Douglas Partners
Geotechnics / Environment / Groundwater

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Level 23
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SYDNEY NSW 2000

Project 86043.00
03 August 2017
R.002.Rev0
SCP:cjm

Attention: Joe Zannino

Email: JoeZannino@citta.com.au

Dear Sirs

Geotechnical Desktop Assessment
Proposed Residential Development
2-4 Lyon Park Road, Macquarie Park

1. Introduction

This letter report presents the results of a desktop geotechnical assessment undertaken by Douglas Partners Pty Ltd (DP) for the construction of a proposed road along the north-eastern side of 2-4 Lyon Park Road. The assessment was commissioned in an email dated 29 June 2017 by Joe Zannino of Citta Property Group Pty Limited on behalf of Frasers Property Australia Pty Ltd.

The proposed road will extend from Lyon Park Road along the north-eastern boundary of 2-4 Lyon Park Road, then onto a new bridge over Shrimptons Creek. Only the south-eastern abutment of the bridge is considered in this report. (Reference should be made to DP Report 86043.00.R.001 in relation to the abutment on the north-western side of Shrimptons Creek.) The development also includes new car parking and landscaping areas adjacent to the new road.

The aim of the desktop assessment is to consider the subsurface soil and groundwater conditions expected at the development areas in order to provide information on the proposed road and bridge construction.

The assessment included:

- the review of selected mapping for the area;
- review of DP's database of past investigations in the vicinity of the site; and,
- a brief site visit to assess site conditions and make observations.

2. Site Description

The greater site at 2-4 Lyon Park Road is occupied by a central, multi-storey commercial building with three basement parking levels. The ground around the site is generally paved, with some grassed and vegetated garden areas towards the rear (north-west) of the site. Ground levels at the site slope gently down from RL 52 towards the southern corner of the site, to approximately RL 41 at Shrimptons Creek at the rear of the site.

From discussions with the Facilities Manager at the site, it is understood that the rear portion of the site is susceptible to flooding from Shrimptons Creek, and that a recent flood event resulted in damage to a chain-link fence at the rear of the site, apparently by undermining of the shallow concrete foundations.

Of particular interest to this report is the subject area within 12–15 m of the north-eastern boundary, which is proposed for the road development. In this area, the site is currently generally paved, as shown in Figure 1, with some landscaped garden areas.

Figure 1: Subject new road alignment, facing west from Lyon Park Road

The existing pavement comprises interlocking segmental paving blocks and is generally in good condition. Pavement levels within the subject development area slope gently down from approximately RL 49 at Lyon Park Road towards Shrimptons Creek at approximately 1° to 2°.

Beyond the pavements, still within the subject development area, ground levels generally slope down at approximately 8° to 10° from RL 46 at the pavement edge to approximately RL 41 at Shrimptons Creek. The site is generally grassed in this area, with some trees, and the surface of some sandstone boulders is visible at ground surface indicating filling or disturbed soils. The ground surface was observed to be moist at the time of the site visit.

Higher ground levels on the adjoining site at 6 Lyon Park Road (for pavement areas, at approximately RL 49) are supported by crib walls on the common site boundary and, in some parts of the site, by batters within the proposed road corridor. The difference in level between the sites increases towards Shrimptons Creek and at the rear half of the site, mulched and vegetated batters rise along the proposed corridor towards the toe of the crib walls. The batters are up to 2 m high, and support crib walls of typical heights of 1.5 m to 2 m (see Figure 2)

Figure 2: Mulched batter slope and crib wall at rear boundary between 2-4 and 6 Lyon Park Road

At the rear of the proposed road corridor, a stormwater outlet drains to Shrimptons Creek. The outlet has steep sandstone boulder headwalls and concrete sidewalls, built into the steeper batter in this area. The concrete and sandstone apron may possibly be constructed on sandstone blocks or outcrop, but was not accessible for safe inspection during the site visit.

Figure 3: Stormwater outlet at Shrimptons Creek, with concrete sidewalls and sandstone boulder headwall, as viewed from north-western bank.

Sandstone boulders were visible within the soil creek banks, and the banks had apparently been subject to previous erosion in some areas. A chain-link fence generally separated the site from Shrimptons Creek, with the area of fencing undermined by recent erosion located south-west of the subject area.

3. Regional Mapping

Reference to the Sydney 1:100 000 Soil Landscape Series Sheet indicates that the site is underlain by the residual Lucas Heights soil landscape towards Shrimptons Creek, and by the erosional Glenorie soil landscape towards Lyon Park Road. These natural soils are likely to comprise sandy clay and clayey sand soils, possibly with some silty clay.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is mapped as underlain by Hawkesbury Sandstone under most of the site, with a boundary with Ashfield Shale towards Lyon Park Road. The Ashfield Shale, where present, would be underlain by Hawkesbury Sandstone, possibly with a transitional Mittagong Formation (interbedded shale, laminite and sandstone) between the two layers. DP experience is consistent with the Hawkesbury Sandstone mapping.

There are no registered groundwater bores in the immediate area (ie. around the proposed road corridor and new bridge). The site is outside of areas of known salinity or acid sulphate soils, although some layers within the Glenorie soil landscape can be associated with local occurrence of salinity.

4. Past Field Work

DP has undertaken past investigation and inspections on the subject site. Five boreholes were undertaken near the proposed development corridor in August 2000, prior to the construction works for the existing building (refer to Bores 1 to 5, at the locations shown in Drawing 1, attached). The test bore reports for these locations are also attached, together with the relevant notes. The boreholes indicate that:

- Poorly compacted filling was present to depths of up to 1.8 m, (though noting that the earthworks involved in the construction of the existing building and pavements are likely to have altered this upper profile, potentially removing some or all of the unsuitable filling and/or the placement of new, possibly engineered filling).
- The natural soils underlying the filling generally comprised soft, firm and firm to stiff silty, sandy clay, sometimes with ironstone gravel. While a thick band of ironstone was noted at Bore 1,given the drilling methods, it is considered likely that this refers to a layer of weakly iron-cemented soil or rock, or ironstone gravel, rather than solid ironstone.
- Sandstone was identified underlying the natural soils at Bores 2 to 5, at levels falling from RL 45 at Bore 5 to RL 42.9 at Bore 2. The sandstone ranged from extremely low strength, improving to high strength, with strength generally improving with depth.
- No free groundwater was observed whilst augering at borehole locations, however it was noted that “considerable seepage and saturated soil conditions” were present, particularly over the rear third of the site during the investigation. These observations suggest that very high moisture levels were present within the soils, though seepage flows may be localised, temporary or slow-flowing (based on the absence of water inflow to boreholes whilst augering).

5. Comments

5.1 Proposed Development

The construction of a new pavement is proposed along the north-eastern side of the site, with a pavement width of approximately 7 m.

The works will include associated works to relocate existing services in the area, which will include the construction of additional parking bays, landscaping and moving electrical substation(s).

A new bridge is proposed at Shrimptons Creek, and the south-eastern bridge abutment will be located within the subject road corridor. The precise location of the south-eastern abutment is not currently known.

5.2 Geotechnical and Hydrogeological Model

The geotechnical and hydrogeological model for the subject area is as follows:

- **Filling** – comprising likely well-compacted, possibly partly lime-stabilised filling below existing pavements (only), and uncontrolled, poorly compacted filling below the grassed and landscaping areas, including an upper layer of topsoil; to typical depth of 0.5 m to 1.5 m below ground level, but likely deeper in some areas; underlain by,
- **Sandy and Silty Clay** – soft and firm to stiff, moist to wet, to depths of approximately 2 m to 3 m below ground level, possibly deeper towards Shrimptons Creek; underlain by,
- **Sandstone** – extremely low strength sandstone, improving with depth to high strength sandstone, possibly including bands of iron-cemented strong and weak (bleached) layers.

Groundwater levels are generally expected to be within the sandstone, or towards the base of the natural soils. Towards Shrimptons Creek, where deeper rock may be present, groundwater levels may be within the soils. Temporary groundwater levels are likely to be present towards the base of the filling and natural soil layers, particularly following rainfall, with rainfall events expected to result in elevated groundwater levels, particularlyin the vicinity of Shrimptons Creek. Based on previous observations at the site, these elevated groundwater levels may persist for some time following wet weather, although they may also be alleviated by drainage installed during the site development.

Even where groundwater levels are not elevated, soils in the vicinity of Shrimptons Creek are likely to have a relatively high moisture content.

5.3 Foundations

Foundations for the new bridge should be taken down to bear on sandstone bedrock. Pile foundations are likely to be required. Contiguous flight auger (CFA) piles may be required for construction if groundwater levels are above the bedrock surface to avoid the risk of sidewall collapse and major seepage inflows. Augered pile holes with temporary casing, to protect against sidewall collapse may be considered, but such temporary casing may be ineffective if groundwater inflow into the pile hole is significant.

Where pile foundations are taken down to bear on at least consistent, medium strength sandstone, an allowable bearing pressure of up to 3.5 MPa may be adopted for preliminary design purposes. Specific geotechnical investigation using cored boreholes at the abutment locations would generally be required in advance of the works to confirm the bearing capacity. Higher bearing capacities could potentially be achieved, subject to the material encountered by the investigation.

Shallow foundations may be appropriate for minor structures associated with the road extension, such as the substation(s). These relatively lightly loaded structures could be founded on at least stiff, natural clay soils, based on an allowable bearing capacity of 100 kPa. It would generally not be appropriate to support structures on filling that has not been placed and engineered for that purpose.

5.4 Earthworks

The proposed road formation levels have not been confirmed at this time, and are likely to depend on the desired road alignment, interface with adjacent buildings and roads, and construction practicalities. Such practicalities may include the following:

- Saturated soil conditions identified towards the rear of the site during previous investigations suggests that there may be difficulties undertaking earthworks during wet weather. Raised road levels may be an alternative approach, to limit the influence of any soft or saturated soils on the road formation, although batter stability will need to be appropriately addressed. Also, the construction of engineered fill platforms (eg. for abutments) over soft soils may requires either the improvement/treatment of such soils to reduce consolidation settlement to acceptable levels, or the removal and replacement of such soft soils. The use of rockfill with concrete pavements and approach slabs is a common solution for the design of pavement batters that will be subject to flooding.
- Care will need to be taken during any excavation for earthworks below the adjacent crib walls, as this may cause movement of the walls. Assessment and investigation will generally be required to assess the foundation level of the existing boundary retaining walls, foundation strata and any passive support requirements which may dictate the approach to excavation. The following options may be available for excavation:
  - Embedded piled shoring walls could be used to support the existing boundary retaining walls, and allow ready excavation in front of the wall. A piled wall is likely to be required to provide rigidity to the sides of the excavation. This will require some offset from the wall in order to install the piles, and piles may need to be relatively closely spaced or possibly contiguous to provide appropriate support;
  - Alternatively, where excavation is only required for the short-term and the soil below the foundation is of adequate strength, the use of a ‘hit-and-miss’ excavation and replacement sequence (ie. in alternate ‘panels’) could be considered. This can be a relatively time consuming process, but can avoid the need for pile installation. The width of panels would require further assessment;
  - It is possible in areas of limited excavation, if favourable ground conditions and wall design exist, that batters and benches may be used. This may not be compatible with the current road geometry, particularly towards the rear of the site, where steep batters are already present below the ‘high-side’ retaining wall along the site boundary.

Given the above, the proposed road would ideally limit excavation requirements below the crib wall.

5.5 Pavements

In order to provide a suitable subgrade for pavement and road construction, it is suggested that the following procedures be adopted:

- Excavate the existing pavement materials and filling, and stockpile for possible re-use. Some of the filling, particularly towards the rear of the site, may be unsuitable for re-use. Where re-use is practical, the filling is nonetheless expected to require removal of unsuitable material (e.g. organic material, over-wet or highly saturated soils and sandstone cobbles/boulders) and moisture treatment prior to reuse as engineered filling.

- Proof roll the underlying firm to stiff clay to identify any exceptionally soft and yielding areas which should be removed;
- Place filling in uniform layers and compact to an appropriate density/compaction level. The appropriate densities and compaction ratios will depend on the filling material adopted, proposed traffic loading and level of the fill layer relative to the top-of-subgrade.

It is recommended that any new filling is placed as controlled filling, with inspection and testing of compaction activities undertaken at a suitable frequency. Guidance on test frequencies may be sought from AS3798 *Guidelines on Earthworks for Commercial or Residential Developments.*

Appropriate surface and subsurface drainage is critical to pavement performance, and should be carefully considered in the design. Towards the rear of the site, where saturated soil conditions have been previously observed, this may require the use of a drainage blanket to limit the ingress of moisture into the road pavement layers. The efficacy of such drainage will depend on the pavement levels and vulnerability of the drainage system to flooding from Shrimptons Creek. Robust subsoil drainage should be installed to at least 500 mm below subgrade level along the high side of all pavement areas and adjacent to garden/grassed areas

In the absence of specific laboratory testing it is suggested that a CBR of 5% is adopted as a preliminary value for pavement design, given the previous earthworks at the site and reasonable condition of the existing pavement. This CBR assumes that equilibrium moisture content is maintained within the subgrade, and thus is reliant on appropriate subsurface drainage.

6. Limitations

Douglas Partners (DP) has prepared this report for this project at 2-4 Lyon Park, Macquarie Park in accordance with DP’s proposal dated 7 June 2017 and subsequent email of 30 June and acceptance received from Joe Zannino of Citta Property Group Pty Limited (on behalf of Frasers Property Group Pty Ltd.) dated 30 June 2017. The work was carried out under DP’s Conditions of Engagement . This report is provided for the exclusive use of Fraser Property Australia 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 sub-surface conditions observed on the site and inferred from geological and other mapping and nearby DP investigations. Sub-surface 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.

The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site. The advice may also be limited by scope constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached notes 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 scope for work for this report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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.

Please contact the undersigned if you have any questions on this matter.

Yours faithfully
Douglas Partners Pty Ltd

Sally Peacock
Geotechnical Engineer/Associate

Reviewed by

Bruce McPherson
Principal

Attachments:
- About this Report
- Drawing 101
- Previous Test Bore Results and Related Notes

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

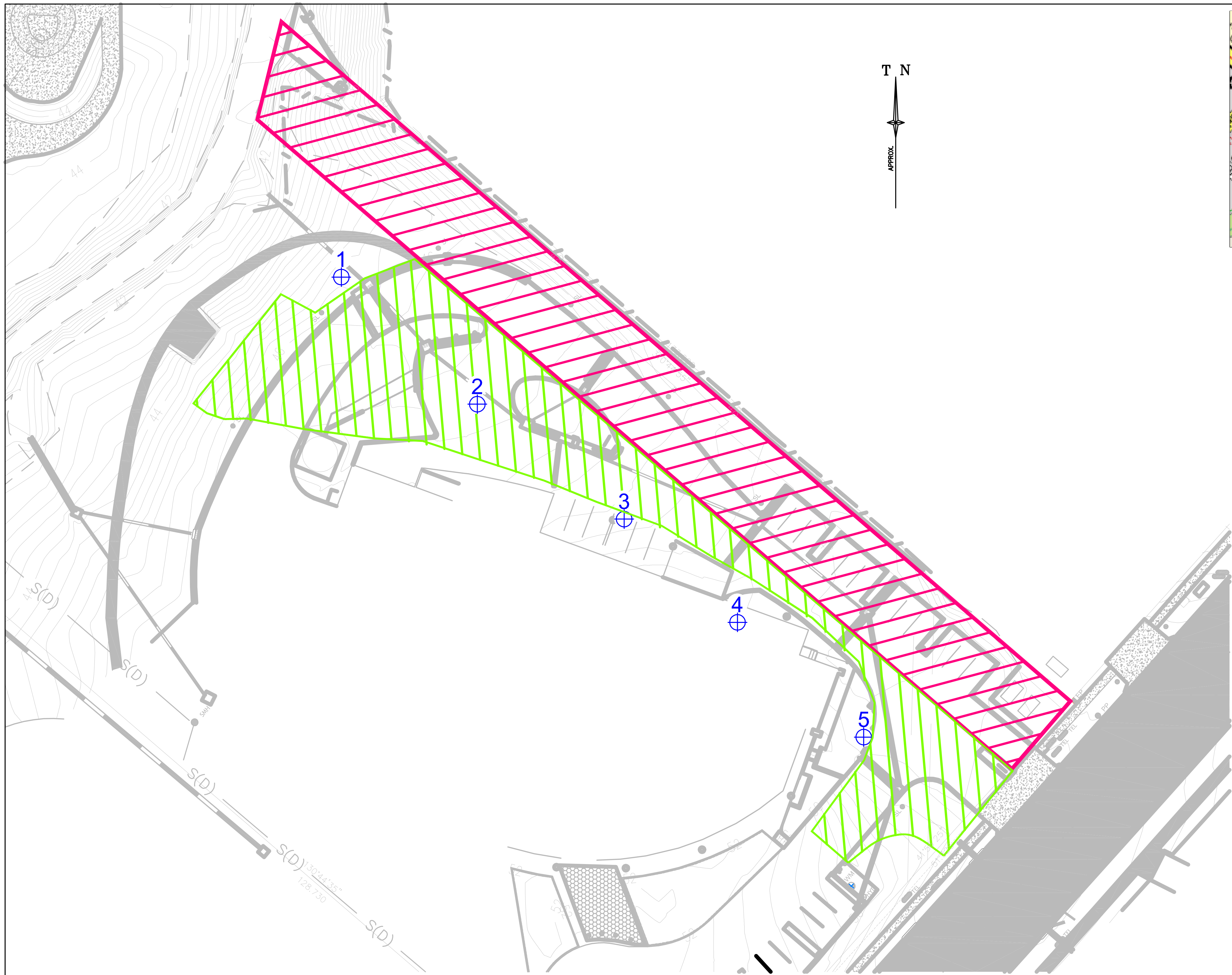
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later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

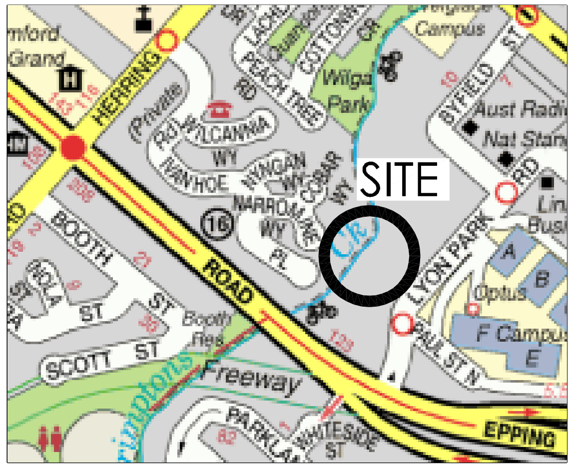
Information for Contractual Purposes

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.





Locality Plan

LEGEND

⊕ Previous Test Boreholes (1-5)

▨ Proposed Road

▨ Associated Development

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CLIENT: Frasers Property Australia Pty Ltd
OFFICE: Sydney
DRAWN BY: Vojta
SCALE: 1:500@A3
DATE: 24.7.2017

TITLE: **Previous Test Locations**
Proposed Road - Ivanhoe Estate
2-4 Lyon Park Road, Macquarie Park

PROJECT No: 86043.00
DRAWING No: 101
REVISION: 0

TEST BORE REPORT

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTI STOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

DATE: 1 AUGUST 00
PROJECT No.: 29190
SURFACE LEVEL: 45.12

BORE No. 1
SHEET 1 **OF** 1

Depth m	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Results	Headspace PID (ppm)
0	FILLING - poorly compacted, light brown to brown clay filling with a trace of silt and gravel	A S	0.5 0.95	1,1,2 N=3	2
1.4	CLAY - firm, brown mottled red brown clay with a trace of ironstone gravel				
1.8	IRONSTONE				
2.0	TEST BORE DISCONTINUED AT 2.0 METRES - auger refusal				

RIG: B40 **DRILLER:** DRIVER **LOGGED:** CARLE **CASING:**

TYPE OF BORING: 100mm DIAMETER SPIRAL FLIGHT AUGER

GROUND WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED

REMARKS: TBM GRATE IN LYON PARK ROAD RL 48.22

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp Pocket Penetration (kPa)	V shear vane (kPa)

CHECKED:
Initials:
Date: 10/8

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TEST BORE REPORT

BORE No: 2

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTISTOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

PROJECT No: 29190
SURFACE LEVEL: 45.91
DIP OF HOLE: 90°

DATE: 2/8/00
SHEET 1 **OF** 1
AZIMUTH:

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - poorly compacted, dark brown silty sandy clay		▩							
							S			1,2,4 N=6
0.75	FILLING - poorly compacted, dark grey and yellow brown sandy clay and gravel filling		▩							
1.1	FILLING - crushed sandstone and gravel filling		▩							
1.7	SANDY CLAY - firm to stiff, light grey and yellow brown sandy clay		▨							
							S			3,4,4 N=8
3.0	SANDSTONE - extremely low to very low strength, light grey brown sandstone		░							
3.5	TEST BORE DISCONTINUED AT 3.5 METRES						A			

RIG: B40 **DRILLER:** DRIVER **LOGGED:** PARMAR **CASING:** UNCASED
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 3.5m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials:
Date: 10/8

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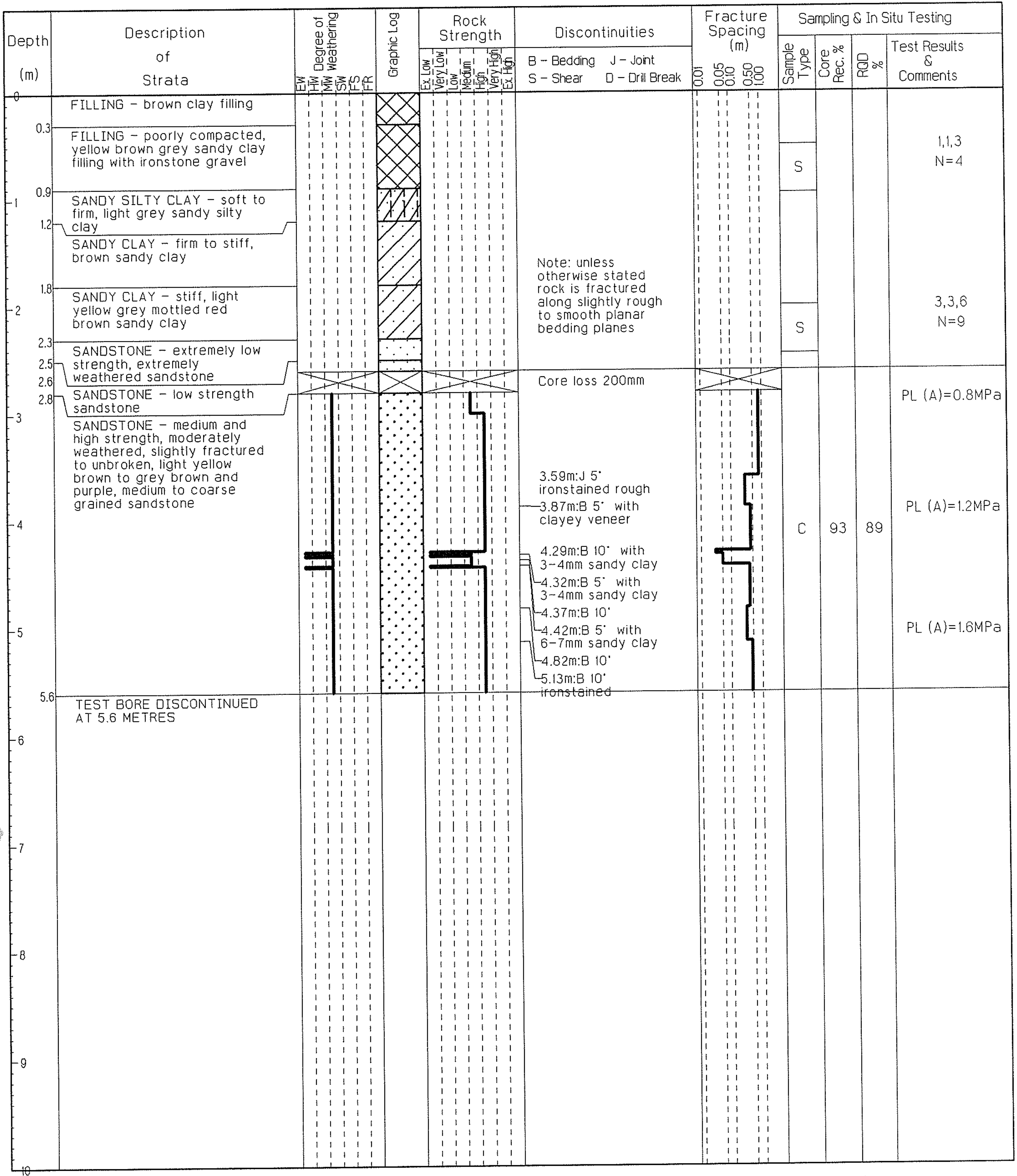
TEST BORE REPORT

BORE No: 3
DATE: 2/8/00
SHEET 1 **OF** 1
AZIMUTH:

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTISTOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

PROJECT No: 29190
SURFACE LEVEL: 46.76
DIP OF HOLE: 90°



Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - brown clay filling					
0.3	FILLING - poorly compacted, yellow brown grey sandy clay filling with ironstone gravel		S			1,1,3 N=4
0.9	SANDY SILTY CLAY - soft to firm, light grey sandy silty clay					
1.2	SANDY CLAY - firm to stiff, brown sandy clay	Note: unless otherwise stated rock is fractured along slightly rough to smooth planar bedding planes				
1.8	SANDY CLAY - stiff, light yellow grey mottled red brown sandy clay		S			3,3,6 N=9
2.3	SANDSTONE - extremely low strength, extremely weathered sandstone					
2.5						
2.6	SANDSTONE - low strength sandstone	Core loss 200mm				
2.8	SANDSTONE - medium and high strength, moderately weathered, slightly fractured to unbroken, light yellow brown to grey brown and purple, medium to coarse grained sandstone	3.59m:J 5° ironstained rough 3.87m:B 5° with clayey veneer 4.29m:B 10° with 3-4mm sandy clay 4.32m:B 5° with 3-4mm sandy clay 4.37m:B 10° 4.42m:B 5° with 6-7mm sandy clay 4.82m:B 10° 5.13m:B 10° ironstained	C	93	89	PL (A)=0.8MPa PL (A)=1.2MPa PL (A)=1.6MPa
5.6	TEST BORE DISCONTINUED AT 5.6 METRES					

RIG: B40
DRILLER: DRIVER
LOGGED: PARMAR
CASING: GL TO 2.6m
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 2.6m,NMLC CORING TO 5.6m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials: [illegible]
Date: 10/8

Douglas Partners
Geotechnics · Environment · Groundwater

TEST BORE REPORT

CLIENT: LIPMAN PTY LTD
PROJECT: PROPOSED MULTI STOREY BUILDING
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE

DATE: 1 AUGUST 00
PROJECT No.: 29190
SURFACE LEVEL: 47.3

BORE No. 4
SHEET 1 **OF** 1

Depth m	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Results	Headspace PID (ppm)
0	FILLING - poorly compacted, brown, slightly sandy clay filling				
		A*	0.5	1,2,4 N=6	2
		S			
	- 0.95m - traces of wood		0.95		
1.3	CLAY - red brown clay with a trace of silt and sand				
1.7	SILTY SANDY CLAY - grey silty sandy clay	A	1.8		2
2.0	CLAY - firm, red brown clay		2.0	2,3,5 N=8	
		S			
			2.45		
2.8	SANDSTONE - extremely low strength, light grey sandstone with some clay				
3.5	TEST BORE DISCONTINUED AT 3.5 METRES - auger refusal				

RIG: B40
DRILLER: DRIVER
LOGGED: CARLE
CASING:
TYPE OF BORING: 100mm DIAMETER SPIRAL FLIGHT AUGER
GROUND WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS: *DENOTES DUPLICATE SAMPLE Z1 TAKEN

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp Pocket Penetration (kPa)	V shear vane (kPa)

CHECKED:
Initials:
Date: 10/8

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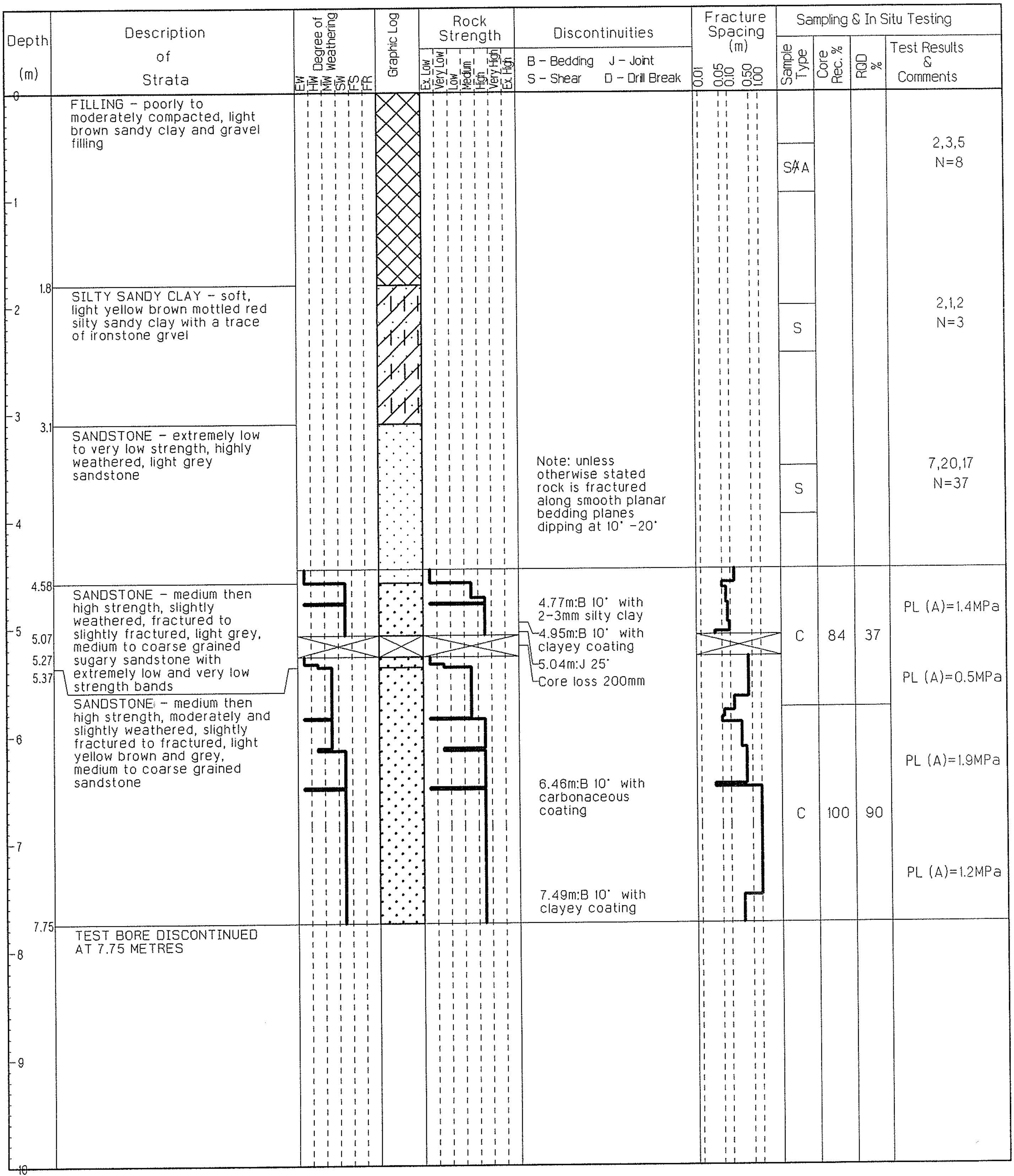
TEST BORE REPORT

BORE No: 5
CLIENT: LIPMAN PTY LTD
PROJECT No: 29190
DATE: 3/8/00
PROJECT: PROPOSED MULTISTOREY BUILDING
SURFACE LEVEL: 48.05
SHEET 1 **OF** 1
LOCATION: 2-4 LYON PARK ROAD, NORTH RYDE
DIP OF HOLE: 90°
AZIMUTH:



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	FILLING - poorly to moderately compacted, light brown sandy clay and gravel filling		S/A			2,3,5 N=8
1.8	SILTY SANDY CLAY - soft, light yellow brown mottled red silty sandy clay with a trace of ironstone grvel		S			2,1,2 N=3
3.1	SANDSTONE - extremely low to very low strength, highly weathered, light grey sandstone	Note: unless otherwise stated rock is fractured along smooth planar bedding planes dipping at 10° -20°	S			7,20,17 N=37
4.58 5.07 5.27 5.37	SANDSTONE - medium then high strength, slightly weathered, fractured to slightly fractured, light grey, medium to coarse grained sugary sandstone with extremely low and very low strength bands	4.77m:B 10° with 2-3mm silty clay 4.95m:B 10° with clayey coating 5.04m:J 25° Core loss 200mm	C	84	37	PL (A)=1.4MPa PL (A)=0.5MPa
	SANDSTONE - medium then high strength, moderately and slightly weathered, slightly fractured to fractured, light yellow brown and grey, medium to coarse grained sandstone	6.46m:B 10° with carbonaceous coating 7.49m:B 10° with clayey coating	C	100	90	PL (A)=1.9MPa PL (A)=1.2MPa
7.75	TEST BORE DISCONTINUED AT 7.75 METRES					

RIG: B40
DRILLER: DRIVER
LOGGED: PARMAR
CASING: GL TO 4.45m
TYPE OF BORING: SPIRAL FLIGHT AUGER TO 4.45m,NMLC CORING TO 7.75m
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_S (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials: WB
Date: 10/8

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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

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, 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 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 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate 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 $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

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 longer 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:

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

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

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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
$\underline{\nabla}$	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

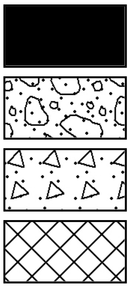
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

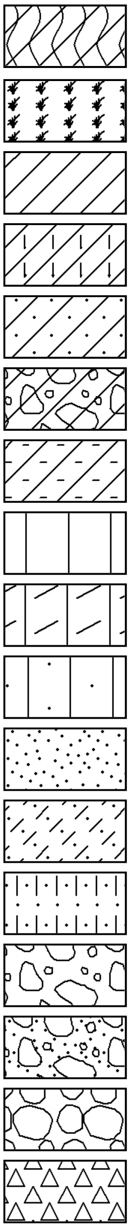
Graphic Symbols for Soil and Rock

General



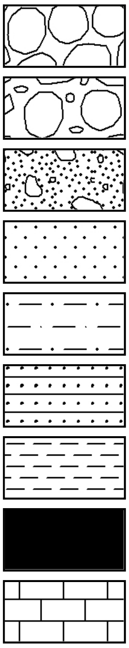
Asphalt
Road base
Concrete
Filling

Soils



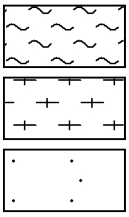
Topsoil
Peat
Clay
Silty clay
Sandy clay
Gravelly clay
Shaly clay
Silt
Clayey silt
Sandy silt
Sand
Clayey sand
Silty sand
Gravel
Sandy gravel
Cobbles, boulders
Talus

Sedimentary Rocks



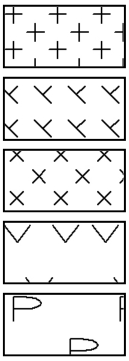
Boulder conglomerate
Conglomerate
Conglomeratic sandstone
Sandstone
Siltstone
Laminite
Mudstone, claystone, shale
Coal
Limestone

Metamorphic Rocks



Slate, phyllite, schist
Gneiss
Quartzite

Igneous Rocks



Granite
Dolerite, basalt, andesite
Dacite, epidote
Tuff, breccia
Porphyry