Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Investigation

Proposed Anatomy Building
University of Newcastle
Callaghan

Prepared for
University of Newcastle

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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
Author		
Reviewer		

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Report on Geotechnical Investigation
Proposed Anatomy Building
University of Newcastle, Callaghan

1. Introduction

This report presents the results of a geotechnical investigation for a proposed three to five storey Anatomy Building to be constructed within the University of Newcastle, Callaghan. The work was carried out for The University of Newcastle.

The purpose of the investigation was to provide the following:

- Subsurface conditions at the site;
- Comments on suitable footing types and geotechnical parameters for footing design;
- Geotechnical parameters for retaining wall design;
- Comments on excavatability;
- Site sub-soil classification in accordance with AS1170.4 (earthquake actions).

For purposes of the investigation the client supplied a detail survey plan of the site by Monteath & Powys Pty Ltd, Ref No 07/126, Sheet 1 of 5, Rev A, dated 24 May 2010.

Ground Test, a subsidiary of Douglas Partners Pty Ltd (DP), has previously undertaken a geotechnical investigation on the adjoining Medical Sciences Building (Ref 1), the results of which have been referred to in the preparation of this report.

2. Site Description

The site is located in the north-western part of the University of Newcastle as indicated on the Locality Plan on Drawing 1, attached.

The area of the proposed anatomy building is currently undeveloped and slopes down to the north-west at about 7^o to 10^o. The site contains a number of mature trees, landscaped gardens and pedestrian pathways.

The following photos show the site:

Figure 1: Looking west / north-west towards Bore 1

Figure 2: Looking south-east towards Bore 2

Figure 3: Looking north / north-west across the site

Reference to the 1:100,000 Newcastle Coalfield Regional Geology sheet indicates that the site is underlain by the Permian aged Tomago Coal Measures, which typically include siltstone, sandstone, coal, tuff and carbonaceous claystone.

3. Field Work

3.1 Methods

The field work was undertaken on 24 August 2010, and comprised the drilling of two bores to depths of 5.4 m to 7.5 m using a 4WD mounted rig (Bores 1 and 2). The bores were drilled using a v-bit solid fight auger to refusal, followed by NMLC diamond drill coring to the termination depth. The approximate locations of the bores are indicated on Drawing 1, attached.

The bores were set out by a geotechnical engineer from DP who also logged the subsurface profile in each bore and took regular samples for laboratory testing and identification purposes. Standard penetration tests (SPT) and pocket penetrometer tests were performed at selected depths and locations.

The test locations were positioned by measuring from existing site features. Surface levels at each of the bore locations were obtained by interpolating between contours on the site survey plan, and are therefore approximate only.

3.2 Results

The subsurface conditions encountered are presented in detail in the attached borehole logs. These should be read in conjunction with the general notes preceding them, which explain the descriptive terms and classification methods used in the reports. The following is a summary of these subsurface conditions.

From (m)	To (m)	Description
0.0	0.1	**Topsoil:** encountered in Bore 2
0.0	0.8	**Filling**: generally comprising sandy clay; encountered in Bore 1
0.1 / 0.8	1.2 / 2.0	**Clay:** generally very stiff to hard
1.2 / 2.0	Termination Depth (5.4 / 7.5)	**Bedrock:** generally comprising siltstone and sandstone, and including laminite; extremely low strength initially, increasing to low to medium strength

The following table summarises the depth to bedrock in each of the bores.

Table 1: Summary of Rock Depths

Bore	Approximate Surface RL (m)	Depth to Top of Rock (m)	Approximate RL of Top of Rock (AHD)	Depth to v-bit Refusal (m)	Approximate RL of v-bit Refusal (AHD)
1	21.5	2.0	19.5	4.5	17.0
2	19.0	1.2	17.8	2.6	16.4

Groundwater was not encountered in either of the bores above the depth of coring during drilling. After coring commenced, groundwater observations were obscured by the drilling fluids. A temporary standpipe was installed in Bore 1 on completion of drilling. Standing water was bailed from the standpipe on 3 September 2010, followed by measurement of groundwater depth on 6 September 2010, about two weeks after drilling.

The following table summarises the groundwater observations in the standpipe.

Table 2: Summary of Groundwater Observations

Location	Approximate Surface Level (AHD)	Depth to Groundwater on 6/9/10 (m)	Approximate Groundwater Level (AHD)
1	21.5	5.4	16.1

It should be noted that groundwater levels are affected by factors such as climatic conditions and soil permeability and will therefore vary with time.

4. Laboratory Testing

Laboratory testing comprised 29 point load index tests to assess rock strength. The detailed results are attached and are summarised on the attached borehole logs.

5. Proposed Development

The proposed development includes construction of the proposed anatomy building, which will initially be a three storey structure, however could be increased to a five storey structure either prior to construction commencing or at some time in the future.

The structural engineer has indicated that working column loads will be in the order of about 2000 kN to 2500 kN for the three storey structure, increasing to up to 4500 kN for the five storey structure.

Preliminary information indicates that construction of the building will require excavations of up to about 2 m along the southern boundary for a partial basement.

6. Comments

6.1 Footings

Footings for column loads in the order of 2000 kN to 4500 kN should be founded in rock, which was encountered below depths of 2.0 m in Bore 1 and 1.2 m in Bore 2 drilled for the current investigation, and was encountered at depths of between 0.8 m to 1.5 m in three of the bores drilled in 1975 for the proposed medical sciences building (Ref 1).

It is understood that excavations of up to 2 m are proposed along the southern limits of the building (ie near Bore 1), therefore it is possible that rock could be encountered near the base of the excavation in this area of the site.

Footings should be founded on rock in all areas, to reduce the risk of differential settlement. It may be possible to support the structural loads on appropriately sized pad footings, however depending on finished floor levels, it is possible that deeper piled foundations could be required in some areas.

Pad footings founded in extremely low strength or better rock, which was encountered at depths of between 1.2 m and 2.0 m in the bores, may be proportioned for a maximum allowable bearing pressure of 700 kPa.

Geotechnical inspection should be undertaken to confirm suitable founding strata. The foundation material will be at risk of softening if left exposed to the elements, therefore it may be prudent to place a blinding layer of concrete following geotechnical inspection.

Where required, bored concrete piles, founded at least four pile diameters below the ground surface may be designed based on the parameters presented in Table 3, below.

Table 3: Geotechnical Parameters for Pile Design

Estimated depth to top of layer (m)	Layer description	Maximum allowable shaft adhesion[1] (kPa)	Maximum allowable end bearing (kPa)
0.8	CLAY – Very stiff to hard	20	-
1.2 / 2.0	Extremely Low strength rock	50	700
3.0 / 5.2	Low to Medium Strength rock	150	1500

[1] Shaft adhesion should be ignored in the upper 1.5 pile diameters

The parameters provided are conditional on the removal of any smearing on the shaft of the pile bore and removal of all loose material at the base of the pile bore. If any water collects in the base of the pile holes, this should be removed, and the base checked for potential softening and over-drilled as necessary, prior to pouring of concrete. Appropriate founding strata should be confirmed during construction.

Contractors should confirm the capacity of their equipment to install piles to the depths required.

Settlement of piles is expected to be in the order of 1% of the pile diameter, or less.

6.2 Mine Subsidence

The site is not located within a proclaimed mine subsidence district. Enquiries with the Mine Subsidence Board (MSB) indicate that the site is not undermined and is not subject to any building restrictions imposed by the MSB.

A copy of correspondence received from the MSB is attached.

6.3 Excavations

It is understood that excavations of up to about 2 m are proposed in the vicinity of the southern boundary of the building, near Bore 1.

The logs show that while excavations will predominantly be through very stiff to hard clay, rock will likely be encountered near the base of bulk excavations, and also in footing excavations.

Bores 1 and 2 encountered v-bit refusal at depths of 4.5 m and 2.6 m, respectively, which is below the expected maximum depth of excavation.

It is therefore anticipated that bulk excavation will generally be achievable using conventional hydraulic equipment. Production rates may slow and moderate to heavy ripping may be needed near the base of excavations.

It is important to note that excavatability of rock is dependent not only on rock strength, but also on the presence, orientation and extent of discontinuities such as jointing and fracturing and other factors. For example, low strength rock with few discontinuities can be more difficult to excavate than highly fractured high strength rock.

Selection of excavation methods and equipment should take into account the particle size distribution of excavated material which is intended for re-use as engineered fill.

6.4 Engineered Filling

If filling is required to support slabs or other lightly loaded structural elements, then it should be placed to the requirements of engineered filling. The following procedure is recommended for placement of engineered filling, if required:

- Remove topsoil, uncontrolled filling and deleterious materials;
- Test roll the surface in order to determine any soft zones and assess moisture condition. Moisture contents should be in the range OMC -3 (dry) to OMC where OMC is the optimum content at standard compaction;
- Compact the tyned natural surface to a dry density ratio of at least 100% Standard. The compacted clay subgrade should be left exposed for a minimum of time prior to placement of pavement layers and floor slabs, to minimise the occurrence of desiccation cracking;
- Suitable filling should be placed in horizontal layers not exceeding 300 mm loose thickness and compacted to a dry density ratio of at least 100% Standard. Moisture content should be in the range as stated above.

Geotechnical inspections and testing should be performed during construction in accordance with AS3798 (Ref 2).

6.5 Retaining Walls

It is understood that retaining walls may be required as part of the construction, although their exact locations and heights of material to be retained are unknown. It is expected that retaining walls will retain natural soil / rock and engineered filling.

For permanent retaining walls, where the wall will be free to deflect, design should be based on “active” (K_a) earth pressure coefficients, assuming a triangular earth pressure distribution. This would comprise any non-propped or laterally unrestrained walls (eg cantilever type walls).

Where structures or services are near the crest, or if the retaining walls are laterally restrained by the structure and not free to deflect, retaining wall design should be based on "at–rest" (K_o) earth pressure coefficients.

The suggested long term (permanent) design soil parameters are shown in Table 4 below. These parameters are applicable to natural clay, compacted engineered filling and rock. The earth pressure coefficients are for level backfill. Any additional surcharge loads, including those imposed by inclined slopes behind the wall, during or after construction, should be accounted for in design.

Table 4: Geotechnical Parameters for Retaining Structures

Parameter	Symbol	Natural Clay, Engineered Filling or Extremely Low Strength Rock	Low Strength Bedrock
Bulk Density (kN/m^3)	γ	20	22
Effective Cohesion (kPa)	c'	5	-
Angle of Friction	ϕ'	25^o	27^o
Active earth pressure coefficient – cantilever design (free to deflect)	K_a	0.3	0.35
At-rest earth pressure coefficient – propped/restrained wall	K_o	0.55	0.5
Passive earth pressure coefficient	K_p	2.5	-
Passive Pressure	P_p	-	400

A factor of safety of at least 2.5 should be incorporated into retaining wall design based on the earth pressure co-efficients presented in the table above.

Retaining walls not designed for hydrostatic pressure should include free draining single size (10 mm single size gravel or coarser) aggregate backfill at the rear of the wall, with a slotted drainage pipe at the base of the backfill. The pipes should discharge to the stormwater drainage system. The backfill should be encapsulated within geotextile fabric.

6.6 Excavation Batters

Temporary batter slopes of up to 2 m height are expected during construction. Permanent batter slopes are not anticipated.

The following temporary batters are recommended:

Table 5: Recommended Temporary Batter Slopes

Strata	Maximum Slope (H:V)
Natural clay / Extremely low strength rock	1:1*
Very low strength rock	0.75:1*

Notes: * Subject to geotechnical inspection during construction; dependent on jointing

Rock cuttings should be inspected by a suitably qualified engineering geologist / geotechnical engineer, during excavation / trimming, to confirm the above batter slopes and identify the need or otherwise for bolting or other temporary support measures.

All batter slopes should be protected from erosion. Surface water should be diverted away from slopes by installation of a dish drain at the crest of slopes.

If permanent batter slopes are required, additional advice should be obtained from this office.

6.7 Earthquake Loading Factors

With reference to AS1170.4 – 2007 (Ref 3), the following factors are considered appropriate to this site:

- Hazard Factor (Z): 0.11
- Soil Sub-class: C_e

7. References

1. Ground Test Pty Ltd, “Report on Foundation Conditions, Proposed Medical Sciences Building, University of Newcastle, Birmingham Gardens”, Report No SSI/1-5059, September 1975.
2. Australian Standard AS 3798-2007: Guidelines on Earthworks for Commercial & Residential Development.
3. Australian Standard AS 1170.4-2007 “Structural Design Actions, Part 4: Earthquake Actions in Australia”, Standards Australia.

8. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at The University of Newcastle, Callaghan in accordance with DP’s proposal dated 16 August 2010 and acceptance received from Mr David Quayle of The University of Newcastle dated 16 August 2010. The work was carried out under DP Conditions of Engagement. This report is provided for the exclusive use of The University of Newcastle for the specific project and purpose 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.

The results provided in the report are considered to be indicative of the sub-surface conditions on the site only to the depths investigated at the specific sampling and/or testing locations, and only at the time the work was carried out. DP's advice may be based on observations, measurements, tests or derived interpretations. The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time. The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site. Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from review by others of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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Appendix A

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

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

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.

Symbols & Abbreviations

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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

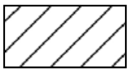
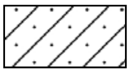
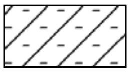
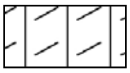
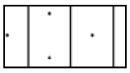
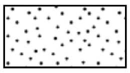
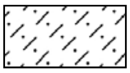
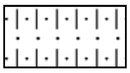
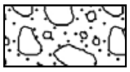
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

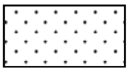
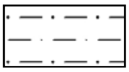
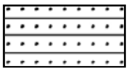
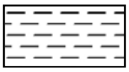
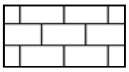
General

■	Asphalt
	Road base
	Concrete
	Filling

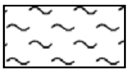
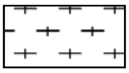
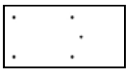
Soils

	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

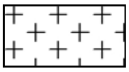
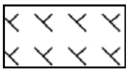
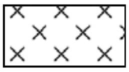
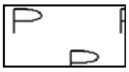
Sedimentary Rocks

	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

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 Descriptions

Rock Quality Designation

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

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

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:

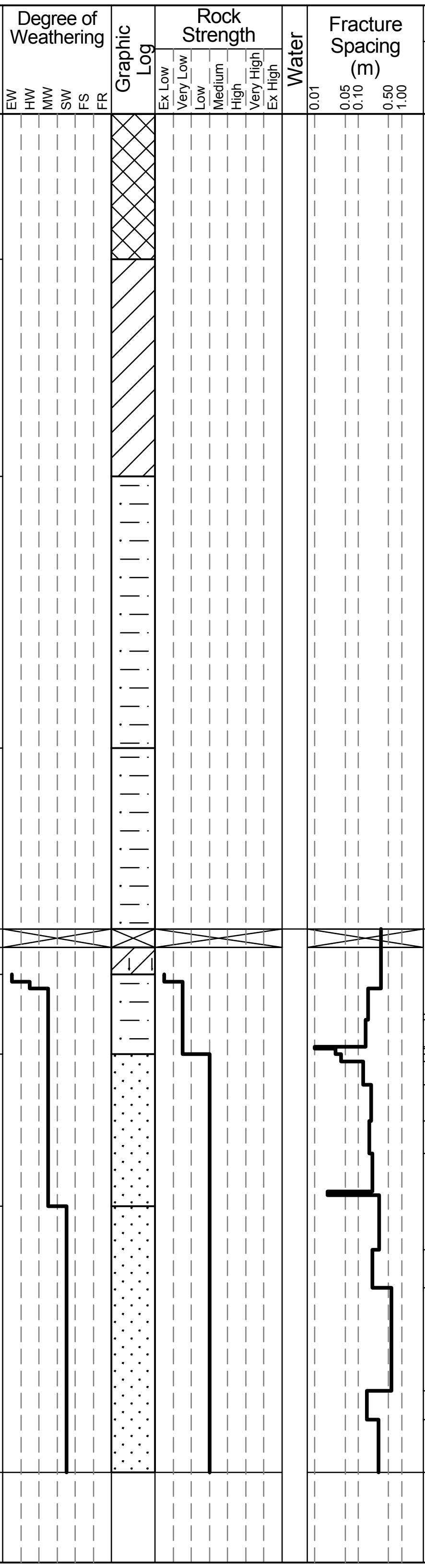
Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

BOREHOLE LOG

CLIENT: University of Newcastle
PROJECT: Proposed Anatomy Facility
LOCATION: Callaghan

SURFACE LEVEL: 21.5 AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 49652
DATE: 24 Aug 10
SHEET 1 OF 1



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
21		FILLING - Generally comprising dark brown sandy clay, some gravel, moist. Trace coal fragments		D			
20	0.8	CLAY - Very stiff light brown mottled grey, trace sand, M≯ Wp. Trace red-brown ferruginised zones to 15mm in diameter (residual)		S pp			3,5,8 N = 13 300 - 350 kPa
19	2.0	SILTSTONE - Extremely low strength, extremely weathered, brown grey and red-brown siltstone. Hard silty clay, M<Wp		S pp			3,15,20/70mm, bouncing >600 kPa
18	3.5	SILTSTONE - Extremely low strength, extremely weathered, brown with some orange-brown and grey speckling siltstone. Hard silt, some clay, some sand, M<Wp. Sand is fine to medium grained, mostly fine grained At 4.5m, v-bit refusal		S			2,13,20 N = 33
17	4.5	CORE LOSS	4.5m: CORE LOSS: 100mm	C	93	60	PL(I) = 0.07 MPa PL(I) = 0.11 MPa PL(D) = 0.09MPa PL(I) = 0.16MPa PL(A) = 0.7MPa PL(D) = 0.56MPa
	4.6	SILTY CLAY - Hard dark brown silty clay					
	4.75	SILTSTONE - Extremely low strength to very low strength, extremely weathered to moderately weathered dark grey siltstone	4.95m: P 10°, Fe stn, pl, ro 4.97m: P 10°, chs un, un, sm From 4.83m to 5.16m - J 95°, clr, pl, sm 5.15m: P 0°, Fe stn, pl, ro 5.16m: P 0°, Fe stn, pl, ro				
16	5.19	SANDSTONE - Low to medium strength, moderately weathered brown medium grained sandstone with some fine grained sandstone and siltstone thinly interlaminated. Trace carbonaceous laminations	5.19m: P 0°, Fe stn, pl, sm 5.23m: P 5°, cbs un, pl, sm 5.36m: P 0°, Fe stn, pl, sm 5.56m: P 5°, Fe stn, pl, ro 5.74m: P 10°, Fe stn, pl, sm				
15	6.03	SANDSTONE - Low to medium strength, slightly weathered light grey fine grained sandstone interlaminated with dark grey siltstone. Some iron staining. Trace carbonaceous laminations	6.27m: P 0°, Fe stn, pl, ro From 6.37m to 6.55m - J, 90°, un, sm 7.05m: P 0°, Fe stn, pl ro 7.21m: P 0°, Fe stn, pl, ro	C	100	100	PL(A) = 0.13MPa PL(D) = 0.21MPa PL(A) = 0.09MPa PL(D) = 0.08MPa PL(A) = 0.15MPa PL(D) = 0.24MPa PL(A) = 0.34MPa PL(D) = 0.22MPa PL(A) = 0.36MPa PL(D) = 0.28MPa
14	7.5	Bore discontinued at 7.5m, limit of investigation					

RIG: Isuzu 4WD Rig **DRILLER:** Atkins Drilling **LOGGED:** Foote **CASING:** 4.5m NW

TYPE OF BORING: Solid flight auger to v-bit refusal at 4.5m, then NMLC coring to 7.5m

WATER OBSERVATIONS: No free groundwater observed above 4.5m during drilling; groundwater measured at 5.4 m in temporary standpipe on 6/9/10

REMARKS: *Surface levels are approximate only

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

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Initials:
Date:

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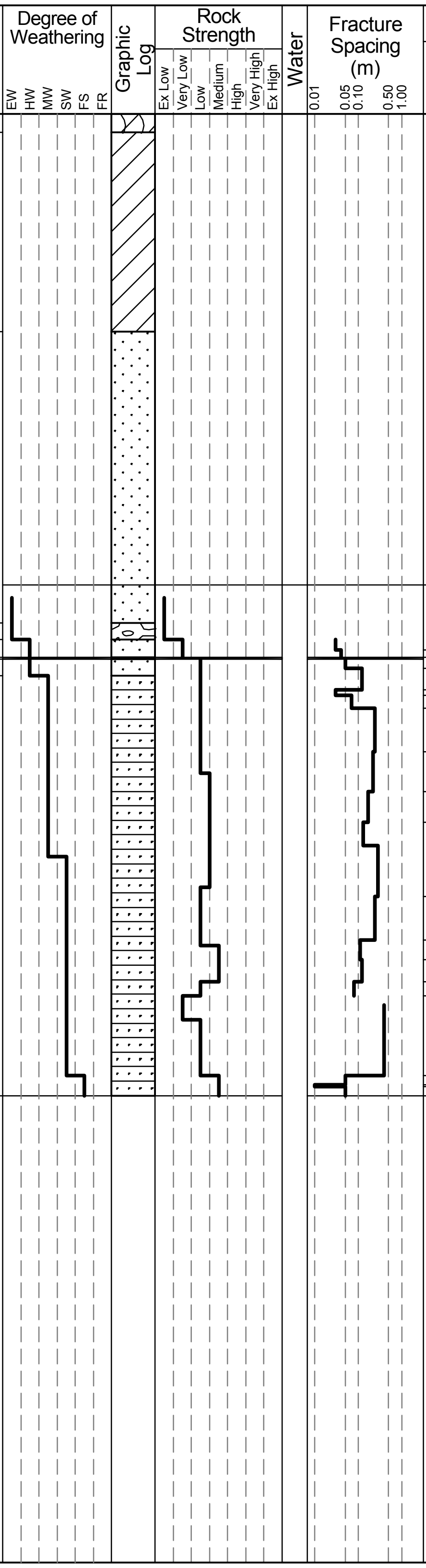
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: University of Newcastle
PROJECT: Proposed Anatomy Facility
LOCATION: Callaghan

SURFACE LEVEL: 19.0 AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 49652
DATE: 24 Aug 10
SHEET 1 OF 1



RL	Depth (m)	Description of Strata
19	0.1	TOPSOIL - Dark brown sandy clayey silt topsoil, humid
		CLAY - Very stiff to hard grey silty clay, M≪Wp
18	1	
	1.2	SANDSTONE - Extremely low strength, extremely weathered, grey mottled orange-brown fine grained sandstone. Dense silty fine grained sand. Behaves low plastic.
17	2	
		At 2.6m, v-bit refusal, start coring
	2.81	CONGLOMERATE - Extremely low strength, extremely weathered, red-brown and grey conglomerate. Gravel phases weathered as well. hard clayey fine grained sand
	2.9	
16	3	
	3.0	SANDSTONE - Very low strength, highly weathered grey, orange-brown and red-brown fine grained sandstone
	3.01	CORE LOSS
	3.1	SANDSTONE - Low strength, hightly weathered grey and red-brown fine grained sandstone
15	4	
		LAMINITE - Low to medium strength, moderately to slightly weathered medium grained sandstone and siltstone thinly interlaminated. Trace thin carbonaceous laminations
14	5	
	5.42	Bore discontinued at 5.42m, limit of investigation
13	6	
12	7	

Discontinuities

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

- 2.93m: P, 5°, cln, un, sm
- 2.96m: J 70°, cly inf, pl, 4mm thick
- 3m: CORE LOSS: 10mm
- 3.06m: J 70°, cly inf, pl, 2mm thick
- 3.18m: J 30°, Fe stn, pl
- 3.21m: J 25°, he, Fe stn, pl
- 3.28m: P 0°, chs un, pl, sm
- 3.52m: P 0°, Fe stn, pl, ro
- 3.74m: P 0°, Fe stn, pl, ro
- 3.91m: P 0°, clm, pl, sm
- 4.32m: P 5°, Fe stn, pl, sm
- 4.56m: P 0°, Fe stn, pl, ro
- 4.67m: P 0°, Fe stn, pl, sm
- 4.79m: P 5°, chs un, cu, sm
- From 4.87m to 4.92m - fragmented zone
- 5.31m: P 5°, Fe stn, pl, sm
- 5.36m: P 10°, Fe stn, pl, ro
- 5.37m: J 80°, Fe stn, pl, ro

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
pp S			>400 kPa 4,11,15 N = 26
S			4,15/50mm bouncing
C	94	60	PL(A) = 0.19MPa PL(D) = 0.27MPa PL(A) = 0.11MPa PL(D) = 0.08MPa PL(A) = 0.23MPa
C	95	79	PL(A) = 0.06MPa PL(D) = 0.08MPa PL(A) = 0.31MPa PL(D) = 0.18MPa PL(A) = 0.13MPa PL(D) = 0.19MPa PL(A) = 0.11MPa PL(D) = 0.11MPa

RIG: Isuzu 4WD Rig **DRILLER:** Atkins Drilling **LOGGED:** Foote **CASING:** 2.6M NW

TYPE OF BORING: Solid flight auger to refusal at 2.6m, then NMLC coring to 5.42m

WATER OBSERVATIONS: No free groundwater observed above 2.6m; groundwater obscured by drilling fluids below 2.6m

REMARKS: *Surface levels are approximate only

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

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Date:

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Photo 1 – Bore 1: 4.5 m to 7.5 m

Photo 2 – Bore 2: 2.6 m to 5.5 m

Geotechnical Investigation for Proposed Anatomy Facility, University of Newcastle	PROJECT 49652	Douglas Partners Geotechnics · Environment · Groundwater

POINT LOAD TEST REPORT

CLIENT : University of Newcastle
PROJECT : Anatomy Facility
LOCATION : University of Newcastle
TEST METHOD: AS 4133.4.1

DATE: 30-Aug-10
PROJECT NO : 49652
TESTED BY : TAC

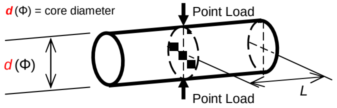
BORE: 1

SHEET: 1 OF 1

DEPTH (m)	ROCK DESCRIPTION	TEST TYPE Axial (A), Diametral (D) Irregular (I)	DIMENSIONS Min. Width (W) (mm)	Depth (d) (mm)	FAILURE READING (KN)	POINT LOAD INDEX, $I_{s(50)}$ Axial (A) or Irregular (I)	POINT LOAD INDEX $I_{s(50)}$ Diametral (D)	INTERPRETED ROCK STRENGTH
4.90	Siltstone	I	52	34	0.15	0.07	-	V LOW
4.90	Siltstone	I	52	30	0.22	0.11	-	LOW
5.25	Siltstone / fine gr sandstone	D		52	0.23	-	0.09	V LOW
5.25	Siltstone / fine gr sandstone	I	30	40	0.28	0.16	-	LOW
5.80	Sandstone	D		52	1.48	-	0.56	MEDIUM
5.80	Sandstone	A	52	30	1.47	0.70	-	MEDIUM
6.20	Laminite	D		52	0.57	-	0.21	LOW
6.20	Laminite	A	52	35	0.31	0.13	-	LOW
6.25	Laminite	D		52	0.22	-	0.08	V LOW
6.25	Laminite	A	52	24	0.16	0.09	-	V LOW
6.45	Laminite	D		57	0.74	-	0.24	LOW
6.45	Laminite	A	52	29	0.31	0.15	-	LOW
7.03	Laminite	D		52	0.59	-	0.22	LOW
7.03	Laminite	A	52	34	0.78	0.34	-	MEDIUM
7.48	Laminite	D		52	0.75	-	0.28	LOW
7.48	Laminite	A	52	28	0.71	0.36	-	MEDIUM

Diametral Test:
CHECK $L > 0.5 \cdot d$
L = distance from load point to nearest free end
d = distance between load points

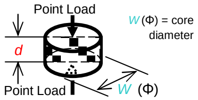
Equivalent core diameter: $d_e = d$



Axial Test:
CHECK $0.3 \cdot W < d < W$

Equivalent core diameter:

$$d_e = \sqrt{4 \cdot \frac{d}{\pi} \cdot W}$$



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POINT LOAD TEST REPORT

CLIENT : University of Newcastle
PROJECT : Anatomy Facility
LOCATION : University of Newcastle
TEST METHOD: AS 4133.4.1

DATE: 30-Aug-10
PROJECT NO : 49652
TESTED BY : TAC

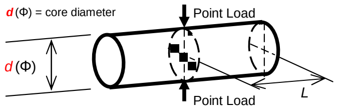
BORE: 2

SHEET: 1 OF 1

DEPTH (m)	ROCK DESCRIPTION	TEST TYPE Axial (A), Diametral (D) Irregular (I)	DIMENSIONS		FAILURE READING (KN)	POINT LOAD INDEX, $I_{s(50)}$ Axial (A) or Irregular (I)	POINT LOAD INDEX $I_{s(50)}$ Diametral (D)	INTERPRETED ROCK STRENGTH
			Min. Width (W) (mm)	Depth (d) (mm)				
3.15	Laminite	D		52	0.71	-	0.27	LOW
3.15	Laminite	A	52	40	0.49	0.19	-	LOW
3.70	Laminite	D		52	0.22	-	0.08	V LOW
3.70	Laminite	A	52	38	0.28	0.11	-	LOW
4.00	Laminite	A	52	33	0.52	0.23	-	LOW
4.45	Laminite	D		52	0.20	-	0.08	V LOW
4.45	Laminite	A	52	40	0.15	0.06	-	V LOW
4.70	Laminite	D		52	0.47	-	0.18	LOW
4.70	Laminite	A	52	35	0.74	0.31	-	MEDIUM
5.15	Laminite	D		52	0.51	-	0.19	LOW
5.15	Laminite	A	52	34	0.30	0.13	-	LOW
5.30	Laminite	D		52	0.30	-	0.11	LOW
5.30	Laminite	A	52	35	0.27	0.11	-	LOW

Diametral Test:
CHECK $L > 0.5 \cdot d$
L = distance from load point to nearest free end
d = distance between load points

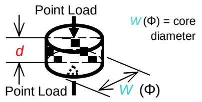
Equivalent core diameter: $d_e = d$



Axial Test:
CHECK $0.3 \cdot W < d < W$

Equivalent core diameter:

$$d_e = \sqrt{4 \cdot \frac{d}{\pi} \cdot W}$$



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Initials
Date

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Geotechnics · Environment · Groundwater

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SUBSIDENCE
BOARD

In reply please send to: Newcastle District Office

Our reference: FN10-25260N1 PA.VS

Your reference: email enq.

Contact: Phil Alexander (02) 4908 4352

RECEIVED
14 SEP 2010
BY:

Douglas Partners Pty Ltd
Box 324
HRMC NSW 2310

ATTENTION: JULIE WHARTON

9 September 2010

Dear Ms Wharton

ENQUIRY NO: TENQ10-05983N1
LOT 16 DP 817507 NO 130 UNIVERSITY DR CALLAGHAN

This property is not within a proclaimed Mine Subsidence District and is not subject to any building restrictions imposed by the Mine Subsidence Board.

The provisions of the Mine Subsidence Compensation Act cover any improvement erected on this land.

Yours faithfully

Phil Alexander
District Manager

NEWCASTLE
Ground Floor
NSW Government Offices
117 Bull Street
Newcastle West 2302
PO Box 488G Newcastle 2300
Telephone: (02) 4908 4300
Facsimile: (02) 4929 1032
DX 4322 Newcastle West

PICTON
100 Argyle Street
Picton 2571
PO Box 40 Picton 2571
Telephone: (02) 4677 1967
Facsimile: (02) 4677 2040
DX 26053 Picton

SINGLETON
The Central Business Centre
Unit 6, 1 Pitt Street
Singleton 2330
PO Box 524 Singleton 2330
Telephone: (02) 6572 4344
Facsimile: (02) 6572 4504

WYONG
Suite 3 Feldwin Court
30 Hely Street
Wyong 2259
PO Box 157 Wyong 2259
Telephone: (02) 4352 1646
Facsimile: (02) 4352 1757
DX 7317 Wyong

HEAD OFFICE
PO Box 488G
Newcastle 2300
Telephone: (02) 4908 4395
Facsimile: (02) 4929 1032

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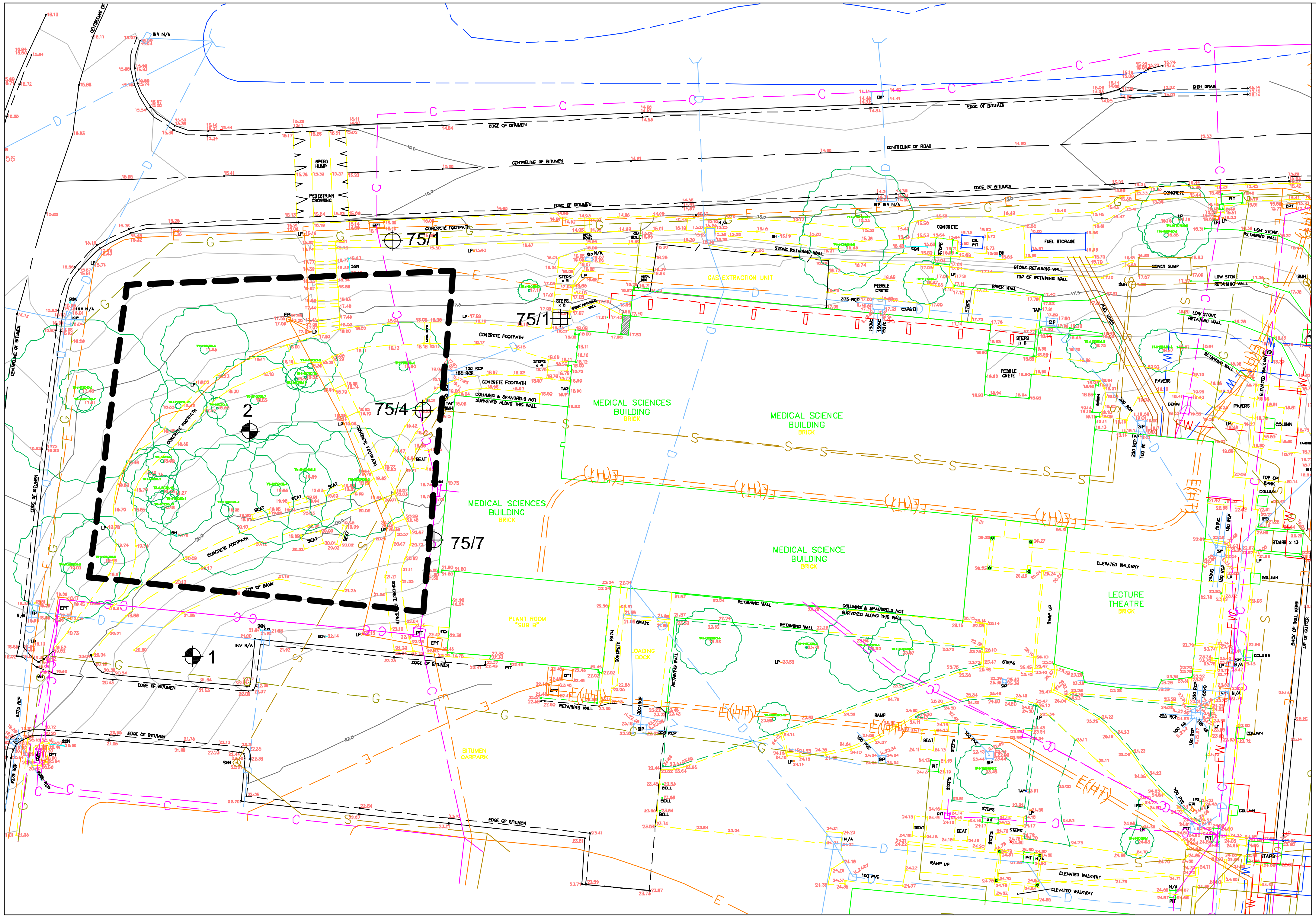

Locality Plan

NOTE: Base drawing from Detail Survey Plan by Monteath & Powys Pty Ltd, Ref 02/126, Rev A, 24/5/10

0 2 4 6 8 10 15 20 30 40m

1:400 @ A3

LEGEND

- ◐ Approximate Borehole Locations (Current Investigation)
- ⊕ Approximate Borehole Locations (Previous Investigation, Ref 1)
- ⊞ Approximate Test Pit Locations (Previous Investigation, Ref 1)
- ▭ Approximate Proposed Building Outline

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: University of Newcastle		TITLE: **Test Location Plan**		PROJECT No: 49652
OFFICE: Newcastle	DRAWN BY: PLH	**Proposed Anatomy Building**	N	DRAWING No: 1
SCALE: As shown	DATE: 23.09.2010	**University of Newcastle, Callaghan**		REVISION: 0