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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED WALLACE WURTH REDEVELOPMENT
CORNER OF HIGH STREET AND BOTANY STREET
UNSW, KENSINGTON

Prepared for
BOVIS LEND LEASE

Project 71543
March 2010

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Douglas Partners Pty Ltd
ABN 75 053 980 117

96 Hermitage Road
West Ryde NSW 2114
Australia

PO Box 472
West Ryde NSW 1685

Phone (02) 9809 0666
Fax (02) 9809 4095
sydney@douglaspartners.com.au

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TABLE OF CONTENTS

STE:lll
Project 71543
19 March 2010

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1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for the proposed Wallace Wurth Redevelopment within University of New South Wales at the corner of High Street and Botany Street, Kensington. The investigation was commissioned by Bovis Lend Lease Pty Ltd.

It is understood that the proposed development includes construction of two seven-storey extensions to the north and south of the "M-Wing" which is located on the eastern side of the existing Wallace Wurth building. The extensions will include a lower ground floor which will be linked to the existing lower ground floor of the M-Wing and will require excavation to depths of approximately 2.5 m to 3 m. An additional level will be constructed above the existing Wallace Wurth building, which will also be refurbished and upgraded. The investigation was required for design and planning purposes.

The investigation included seven boreholes (including one inclined borehole) and installation of three groundwater monitoring wells for sampling and measurement of groundwater levels. Laboratory testing of selected rock core samples was undertaken, followed by engineering analysis and reporting. Details of the field work are given in the report, together with comments on design and construction practice.

Douglas Partners Pty Ltd (DP) carried out a Phase 1 Contamination Assessment of the site in conjunction with the geotechnical investigation, the results of which have been reported separately (Project No. 71543.01, dated March 2010).

2. BACKGROUND

DP carried out geotechnical and contamination assessments for the C25 Cancer Research Facility which has recently been completed and is located immediately to the west of the Wallace Wurth building. DP also provided extensive geotechnical and contamination services during construction of the C25 building which required excavation to depths of approximately 6-7 m. The C25 site was previously used as a quarry for extraction of deeply weathered igneous material (essentially clay). The C25 site was underlain by variable and problematic ground conditions with deep contaminated sandy filling, shallow groundwater, and highly fractured rock resulting from a major igneous intrusion (dyke). Extensive grouting of the rock below the bulk excavation level was carried out to reduce groundwater inflows and facilitate a drained basement.

Geotechnical inspections carried out during excavation on the C25 site indicated that the quarry and deep filling did not extend up to the Wallace Wurth building. Sandstone bedrock was exposed at relatively shallow depth (approximately RL 53-54 m) along the western side of the Wallace Wurth building. Highly weathered igneous rock/clay was exposed over a large area on the north-western part of the C25 site and is possibly associated with a diatreme (volcanic neck). A near-vertical igneous dyke, approximately 1 m wide, and striking approximately north-west to south-east, intercepted the excavation along the western side of the Wallace Building. The inferred location of the dyke is shown on Drawing 1 in Appendix B. The sandstone along the sides of the dyke was generally highly fractured and of very low strength then grading to medium to high strength at distances of about 3 m from the dyke. High and very high strength, indurated sandstone was also encountered close to the igneous intrusions.

A concrete lined drainage channel, about 1.5 to 2 m wide, runs below the western side of the Wallace Wurth Building. It is understood that the channel was constructed to manage water seepage that was encountered during excavation and construction of the Wallace Wurth

Building. The base of the channel is estimated to be 3 m below ground level (approximately RL53.5 m). Sandstone bedrock was previously observed in the sides of the channel. At the time of the investigation for the C25 site, a constant 'trickle' of water, about 50 mm deep, was running along the base of the channel. A thick iron oxide sludge was observed in the base of the channel.

Douglas Partners have been provided with various structural drawings for the Wallace Wurth building prepared by Morrison & Little Pty Ltd (dated February 1975). The drawings indicate that the footing system for the building includes pad footings founded on rock. The pads vary in size from 0.7 m × 0.7 m × 0.7 m deep to 2 m × 2 m × 1.3 m deep. The base of the pad footings vary from RL 49.8 to 51.8 m. The design bearing pressure for the rock foundation is not shown on the drawings and has not been confirmed.

3. SITE DESCRIPTION

The site, including the existing Wallace Wurth building and the proposed extensions, is a rectangular-shaped lot with a plan area of approximately 3,000 m^2 and a northern frontage to High Street and an eastern frontage to Botany Street. It is located on relatively level ground which extends for about 50 m to the west, where ground slopes then fall to the west (to the lower campus) at an average slope of approximately 10 degrees. Ground surface levels within the site generally range from RL 55.5 to RL 56.5 m relative to Australian Height Datum (AHD).

At the time of the investigation, the five to six-storey Wallace Wurth building occupied most of the site. The area to the north of the M-Wing was occupied by a single-storey brick building. The area to the south of the W-Wing included a dangerous goods storage area and a concrete driveway and loading dock with entry to lower ground floor.

Several medium to large trees are present on the site.

A Sydney Water easement for drainage runs along the northern and eastern sides of the site and is understood to include a 0.75 m diameter steel water main.

4. GEOLOGY

Reference to the Sydney 1:100 000 Series Geological Sheet indicates that the site is underlain by Quaternary aged sediments comprising medium to fine grained Aeolian sand (deposited by transgressive dunes) overlying Hawkesbury Sandstone which typically comprises medium to coarse grained quartz sandstone with some shale bands or lenses.

The geological mapping was generally confirmed by the fieldwork which identified filling and sand overlying sandstone. Weathered igneous rock associated with a dyke was intercepted on the site. One of the dykes was previously identified on the C25 site. Dykes are steeply dipping, often near-vertical igneous rock intrusions, which in Sydney, can vary from less than 1 m to about 6 m wide. Many dykes contain completely weathered basalt/dolerite (clay) near the surface but sometimes this grades to medium or high strength rock at depth. The rock (i.e. sandstone) adjacent to dykes can be highly fractured and variable in strength due to the effects of the intrusion. Thermal alteration can sometimes also result in abnormally high strength host rock. Water seepage along the sides of the dyke can also occur due to the highly fractured host rock and deeply weathered igneous rock. These typical observations of dykes are generally consistent with conditions encountered on the C25 site (as discussed in Section 2).

5. FIELD WORK METHODS

The field work included seven boreholes (BH 1 to 7 inclusive) and installation of three groundwater monitoring wells (BH 1, 3 and 6). All of the boreholes were drilled vertically, with the exception on BH2, which was drilled at an inclination of 60 degrees below horizontal, in a northerly direction. BH2 was inclined in an attempt to intercept the igneous dyke previously encountered on the C25 site.

The boreholes were drilled to depths of 5.3 m to 18.3 m using a bobcat-mounted drilling rig. The boreholes were initially drilled using spiral flight augers then rotary washboring within the soil and weathered rock to depths of 0.4 m to 5.8 m. The boreholes were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the rock. A combination of rotary drilling and NMLC coring was used in BH3 due to slow drilling progress and blockage of the core barrel. Standard Penetration Tests (SPT's) were carried out within the boreholes at 1.5 m depth intervals to sample the soil and assess the in-situ strength of the materials.

Soil samples and rock cores 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.

Groundwater observations were made during auger drilling of the boreholes and groundwater monitoring wells (50 mm diameter slotted PVC) were installed in BH 1, 3 and 6 to allow for future measurement of groundwater levels and sampling of the groundwater for the contamination assessment. The groundwater levels within the monitoring wells were measured on 11/2/2010 and again on 5/3/2010.

The ground surface levels at the test locations were interpolated from spot levels shown on the survey plan by Degotardi, Smith & Partners Pty Ltd (Drawing No. 32141A01, dated 5/2/2010).

6. FIELD WORK RESULTS

Details of the conditions encountered in the boreholes are given in Appendix A, together with colour photographs of the rock core samples and notes defining classification methods and descriptive terms. The borehole locations are shown on Drawing 1 in Appendix B.

6.1 Subsurface Soils and Rock

The boreholes generally encountered sandy filling to depths of 0.7 m to 3.3 m overlying natural sand then sandstone bedrock at depths of 3.1 m to 5.3 m (RL50.6 to 53.3 m). Sandstone was encountered at a shallower depth of 0.4 m (RL52.9 m) in BH4 as this borehole was drilled from a lower elevation in the loading dock. Igneous rock was encountered in BH2 and BH3 and traces of igneous rock and evidence of indurated sandstone was encountered in BH1.

The various strata are summarised below:

PAVEMENTS: asphaltic concrete 150 mm thick over roadbase 200 mm thick was directly overlying sandstone bedrock in BH4 (located in the loading dock).

FILLING: with the exception of BH4, sandy filling was generally encountered to depths of between 0.7 m to 2.0 m, however, filling was also encountered to a depth of 3.3 m and 3.0 m in BH6 and BH7, respectively. The filling comprised sand with inclusions of sandstone gravel, concrete, ceramic tile, glass, slag and metal fragments. The filling in BH7 below 1.5 m depth was logged as possibly natural.

SAND: below the filling, loose to medium dense sand was encountered to depths of between 3.1 m to 5.3 m. Sandy clay was encountered below the sand in BH6 to a depth of 4.3 m.

SANDSTONE: with the exception of BH2 and BH3, sandstone bedrock was encountered at depths of between 0.4 m to 5.3 m (RL50.6 to 53.3 m). The sandstone included variable very low to medium strength rock about 0.5 m to 3 m thick overlying more uniform medium to high strength (or better) rock at depths of 2.2 m to 6.4 m (RL50.0 to RL51.3 m). The sandstone in BH1 included very low strength bands of igneous rock to a depth of 6.3 m (RL53.3 m) over high to very high strength indurated sandstone.

IGNEOUS ROCK: The inclined BH2 encountered extremely low to very low strength igneous rock with bands of medium to high strength indurated sandstone to a depth of 12 m over very low to medium strength sandstone with igneous intrusions (possibly representing the edge of the dyke). It is not possible to accurately determine the width or strike of the dyke from the recovered core, however, the presence of the dyke in BH2 is consistent with the dyke encountered on the C25 site. It appears that the dyke is about 1 m to 2 m wide with fractured variable strength sandtone at least 2 m wide on each side.

BH3 encountered extremely low to very low strength igneous rock to a depth of 17.2 m over fresh, high to very high strength igneous rock. The width or strike of the dyke cannot be determined from this vertical borehole.

6.2 Groundwater

Groundwater was not encountered during auger drilling of the boreholes to depths of 2.5 m to 3.6 m. A summary of the measured groundwater levels within the groundwater monitoring wells is provided in Table 1. Following the measurement on 11/2/10, the groundwater wells were purged for environmental sampling purposes.

Table 1 – Summary of Measured Groundwater Levels

Date	Depth (RL) to Groundwater		
	BH 1	**BH 3**	**BH 6**
11/2/10 (prior to purging)	4.8 (51.6)	5.8 (50.4)	4.8 (51.7)
5/3/10	5.1 (51.3)	4.9 (51.3)	4.9 (51.6)

7. POINT LOAD STRENGTH INDEX TESTING

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist rock strength classification. The results of the testing are shown on the bore logs at the appropriate depth.

It is noted that Is_{50} tests are not readily carried out on extremely low to very low strength rock and hence strength classification for the weaker rock is based on visual/tactile assessments of the rock core. The Is_{50} values for the various rock strata are described below together with the estimated unconfined compressive strength (UCS) which is based on a UCS:Is_{50} ratio of 20.

The Is_{50} values for the sandstone cores generally ranged from 0.4 MPa to 2.0 MPa corresponding to medium to high strength classification (estimated UCS ranging from 8 MPa to

40 MPa). Some higher Is_{50} values of up to 4.9 MPa (estimated UCS of 98 MPa) were however measured on samples of very high strength indurated sandstone in BH1 and BH2. An Is_{50} value of 9.2 MPa (estimated UCS of 180MPa) was measured on the fresh igneous rock in BH3 at a depth of 17.5 m.

8. GEOTECHNICAL MODEL

Two geotechnical cross sections (Sections A-A' and B-B') showing the interpreted subsurface profile between the boreholes are shown on Drawings 2 and 3 in Appendix B. The sections show interpreted geotechnical divisions of underlying soil and rock together with the extent of the existing and proposed buildings. The orientations of the cross-sections are shown on Drawing 1. A summary of the depths (and reduced level) to the top of the various rock strata is provided in Table 2.

The interpreted geotechnical model for the site includes:

- sandy filling to depths of 1 m to 2 m across most of the site with some areas of localised deeper filling adjacent to the existing lower ground floor (i.e. backfill).
- natural loose to medium dense sands underlying the filling to depths of 3 m to 5.5 m. The thickness of the sand layer generally increases towards the south as the rock surface also falls to the south. Sandy clay may be present above the rock surface at some locations, as encountered in BH6.
- below the sand is a somewhat variable bedrock rock profile comprising sandstone which is intersected by at least two igneous dykes. The sandstone surface generally falls towards the south-east from about RL53.5 to RL50.5 m. The sandstone profile is expected to comprise a variable layer of fractured, very low to medium strength rock about 0.5 m to 3 m thick overlying more uniform slightly fractured, medium or better strength rock at depths of about 2.5 m to 6.5 m (RL50 to RL51.5 m).
- based on the dyke encountered on the C25 site and the igneous rock encountered in BH2, it is expected that a near vertical dyke about 1 m to 2 m wide crosses the north-eastern corner of the site on a north-west to south-east strike (as shown on Drawing 1). The

sandstone either side of the dyke is expected to be highly fractured and of extremely low to very low strength but will also include high to very high strength indurated sandstone.

- based on the igneous rock encountered in BH3 and the bands of igneous rock and indurated sandstone encountered in BH1, it is inferred that a dyke may also cross the northern part of the site, on a similar north-west to south-east strike (the possible location is also shown on Drawing 1). It must be recognised that this is an assumption only based on available information and may not be correct. The igneous rock at these two borehole locations may be related to separate dykes running in totally opposite directions.
- the dykes are expected to comprise extremely low to very low strength igneous rock and possibly clay to depths of about 15 m to 20 m grading to fresh, medium to very high strength igneous rock. This was observed in BH3 and is consistent with previous investigation and piling on the C25 site.
- monitoring to date indicates that a groundwater table exists within the rock profile at approximately RL51.5 m. Fluctuations in the groundwater level may be expected following periods of prolonged or heavy rainfall. Perched groundwater seepage will also occur at a higher level, along the top of the rock surface, particularly following periods of rainfall. Ongoing groundwater measurements should be carried out within the groundwater monitoring wells to assess possible fluctuations in groundwater levels.
- it is likely that the groundwater contains relatively high levels of iron as evident from iron-oxide sludge observed in the base of the drainage channel below the western side of the Wallace Wurth building.

Table 2 – Summary of Depths and Reduced Levels (RL) to the Top of Various Strata

Strata	Depth (RL) to Top of Strata						
	BH 1	BH 2	BH 3	BH 4	BH 5	BH 6	BH 7
Surface/Filling	0 (56.4)	0 (55.8)	0 (56.2)	0 (53.3)	0 (55.9)	0 (56.5)	0 (55.8)
Sand	1.0 (55.4)	1.8 (54.2)	0.7 (55.5)		2.0 (53.9)	3.3 (53.2)	3.0 (53.8)
Igneous Rock : EL-VL	-	4.3 (52.1)(B)	3.2 (53.0)(D)				
Igneous Rock : H-VH	-		17.2 (39.0)				
Sandstone : EL-VL	-	12.0 (45.4)(C)		0.4 (52.9)(E)	5.3 (50.6)		4.9 (50.9)
Sandstone : L-M	3.1 (53.3)(A)					4.3 (52.2)(F)	
Sandstone : M-H	6.4 (50.0)			2.2 (51.1)	5.8 (50.1)	5.2 (51.3)	5.2 (50.6)

Notes: EL= extremely low strength VL= very low strength L = low strength
M = medium strength H = high strength VH = very high strength
(A) High strength with very low strength igneous bands of rock in BH1 at 3.1m depth
(B) With medium to high strength sandstone bands in BH2 at 4.3m depth
(C) With high strength sandstone bands in BH2 at 12.0m depth
(D) With medium strength bands in BH3 below 11.0m depth
(E) Medium strength from 0.35-0.9m in BH4
(F) With very low strength bands in BH6 at 4.3m depth

Further investigation will be required to determine the extent of the igneous dykes across the site for detailed design purposes. This investigation can be carried out during construction whilst making allowances in the design for variable foundation conditions. Alternatively, further investigation could be carried out earlier, however, access to the site is restricted by existing buildings. It is also noted that even with further extensive investigation, the actual rock profile and details of the dykes will only become truly apparent at the time of excavation when the rock is exposed and test pits can be excavated to delineate the width and strike of the dykes.

9. COMMENTS

9.1 Proposed Development

Based on drawings by Wilson Architects Pty Ltd and Lahznimmo Architects Pty Ltd (architects in association) dated 5 March 2010, it is understood that the proposed development includes construction of two seven-storey extensions to the north (northern extension) and south (southern extension) of the Wallace Wurth M-Wing. The extensions will include a lower ground floor (RL53.5 m) which will be linked to the existing lower ground floor of the M-Wing. The lower ground floors will require excavation to depths of approximately 2.5 m to 3 m. An additional level will be constructed above the existing Wallace Wurth building, which will also be refurbished and upgraded.

9.2 Earthworks

9.2.1 Excavation Conditions

Excavations will require removal of mostly sandy soils. Sandstone may be encountered close to the bulk excavation level on the northern end of the site with the rock surface then gradually falling to 2.5 - 3 m below the bulk excavation level on the southern end of the site. The rock surface is, however, expected to be located at or close to the bulk excavation level on the north-western corner of the southern extension (see BH4). The rock may vary from extremely low to very high strength sandstone along the sides of the dyke.

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 to high 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 medium to high strength, and particularly very high strength rock, may be very slow (even with large dozers) and therefore pre-splitting or rock hammering may be necessary to improve efficiency. The very high strength sandstone encountered on the C25 site was very difficult to excavate and required rock saws and rock hammering. Contractors may inspect the rock core samples at the DP office in West Ryde prior to submitting final tenders and should be asked to make their own assessment on the equipment required to complete bulk and detailed earthworks. Rock cores are generally kept for 6 months after drilling unless longer holding times are requested.

9.2.2 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 Phase 1 Contamination Assessment report for details on the contamination status and waste classification of site soils.

9.2.3 Groundwater

Groundwater was measured at RL51.3 to RL51.7 within the groundwater monitoring wells on the site, which is approximately 2 m below the proposed lower ground floor level. In the absence of more detailed monitoring it is suggested that allowance for a potential rise in the groundwater level to RL52.5 m should be included in the design. This suggested groundwater level is 1 m below the proposed floor level but may be applicable for localised deeper structures such as lift shafts. Some groundwater seepage should also be expected at a higher level, along the top of the rock surface, with seepage flows expected to increase following periods of extended rainfall.

During construction, it is anticipated that groundwater seepage should be readily controlled by subfloor drainage and perimeter drains connected to a "sump-and-pump" dewatering system. The need for ongoing dewatering, after construction, will depend on whether the basement is designed as a drained basement or water tight (tanked) basement. A drained basement will require permanent subfloor drainage below the basement floor slab connected to a sump and pump dewatering system. A tanked basement will avoid the need for dewatering after construction, however the tanked basement may be considerably more expensive than the drained basement and is probably not warranted for this site. A tanked basement would need to be designed to resist uplift forces associated with groundwater pressure, for which preliminary design could be based on a groundwater level at the rock surface.

Inspection of the existing lower ground floor levels for the Wallace Wurth building together with a review of available drainage plans (if available) may provide valuable information on drainage provisions for the new building.

It is possible that the seepage into the basement may give rise to precipitation of red brown iron oxide sludge from the groundwater. Perimeter and subfloor drains should be designed for easy access to allow for inspection, maintenance and periodic cleaning.

9.2.4 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings and pavements before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to vibrations or construction related activities can be accurately assessed.

9.2.5 Vibrations

It is anticipated that rock excavation, where required, will result in vibration of the surrounding ground. Where impact breakers are required in the vicinity of adjacent buildings it would be prudent to monitor and limit vibrations on these structures. Generally, a maximum peak particle velocity of 8 mm/sec (in any component direction) at foundation level of adjacent structures is suggested for both structural and human comfort considerations. This vibration limit may need to be reviewed if there is any sensitive equipment located within the adjacent buildings (i.e. medicinal research facilities).

Based on vibration monitoring carried out by DP at various excavation sites in Sydney it is anticipated that vibrations resulting from a 2000 kg rock hammer would be less than 8 mm/sec at distances of more than 10 m from the excavation. However, as the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of excavation. The trial may indicate that smaller or different types of excavation equipment should be used.

9.3 Excavation Support

9.3.1 General

Vertical excavations within the soils and extremely low to low strength rock will require retaining/shoring support during construction and as part of the final basement structure. Although the excavation is generally set back 6 m to 7 m from the site boundaries, it is anticipated that shoring will be required to support the soils and to reduce disturbance of adjacent services and the Sydney Water easement.

9.3.2 Batter Slopes and Vertical Rock Faces

Temporary slopes within the shored excavation should generally be battered at no steeper than 1.5H:1V (Horizontal:Vertical) in the sandy soils and 0.75H:1V in any extremely low and low strength rock.

Excavations in sandstone of low to medium or greater strength should generally be self-supporting (subject to joint orientation) and may be cut vertically. It is anticipated that the extent of vertical rock excavation on this site will be limited to localised deeper excavations for services and lift shafts below the bulk excavation level. All vertical rock faces must be progressively inspected by a geotechnical engineer at 1.5 m depth intervals to check for adversely inclined joints and detached blocks and to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcreting of fractured or highly weathered zones or rock bolting where adverse joints form potentially unstable wedges of rock.

9.3.3 Retaining Walls/Shoring

Due to the presence of loose sands and possible seepage along the rock surface the preferred shoring system for this site is a contiguous pile wall. A secant pile wall could be used to provide a relatively impermeable wall, however, this is not necessarily warranted for the expected subsurface conditions. The shoring piles should be taken down to rock and founded below the bulk excavation level. The preferred shoring systems are described below:

- **Contiguous Piled Wall** – comprises closely spaced/touching Continuous Flight Auger (CFA) piles. It will be necessary to ensure that gaps between piles are generally limited to less than about 30 mm to 50 mm to minimise the risk of sand loss from behind the wall. Any gaps between the contiguous piles should be progressively patched as the excavation proceeds (at every 1.5 m depth of excavation) to prevent such sand loss. The gaps in between the piles are typically filled with dry-packed grout, however, shotcrete or grout injection may be required where the wall is designed to form part of the basement structure.

- **Secant Pile Wall** - comprises interlocking CFA piles or CFA piles with jet grouted columns in between piles. This system can generally provide an effective seal to minimise sand loss and water inflow from behind the wall, and if adequately supported, minimise lateral deflections. This type of wall is generally more expensive and is used for deep excavations below the water table and may not be necessary for this site. The pile wall can be incorporated into the vertical load carrying footing system and can generally form part of the basement structure.

The following shoring systems have been considered but are not recommended due to the limitations and risks associated with there use, as outlined below:

- **Sheet Piles** - are generally suitable for shallower excavations in soil and where there are no movement/vibration sensitive structures adjacent to the excavation. Sheet piles are unlikely to be suitable for this site due to risks associated with vibration induced settlement of surrounding residential buildings that may be founded on shallow foundations. The piles cannot penetrate rock and this may result in buckling of the piles and/or large gaps at the base of the wall through which seepage and sand loss may occur. Generally sheet pile walls experience higher lateral deflections than other shoring systems.
- **Geocast Walls** - are proprietary walls and are generally suitable for shallower excavations in soil where there are no movement sensitive structures adjacent to the excavation. This type of wall can be cost effective and produce a reasonable finish for the basement walls. The presence of sandstone will, however, present difficulties for this type of wall construction and may result in uneven founding and gaps below the base of the wall that may allow seepage and sand loss. If this system is to be considered, detailed information on the proposed methodology and assurances on the suitability of this construction for the site should be sought from the contractor.

9.3.4 Design

The design of the shoring will depend somewhat upon whether it is cantilevered or restrained by multiple rows of temporary rock anchors. It is anticipated that one row of rock anchors may be required to provide lateral support to the shoring wall to limit wall movements.

It is suggested that design of cantilevered shoring systems (or shoring with a single row of anchors) be based on a triangular earth pressure distribution based on earth pressure coefficients provided in Table 3. 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 is to be minimised.

Table 3 – Recommended Parameters for Retaining Wall/Shoring Design

Material	**Earth Pressure Coefficient**		**Bulk Unit Weight (kN/m^3)**
	Active (Ka)	**At Rest (Ko)**	
Filling and Sand	0.4	0.6	20
Extremely Low to Low Strength Rock	0.3	0.45	21
Low to Medium Strength or Greater Rock	0*	0*	22

Note * Provided that no adverse jointing is present in the rock (to be confirmed by progressive inspection by a geotechnical engineer)

Where more than one row of temporary anchors is used it is suggested that design of shoring is based on a trapezoidal earth pressure distribution. Where there are no movement sensitive structures or services in close proximity to the excavation the maximum pressure (kPa) could be calculated using 5H (H equals the depth to the base of the excavation). Where the wall movement is to be minimised the maximum pressure could be calculated using 7H. The pressure distribution should increase from zero at the surface to the maximum value at a depth of 0.2 H and then decrease from the maximum at a depth of 0.8H back to zero at the base of the excavation.

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

Passive lateral resistance for piles founded in rock below the base of the excavation may be based on an ultimate passive restraint equal to 600 kPa in extremely low to very low strength rock and 2000 kPa in low to medium or greater strength rock. A factor of safety must be applied to the ultimate values to limit wall movement that is required to mobilise the passive resistance. Passive resistance should be assumed to start at least 0.5 m below bulk excavation level due to disturbance and fracturing of the rock.

Shoring walls should be designed for hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided.

9.3.5 Rock Anchors

The design of temporary rock anchors for the support shoring systems may be based on an allowable bond stress of 70 kPa in extremely low to very low strength rock and 300 kPa in low to medium or greater strength rock. The anchors should be bonded behind a line drawn up at 45° from the base of the shoring, and "lift-off" tests should be carried out to confirm the anchor capacities. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities, particularly for bonds formed in medium to high strength rock. It should be noted that permission will be required from adjacent property owners prior to installing bolts/anchors below their land. Consideration should be given to the location and depth of adjacent services when installing anchors.

It is anticipated that the building will restrain the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would generally need careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

9.4 Foundations

9.4.1 New Footings

All new structures should be uniformly founded on rock. Pad footings founded on sandstone should be possible for some areas of the northern and southern extension where competent sandstone is reasonably close to the bulk excavation level. Where the rock is deeper than say 1 m below the bulk level it will probably be beneficial to use piles to avoid additional excavation and support of sandy soils. Piles will most likely be required at some locations due to weak rock and dykes and therefore a piled foundation system may prove more economical.

The locations of the dykes and extent of highly fractured and weak sandstone on the site will require further investigation. Some of the new building columns will most likely be located close or on dykes or weak sandstone. At the surface, the dykes may include extremely low strength rock, or possibly very stiff to hard clay, and may not be suitable to support the column loads. If the dyke is reasonably narrow it may be possible to bridge across the dyke with pad footings either side. Alternatively piles could be used to penetrate to underlying sandstone or fresh igneous rock (encountered at 17.2 m in BH3). Further investigation may be required to confirm the depth to suitable foundations at affected columns for detailed design. On the C25 site, additional cored boreholes were drilled at selected column locations to allow the piling contractor to assess suitable founding levels. It is understood that most piles on the C25 site encountered competent rock or practical refusal within 10 m to 20 m depth.

Open bored piles will not be suitable due to the collapsing sandy soils. Bored piles with temporary liners to support the sandy soils could be considered, however, this piling technique can be time consuming and difficult. Continuous flight auger (CFA) piles would be suitable and avoid problems associated with collapsing soils and also groundwater seepage. Large piling rigs fitted with rock augers will be required to penetrate medium to high strength rock and productivity in this material will be slow. It may not be possible to effectively penetrate thick bands of very high strength rock. The piling contractor should be made aware of the ground conditions and should make their own assessment on the type of equipment required to carry out the piling works.

Footings founded on variable very low to medium strength sandstone (generally at the rock surface) may be designed for an allowable end bearing pressure of 1500 kPa whilst footings on uniform medium strength or greater sandstone or igneous rock could be designed for an allowable end bearing pressure of 3500 kPa. An allowable shaft adhesion equal to 10% of the end bearing values may be assumed for the rock strata. It expected that deep pads or short piles will generally be required to reach the medium to high strength rock which was present at about RL50 to 51.5 m. Higher bearing pressures of about 6000 kPa could be justified on high strength rock, however, further investigation would be required to confirm the depth and extent of this strata across the site. Footings designed using these allowable parameters could expect settlements of less than 1% of the footing width.

As a guide, pad footings founded on the extremely low strength igneous rock may be suitable for an allowable end bearing pressure of about 500 - 700 kPa, whilst piles could be designed for an allowable end bearing pressure of about 1000 - 1500 kPa and an allowable shaft adhesion of 30 kPa. Differential settlements may occur between footings founded on sandstone and footings founded on weak igneous rock. Further review and analysis will be required if varying foundations and footings are proposed. Preferably all footings should be founded on uniformly consistent materials where possible.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters.

9.4.2 Existing Footings

Preliminary assessment on the capacity of the existing footings to accommodate additional loads may be based on the design parameters provided in Section 9.4.1.

The original structural drawings for the Wallace Wurth building by Morrison & Little Pty Ltd indicate that the footing system comprises pad footings founded on rock. The pads vary in size from 0.7 m × 0.7 m × 0.7 m deep to 2 m × 2 m × 1.3 m deep. The base of the pad footings vary from RL 49.8 to 51.8 m. The design bearing pressure for the rock foundation is not shown on the drawings and has not been confirmed, but given the age of the building, it is likely that the footings would have been designed in accordance with Ordinance 70 for which a maximum bearing pressure of 2300 kPa was allowed.

Based on the borehole data and expected founding levels, it is anticipated that the footings are probably founded on medium to high strength sandstone which may be suitable for an allowable end bearing pressure of 3500 kPa. It is however possible that some of the footings may also be founded on the upper variable strength rock and possibly weak igneous rock which must be considered in the assessment.

Further investigation and review of more heavily loaded or sensitive pad footings may be carried out to confirm foundation strata and to assess the capacity of the footings to accommodate additional loads if this is considered to be critical to the design and performance of the building.

9.5 Floor Slabs and Pavements

During construction of pavements and floor slabs all topsoil, organic and deleterious material should be stripped and stockpiled separately for disposal or use in landscaping areas. Proof rolling of the exposed subgrade (soil only) should be carried out under the supervision of a geotechnical engineer to detect any soft or heaving areas. Any soft spots detected during proof rolling would need to be stripped to a competent base and replaced with engineered filling.

Engineered filling should be placed in layers and compacted to a minimum dry density ratio of 98% relative to standard compaction (or equivalent density index of 75% for sandy soils). The density ratio should be increased to 100% standard compaction (or equivalent density index of 80%) within 0.3 m of the subgrade surface. The existing filling, sand and excavated rock on site should generally be suitable for re-use as engineered filling provided it has a maximum particle size of 70 mm and moisture content within 2% of OMC.

Subject to the subgrade preparation outlined above, the design of pavements on sandy subgrade may be based on a CBR value of 5%. Design of pavements on weathered rock may be based on a CBR value of 10%. These CBR values assume all pavements are protected by adequate surface and subsoil drainage to minimise the risk of water infiltration and softening of pavement materials.

Control joints should be provided in the slab at the soil and rock interface to reduce the effects of differential movement and possible cracking of the slabs at these locations.

9.6 Earthquake Design

In accordance with AS1170-2007 “Structural Design Actions, Part 4 : Earthquake Actions in Australia” a hazard factor (Z) of 0.08 and a site subsoil Class B_e is considered to be appropriate for the site. This assumes that all structural loads are uniformly founded on very low strength or better rock.

10. LIMITATIONS

Douglas Partners (DP) has prepared this report for the Wallace Wurth Redevelopment at UNSW, Kensington in accordance with the letter by Bovis Lend Lease Pty Ltd (BLL) dated 20

January 2010. This report is provided for the exclusive use of the BLL 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 is based on observations, measurements, tests or derived interpretations. The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time. The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site. Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others, 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

Reviewed by

Scott Easton
Senior Associate

Michael J Thom
Principal

APPENDIX A

Notes Relating to this Report
Results of Field Work

Douglas Partners
Geotechnics • Environment • Groundwater

NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring 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.

Description and Classification Methods

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

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from ‘feel’ and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, “Methods of Testing Soils for Engineering Purposes” — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the ‘N’ value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\, c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein 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. 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 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, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

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

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company 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 condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

Reproduction of Information for Contractual Purposes

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Tender Documents”, published by the Institution of Engineers, Australia. Where information obtained from this investigation 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. The Company 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 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.

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DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{S(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{S(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

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

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

SEDIMENTARY ROCK TYPES

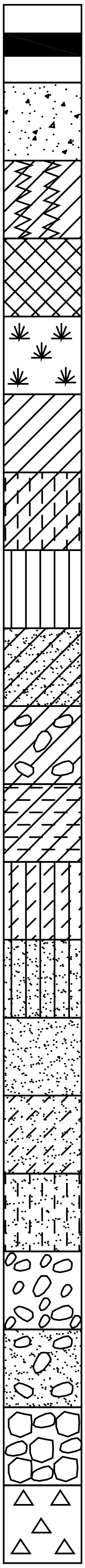
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.

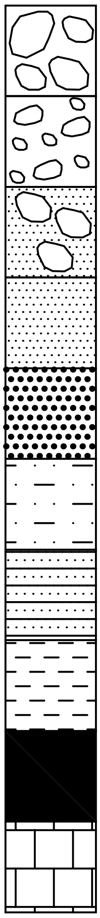
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



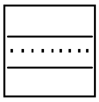
- BITUMINOUS CONCRETE
- CONCRETE
- TOPSOIL
- FILLING
- PEAT
- CLAY
- SILTY CLAY
- SILT
- SANDY CLAY
- GRAVELLY CLAY
- SHALY CLAY
- CLAYEY SILT
- SANDY SILT
- SAND
- CLAYEY SAND
- SILTY SAND
- GRAVEL
- SANDY GRAVEL
- COBBLES/BOULDER
- TALUS

SEDIMENTARY ROCK

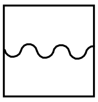


- BOULDER CONGLOMERATE
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE FINE GRAINED
- SANDSTONE COARSE GRAINED
- SILTSTONE
- LAMINITE
- MUDSTONE, CLAYSTONE, SHALE
- COAL
- LIMESTONE

SEAMS

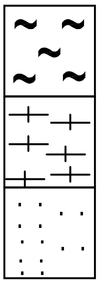


SEAM
>10mm



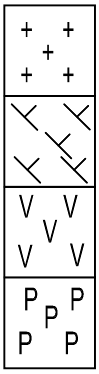
SEAM
<10mm

METAMORPHIC ROCK



- SLATE, PHYLLITE, SCHIST
- GNEISS
- QUARTZITE

IGNEOUS ROCK



- GRANITE
- DOLERITE, BASALT
- TUFF
- PORPHYRY

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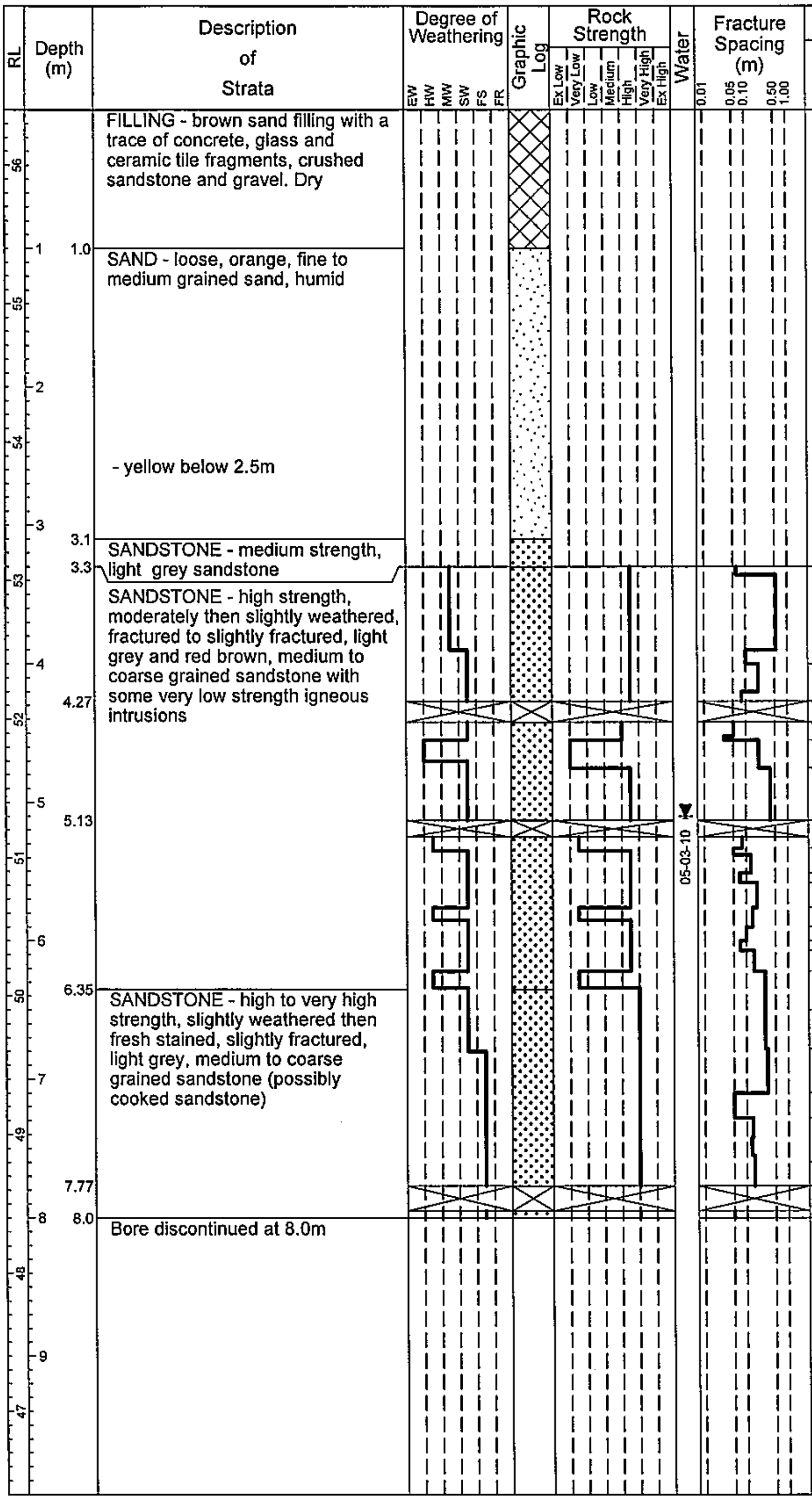
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 56.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 71543
DATE: 08 Feb 10
SHEET 1 **OF** 1

Depth (m)	Description of Strata
	FILLING - brown sand filling with a trace of concrete, glass and ceramic tile fragments, crushed sandstone and gravel. Dry
1.0	SAND - loose, orange, fine to medium grained sand, humid - yellow below 2.5m
3.1	SANDSTONE - medium strength, light grey sandstone
3.3	SANDSTONE - high strength, moderately then slightly weathered, fractured to slightly fractured, light grey and red brown, medium to coarse grained sandstone with some very low strength igneous intrusions
4.27	
5.13	
6.35	SANDSTONE - high to very high strength, slightly weathered then fresh stained, slightly fractured, light grey, medium to coarse grained sandstone (possibly cooked sandstone)
7.77	
8.0	Bore discontinued at 8.0m



Discontinuities

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

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

3.36m: J60°, healed, curved, ironstained
3.9m: B0°, ironstained
4m: B10°, clay smear
4.2m: J45°, rough
4.27m: CORE LOSS: 150mm
4.42-4.52m: (x2) B0°- 5°, ironstained
4.55-4.70m: igneous rock (dyke) intrusion
4.75m: J55°, rough
5.13m: CORE LOSS: 120mm
5.33m: J60°, clayey, ironstained
5.51m: J55°, clay smear
5.58m: J85°, rough, clay smear
5.76m: B5°, sandy clay band
5.9m: B20°, ironstained
6m: J, subvertical, rough
6.07m: B0°, ironstained
6.22m: B0°, sandy clay band
6.5m: J25°, ironstained
6.78-7.05m: J80°, rough
7.1-7.23m: (x2) J50°, rough
7.28m: J75°- 85°, rough
7.6m: J35°, ironstained, rough
7.77m: CORE LOSS: 180mm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
A			
A			
S			1,3,5 N = 8
A			
S			1,3,3 N = 6
C	88	60	PL(A) = 1.7MPa PL(A) = 2MPa PL(A) = 2MPa
C	100	60	
C	89	79	PL(A) = 1.9MPa PL(A) = 3.1MPa

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 3.3m
TYPE OF BORING: Solid flight auger to 3.3m; NMLC-Coring to 8.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Groundwater well installed to 7.8m. Water measured in well at 5.1m on 5/3/10

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

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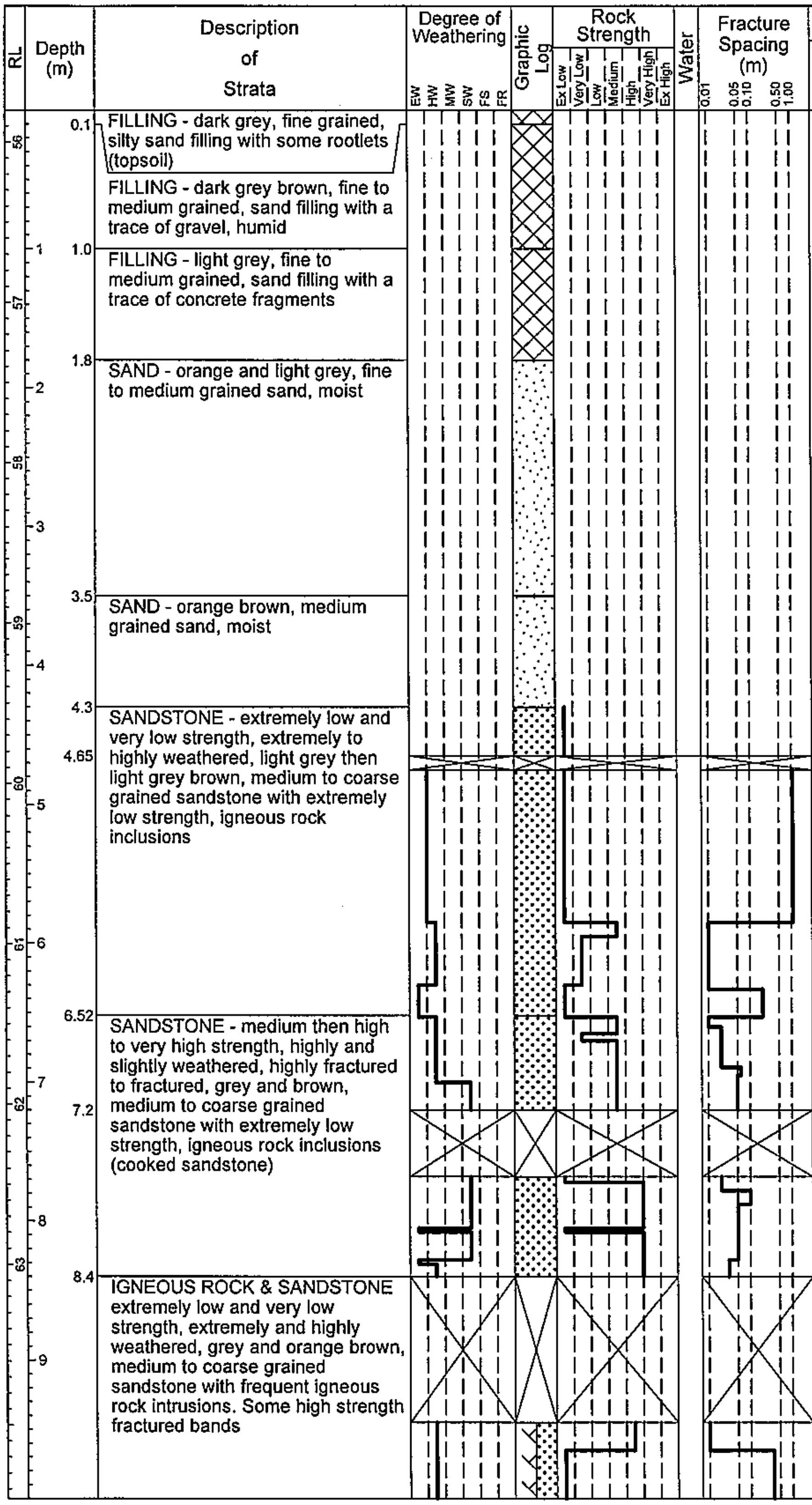
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 55.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 60°/0

BORE No: 2
PROJECT No: 71543
DATE: 01 Feb 10
SHEET 1 **OF** 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	FILLING - dark grey, fine grained, silty sand filling with some rootlets (topsoil)		A A A			
	FILLING - dark grey brown, fine to medium grained, sand filling with a trace of gravel, humid		A/E A			
1.0	FILLING - light grey, fine to medium grained, sand filling with a trace of concrete fragments		A/E			
1.8	SAND - orange and light grey, fine to medium grained sand, moist		A/E			
3.5	SAND - orange brown, medium grained sand, moist	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10° or joints	A			
4.3	SANDSTONE - extremely low and very low strength, extremely to highly weathered, light grey then light grey brown, medium to coarse grained sandstone with extremely low strength, igneous rock inclusions					
4.65		4.65m: CORE LOSS: 100mm 4.75-4.85m: dyke	C	92	0	
						PL(A) = 0.7MPa
		6.33-7.25m: dyke	C	100	0	
6.52	SANDSTONE - medium then high to very high strength, highly and slightly weathered, highly fractured to fractured, grey and brown, medium to coarse grained sandstone with extremely low strength, igneous rock inclusions (cooked sandstone)	6.53-7.25m: highly fractured to fractured sandstone (dyke) in subvertical joints				
7.2		7.2m: CORE LOSS: 480mm				
		7.68-7.78m: fragmented in 0.02m intervals 7.88-8.13m: J, subvertical, rough 8.05-8.08m: clay band 8.13m: J75°, rough 8.28-8.31m: clay band 8.31m: J, subvertical, ironstained	C	62	0	PL(A) = 1.9MPa PL(A) = 4.9MPa
8.4	IGNEOUS ROCK & SANDSTONE extremely low and very low strength, extremely and highly weathered, grey and orange brown, medium to coarse grained sandstone with frequent igneous rock intrusions. Some high strength fractured bands	8.4m: CORE LOSS: 1050mm 9.45-9.65m: fragmented to 0.02m intervals	C	34	0	PL(A) = 1.3MPa

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 2.6m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.65m; NMLC-Coring to 14.4m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 14.4m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials: STE
Date: 17/3/10

Douglas Partners
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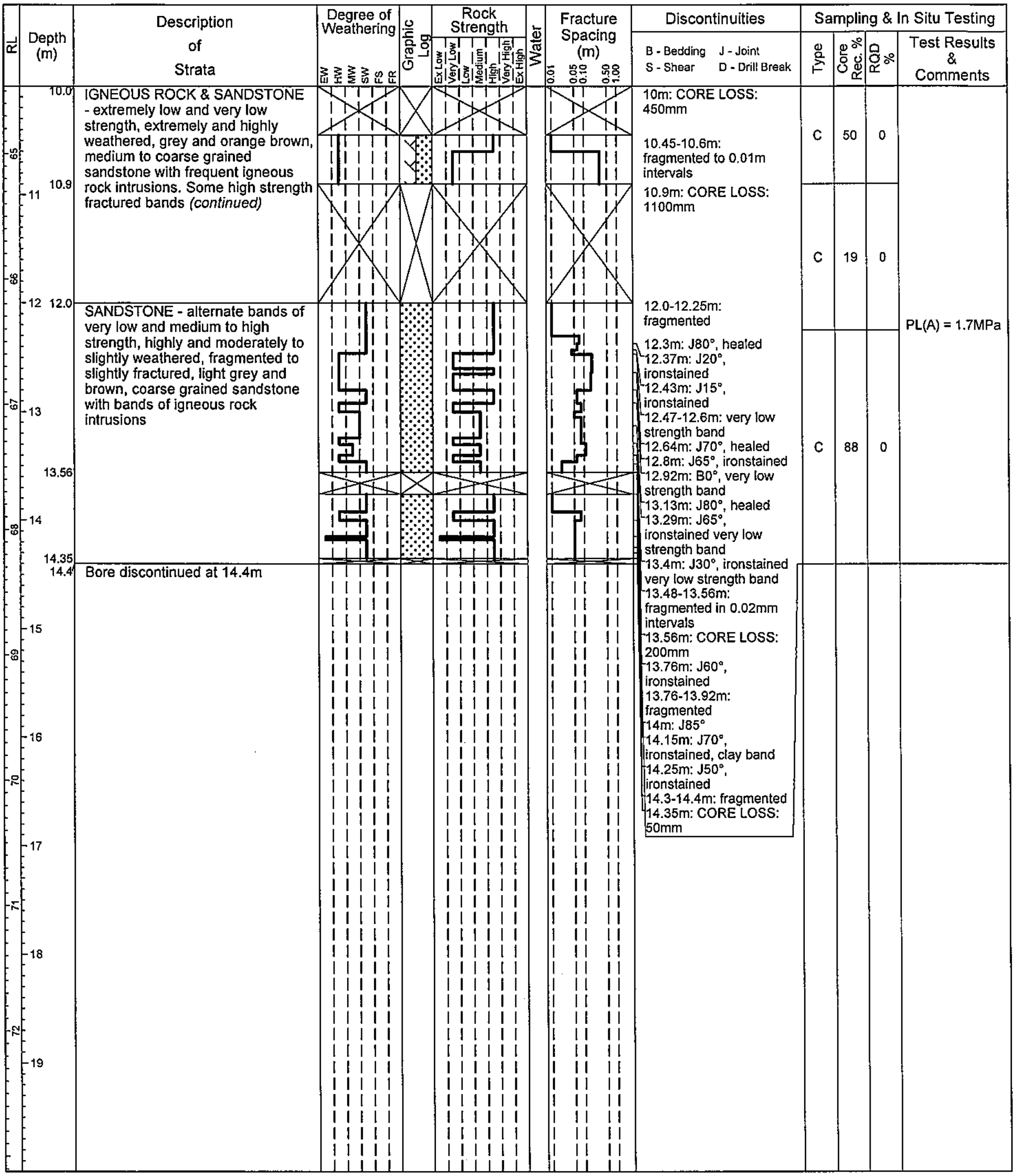
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 55.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 60°/0

BORE No: 2
PROJECT No: 71543
DATE: 01 Feb 10
SHEET 2 **OF** 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
10.0	IGNEOUS ROCK & SANDSTONE - extremely low and very low strength, extremely and highly weathered, grey and orange brown, medium to coarse grained sandstone with frequent igneous rock intrusions. Some high strength fractured bands *(continued)*	10m: CORE LOSS: 450mm 10.45-10.6m: fragmented to 0.01m intervals	C	50	0	
10.9		10.9m: CORE LOSS: 1100mm	C	19	0	
12.0	SANDSTONE - alternate bands of very low and medium to high strength, highly and moderately to slightly weathered, fragmented to slightly fractured, light grey and brown, coarse grained sandstone with bands of igneous rock intrusions	12.0-12.25m: fragmented 12.3m: J80°, healed 12.37m: J20°, ironstained 12.43m: J15°, ironstained 12.47-12.6m: very low strength band 12.64m: J70°, healed 12.8m: J65°, ironstained 12.92m: B0°, very low strength band 13.13m: J80°, healed 13.29m: J65°, ironstained very low strength band 13.4m: J30°, ironstained very low strength band 13.48-13.56m: fragmented in 0.02mm intervals	C	88	0	PL(A) = 1.7MPa
13.56		13.56m: CORE LOSS: 200mm 13.76m: J60°, ironstained 13.76-13.92m: fragmented 14m: J85° 14.15m: J70°, ironstained, clay band 14.25m: J50°, ironstained 14.3-14.4m: fragmented				
14.35		14.35m: CORE LOSS: 50mm				
14.4	Bore discontinued at 14.4m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 2.6m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.65m; NMLC-Coring to 14.4m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 14.4m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

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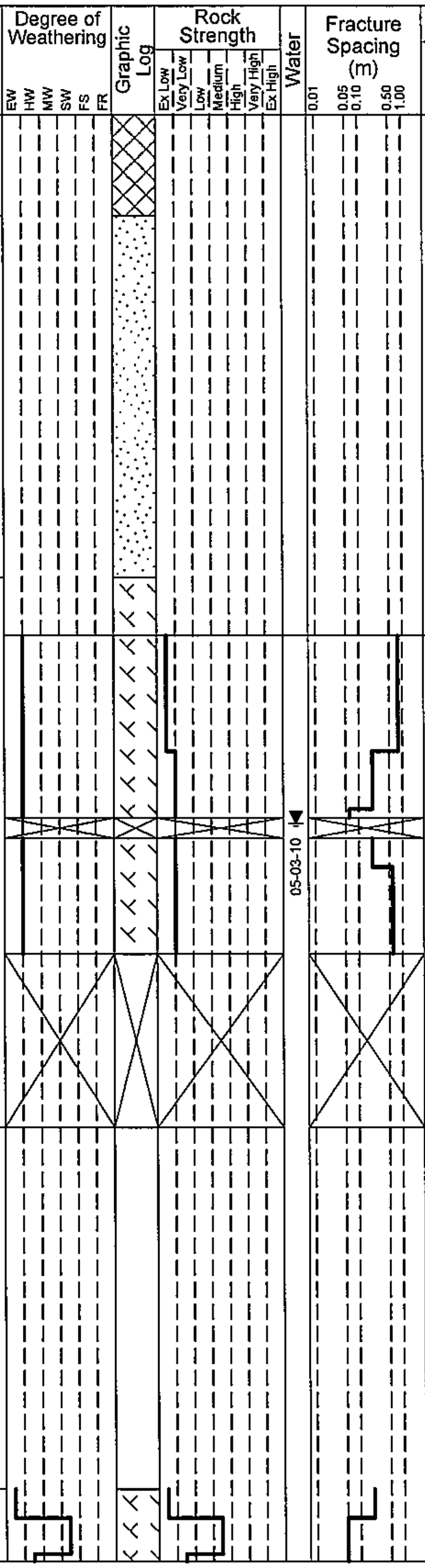
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 56.2 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 71543
DATE: 3-8 Feb 2010
SHEET 1 OF 2



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

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	FILLING - dark brown, sand filling		A A			
0.7	SAND - medium dense, light brown, fine to medium grained sand		S			4,6,9 N = 15
	- yellow below 2.2m		S A			3,6,11 N = 17
3.2	IGNEOUS ROCK (DYKE) - extremely low to very low strength, extremely to highly weathered, light grey to red brown, igneous rock (dyke)	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	C	90	0	
4.86		4.86m: CORE LOSS: 140mm				
5.8		5.8m: CORE LOSS: 1200mm	C	40	0	
7.0	ROTARY DRILLING	7.0-9.5m: rotary drilling in extremely weathered dyke	R			
9.5	IGNEOUS ROCK - description next page	9.7-9.95m: fragmented into 0.05mm intervals	C	83	0	PL(A) = 0.8MPa

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 3.6m

TYPE OF BORING: Solid flight auger to 3.6m; NMLC-Coring to 7.0m; Rotary to 9.5m; NMLC-Coring to 13.0m; Rotary to 14.5m; NMLC-Coring to 15.15m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 18.0m. Water measured in well at 4.9m on 5/3/10

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

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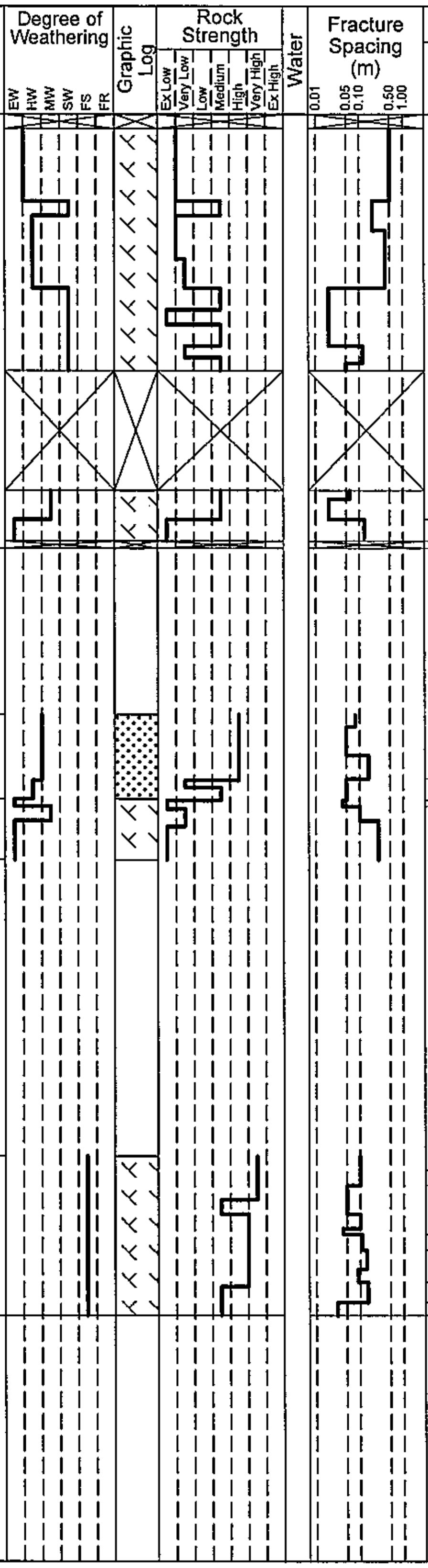
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 56.2 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 71543
DATE: 3-8 Feb 2010
SHEET 2 **OF** 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
46 45 44 43	10.0 11 11.77 12 12.95 13	IGNEOUS ROCK (DYKE) - extremely low to very low and medium strength, extremely to highly weathered, fractured, light grey brown to red brown, igneous rock with medium strength sandstone bands	10m: CORE LOSS: 100mm 10.6m: J70°, clay band 11.2-11.6m: fragmented in 0.02mm intervals 11.77m: CORE LOSS: 830mm 12.6-12.8m: fragmented in 0.02 to 0.05mm intervals 12.8-12.95m: clay band 12.95m: CORE LOSS: 50mm	C	69	0	PL(A) = 0.6MPa PL(A) = 0.5MPa
42	13.0 14	ROTARY DRILLING	13.0-14.15m: rotary drilling to weathered rock	R			
41	14.15	SANDSTONE - high then medium strength, highly to moderately weathered, fractured, red brown, coarse grained sandstone (cooked sandstone)	14.23m: J10°, 30mm clay 14.23-14.43m: (x4) B0°-5°, ironstained 14.59m: B0°, 20mm clay	C	100	0	PL(A) = 2.1MPa
	14.73 15	IGNEOUS ROCK (DYKE) - extremely low to very low strength, extremely weathered, light grey brown, igneous rock (dyke)	14.74-14.8m: extremely low strength dyke 14.78-15.15m: extremely low strength dyke				
40	15.15 16 17	ROTARY DRILLING		R			
39 38	17.2 18	IGNEOUS ROCK (DYKE) - high to very high strength, fresh stained, fractured to slightly fractured, light greenish grey and brown, igneous rock (dyke). Some medium strength bands	17.2m: J, subvertical, healed, ironstained 17.3m: J70°, healed, ironstained 17.4-17.6m: J, subvertical, partially healed, fragmented into 0.05mm intervals 17.74m: J85°, partially healed 17.85m: J70° 18.07m: J65°, rough 18.21-18.3m: J75°, clay smear & fragmented into 0.03mm intervals	C	100	63	PL(A) = 9.2MPa PL(A) = 3MPa PL(A) = 0.7MPa
37	18.3 19	Bore discontinued at 18.3m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 3.6m

TYPE OF BORING: Solid flight auger to 3.6m; NMLC-Coring to 7.0m; Rotary to 9.5m; NMLC-Coring to 13.0m; Rotary to 14.5m; NMLC-Coring to 15.15m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 18.0m. Water measured in well at 4.9m on 5/3/10

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

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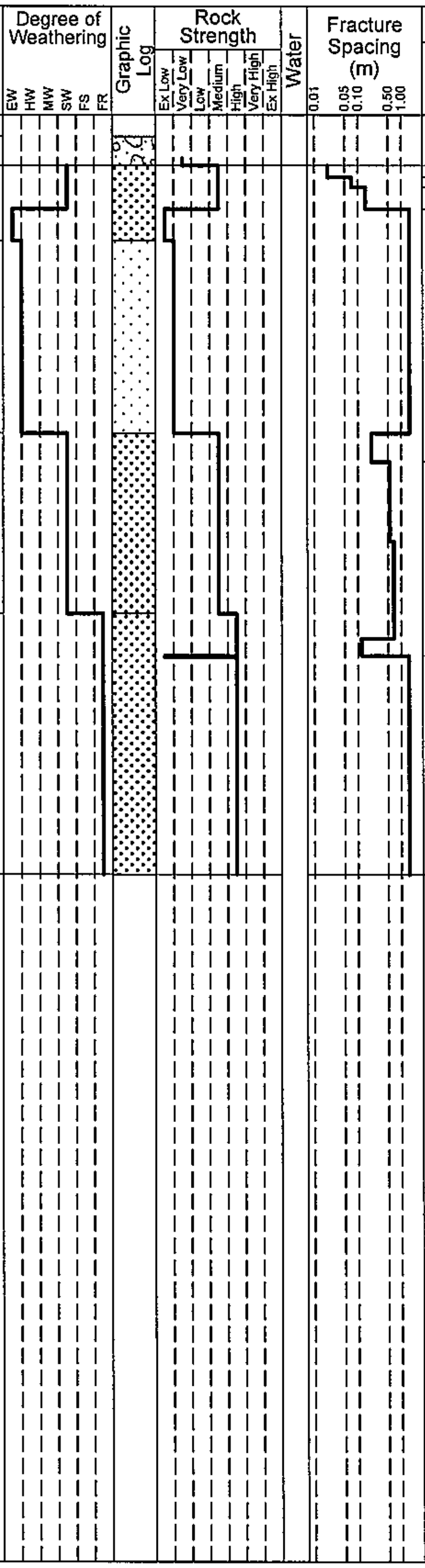
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 53.3 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 71543
DATE: 29 Jan 10
SHEET 1 **OF** 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.15	ASPHALTIC CONCRETE	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10° or joints	A			
0.35	ROADBASE GRAVEL		A			
0.87	SANDSTONE - medium strength, slightly weathered, fractured to slightly fractured, light grey brown, medium to coarse grained sandstone 0.65-0.88m: clay band	0.43m: B5°, ironstained 0.5m: B0°, clay veneer 0.65m: B0°, clay smear, ironstained	A A C	100	0	PL(A) = 0.8MPa
2.2	SANDSTONE - extremely to very low strength, extremely to highly weathered, light grey to grey, fine grained sandstone		C	100	0	
3.45	SANDSTONE - medium strength, slightly weathered, slightly fractured, light grey and brown, medium to coarse grained sandstone	2.2m: B0°, sandy clay band 2.4m: B5°, clay smear 2.95m: B0°- 5°, clay smear				PL(A) = 0.6MPa PL(A) = 0.7MPa
5.25	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained massive sandstone	3.62m: B5°, clay veneer 3.74m: B0°, 10mm clay 3.74-5.25m: massive	C	100	100	PL(A) = 1.5MPa PL(A) = 1.8MPa
	Bore discontinued at 5.25m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 0.35m
TYPE OF BORING: Solid flight auger to 0.35m; NMLC-Coring to 5.25m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ≡ Water level

CHECKED
Initials: STC
Date: 17/3/10

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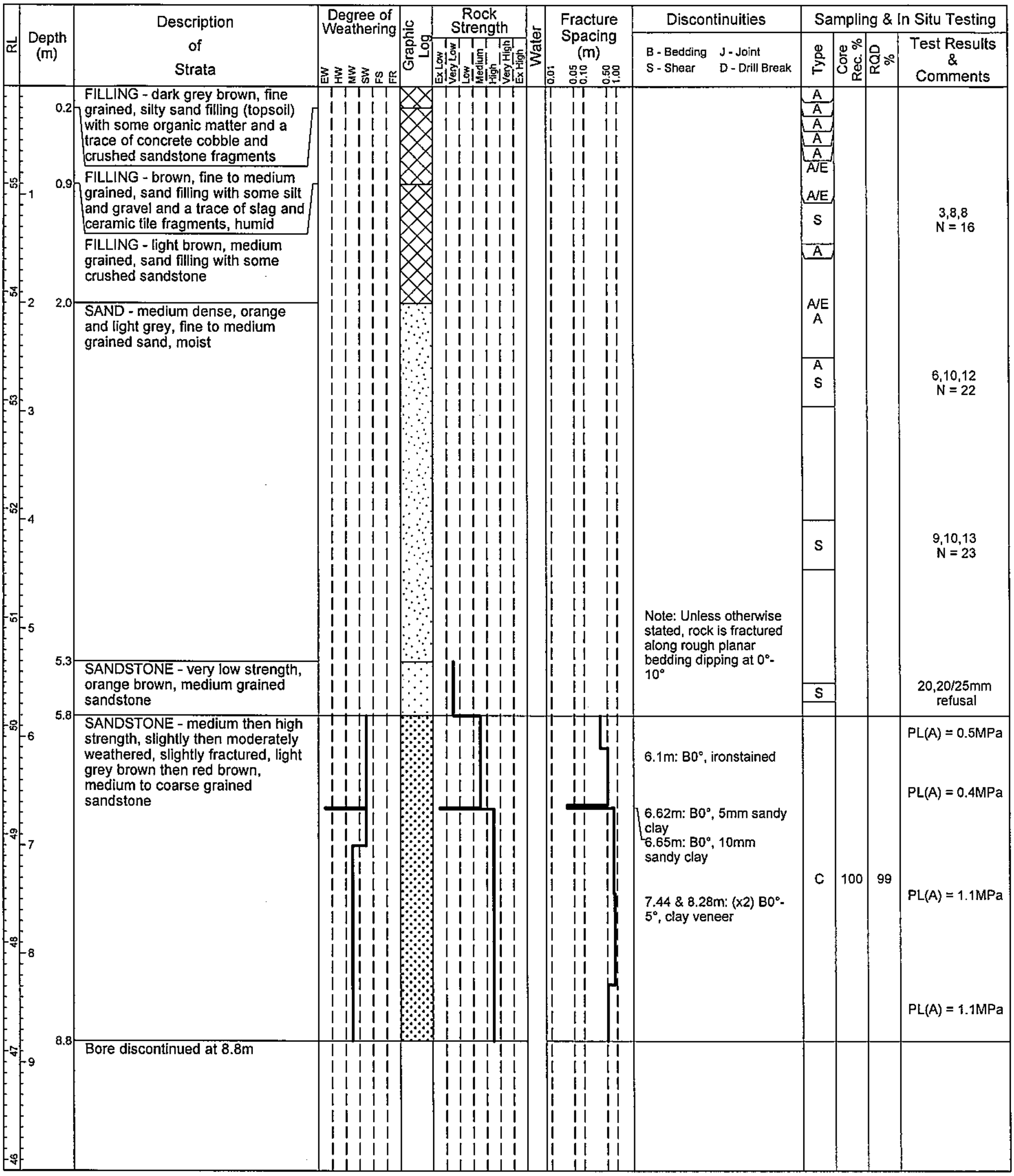
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 55.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 71543
DATE: 01 Feb 10
SHEET 1 OF 1



RL	Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	0.2	FILLING - dark grey brown, fine grained, silty sand filling (topsoil) with some organic matter and a trace of concrete cobble and crushed sandstone fragments		A A A A A			
55	0.9 1	FILLING - brown, fine to medium grained, sand filling with some silt and gravel and a trace of slag and ceramic tile fragments, humid		A/E A/E S			3,8,8 N = 16
		FILLING - light brown, medium grained, sand filling with some crushed sandstone		A			
54 53 52 51	2.0 2 3 4 5	SAND - medium dense, orange and light grey, fine to medium grained sand, moist	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10°	A/E A A S S			6,10,12 N = 22 9,10,13 N = 23
	5.3	SANDSTONE - very low strength, orange brown, medium grained sandstone		S			20,20/25mm refusal
50 49 48	5.8 6 7 8	SANDSTONE - medium then high strength, slightly then moderately weathered, slightly fractured, light grey brown then red brown, medium to coarse grained sandstone	6.1m: B0°, ironstained 6.62m: B0°, 5mm sandy clay 6.65m: B0°, 10mm sandy clay 7.44 & 8.28m: (x2) B0°-5°, clay veneer	C	100	99	PL(A) = 0.5MPa PL(A) = 0.4MPa PL(A) = 1.1MPa PL(A) = 1.1MPa
47 46	8.8 9	Bore discontinued at 8.8m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 5.8m

TYPE OF BORING: Solid flight auger to 5.5m; Rotary to 5.8m; NMLC-Coring to 8.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

Douglas Partners
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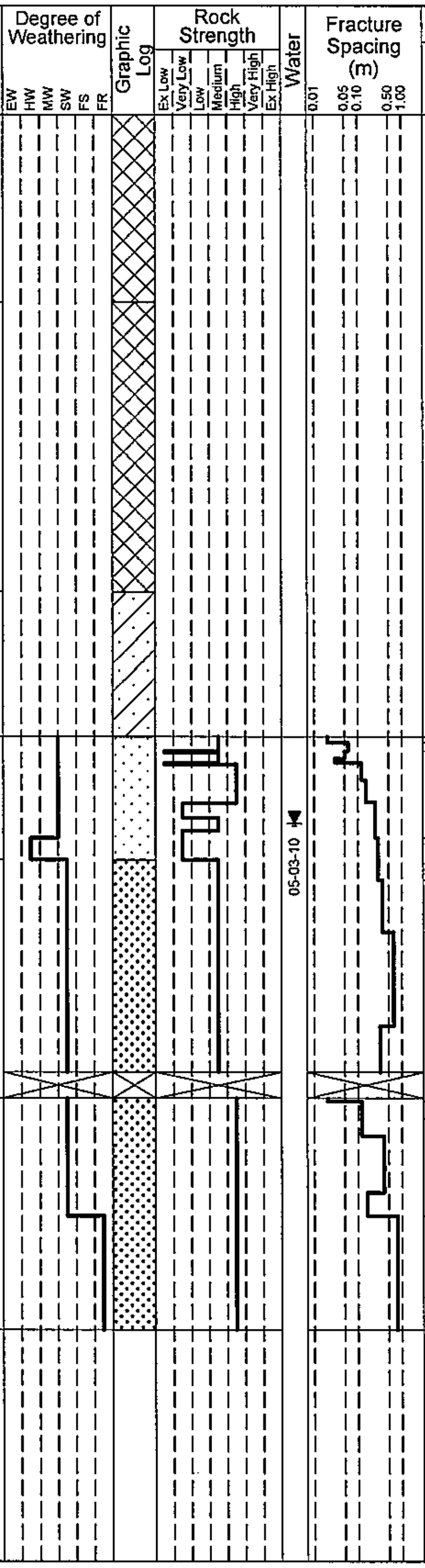
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 56.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 71543
DATE: 09 Feb 10
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	FILLING - grey brown, sand filling with some sandstone gravel, glass and concrete fragments and a trace of shell fragments, ceramic tile fragments, plastic and steel wire pieces		A A A* A			
1.3	FILLING - yellow sand filling with some sandstone gravel		A			
			S			1,2,1 N = 3
			A/E A/E			
			S			0,1,3 N = 4
			A			
3.3	SANDY CLAY - grey brown, sandy clay	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	A A			
4.3	SANDSTONE - medium strength, highly and moderately to slightly weathered, fractured to slightly fractured, light grey brown, fine to medium grained sandstone. Some very low strength bands 4.49-4.76m: high strength band	4.3-4.34m: (x2) B0°, ironstained 4.34m: J65°, rough, clay smear 4.4-4.75m: (x5) B0°- 5°, clay smear/veneer 5m: B0°, sandy clay band	C	100	62	PL(A) = 1.7MPa
5.15	SANDSTONE - medium strength, fresh, slightly fractured, light grey, coarse grained sandstone	5.29m: J45°, clay smear & rough 5.65m: B5°, ironstained 6.3m: J75°, slightly ironstained				PL(A) = 0.8MPa
6.62		6.62m: CORE LOSS: 180mm				
6.8	SANDSTONE - high strength, slightly weathered and fresh, slightly fractured, light grey and red brown, coarse grained sandstone	6.82m: B5°, ironstained 6.94 & 7.06m: (x2) B5°, ironstained 7.45m: J80°, partially healed 7.61m: B5°, clay smear	C	93	93	PL(A) = 2.2MPa PL(A) = 2.4MPa PL(A) = 1.6MPa
8.4	Bore discontinued at 8.4m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 3.5m

TYPE OF BORING: Solid flight auger to 3.0m; Rotary to 4.3m; NMLC-Coring to 8.4m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 7.7m. Water measured in well at 4.9m on 5/3/10

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

Douglas Partners
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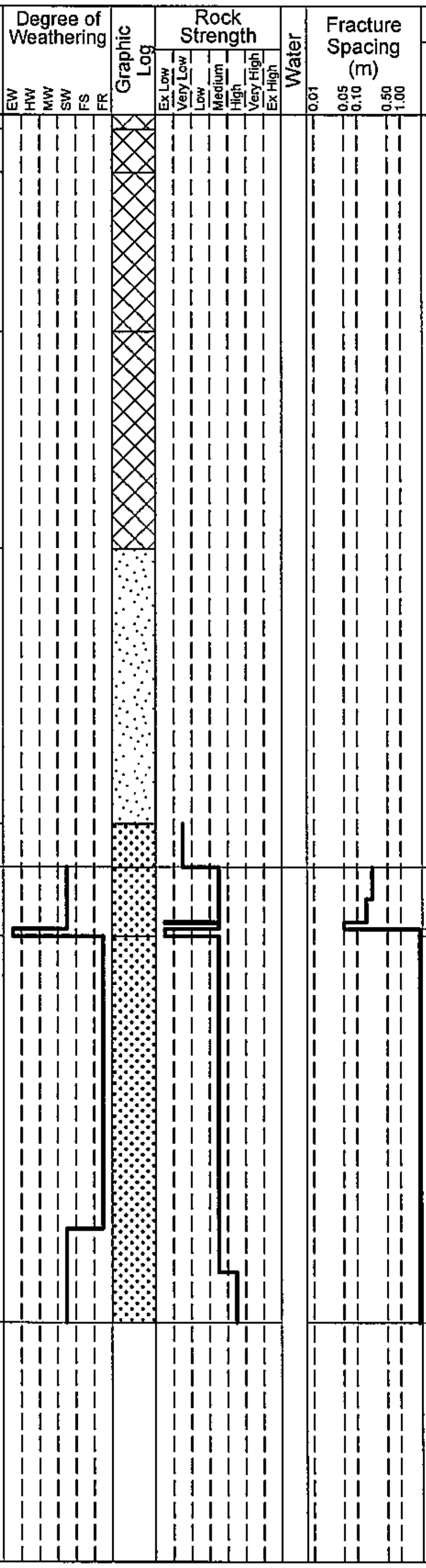
BOREHOLE LOG

CLIENT: Bovis Lend Lease
PROJECT: Wallace Wurth Redevelopment
LOCATION: Cnr High & Botany St, UNSW, Kensington

SURFACE LEVEL: 55.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 71543
DATE: 29 Jan 10
SHEET 1 **OF** 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
0.1	FILLING - brown, fine grained, silty sand filling (topsoil) with some gravel and rootlets		A A			
0.4	FILLING - brown sand and crushed sandstone filling, humid		A/E A/E			
	FILLING - light to dark brown, fine to medium grained sand filling, with a trace of gravel, humid to moist 1.3-1.45m: concrete fragments		A/E			
1.5	FILLING - light brown, fine to medium grained sand with some sandstone gravel (possible natural)		A A A S			4,5,5 N = 10
3.0	SAND - medium dense, orange brown, medium grained sand, moist	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	A A A S			11,15,20/10mm refusal
4.9	SANDSTONE - very low strength, light grey brown, medium grained sandstone					
5.2	SANDSTONE - medium strength, moderately weathered, fractured to slightly fractured, red brown, medium grained sandstone	5.42m: B5°, clay veneer 5.58m: B10°, 10mm sandy clay 5.63m: B5°, 50mm clay 5.68-8.35m: massive sandstone	C	100	97	PL(A) = 0.6MPa
5.68	SANDSTONE - medium then high strength, slightly weathered and fresh, unbroken, light grey and red brown, medium to coarse grained, massive sandstone					PL(A) = 0.8MPa PL(A) = 0.9MPa PL(A) = 1.4MPa
8.35	Bore discontinued at 8.35m					

RIG: Bobcat **DRILLER:** Steve S **LOGGED:** SI **CASING:** HW to 5.0m
TYPE OF BORING: Solid flight auger to 5.2m; NMLC-Coring to 8.35m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 17/3/10

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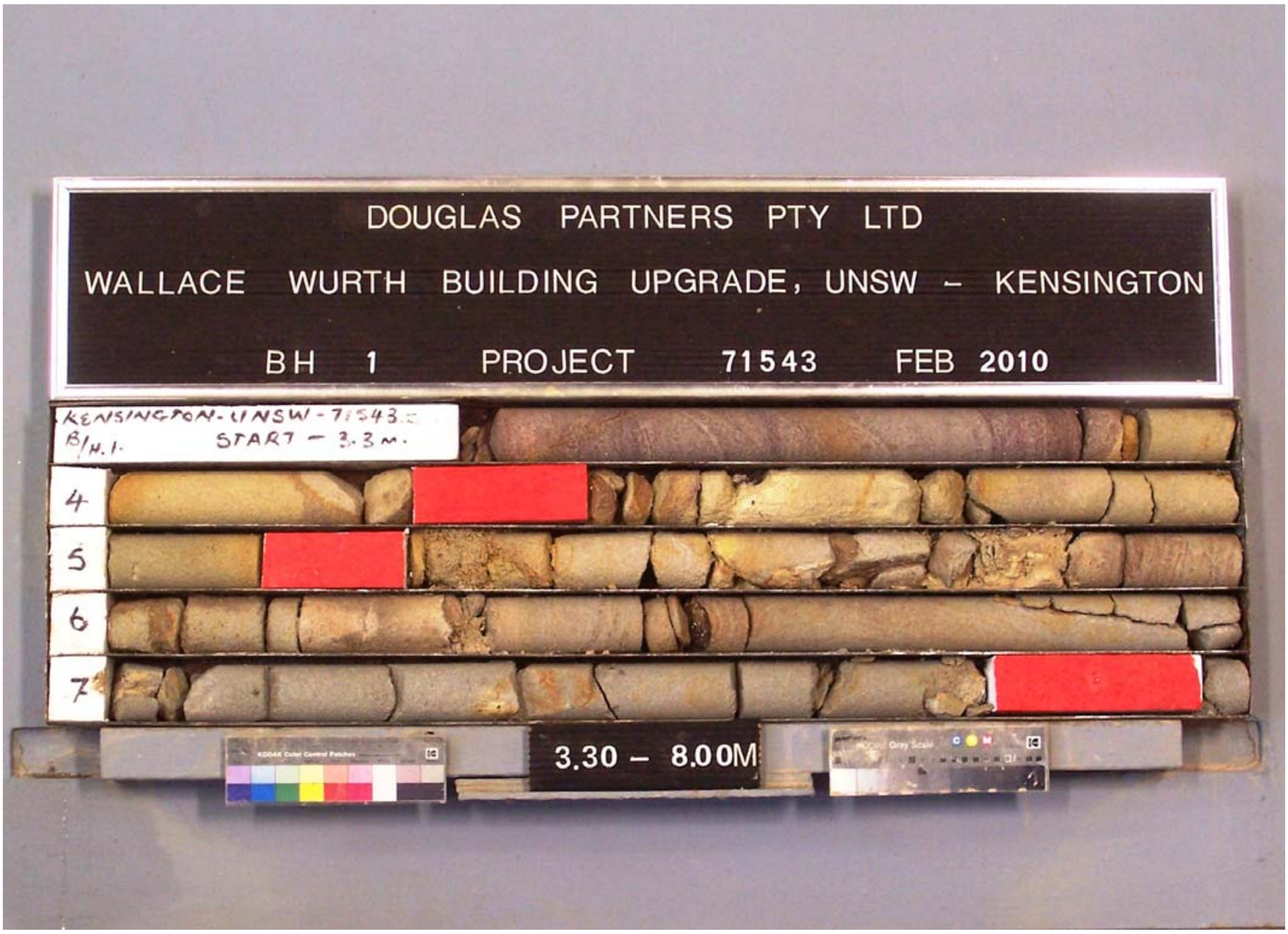
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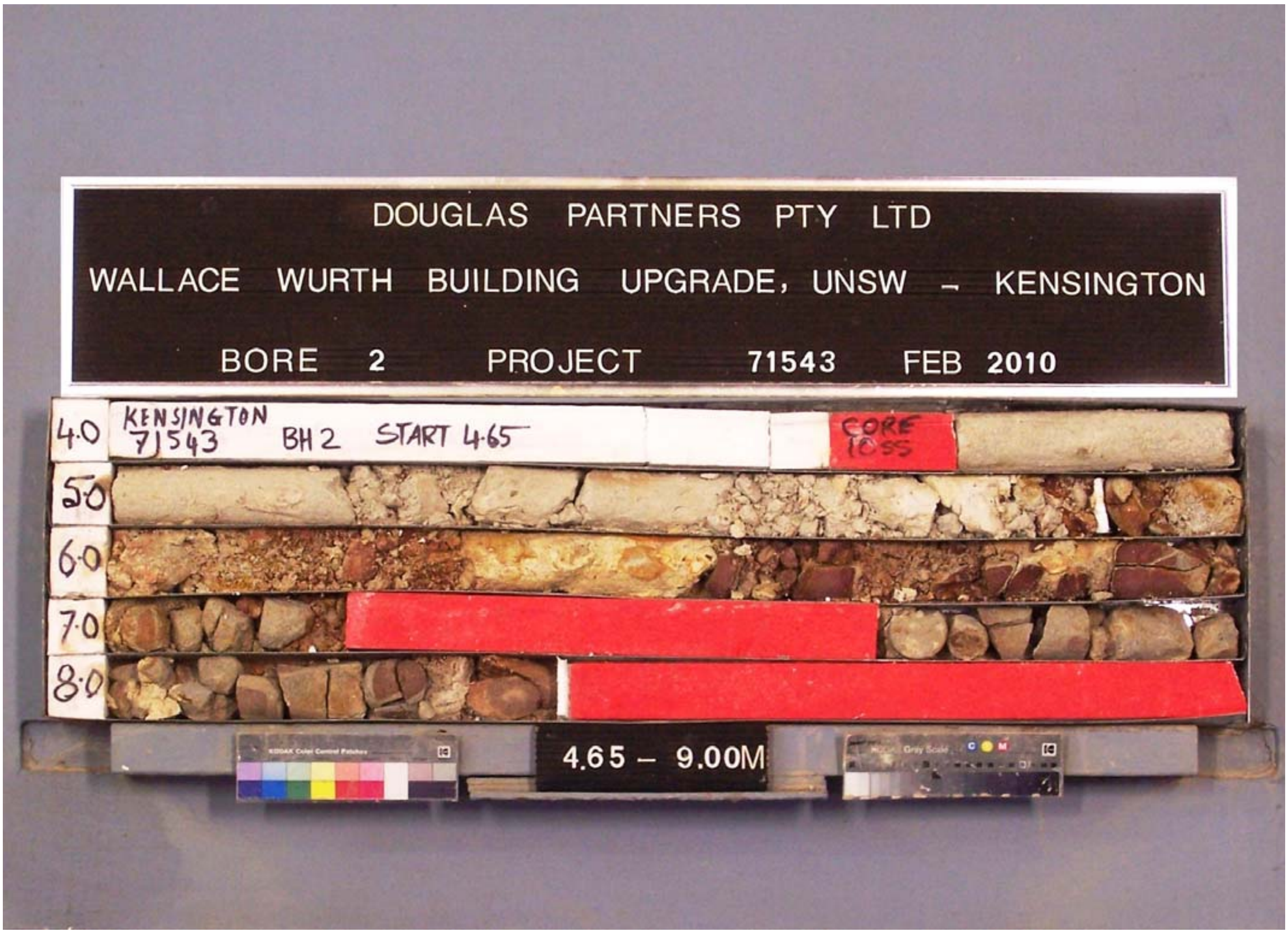
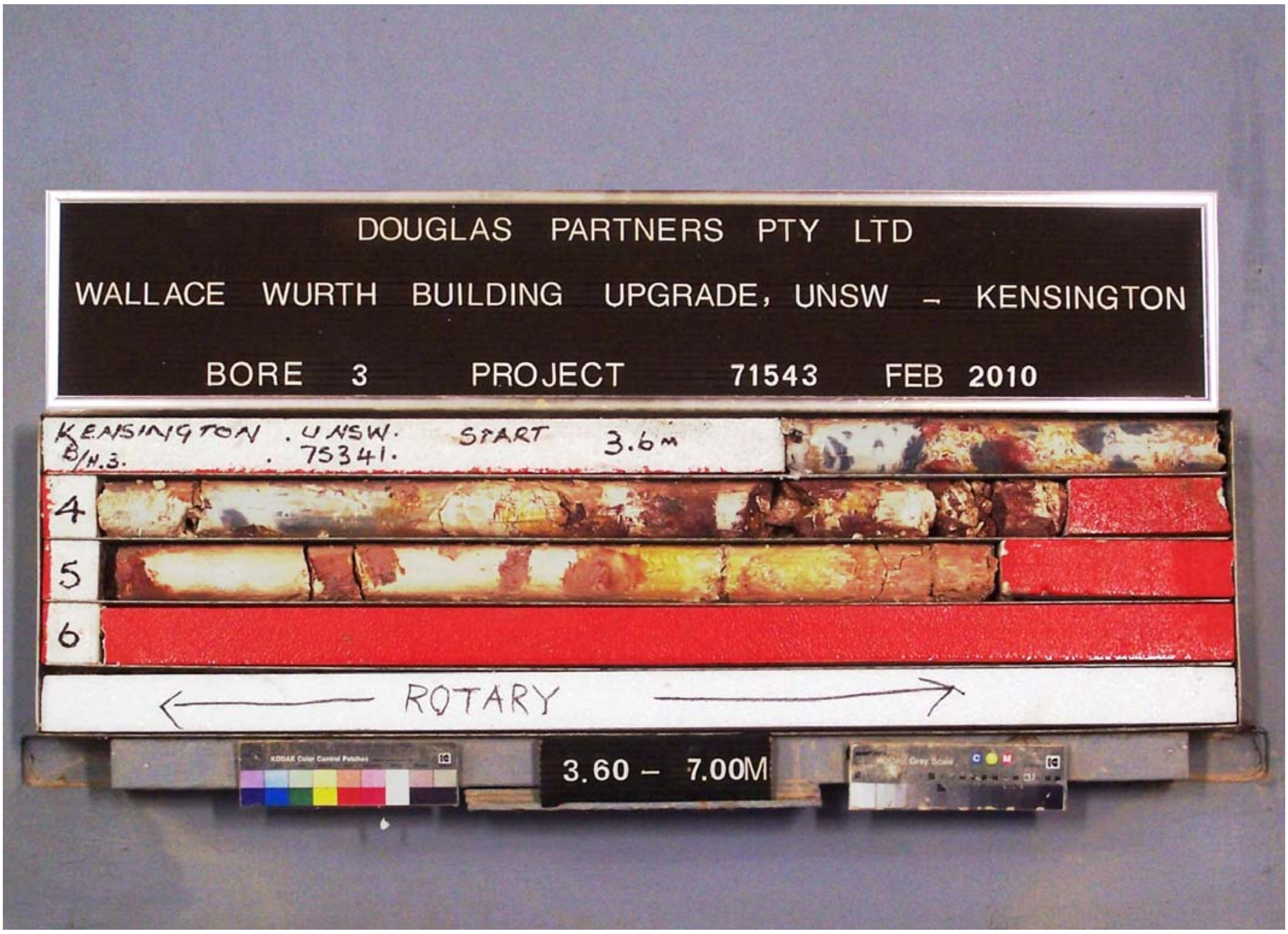
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BH 1 PROJECT 71543 FEB 2010
KENSINGTON-UNSW-71543
B/H.1. START – 3.3M.
4
5
6
7
KODAK Color Control Patches
3.30 – 8.00M
KODAK Gray Scale


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 2 PROJECT 71543 FEB 2010
4.0
KENSINGTON 71543 BH2 START 4.65
CORE LOSS
5.0
6.0
7.0
8.0
4.65 – 9.00M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 2 PROJECT 71543 FEB 2010
9.0
10.0
SS
12
13.
9.00 –14.00M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW - KENSINGTON
BORE 2 PROJECT 71543 FEB 2010
U.N.S.W. KENSINGTON - 71543
B/H.2. @ 60°
START 12.25m.
14
END OF HOLE 14.4m
KODAK Color Control Patches
14.00 – 14.40M
Gray Scale
C
Y
M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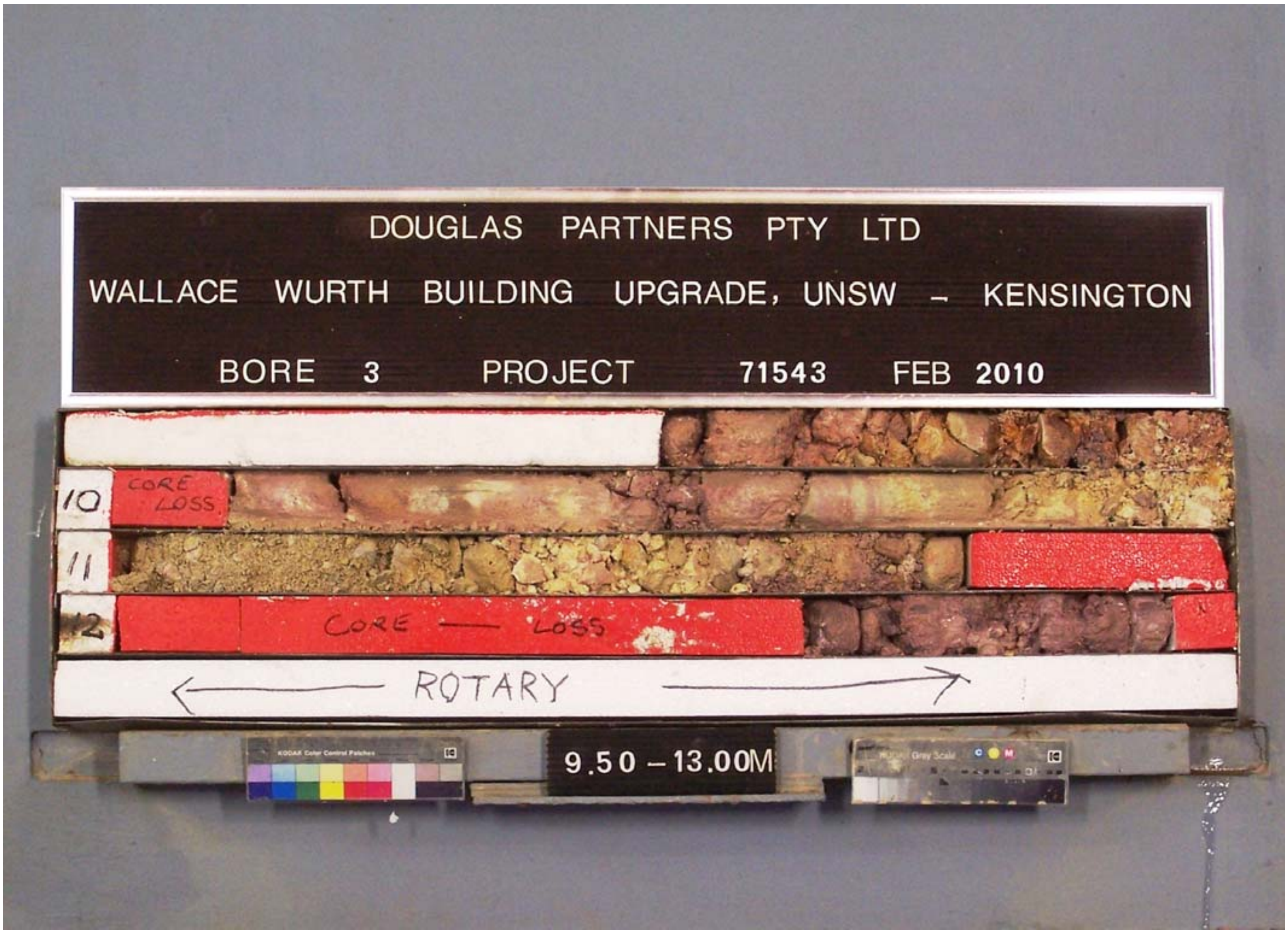
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 3 PROJECT 71543 FEB 2010
KENSINGTON . UNSW. START 3.6m
B/H.3. . 75341.
4
5
6
ROTARY
3.60 – 7.00M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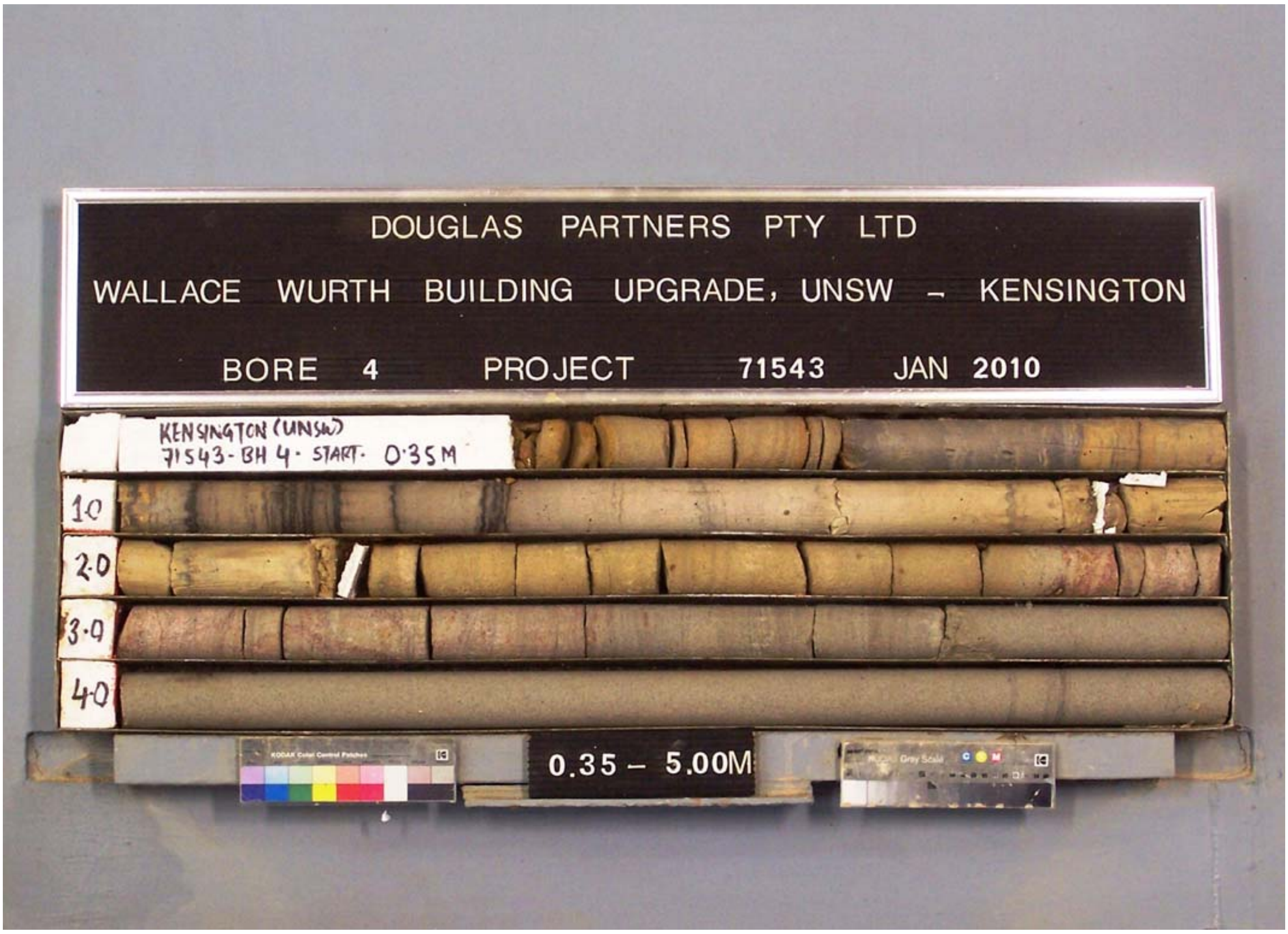
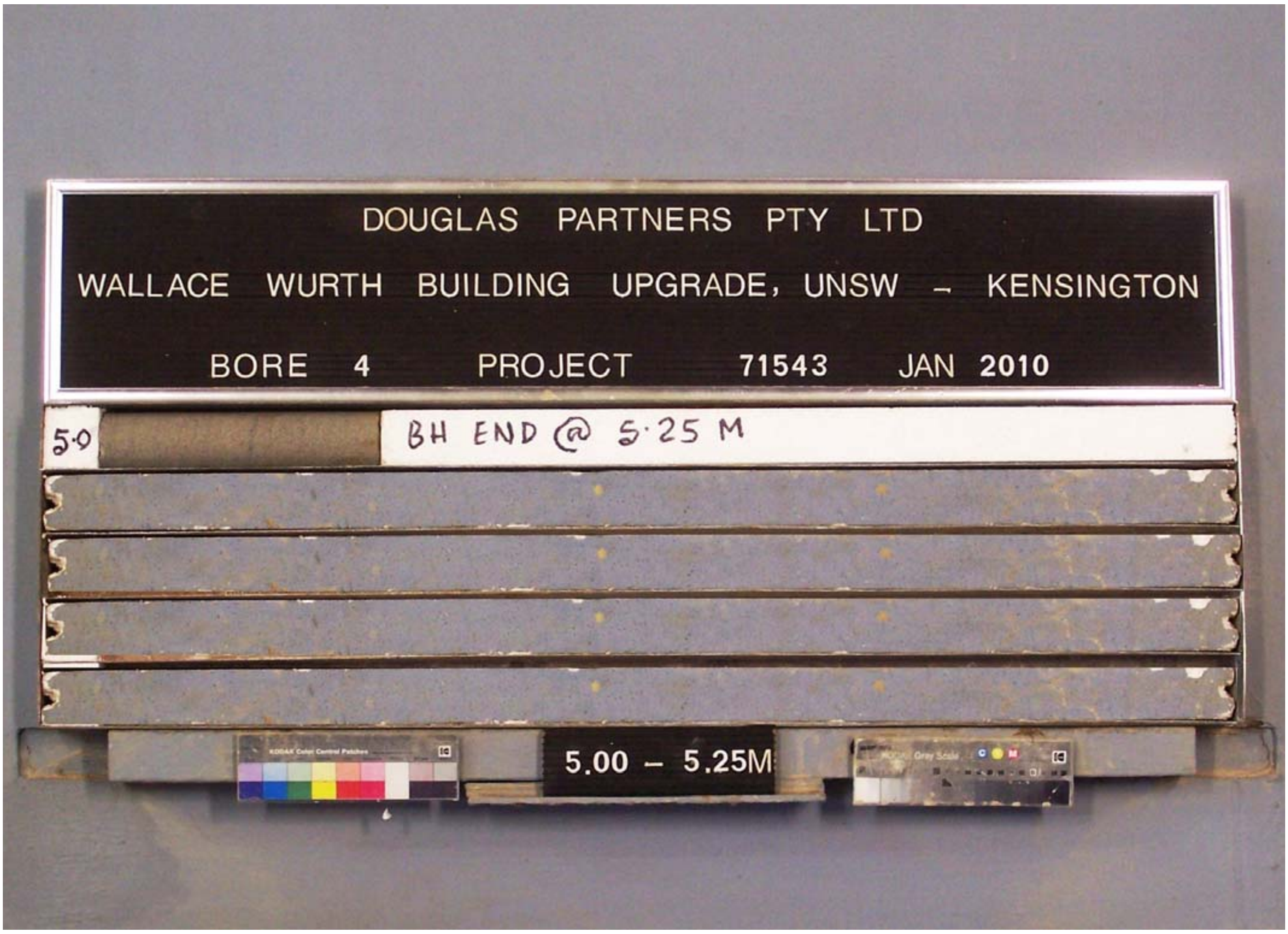
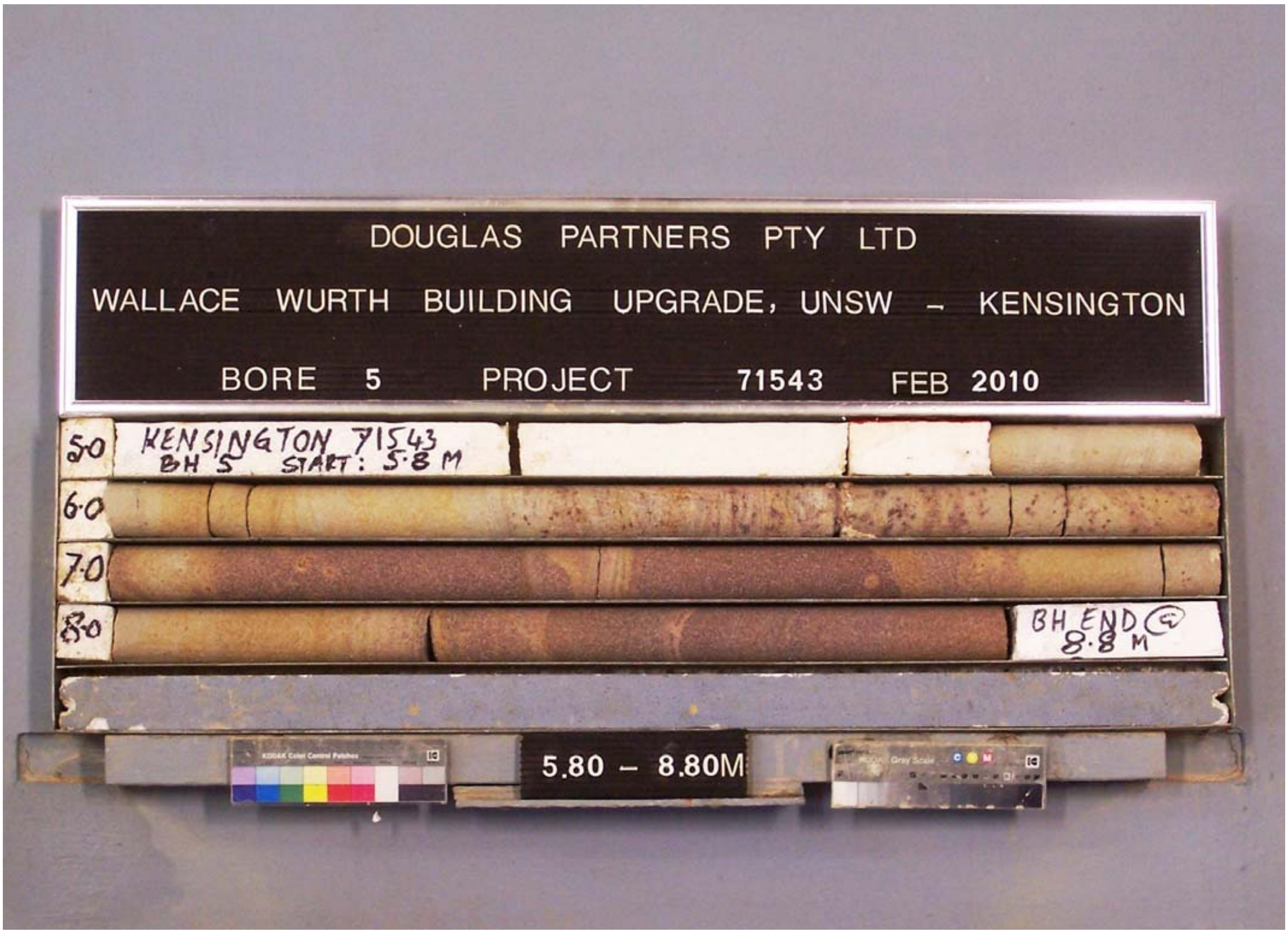
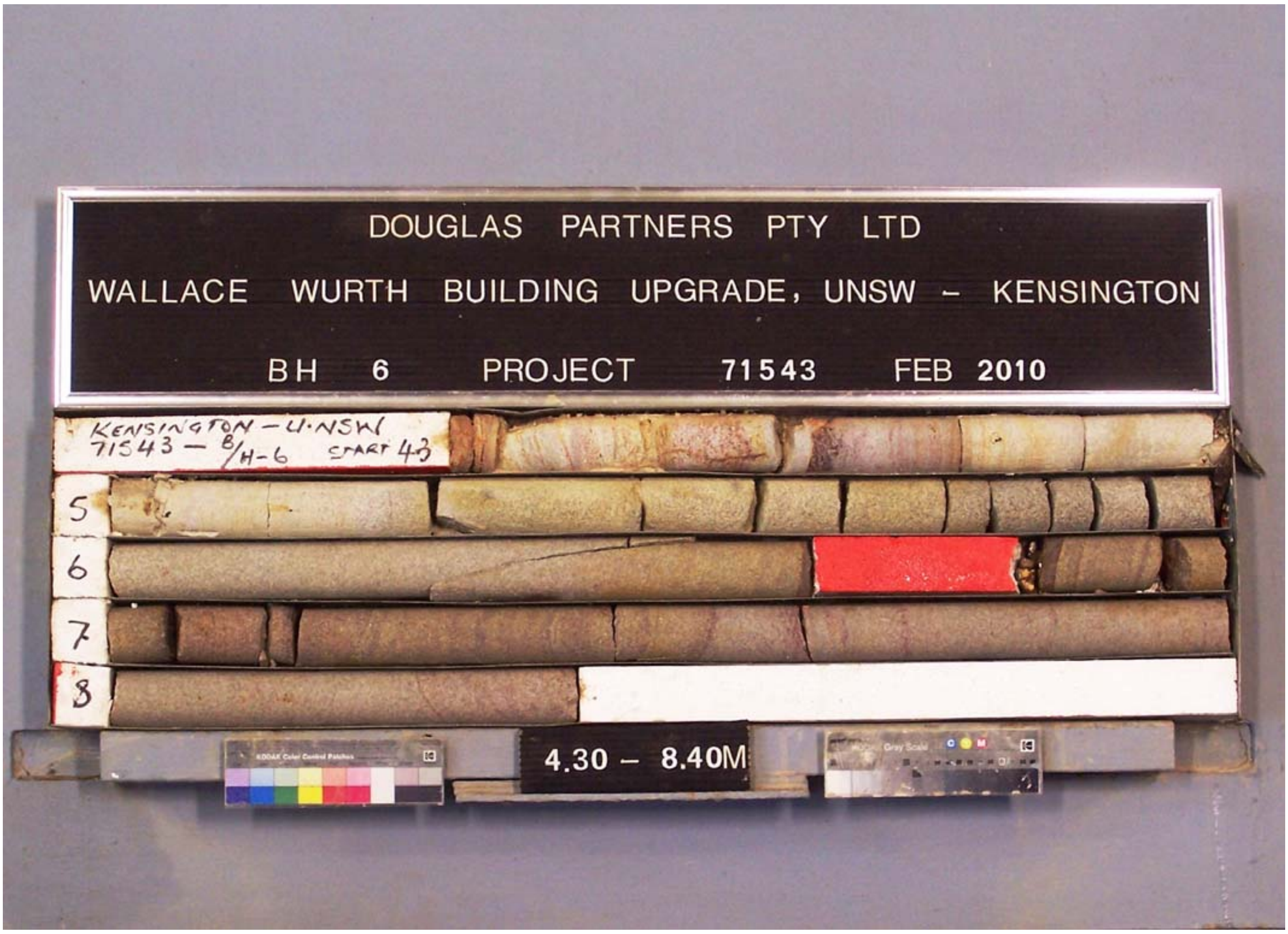
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 3 PROJECT 71543 FEB 2010
10
CORE LOSS
11
12
CORE — LOSS
ROTARY
9.50 – 13.00M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW - KENSINGTON
BORE 3 PROJECT 71543 FEB 2010
14
15
← ROTARY DRILLING →
16
17
18
BH END @ 18·3 M
14.15 – 18.30M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 4 PROJECT 71543 JAN 2010
KENSINGTON (UNSW)
71543-BH 4. START. 0.35M
1.0
2.0
3.0
4.0
0.35 – 5.00M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 4 PROJECT 71543 JAN 2010
5.0
BH END @ 5.25 M
5.00 – 5.25M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW - KENSINGTON
BORE 5 PROJECT 71543 FEB 2010
5.0
KENSINGTON 71543
BH 5 START: 5.8 M
6.0
7.0
8.0
BH END @
8.8 M
KODAK Color Control Patches
5.80 – 8.80M
KODAK Gray Scale
C Y M


DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BH 6 PROJECT 71543 FEB 2010
KENSINGTON – U.NSW
71543 – B/H-6 START 4.3
5
6
7
8
4.30 – 8.40M

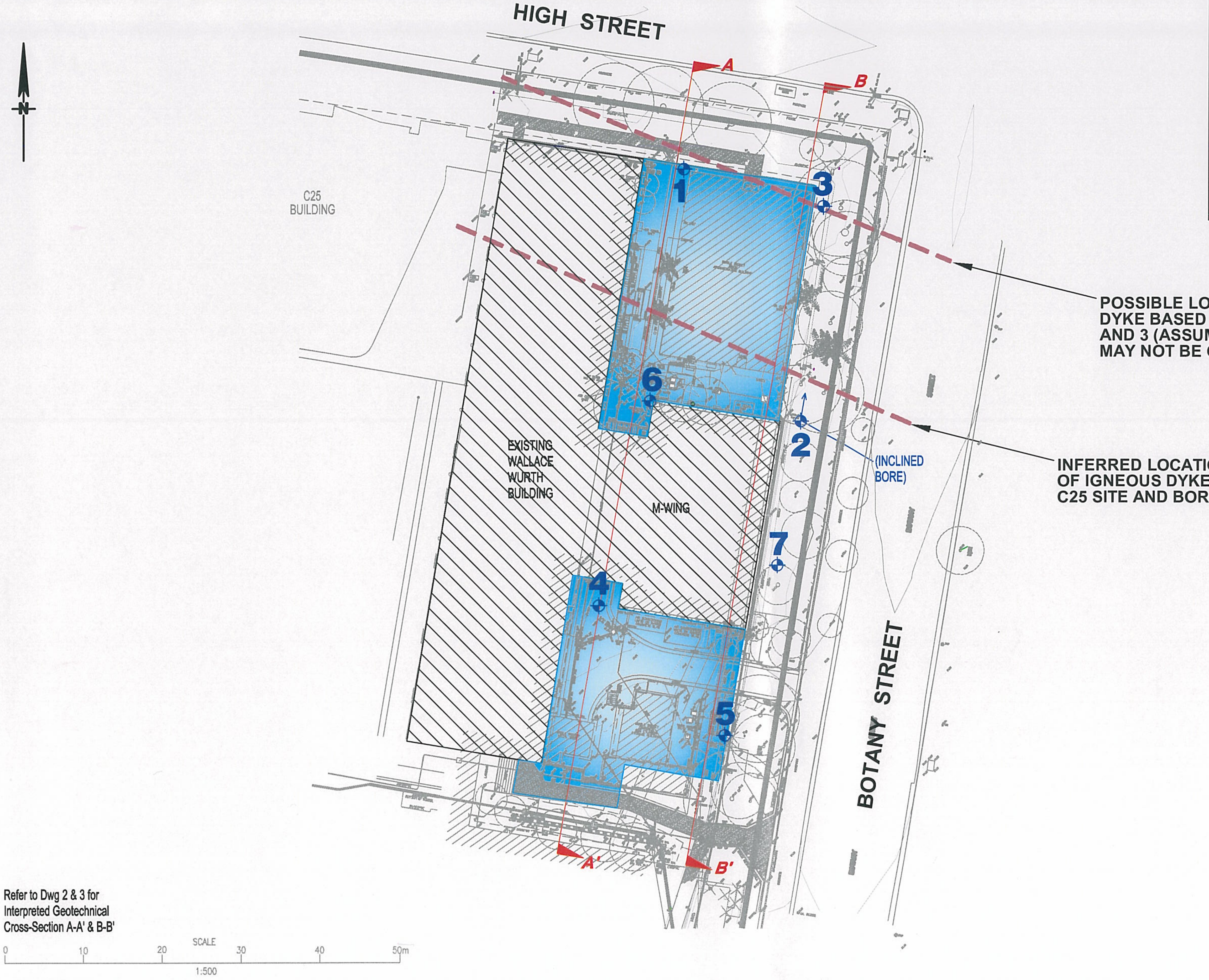

DOUGLAS PARTNERS PTY LTD
WALLACE WURTH BUILDING UPGRADE, UNSW – KENSINGTON
BORE 7 PROJECT 71543 FEB 2010
NSW KENSINGTON
71543. B/H-7. START. 5.2
START
6
7
8
FIN 8.3 m.
5.20 – 8.30M

APPENDIX B

Drawing No. 1 – Location of Tests

Drawing No. 2 – Inferred Geotechnical Cross Section (A-A')

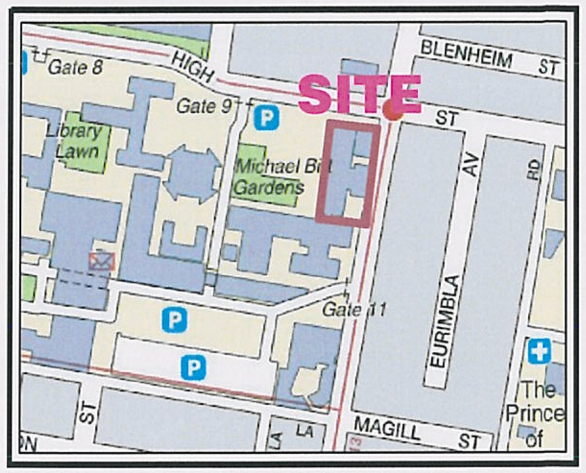
Drawing No. 3 – Inferred Geotechnical Cross Section (B-B')



POSSIBLE LOCATION OF DYKE BASED ON BORES 1 AND 3 (ASSUMED ONLY AND MAY NOT BE CORRECT)

INFERRED LOCATION OF IGNEOUS DYKE FROM C25 SITE AND BORE 2

Refer to Dwg 2 & 3 for
Interpreted Geotechnical
Cross-Section A-A' & B-B'



LOCALITY PLAN

LEGEND

⊕ TEST BORE LOCATION

🟦 AREA OF PROPOSED SEVEN-STOREY EXTENSIONS

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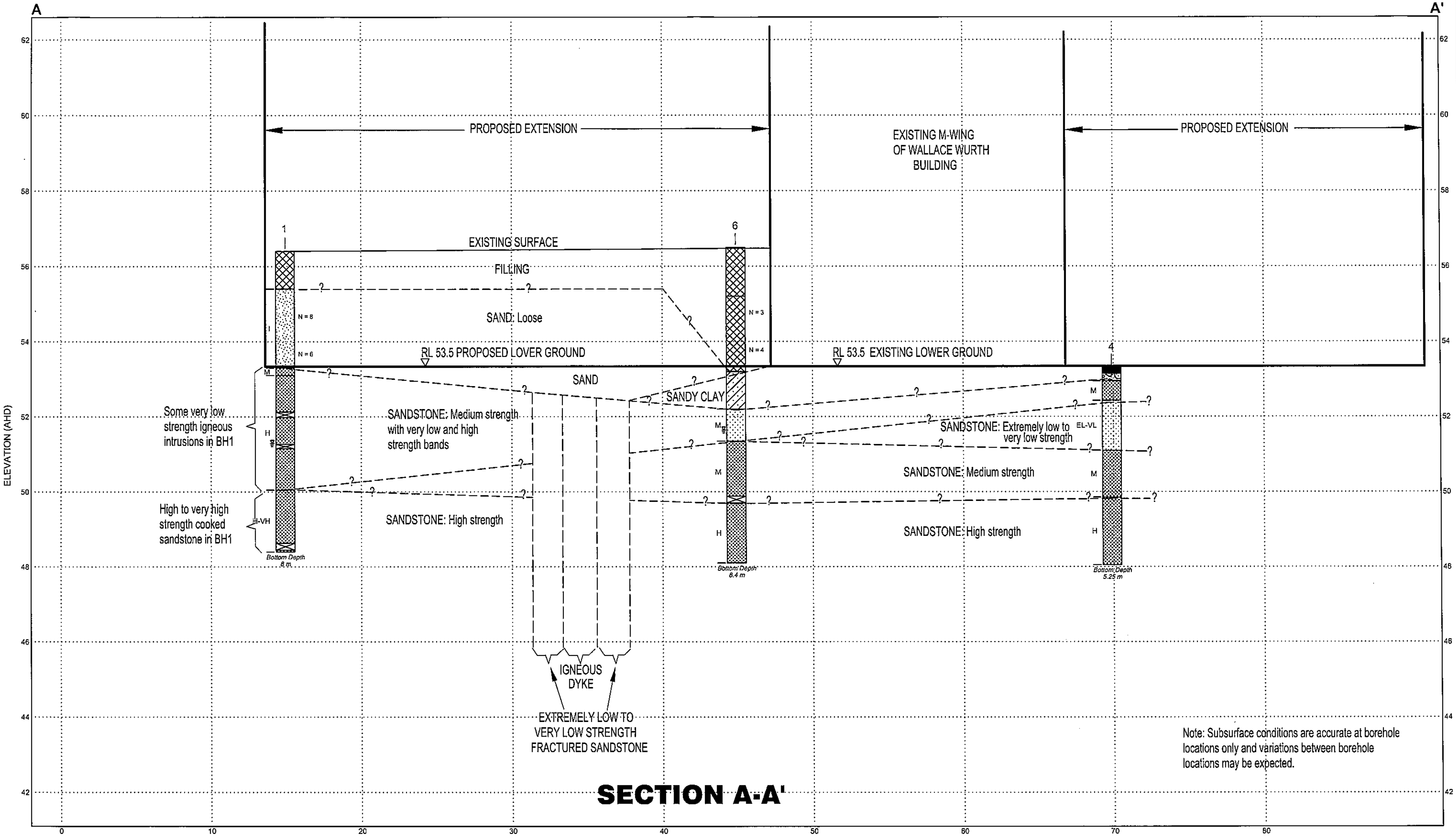
CLIENT: Bovis Lend Lease
DRAWN BY: PSCH
SCALE: 1:500 @ A3
OFFICE: Sydney
APPROVED BY: STE
DATE: 23.2.2010

TITLE: **Location of Tests**
Proposed Wallace Wurth Redevelopment
Cnr High St & Botany St, KENSINGTON (UNSW)

PROJECT No: 71543
DRAWING No: 1
REVISION: A



Note: Subsurface conditions are accurate at borehole locations only and variations between borehole locations may be expected.

SECTION A-A'

LEGEND

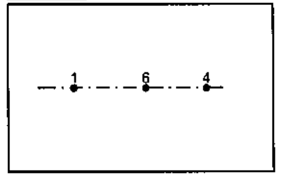
- ■ Asphaltic Concrete
- ☒ CORE LOSS
- ▩ Filling
- ▦ Roadbase
- ▒ Sand
- ▓ Sandstone coarse grained
- ░ Sandstone fine grained
- ▨ Sandy Clay

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

SOIL CONSISTENCY
l - loose
md - medium dense

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

SITE MAP



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CLIENT: Bovis Lend Lease
DRAWN BY: LD
SCALE: 1: 250 (H) 1: 100 (V)
OFFICE: Sydney
APPROVED BY: STE
DATE: 16.03.2010

TITLE: **INTERPRETED GEOTECHNICAL CROSS-SECTION A-A'**
Wallace Wurth Redevelopment
Cnr High St & Botany St, UNSW, Kensington

PROJECT No: 71543
DRAWING No: 2
REVISION: 0

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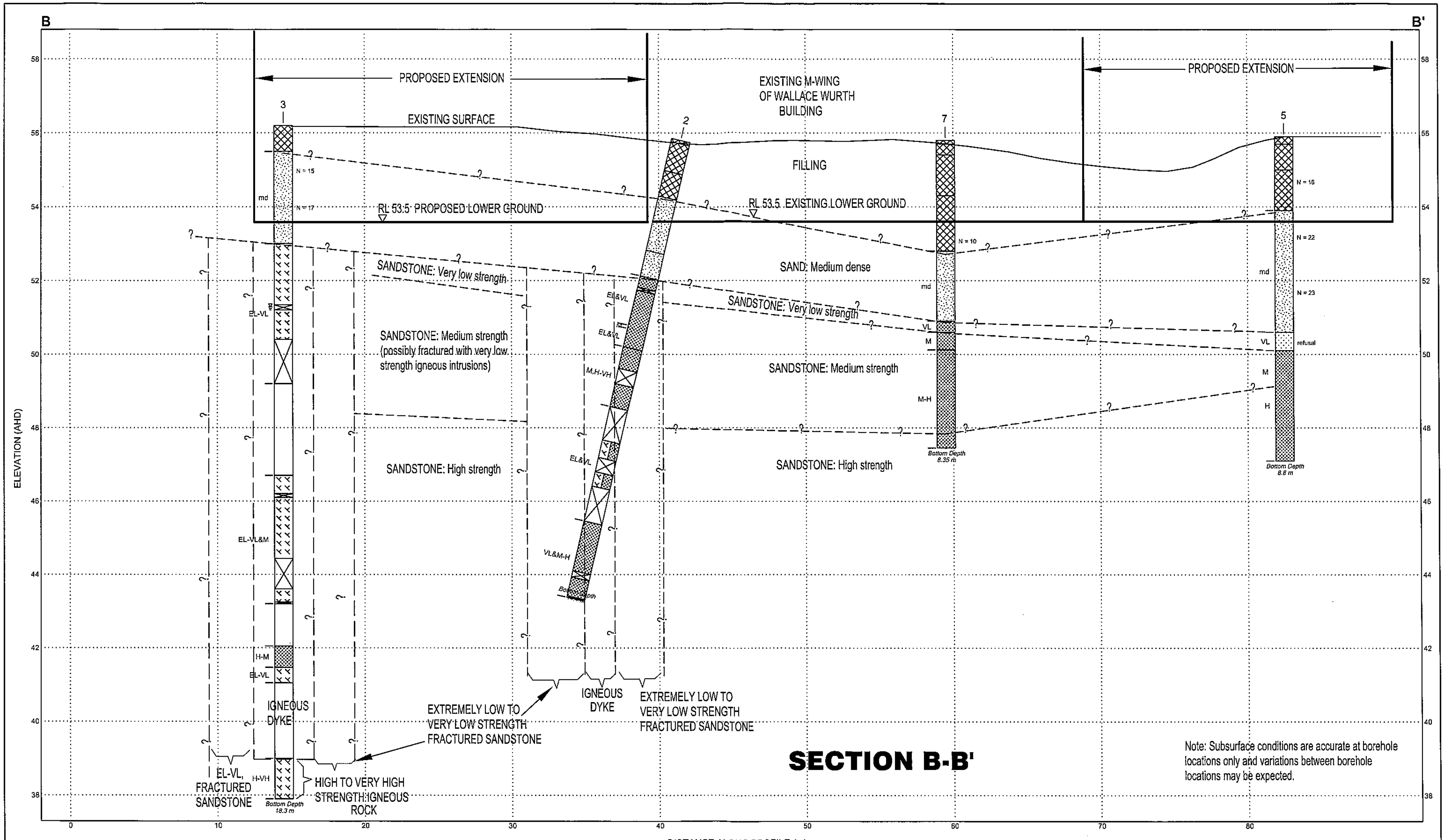


SECTION B-B'

Note: Subsurface conditions are accurate at borehole locations only and variations between borehole locations may be expected.

LEGEND

☒ CORE LOSS
Dyke
☐ Blank Lithology (with border)
Filling
Sand
Sandstone coarse grained
Sandstone fine grained

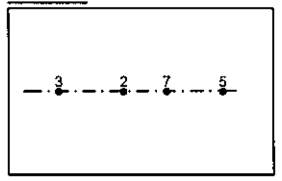
ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

SOIL CONSISTENCY

vs	very soft	vl	very loose
s	soft	l	loose
f	firm	md	medium dense
st	stiff	d	dense
vst	very stiff	vd	very dense
h	hard		

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

SITE MAP



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CLIENT: Bovis Lend Lease
DRAWN BY: LD
SCALE: 1: 250 (H) 1: 100 (V)
OFFICE: Sydney
APPROVED BY: STE
DATE: 16.03.2010

TITLE: **INTERPRETED GEOTECHNICAL CROSS-SECTION B-B'**
Wallace Wurth Redevelopment
Cnr High St & Botany St, UNSW, Kensington

PROJECT No: 71543
DRAWING No: 3
REVISION: 0

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