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REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT
128 HERRING ROAD
MACQUARIE PARK

Prepared for
LIPMAN PROPERTIES PTY LTD

Project 71476.00
December 2009

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REPORT ON GEOTECHNICAL INVESTIGATION
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1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for the proposed residential development at 128 Herring Road, Macquarie Park. The investigation was commissioned by Lipman Properties Pty Ltd.

Based on preliminary information provided by Lipman Properties, it is understood that the proposed development will include the staged construction of five residential unit blocks (Buildings A to E). The buildings will comprise twelve levels above ground with two to three levels of basement carparking. The lowest basement levels will generally require excavation to depths of approximately 8 m to 9 m. The project will involve the initial subdivision of the site and construction of Building A, then progressive development of the remaining buildings. For this reason, the present investigation has targeted Building A with a greater density of boreholes. The lowest basement levels will generally require excavation to depths of approximately 8 m to 9 m.

The field work for the investigation included the drilling of eight boreholes and installation of two 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. 71476.01, dated December 2009).

2. SITE DESCRIPTION

The site of the proposed development is situated on the north-eastern side of Morling College and covers a total area of approximately 17,000m^2. Herring Road runs along the south-eastern site boundary. The Stage 1 site (Building A) is located on the eastern corner of the overall development site and covers an area of approximately 5,000 m^2.

The site and surrounding area are located on a gentle north-facing hill which generally falls from Herring Road to an open water-course located close to the north-western boundary. Within the site, ground surface generally falls from approximately RL 68 to RL 56, relative to Australian Height Datum (AHD), at an average slope of approximately 3 to 5 degrees. The lower north-western part of the site is a reasonably level playing field that has been formed by filling approximately 1 m to 2 m thick. Part of the site is located on the northern side of the water-course where groundslopes begin to rise gently to the north.

At the time of the investigation the site was generally occupied by one to two-storey brick buildings with surrounding grassed and garden areas and numerous scattered mature trees. An earth mound approximately 2 m high was located along the Herring Road boundary.

The property to the south-west of the site is occupied by Morling College and includes a number of one to two-storey brick buildings which are set back approximately 10 m to 20 m from the common boundary.

The property to the north-east of the site is generally undeveloped and covered with grass, with the exception of a brick building (Dunmore Lang College) which is located towards the Herring Road boundary and set back approximately 15 m to 20 m from the common boundary

On the adjacent property to the north-west of the site there was a multi-level commercial building under construction.

3. GEOLOGY

Reference to the Sydney 1:100 000 Series Geological Sheet indicates the site is underlain by Ashfield Shale and that the site is close to boundaries with Hawkesbury Sandstone to the north and east of site. Ashfield Shale typically comprises black to dark grey shale and laminite (interbedded shale, siltstone and fine grained sandstone) and typically weathers to form clays of medium to high plasticity. Hawkesbury Sandstone typically comprises medium to coarse grained quartz sandstone with some shale bands or lenses. The geological mapping was confirmed by the field work which identified residual soils then laminite overlying sandstone bedrock. The laminite may be part of the Mittagong Formation which is a transitional rock unit between the Ashfield Shale and Hawkesbury Sandstone.

4. FIELD WORK METHODS

The field work included eight boreholes (BH1 to BH8 inclusive) drilled to depths of 11.95 m to 12.2 m using a truck-mounted drilling rig and the installation of two groundwater monitoring wells.

The boreholes were initially drilled using spiral augers and rotary washboring within the soil and extremely weathered rock to depths of 1.1 m to 4.7 m. The boreholes were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the bedrock.

Standard Penetration Tests (SPT's) were carried out at regular intervals below depths of 1.0 m to sample the soil and extremely weathered rock and to assess the in-situ strength of the materials. Disturbed soil samples were also retrieved from the boreholes during drilling for identification and classification purposes.

The 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 monitoring wells (50 mm diameter slotted PVC) were installed in BH 2 and BH 8 to depths of 12.0 m, to allow for measurement of the groundwater level during the investigation period and sampling of the groundwater for the contamination assessment. No long term monitoring of groundwater levels was carried out.

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

The ground surface level at each of the borehole locations was interpolated from spot heights relative to Australian Height Datum (AHD) shown on the survey plan by Barrie Green and Associates Pty Ltd (Job No 6041, dated August 2009).

5. FIELD WORK RESULTS

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

The boreholes penetrated a subsurface profile typically comprising topsoil and filling to depths of 0.1 m to 1.4 m, then residual clay to depths of 0.5 m to 4.7 m overlying laminite then sandstone bedrock. The various strata are summarised below.

TOPSOIL: silty/clayey topsoil approximately 100 mm thick was encountered at most locations (300 mm thick in BH6).

FILLING: silty clay filling was encountered to a depth of 1.4 m in BH8.

CLAY: stiff to very stiff natural clay was encountered to depths of between 0.5 m to 1.3 m in BH1 to BH6 inclusive and to depths of 2.5 m and 4.7 m in BH7 and BH8, respectively.

SANDSTONE/ LAMINITE interbedded fine grained sandstone and siltstone (laminite) was encountered below the clay and extended to depths of between 5 m to 9 m. The laminite profile was quite variable but generally included extremely low to very low strength rock approximately 2.5 m thick over low to medium strength rock grading to medium to high strength rock at depths of approximately 5 m to 7 m. Some bands of higher and lower strength rock were encountered within the interbedded extremely low to high strength rock profile.

SANDSTONE more uniform sandstone bedrock was encountered at depths between 5 m to 9 m. The sandstone generally included medium to high strength rock approximately 1 m to 3 m thick overlying high strength, slightly fractured to unbroken rock. High to very high strength sandstone was encountered at depths of between 10.3 m to 11.0 m in BH2, 3, 5 and 8. A 0.5 m thick band of highly fractured, very low to low strength sandstone was encountered in BH7 at a depth of 9.8 m (possibly associated with fault).

The rock, mainly the laminite, included numerous moderately and steeply dipping joints with dips ranging from 30 degrees below horizontal to sub-vertical. Zones of crushed rock (possible shear zones) were identified in the rock cores at some locations.

No free groundwater was observed during augering of the boreholes (i.e. within depths of 1.1 m to 4.7 m) and the use of water during wash boring and coring within the bedrock prevented the measurement of groundwater below this depth. The water level within the groundwater monitoring wells was measured at 7.2 m (RL 57.7 AHD) in BH2 on 7/12/09 and at 5.0 m depth (RL 59.9 AHD) in BH2 on the 17/12/09.

6. POINT LOAD STRENGTH TESTS

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values. 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 rock cores ranged from 0.2 MPa to 3.7 MPa, corresponding to a low to very high strength classification (estimated UCS ranging from 4 MPa to 74 MPa). Generally the Is_{50} values within the upper interbedded rock profile ranged from 0.2 MPa to 1.0 MPa and within the underlying sandstone the Is_{50} values generally ranged from 1.0 MPa to 2.5 MPa. Higher Is_{50} values of up to 3.7 MPa were measured on very high strength bands of rock/ironstone within the upper interbedded profile and within the underlying very high strength sandstone.

7. 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 rock together with the extent of the proposed basement excavations. A summary of the depths (and reduced level) to the top of the various rock strata is provided in Table 1. The orientations of the cross-sections are shown on Drawing 1.

The interpreted geotechnical model for the site includes:

- topsoil to depths of approximately 0.1 m over most of the site (slightly shallower and deeper in some locations);

- clayey filling to depths of approximately 1 m to 2 m below the playing field and along the banks of the water-course on the north-western end of the site. Filling is also present in the earth mounds along the Herring Road frontage;
- residual stiff to very stiff clay to depths of approximately 0.5 m to 1.5 m across most of the site, increasing to depths of 2.5 m to 5 m on the filled north-western end of the site;
- below the clay is a variable bedrock profile of interbedded fine grained sandstone and siltstone extending to depths of between 5 m to 9 m. The interbedded rock is probably part of the Mittagong Formation which is a transitional rock unit between the Ashfield Shale and Hawkesbury Sandstone. The thickness of the interbedded rock profile reduces towards the lower north-western end of the site and was not present in BH8. The interbedded rock profile generally includes extremely low to very low strength rock approximately 2.5 m thick grading to medium to high strength rock at depths of approximately 5 m to 7 m. Bands of lower and higher strength rock are generally present throughout the interbedded rock profile. The laminite is typically more fractured and jointed than the underlying sandstone.
- more uniform sandstone bedrock (Hawkesbury Sandstone) is present below the interbedded rock profile at depths of between 5 m to 9 m (reducing in depth towards the north-western end of the site). The sandstone generally includes medium to high strength rock approximately 1 m to 3 m thick overlying high strength, slightly fractured and unbroken rock. High to very high strength sandstone is present at some locations below depths of 10 m to 11 m.
- groundwater seepage should be expected near the interface of the residual clay and rock surface. It is anticipated that groundwater seepage may also occur within fractured zones and joints within the underlying rock profile. Groundwater seepage flows are likely to increase following periods of extended wet weather.

Table 1 – Summary of Depths (and Reduced Level) to Top of Various Rock Strata

Borehole Location	Surface RL (AHD)	Depth & Reduced Level to Top of Various Rock				
		EL-VL Sandstone/ Laminite	**L-M Sandstone/ Laminite**	**M-H Sandstone/ Laminite**	**M-H Sandstone**	**H Sandstone**
1	65.6	1.0 (64.6)	3.0 (62.6)	5.0 (61.6)		8.0 (57.6)
2	64.9	0.7 (64.2)	3.3 (61.6)	7.0 (57.9)		8.9 (56.0)
3	64.5	0.5* (64.0)	4.6 (60.1)	7.1 (57.4)		9.0 (55.5)
4	66.5	1.3* (65.2)	4.7 (61.8)	6.5 (60.0)		7.5 (59.0)
5	66.4	1.0* (65.4)			6.1 (60.3)	8.1 (58.3)
6	62.5	0.8 (61.7)		4.0 (58.5)	5.9 (56.6)	8.1 (54.4)
7	59.6	2.5 (57.1)			5.0 (54.6)	8.3** (51.3)
8	58.9	4.7 54.2)			4.9 (54.0)	8.7 (50.2)

Notes: Bracketed numbers are the Reduced Level (to AHD) for the top of the stratum
EL = extremely low strength rock M = medium strength rock
VL = very low strength rock H = high strength rock
L = low strength rock
* interbedded extremely low to high strength rock below 1.2m in BH3, 3.2m in BH4, 3.4m in BH5
** fractured, very low strength rock in BH7 from 9.8m to 10.3m.

8. COMMENTS

8.1 Proposed Development

Based on concept architectural drawings by Turner + Associates Pty Ltd (dated 2 December 2009) it is understood that the proposed development will include the staged construction of five residential unit blocks (Buildings A to E). The buildings will comprise twelve levels above ground and two to three levels of linked basement carparking. The project will involve the initial subdivision and construction of Building A then progressive development of the remaining buildings. The lowest basement levels for Building A to D range from RL56.6 to RL51.3 and will generally require excavation to depths of approximately 8 m to 9 m. Slightly shallower excavation to approximately 6 m to 7 m depth will be required for Building E, with a basement level of RL 59.4.

The investigation has been carried out with a higher density of boreholes within or close to the footprint of Building A, as this will be the first building to be constructed. Additional boreholes should be carried out within the remaining building footprints to provide a similar coverage and assess the uniformity or variability in subsurface conditions across the site.

8.2 Site Preparation and Earthworks

8.2.1 Excavation Conditions

The investigation indicates that excavation for the basements will require the removal of soil and extremely low to low strength rock to depths of approximately 3 m to 5 m followed by low to medium then medium and high strength rock in the lower half of the basement. It is important to note that the upper layers of interbedded rock contains bands of medium to very high strength rock and ironstone. Slightly fractured to unbroken, medium and high strength sandstone is expected towards the base of the excavation at most locations, particularly on the north-western part of the site where the thickness of the interbedded rock profile is less.

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 rock may be low (even with large dozers) and therefore some pre-splitting or rock hammering may be necessary to improve efficiency. The underlying slightly fractured to unbroken sandstone may be effectively unrippable in which case large hydraulic rock breakers in conjunction with heavy ripping will be required to remove this material. Rock saws may also be used around the perimeter of the excavation to reduce vibrations and reduce over-break of the rock.

The excavation rate that can be achieved within the medium to high strength rock varies considerably and is dependent upon the degree of jointing in the rock, the rock strength, the type of machinery being used and the skill of the operator. Some of these factors vary between individual contractors and it is therefore recommended that bulk excavation tenderers be required to make their own assessment of the equipment required to carry out the work.

Contractors may inspect the rock core samples at the DP office in West Ryde prior to submitting final tenders (rock cores are generally kept for 6 months after drilling unless longer holding times are requested).

8.2.2 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.

8.2.3 Groundwater Seepage

Groundwater was not observed during auger drilling of the boreholes to maximum depths of 4.7 m however groundwater was later measured within the groundwater monitoring wells at depths of 5.0 m (BH2) and 7.2 m (BH8). The measured groundwater level is probably associated with a perched groundwater table near the interface of residual clay and bedrock and minor seepage through fractures and joints in the rock.

During construction, it is anticipated that groundwater seepage should be readily controlled by 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 clay/rock interface.

8.2.4 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings, pavements and structures 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.

8.2.5 Vibrations

It is anticipated that the proposed rock excavation will result in vibration of the surrounding ground, however, it is noted that the basement footprints are set back 10 m or more from adjacent buildings. 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.

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.

8.3 Excavation Support

8.3.1 Batter Slopes and Excavation Faces

Due to the set back distances from boundaries and structures it is anticipated that excavations may be battered in soil and the interbedded rock followed by vertical excavation in the uniform medium to high strength sandstone below depths of approximately 6 m to 8 m for Buildings A, B and E and depths of 5 m to 6 m for Buildings C and D. Alternatively shoring may be adopted within the soil and interbedded rock to minimise earthworks and the volume of material to be removed.

The maximum batter slopes shown in Table 2 are recommended for the design of temporary and permanent batters. These batters are subject to assessment of jointing in the rock by a geotechnical engineer. If adverse jointing is present in the rock then flatter batters or stabilisation using rock bolts may be required.

Table 2 – Recommended Maximum Batter Slopes

Material	Temporary Batter Slope (H:V)	Permanent Batter Slope (H:V)
Filling or natural clay soils	1.5:1	2:1
Sandstone/laminite: Extremely low to very low strength	1:1	1:5
Sandstone/laminite : Low to medium and medium to high strength	0.5:1	1:1
Sandstone : Medium to high strength	Vertical*	Vertical*

Notes: *Vertical excavation is subject to jointing and geotechnical inspection

The interbedded sandstone and laminite is expected to deteriorate and break down if left exposed to weather. It is therefore recommended that any soil and sandstone/laminite faces that are exposed over the long term should be covered with mesh reinforced shotcrete pinned to the face with dowels. A minimum shotcrete thickness of 80 mm should be adopted unless stability issues dictate a greater thickness is required. The need for shotcrete of rock faces may be reassessed by a geotechnical engineer at the time of excavation.

Excavations in uniform medium or greater strength sandstone will generally be self-supporting (subject to joint orientation) and may be cut vertically. It is possible that some of the less fractured medium strength or stronger sandstone/laminite may also be self-supporting and therefore able to be cut vertically, however, this will need to be assessed by a geotechnical engineer at the time of excavation. 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 to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcrete of fractured or highly weathered zones or rock bolts/anchors where adverse joints form potentially unstable wedges of rock.

8.3.2 Retaining Walls/Shoring

Vertical excavations within the soils and interbedded sandstone/laminite, if adopted, will require both temporary and permanent lateral support during and after excavation. It is anticipated that a bored soldier pile wall with shotcrete or timber infill panels would be suitable. Temporary or permanent soil nail and shotcrete walls may also be considered for support of the soil and rock where there are no movement sensitive structures within the zone of influence of the excavation. Further advice on the design and construction of soil nails should be sought if this alternative is to be considered.

Typically, soldier piles are spaced at approximately 2 m to 3 m centres however closer spaced piles may be required to limit wall movements or collapse of infill materials where structures or services are located in close proximity to the excavation. Generally shotcrete panels should be constructed in 2 m depth intervals within soil and extremely low to very low strength rock and then 3 m depth intervals within low to medium strength rock or better.

Preferably, shoring piles should be founded at least 1.0 m below the base of the bulk excavation level in order to provide lateral restraint at the base of the excavation and avoid the risk of adversely inclined joints or wedges undermining the base of the piles. It may be possible to terminate the shoring piles within free standing medium to high strength sandstone above the bulk excavation level, however it will be important for a geotechnical engineer to assess the stability of the rock directly beneath each pile. The toe of the piles above bulk excavation will also need to be restrained with rock bolts or anchors.

Shoring piles may be used to carry vertical structural loads and may be designed on the basis of the allowable foundation pressures given in Section 8.5. A reduction in bearing pressure will generally apply for piles founded close to, or on the edge of vertical (or steep) rock excavations.

Suitably sized drilling rigs fitted with rock augers will be required to penetrate medium and high strength rock and coring buckets may be required to penetrate high to very strength rock.

8.3.3 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 at least one or two rows of rock anchors will be required to provide lateral restraint to shoring piles above the top of medium to high strength sandstone.

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 Earth Pressure Coefficients and Bulk Unit Weights

Material	**Earth Pressure Coefficient**		**Bulk Unit Weight (kN/m^3)**
	Active (Ka)	**At Rest (Ko)**	
Filling and Residual clay	0.35	0.5	20
Sandstone/laminite: Extremely low to very low strength	0.3	0.45	22
Sandstone/laminite: Low to medium strength	0.2	0.3	22
Sandstone/laminite: Medium to high strength	0.1	0.2	22

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

Preliminary design for lateral earth pressures for walls with more than one row of anchors may be based on a uniform rectangular earth pressure distribution. The additional lateral pressures due to surcharge loading behind the wall and hydrostatic pressures (if appropriate) must also be considered. Where lateral movement is less critical (as generally expected for this site) a pressure distribution of 4H may be considered (where H is the depth to the top of the medium to high strength sandstone). For situations where movements are critical, a higher uniform pressure of 6H may be adopted. For detailed design of walls greater than 5 m high a computer analysis package such as WALLAP, FLAC, PLAXIS or similar should be used to model the excavation and anchoring sequence, to refine the design and provide estimates of possible lateral movements.

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

Passive resistance for piles founded below the base of the excavation may be estimated from the ultimate passive pressures provided in Table 4. 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.

Table 4 – Allowable Passive Resistance for Piles

Material Description	**Ultimate Passive Resistance (kPa)**
Sandstone/laminite: Extremely low to very low strength	600
Sandstone/laminite: Low to medium strength	900
Sandstone/laminite: Medium to high strength	1800
Sandstone: Medium to high strength	6000

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

8.3.4 Ground Anchors

The design of temporary and permanent ground anchors for the support of excavations and/or shoring systems may be carried out on the basis of the maximum allowable bond stresses given in Table 5.

Table 5 – Bond Stresses for Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)
Sandstone/laminite: Extremely low to very low strength	80
Sandstone/laminite: Low to medium strength	100
Sandstone/laminite: Medium to high strength	300
Sandstone: Medium to high strength	500

The parameters given above assume that anchor holes are clean and adequately flushed. 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. It should be noted that permission will be required from adjacent property owners prior to installing bolts/anchors below their land.

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 require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

8.4 Excavation Induced Ground Movements

For deep rock excavations, as proposed on the site, there is a possibility that there will be some horizontal movement due to stress relief effects. Release of these stresses due to the excavation will generally cause horizontal movements along the rock bedding surfaces and partings. Generally, it is not practicable to provide restraint for the relatively high in-situ horizontal stresses.

Based on monitoring experience for excavations in the Sydney region, excavations of over 70 m in length may give rise to lateral stress relief movements on the adjacent ground surface in the order of 1 mm to 2 mm per metre depth of rock excavation. It is noted that this estimate of ground movement generally relates to Hawkesbury Sandstone and stress relief movements are likely to be less within the more fractured laminite present on the site. Empirical data suggest that most of the movement occurs during or shortly after the bulk excavation phase.

8.5 Foundations

Following bulk excavation it is anticipated that medium to high strength sandstone will be exposed at or close to the lowest basement level. Below Buildings A, B and E it is expected that uniform medium to high strength sandstone may be approximately 1 m below the bulk excavation in some locations (i.e. BH2 and 3 indicate medium to high strength sandstone below about RL 55.5 to RL 56.0).

All structural loads should be uniformly supported on underlying bedrock for which pad footings should generally be appropriate. Deepened pad footings or bored piles may be used to reach the underlying high strength sandstone for higher load carrying capacities.

Depending on the final design and building layout it is possible that some columns/footings may be located close to adjacent excavations. Where pad footings or pile shafts are within a line extending upwards at an angle of 45° from the base of adjacent excavations a reduction of the allowable bearing pressure or shaft adhesion parameters may be appropriate. Generally the design parameters provided in Table 9 should be halved for footings and piles close to adjacent excavations, however this will depend on the jointing in the rock and specific advice should be sought when the column/pile layout is confirmed.

Recommended maximum allowable pressures and modulus values for the various foundation materials are presented in Table 6. These parameters apply to the design of spread foundations, such as pads or strip footings, and rock socketed bored piles.

Table 6 – Recommended Design Parameters and Modulus Values for Foundation Design

Foundation Stratum	Maximum Allowable Pressure End Bearing (kPa)	Maximum Allowable Pressure Shaft Adhesion[1] (kPa)	Elastic Modulus (MPa)
Sandstone/laminite: Extremely low to very low strength	700	70	100
Sandstone/laminite: Low to medium strength	1000	100	200
Sandstone/laminite: Medium to high strength	2000	200	300
Sandstone: Medium to high strength	3500	350	500
Sandstone: High strength or better	6000	600	1000

Notes:

(1) Shaft adhesion applicable for the design of bored piers, uncased over rock socket length, where adequate sidewall cleanliness and roughness achieved.

Foundations proportioned on the basis of the above parameters would be expected to experience total settlements of less than 1% of the footing width (or pile diameter) under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

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

Spoon testing should be undertaken in at least one-third of high level footings which are proportioned on the basis of allowable bearing pressures of greater than 3500 kPa. For spread footings designed using allowable bearing pressures of 6000 kPa, spoon testing should be undertaken in at least half of all footing locations. The purpose of spoon testing is to check that no significant weak seams exist below the base of the footing within a depth equal to 1.5 times the least footing dimension. If the testing identifies the presence of weak seams then the footing will either have to be deepened or widened to reduce the actual bearing pressure.

8.6 Pavements & Floor Slabs

During construction of pavements and access roads outside the basement area it is recommended that 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 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 stiff base and replaced with engineered filling.

Engineered filling should be placed in maximum 200 mm thick loose layers and compacted to a minimum dry density ratio of 98% Standard compaction with moisture contents within 2% of optimum moisture content (OMC). The compaction should be increased to a dry density ratio of 100% Standard compaction within 0.3 m of the subgrade surface. The existing filling, clay 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 (where possible, preference should be given to the use of granular material).

If the exposed pavement subgrade is unsuitable (i.e. heaving) then it will generally be necessary to construct a bridging layer. Such treatment may be required if pavements are to be constructed on deeper filling such as encountered on the lower north-western end of the site. The extent of the bridging layer and most suitable form of construction should be determined on site by a geotechnical engineer. As a guide, a bridging layer could be constructed by excavation to a depth of 0.5 m followed by placement of a geofabric layer then compacted granular material (possibly including medium to high strength ripped sandstone from the site).

Subject to the subgrade preparation outlined above, the design of pavements on engineered filling or clay subgrade may be based on a CBR value of 3%. Design of pavements on weathered rock may be based on a CBR value of 5% for extremely low to very low strength rock and 10% for low strength rock or better. 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.

9. LIMITATIONS

Douglas Partners (DP) has prepared this report for this project at 128 Herring Road, Macquarie Park in accordance with the consultancy agreement between Lipman Properties Pty Ltd and DP dated 10 November 2009. This report is provided for the exclusive use of the Lipman Properties for the specific project and purpose as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

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

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

DOUGLAS PARTNERS PTY LTD

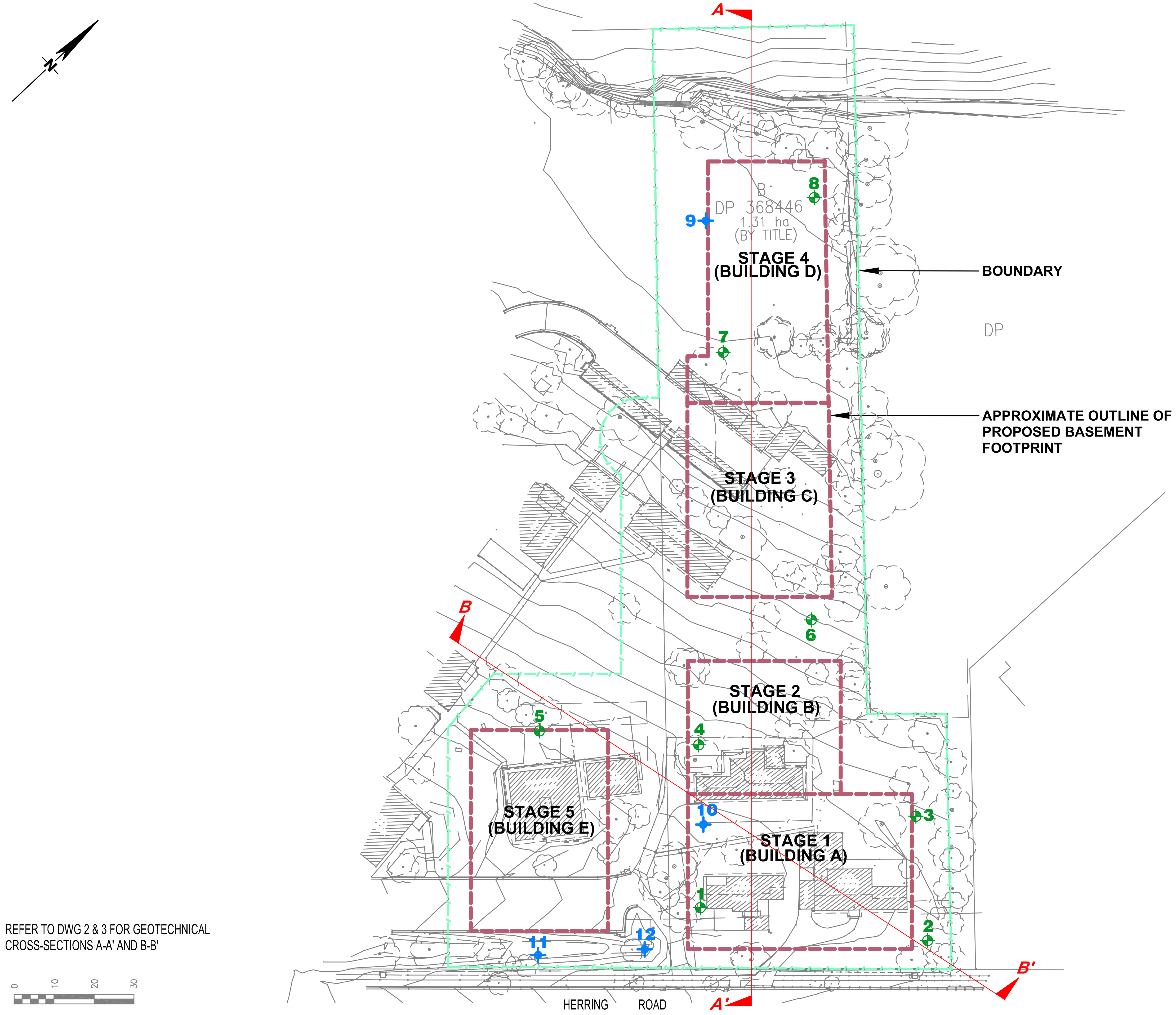
Reviewed by

Scott Easton
Senior Associate

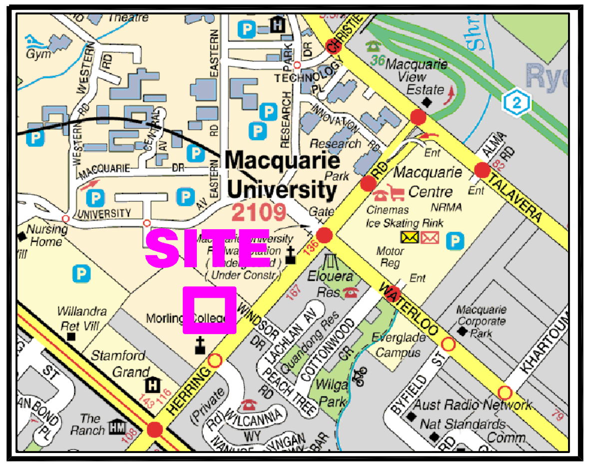
Dr Terry Wiesner
Principal

APPENDIX A

Drawing No. 1 to 3



REFER TO DWG 2 & 3 FOR GEOTECHNICAL
CROSS-SECTIONS A-A' AND B-B'



LOCALITY PLAN

LEGEND

- GEOTECHNICAL BOREHOLE (1-8)
- ENVIRONMENTAL BOREHOLE (9-12)

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: Lipman Properties Pty Ltd

DRAWN BY: PSCH
SCALE: 1:1000 @ A3
OFFICE: Sydney

APPROVED BY:
DATE: 16.12.2009

TITLE: **Location of Tests**
Proposed Residential Development
128 Herring Road, MACQUARIE PARK

PROJECT No: 71476
DRAWING No: 1
REVISION: A

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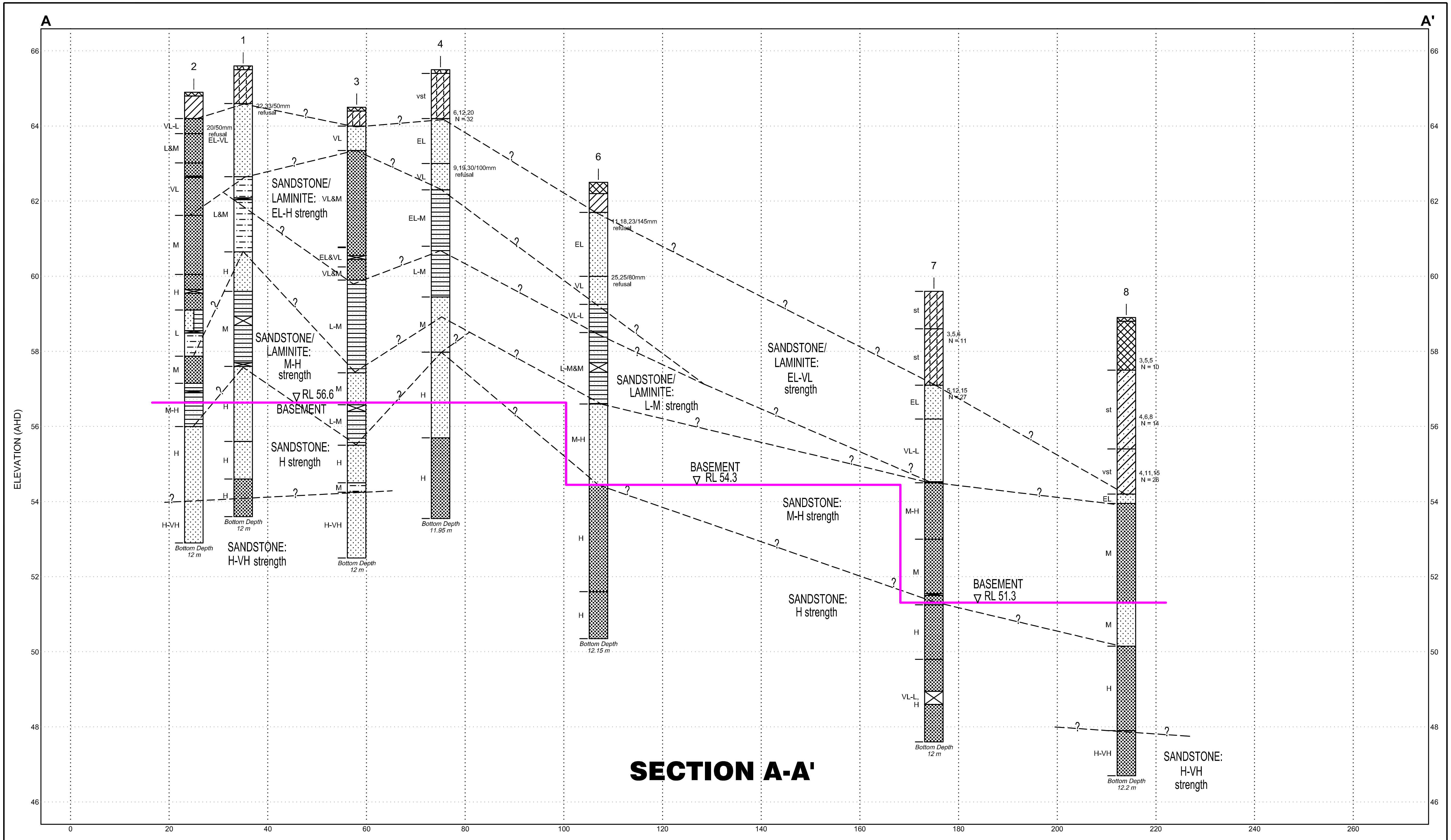

SECTION A-A'

LEGEND

☒ CORE LOSS

▨ Clay

▩ Filling

☰ Laminite

▩ Sandstone coarse grained

░ Sandstone fine grained

▨ Sandy Clay

☰ Siltstone

▨ Silty Clay

▨ Topsoil

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

SOIL CONSISTENCY
st - stiff
vst - very stiff
h - hard

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

Douglas Partners *Geotechnics · Environment · Groundwater*	CLIENT: Lipman Properties Pty Ltd			TITLE: **CROSS SECTION A-A'** **Proposed Residential Development** **128 Herring Road, Macquarie Park**	PROJECT No: 71476
	DRAWN BY: LD	SCALE: As shown	OFFICE:		DRAWING No: 2
	APPROVED BY:		DATE: 16.12.2009		REVISION: 1

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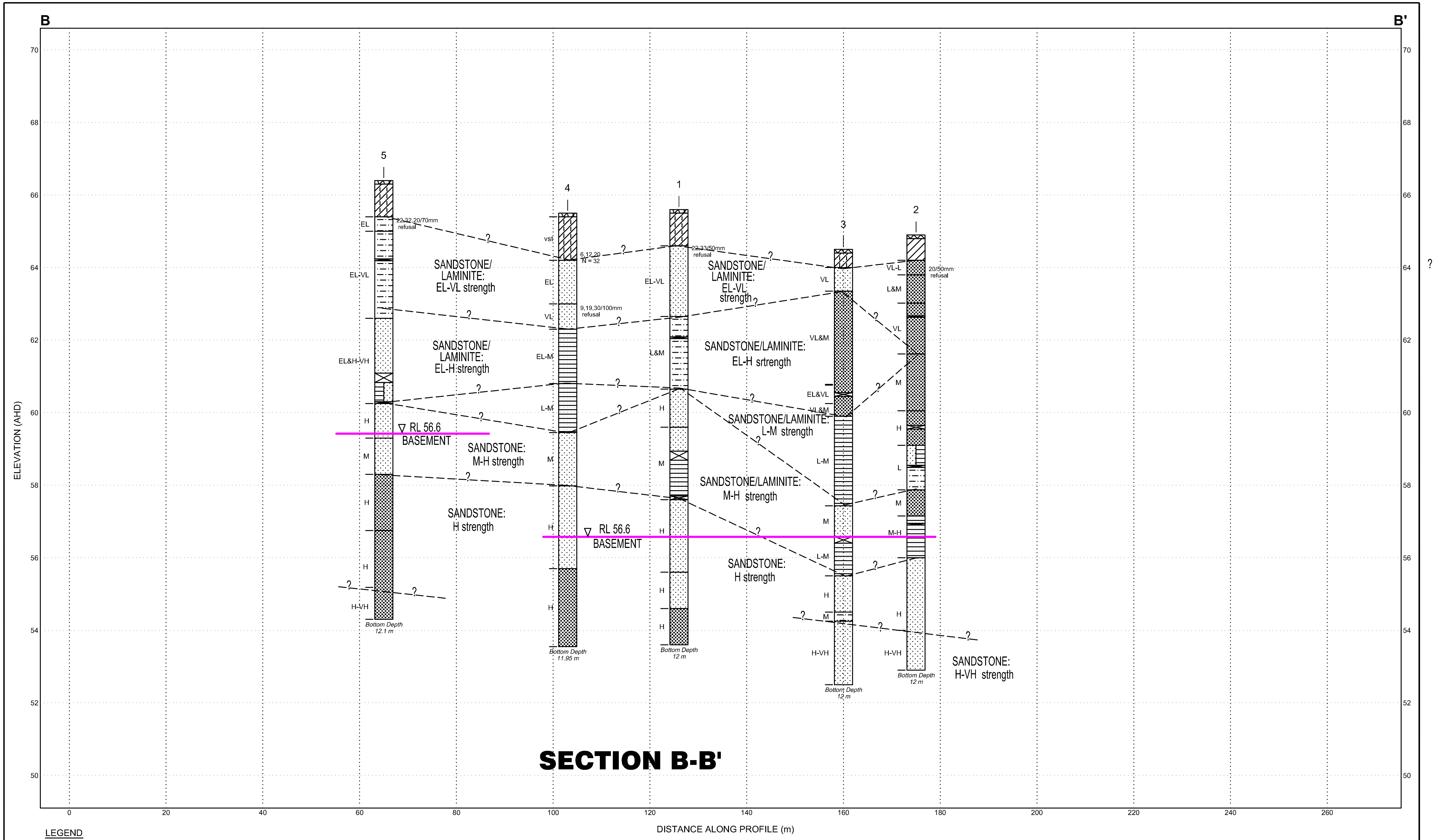

SECTION B-B'

LEGEND

☒ CORE LOSS
▨ Clay
▩ Filling
☰ Laminite
▦ Sandstone coarse grained
░ Sandstone fine grained
▤ Siltstone
▨ Silty Clay
▨ Topsoil

ROCK STRENGTH

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

SOIL CONSISTENCY

st - stiff
vst - very stiff
h - hard

TESTS / OTHER

N - Standard penetration test value
▽ - Water level

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: Lipman Properties Pty Ltd		
DRAWN BY: LD	SCALE: As shown	OFFICE:
APPROVED BY:		DATE: 17.12.2009

TITLE: **CROSS SECTION B-B'**
Proposed Residential Development
128 Herring Road, Macquarie Park

PROJECT No: 71476
DRAWING No: 3
REVISION: 1

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APPENDIX B

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:—

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

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

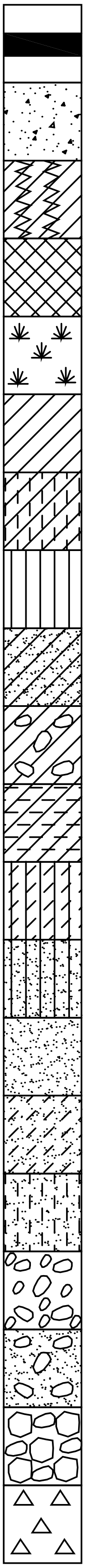
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

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.

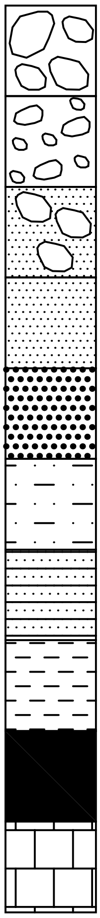
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



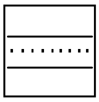
- BITUMINOUS CONCRETE
- CONCRETE
- TOPSOIL
- FILLING
- PEAT
- CLAY
- SILTY CLAY
- SILT
- SANDY CLAY
- GRAVELLY CLAY
- SHALY CLAY
- CLAYEY SILT
- SANDY SILT
- SAND
- CLAYEY SAND
- SILTY SAND
- GRAVEL
- SANDY GRAVEL
- COBBLES/BOULDER
- TALUS

SEDIMENTARY ROCK

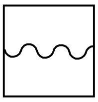


- BOULDER CONGLOMERATE
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE FINE GRAINED
- SANDSTONE COARSE GRAINED
- SILTSTONE
- LAMINITE
- MUDSTONE, CLAYSTONE, SHALE
- COAL
- LIMESTONE

SEAMS

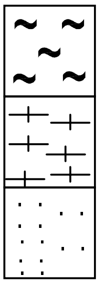


SEAM
>10mm



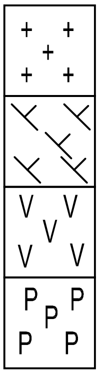
SEAM
<10mm

METAMORPHIC ROCK



- SLATE, PHYLLITE, SCHIST
- GNEISS
- QUARTZITE

IGNEOUS ROCK



- GRANITE
- DOLERITE, BASALT
- TUFF
- PORPHYRY

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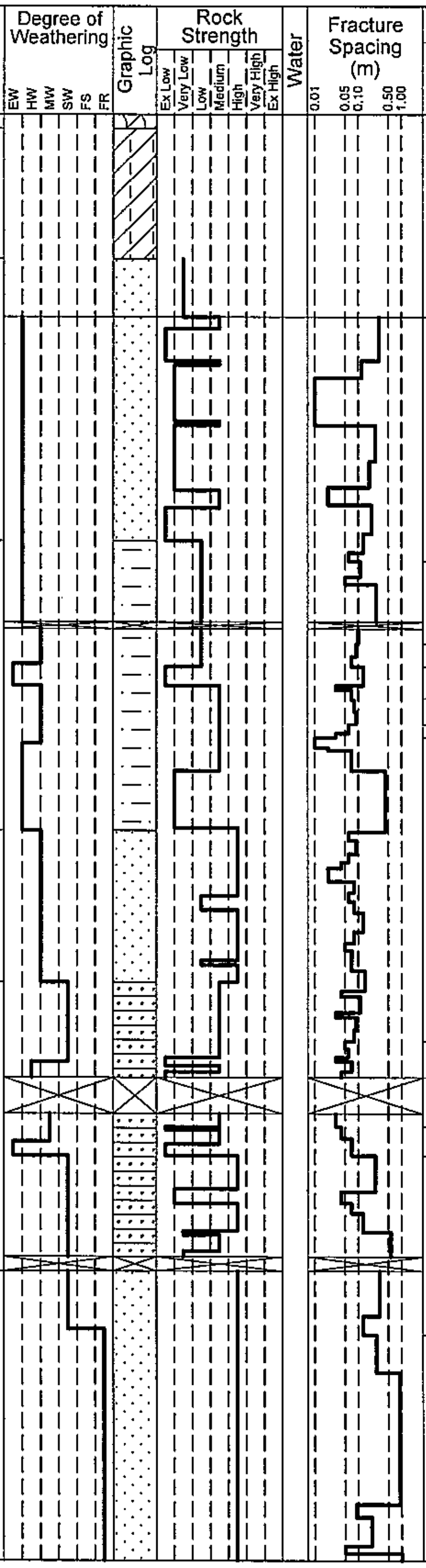
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 65.6 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 71476
DATE: 10 Nov 09
SHEET 1 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
	0.1	TOPSOIL - grey brown, fine grained silty sand topsoil with some gravel, humid		A			
65		SILTY CLAY - red brown, silty clay with a trace of fine grained sand and ironstone gravel, damp	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10° or joints	A			
	1.0	SILTSTONE - extremely low to very low strength, extremely to highly weathered, light grey and red brown, siltstone with some medium strength ironstone bands		A S			22,33/50mm refusal
64 63			1.4-2.9m: highly weathered rock with ironstone bands	C	98	0	
	2.95	SILTSTONE - low and medium strength, highly to moderately weathered, fractured, light grey brown and grey, siltstone with some extremely low and very low strength bands	3.03m: J30°, smooth 3.09-3.25m: (x4) B0°, clay smear				
62 61	3.51		3.51m: CORE LOSS: 50mm 3.66m: J45°, healed, ironstained 3.82m: B0°, clay band 4.04m: J60°, clay smear 4.22m: J45°, smooth, clay smear 4.31-4.54m: B0°- 5° (numerous) ironstained	C	100	7	PL(A) = 0.4MPa
60	4.95	SANDSTONE - high strength, highly to moderately weathered, fractured, grey and brown, fine grained sandstone with approximately 20% siltstone bands and laminations	5.02-5.92m: (x14) B0°- 5°, ironstained				PL(A) = 1.3MPa
59	6.0	LAMINITE - medium strength, moderately and slightly weathered, fractured, interbedded light grey to grey, fine grained sandstone and siltstone. Some extremely low and very low strength bands	6.1m: J70°, ironstained 6.33m: J20°, 45°, steeped 6.41-6.55m: (x4) B0°- 5°, ironstained	C	86	25	PL(A) = 0.4MPa
	6.66		6.66m: CORE LOSS: 250mm 7m: B0°, 10mm clay 7.2m: J45°, ironstained, rough				
58		- some high strength bands from 7.2m	7.6m: J85°, rough	C	91	86	PL(A) = 1MPa
	7.9		7.9m: CORE LOSS: 100mm				
57	8.0	SANDSTONE - high strength, slightly weathered then fresh, slightly fractured, light grey, fine grained sandstone with some siltstone laminations and bands	8.31m: J70°, healed, ironstained 8.44-9.61m: (x3) B0°- 5°, ironstained & clay veneer	C	100	95	PL(A) = 1.1MPa
56			9.9m: B0°, 3mm clay				PL(A) = 1.5MPa

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 1.0m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.4m; NMLC-Coring to 12.0m

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
Uₓ	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: 22/12/09

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BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 65.6 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 71476
DATE: 10 Nov 09
SHEET 2 **OF** 2

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low Very Low Low Medium High Very High Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
55	10.0	SANDSTONE - high strength, fresh, slightly fractured, light grey, fine grained sandstone	FR		High				C	100	95	PL(A) = 2.1MPa
54	11 11.0	SANDSTONE - high strength, fresh, slightly fractured, light grey, medium grained sandstone	FR		High			10.96-11.4m: (x2) B0°, clay veneer	C	100	100	PL(A) = 1.5MPa
53	12 12.0	Bore discontinued at 12.0m										
52	13											
51	14											
50	15											
49	16											
48	17											
47	18											
46	19											

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 1.0m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.4m; NMLC-Coring to 12.0m

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: 22/12/09

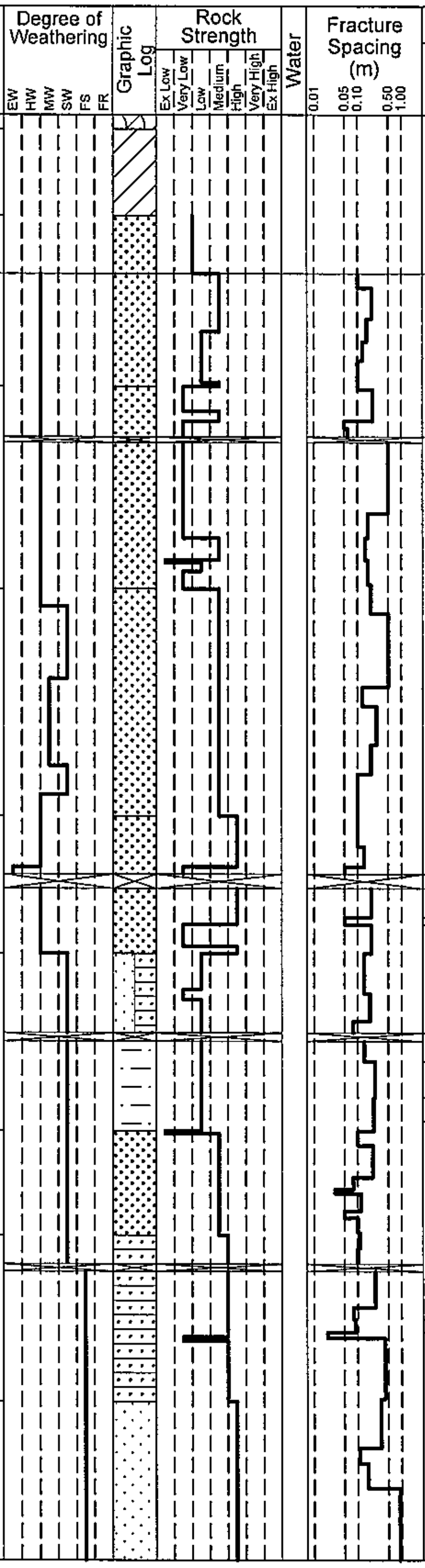
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 64.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 71476
DATE: 13 Nov 09
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	TOPSOIL - light grey brown, fine grained, silty sand topsoil, humid	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	A			
	CLAY - stiff, orange clay with ironstone gravel		A			
0.7	SANDSTONE - very low to low strength, light grey brown, fine to medium grained sandstone		A			20/50mm refusal
1.1	SANDSTONE - low and medium strength, highly to moderately weathered, slightly fractured, light grey brown and red brown, medium to coarse grained sandstone	1.2 & 1.41m: (x2) B20°, clay veneer 1.57-1.8m: (x3) B5°- 10°, ironstained & clay smear	S C	 97	 64	PL(A) = 0.7MPa
1.88 2.23	SANDSTONE - very low strength, highly to moderately weathered, light grey brown and red brown, medium to coarse grained sandstone with some medium strength bands	1.9m: J50°, clayey 2.12 & 2.17m: (x2) B0°, clay smear 2.23m: CORE LOSS: 40mm 2.93m: J, subvertical 3.08 & 3.25m: (x2) B0°-10°, clay smear	C	100	76	PL(A) = 0.2MPa PL(A) = 0.7MPa
3.28	SANDSTONE - medium strength, moderately and slightly weathered, slightly fractured, light grey brown and red brown, medium to coarse grained sandstone	3.45-3.72m: J75°- 80°, curved, rough 3.96 & 4.09m: (x2) B0°, ironstained & clay smear 4.36m: J50°, clay smear 4.56-4.80m: J75°- 90°, curved, 5mm clay infill	C	92	88	PL(A) = 0.4MPa
4.85 5.25	SANDSTONE - high strength, highly to moderately weathered, slightly fractured, red brown, medium grained sandstone with some very low strength bands	4.88-9.50m: (x4) B0°- 5°, clay smear 5.2m: B0°, clay band 5.25m: CORE LOSS: 100mm 5.35m: J30°, clay smear 5.55m: J25°, ironstained	C	94	61	PL(A) = 1.2MPa PL(A) = 0.2MPa
5.8 6.35	SANDSTONE/LAMINITE - low strength, slightly weathered slightly fractured, light grey to grey, fine grained sandstone with interbedded siltstone bands and laminations	5.6-6.08m: (x3) B0°- 5°, 10mm clay bands 6.27m: B0°, clay smear 6.35m: CORE LOSS: 60mm 6.55m: J35°, crushed rock 6.8m: J75°, smooth	C	97	89	PL(A) = 0.2MPa
7.03	SANDSTONE - medium strength, slightly weathered, slightly fractured, light grey brown, medium grained sandstone	6.96-7.73m: (x9) B0°- 5°, ironstained				PL(A) = 0.9MPa
7.75 7.95	LAMINITE - high strength, fresh stained, slightly fractured, finely interbedded light grey to grey, fine grained sandstone and dark grey siltstone. 50% siltstone laminae 7.75-7.85m: very low strength band	7.95m: CORE LOSS: 50mm 8.25m: B5°, ironstained 8.33m: J30°, healed, ironstained 8.42m: J20°, smooth	C	100	88	PL(A) = 1.2MPa
8.9	SANDSTONE - high then high to very high strength, fresh stained then fresh, slightly fractured, light grey brown to grey, fine grained sandstone with some siltstone bands and laminations					PL(A) = 2.5MPa

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 1.0m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.1m; NMLC-Coring to 12.0m

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: 22/12/09

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 64.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 71476
DATE: 13 Nov 09
SHEET 2 **OF** 2

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
54	11	SANDSTONE - high then high to very high strength, fresh stained then fresh, slightly fractured, light grey brown to grey, fine grained sandstone with some siltstone bands and laminations *(continued)*						10.4-10.7m: J75°, rough 10.81-10.88m: (x2) B0°-5°, clay bands 11.4-11.48m: (x4) B0°-5°, clayey 11.52m: B10°, clay veneer	C	100	87	PL(A) = 1.7MPa PL(A) = 3MPa
53	12 12.0	Bore discontinued at 12.0m										
52	13											
51	14											
50	15											
49	16											
48	17											
47	18											
46	19											
45												

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 1.0m
TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.1m; NMLC-Coring to 12.0m
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: 22/12/09

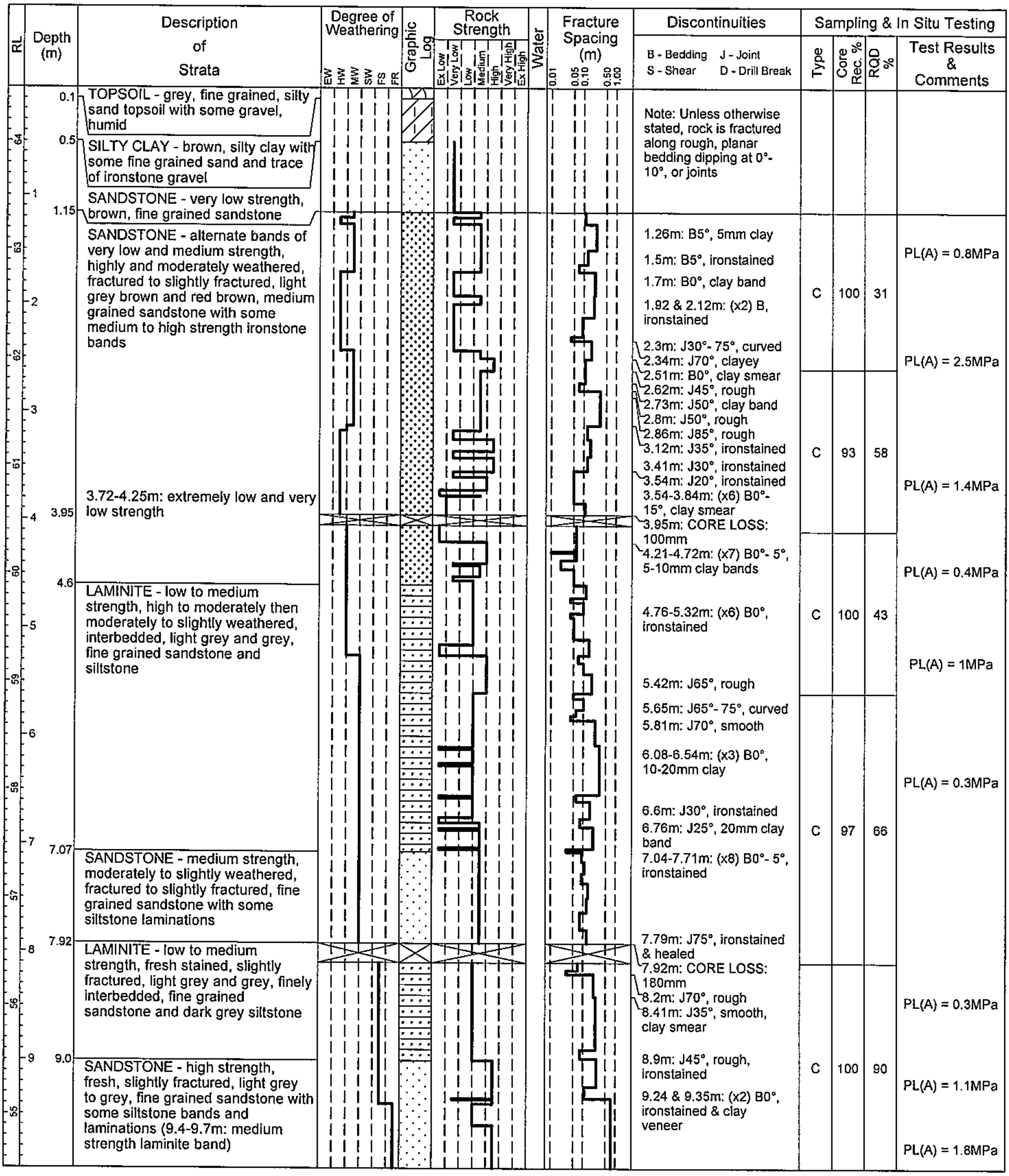
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 64.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 71476
DATE: 16-17/11/09
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL - grey, fine grained, silty sand topsoil with some gravel, humid	Note: Unless otherwise stated, rock is fractured along rough, planar bedding dipping at 0°-10°, or joints				
0.5	SILTY CLAY - brown, silty clay with some fine grained sand and trace of ironstone gravel					
1.15	SANDSTONE - very low strength, brown, fine grained sandstone					
	SANDSTONE - alternate bands of very low and medium strength, highly and moderately weathered, fractured to slightly fractured, light grey brown and red brown, medium grained sandstone with some medium to high strength ironstone bands 3.72-4.25m: extremely low and very low strength	1.26m: B5°, 5mm clay 1.5m: B5°, ironstained 1.7m: B0°, clay band 1.92 & 2.12m: (x2) B, ironstained 2.3m: J30°- 75°, curved 2.34m: J70°, clayey 2.51m: B0°, clay smear 2.62m: J45°, rough 2.73m: J50°, clay band 2.8m: J50°, rough 2.86m: J85°, rough 3.12m: J35°, ironstained 3.41m: J30°, ironstained 3.54m: J20°, ironstained 3.54-3.84m: (x6) B0°-15°, clay smear	C C	100 93	31 58	PL(A) = 0.8MPa PL(A) = 2.5MPa PL(A) = 1.4MPa
3.95		3.95m: CORE LOSS: 100mm 4.21-4.72m: (x7) B0°- 5°, 5-10mm clay bands				PL(A) = 0.4MPa
4.6	LAMINITE - low to medium strength, high to moderately then moderately to slightly weathered, interbedded, light grey and grey, fine grained sandstone and siltstone	4.76-5.32m: (x6) B0°, ironstained 5.42m: J65°, rough 5.65m: J65°- 75°, curved 5.81m: J70°, smooth 6.08-6.54m: (x3) B0°, 10-20mm clay 6.6m: J30°, ironstained 6.76m: J25°, 20mm clay band	C C	100 97	43 66	PL(A) = 1MPa PL(A) = 0.3MPa
7.07	SANDSTONE - medium strength, moderately to slightly weathered, fractured to slightly fractured, fine grained sandstone with some siltstone laminations	7.04-7.71m: (x8) B0°- 5°, ironstained 7.79m: J75°, ironstained & healed				
7.92	LAMINITE - low to medium strength, fresh stained, slightly fractured, light grey and grey, finely interbedded, fine grained sandstone and dark grey siltstone	7.92m: CORE LOSS: 180mm 8.2m: J70°, rough 8.41m: J35°, smooth, clay smear				PL(A) = 0.3MPa
9.0	SANDSTONE - high strength, fresh, slightly fractured, light grey to grey, fine grained sandstone with some siltstone bands and laminations (9.4-9.7m: medium strength laminite band)	8.9m: J45°, rough, ironstained 9.24 & 9.35m: (x2) B0°, ironstained & clay veneer	C	100	90	PL(A) = 1.1MPa PL(A) = 1.8MPa

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 1.0m
TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.15m; NMLC-Coring to 12.0m
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: 22/12/09

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 64.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 71476
DATE: 16-17/11/09
SHEET 2 **OF** 2

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
	10.0	SILTSTONE - medium strength, fresh, slightly fractured, dark grey siltstone						10.08m: B0°, 10mm clay				PL(A) = 0.6MPa
	10.25	SANDSTONE - high to very high strength, fresh, slightly fractured then unbroken, light grey, fine grained sandstone							C	100	100	PL(A) = 3.7MPa PL(A) = 3MPa
	12.0	Bore discontinued at 12.0m										

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 1.0m
TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.15m; NMLC-Coring to 12.0m
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: 22/12/09

Douglas Partners
Geotechnics · Environment · Groundwater

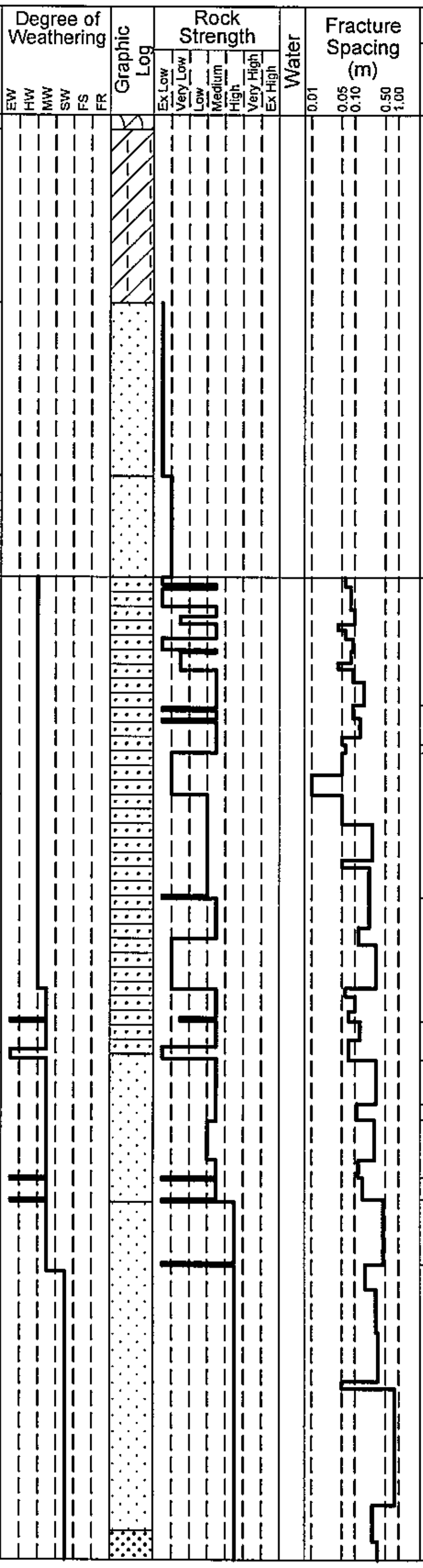
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 65.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 71476
DATE: 10-11/11/09
SHEET 1 OF 2

Depth (m)	Description of Strata
0.1	TOPSOIL - grey brown, fine grained, silty sand topsoil with some clay and rootlets, moist
	SILTY CLAY - very stiff, red brown, silty clay with a trace of fine grained sand and ironstone gravel, damp
1.3	SANDSTONE - extremely low strength, light grey and brown, fine grained sandstone with clay and ironstone bands (soil properties)
2.5	SANDSTONE - extremely low to very low strength, light grey and brown, fine grained sandstone with clay bands
3.2	LAMINITE - alternate bands of extremely low to medium strength, highly to moderately weathered, fractured, light grey to grey and red brown, interbedded fine grained sandstone and siltstone
4.7	LAMINITE - low to medium strength, highly to moderately weathered, fractured to slightly fractured, grey and brown, interbedded fine grained sandstone and siltstone
6.5	SANDSTONE - medium strength, moderately weathered, fractured and slightly fractured, light grey brown to red brown, fine grained sandstone with siltstone laminations and bands
7.52	SANDSTONE - high strength, moderately then slightly weathered, slightly fractured, light grey and brown, fine grained sandstone with some siltstone laminations
9.8	SANDSTONE - see next page



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

3.26-3.79m: (x8) B5°-15°, ironstained & clay bands

3.83m: J80°, clay smear, smooth
4.08-4.17m: (x2) B0°- 5°, 10mm clay bands
4.35m: J40°, clay smear
4.41-4.70m: fragmented into 0.01mm intervals

4.9m: J45°, smooth

5.16m: J40°- 85°, curved, rough
5.41m: B10°, 10mm clay

6.04-6.20m: (x3) B0°-15°, ironstained
6.27m: B0°, 10mm clay
6.47m: J30°, 50mm clay band
6.54m: J20°- 35°, curved, healed
6.84m: J65°- 90°, curved, ironstained, partially healed
6.95-7.23m: (x2) B, ironstained
7.35m: B0°, 10mm clay
7.5m: B0°, 15mm clay
7.95m: B0°, 5mm clay
8.12-9.87m: (x6) B0°- 5°, ironstained

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			6,12,20 N = 32
S			9,19,30/100mm refusal
C	100	30	PL(A) = 0.5MPa
			PL(A) = 0.5MPa
C	100	42	
			PL(A) = 0.6MPa
C	100	48	PL(A) = 0.9MPa
			PL(A) = 0.3MPa
			PL(A) = 1.2MPa
C	100	95	
			PL(A) = 1.3MPa
			PL(A) = 1.6MPa
C	100	100	

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.2m; NMLC-Coring to 11.95m
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: 22/12/09

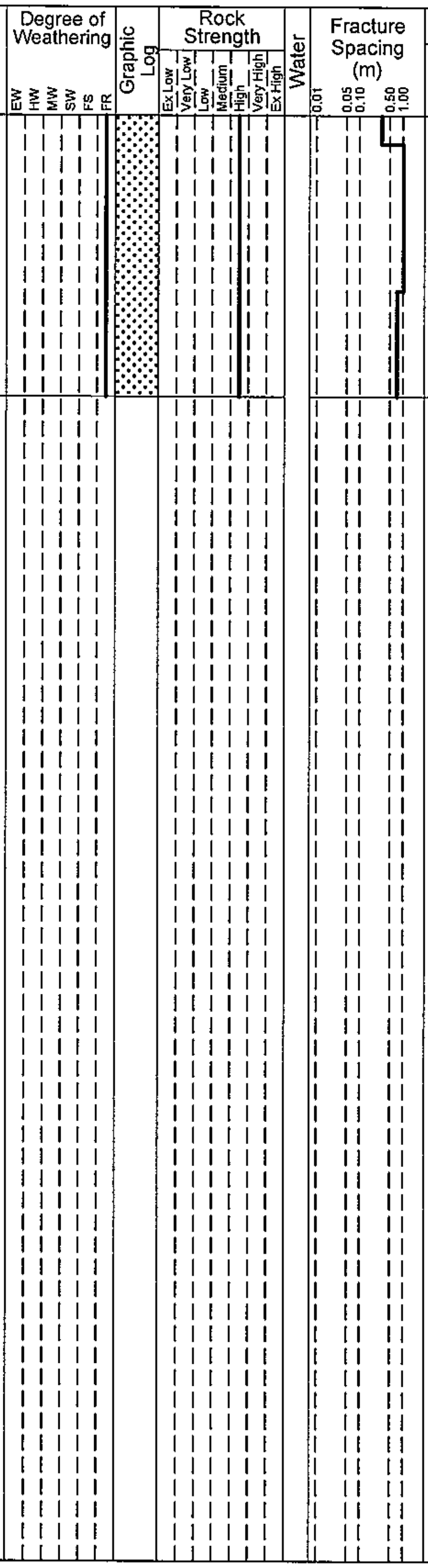
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 65.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 71476
DATE: 10-11/11/09
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
55		SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained sandstone *(continued)*	10.2 & 11.22m: (x2) B0°, clay smear				PL(A) = 1.1MPa
	11			C	100	100	
54							PL(A) = 2MPa
	12 11.95	Bore discontinued at 11.95m					
53							
	13						
52							
	14						
51							
	15						
50							
	16						
49							
	17						
48							
	18						
47							
	19						
46							

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.2m; NMLC-Coring to 11.95m
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: 22/12/09

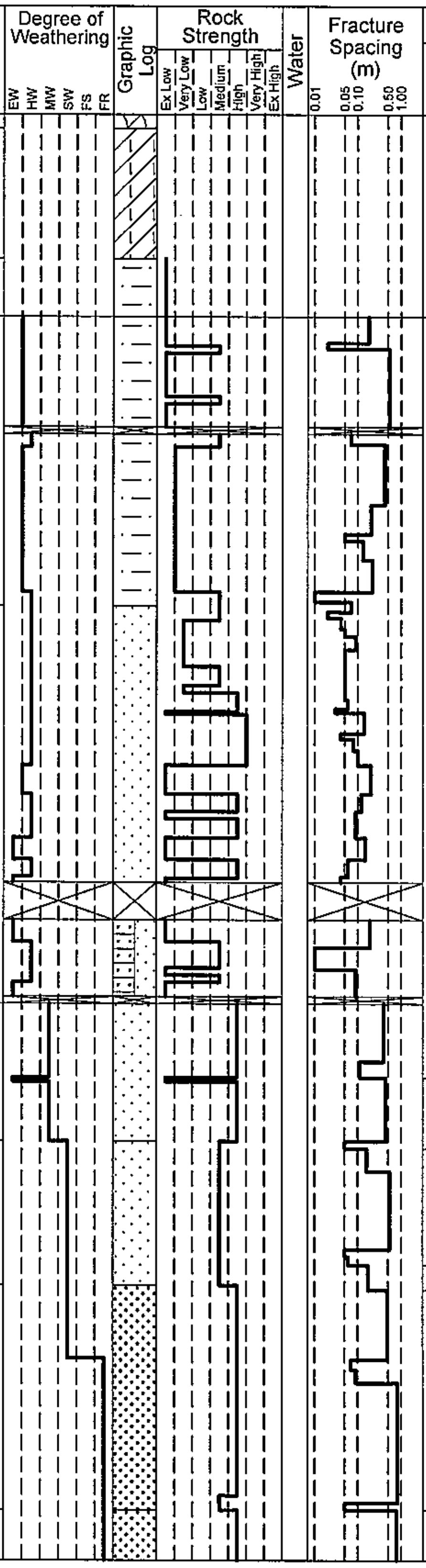
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 66.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 71476
DATE: 11-12/11/09
SHEET 1 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
	0.1	TOPSOIL - light grey brown, fine grained, silty sand topsoil with some gravel, humid		A			
66		SILTY CLAY - red brown, silty clay with a trace of fine grained sand and ironstone gravel, damp	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	A			
	1 1.0	SILTSTONE - extremely low strength, grey brown, fine grained sandstone		A			
				S			22,32,20/70mm refusal
65	1.4	SILTSTONE - extremely low to very low strength, extremely to highly weathered, light grey and red brown, siltstone with some medium strength ironstone bands	1.4-6.15m: highly weathered rock with ironstone bands	C	94	0	
	2 2.16		2.16m: CORE LOSS: 50mm				
64				C	100	0	
	3						
63	3.4	SANDSTONE - alternate bands of extremely low and high to very high strength, extremely and highly weathered, fractured, light grey and red brown, fine grained sandstone with some extremely low strength bands		C	100	0	PL(A) = 0.9MPa
	4						PL(A) = 3.4MPa
62							
	5			C	83	23	PL(A) = 2.8MPa
61	5.31		5.31m: CORE LOSS: 260mm				
	6			C	90	46	
60	6.1 6.15	SANDSTONE - high strength, moderately weathered, slightly fractured, light grey and red brown, fine grained sandstone	6.1m: CORE LOSS: 50mm 6.55m: B0°, clay veneer 6.66m: B0°, 30mm clay bands				PL(A) = 1.6MPa
59	7 7.1	SANDSTONE - medium strength, slightly weathered, slightly fractured, light grey brown and grey, fine grained sandstone with some siltstone laminations and bands	7.1m: J30°, healed, ironstained 7.15-7.31m: (x2) B0°, 2mm clay 7.85 & 7.90m: (x2) B0°-5°, ironstained	C	100	90	PL(A) = 0.8MPa
58	8 8.1	SANDSTONE - high strength, slightly weathered then fresh, slightly fractured, light grey and brown, medium grained sandstone with some siltstone laminations and bands	7.96m: J70°, ironstained 8.13-8.68m: (x3) B0°- 5°, ironstained 8.77m: B0°, 3mm carbonaceous band				PL(A) = 1.6MPa
57	9			C	100	86	PL(A) = 1.7MPa
	9.65	SANDSTONE - see next page	9.6m: J70°, rough 9.65m: B0°, clay smear				

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 1.1m
TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.4m; NMLC-Coring to 12.10m
WATER OBSERVATIONS:
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: 22/12/09

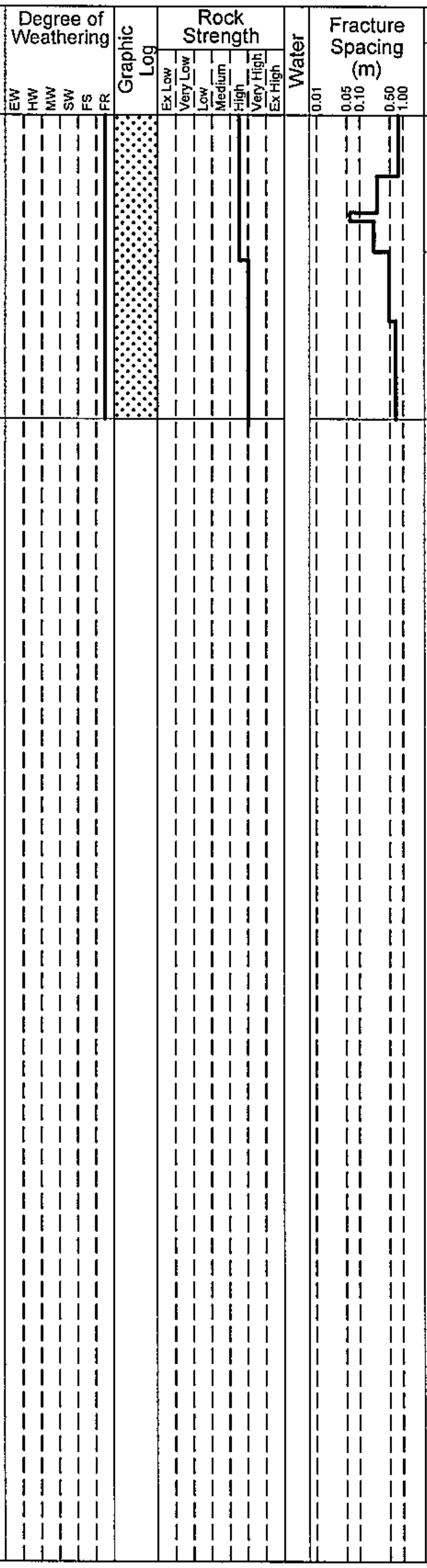
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 66.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 71476
DATE: 11-12/11/09
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
56	11	SANDSTONE - high then very high strength, fresh, slightly fractured, light grey, medium grained sandstone *(continued)*	10.42m: J55°- 75°, curved, rough 10.73m: J75°- 85°, curved, rough 10.94 & 11.42m: (x2) B0°, clay smear	C	100	97	PL(A) = 1.4MPa
55		- siltstone laminations from 11.4m					PL(A) = 3.1MPa
	12 12.1	Bore discontinued at 12.1m					
54	13						
53	14						
52	15						
51	16						
50	17						
49	18						
48	19						
47							

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 1.1m
TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 1.4m; NMLC-Coring to 12.10m
WATER OBSERVATIONS:
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: 22/12/09

Douglas Partners
Geotechnics · Environment · Groundwater

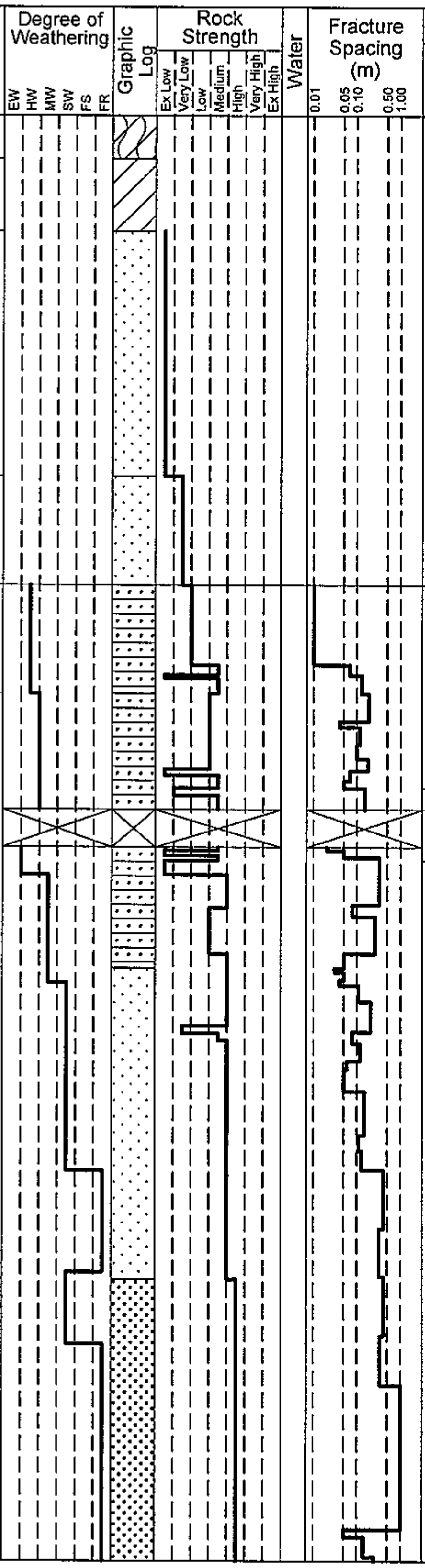
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 62.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 71476
DATE: 12 Nov 09
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	TOPSOIL - light grey brown, silty clay topsoil, humid		A			
0.3	CLAY - red brown, clay with trace silt and ironstone gravel, damp		A A			
0.8	SANDSTONE - extremely low strength, light grey and orange, fine grained sandstone with ironstone bands		S			11,18,23/145mm refusal
2.5	SANDSTONE - very low strength, fine grained, light grey and orange sandstone	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	S			25,25/80mm refusal
3.25	LAMINITE - very low to low strength, highly weathered, light grey, finely interbedded fine grained sandstone and siltstone with some medium to high strength ironstone bands	3.25-3.80m: J, subvertical, rough, irregular & fragmented into 0.01mm intervals 3.87m: B0°, 10mm clay	C	100	70	
4.0	LAMINITE - low to medium and medium strength, highly to moderately weathered, fractured to slightly fractured, light grey brown, finely interbedded, fine grained sandstone and siltstone with some extremely low strength bands	4.23m: J75°, clayey 4.53m: B, 20mm clay 4.65m: B, 10mm clay				PL(A) = 0.3MPa
4.8		4.8m: CORE LOSS: 260mm 5.08m: B0°, 20mm clay 5.15-5.25m: clay band 5.46-8.44m: (x21) B0°-5°, ironstained	C	87	52	PL(A) = 2.5MPa
5.9	SANDSTONE - medium to high strength, slightly weathered to fresh, fractured to slightly fractured, light grey and brown, fine grained sandstone - grey from 7.2m	6.6m: J15°, ironstained & healed				PL(A) = 1MPa PL(A) = 1.1MPa
8.05	SANDSTONE - high strength, slightly weathered and fresh, slightly fractured and unbroken, light grey, medium to coarse grained sandstone	8.78-9.97m: (x5) B0°- 5°, clay veneer/smear	C	100	100	PL(A) = 2.2MPa PL(A) = 1.4MPa
			C	100	94	



RIG: DT 100
DRILLER: Steve Y
LOGGED: SI/RGB
CASING: HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.25m; NMLC-Coring to 12.15m
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: 22/12/09

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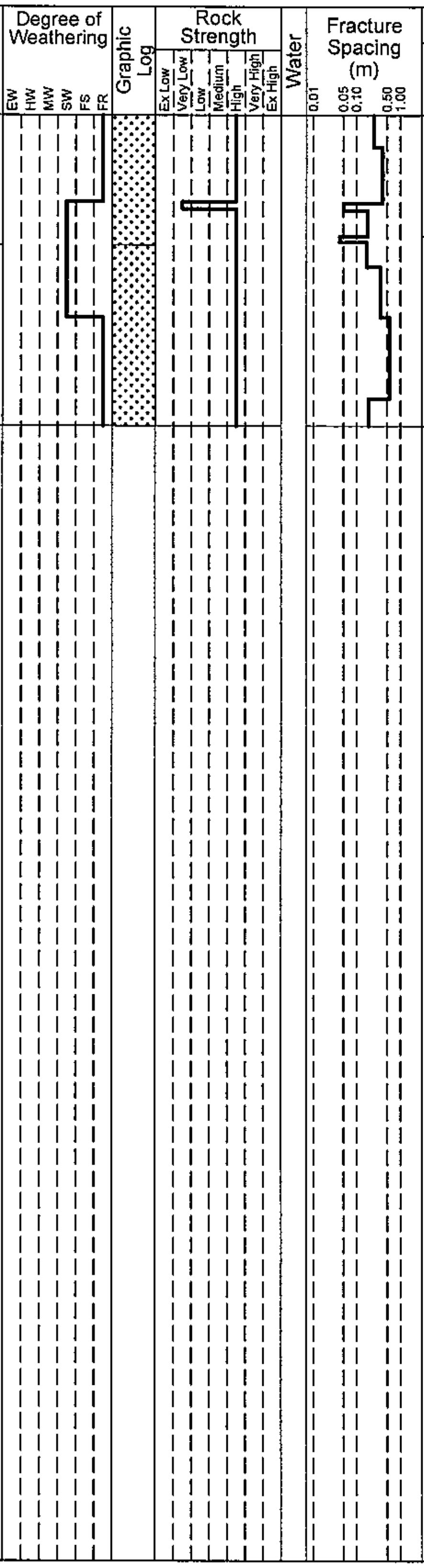
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 62.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 71476
DATE: 12 Nov 09
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
52	10.0	SANDSTONE - high strength, slightly weathered and fresh, slightly fractured, light grey, medium to coarse grained sandstone	10.6m: B5°, 10mm sandy clay	C	100	94	PL(A) = 1.1MPa
51	10.9	SANDSTONE - high strength, slightly weathered then fresh, light grey and brown, medium to coarse grained sandstone	10.84-11.96m: (x5) B0°-5°, ironstained				PL(A) = 2MPa PL(A) = 2MPa
50	12.15	Bore discontinued at 12.15m					

RL scale: 52, 51, 50, 49, 48, 47, 46, 45, 44, 43
Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI/RGB **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.25m; NMLC-Coring to 12.15m
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: 22/12/09

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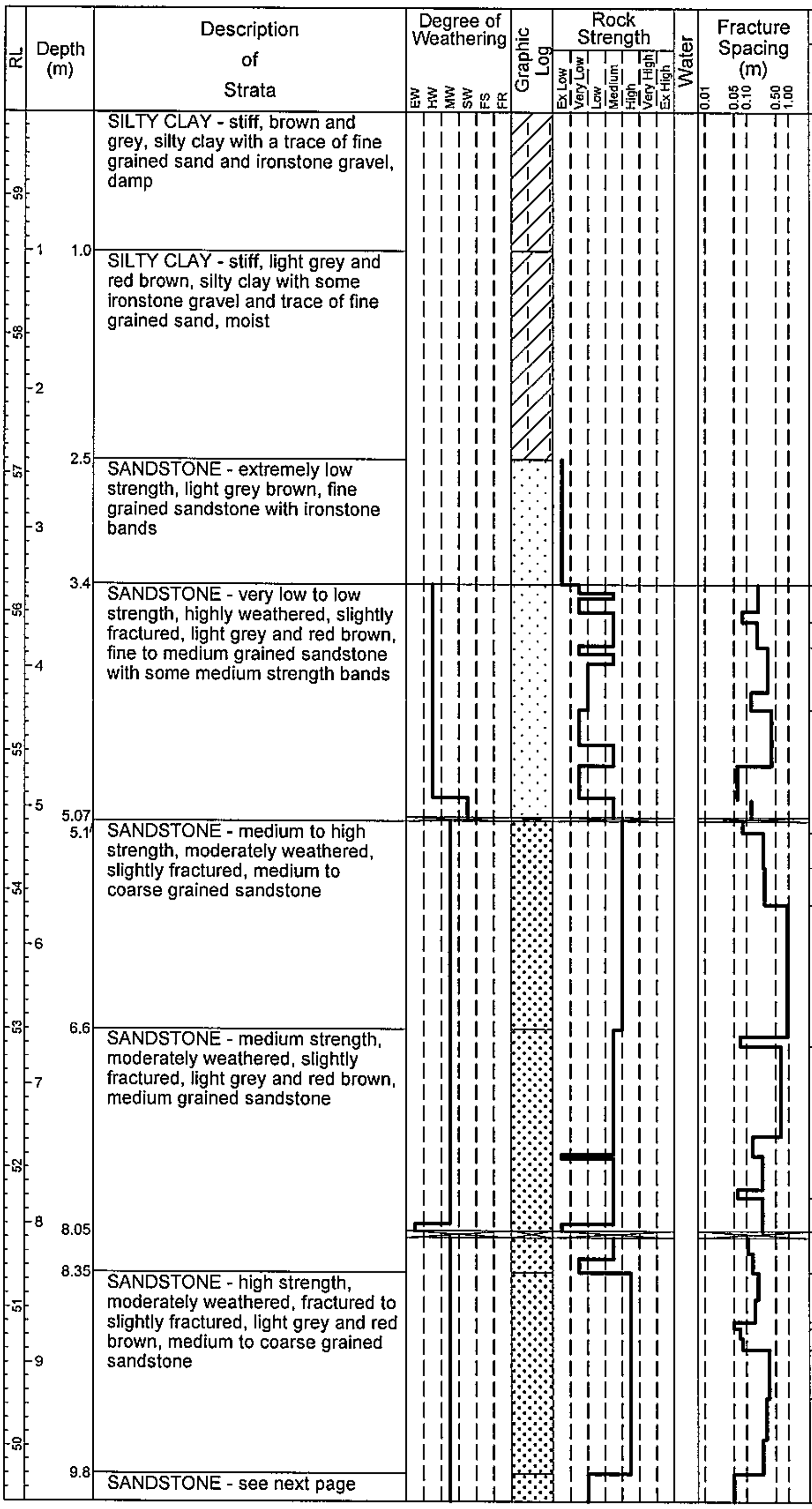
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 59.6 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 71476
DATE: 17 Nov 09
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
	SILTY CLAY - stiff, brown and grey, silty clay with a trace of fine grained sand and ironstone gravel, damp		A/E A/E			
1.0	SILTY CLAY - stiff, light grey and red brown, silty clay with some ironstone gravel and trace of fine grained sand, moist		A/E S			3,5,6 N = 11
2.5	SANDSTONE - extremely low strength, light grey brown, fine grained sandstone with ironstone bands	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	S			5,12,15 N = 27
3.4	SANDSTONE - very low to low strength, highly weathered, slightly fractured, light grey and red brown, fine to medium grained sandstone with some medium strength bands	3.59m: B5°, clay band 3.67m: J30°, ironstained, rough 3.85m: B15°, 2mm clay 4.17m: J65°, clayey 4.3m: J30°, 2mm clay 4.7-4.94m: (x4) B5°- 10°, clayey	C	98	29	PL(A) = 0.7MPa PL(A) = 0.4MPa
5.07 5.1	SANDSTONE - medium to high strength, moderately weathered, slightly fractured, medium to coarse grained sandstone	5.07m: CORE LOSS: 30mm 5.18m: J20°, ironstained 5.43m: (x2) J90°, healed & 30°, rough, some crushed rock 5.7m: B5°, ironstained	C	98	89	PL(A) = 1.3MPa PL(A) = 1MPa
6.6	SANDSTONE - medium strength, moderately weathered, slightly fractured, light grey and red brown, medium grained sandstone	6.65m: B10°, clay veneer 6.72m: J40°, ironstained 7.37m: J55°, rough 7.51m: J30°, 10mm clay 7.75m: J80°, healed 7.81m: J75°- 80°, curved, ironstained				PL(A) = 0.7MPa
8.05 8.35	SANDSTONE - high strength, moderately weathered, fractured to slightly fractured, light grey and red brown, medium to coarse grained sandstone	8.05m: CORE LOSS: 50mm 8.35m: J75°- 85°, curved, rough, ironstained 8.7m: J75°, rough 8.75m: J70°, ironstained 8.9-9.2m: (x2) J85° & 75°, ironstained 9.25m: J80°, rough	C	100	76	PL(A) = 1.7MPa PL(A) = 1.5MPa
9.8	SANDSTONE - see next page	9.8m: (x2) J75°, 85°, healed, ironstained	C	73	34	

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 2.5m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.4m; NMLC-Coring to 12.0m

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: 22/12/09

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BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 59.6 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 7
PROJECT No: 71476
DATE: 17 Nov 09
SHEET 2 **OF** 2

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low Very Low Low Medium High Very High Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sampling & In Situ Testing Type	Core Rec. %	RQD %	Test Results & Comments
49	10.65	SANDSTONE - very low to low then high strength, moderately weathered, highly fractured to fractured, brown, medium to coarse grained sandstone *(continued)*						10.0-11.7m: J, subvertical, fragmented into 0.01-0.05mm interval 10.65m: CORE LOSS: 350mm	C	73	34	
48	11 12 12.0							11.55m: J65°, rough, ironstained 11.85m: J70°, rough, ironstained 11.95m: J85°, rough, ironstained	C	100	0	PL(A) = 1.1MPa
47 46 45 44 43 42 41 40	13 14 15 16 17 18 19	Bore discontinued at 12.0m										

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HQ to 2.5m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.4m; NMLC-Coring to 12.0m

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: 22/12/09

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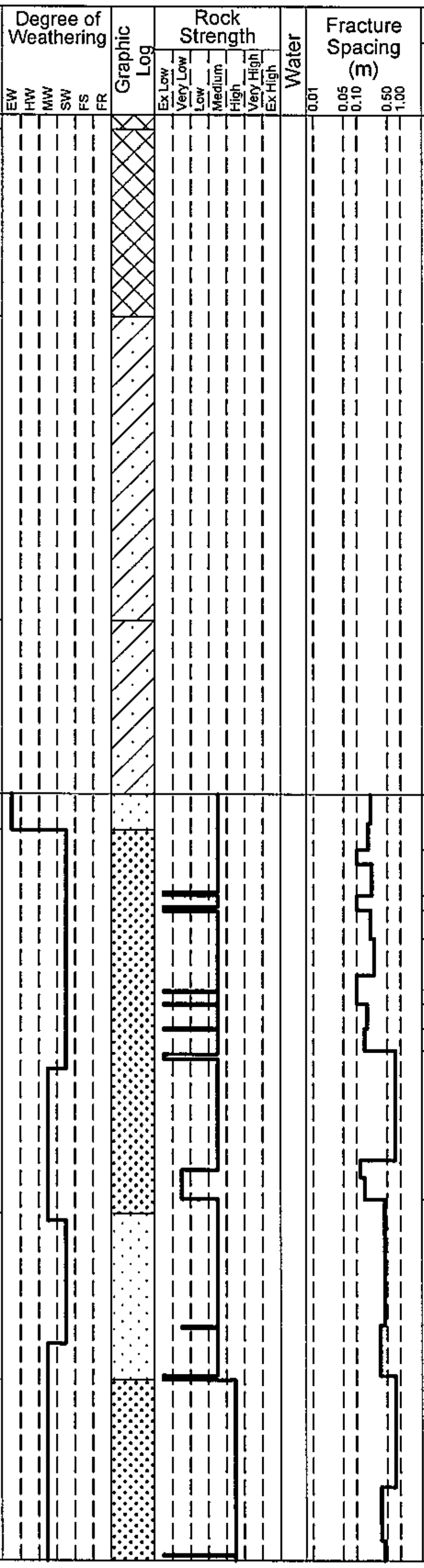
BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 58.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 8
PROJECT No: 71476
DATE: 13 Nov 09
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 - grey, silty clay topsoil with some rootlets, damp		A			
	FILLING - brown, silty clay filling with some fine to medium grained sand and a trace of ironstone gravel, damp		A A S			3,5,5 N = 10
1.4	SANDY CLAY - stiff, light grey and red brown, sandy clay with ironstone gravel		S			4,6,8 N = 14
3.5	SANDY CLAY - very stiff, light grey, red and orange, sandy clay with ironstone bands (extremely weathered sandstone)	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping at 0°-10° or joints	S			4,11,15 N = 26
4.7	SANDSTONE - extremely low strength, light grey and orange sandstone					
4.95	SANDSTONE - medium strength, moderately and slightly weathered, slightly fractured, light grey and orange brown, medium to coarse grained sandstone with some extremely low strength bands	4.9m: J45°- 75°, curved, clay infill 5.08 & 5.18m: (x2) B0°-10°, clay veneer 5.4m: J25°, clay smear 5.5m: B10°, 15mm clay band 5.7 & 5.95m: (x2) B0°-5°, clay veneer 6.05m: B5°, 5mm clay 6.15m: B5°, 5mm clay & J, subvertical rough 6.32m: B0°, 2mm clay 6.47m: J45°, rough 7.35m: J60°, smooth, clay smear 7.5m: B0°, 5mm clay	C C	100 100	80 89	PL(A) = 0.7MPa PL(A) = 0.7MPa
7.6	SANDSTONE - medium strength, moderately and slightly weathered, slightly fractured, light grey and red brown, fine to medium grained sandstone	8.37m: B0°, 10mm clay				PL(A) = 0.9MPa PL(A) = 0.7MPa
8.75	SANDSTONE - high strength, moderately weathered, slightly fractured, light brown, medium to coarse grained sandstone	8.72m: B5°, 20mm clay 9.86m: B0°, 2mm clay	C	100	98	PL(A) = 1.8MPa

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.7m; NMLC-Coring to 12.10m
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: 22/12/09

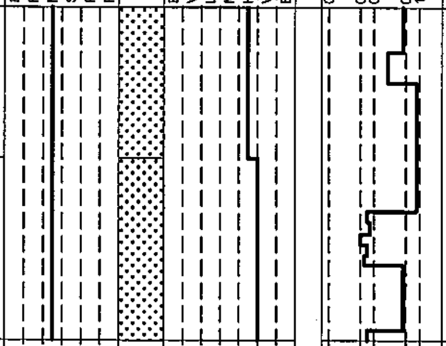
Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Lipman Properties Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 128 Herring Road, Macquarie Park

SURFACE LEVEL: 58.9 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 8
PROJECT No: 71476
DATE: 13 Nov 09
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
		SANDSTONE - high strength, moderately weathered, slightly fractured, light brown, medium to coarse grained sandstone *(continued)*	10.3-11.35m: (x3) B5°-15°, clay veneer	C	100	98	PL(A) = 2.2MPa
48	11 11.0	SANDSTONE - high to very high strength, moderately weathered, fractured to slightly fractured, medium to coarse grained sandstone	11.42m: J25°, rough 11.64m: J35°, ironstained	C	100	55	PL(A) = 3.1MPa
47	12						
	12.2	Bore discontinued at 12.2m	12.13m: J85°, healed				PL(A) = 2.6MPa
46	13						
45	14						
44	15						
43	16						
42	17						
41	18						
40	19						
39							

RIG: DT 100 **DRILLER:** Steve Y **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.7m; NMLC-Coring to 12.10m
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: 22/12/09

Douglas Partners
Geotechnics · Environment · Groundwater

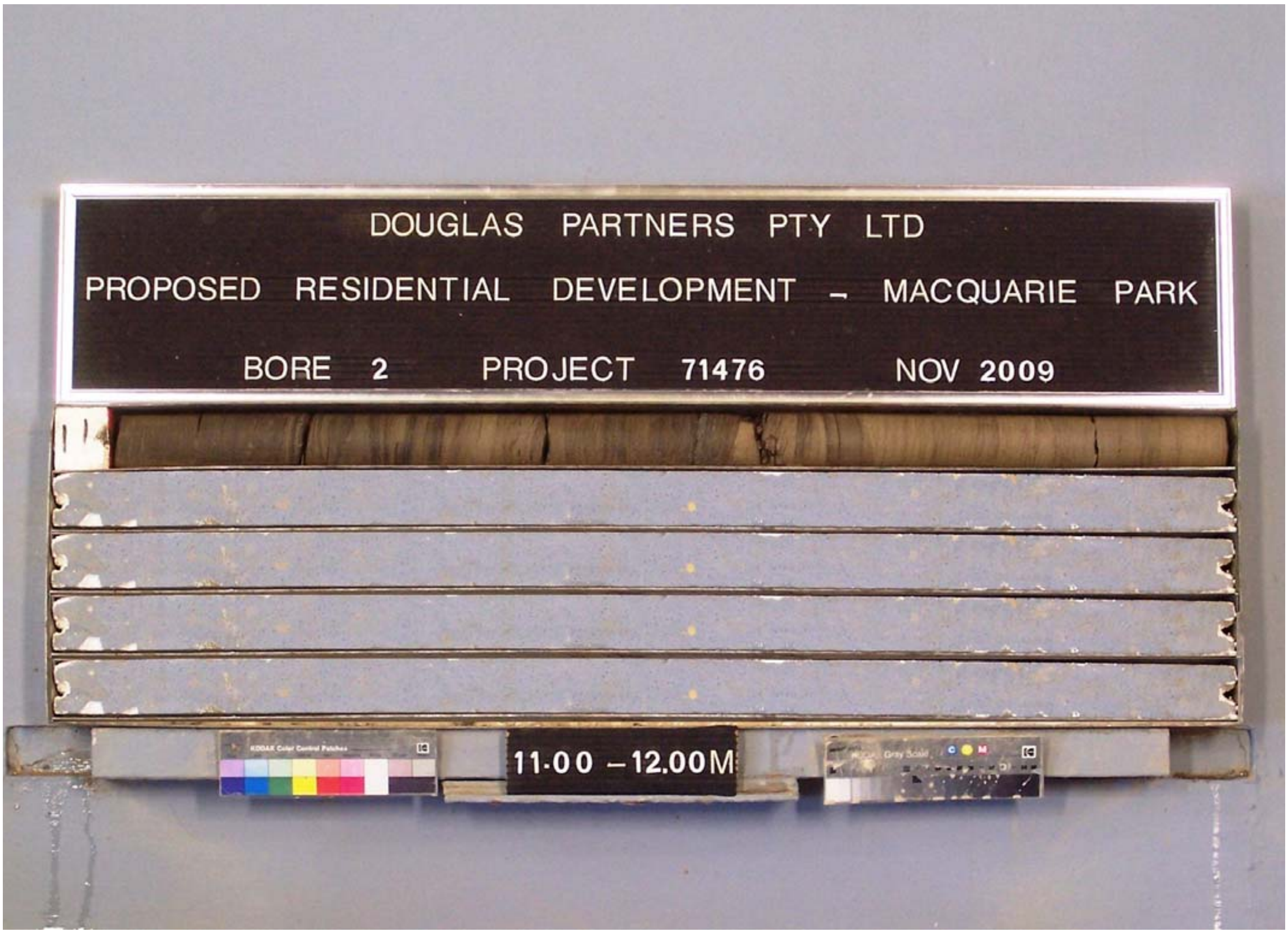

DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 1 PROJECT 71476 NOV 2009
MAQAURIE PARK
71476 10/11/09
BH1 START 1.4 mtrs
2
3
C/L 0.05m
4
5
1.40 – 6.00M
KODAK Color Control Patches
KODAK Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 1 PROJECT 71476 NOV 2009
6
CORE LOSS 0.25M
7
CORE LOSS 0.10M
8
9
10
6.00 –11.00M
KODAK Color Control Patches
KODAK Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 1 PROJECT 71476 NOV 2009
11
KODAK Color Control Patches
11·00 –12.00M
Gray Scale
C
Y
M

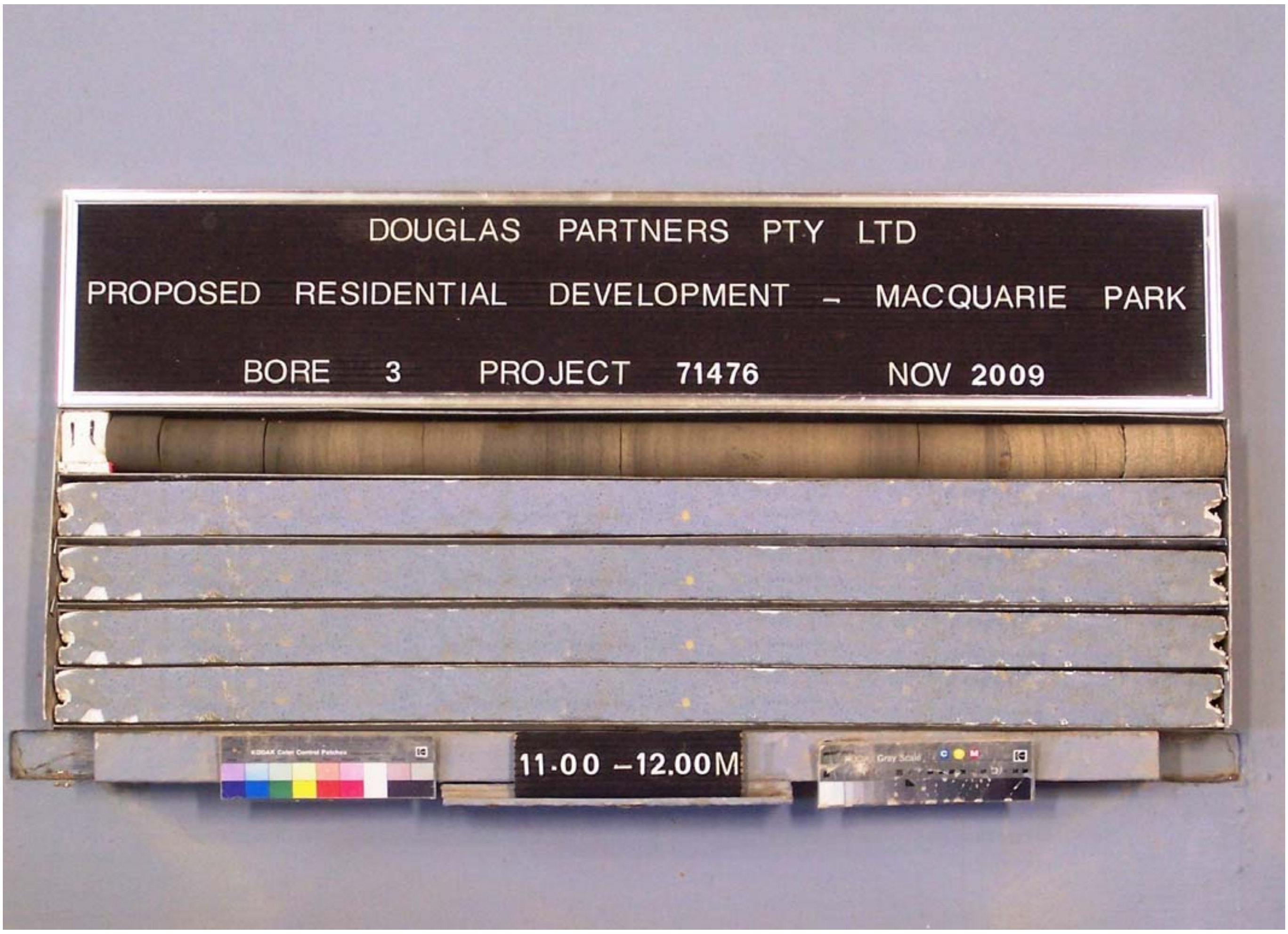

DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 2 PROJECT 71476 NOV 2009
MACQUARIE PARK
71476 13/11/09
BH2 START 1.1m
2
50mm loss
3
4
5
100mm loss
1.10 – 6.00M
KODAK Color Control Patches
Kodak Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 2 PROJECT 71476 NOV 2009
6
7
60mm LOSS
8
9
10
6.00 – 11.00M
KODAK Color Control Patches
Kodak Gray Scale

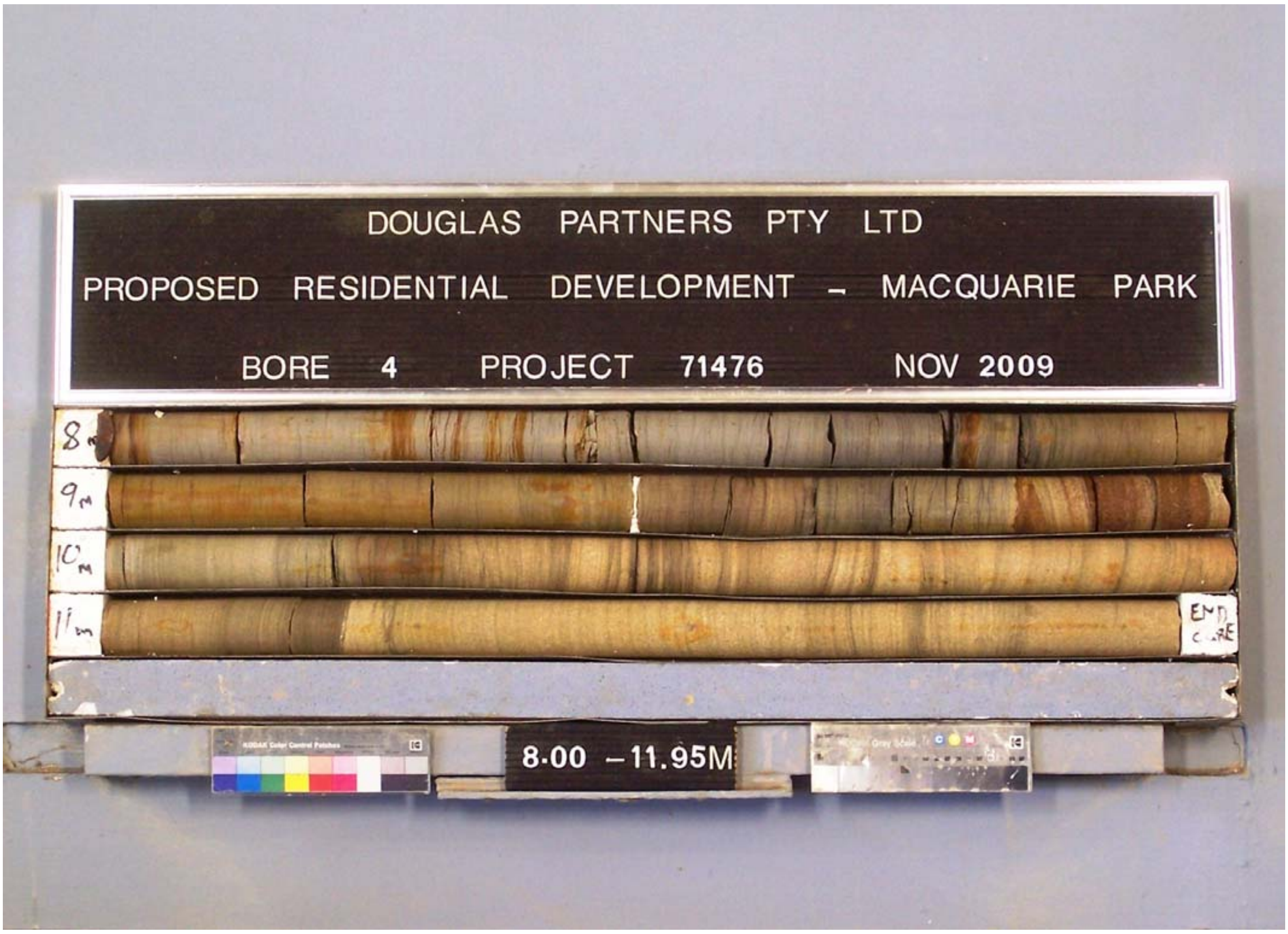

DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 2 PROJECT 71476 NOV 2009
11.00 –12.00M
KODAK Color Control Patches
Kodak Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 3 PROJECT 71476 NOV 2009
MACQUARIE PARK
71476 16/11/09
BH 3 START 45M
2
3
4
MM
5
1·15 – 6.00M
KODAK Color Control Patches
Gray Scale
C
M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 3 PROJECT 71476 NOV 2009
6
7
8
170 MM
LOSS.
9
10
6·00 –11.00M
KODAK Color Control Patches
Gray Scale
C
M

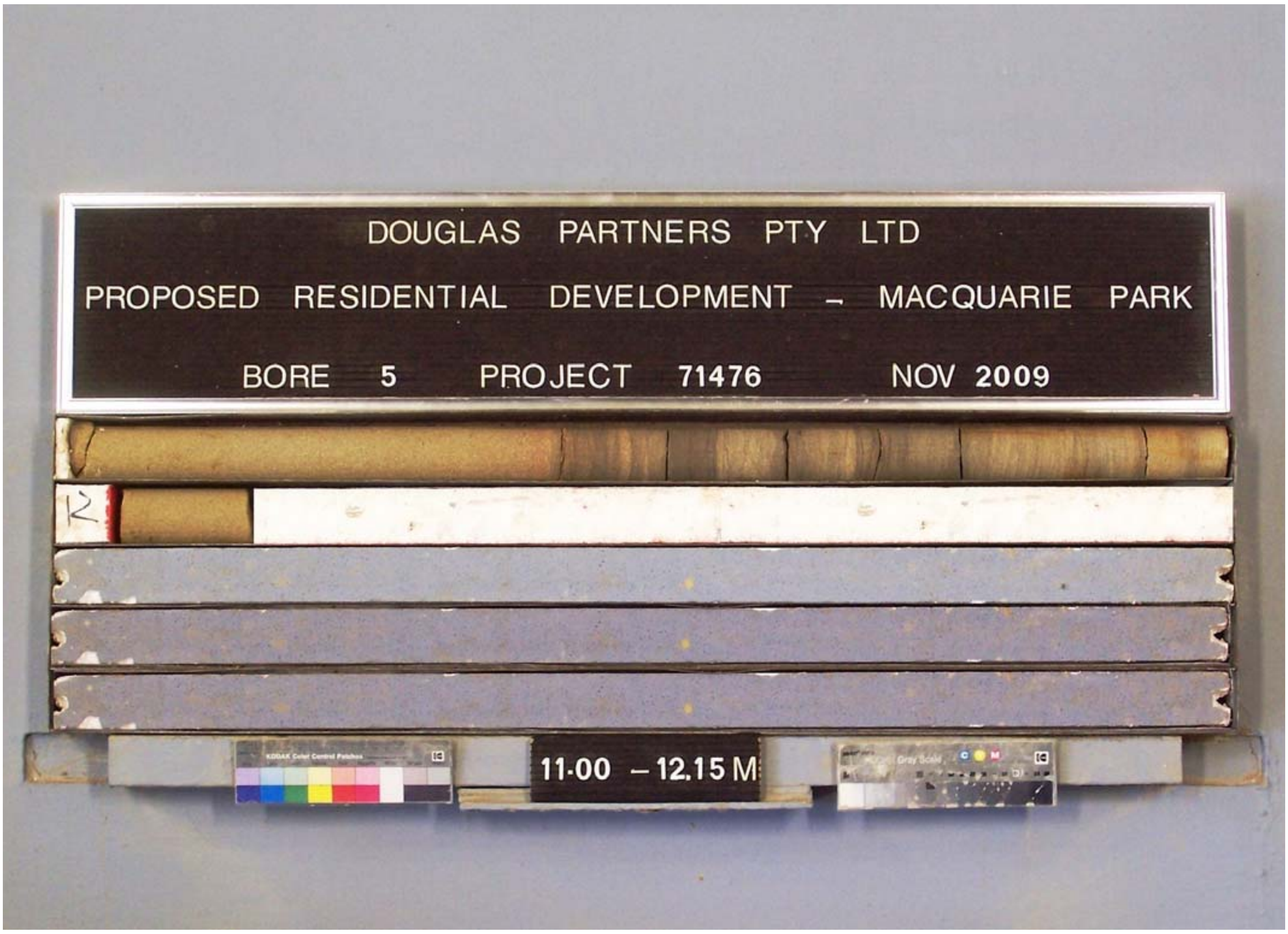

DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT - MACQUARIE PARK
BORE 3 PROJECT 71476 NOV 2009
11.00 -12.00M
KODAK Color Control Patches
Gray Scale
C
Y
M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 4 PROJECT 71476 NOV 2009
MACQUARIE PARK: BH:4:
71476: 11-11-09
3M
START 3·20M
4.0M
5.0M
6.0M
7.0M
3·20 – 8.00M
KODAK Color Control Patches
KODAK Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 4 PROJECT 71476 NOV 2009
8M
9M
10M
11M
END CORE
8·00 –11.95M
KODAK Color Control Patches
KODAK Gray Scale


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 5 PROJECT 71476 NOV 2009
MACQUARIE PARK
71476 11/11/09
BH5 START 1.4m
2
CORE LOSS 0.05M
3
4
5
CORELOSS 0.25M
KODAK Color Control Patches
1.40 – 6.00M
KODAK Gray Scale
C Y M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 5 PROJECT 71476 NOV 2009
6
LOSS
7
8
10
11
KODAK Color Control Patches
6.00 – 11.00M
KODAK Gray Scale
C Y M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 5 PROJECT 71476 NOV 2009
12
KODAK Color Control Patches
11·00 – 12.15 M
Kodak Gray Scale
C Y M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 6 PROJECT 71476 NOV 2009
MACQUARIE PARK 71476 BH 6
3.25m Start
4
CORE LOSS 0.25M
5
6
7
3·25 – 7.00M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 6 PROJECT 71476 NOV 2009
8
9
10
11
12m
8.00 – 12.15 M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 7 PROJECT 71476 NOV 2009
MACQAURIE PARK 17/11/09
71476 BH7 START 3.4 MTRS
4
5
6
7
3·40 – 8.00M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 7 PROJECT 71476 NOV 2009
8
9
10
CORE LOSS 0·36M
11
8·00 –12.00M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 8 PROJECT 71476 NOV 2009
MACQUARIE PARK 13/11/09
71476 BH8 START 4.7M
5
6
7
8
KODAK Color Control Patches
4.70 – 9.00M
KODAK Gray Scale
C Y M


DOUGLAS PARTNERS PTY LTD
PROPOSED RESIDENTIAL DEVELOPMENT – MACQUARIE PARK
BORE 8 PROJECT 71476 NOV 2009
9
10
11
END CORE
KODAK Color Control Patches
9.00 – 12.20M
KODAK Gray Scale
C Y M