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Geotechnics / Environment / Groundwater

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Report on
Geotechnical Investigation

Proposed Commercial Development
Site 8B, Sydney Olympic Park

Prepared for
Fitzpatrick Investments Pty Ltd

Project 72233
February 2011

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Geotechnics | Environment | Groundwater

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	Signature	Date
Author		
Reviewer		

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Table of Contents

Report on Geotechnical Investigation
Proposed Commercial Development
Site 8B, Sydney Olympic Park

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed commercial development at Site 8B, located on Murray Rose Avenue within the commercial precinct of the Sydney Olympic Park at Homebush Bay. The work was commissioned by Fitzpatrick Investments Pty Ltd.

The proposed development includes construction of a six-storey commercial building with two levels of basement carparking. The basement carpark will be linked to the existing basement for the adjacent commercial building on Site 8A, which is located to the east of the site. The lowest basement level will require bulk excavation to depths of approximately 6 m to 6.5 m. The investigation was carried out to provide information on the subsurface profile for design and planning purposes.

The investigation included four rock cored boreholes and laboratory testing of selected rock core samples, followed by engineering analysis and reporting. Details of the field and laboratory work are given in the report, together with comments on design and construction practice.

Douglas Partners Pty Ltd (DP) carried out an in situ waste classification of the site in conjunction with the geotechnical investigation. The results have been reported separately (Project No. 72233, dated 11 February 2011).

2. Previous Investigations and Experience

DP previously carried out a geotechnical investigation for Site 8A on the adjacent property to the east of the subject site (Site 8B) and prepared a report dated 5 December 2006 (Project 44346). The investigation included three rock cored boreholes with one of the boreholes (BH1) located close to the north-eastern corner of Site 8B. The borehole log for BH1 is included in Appendix A of this report and the location is shown on Drawing 1 in Appendix B. The surface level shown the log for BH1 is based on an assumed datum and cannot be accurately related to Project Datum. It is presumed that the surface level would have been close to RL116.3 m based on the general topography and surface levels on Site 8B.

DP subsequently carried out geotechnical inspections for the bulk earthworks and anchoring during construction of the two level basement on Site 8A. The subsurface conditions exposed on the western end of the basement, adjacent to Site 8B typically included filling to depths of 2 m to 3 m over stiff residual clay then shale and laminite at depths of approximately 3 m to 4 m. The rock was of extremely low strength grading to medium strength towards the base of the excavation at a depth of approximately 6 m. Some relatively minor seepage was encountered along the top of the clay and rock and also through the rock in the base of the excavation. This seepage was more concentrated on the eastern end of the site where the soil and rock profile was deeper. It is understood that the seepage flows were controlled by sump-and-pump techniques.

3. Site Description and Geology

The site is a rectangular shaped lot with a plan area of 1170 m^2 and a southern frontage to Murray Rose Avenue. At the time of the investigation the site was an asphaltic concrete paved carpark. The site is relatively level with the ground surface level typically varying from between RL 116 m to RL116.4 m relative to Project Datum.

On the adjacent property to the east of the site (Site 8A) is a six-storey commercial building with two levels of basement carparking. Along the western end of the basement, along the common boundary with Site 8B, is a batter slope covered with a protective layer of shotcrete. The batter slope will be removed to link the basements below Site 8A and 8B.

The Olympic Park Train Station is located to the south of the site on the southern side of Murray Rose Avenue. Drawings provided by Bates Smart Pty Ltd indicate that the railway tunnel is located approximately 35 m from the site with the base of the tunnel approximately 10 m below the ground surface. Specific details of the tunnel and its construction have not been confirmed.

Reference to the Sydney 1:100 000 Geological Series Map indicates that the site is underlain by Ashfield Shale which typically comprises black to dark grey shale, siltstone and laminite (finely interbedded siltstone and fine grained sandstone). These rocks typically weather to a residual clay profile of medium to high plasticity. The geological mapping was confirmed by the field work which identified residual soils overlying bedrock including shale, laminite and siltstone.

4. Field Work Methods

The field work for the investigation included four boreholes (BH101 to 104, inclusive) drilled to depths of between 8.7 m to 9 m using a truck-mounted rig. The boreholes were initially drilled to depths of between 3.0 m to 4.0 m using spiral flight augers and rotary drilling within the soils and extremely low to very low strength rock. The boreholes were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the bedrock. Standard Penetration Tests (SPTs) were carried out within the boreholes at 1.5 m depth intervals to sample the soil and weathered rock and to assess the in situ strength of the materials.

The rock cores recovered from the boreholes were returned to the DP office where they were logged by a geologist, the cores photographed and Point Load Strength Index (Is_{50}) tests carried out on selected samples of the rock core.

The ground surface levels at the test locations were interpolated from surface levels shown on the survey plan by Land Partners Pty Ltd (Plan No 72664, dated 17/9/10). These ground surface levels appear to refer to a project datum and not to Australian Height Datum (AHD).

The borehole locations are shown on Drawing 1 in Appendix B.

5. Field Work Results

Details of the subsurface conditions encountered are given in the borehole logs in Appendix A, together with colour photographs of rock core samples and notes defining classification methods and descriptive terms.

The sequence of subsurface materials encountered within the boreholes, in increasing depth order, is described below:

PAVEMENT:	asphaltic concrete 20 mm thick over a 300 mm to 400 mm thick layer of sand and gravel filling;
FILLING:	encountered below the pavement in BH1, 2 and 3 to depths ranging from 0.6 m to 1.6 m. The filling generally comprised clay with some sand and gravel;
CLAY:	stiff clay was encountered below the filling to depths of between 1.3 m to 2.4 m (RL115.0 m to RL113.5 m);
ROCK:	shale and laminite were encountered below the clay at depths of between 1.3 m to 2.4 m (RL115.0 m to RL113.5 m). The rock typically comprised a 1 m to 2 m thick layer of extremely low to very low strength shale over fragmented to fractured, extremely low to medium strength laminite grading to slightly fractured to unbroken laminite below depths of approximately 6 m to 6.5 m (RL109 m to RL110 m).

No groundwater was observed during the auger drilling of the boreholes to depths of 2.5 m. The use of water for rotary drilling and coring prevented observation of groundwater below this depth.

6. Point Load Strength Tests

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the bore logs at the appropriate depth. The Is_{50} values were used to provide an estimate of the unconfined compressive strength (UCS) based on a UCS:Is_{50} ratio of 20:1. The Is_{50} values for the rock typically ranged from 0.2 MPa to 1.5 MPa, corresponding to a low to high strength classification (estimated UCS ranging from 4 MPa to 30 MPa).

7. Proposed Development

Architectural plans by Bates Smart Pty Ltd (dated 21 October 2010) indicate that the proposed development includes the construction of a six-storey commercial building with two levels of basement carparking. The lowest basement level (RL109.95 m) will require bulk excavation to depths of approximately 6 m to 6.5 m. The basement excavation will extend up to the site boundaries.

8. Comments

8.1 Site Preparation and Earthworks

8.1.1 Excavation Conditions

The investigation indicates that excavation for the basement will require the removal of mostly soil and extremely low to medium strength rock. Slightly fractured to unbroken, medium and high strength rock is expected within the deeper parts of the excavation, particularly for the detailed excavation of footings, services and lift pits.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of medium strength bands within the weathered rock sequence. Excavation of low to medium strength rock may require moderate ripping with an excavator whilst excavation of medium and high strength rock will require heavy ripping with a large excavator or bulldozer. Productivity within slightly fractured to unbroken, medium to high strength rock may be low (even with large dozers) and therefore some pre-splitting or rock hammering may be necessary to improve efficiency.

8.1.2 Seepage

Some seepage may be expected along the top of the clay and rock surface and may also occur within fractured zones and joints within the underlying rock profile. Observations made during bulk excavation on Site 8A suggest that seepage flows should be minor during relatively dry periods. Seepage flows are likely to increase following periods of extended wet weather. During construction and in the long term, it is anticipated that seepage flows should be readily controlled by perimeter drains connected to a "sump-and-pump" system. A drained basement will require permanent subfloor drainage below the basement floor slab connected to a sump and pump system with periodic pumping.

8.1.3 Dilapidation Surveys

Consideration should be given to undertaking dilapidation surveys on surrounding buildings and pavements that may be affected by the basement construction. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

8.1.4 Noise and Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s component peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 *"Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)"* indicates an acceptable day time limit of 8 mm/s component PPVi for human comfort.

Based on the experience of DP and reference to AS2670, it is suggested that a maximum component PPVi of 8 mm/sec (applicable at the foundation level of existing buildings) be employed at this site to reduce the risk of structural damage to surrounding buildings.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used. The initial stages of the excavation, during the vibration trial, should be undertaken in the centre of the site to minimise the risk of damage to surrounding structures.

8.1.5 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the current *Waste Classification Guidelines* (DECC, April 2008). Reference should be made to the DP in situ waste classification report (Project No. 72233, dated 11 February 2011) for details on the contamination status and waste classification of the site soils.

8.2 Excavation Support

Vertical excavations within the soils and rock will require both temporary and permanent lateral support during and after excavation. It is anticipated that a bored soldier pile wall with shotcrete infill panels would be suitable. Typically, soldier piles are spaced at approximately 2 m to 3 m centres however closer spaced piles may be required to limit wall movements, or collapse of infill materials, where structures or services are located in close proximity to the excavation. Generally, shotcrete panels should be constructed in 2 m maximum depth intervals within soil and extremely low to very low strength rock, and then in 3 m depth intervals within low to medium strength rock or better.

Shoring piles should be founded on rock at least 1.0 m below the base of the bulk excavation and nearby localised excavation level in order to provide lateral restraint at the base of the excavation. Suitably sized drilling rigs fitted with rock augers will be required to penetrate medium and high strength rock. Shoring piles may be used to carry vertical structural loads and may be designed on the basis of the allowable foundation pressures given in Section 8.3.

The design of the shoring will depend somewhat upon whether it is cantilevered or restrained by rows of temporary rock anchors. It is anticipated that at least one or two rows of rock anchors will be required to provide lateral restraint to shoring piles.

It is suggested that design of shoring systems with a single row of anchors be based on a triangular earth pressure distribution based on earth pressure coefficients provided in Table 1. 'Active' earth pressures (Ka) may be used where some wall movement is acceptable, and 'at rest' earth pressures (Ko) should be used where wall movement needs to be limited. Cantilevered shoring piles are unlikely to be suitable for use with this excavation.

Table 1: Recommended Earth Pressure Coefficients and Bulk Unit Weights

Retained Material	Bulk Density kN/m³	Earth Pressure Coefficient	
		Active (Ka)	At Rest (Ko)
Filling	20	0.4	0.6
Stiff clay	20	0.35	0.5
Extremely to very low strength rock	22	0.2	0.3
Low to medium strength rock	22	0.1	0.2
Medium to high strength rock	24	0	0

All surcharge loads should be allowed for in the shoring design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Preliminary design for lateral earth pressures for walls with more than one row of anchors may be based on a uniform rectangular earth pressure distribution. The additional lateral pressures due to surcharge loading behind the wall and hydrostatic pressures (if appropriate) must also be considered. Where lateral movement is less critical a pressure distribution of 4H may be considered (where H is the depth to the top of the slightly fractured to unbroken, medium to high strength rock). For situations where movements are critical, a higher uniform pressure of 6H is suggested. For detailed design a computer analysis package such as WALLAP, FLAC, PLAXIS or similar should be used to model the excavation and anchoring sequence, to refine the design and provide estimates of possible lateral movements.

The design of temporary and permanent support will also need to consider the possibility that 45° joints in the rock will daylight near the base of the shoring wall leading to wedges of rock which need to be supported by the temporary and permanent retaining structures. The support system would typically comprise rock bolts or anchors spaced at 2 m to 3 m centres over the rock face. These anchors should have their bond lengths formed in rock behind a line projected up at 45° from the base of the shoring. As a guide, the support system should be designed to withstand a horizontal force per unit width of $4.2H^2$ (kN) where H is the height of the excavation in metres. This approximation of the horizontal force required to support a 45° wedge is based on an anchor inclination of 10° below horizontal, an average bulk weight of 21 kN/m^3, and friction angle of 25° and cohesion of 0 kPa along the failure plane. Given that there is a low probability that a joint would run the full length and height of the excavation it suggested that this aspect of design may be carried out for a factor of safety of 1.1.

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to a sump dewatering system.

Passive resistance for piles founded in medium to high strength rock below the base of the excavation (including allowance for detailed excavation for services or footings) may be based on an ultimate passive restraint equal 4000 kPa in medium to high strength or greater rock. A factor of safety must be applied to these ultimate values. The top 0.5 m of the rock socket should be ignored due to possible disturbance and tolerance effects.

Temporary ground anchoring or alternative means of support are required until permanent support can be provided by the completed structure. Ground anchors could be constructed in the conventional manner by drilling and grouting the anchors which may be designed on the basis of the maximum allowable bond stresses given in Table 2.

Table 2: Allowable Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)
Extremely low to very low strength	50
Low to medium strength rock	200
Medium to high strength rock	500

The parameters given in Table 2 assume that drilled holes are clean and adequately flushed. Lift-off tests should be carried out to confirm the bond capacities. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities.

8.3 Foundations

It is expected that the proposed bulk excavation level at approximately RL 109.7 m will expose slightly fractured to unbroken, medium to high strength laminite at, or close to the base of the excavation. On the eastern end of the site the medium and high strength rock may be approximately 0.5 m to 1 m below the bulk excavation level.

All structural loads should be uniformly supported on the underlying medium to high strength rock for which pad footings should be appropriate. Footings founded on medium to high strength rock may be designed for an allowable end bearing pressure of 6000 kPa. Footings proportioned on this basis would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing should be undertaken in at least one-third of the footings which are proportioned on the basis of allowable bearing pressures of greater than 6000 kPa. The purpose of spoon testing is to check that no significant weak seams exist below the base of the footing within a depth equal to 1.5 times the least footing dimension. If the testing identifies the presence of weak seams then the footing will either have to be deepened or widened to reduce the actual bearing pressure.

9. Limitations

Douglas Partners (DP) has prepared this report for this proposed commercial development at Site 8B within the Sydney Olympic Park at Homebush Bay in accordance with DP’s proposal dated12 January 2011. The work was carried out under DP Conditions of Engagement. This report is provided for the exclusive use of Fitzpatrick Investments Pty Ltd and their agents 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 are encountered elsewhere on the site. If 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 "About 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.

Douglas Partners Pty Ltd

Appendix A

About this Report
Results of Field Work

About this Report

Introduction

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

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

Copyright

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.

Soil Descriptions

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Description and Classification Methods

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

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

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

Transported soils may be further subdivided into:

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

July 2010

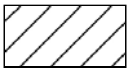
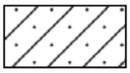
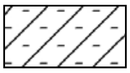
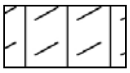
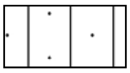
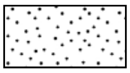
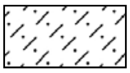
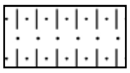
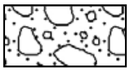
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

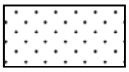
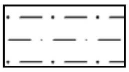
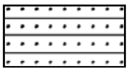
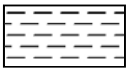
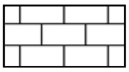
General

■	Asphalt
	Road base
	Concrete
	Filling

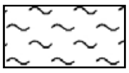
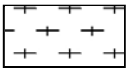
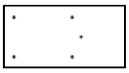
Soils

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

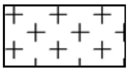
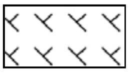
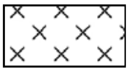
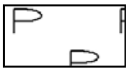
Sedimentary Rocks

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

Metamorphic Rocks

	Slate, phyllite, schist
	Gneiss
	Quartzite

Igneous Rocks

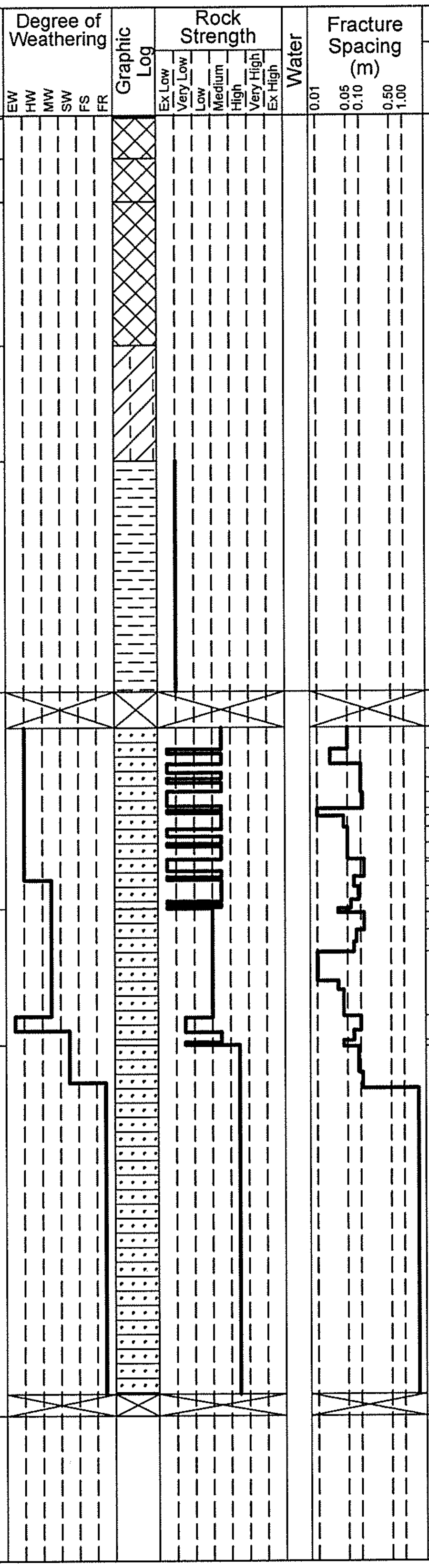
	Granite
	Dolerite, basalt, andesite
	Dacite, epidote
	Tuff, breccia
	Porphyry

BOREHOLE LOG

CLIENT: Fitzpatrick Investments Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: Site 8B, Sydney Olympic Park

SURFACE LEVEL: 116.0 ^
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 72233
DATE: 21/1/2011
SHEET 1 OF 1



RL	Depth (m)	Description of Strata
116	0.02	ASPHALTIC CONCRETE
	0.3	FILLING - brown sand filling with some igneous and sandstone gravel, humid
	0.6	FILLING - brown, slightly gravelly sand filling with some clay, humid
115	1	FILLING - brown silty clay filling with some sand and ironstone and igneous gravel, Mc<PL
	1.6	SILTY CLAY - stiff, light grey and orange brown, silty clay with a trace of ironstone gravel, Mc<PL
114	2	
	2.4	SHALE - extremely low to very low strength, grey shale with some medium strength ironstone bands
113	3	
112	4.0	LAMINITE - alternate bands of extremely low and medium strength, extremely to highly weathered, fragmented to fractured, light grey brown to brown laminite
	4.25	
111	5	
	5.5	LAMINITE - low to medium then medium strength, moderately weathered, fragmented to fractured, brown laminite with some very low strength bands
110	6	
	6.44	LAMINITE - high strength, slightly weathered then fresh, slightly fractured then unbroken, light grey to grey laminite with approximately 25% fine grained sandstone laminations
109	7	
108	8	
	8.85	8.85-9.0m: core left down the hole
107	9.0	Bore discontinued at 9.0m

Discontinuities

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

Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°

4m: CORE LOSS: 250mm
4.25-4.40m: B (x3) 0°, Ds (10-15mm)
4.4-4.5m: 13 (x5) 0°, fe
4.6m: B0°, Ds (30mm)
4.7m: B5°, Ds (50mm)
4.81-4.86m: fg, fe
4.86-4.94m: fe, Ds (10mm)
4.94-5.16m: B (x4) 0°, fe
5.05m: J30°, pl, fe, ro
5.16-5.24m: cly band
5.18m: B0°, 10mm cly
5.35m: J, sv, st, fe, ro
5.44m: B0°, Ds (20mm)
5.53m: J35°, 75°, cu, fe, ro
5.65m: J, sv, fe, ro
5.8m: J35°, fe, ro
5.8-6.0m: DIB
6.24-6.34m: Ds (100mm)
6.41m: B0°, Ds (10mm)
6.45-6.74m: B (x4) 0°, cly, vn
8.85m: CORE LOSS: 150mm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
E*			
E			
E			
S			1,2,3 N = 5
A/E			pp = 150
S			18,28/150mm refusal
			PL(A) = 0.8
C	93	25	
			PL(A) = 0.2
			PL(A) = 1.1
			PL(A) = 1.4
C	93	93	
			PL(A) = 1.5

RIG: DT 100 **DRILLER:** Steve Salib **LOGGED:** SI **CASING:** HW to 4.0m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 4.0m; NMLC-Coring to 9.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Mc = Moisture content; PL = Plastic limit; *Denotes field replicate sample BD2/210111 collected. ^Project Datum

SURVEY DATUM:

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
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DOUGLAS PARTNERS PTY

SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 101 PROJECT 72233 JAN 2011

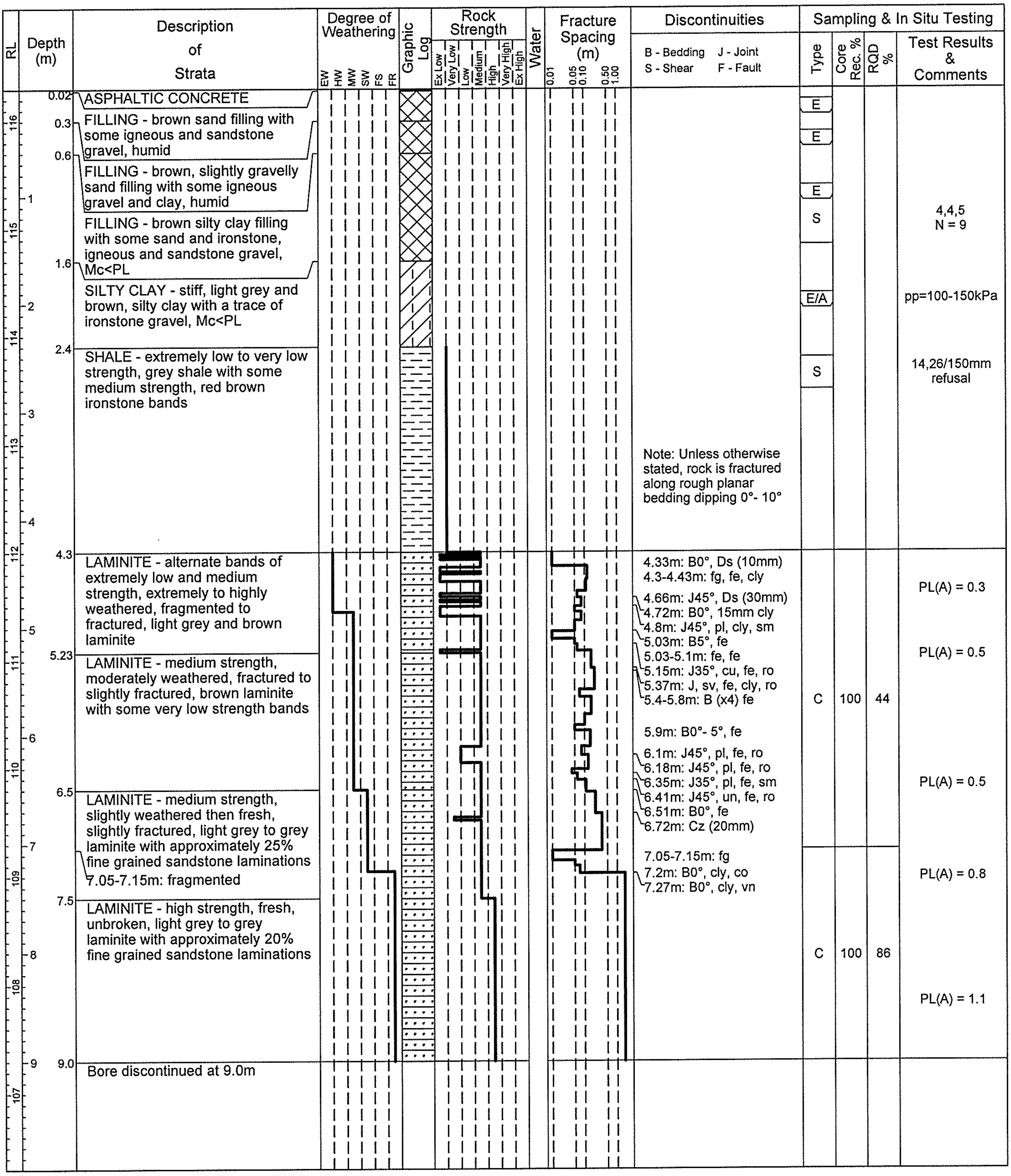
4.0 – 9.0m

BOREHOLE LOG

CLIENT: Fitzpatrick Investments Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: Site 8B, Sydney Olympic Park

SURFACE LEVEL: 116.3 ^
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 72233
DATE: 21/1/2011
SHEET 1 **OF** 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALTIC CONCRETE		E			
0.3	FILLING - brown sand filling with some igneous and sandstone gravel, humid		E			
0.6	FILLING - brown, slightly gravelly sand filling with some igneous gravel and clay, humid		E			
1.6	FILLING - brown silty clay filling with some sand and ironstone, igneous and sandstone gravel, Mc<PL		S			4,4,5 N = 9
	SILTY CLAY - stiff, light grey and brown, silty clay with a trace of ironstone gravel, Mc<PL		E/A			pp=100-150kPa
2.4	SHALE - extremely low to very low strength, grey shale with some medium strength, red brown ironstone bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			14,26/150mm refusal
4.3	LAMINITE - alternate bands of extremely low and medium strength, extremely to highly weathered, fragmented to fractured, light grey and brown laminite	4.33m: B0°, Ds (10mm) 4.3-4.43m: fg, fe, cly 4.66m: J45°, Ds (30mm) 4.72m: B0°, 15mm cly 4.8m: J45°, pl, cly, sm 5.03m: B5°, fe 5.03-5.1m: fe, fe				PL(A) = 0.3 PL(A) = 0.5
5.23	LAMINITE - medium strength, moderately weathered, fractured to slightly fractured, brown laminite with some very low strength bands	5.15m: J35°, cu, fe, ro 5.37m: J, sv, fe, cly, ro 5.4-5.8m: B (x4) fe 5.9m: B0°- 5°, fe 6.1m: J45°, pl, fe, ro 6.18m: J45°, pl, fe, ro 6.35m: J35°, pl, fe, sm	C	100	44	PL(A) = 0.5
6.5	LAMINITE - medium strength, slightly weathered then fresh, slightly fractured, light grey to grey laminite with approximately 25% fine grained sandstone laminations 7.05-7.15m: fragmented	6.41m: J45°, un, fe, ro 6.51m: B0°, fe 6.72m: Cz (20mm) 7.05-7.15m: fg 7.2m: B0°, cly, co 7.27m: B0°, cly, vn				PL(A) = 0.8
7.5	LAMINITE - high strength, fresh, unbroken, light grey to grey laminite with approximately 20% fine grained sandstone laminations		C	100	86	PL(A) = 1.1
9.0	Bore discontinued at 9.0m					

RIG: DT 100 **DRILLER:** Steve Salib **LOGGED:** SI **CASING:** HW to 2.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 4.3m; NMLC-Coring to 9.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Mc = Moisture content; PL = Plastic limit. ^Project Datum

SURVEY DATUM:

SAMPLING & IN SITU TESTING LEGEND

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

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DOUGLAS PARTNERS PTY

SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 102 PROJECT 72233 JAN 2011

4.3 – 9.0m

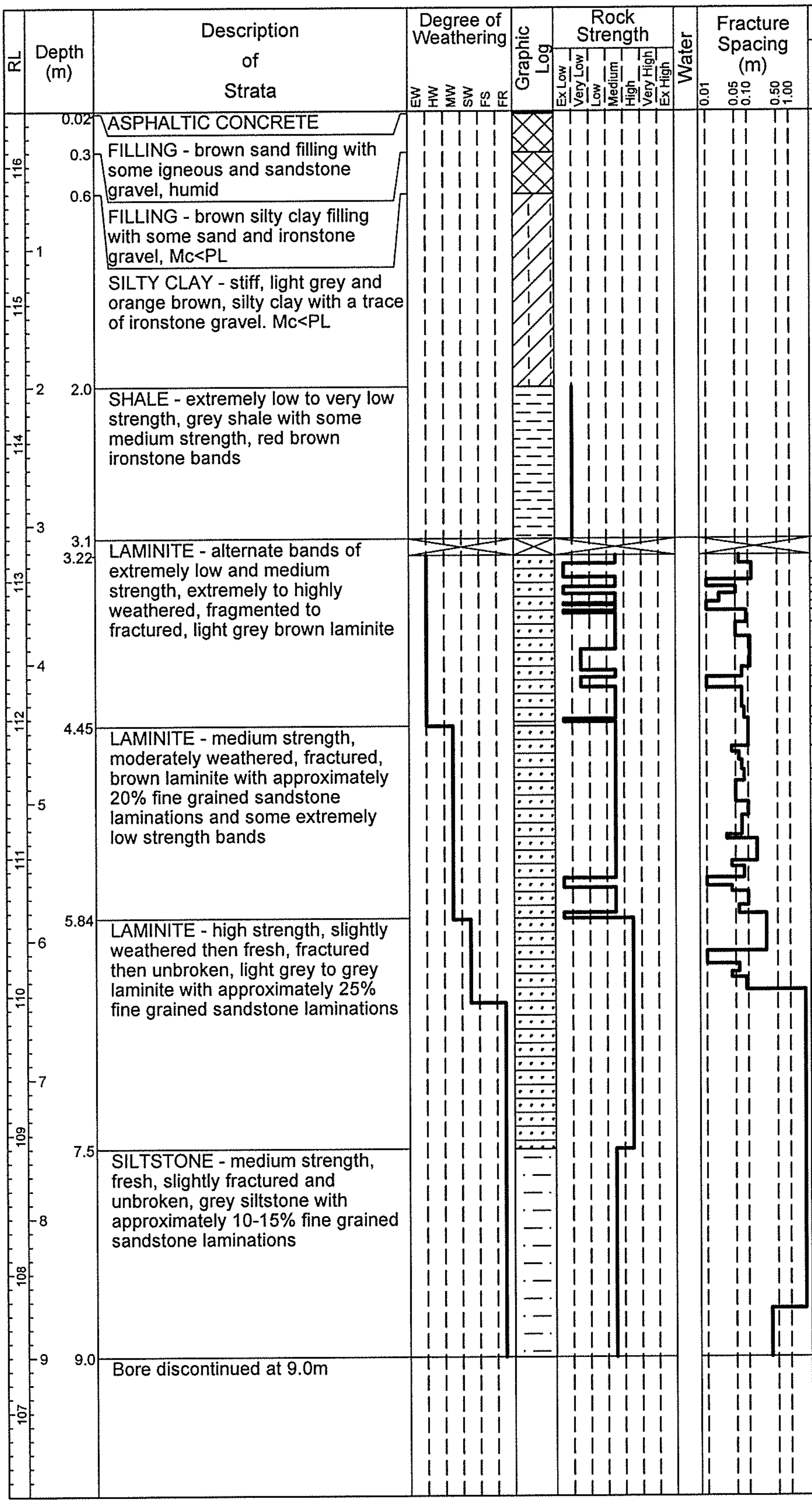
BOREHOLE LOG

CLIENT: Fitzpatrick Investments Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: Site 8B, Sydney Olympic Park

SURFACE LEVEL: 116.4 ^
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 72233
DATE: 20/1/2011
SHEET 1 **OF** 1

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALTIC CONCRETE		A/E			
0.3	FILLING - brown sand filling with some igneous and sandstone gravel, humid		E/A			
0.6	FILLING - brown silty clay filling with some sand and ironstone gravel, Mc<PL		A/E*			
	SILTY CLAY - stiff, light grey and orange brown, silty clay with a trace of ironstone gravel. Mc<PL		S			3,4,4 N = 8
2.0	SHALE - extremely low to very low strength, grey shale with some medium strength, red brown ironstone bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding, dipping 0°- 10°				
3.1 3.22	LAMINITE - alternate bands of extremely low and medium strength, extremely to highly weathered, fragmented to fractured, light grey brown laminite	3.1m: CORE LOSS: 120mm 3.28-3.40m: Ds (120mm) 3.4-3.45m: fg, fe 3.45-3.5m: Ds (50mm) 3.5-3.56m: fg, fe 3.56-3.57m: Ds (10mm) 3.62-3.63m: Ds (10mm) 3.71 & 3.87m: B0°- 5°, fe 3.90-4.03m: Ds band 4.03m: J80°, pl, cly, ro 4.10-4.18m: fg 4.18m: J60°, pl, ro 4.32-4.40m: J30° & 45°, st, fe, ro	C	96	29	PL(A) = 0.6
4.45	LAMINITE - medium strength, moderately weathered, fractured, brown laminite with approximately 20% fine grained sandstone laminations and some extremely low strength bands	4.40-4.42m: Ds (20mm) 4.45m: J30°, pl, fe, ro 4.6m: J60°, pl, fe, sm 4.77m: J60°, pl, fe, sm 4.85-5.55m: B (x10) 0°-10°, fe, cly, un				PL(A) = 0.4 PL(A) = 0.6
5.84	LAMINITE - high strength, slightly weathered then fresh, fractured then unbroken, light grey to grey laminite with approximately 25% fine grained sandstone laminations	5.61 & 5.65m: B0°, Ds (10mm) 5.65-5.81m: J, sv, un, fe, ro 5.81-5.84m: Ds (30mm) 6.08-6.17m: fg 6.17-6.36m: B (x4) 0°, cly, vn	C	100	91	PL(A) = 1.4 PL(A) = 1.4
7.5	SILTSTONE - medium strength, fresh, slightly fractured and unbroken, grey siltstone with approximately 10-15% fine grained sandstone laminations	8.65m: J70°, un, fe, cly, ro				PL(A) = 0.9 PL(A) = 0.7
9.0	Bore discontinued at 9.0m					



RIG: DT 100 **DRILLER:** Steve Salib **LOGGED:** SI **CASING:** HW to 2.6m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.6m; Rotary to 3.1m; NMLC-Coring to 9.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Mc = Moisture content; PL = Plastic limit; *Denotes field replicate sample BD2/200111 collected. ^Project Datum

SURVEY DATUM:

SAMPLING & IN SITU TESTING LEGEND

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

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Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY

SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 103 PROJECT 72233 JAN 2011

3.1 – 8.0m

DOUGLAS PARTNERS PTY LTD

SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 103 PROJECT 72233 JAN 2011

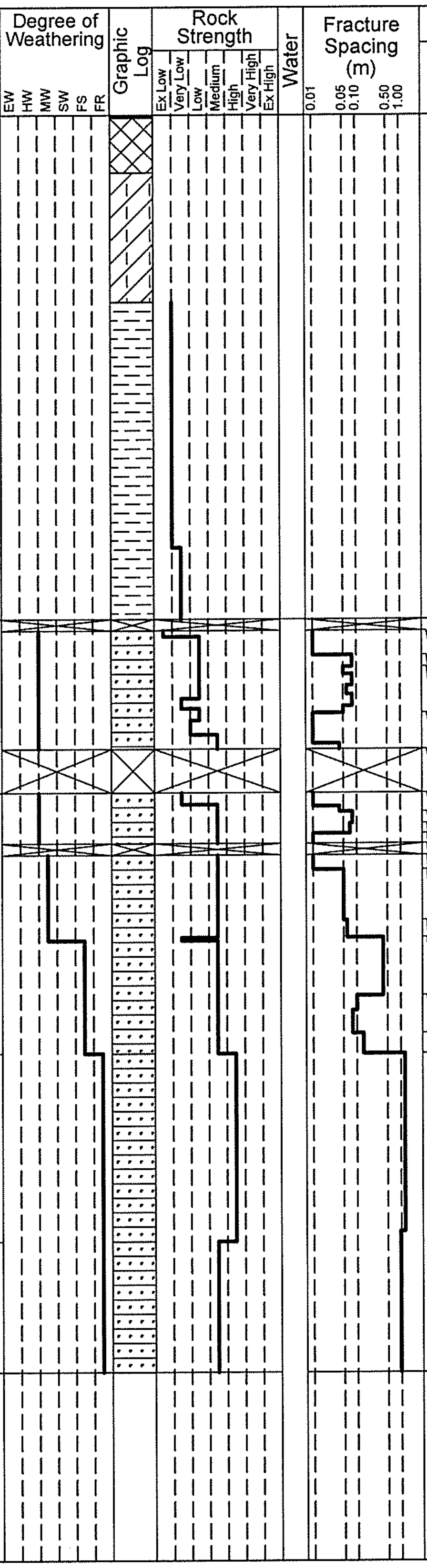
8.0 – 9.0m

BOREHOLE LOG

CLIENT: Fitzpatrick Investments Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: Site 8B, Sydney Olympic Park

SURFACE LEVEL: 116.3 ^
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 72233
DATE: 20/1/2011
SHEET 1 **OF** 1



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	0.02	ASPHALTIC CONCRETE					
116	0.4	FILLING - brown sand filling with some igneous and sandstone gravel, humid		A			
115	1	SILTY CLAY - stiff, light grey and orange brown, silty clay with a trace of ironstone gravel, Mc<PL		A/E A/E S			3,9,23 N = 32
114 113	1.3 2 3	SHALE - extremely low to very low strength, grey shale with some medium strength, red brown ironstone bands - becoming very low strength from approximately 3.0m	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10°	A			
112	3.5 3.58 4	LAMINITE - low strength, highly to moderately weathered, fragmented to fractured, grey brown laminite with approximately 20% fine grained sandstone laminations and some very low strength bands	3.5m: CORE LOSS: 80mm 3.58-3.62m: Ds (40mm) 3.58-3.75m: fg 3.75m: J40°, pl, fe, ro 3.83-4.02m: B (x4) 0°, fe 4.15m: J55°, pl, fe, sm 4.15-4.36m: Dib 4.4m: CORE LOSS: 300mm	C	75	0	PL(A) = 0.2 PL(A) = 0.4
111 110	4.7 5 5.13 6	LAMINITE - medium strength, moderately weathered then fresh stained, fragmented to fractured then slightly fractured, light grey to grey brown laminite with approximately 20% fine grained sandstone laminations	4.7-4.79m: fg, fe 4.83m: J30°, 90°, st, fe, ro 4.91m: J45°, pl, fe, sm 4.98m: B0°, cly, fe 5.05m: CORE LOSS: 80mm 5.13-5.23m: fg 5.23m: J70°, pl, fe, ro 5.23-5.58m: B (x6) 0°-5°, fe, cly 5.58m: J, sv, cu, he, fe 5.7m: B0°, fe, cly 6.10-6.28m: B (x3) 0°, fe 6.36m: J, sv, un, fe, ro	C	96	58	PL(A) = 0.5 PL(A) = 0.9
109	6.5 7	LAMINITE - high strength, fresh, unbroken, grey laminite with approximately 30% fine grained sandstone laminations	6.5m: J45°, un, fe, ro 7.73m: J35°, pl, fe, cly, ro	C	100	100	PL(A) = 1.2
108	7.8 8	LAMINITE - medium strength, fresh, slightly fractured, grey laminite with approximately 20% fine grained sandstone laminations					PL(A) = 0.6
107	8.7 9	Bore discontinued at 8.7m					

RIG: DT 100 **DRILLER:** Steve Salib **LOGGED:** SI **CASING:** HW to 1.6m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.6m; Rotary to 3.5m; NMLC-Coring to 8.7m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Soft bone 4.4m to 4.7m. Mc = Moisture content; PL = Plastic limit. ^Project Datum

SURVEY DATUM:

SAMPLING & IN SITU TESTING LEGEND

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

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DOUGLAS PARTNERS PTY

SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 104 PROJECT 72233 JAN 2011

3.5 – 8.0m

DOUGLAS PARTNERS PTY LTD

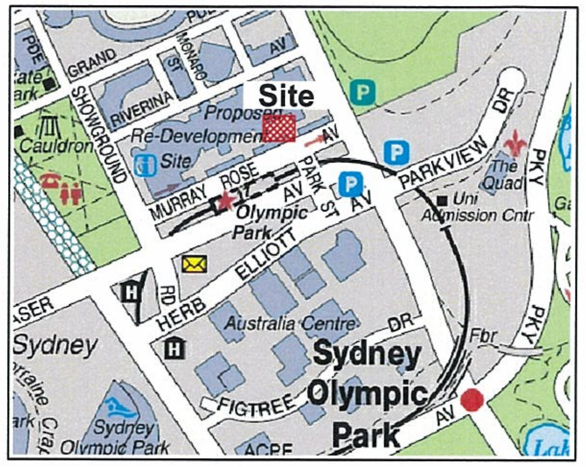
SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH

BORE 104 PROJECT 72233 JAN 2011

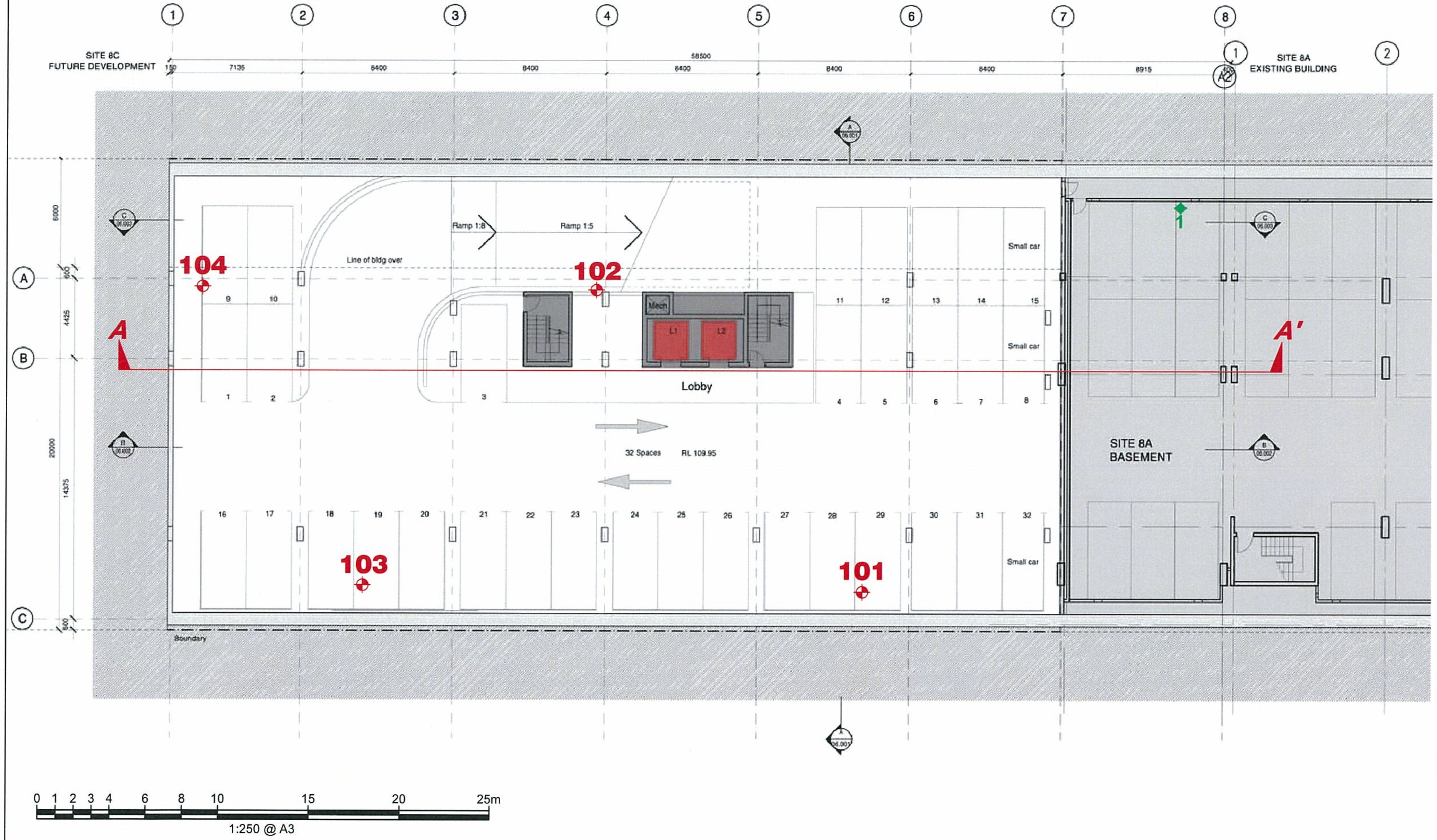
8.0 – 8.7m

Appendix B

Drawing 1 – Location of Boreholes
Drawing 2 –Geotechnical Cross-Section A-A’



Locality Plan



NOTE:
Base drawing from Bates Smart Pty Ltd
(Drawing A02.002[B], dated 16/11/2010)

LEGEND

✦ Previous DP Borehole (Site 8A, Project 44346)

⊕ Current DP Borehole (Site 8B)

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Fitzpatrick Investments Pty Ltd
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: As shown
DATE: 14.2.2011

TITLE: **Location of Tests**
Proposed Commercial Development
Site 8B, Murray Rose Avenue, SYDNEY OLYMPIC PARK

PROJECT No: 72233
DRAWING No: 1
REVISION: A

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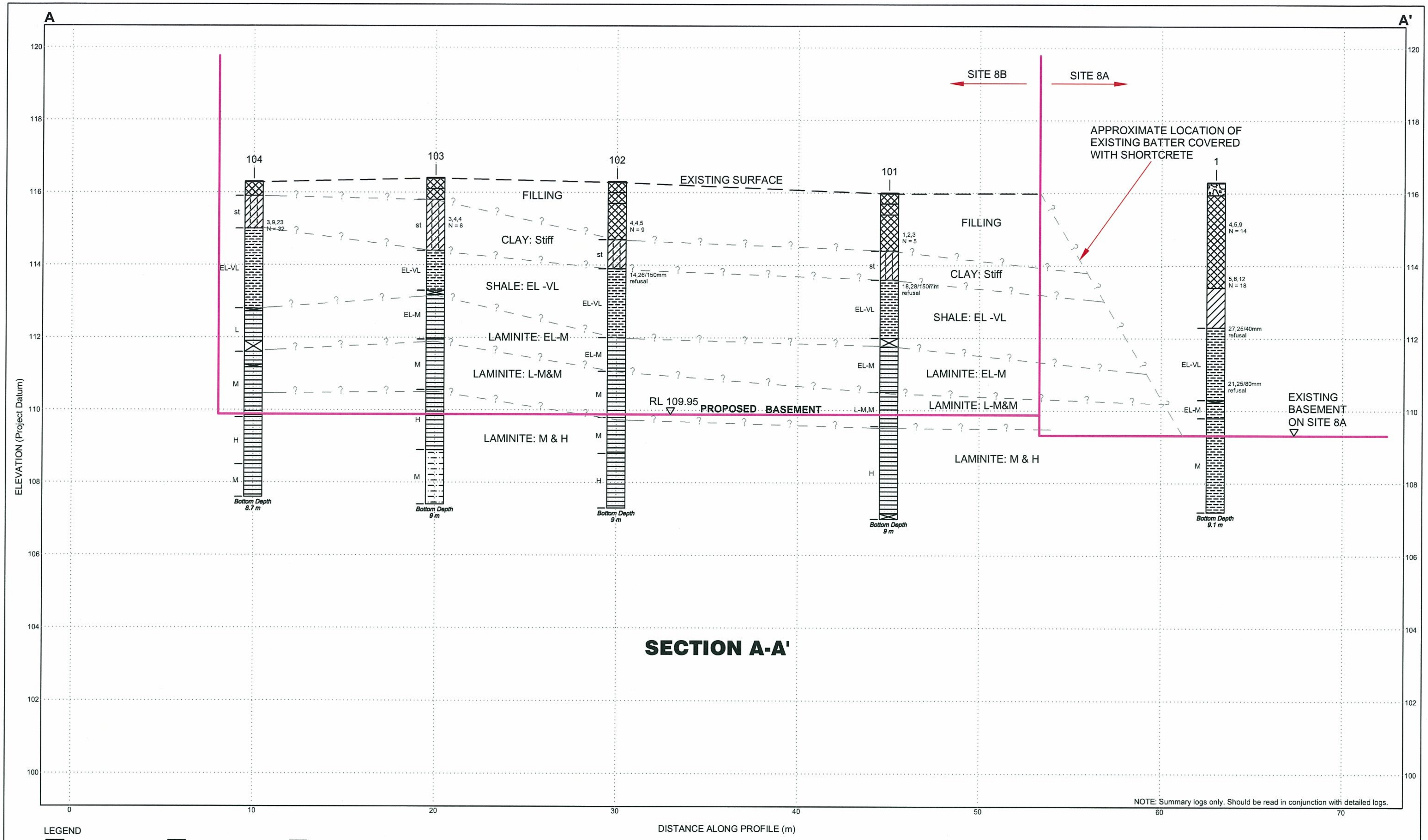

SECTION A-A'

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

LEGEND

- ☒ Core Loss
- ■ Asphaltic Concrete
- ▨ Clay
- ▩ Filling
- ▤ Laminite
- Sandy Gravel
- Shale
- Siltstone
- ▨ Silty Clay

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

SOIL CONSISTENCY
st - stiff

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

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CLIENT: Fitzpatrick Investments Pty Ltd		
OFFICE: Sydney	DRAWN BY: STE/LD	
SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 24.02.2011	

TITLE: **Geotechnical Cross Section A-A'**
Proposed Commercial Development
Site 8B, Sydney Olympic Park, Homebush Bay

PROJECT No:	72233
DRAWING No:	2
REVISION:	1

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