ltd
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: 1:250 @ A3
DATE: 16.3.2015

TITLE: **Location of Boreholes**
Proposed Residential Development
871-877 Pacific Highway, CHATSWOOD

N

PROJECT No: 84722
DRAWING No: 1
REVISION: 0



NOTE: Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ■ Asphalt
- ▨ Clay
- ▩ Filling
- ▤ Shale

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

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

BEL - Bulk excavation level

0 2
Horizontal Scale (metres)

Vertical Exaggeration = 2.0

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Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Megland Group Pty Ltd
OFFICE: Sydney
DRAWN BY: AL/PSCH
SCALE: 1:100 (H) 1:50 (V) @ A3
DATE: 16.03.2015

TITLE: **Cross Section A-A'**
Proposed Residential Development
871-877 Pacific Highway, Chatswood

PROJECT No: 84722
DRAWING No: 2
REVISION: 0

Appendix C

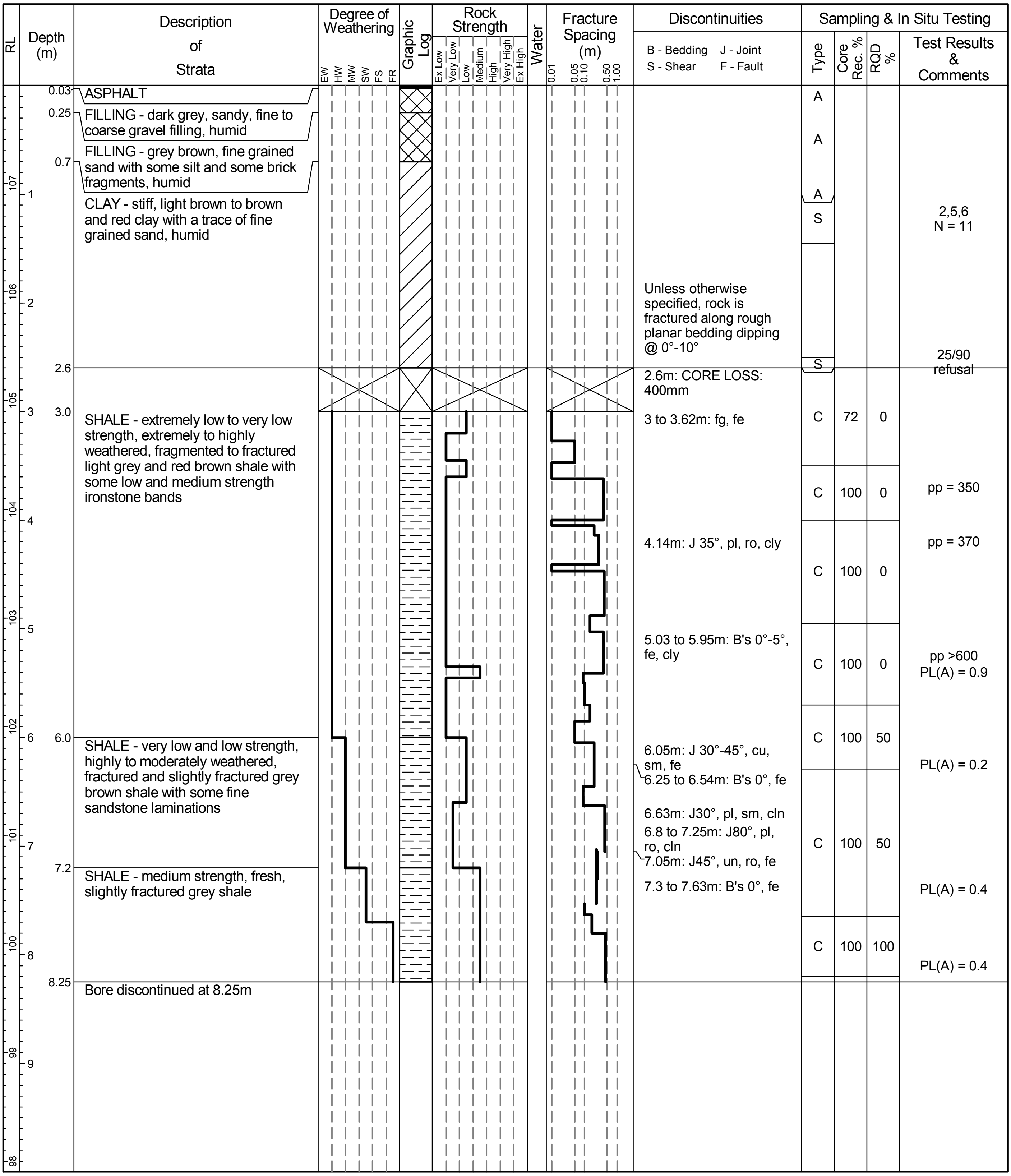
Field Work Results

BOREHOLE LOG

CLIENT: Megland Group Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 871-877 Pacific Highway, Chatswood

SURFACE LEVEL: 107.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 84722
DATE: 18/2/2015
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.03	ASPHALT		A			
0.25	FILLING - dark grey, sandy, fine to coarse gravel filling, humid		A			
0.7	FILLING - grey brown, fine grained sand with some silt and some brick fragments, humid		A			
	CLAY - stiff, light brown to brown and red clay with a trace of fine grained sand, humid		S			2,5,6 N = 11
		Unless otherwise specified, rock is fractured along rough planar bedding dipping @ 0°-10°	S			25/90 refusal
2.6		2.6m: CORE LOSS: 400mm				
3.0	SHALE - extremely low to very low strength, extremely to highly weathered, fragmented to fractured light grey and red brown shale with some low and medium strength ironstone bands	3 to 3.62m: fg, fe	C	72	0	
			C	100	0	pp = 350
		4.14m: J 35°, pl, ro, cly	C	100	0	pp = 370
		5.03 to 5.95m: B's 0°-5°, fe, cly	C	100	0	pp >600 PL(A) = 0.9
6.0	SHALE - very low and low strength, highly to moderately weathered, fractured and slightly fractured grey brown shale with some fine sandstone laminations	6.05m: J 30°-45°, cu, sm, fe 6.25 to 6.54m: B's 0°, fe	C	100	50	PL(A) = 0.2
		6.63m: J30°, pl, sm, cln 6.8 to 7.25m: J80°, pl, ro, cln 7.05m: J45°, un, ro, fe	C	100	50	
7.2	SHALE - medium strength, fresh, slightly fractured grey shale	7.3 to 7.63m: B's 0°, fe				PL(A) = 0.4
			C	100	100	PL(A) = 0.4
8.25	Bore discontinued at 8.25m					

RIG: DT 100 **DRILLER:** LC **LOGGED:** SI/MP **CASING:** HW to 2.6m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 2.6m; NMLC Coring to 8.25m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 8.25m; Screen 2.25 to 8.25m, Gravel 1.7 to 8.25m, Bentonite 1.3 to 1.7m, Backfill to GL with gatic cover

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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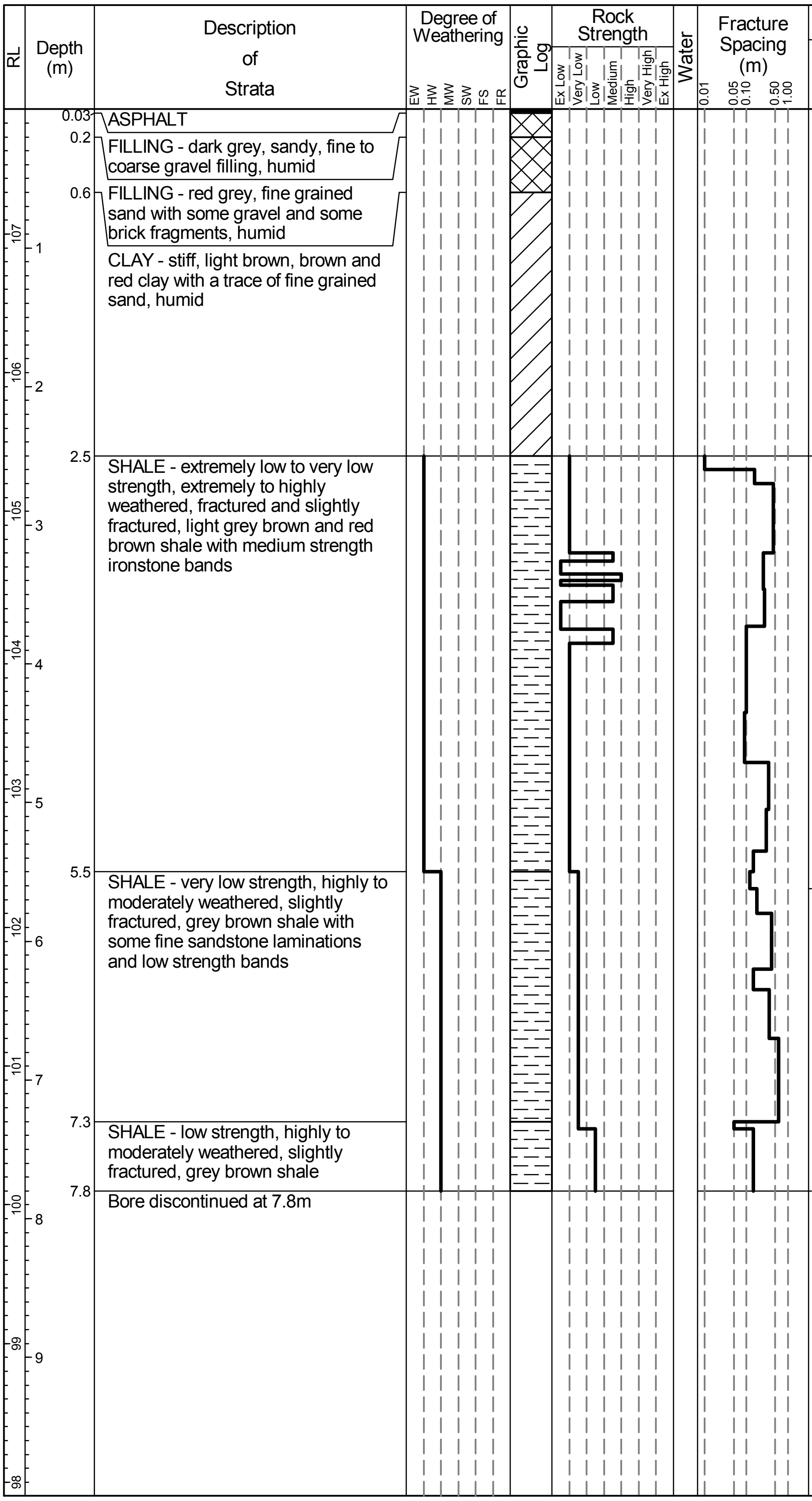
BOREHOLE LOG

CLIENT: Megland Group Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 871-877 Pacific Highway, Chatswood

SURFACE LEVEL: 107.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 84722
DATE: 19/2/2015
SHEET 1 OF 1

Depth (m)	Description of Strata
0.03	ASPHALT
0.2	FILLING - dark grey, sandy, fine to coarse gravel filling, humid
0.6	FILLING - red grey, fine grained sand with some gravel and some brick fragments, humid
	CLAY - stiff, light brown, brown and red clay with a trace of fine grained sand, humid
2.5	SHALE - extremely low to very low strength, extremely to highly weathered, fractured and slightly fractured, light grey brown and red brown shale with medium strength ironstone bands
5.5	SHALE - very low strength, highly to moderately weathered, slightly fractured, grey brown shale with some fine sandstone laminations and low strength bands
7.3	SHALE - low strength, highly to moderately weathered, slightly fractured, grey brown shale
7.8	Bore discontinued at 7.8m



Discontinuities

B - Bedding J - Joint
S - Shear F - Fault

Unless otherwise specified, rock is fractured along rough planar bedding dipping @ 0°-10°

2.76m: J35°,pl, ro, fe

3.2 to 4.65m: B's 0°-5°, fe, cly

4.71m: J45°, pl, ro, fe, cly

5.05 to 5.2m: cly

5.35 to 5.5m: J30°, pl, ro, fe

5.62m: J70°, un, ro, cly

5.8m: B0°, cly, 10mm

6.2m: J70°, un, ro, cly

6.35m: J30°, pl ro, cly

6.7 & 7.3m: B's, 0°, fe

7.35 to 7.65m: B's 0°-5°, fe

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			3,6,5 N = 11
C	100	0	
C	100	0	pp = 250 PL(A) = 1.3 pp = 300 PL(A) = 0.3
C	100	0	pp = 400
C	100	20	PL(A) = 0.2

RIG: DT 100 **DRILLER:** DL **LOGGED:** SI/MP **CASING:** HW to 1.1m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 2.5m; NMLC Coring to 7.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 7.8m; Screen 1.8 to 7.8m, Gravel 1.5 to 7.8m, Bentonite 1.0 to 1.5m, Backfill to GL with gatic cover

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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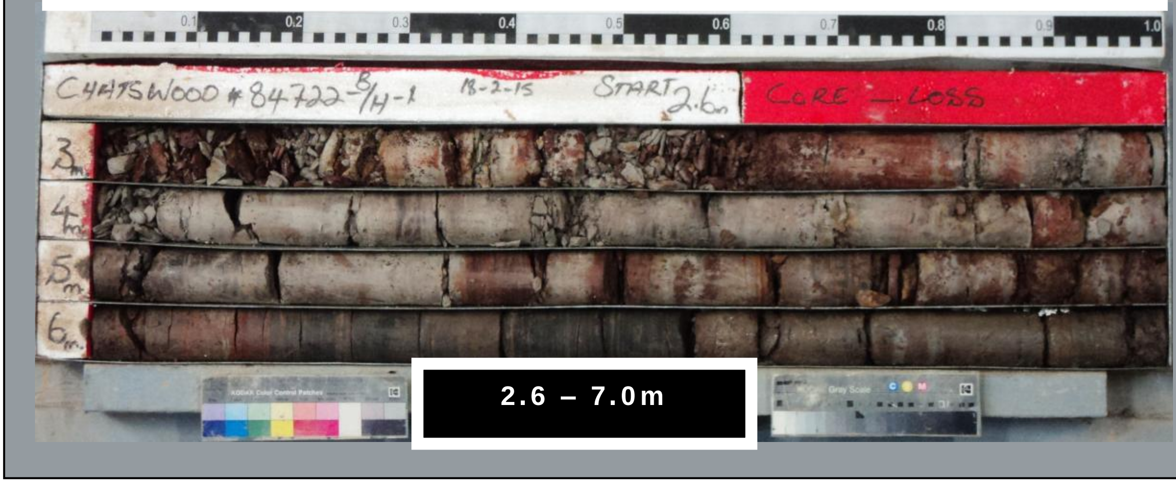
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – CHATSWOOD

BORE 1 PROJECT 84722 FEB 2015



2.6 – 7.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – CHATSWOOD

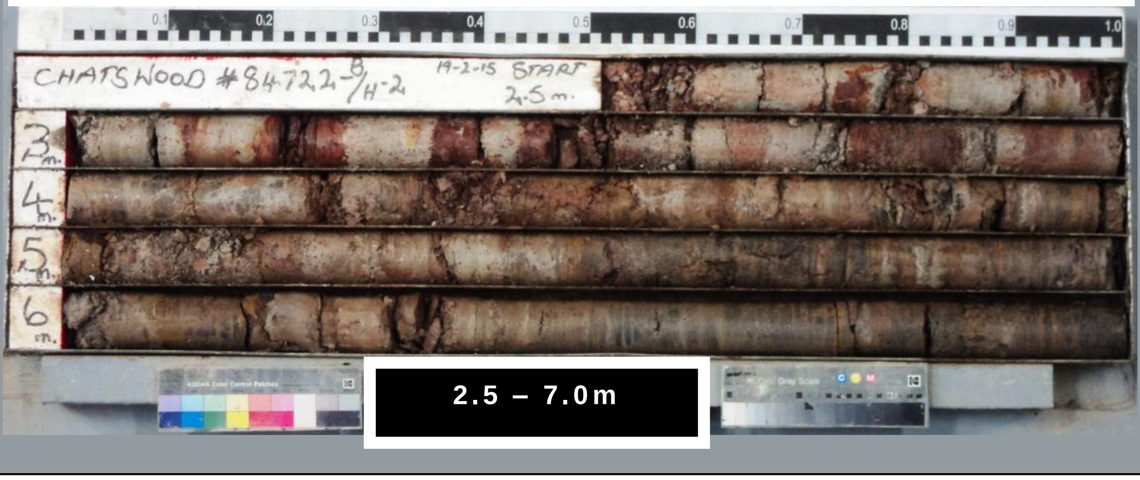
BORE 1 PROJECT 84722 FEB 2015

7.0 – 8.25m

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – CHATSWOOD

BORE 2 PROJECT 84722 FEB 2015

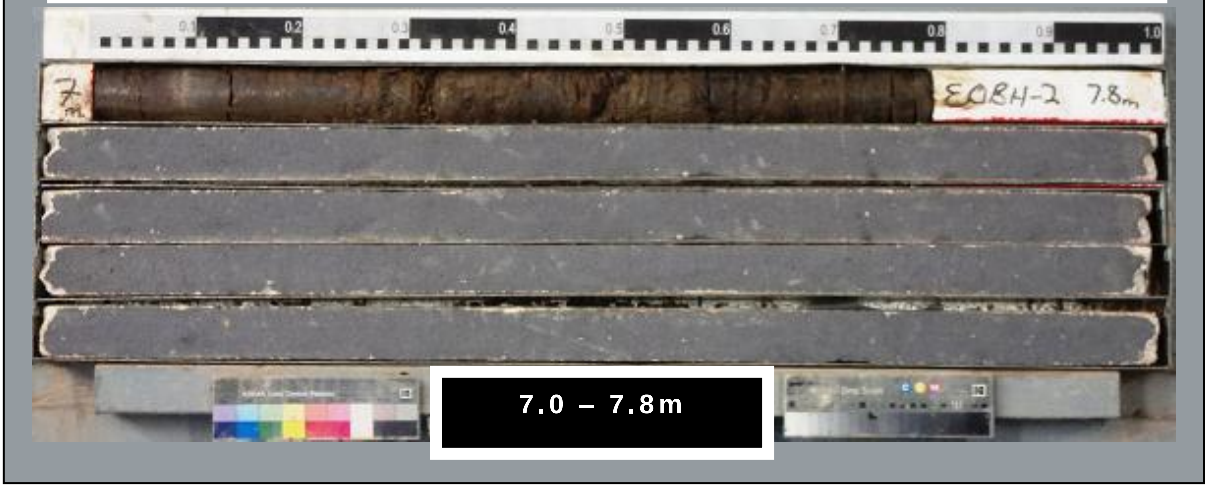


2.5 – 7.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – CHATSWOOD

BORE 2 PROJECT 84722 FEB 2015



7.0 – 7.8m