Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Mixed Use Development
23 − 41 Lindfield Avenue & 9 −11 Havilah Lane
Lindfield

Prepared for
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Report on Geotechnical Investigation
Proposed Mixed Use Development
23 − 41 Lindfield Avenue & 9 − 11 Havilah Lane, Lindfield

1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for a mixed use development proposed for 23-41 Lindfield Avenue and 9-11 Havilah Lane, Lindfield. The work was commissioned by Mr Mathew Wagstaff of Aqualand Projects Pty Ltd (Aqualand), and was carried out in accordance with the agreed scope of works, as outlined in DP's proposal dated 3 December 2014.

Based on the architectural plans for the development (twelve sheets prepared by Crone Partners, Project No.CA 2924, Drawing Nos. SK001 to SK012) Douglas Partners Pty Ltd (DP) understands that Aqualand propose to construct a mixed use development comprising residential apartments and retail space. The building will include three (Havilah Lane side) to four (Lindfield Avenue side) basement and lower ground floor levels for car parking overlain by one ground floor level of retail space and a residential entrance lobby, including a few residential apartments. Above the retail levels two residential apartment towers are proposed. Tower A will extend to seven levels along the Lindfield Avenue frontage and Tower B will have seven levels along Havilah Lane. The proposed building will essentially occupy the full site area. The plans show the lowest basement floor level will lie at RL 85.4 m relative to Australian height datum (AHD), thus requiring excavation of the site to an approximate depth of up to 13 m, plus any additional footing, lift-well and site preparation depths.

The purpose of the investigation was:

- to determine the subsurface conditions present at the site; and
- to provide advice on site excavation, retention, foundations and site drainage.

The geotechnical investigation included the drilling of seven boreholes, laboratory testing of selected soil and rock core samples recovered from the boreholes, followed by engineering analysis and reporting. The details of the field and laboratory work are presented in the report, together with comments on design and construction practice.

DP has also undertaken a detailed site investigation for contamination for this site. The results of the assessment are presented in DP's "Report on Detailed Site Investigation", Project 73174.03, dated January 2015.

2. Background

This report presents the results of a geotechnical investigation undertaken by DP in 2012 for a previous site owner and for similar proposed development. Since 2012 the site has been purchased by Aqualand, who have also purchased additional smaller adjoining landholdings. Accordingly, DP has completed the drilling of an additional borehole (BH101) in one of the adjoining sites and has updated the previous report to include the results of both the 2012 and current (2104) investigations. DP's environmental reports from 2012 have also been updated and augmented with the drilling of additional boreholes (BH102 to 109).

3. Site Description

The site is located between the eastern side of Lindfield Avenue and the western side of Havilah Lane, north of their intersections with Kochia Lane at Lindfield. The site is an irregularly shaped parcel of land that is an amalgamation of several smaller sites that previously fronted Lindfield Avenue and/or Havilah Lane. The site covers an area of approximately 3800 m^2.

The ground surface at and surrounding the site falls to the north east at a gentle grade of approximately three to five degrees. Ground surface levels fall from an approximate reduced level of RL 98 m AHD at the south eastern corner of the site to approximately RL 94 m AHD at the north eastern site corner.

Existing site development includes a two storey retail centre with part basement level loading dock and part under-croft at-grade car parking that occupies approximately 50% of the site. In addition, a small retail building is situated at the south western corner and includes at-grade car parking to its rear with further on-grade car parking space present at the north east corner adjacent to Havilah Lane. Existing attached terrace retail buildings are present along the northern part of the Lindfield Avenue site boundary. Exposed site areas are covered with asphalt or concrete pavements, although a small grassed area is also present near the site centre. Construction of the existing larger retail building has resulted in a basement floor level of approximately 95.3 m AHD, meaning the site has already been excavated to depths of between 0 m (Havilah Lane site boundary) and 3 m (Lindfield Avenue site boundary).

4. Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale, with the Hawkesbury Sandstone Formation situated at slightly lower elevations and outcropping to the north and east of the site. The Ashfield Shale Formation generally consists of black to dark-grey shale and laminate. The weathered portion of this formation typically includes clays and silty clays of medium to high plasticity.

The Hawkesbury Sandstone Formation generally consists of medium to coarse grained quartz sandstone, with very minor shale and laminite lenses.

The field work confirmed the presence of predominantly shale and siltstone bedrock in the upper portion of the boreholes, with sandstone also intersected from depths of 5 m to 12 m. Overlaying soils comprised pavement materials and some filling, as well as residual clay and shaly clay.

5. Field Investigation

5.1 Field Work Methods (2012)

The initial field work was conducted over four days on 14, 17, 18 and 19 September 2012. The geotechnical investigation included:

- A walkover inspection of the site by a senior geotechnical engineer.
- Drilling of two boreholes (BH1 and BH2) using a truck-mounted DT100 drill rig. Initially the bores were drilled using solid flight augers fitted with a Tungsten-Carbide (TC) bit until practical refusal on weathered rock (and shaly clay) occurred at an approximate depth of 2.5 m in each borehole. Drilling was then advanced for 1 m to 1.5 m in lower strength rock with rotary wash bore methods and then further advanced to depths of 15 m and 12 m respectively in higher strength rock using NMLC diamond core methods.
- Standard penetration tests (SPTs) at 1 m depth in the overburden materials where silty clay was intersected in BH1 and BH2.
- Drilling of a further four boreholes (BH3 to BH6) using a track-mounted Dando-Terrier drill rig. Initially the bores were drilled using solid flight augers fitted with a Tungsten-Carbide (TC) bit until practical refusal on weathered rock (and shaly clay) occurred at depths of 2.5 m to 3.7 m. Drilling was then advanced in boreholes BH3 and BH4 for up to 3 m in lower strength rock with rotary wash bore methods and then further advanced to depths of 10 m and 9 m respectively in higher strength rock using NMLC diamond core methods.
- Collection of soil and rock core samples from the boreholes for examination, logging and to provide laboratory test specimens for point load strength index testing.

Standpipe piezometers were installed in boreholes BH1 and BH4 to assist the measurement of groundwater levels and to facilitate groundwater sampling for contamination testing purposes. Details of the piezometers are presented in DP's Report on Detailed Site Investigation. All other boreholes were backfilled with drilling spoil on completion of drilling.

Borehole locations were selected in consultation with the previous site owner and are shown on Drawing 2, in Appendix B. Locations were chosen based on drill rig accessibility, existing and proposed building geometries, proposed excavations and existing buried services. Prior to drilling at the site, bore locations were scanned for the presence of in-ground (buried) service lines. The surface level for each bore was levelled by a technical officer, relative to permanent survey mark SSM163033 (Lindfield Avenue), for which a reduced level of RL 99.50 AHD was indicated.

5.2 Field Work Methods (2014)

The current field work was conducted on 12 December 2014. The additional investigation included:

- A walkover inspection of the site by a senior geotechnical engineer.
- Drilling of one borehole (BH101) using a MD200 drill rig. Initially the bore was drilled using solid flight augers fitted with a Tungsten-Carbide (TC) bit to 2.5 m depth and then rotary wash bore methods in weaker rock to 5 m depth. Drilling was then advanced to a depth of 8.1 m in higher strength rock using NMLC diamond core methods.

- Standard penetration tests (SPTs) at 1.5 m depth intervals in the overburden materials.
- Collection of soil and rock core samples from the boreholes for examination, logging and to provide laboratory test specimens for point load strength index testing.

Additional boreholes (BH102 to BH109) were also drilled to depths of between 1 m and 2 m to assist the environmental site investigation. Details of these boreholes are presented in DP's Report on Detailed Site Investigation.

The borehole location was selected in consultation with the client and is shown on Drawing 2, in Appendix B.

5.3 Field Work Results

Details of the subsurface conditions encountered are given on the borehole logs presented in Appendix C, together with notes defining classification methods and descriptive terms.

A summary of the typical sequence of subsurface conditions encountered during the field investigation is presented below:

Pavements and Filling:	An asphalt wearing course approximately 50 mm thick, overlying granular filling including road base gravels and sand to depths of 0.7 m at BH1 and 0.3 m at BH101. Sandy clay filling with grass roots to a depth of 0.2 m at BH2. Concrete pavements of between 120 mm and 150 mm thick at BH3 to BH6 overlying road base gravels, crushed sandstone and sand filling to depths of between 0.25 m and 0.4 m.
Residual Clay: (Natural)	Intersected below the pavement and filling layers and extending to depths of between 2.8 m and 4.5 m. Consisting of light brown mottled orange-brown, clay and grey and red-brown shaly clay. The clay was generally stiff to very stiff, occasionally hard and damp to moist.
Weathered Rock:	Intersected from depths of 2.8 m to 4.5 m and primarily consisting of shale, interbedded shale and siltstone, and laminite, with sandstone at depth. The shale, siltstone and laminite were initially extremely to highly weathered to approximate depths of 9 m in BH1 and BH2 and 6 m in BH3 and BH4, then slightly weathered below. Slightly weathered rock was generally of low to medium strength, whereas the more weathered rock was less uniform and typically of extremely low to medium strength. The rock was typically fractured to slightly fractured. The degree of fracturing varied considerably, particularly in more weathered bedrock, although was less fractured within the sandstone layers. Bedding was essentially near horizontal and joints ranged in angle from 25 to 90 degrees in shale and siltstone (typically 40 to 70 degrees) and from 30 to 55 degrees in sandstone. A few thin clay seams and clay smears were identified in the rock core samples, generally along bedding separations.

5.4 Groundwater

No free groundwater was encountered during auger drilling. Once water was introduced into the borehole to facilitate rotary and NMLC drilling, further observation of groundwater seepage flows and levels was precluded. Long term/ongoing groundwater depth monitoring was beyond the scope of the investigation, although standpipe piezometers were installed in boreholes BH1 and BH4 as part of the 2012 contamination assessment. After development of the standpipes and prior to sampling for contamination purposes, groundwater was encountered in both boreholes at depths of 5.4 m and 4 m respectively during the 2012 assessment work. Recent groundwater observations indicate current groundwater levels of between 1.5 m (Bore MW3) and 2.8 m (Bore 1) below current ground surface levels.

6. Laboratory Testing

Rock core samples were collected from boreholes BH1 to BH4 and BH101 during the field investigations. Several sub-samples of the core were subjected to point load strength index testing in their axial direction for classification according to rock strength. The test results are presented on the log sheets in Appendix C, at the relevant depth.

In addition, laboratory aggressivity testing was undertaken on four soil samples by an external NATA accredited laboratory (Envirolab Services) to determine pH, chloride, sulphate and resistivity levels for exposure classification of buried concrete and steel elements in accordance with AS 2159 Piling Design and Installation 2009. The test results are presented in Appendix D and are summarised in Table 1.

Table 1: Results of Laboratory Soil Testing (Chemical)

BH	Depth (m)	$EC_{1:5}$ (µS/cm)	Texture Class	ECe (dS/m)	$pH_{1:5}$	Cl	SO_4	Classification & Aggressivity		
								Salinity	Concrete	Steel
1	1.0	96	LMC	0.77	4.3	6	63	Non	Moderate	Non
2	2.7	43	LMC	0.34	4.3	6	27	Non	Moderate	Non
3	2.0	52	LMC	0.42	4.5	4	48	Non	Moderate	Non
5	3.5	130	LMC	1.04	4.3	9	190	Non	Moderate	Non

Where: $EC_{1:5}$ = Electrical Conductivity (soil:water paste)
EC_e = Electrical Conductivity corrected for soil texture
SO_4 = Sulphate (mg/kg)
Cl = Chloride (mg/kg)

Salinities are classified by the method of Richards (1954).
Aggressivities to concrete are classified according to AS2159 (2009) – worst case (pH or SO4).
Aggressivities to steel are classified according to AS2159 (2009) – worst case (pH or Cl).
For aggressivity classifications, soils are considered to be in Condition B (low permeability).

7. Proposed Development

The proposed development will include the construction of mixed use building comprising residential apartments and retail space. The building will include three (Havilah Lane) to four (Lindfield Avenue) basement and lower ground floor levels for car parking overlain by one ground floor level of retail space and a residential entrance lobby, including a few residential apartments. Above the retail levels two residential apartment towers are proposed. Tower A will extend to seven levels along the Lindfield Avenue frontage and Tower B will have seven levels along Havilah Lane. The proposed building will essentially occupy the full site area. The plans show the lowest basement floor level will lie at RL 85.4 m relative to Australian height datum (AHD), thus requiring excavation of the site to an approximate depth of up to 13 m, although part of the site has already been excavated to an approximate depth of up to 3 m. Detailed excavations for footings, lift-wells and services may extend to greater depth.

8. Geotechnical Model

The following geotechnical model was developed for the site, based on the results of the geotechnical investigation. Two interpreted geotechnical cross-sections (Section A-A’ and Section B-B’) through the site are presented as Drawings 3 and 4 in Appendix B.

Ground conditions prior to development most likely comprised a thin topsoil and vegetation cover over an approximate 2.5 m to 4.5 m depth of residual clay and shaly clay, underlain by shale and siltstone and then sandstone bedrock. Ground surface levels would have been close to current ground surface levels at the perimeter of the site and probably fell gently towards the north to north east, as is typical of the surrounding area.

As a result of the existing development, the site has been excavated to depths of up to 3 m within central to southern parts of the site with probable minor regrading elsewhere. An approximate reduced level for the existing basement floor can be estimated from the level provided for BH3, undertaken at the basement entrance (RL 95.3 m AHD). The placement of minor thicknesses of filling may have occurred below the existing retail buildings, particularly the smaller structures along Lindfield Avenue, although most filling intersected in the boreholes is probably the result of local site trimming/regrading to facilitate pavement construction. Essentially all of the topsoil and vegetation appears to be gone.

Based on the investigation results, current site conditions can be characterised as including between 2.5 m and 4.5 m of overburden soils (pavements, filling and residual soil) overlying weathered and slightly fractured shale and siltstone bedrock, overlying sandstone bedrock.

Excavations within the footprint of the proposed building will remove all remaining overburden soils and a significant portion of the underlying shale and siltstone, particularly within the southern and western sides of the site where excavations will be deepest. Bulk excavations to an approximate level of RL 85 m AHD will expose medium to high strength sandstone along Lindfield Avenue and low to medium strength laminite and medium strength sandstone along Havilah Lane. Based on the ground surface levels recorded for each bore (BH1 to BH4 and BH101 only), the rock surface lies at the reduced levels listed in Table 2 below.

Table 2: Levels at Rock Surface from Bore Logs

Bore No.	Rock Depth (m)	Rock Surface Level (AHD)
BH1	4.0	93.6
BH2	3.4	92.7
BH3	4.0	91.1
BH4	2.8	91.3
BH101	5.0	87.7

Although the rock core is typically slightly fractured, some core samples indicate a slightly higher degree of fracturing, which is generally represented by short lengths of core loss in the borehole logs. Core samples also show some inclined joints, most dipping at 40 to 70 degrees in shale and siltstone and 30 to 55 degrees in sandstone. These joints may result in the development of wedge failures in perimeter excavation faces unless excavations are battered at flatter slope angles. For vertical or near-vertical excavations, these joints will require special consideration when designing temporary and permanent retention systems, and again during construction.

Although point load index strength testing indicates predominantly very low to low rock strengths for most of the excavation depth, rock strengths are expected to increase to at least low and medium strength close to the base of the excavation and possibly high strength for some areas of sandstone at the excavation base. The excavation sides will expose extremely very low to low strength shale and siltstone and low to medium strength sandstone, as well as overburden soils comprising filling and residual clay and shaly clay. Based on the results of point load strength index tests, together with fracture and joint spacing, the following rock classifications have been determined. Table 3 provides a summary of the rock classifications at each test bore (BH1 to BH4 and BH101 only). Classifications include the lower portion of the distinctly weathered shale within the Class V depth interval, which represents the approximate level at which the SPT test equipment refused or encountered particularly high blow counts.

Table 3: Summary of Geotechnical Model

Bore No.	Surface RL at Bore (AHD)	Depth (m) to / RL (AHD) at Top of Various Bedrock Classes			
		Class V	Class IV	Class III	Class II
BH1	97.6	2.5 / 95.1	6.7 / 90.9	7.4 / 90.2	13.0 / 84.6
BH2	96.1	2.5 / 93.6	5.9 / 90.2	7.5 / 88.6	11.5 / 84.6
BH3	95.1	2.5 / 92.6	5.0 / 90.1	6.6 / 88.5	-
BH4	94.1	2.7 / 91.4	5.1 / 89.0	7.5 / 86.6	-
BH101	92.7	3.8 / 88.9	4.5 / 88.2	5.8 / 86.9	-

9. Comments

9.1 Site Preparation

9.1.1 Excavations

Based on architectural plans for the development, site excavation will extend to between 6 m and 13 m depth and will initially be within pavement materials, filling and residual soil across the site following demolition of existing structures. Excavations will also intersect the underlying very low, low and medium strength shale, siltstone, laminite and to a lesser extent medium strength sandstone near the excavation base. Excavations within soil will require the use of medium sized excavators for excavation efficiency.

Excavations within the underlying slightly fractured rock will require larger excavators (say 30+ tonne) and will require extensive ripping and the use of medium to large sized hydraulic hammers. Detailed rock excavations and/or excavations near vibration sensitive structures or services, may require the use of rock grinders and/or saws to minimise vibratory effects beyond the site boundary. The geometry of rock joints, fractures and bedding planes will assist site excavation, although joint alignment may not coincide with site boundaries, thus additional care will be required to avoid localised wedge failures and over-excavation near the site/excavation perimeter.

Excavations will remove rock of various classes and strengths. Although Table 3 in Section 7 provides approximate reduced levels for the top of each rock class at each test bore, contractors should be aware that higher strength rock is also likely, particularly at the base of deeper excavations, including those where sandstone is encountered. It is probable that Class II sandstone will be exposed on this site or encountered in footing excavations. Hence, contractors tendering the project should select appropriate excavation machinery.

Excavated material to be disposed of off-site should be tested for contaminants to allow Waste Classification Assessment in accordance with NSW EPA requirements. The results of DP's Detailed Site Investigation for contamination should be consulted for information on the environmental condition of the site and the appropriate classification of soil to be disposed of off-site.

9.1.2 Vibrations with Excavation

From DIN 4150-2 (1999) a maximum limit of 20 mm/s PPVi (peak particle velocity of particle motion for any direction component) is recommended to prevent structural damage to commercial and industrial structures. However, some architectural damage such as cracks through rendering, cornices and skirtings may occur below this limit, particularly for buildings that have been poorly constructed. Accordingly, if hammer excavations are required at the site, then it may be necessary to undertake vibration monitoring to ensure vibrations from the excavations do not affect the neighbouring industrial properties, nearby services or possibly the residential properties on the opposite side of Havilah Lane. Dilapidation surveys should be carried out on neighbouring properties immediately adjacent to the site prior to commencement of demolition or other construction activities to allow an appropriate response to claims for damage. For adjoining properties that contain buildings comprising several residential, retail or commercial units, only those closest to the site would require dilapidation surveys.

Ground vibration can, however, be strongly perceptible to humans at levels above 2.5 mm/s vector sum peak particle velocity (VSPPV) and can be disturbing at levels above 5 mm/s VSPPV. Complaints from building occupants are sometimes received when levels are as low as 1 mm/s VSPPV. The Australian Standard AS2670.2-1990 "Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)" indicates an acceptable day time limit of 8 mm/s peak velocity of vertical particle motion (PPVz) for human comfort.

Taking the above into account and given that services buried within the adjoining footpaths are likely to be relatively old, it is recommended that vibration levels at the foundation level of adjoining buildings and within the footpaths be kept below 5 mm/sec peak particle velocity (PPV).

Table 4 provides a guide to the relationship between PPV and different plant types at set distances. This can be used for preliminary cost estimation but should be verified with site-specific trials.

Table 4: Guide to Vibration Caused by Equipment

Equipment Type	Distance (m)	Peak Particle Velocity* (mm/s)	Component Velocity (mm/s)
Krupp 300 or equivalent rock hammer	1.0 1.5 2.0 5.0	15 - 30 10 - 17 7 2 - 4	11 – 21 7 – 12 5 1 – 3
Krupp 600 or equivalent rock hammer	3.5 6 9	10 4 – 5 2	7 3 – 3.5 1
Krupp 900 or equivalent rock hammer	6 9 20	20 10 – 11 4	14 7 – 8 3
3m diameter saw on excavator	0.5 – 1 1 – 2	5 3	3.5 2

Note: * Assumes two components are in-phase to generate peak particle velocity

No data is available for line drilling, however it is anticipated that vibration levels should be similar to, or less than, those due to the use of a rock saw.

Vibration intensity is affected by many factors, including equipment type, rock strength and jointing. As such, although initial selection may be made based on Table 4, it is recommended that a vibration monitoring trial be carried out, to confirm the effects of rock hammer excavation, particularly near to common lot boundaries where adjoining structures are less than 20 m away. Further, if vibrations cannot be minimised during construction, then unattended monitoring during the excavation period would be appropriate.

To minimise the effects of hydraulic rock hammer equipment, the work method should allow for:

- Saw cutting or line drilling of all excavation perimeters in rock strengths of medium or higher;

- Additional cuts within the body of the excavation, if required;
- Excavation of loose or rippable sandstone blocks by bucket or single tyne attachments prior to commencement of rock sawing or hammering. Care should be taken to ensure that existing, loosened blocks do not continue into the adjacent foundation areas;
- Progressive breakage from open excavated faces;
- Selective breakage along open joints where these are present;
- Use of rock hammers in short bursts to prevent generation of resonant frequencies; and
- The movement of large blocks away from structures prior to breaking up for transport from site.

Prospective excavation contractors should note that intact rock strengths may be higher than indicated by classification due to downgrading based on the presence of defects and low strength bands within the rock. Contractors should inspect rock cores before tendering to make their own assessment.

9.2 Retaining Walls

The perimeter of all excavations will require support from both temporary and permanent retaining walls, as there is unlikely to be sufficient room to create batters, particularly for excavations of between 6 m and 13 m depth, although some battering may be possible along the northern side of the basement subject to final basement dimensions. Further retention at lift-wells may also be necessary. Although final structural plans have not been viewed, it is anticipated that retaining walls will include staged, anchored and shotcreted walls to all sides of all excavations, including the construction of anchored soldier piles. However, if the construction of batters is possible, then DP suggests that batter constructions adopt the following grades outlined in Table 5. Flatter slopes may be necessary to prevent or restrict erosion.

Table 5: Maximum Temporary & Permanent Batter Slopes

Material Type	Short Term (H:V)	Long Term (H:V)
Filling and stiff to very stiff clay	1.5:1	2:1
Class V and Class IV bedrock	0.75:1	2:1
Class III or better bedrock	0.5:1	1:1

Notes: 1 – Batter grades in filling should be confirmed during construction.
2 – Adverse jointing in weathered rock may require flatter slopes.

It is suggested that the design of the retaining wall system is based upon an average bulk unit weight of 20 kN/m^3 and 22 kN/m^3 for soil and rock respectively, with a triangular earth pressure distribution based on lateral earth pressure coefficients as listed in Table 6.

Table 6: Lateral Earth Pressure Coefficients

Material Type	Lateral Earth Pressure Coefficient (K_a)		K_o
	Temporary Support	Permanent Support	
Filling, clay and Class V bedrock	0.25	0.3	0.5
Class IV and Class III bedrock	0.2	0.25	0.25
Class II or better bedrock	0.1	0.15	0.15

The infill panels and the soldier piles should be designed as a composite retaining wall for full soil pressure.

The above K values assume that no adversely dipping joints are present. Based on the borehole results it is considered possible that some major through-going adversely orientated joints or faults could be encountered at the site, as some inclined joints were observed in the cores. This should be carefully checked by regular geotechnical inspections during each drop in level in the excavation. If adversely oriented geological structures are identified, then additional anchors may be required.

The horizontal pressures acting on the wall can be estimated based on the following formula:

$$\sigma_z = K\, z\, \gamma$$

Where, σ_z = Horizontal pressure at depth z (m)
K = Earth pressure coefficient
z = Depth (m)
γ = Unit weight of soil or rock (kN/m^3)

Alternatively, estimation of lateral earth pressures for the lower braced or anchored portions of the retention system requires a different calculation method. Earth pressures should be considered to have a trapezoidal distribution. In such cases, the uniform horizontal pressures acting on the retained height of the anchored wall can be calculated based on the following formula:

$$P_c' = 0.4\, \gamma\, H$$

Where, p_c' = Maximum horizontal pressure acting on lower 80% of wall height (kPa)
γ = Unit weight of soil or rock (kN/m^3)
H = Wall Height (m)

The above suggests an earth pressure of 8H, which is based on limiting lateral ground surface movements at the retaining wall perimeter and reducing the degree of potential movement (settlement and lateral deflection) below adjoining structures founded on high level footings and along services (or other infrastructure) founded within ground above a 45 degree influence line draw up and away from the top of Class III rock at the excavation face. However, if some movement of the ground behind the wall is permitted, then lateral earth pressures of 6H could be adopted for the design of anchored walls. All soil parameters listed above remain valid irrespective of the calculation method used.

Given the slightly fractured nature of the bedrock, the design of retaining walls must also consider maximum loads resulting from unstable rock mass wedges. Design of anchors should therefore consider maximum potential loads (in kN) due to wedge failures, which can be considered as being equal to 4.2 times H^2, where H is the height of the retaining wall in metres.

Additional pressures should be allowed for where surcharging occurs from adjacent building footings, traffic or other loadings. Unless positive drainage measures can be incorporated to prevent water pressure build up behind the walls, full hydrostatic head should be allowed for in design while, at the same time, allowing the soil density to reduce to the buoyant condition.

Drainage is normally provided behind the shotcrete walls. The sprayed concrete wall should provide adequate structural support, however it may be appropriate to install a false wall (single brickwork or blockwork) for aesthetic purposes and to avoid dampness.

Care should be exercised in construction to ensure that anchors are installed progressively during excavation, and stressed up, with shotcreting carried out at regular intervals to limit the exposed sections to no greater than say 3 m widths. The first row of anchors should be installed and stressed up to 80% working load prior to excavation of the next row of panels. Stress relief related movement may lead to an increase in the load in anchors.

Prospective piling contractors should note that intact rock strengths are higher than indicated by classification, due to slight downgrading.

9.3 Anchor Design

Soldier piles can be anchored with prestressed type rock anchors and/or passive rock bolts, as appropriate for the supporting stratum and the ability to withstand minor wall deflections. It is suggested that rock anchors be inclined as steeply as possible, but not exceeding 45°, to allow anchoring in the stronger, less fractured rock at depth. Anchors can be installed at greater inclinations but careful consideration is required during design to determine the likelihood of adverse vertical and shear forces developing in the anchor bars. For estimating purposes, DP recommends the following allowable bond stress values listed in Table 7.

Table 7: Allowable Bond Stress

Material Type	Ultimate Bond Stress (kPa)
Filling	0
Residual Clay (very stiff or better) & Class V bedrock	50
Class IV and III bedrock (other than sandstone)	300
Class III bedrock (sandstone)	1000

Notes: 1 – A free length of at least 3 m should be provided to each anchor.
2 – Bond lengths should commence below a plane inclined at 45° from the base of the excavation.

Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the anchor system and method of installation) are used and that the anchor holes are carefully cleaned out prior to grouting. After anchors have been installed, it is recommended that they be tested to 125% of nominal working load and then locked off at between 60% and 80% of their working loads.

Checks should be carried out to ensure that the load is maintained in the anchors throughout the construction period and is not lost due to creep effects or to other causes.

9.4 Foundations

Based on the results of the field investigation, footing types appropriate for the estimated foundation conditions include pad and/or pile footings founding in rock. Piled footings may be required, as depths to rock of reasonable bearing strength could preclude economical construction of pad footings. Footing design may be based on subsoil Class B_e (rock) in accordance with AS1170.4-2007, "Earthquake actions in Australia".

9.4.1 Shallow Foundations

The proposed building will require excavation of the site to depths of between 6 m and 13 m to attain bulk excavation level. Given the anticipated magnitude of column loads, it is likely that all structural loads for the buildings will be mostly supported on large pad or piled footings within medium or higher strength rock. However, for other loads supported by spread footings (pads and strips) founding at shallow depth within the natural soil profile, an allowable bearing pressure of 150 kPa is suggested. Foundations within the existing filling should be avoided.

9.4.2 Deep Foundations

Footings for the new building will most likely comprise a mixture of spread footings (pads) and piles that found within the underlying Class V to Class III shale, siltstone and sandstone at typical depths of between 0 m and 6 m below bulk excavation level.

For the design of foundations, recommended maximum ultimate and allowable (or "serviceability") bearing pressures and estimated elastic modulus values for the foundation materials encountered in the cored boreholes are presented in Table 8.

Table 8: Recommended Parameters for Foundation Design

Material	End Bearing Pressure (MPa)		Shaft Adhesion (MPa)		Field Elastic Modulus, E (MPa)
	Allowable[3]	Ultimate	Allowable[3]	Ultimate	
ELS (Class V) Shale & Siltstone	0.7	3.0	0.07	0.07	50
VLS (Class IV) Shale & Siltstone	1.0	3.0	0.1	0.15	100
LS (Class III) Shale & Siltstone	2.0	8.0	0.2	0.4	200
MS (Class III) Sandstone	3.5	20	0.35	0.8	350
M-HS (Class II) Sandstone [1,2]	6.0	60	0.6	1.5	900

Where: ELS = Extremely Low Strength VLS = Very Low Strength LS = Low Strength MS = Medium Strength

Notes: 1 – For use if Class II sandstone is encountered during footing construction.
2 – The presence of Class II sandstone should be confirmed by an experienced geotechnical engineer.
3 – The tabulated allowable values are based on limiting settlements to <1% of the minimum footing dimension.

Given the deeply weathered rock profiled and presence of numerous clay seams within the upper part of the rock core samples, some spoon testing would be required at pad footings to assess the potential for weak seams that may be present below the foundation level, subject to the parameters used and classification assumed for the foundation. This is particularly so for shale and laminite rock types. If weak seams are identified, additional spoon testing may be required and/or the allowable bearing capacity may need to be reduced.

Pad footings founding near excavations (lift wells, service trenches or similar) must have all loads transferred to rock below an influence line inclined at 45° commencing from the lowest and closest side of the excavation or trench base. Pad footings can be deepened to accommodate this load transfer or alternatively piled footings may be used. Pad footings founding above this line may be designed for only 50% of the relevant tabulated values, subject to specific geotechnical inspection during construction.

Local variations in rock strength and type may occur across the site. All footing excavations should be inspected by a geotechnical engineer or engineering geologist and approved prior to concreting to confirm reduced pressures are not warranted due to extensively weathered or jointed zones. At least one-third of all pad footings should be spoon tested prior to placement of reinforcement and pouring of concrete to ensure the adequacy of the founding material.

9.5 Aggressivity

At all locations soils were found to be non-aggressive to steel. However, at all four locations tested, soils were found to be moderately aggressive to concrete, due to low pH values, using the criteria within Australian Standard AS2159 (2009). The Standard provides advice on structural design life, minimum concrete strengths and minimum reinforcement cover for piles founded in moderately aggressive ground conditions. Where concrete of lower than recommended strength is used then a shorter lifetime may be expected, however, no estimates are given in the Standard of this reduced lifetime.

9.6 Hydrogeological, Groundwater and Seepage

Iron-stained fracture surfaces evident within the core samples suggest at least intermittent seepage flow is likely. From experience, groundwater seepage would typically flow across the underlying rock surface or through open joints, fractures and bedding separations. Accordingly, seepage (if present) would be topographically and geometrically controlled on or within the rock mass.

Although seepage is likely to be present intermittently, the rate of seepage is expected to be low, thus appropriate and typical ‘sump and pump’ construction methods should provide sufficient control during construction. Seepage removal is unlikely to affect nearby structures.

Given the potential for intermittent seepage at this site, it is recommended that a minimum 150 mm thickness layer of compacted, permeable aggregate is incorporated into the subbase below ground floor slabs to prevent capillary rise of potentially saline groundwater.

The high concentration of iron likely to be present in the groundwater will result in the potential formation of a glutinous iron precipitate in sub-floor drains and pumps. Drainage installations should therefore incorporate easy access for periodic maintenance (e.g. rodding of drains, back-flushing).

Based on the observations of the limited duration of pumping of the groundwater during environmental sampling, rapid drawdown was experienced suggesting that the groundwater is of relative low recharge. Seepage flows into the excavation should be expected from depths of 5.5 m at the south western part of the site to 4 m at the south eastern part.

9.7 Pavements

Considering the clay soil and weathered rock profiles that are likely to be exposed on completion of the bulk excavation works, a design CBR value of 3% is suggested for pavements and basement floor slabs founding on clay, increasing to a design CBR value of 8% for pavements and basement floor slabs founding on sandstone at this site. Pavement subgrades should be protected from adverse moisture contents and subsurface seepages by perimeter subsoil drains at all pavement edges.

9.8 Effects on Adjoining Rail Land

The southern corner of the site is the closest point of the proposed excavation works to the rail corridor on the opposite side of Lindfield Avenue. The offset from the development site’s boundary to the closest boundary of the rail corridor is at least 20 m, with the closest structure being the station platform building, offset 23 m from the development. The closest distance for the rail track is 32 m. The ground surface with the rail corridor at this closest offset is within approximately 0.5 m of the existing ground surface fronting the development’s boundary along Lindfield Avenue.

The proposed depth of excavation at the point closest to the rail corridor is equal to the maximum depth of excavation proposed for the development at 13 m. Figure 1 shows the relationship between the development site and the rail corridor.

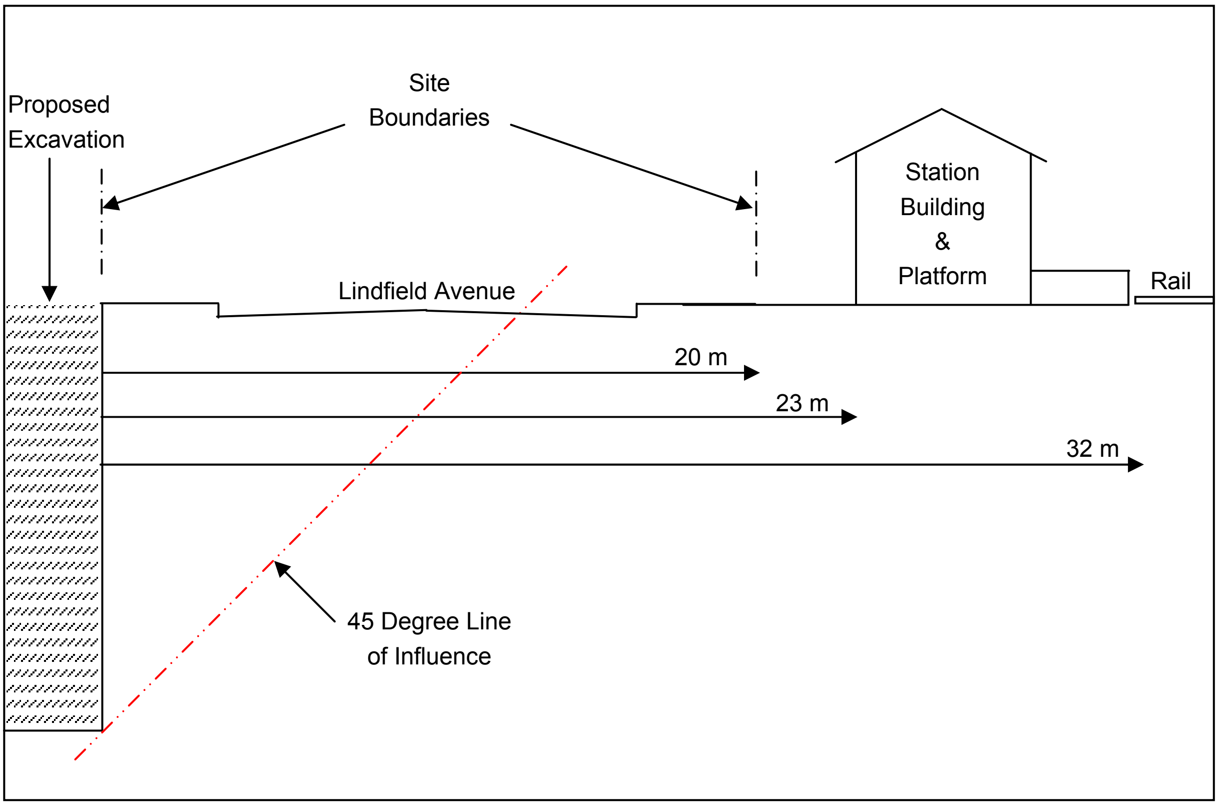

Figure 1: Section showing relationship of site with rail corridor

Based on the geometry outlined in Figure 1, the proposed excavation and site development works are not expected adversely affect the rail corridor, or the structures within.

10. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for the proposed mixed use development at 23-41 Lindfield Avenue and 9-11 Havilah Lane, Lindfield in accordance with DP's proposal dated 3 December 2014 and acceptance received from Mr Mathew Wagstaff from Aqualand Projects Pty Ltd (Aqualand). The work was carried out under DP's conditions of engagement. This report is provided for the exclusive use of Aqualand for the specific project and purpose as described in the report. It should not be used for other projects, other sites or by a third party. DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are considered to be indicative of the subsurface 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. Subsurface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP.

Douglas Partners Pty Ltd

Appendix A

About this Report

Introduction

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

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

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

Site Anomalies

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

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

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Geotechnics | Environment | Groundwater

CLIENT:	Aqualand Projects Pty Ltd		
OFFICE:	Sydney	DRAWN BY:	RCB
SCALE:	NTS	DATE:	21.1.2015

TITLE: **Site Location and Boundaries**
23-41 Lindfield Avenue & 9-11 Havilah Lane, Lindfield

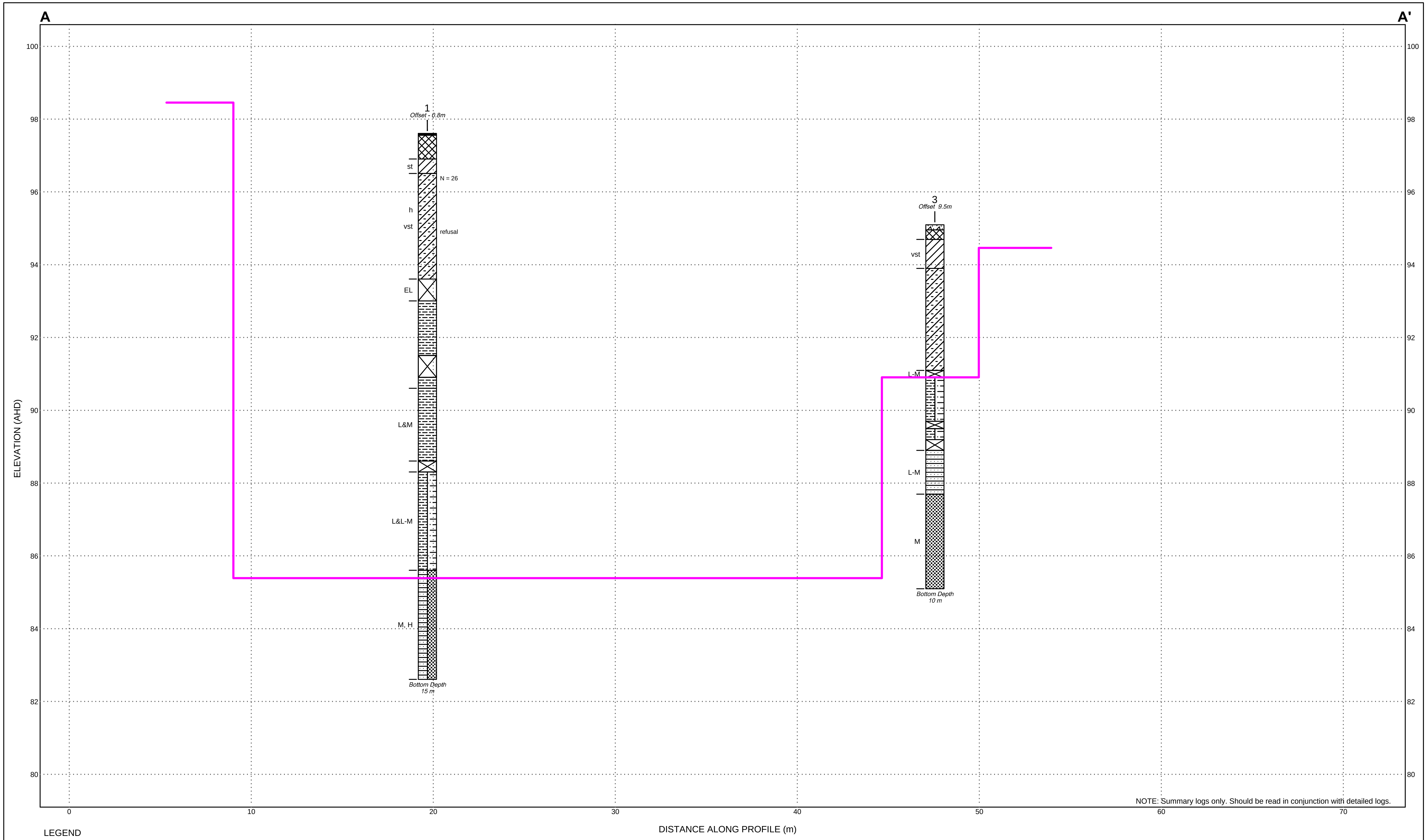
PROJECT No:	73174.02
DRAWING No:	1
REVISION:	A

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Aqualand Projects Pty Ltd
OFFICE: Sydney
DRAWN BY: RCB
SCALE: NTS
DATE: 21.1.2015

TITLE: **Bore and Well Locations**
23-41 Lindfield Avenue & 9-11 Havilah Lane, Lindfield

PROJECT No: 73174.02
DRAWING No: 2
REVISION: A



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

LEGEND

☒	Core Loss	▩	Filling	▨	Shaly Clay
■	Asphaltic Concrete	▤	Laminite		
▨	Clay	▦	Sandstone coarse grained		
▦	Concrete	▤	Shale		

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

0 4
Horizontal Scale (metres)

Vertical Exaggeration = 2.0

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Geotechnics / Environment / Groundwater

CLIENT: Aqualand Projects Pty Ltd		TITLE: Cross-section A - A' Mixed-Use Retail & Residential Development 23-41 Lindfield Avenue & 9-11 Havilah Lane, Lindfield	PROJECT No: 73174.02
OFFICE: Sydney	DRAWN BY: PSCH		DRAWING No: 3
SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 22.1.2015		REVISION: 0

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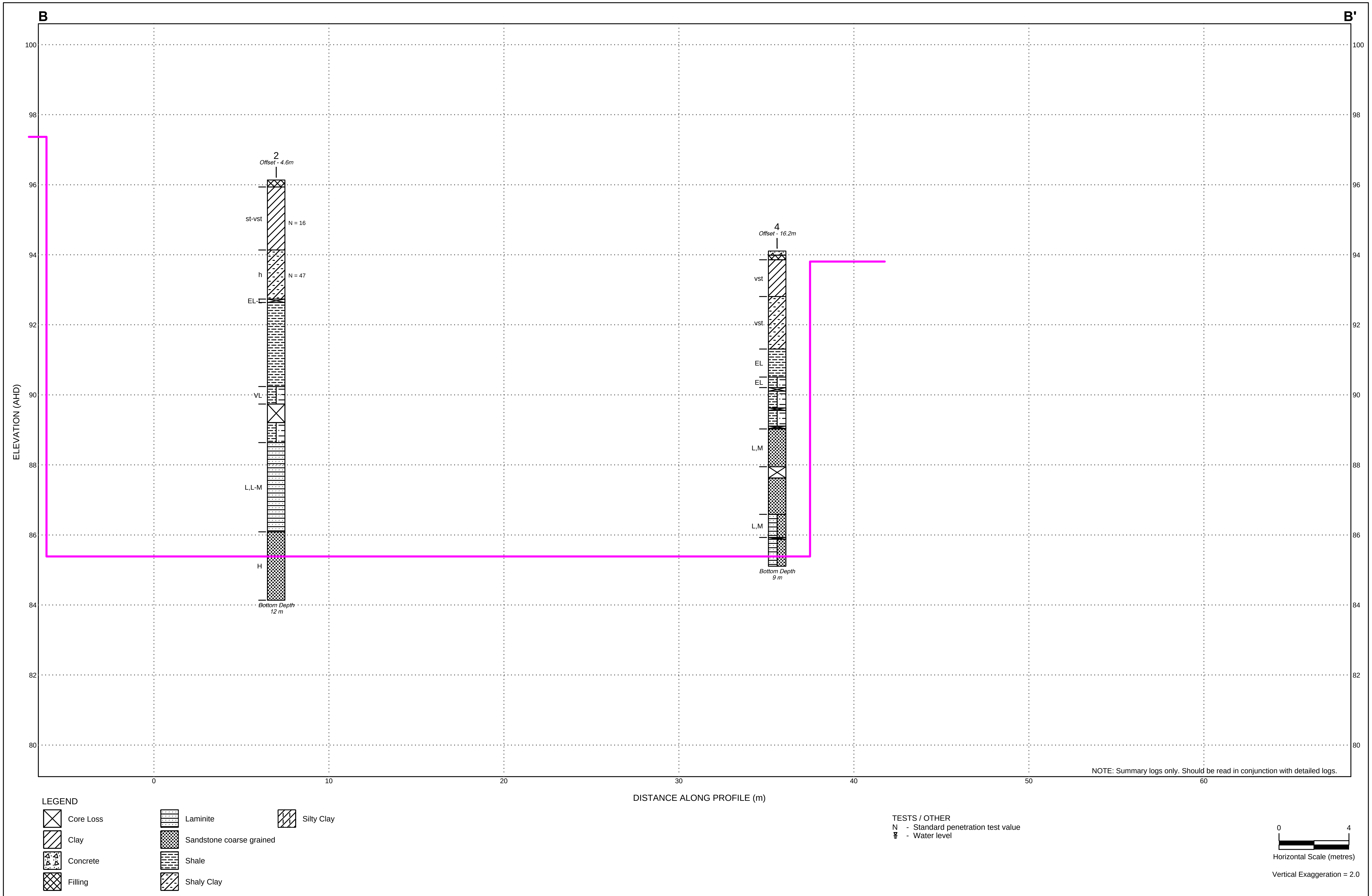

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Aqualand Projects Pty Ltd		TITLE: Cross-section B - B' Mixed-Use Retail & Residential Development 23-41 Lindfield Avenue & 9-11 Havilah Lane, Lindfield	PROJECT No: 73174.02
OFFICE: Sydney	DRAWN BY: PSCH		DRAWING No: 4
SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 22.1.2015		REVISION:

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Appendix C

Field Work Results

Sampling

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

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

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

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

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

The test results are reported in the following form.

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

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

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Description and Classification Methods

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

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

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

Transported soils may be further subdivided into:

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

Rock Descriptions

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and loner sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

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

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

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

Symbols & Abbreviations

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Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

■	Asphalt
▒	Road base
△	Concrete
▦	Filling

Soils

▨	Topsoil
▒	Peat
▨	Clay
▨	Silty clay
▨	Sandy clay
▨	Gravelly clay
▨	Shaly clay
▥	Silt
▥	Clayey silt
▥	Sandy silt
▒	Sand
▒	Clayey sand
▒	Silty sand
○	Gravel
○	Sandy gravel
○	Cobbles, boulders
△	Talus

Sedimentary Rocks

○	Boulder conglomerate
○	Conglomerate
▒	Conglomeratic sandstone
▒	Sandstone
▤	Siltstone
▤	Laminite
▤	Mudstone, claystone, shale
■	Coal
▦	Limestone

Metamorphic Rocks

∼	Slate, phyllite, schist
+	Gneiss
▒	Quartzite

Igneous Rocks

+	Granite
∨	Dolerite, basalt, andesite
×	Dacite, epidote
∨	Tuff, breccia
▢	Porphyry

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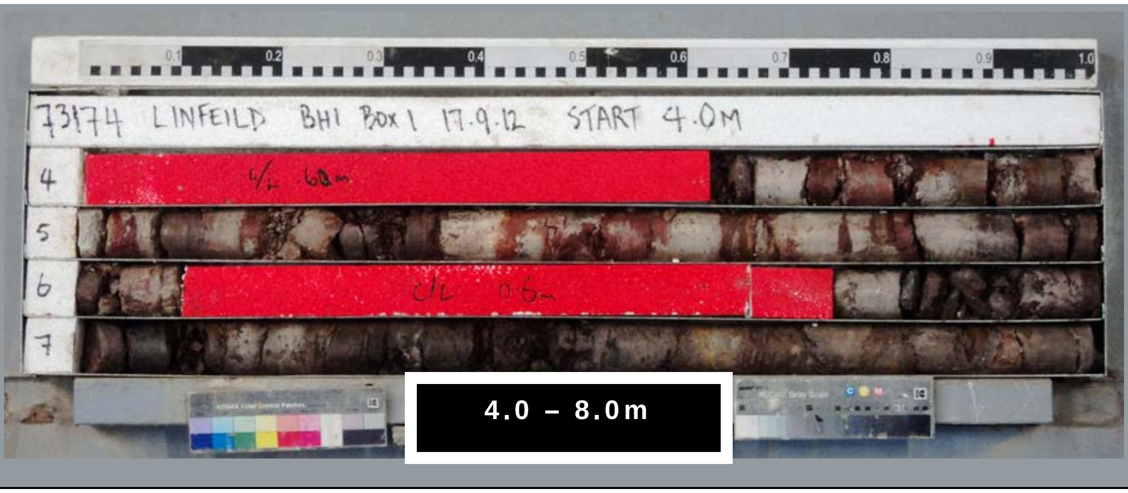
PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 1　　PROJECT 73174　　SEP 2012

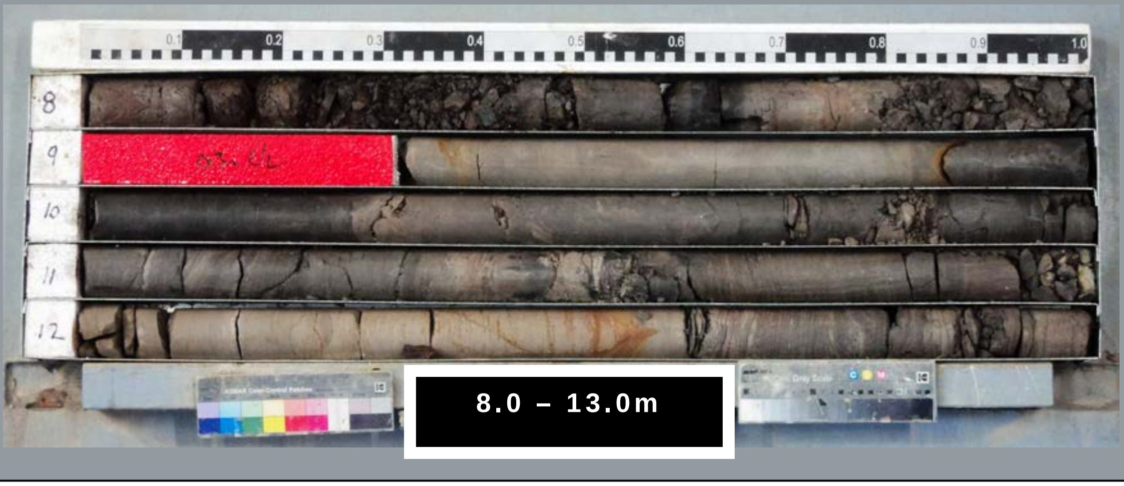


4.0 – 8.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 1　　PROJECT 73174　　SEP 2012



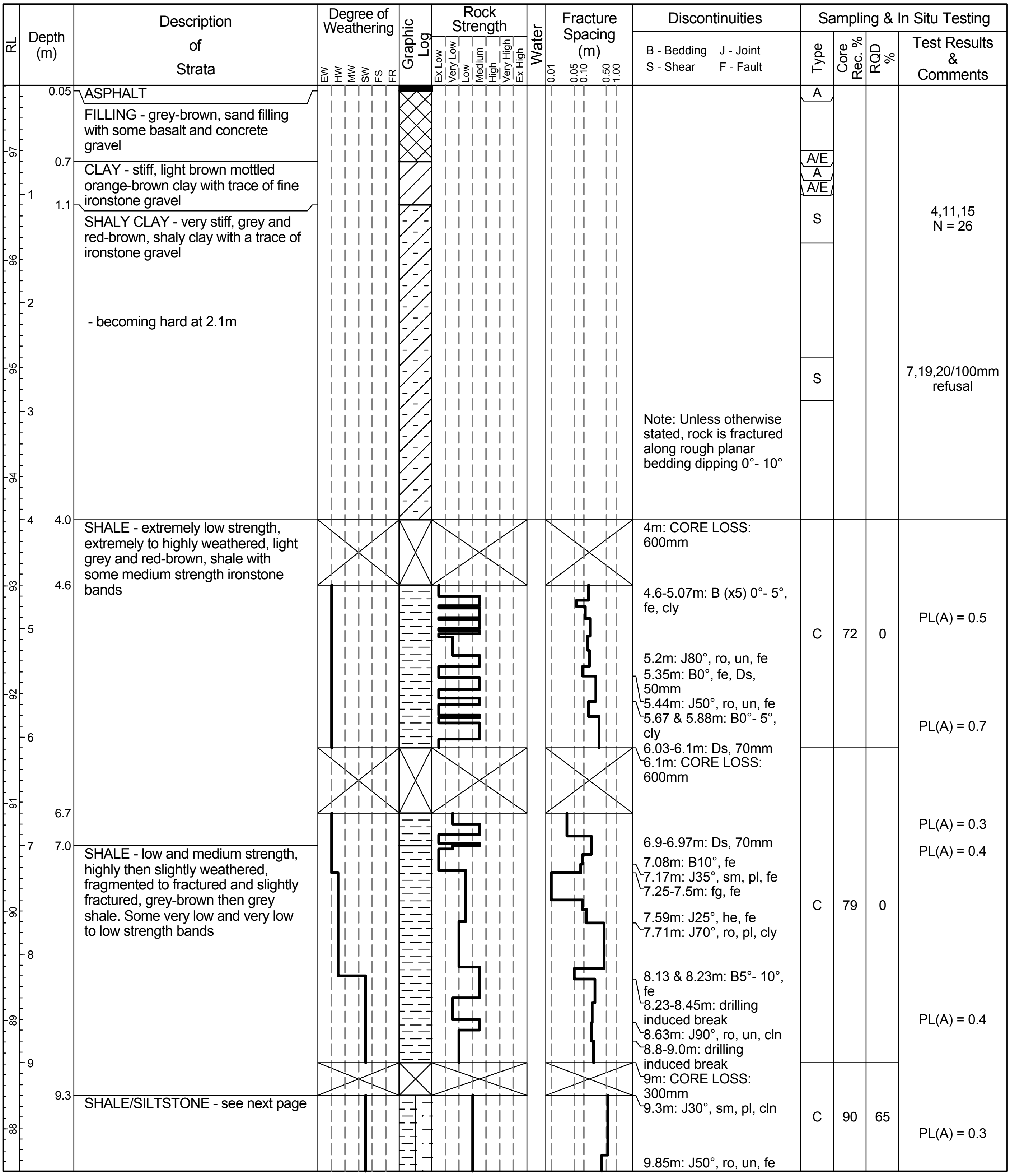
8.0 – 13.0m

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 97.61 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 73174
DATE: 17/9/2012
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALT		A			
0.7	FILLING - grey-brown, sand filling with some basalt and concrete gravel		A/E			
1.1	CLAY - stiff, light brown mottled orange-brown clay with trace of fine ironstone gravel		A A/E			
	SHALY CLAY - very stiff, grey and red-brown, shaly clay with a trace of ironstone gravel - becoming hard at 2.1m	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S S			4,11,15 N = 26 7,19,20/100mm refusal
4.0	SHALE - extremely low strength, extremely to highly weathered, light grey and red-brown, shale with some medium strength ironstone bands	4m: CORE LOSS: 600mm	C	72	0	
4.6		4.6-5.07m: B (x5) 0°- 5°, fe, cly 5.2m: J80°, ro, un, fe 5.35m: B0°, fe, Ds, 50mm 5.44m: J50°, ro, un, fe 5.67 & 5.88m: B0°- 5°, cly 6.03-6.1m: Ds, 70mm				PL(A) = 0.5 PL(A) = 0.7
		6.1m: CORE LOSS: 600mm	C	79	0	
6.7						PL(A) = 0.3
7.0	SHALE - low and medium strength, highly then slightly weathered, fragmented to fractured and slightly fractured, grey-brown then grey shale. Some very low and very low to low strength bands	6.9-6.97m: Ds, 70mm 7.08m: B10°, fe 7.17m: J35°, sm, pl, fe 7.25-7.5m: fg, fe 7.59m: J25°, he, fe 7.71m: J70°, ro, pl, cly 8.13 & 8.23m: B5°- 10°, fe 8.23-8.45m: drilling induced break 8.63m: J90°, ro, un, cln 8.8-9.0m: drilling induced break				PL(A) = 0.4 PL(A) = 0.4
		9m: CORE LOSS: 300mm	C	90	65	
9.3	SHALE/SILTSTONE - see next page	9.3m: J30°, sm, pl, cln 9.85m: J50°, ro, un, fe				PL(A) = 0.3

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

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.0m; NMLC-Coring to 15.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 15.0m (Screen 3.0-15.0m; Gravel 2.5-15.0m; Bentonite 1.5-2.5m; Backfill to GL)

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
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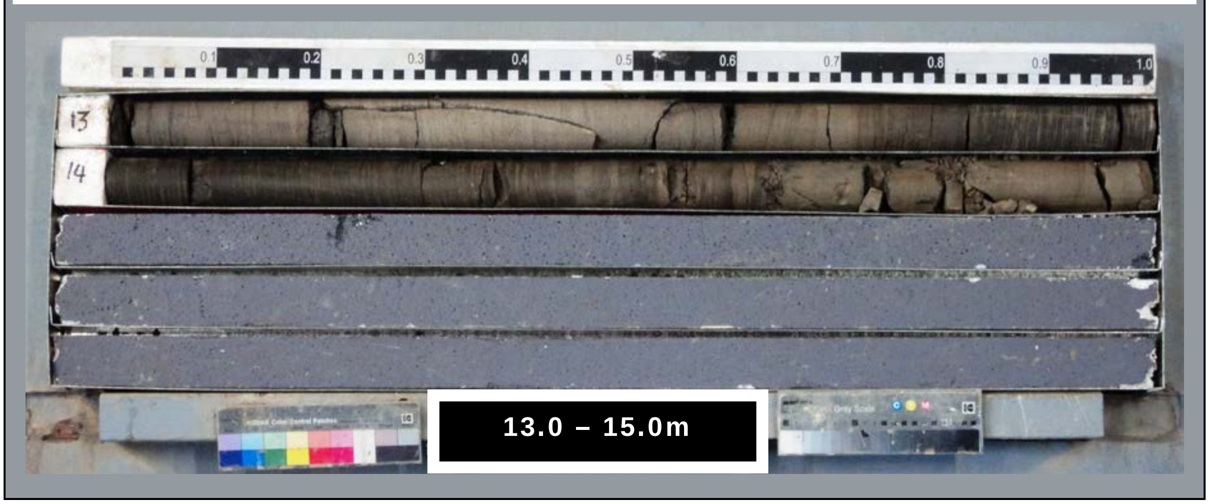
DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 1　　PROJECT 73174　　SEP 2012



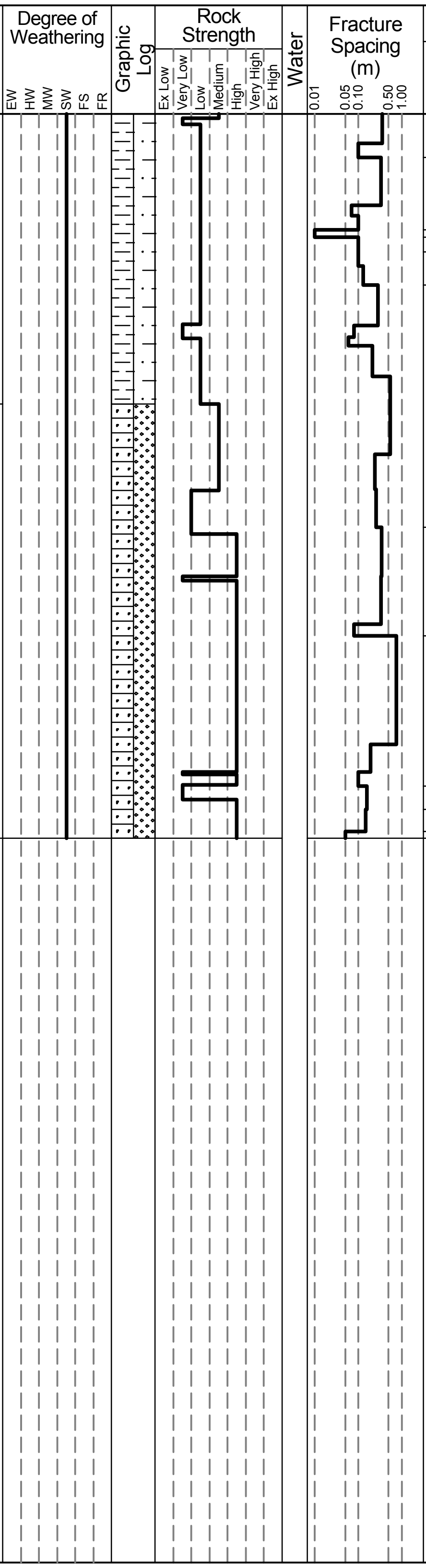
13.0 – 15.0m

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 97.61 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 73174
DATE: 17/9/2012
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
87 86	11	SHALE/SILTSTONE - low and low to medium strength, slightly weathered, fractured and slightly fractured, grey shale/siltstone with some medium and very low strength bands *(continued)*	10.2m: J55°, sl, sm, un, cln 10.3m: J30°, ro, un, cly 10.7m: J35°, ro, un, cln 10.8-10.85m: Cz, 50mm 10.85m: J35°, ro, pl, cln 10.95m: J85°, ro, un, cln 11.18m: J50°, ro, un, cln 11.54m: J85°, ro, pl, cly 11.81m: J85°, ro, un, cln	C	90	65	PL(A) = 0.5 PL(A) = 0.2
85 84 83	12 12.0 13 14	LAMINITE/SANDSTONE - medium then high strength, slightly weathered then fresh, slightly fractured, light grey to grey laminite/medium grained sandstone	12.35m: J65°, he, fe 12.59m: B0°- 5°, Cz, 20mm 12.85m: J45°, ro, pl, cly 13.19m: B0°, cly, 20mm & J80°- 90°, ro, cu, cln 13.52m: J35°, ro, pl, cln 13.6m: B5°, cly, 2mm 14.35m: J45°, ro, un, cln 14.54m: B0°, cly, 5mm 14.64m: J50°, sm, pl, cln 14.8m: J45°- 85°, ro, cu, cln 14.95m: J25°, ro, pl, cln	C	100	81	PL(A) = 0.5 PL(A) = 1.2 PL(A) = 1.2
82 81 80 79 78	15 15.0 16 17 18 19	Bore discontinued at 15.0m target depth reached					

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

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.0m; NMLC-Coring to 15.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 15.0m (Screen 3.0-15.0m; Gravel 2.5-15.0m; Bentonite 1.5-2.5m; Backfill to GL)

SAMPLING & IN SITU TESTING LEGEND

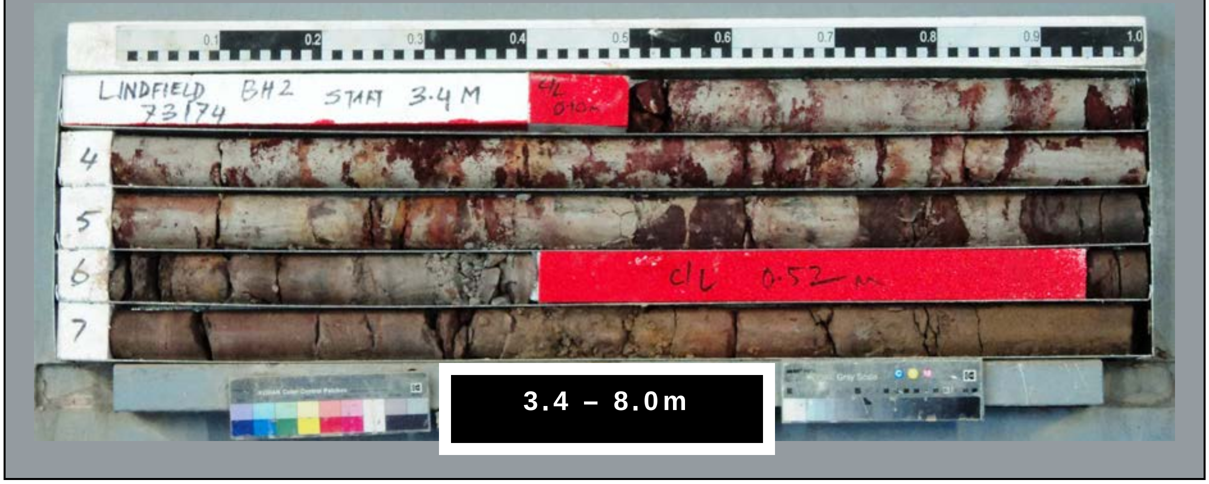
A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 2 PROJECT 73174 SEP 2012



3.4 – 8.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 2 PROJECT 73174 SEP 2012

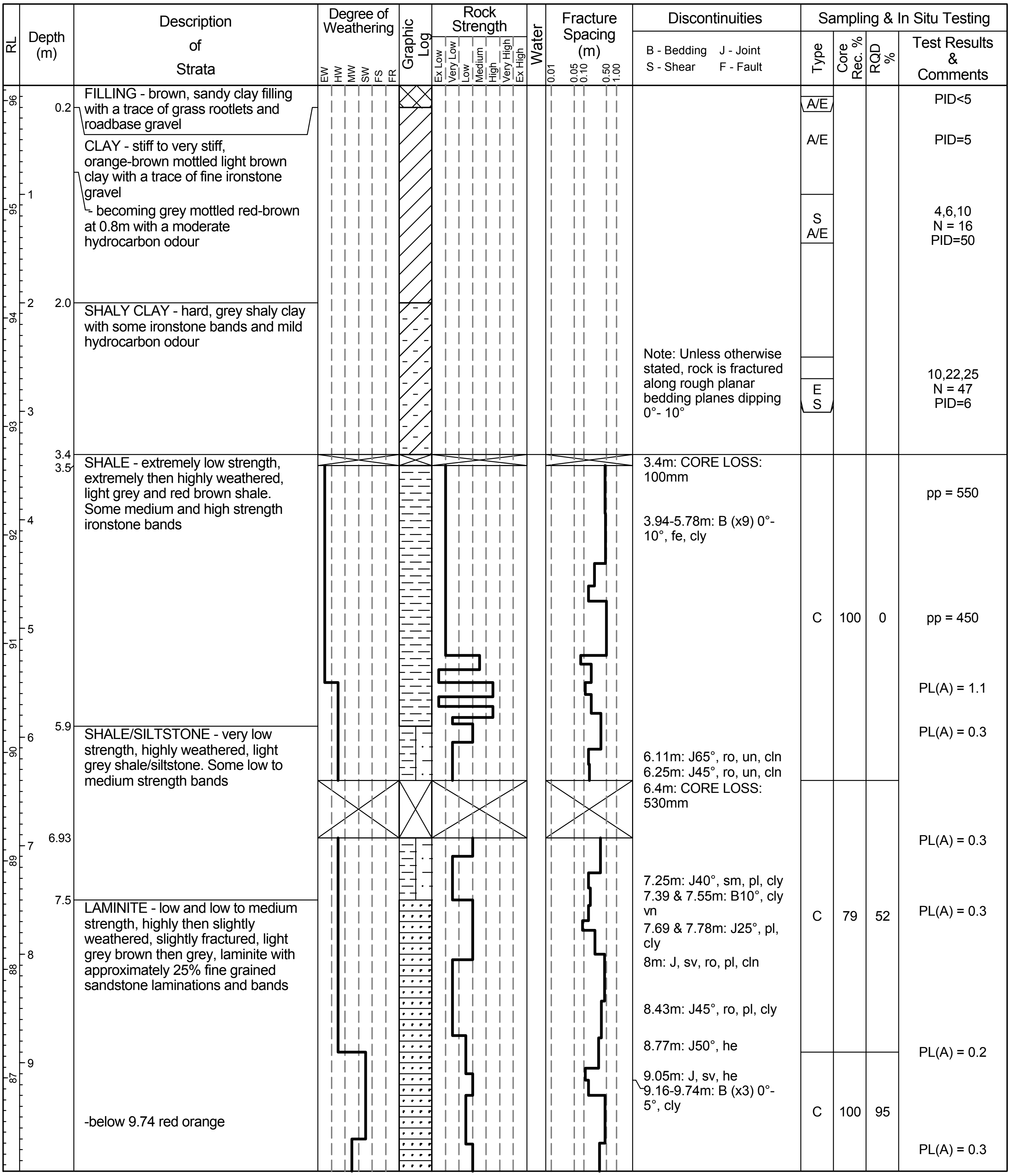
8.0 – 12.0m

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 96.14 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 73174
DATE: 14/9/2012
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.2	FILLING - brown, sandy clay filling with a trace of grass rootlets and roadbase gravel		A/E			PID<5
2.0	CLAY - stiff to very stiff, orange-brown mottled light brown clay with a trace of fine ironstone gravel - becoming grey mottled red-brown at 0.8m with a moderate hydrocarbon odour		A/E S A/E			PID=5 4,6,10 N = 16 PID=50
3.4	SHALY CLAY - hard, grey shaly clay with some ironstone bands and mild hydrocarbon odour	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes dipping 0°- 10°	E S			10,22,25 N = 47 PID=6
3.5		3.4m: CORE LOSS: 100mm				
5.9	SHALE - extremely low strength, extremely then highly weathered, light grey and red brown shale. Some medium and high strength ironstone bands	3.94-5.78m: B (x9) 0°- 10°, fe, cly	C	100	0	pp = 550 pp = 450 PL(A) = 1.1
6.93	SHALE/SILTSTONE - very low strength, highly weathered, light grey shale/siltstone. Some low to medium strength bands	6.11m: J65°, ro, un, cln 6.25m: J45°, ro, un, cln 6.4m: CORE LOSS: 530mm				PL(A) = 0.3
7.5		7.25m: J40°, sm, pl, cly	C	79	52	PL(A) = 0.3
	LAMINITE - low and low to medium strength, highly then slightly weathered, slightly fractured, light grey brown then grey, laminite with approximately 25% fine grained sandstone laminations and bands	7.39 & 7.55m: B10°, cly vn 7.69 & 7.78m: J25°, pl, cly 8m: J, sv, ro, pl, cln 8.43m: J45°, ro, pl, cly 8.77m: J50°, he				PL(A) = 0.3 PL(A) = 0.2
	-below 9.74 red orange	9.05m: J, sv, he 9.16-9.74m: B (x3) 0°- 5°, cly	C	100	95	PL(A) = 0.3

RIG: DT 100 **DRILLER:** SY **LOGGED:** SI **CASING:** HW 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	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

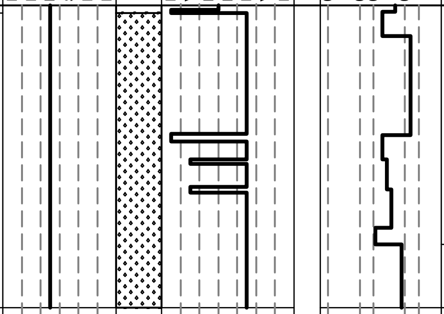
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 96.14 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 73174
DATE: 14/9/2012
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, F - Fault)	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
86 85	10.05 11	SANDSTONE - high strength, moderately weathered, slightly fractured, brown, medium grained sandstone						10.04m: B0°, cly, 20mm 10.2m: J30°, ro, pl, fe 10.86m: B0°, Ds, 40mm 11.02m: B0°, Ds, 30mm 11.22m: B0°, Cz, 10mm 11.47m: B5°, fe, cly vn 11.58m: J25°, ro, pl, cln	C	100	95	PL(A) = 1.2 PL(A) = 2
84 83 82 81 80 79 78 77	12 12.0 13 14 15 16 17 18 19	Bore discontinued at 12.0m target depth reached										

RIG: DT 100 **DRILLER:** SY **LOGGED:** SI **CASING:** HW 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	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

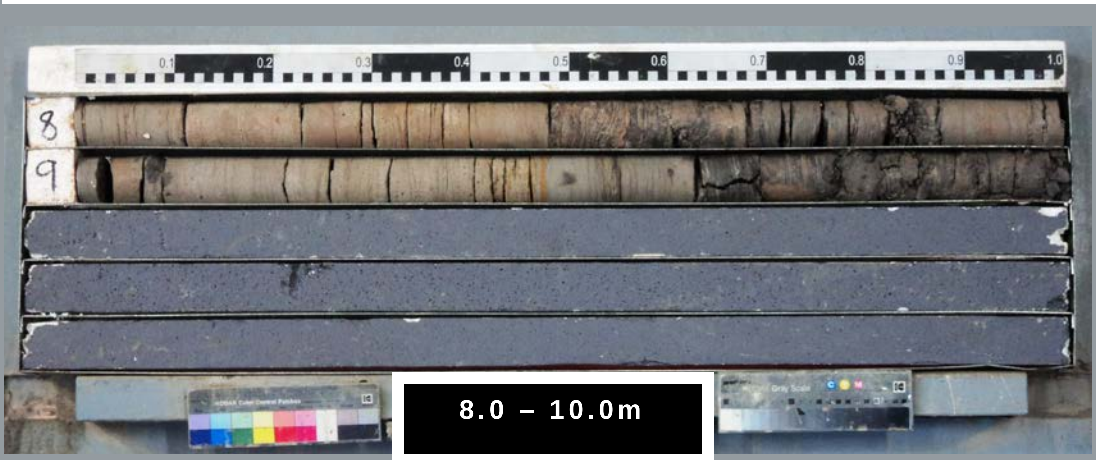
BORE 3 PROJECT 73174 SEP 2012

4.0 – 8.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 3 PROJECT 73174 SEP 2012



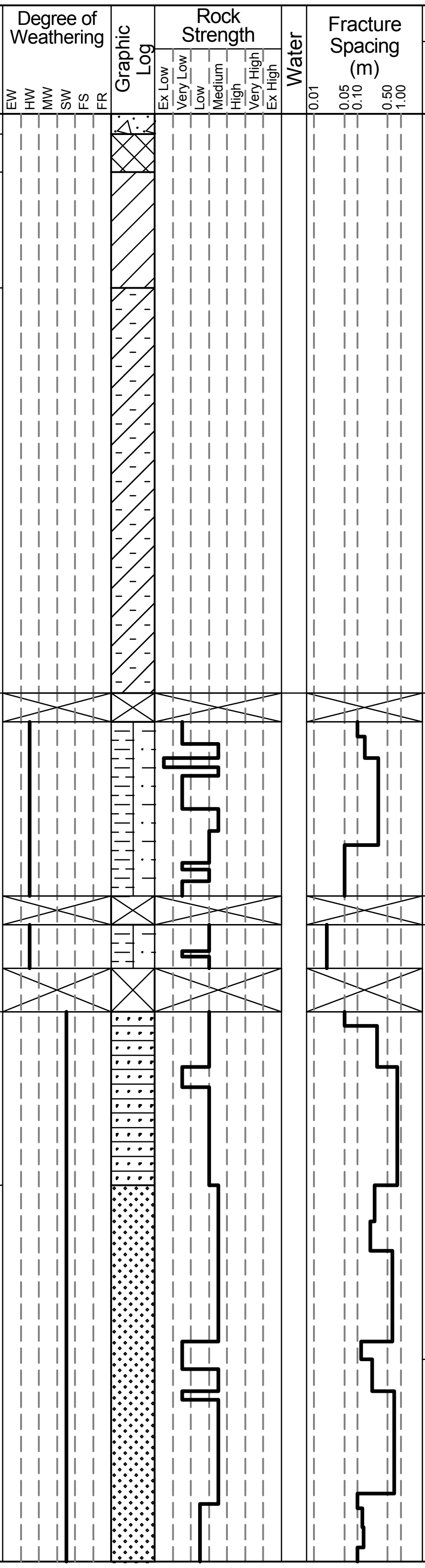
8.0 – 10.0m

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 95.10 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 73174
DATE: 19/9/2012
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	CONCRETE					
0.14	FILLING - light brown-grey, gravelly (concrete and sandstone) sand filling		A/E			PID<5
0.4	CLAY - very stiff, red-brown clay with a trace of ironstone gravel		A/E*			PID<5
1.2	SHALY CLAY - grey mottled red-brown, shaly clay	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	A			PID<5
4.0 4.2	SHALE - low to medium and medium strength, highly weathered, highly fractured to fractured and slightly fractured, light grey to grey brown, shale. Some very low strength bands	4m: CORE LOSS: 200mm 4.3m: J55°, ro, cu, cln 4.45m: B0°, fe, Ds, 60mm	C	60	0	
		4.75m: B10°, Cz, 50mm 5.05m: J30°, sm, pl, cln 5.05-5.4m: B (x7) 0°- 5°, fe, cly	C	100	40	PL(A) = 0.9
5.6		5.4m: CORE LOSS: 200mm 5.6m: J, sv, ro, pl, fe 5.9m: CORE LOSS: 300mm	C	44	0	PL(A) = 0.3
6.2	LAMINITE - low to medium strength, slightly weathered, slightly fractured, grey laminite with approximately 30% fine grained sandstone laminations	6.3m: J30°, sm, pl, cln 6.58-6.62m: Ds, 40mm	C	100	84	PL(A) = 0.3
7.4	SANDSTONE - medium strength, slightly weathered, slightly fractured and fractured, light grey, medium grained sandstone with some low and very low strength laminite bands	7.4-7.82m: B (x3) 0°- 5°, cly, 5-10mm	C	100	100	PL(A) = 0.4
		8.48m: B0°, cly co 8.6m: J30°, sm, pl, cly 8.82-8.87m: Cz, 50mm	C	100	74	PL(A) = 0.6
		9.63m: J, sv, ro, un, cln 9.77-9.82m: Ds, 50mm 9.9m: J90°, ro, un, cln	C	100	100	PL(A) = 0.6
10.0	Bore discontinued at 10.0m					

RIG: Terrier **DRILLER:** SY **LOGGED:** SI **CASING:** PVC to 1.0m

TYPE OF BORING: Solid flight auger to 1.0m; Rotary to 4.0m; NMLC-Coring to 10.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: *Duplicate sample 1/17.9.12 & Duplicate 2/17.9.12 collected

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 4　　PROJECT 73174　　SEP 2012

2.8 – 7.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 4　　PROJECT 73174　　SEP 2012

7.0 – 9.0m

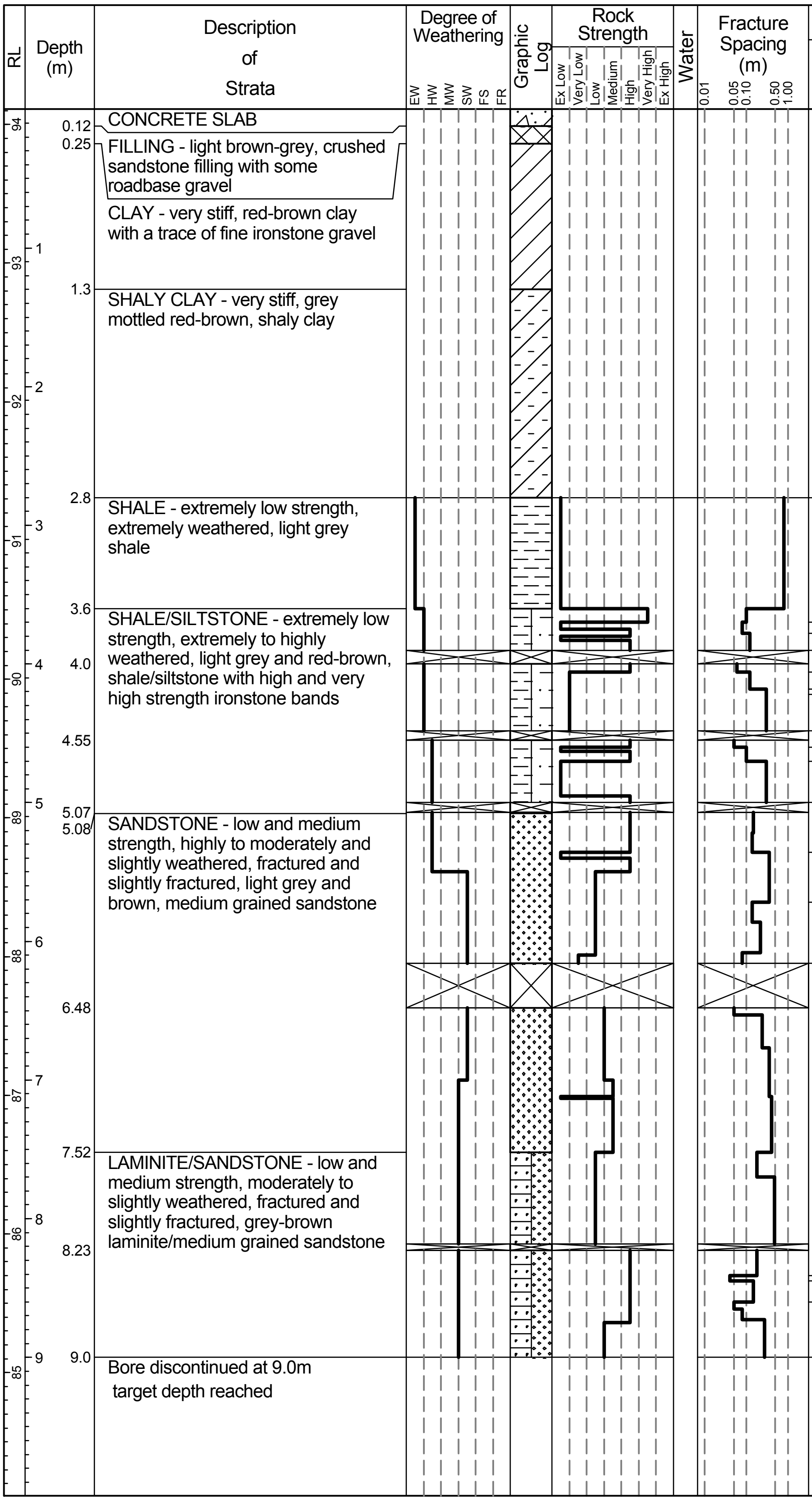
BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 94.11 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 73174
DATE: 17 - 18/9/2012
SHEET 1 OF 1

Depth (m)	Description of Strata
0.12	CONCRETE SLAB
0.25	FILLING - light brown-grey, crushed sandstone filling with some roadbase gravel
	CLAY - very stiff, red-brown clay with a trace of fine ironstone gravel
1.3	SHALY CLAY - very stiff, grey mottled red-brown, shaly clay
2.8	SHALE - extremely low strength, extremely weathered, light grey shale
3.6	SHALE/SILTSTONE - extremely low strength, extremely to highly weathered, light grey and red-brown, shale/siltstone with high and very high strength ironstone bands
4.0	
4.55	
5.07 5.08	SANDSTONE - low and medium strength, highly to moderately and slightly weathered, fractured and slightly fractured, light grey and brown, medium grained sandstone
6.48	
7.52	LAMINITE/SANDSTONE - low and medium strength, moderately to slightly weathered, fractured and slightly fractured, grey-brown laminite/medium grained sandstone
8.23	
9.0	Bore discontinued at 9.0m target depth reached



Discontinuities

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

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

- 3.6m: B5°, cly co
- 3.7m: B0°, fe, Ds, 40mm
- 3.78m: B5°- 10°, fe, cly, 20mm
- 3.9m: CORE LOSS: 100mm
- 4.06m: B0°- 5°, Ds, 70mm
- 4.18m: B/J25°, ro, un, fe
- 4.22-4.35m: Ds, 130mm
- 4.48m: CORE LOSS: 70mm
- 4.6m: J30°, ro, pl, fe, cly
- 4.7m: B0°, cly co
- 5m: CORE LOSS: 70mm
- 5.36m: B0°, Ds, 50mm
- 5.72-6.08m: B (x3) 0°-10°, cly vn
- 6.16m: CORE LOSS: 320mm
- 6.53 & 6.77m: B0°- 5°, cly, 5-10mm
- 7.12m: B0°, cly, 5mm
- 7.52m: B0°, Ds, 20mm
- 7.7m: J70°, ro, un, cly
- 8.18m: CORE LOSS: 50mm
- 8.41m: B15°, cly vn
- 8.45m: J35°, pl, ro, fe
- 8.6-8.73m: B (x3) 10°

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A/E			PID<5
A/E			PID<5
A			
A/E			PID<5
A			
			pp = 500
C	100	0	
			pp = 350
			PL(A) = 3
C	89	0	
C	84	32	PL(A) = 3.1
			PL(A) = 2
			PL(A) = 0.2
C	71	50	
			PL(A) = 0.4
C	100	99	
			PL(A) = 0.5
C	95	84	PL(A) = 0.2
			PL(A) = 1.2
C	100	74	
			PL(A) = 0.3

RIG: Terrier **DRILLER:** SY **LOGGED:** SI **CASING:** PVC to 1.5m

TYPE OF BORING: Diatube to 0.12m; Solid flight auger to 2.7m; Rotary to 2.8m; NMLC-Coring to 9.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 9.0m (Screen 3.0-9.0m; Gravel 2.5-9.0m; Bentonite 1.0-2.5m; Backfill to GL)

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 94.66 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 73174
DATE: 17/9/2012
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water	Well Construction Details
	0.12	CONCRETE SLAB						
	0.4	FILLING - grey-brown/red-brown, sandy clay filling with some basalt gravel	A/E	0.2				
		SILTY CLAY - red-brown, silty clay with a trace of ironstone gravel	A/E	0.6		PID<5		
			A	1.0				
	1.3	SHALY CLAY - grey mottled red-brown, shaly clay	A/E	1.7		PID<5		
			A/E	3.5		PID<5		
	3.7	Bore discontinued at 3.7m - auger (V-bit) refusal						

RIG: Dando **DRILLER:** SS **LOGGED:** SB **CASING:** Uncased
TYPE OF BORING: Diatube to 0.12m; Solid flight auger (V-bit) to 3.7m
WATER OBSERVATIONS: No free groundwater observed
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: ANKA (Civic Centre) Pty Ltd
PROJECT: Mixed-Use Retail & Residential Development
LOCATION: 23-37 Lindfield Avenue, Lindfield

SURFACE LEVEL: 94.66 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 73174
DATE: 17/9/2012
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Well Construction Details
				Type	Depth	Sample	Results & Comments		
	0.16	CONCRETE SLAB							
	0.3	FILLING - grey, gravelly sand filling		A/E	0.25		PID<5		
		CLAY - grey mottled red-brown clay with a trace of fine ironstone gravel - becoming shaly clay at 0.8m		A/E	1.0		PID<5		
				A/E	2.0		PID<5		
	2.5	Bore discontinued at 2.5m - auger (V-bit) refusal							

RIG: Dando **DRILLER:** SS **LOGGED:** SB **CASING:** Uncased
TYPE OF BORING: Diatube to 0.16m; Solid flight auger (V-bit) to 2.5m
WATER OBSERVATIONS: No free groundwater observed
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

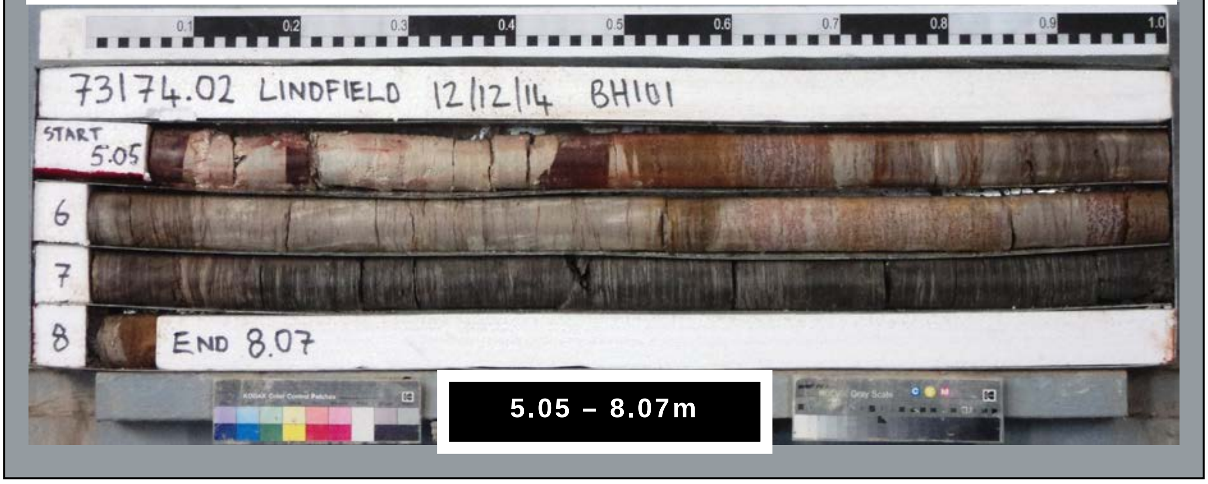
A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED MIXED-USE DEVELOPMENT - LINDFIELD

BORE 101 **PROJECT 73174.02** **DEC 2014**



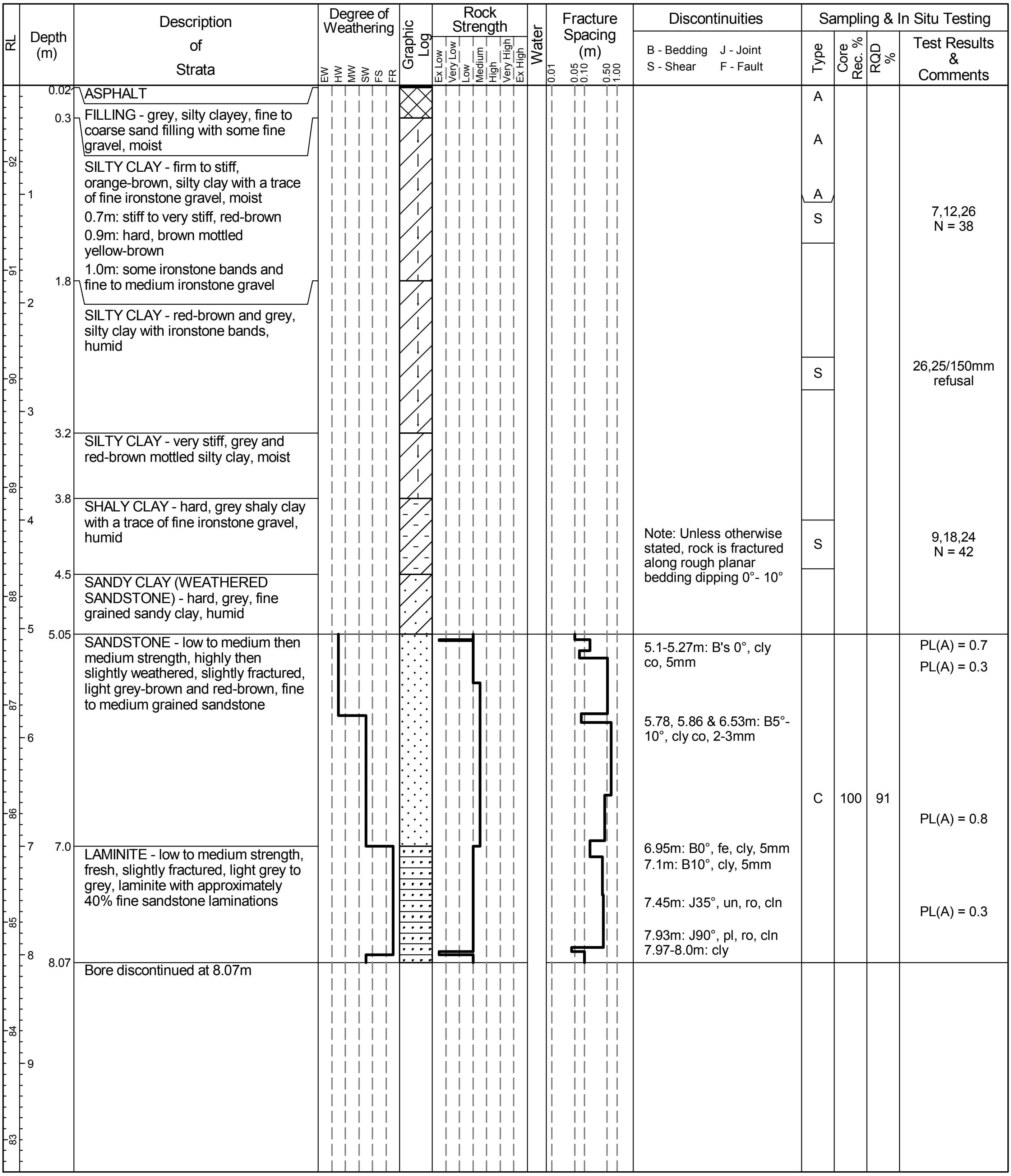
5.05 – 8.07m

BOREHOLE LOG

CLIENT: Aqualand Projects Pty Ltd
PROJECT: Proposed Mixed-Use Development
LOCATION: 23-41 Lindfield Avenue, Lindfield

SURFACE LEVEL: 92.7 AHD
EASTING: 330504
NORTHING: 6261394
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 73174.02/03
DATE: 12/12/2014
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALT		A			
0.3	FILLING - grey, silty clayey, fine to coarse sand filling with some fine gravel, moist		A			
	SILTY CLAY - firm to stiff, orange-brown, silty clay with a trace of fine ironstone gravel, moist 0.7m: stiff to very stiff, red-brown 0.9m: hard, brown mottled yellow-brown 1.0m: some ironstone bands and fine to medium ironstone gravel		A S			7,12,26 N = 38
1.8	SILTY CLAY - red-brown and grey, silty clay with ironstone bands, humid		S			26,25/150mm refusal
3.2	SILTY CLAY - very stiff, grey and red-brown mottled silty clay, moist					
3.8	SHALY CLAY - hard, grey shaly clay with a trace of fine ironstone gravel, humid	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			9,18,24 N = 42
4.5	SANDY CLAY (WEATHERED SANDSTONE) - hard, grey, fine grained sandy clay, humid					
5.05	SANDSTONE - low to medium then medium strength, highly then slightly weathered, slightly fractured, light grey-brown and red-brown, fine to medium grained sandstone	5.1-5.27m: B's 0°, cly co, 5mm 5.78, 5.86 & 6.53m: B5°-10°, cly co, 2-3mm	C	100	91	PL(A) = 0.7 PL(A) = 0.3 PL(A) = 0.8
7.0	LAMINITE - low to medium strength, fresh, slightly fractured, light grey to grey, laminite with approximately 40% fine sandstone laminations	6.95m: B0°, fe, cly, 5mm 7.1m: B10°, cly, 5mm 7.45m: J35°, un, ro, cln 7.93m: J90°, pl, ro, cln 7.97-8.0m: cly				PL(A) = 0.3
8.07	Bore discontinued at 8.07m					

RIG: MD200 **DRILLER:** ID **LOGGED:** KM/SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 5.0m; NMLC-Coring to 8.07m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

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Geotechnics / Environment / Groundwater

Appendix D

Laboratory Test Results

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 79038

Client:

Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Ray Blinman

Sample log in details:

Your Reference:	**73174, Lindfield**	
No. of samples:	17 soils	
Date samples received / completed instructions received	19/09/12 /	19/09/12

Analysis Details:

Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	26/09/12 /	26/09/12
Date of Preliminary Report:	Not issued	

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Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Nancy Zhang
Chemist

Rhian Morgan
Reporting Supervisor

Nick Sarlamis
Inorganics Supervisor

Alex MacLean
Chemist

Paul Ching
Approved Signatory

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil					
Our Reference:	UNITS	79038-4	79038-14	79038-15	79038-16
Your Reference	-------------	BH2	BH1	BH5	BH3
Depth	------------	2.7-2.9	1.0	3.5	2.0
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	21/09/2012	21/09/2012	21/09/2012	21/09/2012
Date analysed	-	21/09/2012	21/09/2012	21/09/2012	21/09/2012
pH 1:5 soil:water	pH Units	4.3	4.3	4.3	4.5
Electrical Conductivity 1:5 soil:water	µS/cm	43	96	130	52
Chloride, Cl 1:5 soil:water	mg/kg	6	6	9	4
Sulphate, SO4 1:5 soil:water	mg/kg	27	63	190	48