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geotechnical engineering consultants

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Urban Apartments Pty Ltd

Proposed Mixed Development
11–15 Deane St & 20 George St, Burwood

Report on
Preliminary Geotechnical Investigation

BURWOOD COUNCIL
211-10 25 FEB 2011

1623-A
19 February 2011

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1623-A
19 February 2011

Urban Apartments Pty Ltd
Shop 8, 338 Liverpool Road
ENFIELD NSW 2136

Attention: Mr George Elias

Dear Sir,

PROPOSED MIXED DEVELOPMENT, 11-15 DEANE ST & 20 GEORGE ST, BURWOOD
PRELIMINARY GEOTECHNICAL INVESTIGATION

We are pleased to present our report on a preliminary geotechnical investigation carried out for the above project.

This report documents field and laboratory investigations and provides discussion and preliminary recommendations for geotechnical aspects of design and construction for the proposed development.

Please contact us if you have any questions regarding this report or if you require further assistance.

For and on behalf of
Asset Geotechnical Engineering Pty Ltd

Mark Bartel

Mark Bartel
BE MEngSc MIEAust CPEng
Principal Geotechnical Engineer

BURWOOD COUNCIL
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Contents

1. INTRODUCTION

1.1 General

This report presents the results of a preliminary geotechnical investigation to support a development application for a proposed mixed commercial and residential development at the above site. The investigation was commissioned by Mister George Elias of Urban Apartments Pty Ltd. The work was carried out in accordance with a proposal by Asset Geotechnical Engineering Pty Ltd dated 19 January 2011, reference P1781.

This report should be read in conjunction with the attached Information Sheets. Particular attention is drawn to the limitations inherent in site investigations and the importance of verifying the subsurface conditions inferred herein.

1.2 Project Summary Details

It is understood that the project involves a residential / commercial development with 4 basement levels for carparking and 16 stories above. Excavation of up to about 15m depth is anticipated for the basement. It is also understood that a railway corridor (including the Burwood Rail Station) is loated on the southern side of Deane Street, and the edge of the excavation appears to be located about 12 m from the rail corridor. The Burwood Rail Station comprises above-ground railway tracks and a railway platform.

1.3 Scope of Work

The objectives of the investigation were to:

A. Provide preliminary information on the surface and subsurface conditions for preliminary design of the proposed excavations, retaining structures and footings, to support a development application.
B. Address RailCorp's requirements with respect to potential impacts on their rail infrastructure.

In order to achieve these objectives, the following scope of work was carried out.

A – Geotechnical Investigation and Reporting

- Review of available reports and maps held within our files.
- Walkover observations of site conditions.
- Drilling and logging of 1 borehole using a truck-mounted rig.
- Laboratory testing, comprising point load strength index testing of recovered rock core.
- Engineering assessment and reporting.

B – RailCorp Requirements

The risk that the proposed development poses to the adjacent rail corridor and associated infrastructure relates to potential instability and / or movement associated with excavation and temporary / permanent shoring works. In order to address RailCorp's requirements, a preliminary slope instability risk assessment has been carried out using the Australian Geomechanics Society (AGS) *Landslide Risk Management*[1], addressing the risk to property only at this stage. Geotechnical input is also provided for preliminary design and construction of temporary shoring and permanent excavation support, to ensure that deflections and subsidence within the RailCorp corridor is within acceptable limits.

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[1] *Landslide Risk Management, Australian Geomechanics, Vol 42, No. 1, March 2007.*

2. FIELDWORK AND LABORATORY TESTING

2.1 Borehole Investigation

The borehole (BH1) was drilled on 1st February 2011 using a truck-mounted drilling rig. The test location is shown on the attached Figure 3.

The borehole was auger drilled to a depth of 4.3 m with Standard Penetration Testing carried out at selected depth intervals to aid with assessment of in situ conditions. The borehole was then cored to a termination depth of 17.5 m.

On completion of logging and sampling, a standpipe piezometer was constructed to allow measurement of groundwater. The piezometer construction comprised a 50 mm PVC pipe hand slotted over the bottom 6 m (i.e. from 11.5 m to 17.5 m depth). The annulus was backfilled with 2 mm size washed sand to a depth of 4 m below ground surface, then a 0.5 m thick bentonite plug / spoil / 0.5 m thick bentonite plug placed above that to within about 0.3 m depth of the ground surface. The piezometer was finished with a cast-iron road box concreted in-place and set flush to the adjacent ground surface.

The test location was set out by our engineer and was located by tape measurements from existing site features. The subsurface conditions encountered were recorded during the progress of the drilling and testing. Recovered rock core was retained for photography and subsequent laboratory testing. The surface level at the test location was assessed from spot levels shown on the supplied plans.

Engineering logs and explanatory notes are attached to this report.

2.2 Laboratory Testing

The recovered rock core was photographed and then delivered to a NATA registered laboratory for point load strength index testing. The test results are attached and are also incorporated on the engineering borehole log. A plot of the test data is also included in Section 4.4.

3. SITE DESCRIPTION

The site is located on the northern side of Deane Street in Burwood, as shown in the attached Figures 1 and 2. It is bounded by Deane Street to the south, Mary Street to the west, George Street and a residential unit building to the north, and a high-rise commercial building to the east. The Burwood railway station is located to the south of Deane Street.

Existing site development comprises four residential unit buildings, with three along Deane Street (numbers 11 to 15) and one on George Street (number 20). The residential unit buildings are of two-story brick construction and appear to be in generally moderate to good condition for their age, estimated at greater than about 30 to 40 years. Associated site development comprises a mixture of concrete and other paving and some vegetation including grass and bushes.

The adjoining building to the east is of concrete frame and blockwork infill construction and has five stories above ground with a partly-buried basement level carpark accessed off of George Street. It appears that the basement level is up to about 2 m below the existing ground surface.

Plate 1 below shows the residential unit buildings along Deane Street and the high-rise commercial building to the east.

Plate 1 – view of Deane Street buildings and commercial building to the east

The regional topography comprises gently sloping terrain. The overall ground surface slopes down to the northwest at less than about 5°.

A brick retaining wall up to about 1.5 m high is located along the western part of the northern boundary of the rail corridor, and an electricity substation is located within the eastern part (see Plate 2). The ground surface rises up to the south at about 15° towards a railway platform. The embankment between the platform and the retaining wall appears to comprise soil filling. The brick retaining wall is in overall moderate condition with some cracking observed.

Plate 2 - view of railway corridor showing electricity substation and brick retaining wall

A brick wall located within the corridor just north of the platform is in poor condition with significant cracking observed (see Plate 3). The brick work at the base of the wall indicates an archway which may have been subsequently buried or backfilled.

Plate 3 – view of damaged brick wall within railway corridor adjacent to railway platform

4. SUBSURFACE CONDITIONS

4.1 Geology

The 1:100,000 Sydney Geological Map indicates the site is underlain by Ashfield Shale, which includes shale with seams of siltstone and sandstone. These rocks typically weather to form residual clay soils of medium to high plasticity.

4.2 Stratigraphy

The following summary description is provided for the conditions observed at the test location for this investigation. The detailed conditions at the test location are recorded on the attached logs. For specific design input, reference should be made to the logs and/or the specific test results, in lieu of the following summary.

Table 1 – Generalised Subsurface Profile (BH1)

Layer	Description	Thickness (m)	Depth to Base (m)
Pavement	CONCRETE	0.05	0.05
Residual	CLAY and Silty CLAY, medium to high plasticity, moisture >>Wp and soft to confirm to about 0.8 m depth, moisture >Wp and stiff to very stiff to about 1.7 m, moisture <=Wp and hard below 1.7m	2.05	2.1
Bedrock	SILTSTONE, extremely weathered, remolds to Sandy CLAY, medium plasticity, moisture < Wp, hard	1.4	3.5
	CLAY with SILTSTONE bands, clay is medium plasticity, moisture >Wp, stiff to very stiff, siltstone is HW, VL to L strength,	1.7	5.2
	SANDSTONE / SILTSTONE, fine grained, CW / XW, EL to VL strength, highly fractured to fractured (assessed Class 5 Sandstone[2])	3.1	8.3
	SHALE / SILTSTONE, fine grained, MW, EL to L strength, highly fractured to fractured (assessed Class 5 / 4 shale)	0.7	9.0
	SHALE / SILTSTONE, fine grained, SW, L to M strength, highly fractured to fractured (assessed Class 4 / 3 Shale)	5.0	14.0
	SHALE / SILTSTONE, fine grained, FR, L to M strength, fractured (assessed Class 3 Shale)	3.5	17.5

It is noted that the shale bedrock below about 8.3 m depth exhibited significant spalling on exposure. This will require consideration with respect to the design and construction of temporary shoring.

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[2] Pells, P.J.N., Mostyn, G. & Walker, B.F., *Foundations on Sandstone and Shale in the Sydney Region*, Australian Geomechanics Journal, December 1998

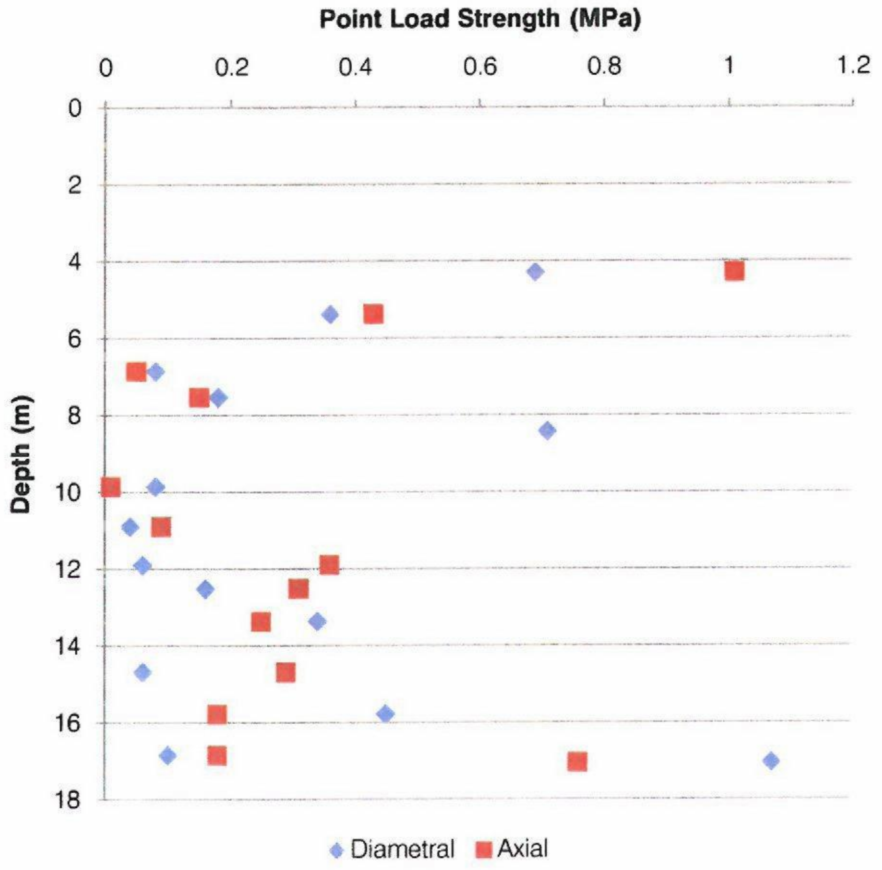
4.3 Groundwater

The piezometer installed during the fieldwork was bailed out at approximately 7:30 PM on the date of the fieldwork (1st February 2011). The water level was recorded at 7.3 m depth at 11:50 AM on the 3rd February 2011. It must be noted that fluctuations in groundwater level can occur due to climatic factors (i.e. rainfall) and other factors (e.g. leaking services).

4.4 Laboratory Test Results

The point load strength index test results are attached and indicate values ranging from 0.01 MPa to 1.07 MPa (i.e. extremely low to medium strength). It should be noted that the diametral tests typically (but not always) failed along bedding planes and the axial tests failed through the rock fabric. The axial tests are therefore considered more representative of the rock substance strength.

A plot of the diametral and axial test results with depth is presented below.



5. DISCUSSIONS & RECOMMENDATIONS

5.1 Excavation Support

It is understood that that the proposed basement level is to extend to the site boundaries. Therefore, temporary and permanent excavation support will be required.

Design of excavation support will need to consider both long–term (i.e. permanent) and short–term (i.e. during construction) loading conditions, as well as the possible impact on adjoining developments.

In the long–term, the floor slabs will provide bracing at the bottom and top of the basement retaining walls, as well as in-between, and therefore the basement retaining walls should be designed as a braced wall for the long–term loading condition.

In the short–term (i.e. during construction), the design of the basement retaining wall will depend on the method of construction adopted. Two common construction techniques include top–down and bottom–up construction.

Top–down construction typically involves:

- construction of the perimeter wall as either contiguous bored piles or semi-contiguous bored piles with shotcrete infill panels;
- construction of internal columns as bored piles;
- pouring the ground floor slab (or sufficient sections of the ground floor slab to provide adequate bracing);
- excavating below the ground floor slab down to subgrade level; and
- pouring the basement floor slab.

Bottom–up construction typically involves:

- constructing the perimeter wall as either contiguous bored piles or semi-contiguous bored piles with shotcrete infill panels;
- options for wall design include cantilever, anchored (rock anchors), and propped (internal props);
- excavating to basement level (installing shotcrete infill panels if semi-contiguous bored piles adopted);
- pouring the ground floor slab and proceeding upwards.

It must be noted that the shale bedrock exhibited noticeable signs of spalling on exposure and therefore if semi-contiguous bored piles are adopted, it will likely be necessary to provide shotcrete infill panels in-between the piles and extending down to the base of the excavation.

In view of the proximity to adjacent structures, particularly the railway corridor to the south, and the regular footprint of the development, we recommend that top-down construction be considered for this site. This would minimise the risk of lateral deflection of the wall and subsidence of adjacent ground, compared with bottom-up construction.

If bottom-up construction is considered, we recommend the use of internal propped walls or anchored walls where the retained height is 2m or more, and either internal propped walls or cantilever walls where the retained height is less than 2m.

Cantilever retaining walls may be designed for a lateral earth pressure coefficient (K_a) of 0.3 where adjacent footings and services are located below the "line of influence", and K_a of 0.45 elsewhere. These coefficients are applicable for the soils overlying bedrock and are also recommended for the assessed Class 5 Sandstone. A reduced K_a value of 0.15 is recommended for the rock below assessed Class 5 Sandstone.

The "line of influence" is defined as a line extending upwards and outwards at 45° above horizontal from the top of the slightly weathered shale/siltstone or the base of the excavation, whichever is higher. The attached Figure 4 illustrates this "line of influence" relative to the southern boundary.

Braced retaining walls may be designed for a uniform lateral earth pressure of $0.65 * \gamma * H * K_a$ where γ = unit weight of retained soil/rock (say 18kN/m^3), H = height of wall, and K_a = earth pressure coefficient (0.3 or 0.45 or 0.15 as noted above).

Subsoil drainage should be provided for the temporary shoring. The permanent basement should be tanked and should allow for a groundwater table to avoid the need for continuous dewatering. In the absence of long-term monitoring, it is suggested that a groundwater level nominally 5 m below existing surface level be adopted for preliminary design. It is also recommended that the piezometer installed during this preliminary investigation be further monitored to gather more data on groundwater fluctuations and to confirm the design recommendations.

Appropriate surcharge loading at the finished surface level should also be adopted for design of the wall.

It is noted that the above lateral earth pressures recommended above do not necessarily allow for the presence of inclined joints/failure planes within the bedrock. It is recommended that, prior to excavation, additional cored boreholes be drilled close to the southern boundary of the subject site in order to further assess the presence of potential joints/failure planes, for detailed design of temporary shoring and permanent support.

Design for the temporary and permanent excavation support should be reviewed by a geotechnical engineer to ensure that the design recommendations have been incorporated.

5.2 Excavation Methodology

The excavation for the proposed basement level is anticipated to be through a mixture of clay soils and extremely to completely weathered siltstone and sandstone to about 9 m depth, and then into moderately weathered and slightly weathered shale/siltstone. The rock is likely to be continuous across adjoining properties. Excavation requirements will be governed by the presence of the rock, and the sensitivity of nearby structures, rail infrastructure, and possible buried services, to vibrations caused by the rock excavation.

The building constructions on the adjacent properties are sensitive to vibrations above certain threshold levels (regarding potential for cracking). The proposed excavation is immediately adjacent to the commercial building to the east, is relatively close to the existing residential development to the north, and is about 10 m from the rail corridor to the south. Close controls by the excavation contractor over the rock excavation are therefore necessary, and are recommended, so that excessive vibration effects are not generated.

Excavation methods should be adopted which limit ground vibrations at the adjoining developments to not more then 10mm/sec. Vibration monitoring will be required to verify that this is achieved. However, if the contractor adopts methods and / or equipment in accordance with the recommendations in Table 2 for a ground vibration limit of 5mm/sec, vibration monitoring may not be required.

The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavation methods are restricted as indicated in Table 2 as follows:

Table 2 – Recommendations for Rock Breaking Equipment

Distance from adjoining structure (m)	Maximum Peak Particle Velocity 5mm/sec		Maximum Peak Particle Velocity 10mm/sec*	
	Equipment	Operating Limit (% of Maximum Capacity)	Equipment	Operating Limit (% of Maximum Capacity)
1.5 to 2.5	Hand operated jackhammer only	100	300 kg rock hammer	50
2.5 to 5.0	300 kg rock hammer	50	300 kg rock hammer or 600 kg rock hammer	100 50
5.0 to 10.0	300 kg rock hammer or 600 kg rock hammer	100 50	600 kg rock hammer or 900 kg rock hammer	100 50

* Vibration monitoring is recommended for 10mm/sec vibration limit.

At all times, the excavation equipment must be operated by experienced personnel, according to the manufacturer's instructions, and in a manner consistent with minimising vibration effects.

Use of other techniques (e.g. chemical rock splitting, rock sawing), although less productive, would reduce or possibly eliminate risks of damage to adjoining property through vibration effects transmitted via the ground. Such techniques may be considered if an alternative to rock breaking is necessary. If rock sawing is carried out around excavation boundaries in not less than 1m deep lifts, a 900 kg rock hammer could be used at up to 100% maximum operating capacity with an assessed peak particle velocity not exceeding 5 mm/sec, subject to observation and confirmation by a geotechnical engineer at the commencement of excavation.

It should be noted that vibrations that are below threshold levels for building damage may be experienced at adjoining developments.

5.3 Footings

Based on the profile encountered in BH 1, it is assessed that slightly weathered or fresh shale/siltstone bedrock (assessed Class 3 / 4 or Class 3) would be encountered at bulk excavation level for the proposed basement. Suitable footings could comprise strip and pad footings, or piles founded within these materials.

Strip and pad footings and rock socketed piles may be designed for the parameters in Table 3 below:

Table 3 – Footing Design Parameters

Founding Stratum	**Maximum allowable design values**		
	End Bearing (kPa)	**Shaft Friction – Compression (kPa)**	**Shaft Friction – Tension (kPa)**
Class 4/ 3 Shale	1,000	150	100
Class 3 Shale	2,500	250	150

Groundwater may be expected within bored pile holes and dewatering by down-hole pump may be required to limit softening of the bases prior to concreting.

An experienced geotechnical engineer should review footing designs to check that the recommendations of the geotechnical report have been included, and should assess footing excavations to confirm the design assumptions.

5.4 Potential Impact on Rail Infrastructure

The proposed development could potentially impact the adjacent rail corridor and associated infrastructure. Impacts relate to potential instability and / or movement associated with excavation and temporary / permanent shoring works.

The attached Figure 4 illustrates an interpreted section through the site extending to the south to include the railway platform. Details within the railway property are indicative only but are considered to be adequate for the purposes of this discussion.

The recommended strategy considered appropriate for the site in order to reduce the risk of potential impacts on the adjacent rail corridor to an acceptable level, is to provide temporary shoring by means of contiguous anchored piles (or semi-contiguous anchored piles with shotcrete infill panels), and permanent support by means of the basement structure. Design recommendations have been provided in the previous sections.

Discussion on slope instability risk is presented in Section 5.4.1. Discussion on potential settlements is presented in Section 5.4.2.

5.4.1 Slope Instability Risk

A limited, preliminary level, risk assessment has been carried out for this site with regard to slope instability, using the methods of the AGS publication *"Landslide Risk Management"*, (Reference 1).

The basis of the preliminary assessment undertaken for this site and important factors relating to slope conditions and the impacts of the development that commonly influence the risks of slope instability are discussed in the attached "Important Information about your Slope Instability Risk Assessment".

The preliminary assessment has been carried out by:

- Consideration of the likely slope failure mechanisms and the likely initiating circumstances that could affect the elements at the site. The type and mode of landslide failure has also been classified.
- **Risk to Property.** For each case, the likely consequences with respect to future development have been considered. The current assessed probability of occurrence of each event has been estimated on a qualitative basis. The consequences and probability of occurrence have been combined for each case to provide the risk assessment.

The following general potential hazards/events are identified for this site and relate to slope instability:

A. slump of excavation

For the hazards / events identified, the elements that are at risk are the proposed development and adjacent site developments comprising buildings, services, and the RailCorp corridor. Table A provides our preliminary risk assessment for the site with respect to risk to property.

Where the proposed temporary support is not designed and constructed in accordance with appropriate engineering standards, a **High risk** is assessed with respect to property. This risk level is considered to be unacceptable.

However, where the proposed temporary support is designed and constructed in accordance with appropriate engineering standards, a **Low risk** is assessed with respect to property. This risk level is considered to be acceptable.

For the purposes of this preliminary assessment, Risk to Life has not been considered. Where design and construction of the development satisfies RailCorp's requirements with respect to potential impact on their rail corridor, it is considered that this would result in an **Acceptable risk** level with respect to life. Further assessment would be needed if it is required to confirm this.

5.4.2 Settlement / Defelections

Design of temporary shoring and permanent support using the recommended lateral earth pressures in this report should ensure that lateral deflections and settlements at the southern boundary of the site are relatively small. Further analysis would be required to quantify the magnitude of deflections and settlement. For the purposes of this preliminary investigation, it is considered likely that such deflections and settlements would be less than about 5 mm, provided that temporary shoring and permanent support is designed in accordance with the recommendations in this report.

It is noted that the northern boundary of the RailCorp property is located about 11 m south of the subject site. Deflections and settlements along the northern boundary are expected to be substantially less than at the southern site boundary, and it is expected there would be negligible impact on the adjoining rail corridor.

Notwithstanding the above, it is recommended that an inclinometer be installed within Deane Street close to the RailCorp property boundary, to allow monitoring during excavation and construction of temporary shoring works for the proposed development. In the event that "significant" deflections occur, action could be taken to increase the lateral support being provided to the excavation (e.g. temporary berm and then additional rock bolting). It is recommended that the magnitude of "significant" deflections that would trigger further action be agreed with RailCorp prior to construction commencing.

5.4.3 *Statement*

Provided that design and construction of the proposed development is carried out in accordance with the recommendations provided in this preliminary report, and in accordance with subsequent recommendations provided during detailed investigations, we consider that the development would not adversely impact the integrity of the adjacent rail corridor. This would be confirmed in writing at the completion of the detailed design phase.

❖ ❖ ❖ ❖ ❖

For and on behalf of
Asset Geotechnical Engineering Pty Ltd

Mark Bartel

Mark Bartel
BE MEngSc MIEAust CPEng
Principal Geotechnical Engineer

Information Sheets

- Important Information
- Abbreviations, Notes & Symbols
- Soil & Rock Terms
- Important Information about Your Slope Instability Risk Assessment

SCOPE OF SERVICES

The geotechnical report ("the report") has been prepared in accordance with the scope of services as set out in the contract, or as otherwise agreed, between the Client and Asset Geotechnical Engineering Pty Ltd ("Asset"). The scope of work may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

RELIANCE ON DATA

Asset has relied on data provided by the Client and other individuals and organizations, to prepare the report. Such data may include surveys, analyses, designs, maps and plans. Asset has not verified the accuracy or completeness of the data except as stated in the report. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations ("conclusions") are based in whole or part on the data, Asset will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to Asset.

GEOTECHNICAL ENGINEERING

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared for a specific client, for a specific project and to meet specific needs, and may not be adequate for other clients or other purposes (e.g. a report prepared for a consulting civil engineer may not be adequate for a construction contractor). The report should not be used for other than its intended purpose without seeking additional geotechnical advice. Also, unless further geotechnical advice is obtained, the report cannot be used where the nature and/or details of the proposed development are changed.

LIMITATIONS OF SITE INVESTIGATION

The investigation programme undertaken is a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions. The data derived from the site investigation programme and subsequent laboratory testing are extrapolated across the site to form an inferred geological model, and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation, the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The engineering logs are the subjective interpretation of subsurface conditions at a particular location and time, made by trained personnel. The actual interface between materials may be more gradual or abrupt than a report indicates.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

Subsurface conditions can be modified by changing natural forces or man-made influences. The report is based on conditions that existed at the time of subsurface exploration. Construction operations adjacent to the site, and natural events such as floods, or ground water fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. Asset should be kept appraised of any such events, and should be consulted to determine if any additional tests are necessary.

VERIFICATION OF SITE CONDITIONS

Where ground conditions encountered at the site differ significantly from those anticipated in the report, it is a condition of acceptance of the report that Asset be notified of any variations and be provided with an opportunity to review the recommendations of this report. Recognition of change of soil and rock conditions requires experience and it is recommended that a suitably experienced geotechnical engineer be engaged to visit the site with sufficient frequency to detect if conditions have changed significantly.

REPRODUCTION OF REPORTS

This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of this Company. Where information from the accompanying report is to be included in contract documents or engineering specification for the project, the entire report should be included in order to minimize the likelihood of misinterpretation from logs.

REPORT FOR BENEFIT OF CLIENT

The report has been prepared for the benefit of the Client and no other party. Asset assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of Asset or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own inquiries and obtain independent advice in relation to such matters.

OTHER LIMITATIONS

Asset will not be liable to update or revise the report to take into account any events or emergent circumstances or fact occurring or becoming apparent after the date of the report.

Abbreviations, Notes & Symbols

METHOD

borehole logs

AS	auger screw *
AD	auger drill *
RR	roller / tricone
W	washbore
CT	cable tool
HA	hand auger
D	diatube
B	blade / blank bit
V	V-bit
T	TC-bit

* bit shown by suffix e.g. ADV

excavation logs

NE	natural excavation
HE	hand excavation
BH	backhoe bucket
EX	excavator bucket
DZ	dozer blade
R	ripper tooth

coring

NMLC, NQ, PQ, HQ

SUPPORT

borehole logs

N	nil
M	mud
C	casing
NQ	NQ rods

excavation logs

N	nil
S	shoring
B	benched

CORE—LIFT

| | casing installed

├─┤ barrel withdrawn

NOTES, SAMPLES, TESTS

D	disturbed
B	bulk disturbed
U50	thin-walled sample, 50mm diameter
HP	hand penetrometer (kPa)
SV	shear vane test (kPa)
DCP	dynamic cone penetrometer (blows per 100mm penetration)
SPT	standard penetration test
N*	SPT value (blows per 300mm) * denotes sample recovered
Nc	SPT with solid cone
R	refusal of DCP or SPT

USCS SYMBOLS

GW	Well graded gravels and gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
GC	Clayey gravels, gravel-sand-clay mixtures.
SW	Well graded sands and gravelly sands, little or no fines.
SP	Poorly graded sands and gravelly sands, little or no fines.
SM	Silty sand, sand-silt mixtures.
SC	Clayey sand, sand-clay mixtures.
ML	Inorganic silts of low plasticity, very fine sands, rock flour, silty or clayey fine sands.
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
OL	Organic silts and organic silty clays of low plasticity.
MH	Inorganic silts of high plasticity.
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity.
PT	Peat muck and other highly organic soils.

MOISTURE CONDITION

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

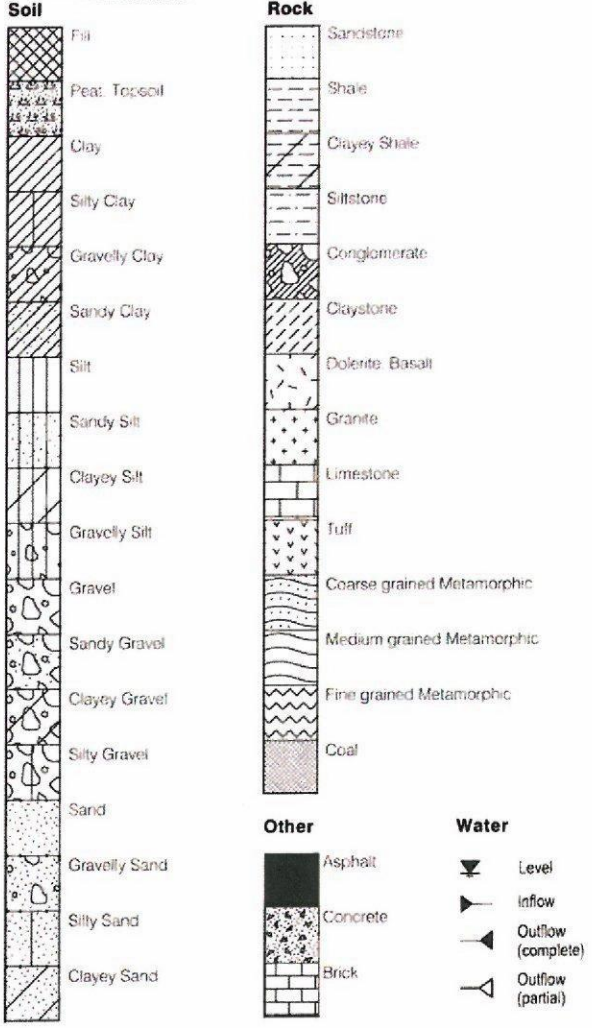
CONSISTENCY

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable

DENSITY INDEX

VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

GRAPHIC LOG



Boundaries

———— known ------ probable .... possible

WEATHERING

XW	extremely weathered
HW	highly weathered
MW	moderately weathered
SW	slightly weathered
FR	fresh

STRENGTH

EL	extremely low
VL	very low
L	low
M	medium
H	high
VH	very high
EH	extremely high

RQD (%)

$$= \frac{\text{sum of intact core pieces > 2 x diameter}}{\text{total length of section being evaluated}} \times 100$$

DEFECTS

type

JT	joint
PT	parting
SZ	shear zone
SM	seam

coating

cl	clean
st	stained
ve	veneer
co	coating

shape

pl	planar
cu	curved
un	undulating
st	stepped
ir	irregular

roughness

po	polished
sl	slickensided
sm	smooth
ro	rough
vr	very rough

inclination

measured above axis and perpendicular to core

AS1726-1993

Soils and rock are described in the following terms, which are broadly in accordance with AS1726-1993.

SOIL

MOISTURE CONDITION

Term	Description
Dry	Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through the hand.
Moist	Feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.
Wet	As for moist, but with free water forming on hands when handled.

Moisture content of cohesive soils may also be described in relation to plastic limit (W_P) or liquid limit (W_L) [>> much greater than, > greater than, < less than, << much less than].

CONSISTENCY OF COHESIVE SOILS

Term	Su (kPa)	Term	Su (kPa)
Very soft	< 12	Very Stiff	100 – 200
Soft	12 – 25	Hard	> 200
Firm	25 – 50	Friable	–
Stiff	50 – 100		

DENSITY OF GRANULAR SOILS

Term	Density Index(%)	Term	Density Index (%)
Very Loose	< 15	Dense	65 – 85
Loose	15 – 35	Very Dense	>85
Medium Dense	35 – 65		

PARTICLE SIZE

Name	Subdivision	Size (mm)
Boulders		> 200
Cobbles		63 – 200
Gravel	coarse	20 – 63
	medium	6 – 20
	fine	2.36 – 6
Sand	coarse	0.6 – 2.36
	medium	0.2 – 0.6
	fine	0.075 – 0.2
Silt & Clay		< 0.075

MINOR COMPONENTS

Term	Proportion by Mass	
	coarse grained	fine grained
Trace	≤ 5%	≤ 15%
Some	5 – 2%	15 – 30%

SOIL ZONING

Layers	Continuous exposures.
Lenses	Discontinuous layers of lenticular shape.
Pockets	Irregular inclusions of different material.

SOIL CEMENTING

Weakly	Easily broken up by hand.
Moderately	Effort is required to break up the soil by hand.

USCS SYMBOLS

Symbol	Description
GW	Well graded gravels and gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
GC	Clayey gravels, gravel-sand-clay mixtures.
SW	Well graded sands and gravelly sands, little or no fines.
SP	Poorly graded sands and gravelly sands, little or no fines.
SM	Silty sand, sand-silt mixtures.
SC	Clayey sand, sand-clay mixtures.
ML	Inorganic silts of low plasticity, very fine sands, rock flour, silty or clayey fine sands.
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
OL	Organic silts and organic silty clays of low plasticity.
MH	Inorganic silts of high plasticity.
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity.
PT	Peat muck and other highly organic soils.

ROCK

SEDIMENTARY ROCK TYPE DEFINITIONS

Rock Type	**Definition** (more than 50% of rock consists of .....)
Conglomerate	... gravel sized (>2mm) fragments.
Sandstone	... sand sized (0.06 to 2mm) grains.
Siltstone	... silt sized (<0.06mm) particles, rock is not laminated.
Claystone	... clay, rock is not laminated.
Shale	... silt or clay sized particles, rock is laminated.

LAYERING

Term	Description
Massive	No layering apparent.
Poorly Developed	Layering just visible. Little effect on properties.
Well Developed	Layering distinct. Rock breaks more easily parallel to layering.

STRUCTURE

Term	Spacing (mm)	Term	Spacing
Thinly laminated	<6	Medium bedded	200 – 600
Laminated	6 – 20	Thickly bedded	600 – 2,000
Very thinly bedded	20 – 60	Very thickly bedded	> 2,000
Thinly bedded	60 – 200		

STRENGTH

Term	Is50 (MPa)	Term	Is50 (MPa)
Extremely Low	<0.03	High	1.0 – 3.0
Very low	0.03 – 0.1	Very High	3.0 – 10.0
Low	0.1 – 0.3	Extremely High	>10.0
Medium	0.3 – 1.0		

NOTE: Is50 = Point Load Strength Index

WEATHERING

Term	Description
Residual Soil	Soil derived from weathering of rock; the mass structure and substance fabric are no longer evident.
Extremely	Rock is weathered to the extent that it has soil properties (either disintegrates or can be remoulded). Fabric of original rock is still visible.
Highly	Rock strength usually highly changed by weathering; rock may be highly discoloured.
Moderately	Rock strength usually moderately changed by weathering; rock may be moderately discoloured.
Slightly	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	Rock shows no signs of decomposition or staining.

DEFECT DESCRIPTION

Type	
Joint	A surface or crack across which the rock has little or no tensile strength. May be open or closed.
Parting	A surface or crack across which the rock has little or no tensile strength. Parallel or sub-parallel to layering/bedding. May be open or closed.
Sheared Zone	Zone of rock substance with roughly parallel, near planar, curved or undulating boundaries cut by closely spaced joints, sheared surfaces or other defects.
Seam	Seam with deposited soil (infill), extremely weathered insitu rock (XW), or disoriented usually angular fragments of the host rock (crushed).

Shape	
Planar	Consistent orientation.
Curved	Gradual change in orientation.
Undulating	Wavy surface.
Stepped	One or more well defined steps.
Irregular	Many sharp changes in orientation.

Roughness	
Polished	Shiny smooth surface.
Slickensided	Grooved or striated surface, usually polished.
Smooth	Smooth to touch. Few or no surface irregularities.
Rough	Many small surface irregularities (amplitude generally <1mm). Feels like fine to coarse sandpaper.
Very Rough	Many large surface irregularities, amplitude generally >1mm. Feels like very coarse sandpaper.

Coating	
Clean	No visible coating or discolouring.
Stained	No visible coating but surfaces are discoloured.
Veneer	A visible coating of soil or mineral, too thin to measure; may be patchy
Coating	Visible coating ≤1mm thick. Thicker soil material described as seam.

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Important Information about your Slope Instability Risk Assessment

1. BASIS OF THE ASSESSMENT

Our assessment of the stability of the land is presented in the framework of Landslide Risk Management (Australian Geomechanics Society, Vol 42, No 1, March 2007). The attached GeoGuides provide further information on landslide risk management and maintenance.

This assessment is based on a visual inspection of the property and also the immediate adjoining land. Limited subsurface investigation may also have been undertaken as part of this appraisal. Slope monitoring has not been carried out within or adjacent to the property for the purpose of this appraisal. The opinions expressed in this report also take into account our relevant local experience.

The property is within an area where landslip and/or subsidence have occurred, or where there is a risk that slope instability may occur. Important factors relating to slope conditions and the impact of development which commonly influence the risks of slope instability are discussed herein.

An owner's decision to acquire, develop or build on land within an area such as this involves the understanding and acceptance of a level of risk. It is important to recognise that soil and rock movements are an ongoing geological process, which may be affected by development and land management within the site or on adjoining land. Soil and rock movements may cause visible damage to structures even where the risk of slope failure is considered low. This report is intended only to assess the risk of slope failure, apparent at the time of inspection.

Our opinion is provided on the present risk of slope instability for the land specifically referenced in the title to this report. Foundations suitable for future building development are discussed in relation to slope stability considerations. Limited foundation advice may be provided. If so, advice is intended to guide the footing design for the proposed development. However, this report is not intended as, is not suitable for, and must not be used in lieu of a detailed foundation investigation for final design and costing of foundations, retaining walls or associated structures.

2. LIMITATIONS OF THE ASSESSMENT PROCEDURE

The assessment procedures carried out for this appraisal are in accordance with the recommendations in Landslide Risk Management (Australian Geomechanics Society, Vol 42, No 1, March 2007), and with accepted local practice.

The following limitations must be acknowledged:-

- the assessment of the stability of natural slopes requires a great degree of judgment and personal experience, even for experienced practitioners with good local knowledge;
- the assessment must be based on development of a sound geological model; slope processes and process rates influencing land sliding or landslide potential will vary according to geomorphologic influences;
- the likelihood that land sliding may occur on a given slope is generally hard to predict and is associated with significant uncertainties;
- different practitioners may produce different assessments of risk;
- actual risk of land sliding cannot be determined; risk changes with time;
- consequences of land sliding need to be considered in a rational framework of risk acceptance;
- acceptable risk in relation to damage to property from landslide activity is subjective; it remains the responsibility of the owner and/or local authority to decide whether the risk is acceptable; the geotechnical practitioner can assist with this judgment;
- the extent and methods of investigation for assessment of landslide risk will be governed by experience, by the perceived risk level, and by the degree to which the risk or consequences of land sliding are accepted for a specific project;
- the assessment may be required at a number of stages of the project or development; frequently (due to time or budget constraints imposed by the client) there will be no opportunity for long-term monitoring of the slope behaviour or groundwater conditions, or for on-going opportunity for the slope processes and performance of structures to be reviewed during and after development; such limitations should be recognised as relevant to the assessment.

3. DEVELOPMENT ON SLOPES

Some risk of slope instability is always attached to the development of land on slopes.

Guidelines for hillside construction and examples of good practices for hillside developments are described in the attached GeoGuides.

Appendix A

Field Investigation Results:

BH 1
Core Photos

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Borehole Log

BH no:	**BH1**
sheet:	1 of 5
job no.:	1623

client:	URBAN APARTMENTS PTY LTD	**started:**	1.2.2011
principal:		**finished:**	1.2.2011
project:	PROPOSED MIXED-USE DEVELOPMENT	**logged:**	MAB
location:	11-15 DEAN ST & 20 GEORGE ST, BURWOOD	**checked:**	MAB
equipment:	Mobile B40 Truck-Mounted Rig	**RL surface:**	20.8 m
diameter:	100mm **inclination:** -90° **bearing:** ---	**datum:**	AHD

drilling information | **material information**

method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	structure and additional observations
ADV	N							CONCRETE	--	--		PAVEMENT
					0.05		CL-CH	CLAY, medium to high plasticity, orange-brown, trace fine gravel	>>Wp	S-F		RESIDUAL (side of hole had sandy / clayey / rubble fill down one side to about 0.5m depth - possible service trench)
HA				20.5								
					0.5							
				20.0					>Wp	St-VSt		
ADV					1.0							
			SPT 2,2,3 N*=5	19.5				mottled light brown, orange-brown and grey, some fine rounded ironstone gravel		VSt	×230 ×250 ×280	
					1.5							
				19.0	1.7		CL	Silty CLAY, medium plasticity, light grey and brown	<=Wp	H		
					2.0							
ADT				18.5	2.1			XW SILTSTONE (remoulds as Sandy CLAY, medium plasticity, light grey and brown)	<Wp			V-bit refusal at 2.1m Steady TC auger progress below 2.1m
					2.5							
				18.0								
					3.0							
				17.5								
					3.5			Clayey SHALE / Shaley CLAY, fine grained, dark grey, XW, EL strength, with bands of Silty CLAY				
				17.0	3.8		CL	Silty CLAY, medium plasticity, white	>Wp	VSt		
					4.0							
			SPT 7, 22, R (N*=R)								×230	
				16.5	4.15			SHALE, fine grained, dark grey, HW, VL-L strength, highly fractured	--			BEDROCK
								Borehole No: BH1 continued as cored hole from 4.3m				
					4.5							
				16.0								
					5.0							

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

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Cored Borehole Log

BH no:	**BH1**
sheet:	2 of 5
job no.:	1623

client:	URBAN APARTMENTS PTY LTD	**started:**	1.2.2011
principal:		**finished:**	1.2.2011
project:	PROPOSED MIXED-USE DEVELOPMENT	**logged:**	MAB
location:	11-15 DEAN ST & 20 GEORGE ST, BURWOOD	**checked:**	MAB
equipment:	Mobile B40 Truck-Mounted Rig	**RL surface:**	20.8 m
diameter:	100mm **inclination:** -90° **bearing:** ---	**datum:**	AHD

drilling information | **material information** | **rock mass defects**

method	support & core-lift	water	RL	depth metres	graphic log core recovery	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	estimated strength MPa EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH	$Is_{(50)}$ MPa × o D=diametral A=axial	RQD %	defect spacing mm 20 60 200 600 2000	defect description type, inclination, thickness, shape, roughness, coating specific / general
				0.05								
			20.5	0.5								
			20.0	1.0								
			19.5	1.5								
				1.7								
			19.0	2.0								
				2.1								
			18.5	2.5								
			18.0	3.0								
			17.5	3.5								
				3.5								
			17.0	3.8								
				4.0								
			16.5	4.15		Continued from non-cored borehole from 4.3m						Numerous BP, clay seams, 0° (not shown)
NMLC				4.3		SHALE/SILSTONE, fine grained, grey/dark grey	MW		D=0.69 A=1.01			
				4.4		Silty CLAY, medium plasticity, light grey to white, moist >Wp,St/VSt	CW					
				4.5								
				4.6		No core 0.12m	--					Numerous BP, clay seams, 0°
			16.0	4.72		Silty CLAY, medium plasticity, light grey to white	CW					
				5.0								

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Cored Borehole Log

BH no: BH1
sheet: 3 of 5
job no.: 1623

client: URBAN APARTMENTS PTY LTD
principal:
project: PROPOSED MIXED-USE DEVELOPMENT
location: 11-15 DEAN ST & 20 GEORGE ST, BURWOOD
equipment: Mobile B40 Truck-Mounted Rig
diameter: 100mm **inclination:** -90° **bearing:** ---
started: 1.2.2011
finished: 1.2.2011
logged: MAB
checked: MAB
RL surface: 20.8 m
datum: AHD

drilling information — method: NMLC; water: 3/2/11, 11:30am (at RL 13.5)

material information

depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	Is(50) MPa (D=diametral, A=axial)
	Silty CLAY, medium plasticity, light grey to white *(continued)*	CW	
5.2	SANDSTONE/SILTSTONE, fine grained, orange-brown / brown with light grey bands	CW - HW	D=0.36 A=0.43
6	SANDSTONE/SILTSTONE, fine grained, orange-brown and light brown, remoulds to Sandy CLAY, some bands which do not remould	CW - XW	D=0.08 A=0.05 D=0.18 A=0.15
8.3	SHALE/SILTSTONE, fine grained, black (interface at top of SHALE inclined at 75°) - subject to crumbling / spalling on exposure	MW SW	D=0.71 D=0.08 A=0.01

RL: 16.5, 16.0, 15.5, 15.0, 14.5, 14.0, 13.5, 13.0, 12.5, 12.0, 11.5, 11.0
depth: 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0

rock mass defects

depth	defect description (type, inclination, thickness, shape, roughness, coating)
~8.6	JT, 75°, pl, ro, clay veneer
~9.5	JT, 80-85°, pl, ro, cl
general	Numerous BP, clay seams, 0° (not shown)

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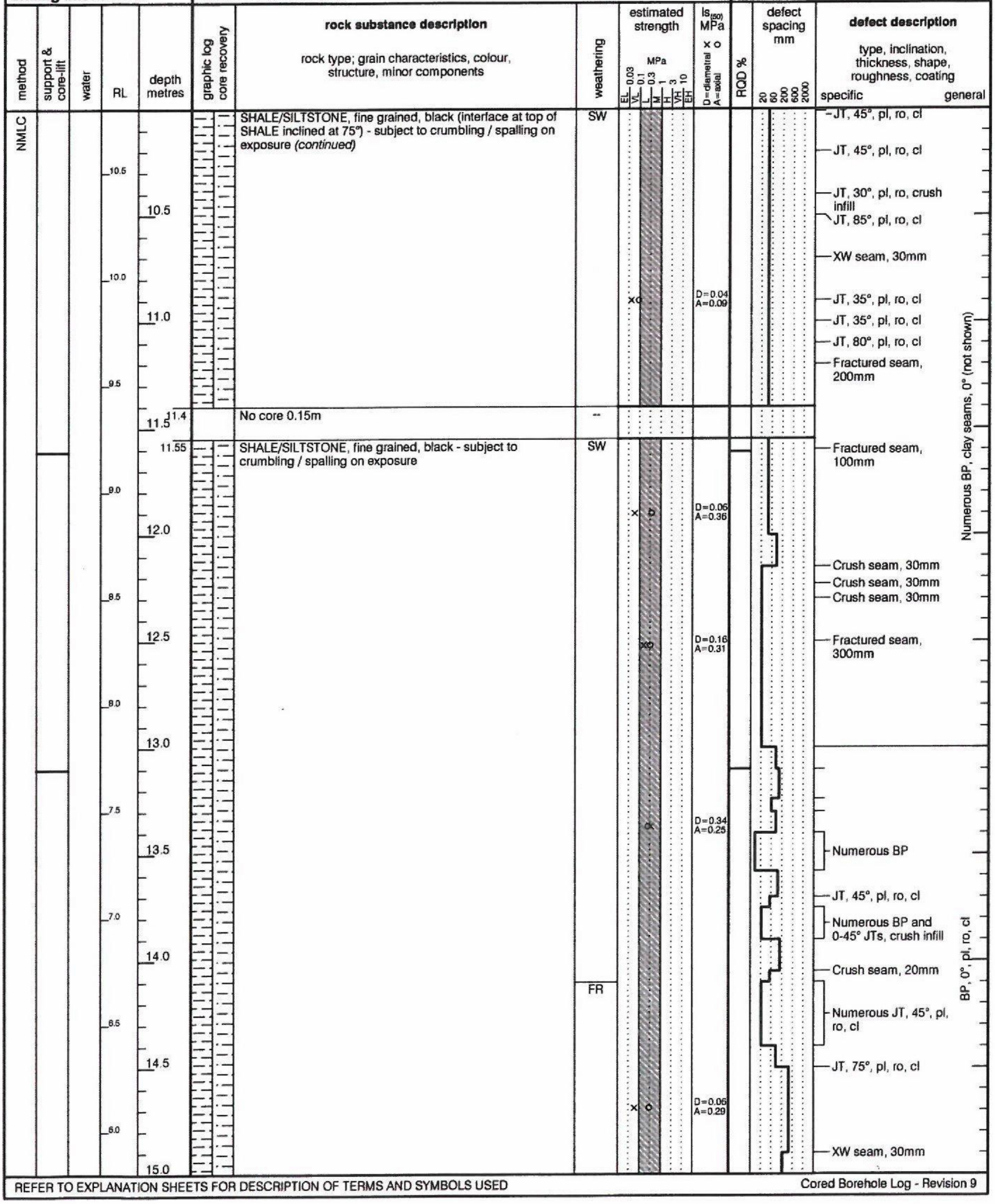
Cored Borehole Log

BH no: BH1
sheet: 4 of 5
job no.: 1623

client: URBAN APARTMENTS PTY LTD
principal:
project: PROPOSED MIXED-USE DEVELOPMENT
location: 11-15 DEAN ST & 20 GEORGE ST, BURWOOD
equipment: Mobile B40 Truck-Mounted Rig
diameter: 100mm **inclination:** -90° **bearing:** ---

started: 1.2.2011
finished: 1.2.2011
logged: MAB
checked: MAB
RL surface: 20.8 m
datum: AHD



drilling information				material information			rock mass defects	
method	support & core-lift	water	RL	depth metres	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D=diametral × A=axial ○	defect description type, inclination, thickness, shape, roughness, coating specific / general
NMLC					SHALE/SILTSTONE, fine grained, black (interface at top of SHALE inclined at 75°) - subject to crumbling / spalling on exposure *(continued)*	SW		JT, 45°, pl, ro, cl
								JT, 45°, pl, ro, cl
			10.5					
								JT, 30°, pl, ro, crush infill
				10.5				JT, 85°, pl, ro, cl
								XW seam, 30mm
			10.0					
							D=0.04 A=0.09	JT, 35°, pl, ro, cl
				11.0				JT, 35°, pl, ro, cl Numerous BP, clay seams, 0° (not shown)
								JT, 80°, pl, ro, cl
								Fractured seam, 200mm
			9.5					
				11.5 11.4	No core 0.15m	--		
				11.55	SHALE/SILTSTONE, fine grained, black - subject to crumbling / spalling on exposure	SW		Fractured seam, 100mm
			9.0					
							D=0.06 A=0.36	
				12.0				
								Crush seam, 30mm
								Crush seam, 30mm
			8.5					Crush seam, 30mm
				12.5			D=0.16 A=0.31	Fractured seam, 300mm
			8.0					
				13.0				
			7.5					
							D=0.34 A=0.25	
				13.5				Numerous BP
								JT, 45°, pl, ro, cl BP, 0°, pl, ro, cl
			7.0					Numerous BP and 0-45° JTs, crush infill
				14.0				Crush seam, 20mm
						FR		
			6.5					Numerous JT, 45°, pl, ro, cl
				14.5				JT, 75°, pl, ro, cl
							D=0.06 A=0.29	
			6.0					
								XW seam, 30mm
				15.0				

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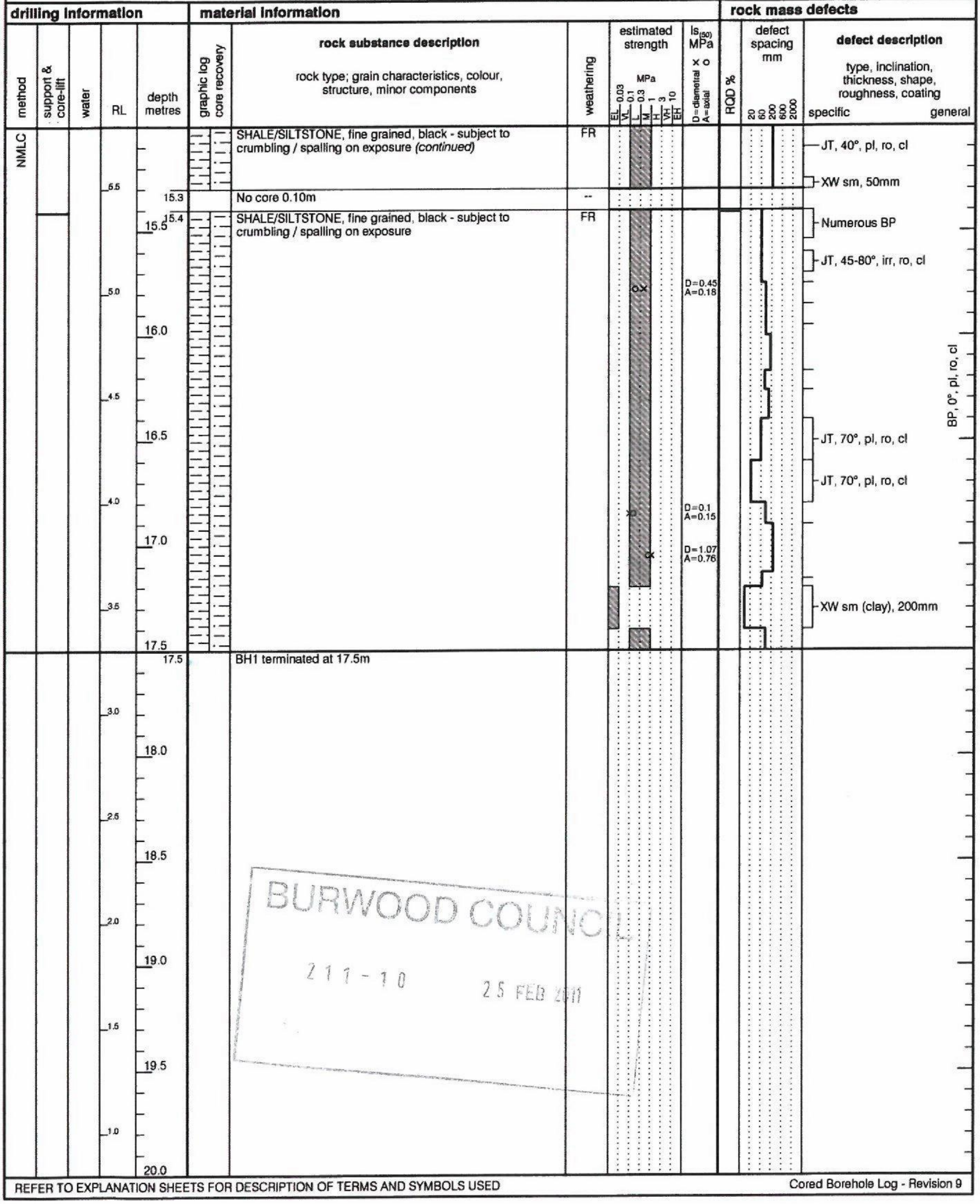
Cored Borehole Log

BH no:	**BH1**
sheet:	5 of 5
job no.:	1623

client:	URBAN APARTMENTS PTY LTD	**started:**	1.2.2011
principal:		**finished:**	1.2.2011
project:	PROPOSED MIXED-USE DEVELOPMENT	**logged:**	MAB
location:	11-15 DEAN ST & 20 GEORGE ST, BURWOOD	**checked:**	MAB
equipment:	Mobile B40 Truck-Mounted Rig	**RL surface:**	20.8 m
diameter:	100mm **inclination:** -90° **bearing:** ---	**datum:**	AHD



depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	Is(50) MPa	defect description (type, inclination, thickness, shape, roughness, coating) specific
	SHALE/SILTSTONE, fine grained, black - subject to crumbling / spalling on exposure *(continued)*	FR		JT, 40°, pl, ro, cl XW sm, 50mm
15.3	No core 0.10m	--		
15.4	SHALE/SILTSTONE, fine grained, black - subject to crumbling / spalling on exposure	FR	D=0.45 A=0.18 D=0.1 A=0.15 D=1.07 A=0.76	Numerous BP JT, 45-80°, irr, ro, cl JT, 70°, pl, ro, cl JT, 70°, pl, ro, cl XW sm (clay), 200mm
17.5	BH1 terminated at 17.5m			

rev	by	date	description
–	–	–	–

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PROPOSED MIXED–USE DEVELOPMENT
11–15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

CORE PHOTOS – BH1 (1 of 2)

drawn: MAB
date: 14.2.11
checked:
scale: ~1:4
fig:
rev: –
job no.: 1623

rev	by	date	description
–	–	–	–

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PROPOSED MIXED-USE DEVELOPMENT
11-15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

CORE PHOTOS – BH1 (2 of 2)

drawn: MAB	**fig:**
date: 14.2.11	
checked:	**rev:** –
scale: ~1:4	**job no.:** 1623

Appendix B

Laboratory Test Results:

Point Load Strength Index (BH 1)

TEST CERTIFICATE

SGS

ABN 44 000 964 278
ph: +61 (0)2 8594 0481
fax: +61 (0)2 8594 0499

SGS Australia Pty Ltd
Unit 15, 33 Maddox Street
(PO Box 6432)
Alexandria NSW 2015
Australia

POINT LOAD STRENGTH INDEX

CLIENT: **Asset Geotechnical**
PO Box 3385 Rouse Hill NSW 2155
PROJECT: **11-13 Deane St, Burwood (1623)**

LAB. NO.	SAMPLE SOURCE	LITHOLOGY	PLATEN SEPARATION DIAM (mm)	PLATEN SEPARATION HEIGHT (mm)	TEST ORIENTATION	POINT LOAD STRENGTH Is (MPa)	POINT LOAD STRENTH $Is_{(50)}$ (MPa)	Type OF FAILURE
63928	BH1	Siltstone	51.7		Diametral	0.68	0.69	FB
	4.34m			27.4	Axial	1.09	1.01	FOB
63929	BH1	Siltstone	51.7		Diametral	0.35	0.36	FOB
	5.38m			44.2	Axial	0.41	0.43	FOB
930	BH1	Siltstone	51.6		Diametral	0.08	0.08	FOB
	6.86m			45.2	Axial	0.05	0.05	FOB
63931	BH1	Siltstone	52.8		Diametral	0.17	0.18	FOB
	7.54m			31.6	Axial	0.16	0.15	FOB
63932	BH1	Siltstone	52.7		Diametral	0.69	0.71	FB
	8.43m			*	Axial	-	-	-
63933	BH1	Siltstone	51.6		Diametral	0.08	0.08	FB
	9.86m			36.8	Axial	0.01	0.01	FOB
63934	BH1	Siltstone	51.5		Diametral	0.04	0.04	FB
	10.90m			34.4	Axial	0.09	0.09	FOB
63935	BH1	Siltstone	51.2		Diametral	0.06	0.06	FB
	11.90m			28.0	Axial	0.37	0.36	FOB

OTES TO TESTING

Testing Device	ELE Point Load Tester
Sample History	Unsoaked
Sampled By:	Client
Job Number:	118-285
Date Tested:	07.02.11
Test Method:	AS 4133.4.1 2007

Failure Type

FOB	Fracture through fabric of specimen oblique to bedding not influenced by weak planes
FB	Fracture along Bedding
FIP	Fracture influenced by pre-existing plane, microfracture, vein, chemical alteration
CPF	Chip or Partial Fracture

* - Insufficient material

Approved Signatory: Chris Lloyd

Date: 07.02.11

Accreditation No. 1459

This document is issued in accordance with NATA's accreditation requirements

TEST CERTIFICATE

ABN 44 000 964 278
ph: +61 (0)2 8594 0481
fax: +61 (0)2 8594 0499

SGS Australia Pty Ltd
Unit 15, 33 Maddox Street
(PO Box 6432)
Alexandria NSW 2015
Australia

POINT LOAD STRENGTH INDEX

CLIENT: **Asset Geotechnical**
PO Box 3385 Rouse Hill NSW 2155
PROJECT: **11-13 Deane St, Burwood (1623)**

LAB. NO.	SAMPLE SOURCE	LITHOLOGY	PLATEN SEPARATION		TEST ORIENTATION	POINT LOAD STRENGTH	POINT LOAD STRENTH	Type OF FAILURE
			DIAM (mm)	HEIGHT (mm)		Is (MPa)	$Is_{(50)}$ (MPa)	
63936	BH1 12.52m	Siltstone	51.2		Diametral	0.16	0.16	FB
				41.2	Axial	0.31	0.31	FOB
63937	BH1 13.37m	Siltstone	50.7		Diametral	0.33	0.34	FB
				33.5	Axial	0.26	0.25	FOB
38	BH1 14.69m	Siltstone	51.4		Diametral	0.06	0.06	FB
				30.6	Axial	0.31	0.29	FOB
63939	BH1 15.78m	Siltstone	51.3		Diametral	0.44	0.45	FB
				36.3	Axial	0.18	0.18	FOB
63940	BH1 16.85m	Siltstone	51.6		Diametral	0.10	0.10	FB
				41.9	Axial	0.15	0.15	FOB
63941	BH1 17.05m	Siltstone	51.6		Diametral	1.05	1.07	FB
				32.4	Axial	0.79	0.76	FOB

NOTES TO TESTING

Testing Device	ELE Point Load Tester
Sample History	Unsoaked
Sampled By:	Client
Job Number:	118-285
Date Tested:	07.02.11
Test Method:	AS 4133.4.1 2007

Failure Type

FOB	Fracture through fabric of specimen oblique to bedding not influenced by weak planes
FB	Fracture along Bedding
FIP	Fracture influenced by pre-existing plane, microfracture, vein, chemical alteration
CPF	Chip or Partial Fracture

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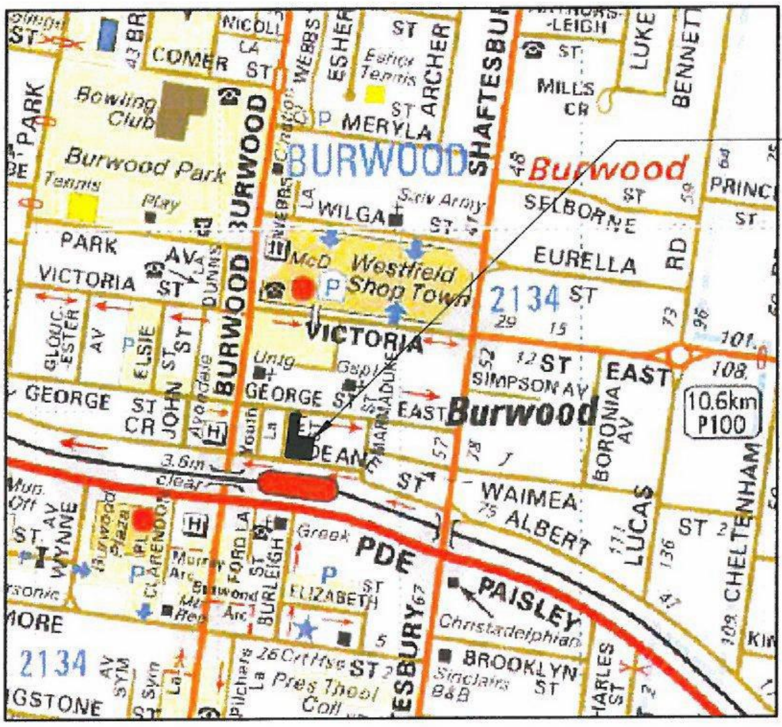
Figures



rev	by	date	description
–	–	–	–

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PROPOSED MIXED-USE DEVELOPMENT
11–15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

SITE LOCALITY

drawn: MAB
date: 14.2.11
checked:
scale: 1:10,000
fig: 1
rev: –
job no.: 1623

rev	by	date	description
-	-	-	-

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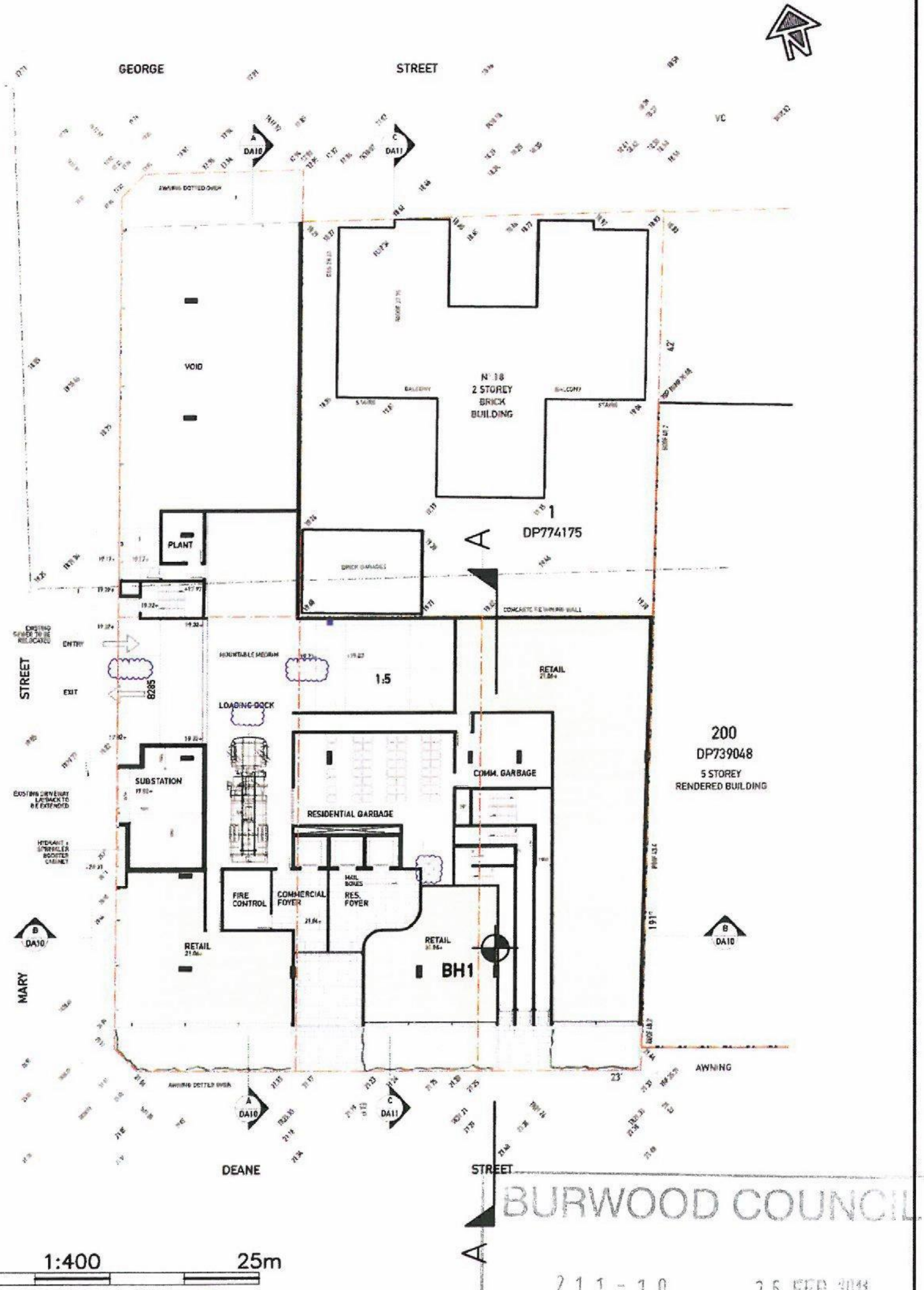
PROPOSED MIXED-USE DEVELOPMENT
11-15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

AIR PHOTO

drawn: MAB
date: 14.2.11
checked:
scale: ~1:800
fig: 2
rev: -
job no.: 1623



rev	by	date	description
–	–	–	–

TEST LOCATION APPROXIMATE
ONLY – SUBJECT TO DETAIL
SURVEY.

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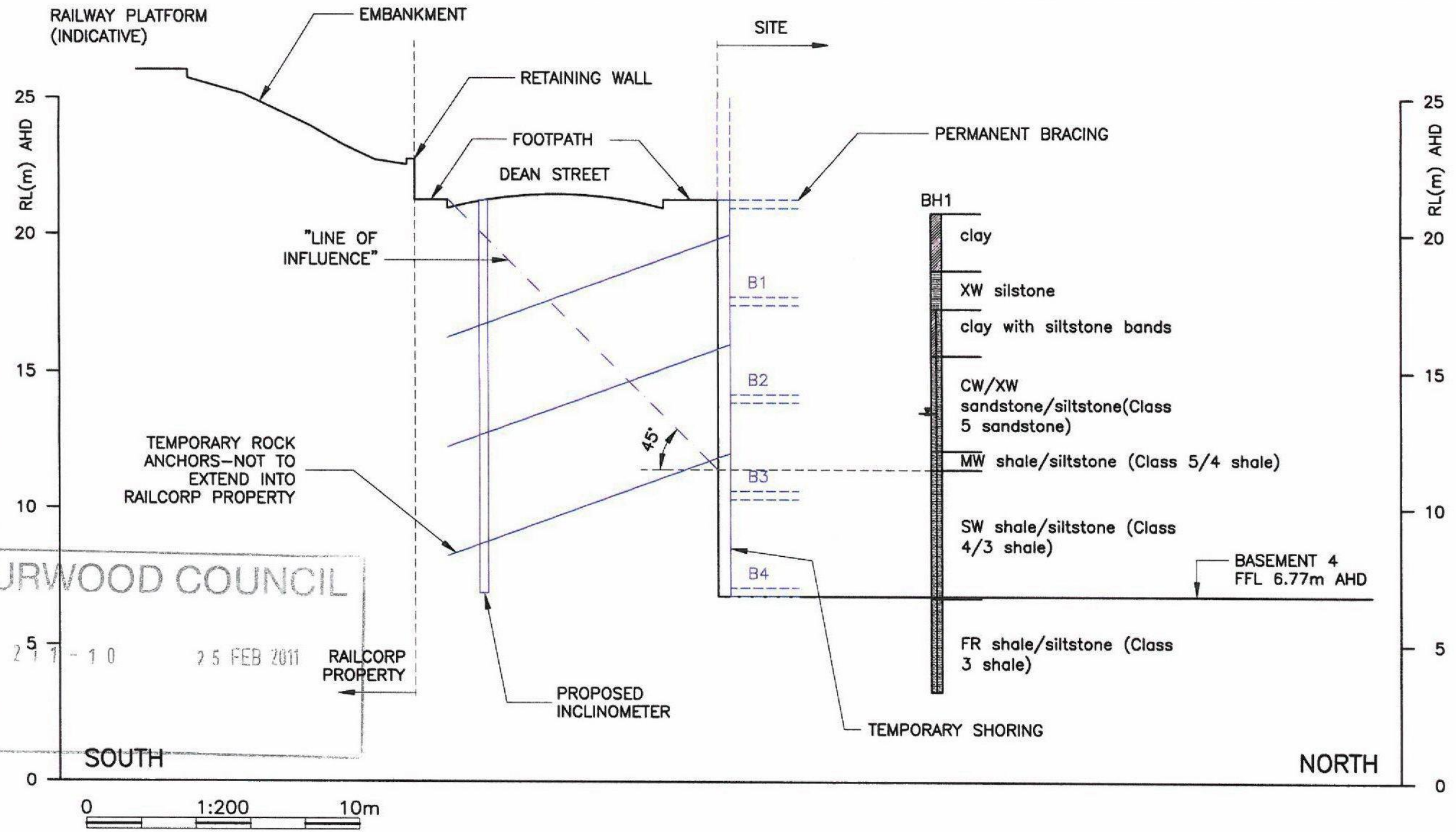
PROPOSED MIXED-USE DEVELOPMENT
11-15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

TEST LOCATION

drawn: MAB
date: 14.2.11
checked:
scale: 1:400
fig: 3
rev: –
job no.: 1623



rev	by	date	description
–	–	–	–

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PROPOSED MIXED–USE DEVELOPMENT
11–15 DEAN ST & 20 GEORGE ST,
BURWOOD
for URBAN APARTMENTS PTY LTD

INTERPRETED SECTION A–A

drawn: MAB
date: 14.2.11
checked:
scale: 1:200
fig: 4
rev: –
job no.: 1623