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Ku-ring-gai Council

Proposed Mixed Use Development Lindfield Community Hub Project

Report on Geotechnical Investigation

3914-R1-Rev2
12 December 2016

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Lindfield Community Hub Project

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Prepared for Ku-ring-gai Council

Prepared by Asset Geotechnical Engineering Pty Ltd

3914-R1-Rev2
12 December 2016

For and on behalf of
Asset Geotechnical Engineering Pty Ltd

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1. INTRODUCTION

1.1 General

This report presents the results of a geotechnical investigation for the above project. The investigation was commissioned on 24 August 2016 by Anthony Fabbro of Ku-ring-gai Council. The work was carried out in accordance with the proposal by Asset Geotechnical Engineering Pty Ltd dated 18 August 2016, reference 3914-P1.

Drawings supplied to us for this investigation comprised:

- Plan showing details and survey levels for the proposed hub by William L. Backhouse Surveyors (reference number CH5525.001, dated 17 November 2015).
- SJB Architects, Lindfield Hub Masterplan, Basement Drawings ((preferred scheme), dated 17/11/16, Rev2.
- SJB Architects, Lindfield Hub Masterplan, 5.1.6 Axonometric View of Basements

Based on the supplied drawings, we understand that the proposed project will be a mixed-use development which will comprise of a council-owned community hub building and a public plaza, a public park, a retail floor space and 95 residential apartments, in addition to construction of a new public street between Bent Street and Beaconsfield Parade, upgrade works to Bent Lane, Bent Street, Woodford Lane and Beaconsfield Parade and 598 car parking spaces. The public, commuter and residential car park is planned to be within three basement levels to 77.3m AHD in the preferred masterplan option.

The residential parking will locally extend to a fourth level of 74.2m AHD.

1.2 Scope of Work

The main objectives of the investigation were to assess the surface and subsurface conditions and to provide comments and recommendations relating to:

- Key geotechnical constraints to the development.
- Basement excavation conditions, methodology and monitoring.
- Maximum allowable temporary and permanent batter slopes.
- Subgrade preparation and earthworks.
- Site Classification to AS2870–2011 “Residential Slabs and Footings”
- Suitable footing systems and geotechnical design parameters for the footing systems.
- Excavation support methodology and design parameters.
- Settlement.
- Groundwater levels and dewatering requirements.
- Soil aggressiveness and corrosion potential.
- Pavement thickness design.
- Potential construction problems for the development and mitigation measures.

In order to achieve the project objectives, the following scope of work was carried out:

- A review of existing regional maps and reports relevant to the site, held within our files.
- Clearance of underground services at proposed test locations.
- Visual observations of surface features.
- Subsurface investigation at eight locations to sample and assess the nature and consistency of subsurface soils and bedrock at accessible areas of the site.

- Laboratory tests on the recovered soil and rock samples to provide engineering data.
- Engineering assessment and reporting.

This report must be read in conjunction with the attached “Important Information about your Geotechnical Report” in Appendix A. Particular attention is drawn to the limitations inherent in site investigations and the importance of verifying the subsurface conditions inferred herein.

2. SITE DESCRIPTION

The site is located west of the Pacific Highway, as shown in Figure 1. The site is bounded by Bent Street to the north, Beaconsfield Parade to the south, Woodford Lane to the east and residential properties to the west.

Topographically, the site is located on the west facing downslope of a region with undulating terrain, with Pacific Highway forming the ridgeline of the slope. The car park is situated on a moderately steep gradient, with slopes ranging between 5º to 10º. Along Bent Street, the slope attains a more gradual transition as it proceeds towards the west. Ground levels typically vary from 87m AHD to 96m AHD at this site.

At the time of investigation, the site was occupied by the council owned car-park serving the strip mall facing Pacific Highway. The properties facing Bent Street are single and double storey brick residential structures, although the structures on 8-12 Bent Street have been demolished.

Site drainage is expected to be by natural run off in the car park, towards the lower elevation region situated at the southwest. Natural seepage into native soils are also expected in the residential properties.

Vegetation was mostly trees and shrubs associated with landscaping within the existing residential properties. Gum trees were observed towards the west of the parking lot, adjacent to Drovers Way. No visible rock outcrops were observed. Review of Ku-ring-gai Council’s acid sulfate soils and salinity overlay indicated that there is no evidence of either potential acid sulfate soils or salinity issues.

3. FIELDWORK & LABORATORY TESTING

3.1 Borehole Investigation

The fieldwork was undertaken on 21, 22 and 26 September 2016 under the full time supervision of a Geotechnical Engineer from Asset, and included subsurface investigation at eight locations.

Buried metallic services and utilities within the site boundaries in the vicinity of the proposed test locations were cleared by an accredited subcontractor and by referring to DBYD utility maps.

Five boreholes (BH1 to BH5) were augered to refusal on bedrock, after which they were continued via NMLC rock coring to target depths of 15.0m. Three hand auger holes (BH6 to BH8) were also performed in the residential properties, where no vehicular access was possible. The hand augers were supplemented by Dynamic Cone Penetrometer (DCP) testing to aid in the assessment of in situ soils. The test locations are shown on the attached Figure 2. Engineering logs are provided in Appendix B together with their explanatory notes.

The test locations were set out by our Geotechnical Engineer by measurements relative to existing site features. The subsurface conditions encountered were logged during drilling. Soil and rock samples were retained for laboratory testing. Surface levels at the test locations were estimated by interpolation from levels shown on the survey plan provided (drawing reference: CH5525.001, dated 17 November 2015).

On completion of logging and sampling, each borehole was backfilled with the drilling spoil. Remaining spoil was left and trimmed neatly flush. Two piezometers were also installed in two of the boreholes (BH2 and BH5), for future monitoring of ground water.

3.2 Laboratory Testing

Soil and rock samples recovered during the fieldwork were delivered to a NATA registered laboratory. Standard compaction and soaked California Bearing Ratio (CBR) tests were performed for the bulk soil samples. Point load strength index testing of rock core was performed on selected rock core from all five boreholes.

Test results are attached. Testing was carried out generally in accordance with AS1289 “Methods of Testing Soil for Engineering Purposes” or as described in the laboratory test results.

4. SUBSURFACE CONDITIONS

4.1 Geology

The 1:100,000 Sydney Geological Map indicates the site is underlain by Ashfield Shale, with Hawkesbury Sandstone at deeper elevations. Interbedded layers of shale and sandstone are to be expected towards the interface of the two strata.

4.2 Subsurface Conditions

A generalised geotechnical model for the site has been developed is shown in Table 1, with a summary of the subsurface the conditions observed at each test location shown in Table 2. For a detailed description of the subsurface conditions, refer the attached engineering logs and explanatory notes. For specific design input, reference should be made to the logs and/or the specific test results, in place of the following summary.

Table 1 - Generalised Site Geotechnical Model

Layer	Origin	Description	Soil Density / Consistency	Assessed Rock Classification[1]
Unit 1	Topsoil	CLAY/ SILTY CLAY, low plasticity, dark brown, traces of fine grained sand, traces of subangular gravel and root fibres.	--	--
Unit 2	Fill	CLAY, low to medium plasticity, dark brown to grey, traces of fine grained sand and subangular gravel, some traces of brick fragments, traces of ironstone inclusions.	--	--
Unit 3	Residual Soil	CLAY, high plasticity, orange brown to grey mottled red, traces of subangular gravel and ironstone inclusions.	Stiff to hard	--
Unit 4	Bedrock	SHALE, extremely weathered, extremely weak to very low strength, logged as clay, brown to red brown to grey, very thinly bedded.	--	Class 5 Shale
Unit 5	Bedrock	Alternating layers of SHALE and SANDSTONE, fine grained, highly to moderately weathered, low to high strength, grey to dark grey with traces of iron stains, poorly developed.	--	Class 4/3 Shale/ Sandstone
Unit 6	Bedrock	Alternating layers of SANDSTONE and SHALE, fine to medium grained, grey, moderately to slightly weathered, medium to very high strength, poorly developed.	--	Class 2 Sandstone /Shale

Table 2 – Generalised Subsurface Conditions (depth to top of stratum, m) [thickness, m]

Layer	Origin	BH1	BH2	BH3	BH4	BH5	BH6	BH7	BH8
	GL (m AHD)	88.0	95.3	95.0	92.0	93.4	92.0	89.0	90.0
Unit 1	Topsoil	0.0	0.0	--	0.0	--	0.0	0.0	0.0
Unit 2	Fill	0.3	--	--	--	0.2	0.4	--	--
Unit 3	Residual Soil	1.2	0.2	0.1	0.3	2.1	0.8	0.2	0.3
Unit 4	Bedrock	1.8 [3.4]	1.4 [6.2]	1.9 [3.5]	3.2 [4.5]	2.5 [6.0]	1.8	2.0	2.5
Unit 5	Bedrock	5.2 [6.0]	7.6 [5.0]	6.4 [2.6]	7.7 [4.7]	8.5 [3.2]	--	--	--
Unit 6	Bedrock	11.2 [>3.8]	13.6 [>2.8]	9.0 [>5.82]	12.45 [>2.8]	11.7 [>3.0]	--	--	--
EOH		15.0	16.4	14.82	15.27	14.74	1.8	2.0	2.5

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

+ Denotes end of hole

Special Note for DCP testing

Particular caution must be used when inferring subsurface conditions from DCP results. Refusal can be encountered on obstructions such as gravel, rock floaters, or other inclusions within a soil mass. Also, the DCP results in clay soils are significantly affected by the insitu moisture content. It is therefore strongly recommended that an experienced Geotechnical Engineer be engaged to confirm the inferred subsurface conditions during construction, and to provide advice where subsurface conditions are significantly different.

4.3 Groundwater

No free groundwater was observed during the drilling. Monitoring wells were installed in boreholes BH2 and BH5. Readings were taken on 29 September 2016. It was observed that in BH2 at the north of the site, the groundwater level was at RL 88.5m AHD (6.8m depth). In BH5 at the south of the site, the groundwater level was observed at RL 89.5m AHD (3.9m depth). In both cases the groundwater is associated with the weathered shale bedrock.

4.4 Laboratory Test Results

Results from the laboratory testing undertaken on selected soil samples are included in Appendix C, and are summarised in Table 3.

Table 3 – Laboratory Test Results

Test Location & Depth	CBR (%)		Moisture Content (%)	
	@ 2.5mm penetration	@ 5mm penetration	Field Moisture	Standard Optimum
BH3 (0.1m – 6.0m)	--	2.5	20.3	20.0
BH4 (0.0m – 7.5m)	--	4.5	19.9	20.0
BH5 (0.12m – 8.5m)	--	6.0	19.0	19.0
BH7a (0.0m – 2.0m)	3.5	--	21.8	23.0

5. DISCUSSIONS & RECOMMENDATIONS

The public, residential and commuter car park is planned to be within three basement levels to 77.3m AHD in the preferred masterplan option with an effective ground (“supermarket”) level of 88.9m AHD, below the existing ground level of approximately 94mAHD. Groundwater is anticipated to be 11m to 12m above the lowest basement level.

Residential parking will locally extend to a fourth level of 74.2m AHD with a surface “park” level of 92m AHD. Based on the anticipated lowest basement finished floor level and from the results of this investigation, it is assessed that this basement level will be about 14m to 15m below the observed groundwater level, and would be within interbedded sandstone and shale bedrock.

Key geotechnical constraints to the development include groundwater control (during construction and long-term), temporary shoring, permanent retaining, and foundation conditions.

Recommendations for design and construction of the development are provided in the following sections. The presence of groundwater at 3.9m to 6.8m depth, average 5.4m depth (RL 89m AHD) towards the centre of the site (i.e. up to 15m above the lowest basement level) and the depth to weathered bedrock (between

1.4m and 3.2m) and for the less weathered stronger rock (between 5.2m to 8.5m depth) will need to be carefully considered with respect to design and construction sequencing of the development.

5.1 Construction Sequence

The following construction sequence is suggested for the basement level for the development:

1. Demolish existing buildings (on 2 Bent Street, 4 Bent Street and 6 Bent Street) and clear landscaping.
2. Remove existing pavements in the parking lot and concrete slabs.
3. Install temporary shoring around the basement perimeter with temporary anchors as required.
4. We have been advised that in addition to maintaining vehicular access to Bent St, Drovers Way and Beaconsfield Pde, access will be required to be provided to Woodford Lane during the course of construction to provided parking and delivery access to the rear of existing commercial allotments along Pacific Highway. Therefore, surcharge from such live loading should be allowed for in design of the proposed excavation support.
5. Install temporary dewatering system (external or internal to basement)
6. Excavate to bulk excavation level.
7. Install pad footings for internal column loads.
8. Carry out detail excavations (e.g. for lift pits) – additional localised dewatering may be required.
9. Construct lower basement ground floor.
10. Pour lower basement roof and continue up to existing ground surface level to provide permanent support to the excavation.
11. Decommission temporary dewatering system.

5.2 Temporary Shoring

It is understood that permanent batter slopes are not proposed for the development. The proposed depth of excavation, the presence of groundwater, and the lack of clearance between the basement and boundary would preclude temporary batters, and therefore temporary shoring will be required. Depending on the design of the shoring, it could also be incorporated into the permanent foundation and retaining works.

A number of possible shoring systems could be considered for the site. These are summarised in Table 4 together with a brief description of the advantages and disadvantages of each.

Table 4 – Summary of Shoring Options

Option	Method	Advantages	Disadvantages
1	Conventional shoring with soldier piles and steel walers, or soldier piles and shotcrete infill panels	Relatively low cost	Risk of instability and loss of ground unless adequate external dewatering is provided. Forms a poor seal against groundwater. Greater amount of dewatering required. Potential draw down of groundwater levels outside of the site with possible adverse effects on adjacent structures.
2	Steel sheet pile (driven or hydraulically installed)	Rapid installation. Lower cost that Option 3. Low permeability water barrier.	Vibration may not be acceptable for adjoining developments. Penetration into rock would be limited.

		Amenable to joint caulking.	Permanent wall required. Will require soil anchors.
3a **or** **3b**	Contiguous or Secant bored piles	Can form part of the permanent structure. Minimum noise and vibration. Can maximise site building space as no temporary wall is required. Permanent water proofing can be incorporated. Low permeability water barrier (secant piling very low permeability compared to contiguous piling)	For secant piles, ensuring complete contact of all piles over full pile length may be difficult. Additional finishing may be required following excavation if a 'smooth' internal wall is required. Relatively high cost. May require soil anchors along boundaries where high level footings are located. Contiguous piles may require additional waterproofing where close contact not achieved.
4	Cutter Soil Mix (CSM) or Diaphragm wall	Practically impervious. Can be used as a permanent wall. Minimise settlement and ground disturbance of adjacent ground and properties.	Expensive. Close supervision of contractors required. May require soil anchors along boundaries where high level footings are located.

Based on the advantages and disadvantages listed in Table 4, we recommend a contiguous (Option 3a) pile wall retention system for the basement excavation. Subject to further ground investigation to greater depths with insitu measurement of groundwater inflow rates, the use of secant piles may become necessary if high water flows become apparent. Option 1 may be an acceptable solution as long as the construction is carefully monitored by the Project Geotechnical Engineer. Drainage to the back of the shotcrete panels will be essential. Option 2 is not likely to be suitable due to the depth of excavation support and adjacent structures. Option 4, Diaphragm wall option, may be too expensive for the scale of the project, however, could still be considered.

The founding depth of the retaining wall piles is a function of the required socket depth to achieve adequate embedment to resist overturning, the required load carrying capacity if the piles are to be incorporated into the permanent works, and the effect on reducing dewatering requirements by socketing into bedrock.

Assessed Class 4/3 shale and sandstone bedrock was encountered at depths between 5.2m to 8.5m depth and assessed Class 2 sandstone and shale was encountered below about 9m to 13.6m depth. Practically, it may be difficult to achieve a substantial socket within the assessed Class 2 bedrock. However, adequate overturning resistance is likely to be achieved within the assessed Class 5 to 4 sandstone and shale rock and the overlying very stiff to hard clays. Control of lateral deflections will also need to be considered (e.g. along the northern boundary facing Bent Street and the southern boundary facing Beaconsfield Parade). Temporary rock anchors are likely to be required.

Design of temporary shoring for carrying vertical loading should be in accordance with Section 5.7, and for lateral pressures it should be in accordance with Section 5.8.

Detailed construction supervision, monitoring and inspections will be required during the piling and subsequent bulk excavation to ensure an adequate standard of workmanship and to minimise potential problems. Regular inspection of the rock cuts as exposed by a competent Geotechnical Engineer will be required to manage the stability of the rock and control of groundwater.

5.3 Dewatering

In order to construct the basement, it will be necessary to dewater to about 0.5m below the proposed bulk excavation level. This would require lowering of the groundwater table to about 12m to 16m below the groundwater level observed for this study. Where lift pits or other local excavations extend below the bulk excavation level, it would be necessary to locally dewater not less than about 0.5m below such local excavations.

Contiguous or secant piles to bedrock together with internal dewatering would minimise the potential risks and should provide adequate control of groundwater for construction purposes. The development should be designed to minimise the risk of settlement induced by groundwater lowering by designing the basement structure as a "tanked" excavation (i.e. with impermeable retaining walls and floor structure). Permanent dewatering is not recommended.

The quantity of seepage expected to flow into the excavation during construction is unknown, but based on previous experience should not be significant. It will depend on the in situ permeability of the soils, the jointing / fracturing of the underlying bedrock, the flow path length, and the type and adequacy of construction of the temporary shoring adopted (e.g. contiguous versus secant piling). At this stage no in situ or laboratory permeability tests of the site subsurface profile has been undertaken. However, based on the borehole soil description of the sandy clays and with reference to empirical charts, we anticipate that the permeability of the sandy clays would be in the order of 10^{-5} to 10^{-7} cm/sec. The mass permeability of the underlying bedrock could be of a similar order to the soils.

Depending on seepage flows/water levels at the time of construction and the type of retention system constructed, we expect that dewatering by internal sumps and pumps would likely be sufficient to permit excavation works.

A temporary dewatering licence may be required from the NSW DPI Water, under Part 5 of the Water Act 1912. As part of the application process, a Dewatering Management Plan (DMP) will be required. This should be prepared by an experienced and qualified hydrogeologist, and should include an assessment of:

- dewatering volumes,
- impact on other groundwater users,
- drawdown effects,
- discharge water quality criteria and anticipated treatment requirements, and
- groundwater quality

5.4 Earthworks

5.4.1 Excavation

The excavation for the proposed development is anticipated to be partially within soils, and mostly within shale and sandstone bedrock. Excavation within the soils and extremely weathered bedrock would be achievable using conventional earthmoving equipment (i.e. hydraulic excavator bucket).

Excavation within the deeper less weathered bedrock will likely require use of ripper tooth fitted to a hydraulic excavator bucket, a dozer fitted with ripper tooth, or a hydraulic hammer fitted to an excavator, possibly supplemented by rock saw and rock splitting techniques.

5.4.2 Vibration Management

Australian Standard AS 2187: Part 2-2006 recommends the frequency dependent guideline values and assessment methods given in BS 7385 Part 2-1993 “Evaluation and measurement for vibration in buildings Part 2” as they “are applicable to Australian conditions”. The standard sets guide values for building vibration based on the lowest vibration levels above which damage has been credibly demonstrated. These levels are judged to give a minimum risk of vibration-induced damage, where minimal risk for a named effect is usually taken as a 95% probability of no effect.

Sources of vibration that are considered in the standard include demolition, blasting (carried out during mineral extraction or construction excavation), piling, ground treatments (e.g. compaction), construction equipment, tunnelling, road and rail traffic and industrial machinery.

For residential structures, BS 7385 recommends vibration criteria of 7.5 mm/s to 10 mm/s for frequencies between 4 Hz and 15 Hz, and 10 mm/s to 25 mm/s for frequencies between 15 Hz to 40 Hz and above. These values would normally be applicable for new residential structures or residential structures in good condition. Higher values would normally apply to commercial structures, and more conservative criteria would normally apply to heritage structures.

However, structures can withstand vibration levels significantly higher than those required to maintain comfort for their occupants. Human comfort is therefore likely to be the critical factor in vibration management.

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

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

Table 5 – Recommendations for Rock Breaking Equipment

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

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

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

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

It is pointed out that the rock classification system used in Table 1 is intended primarily for use in design of foundations, and is not intended to be used to directly assess rock excavation characteristics. Excavation contractors should refer to the detailed engineering logs, core photographs, laboratory strength tests, and inspection of rock core, and should not rely solely on the rock classifications presented in geotechnical engineering reports, when assessing the suitability of their excavation equipment for the proposed development. Further geotechnical advice must be sought if rock excavation characteristics are critical to the proposed development.

It should be noted that vibrations that are below threshold levels for building damage may be experienced at adjoining developments. Rock excavation methodology should also take into account acceptable noise limits as per the “Interim Construction Noise Guideline” (NSW EPA).

5.4.3 Subgrade Preparation

The following general recommendations are provided for subgrade preparation for earthworks, pavements, slab-on-ground construction, and minor structures:

- Strip existing fill and topsoil. Remove unsuitable materials from site (e.g. material containing deleterious matter). Stockpile remainder for re-use as landscaping material or remove from site.
- Excavate residual clayey soils and rock, stockpiling for re-use as engineered fill or remove to spoil. Rock could be stockpiled separately from clayey soils, for select use beneath pavements.
- Where rock is exposed in bulk excavation level beneath pavements, rip a further 150mm.
- Where rock is exposed at footing invert level, it should be free of loose, “drummy" and softened material before concrete is poured.
- Where soil is exposed at bulk excavation level, compact the upper 150mm depth to a dry density ratio (AS1289.5.4.1–2007) not less than 100% Standard.
- Areas which show visible heave under compaction equipment should be over-excavated a further 0.3m and replaced with approved fill compacted to a dry density ratio not less than 100%.

Further advice should be sought where filling is required to support major structures.

Any waste soils being removed from the site must be classified in accordance with current regulatory authority requirements to enable appropriate disposal to an appropriately licensed landfill facility. Further advice should be sought from a specialist environmental consultant, if required.

5.4.4 Filling

Where filing is required, place in horizontal layers over prepared subgrade and compact as per Table 6.

Table 6 – Compaction Specifications

Parameter	Cohesive Fill	Non Cohesive Fill
Fill layer thickness (loose measurement): • Within 1.5m of rear of retaining walls • Elsewhere	0.2m 0.3m	0.2m 0.3m
Density: • Beneath Pavements • Beneath Structures • Upper 150mm of subgrade	≥ 95% Std ≥ 98% Std ≥ 100% Std	≥ 70% ID ≥ 80% ID ≥ 80% ID
Moisture content during compaction	± 2% of optimum	Moist but not wet

Filling within 1.5m of the rear of any retaining walls should be compacted using light weight equipment (e.g. hand-operated plate compactor or ride-on compactor not more than 3 tonnes static weight) in order to limit compaction-induced lateral pressures.

Any soils to be imported onto the site for the purpose of back-filling and re-instatement of excavated areas should be free of contamination and deleterious material, and should include appropriate validation documentation in accordance with current regulatory authority requirements which confirms its suitability for the proposed land use. Further advice should be sought from a specialist environmental consultant if required.

5.4.5 Batter Slopes

Recommended maximum slopes for permanent and temporary batters are presented in Table 7.

Table 7 – Recommended Maximum Batter Slopes

Unit	Maximum Batter Slope (H : V)	
	Permanent	**Temporary**
Residual Clay & Shaley Clay	2 : 1	1 : 1
Class 5/4 Shale / Class 5 Sandstone	1.5 : 1	0.75 : 1
Class 3 or better Shale / Class 4 or better Sandstone	vertical *	vertical *

* subject to inspection by a Geotechnical Engineer and carrying out remedial works as recommended (e.g. shotcrete, rock bolting).

5.5 Site Classification

Due to the presence of trees, fill, and existing site structures (causing abnormal moisture conditions), the site is classified as a Class P (Problem) Site in accordance with AS 2870–2011 “Residential Slabs and Footings”. Footings should be designed as per the recommendations given in Section 5.7.

The above classification and footing recommendations are provided on the basis that the performance expectations set out in Appendix B of AS2870–2011 are acceptable and that future site maintenance is in accordance with CSIRO BTF 18, a copy of which is attached.

5.6 Salinity & Aggressivity

Whilst no specific laboratory testing has been carried out to assess the aggressiveness of soil to concrete and steel, based on the subsurface profile as described above and the site conditions, we consider that the soils would likely be non-saline and non-aggressive with respect to steel and concrete. Further testing would be required to confirm this.

5.7 Footings

Suitable footings might comprise a slab on ground for the basement area and pad footings supporting the upper building loads. It is recommended that all footings are founded on bedrock in order to reduce the risk of differential settlement due to variable founding conditions.

Edge beams for slab, pad footings and rock socketed piles may be designed for the parameters in Table 8.

Table 8 – Footing Design Parameters

Founding Stratum	Maximum Allowable (Serviceability) Values (kPa)			Ultimate Strength Limit State Values (kPa)		
	End Bearing	Shaft Friction – Compression	Shaft Friction – Tension	End Bearing	Shaft Friction - Compression	Shaft Friction – Tension*
Class 5 Shale	700	70	35	2,100	210	105
Class 4 Shale	1,000	100	50	4,500	150	100
Class 3 Shale	2,500	250	175	7,500	750	375
Class 2 Shale	4,000	400	200	12,000	1,200	600
Class 4 Sandstone	2,000	200	100	6,000	600	300
Class 3 Sandstone	3,500	350	175	10,500	1,050	550
Class 2/1 Sandstone	6,000	600	300	18,000	1,800	900

Note: Parameters for Class 4/5 Shale provided for strip and pad footings and bored piles only – these should not be used for CFA, CIS or Steel Screw piles.

* Uplift capacity of piles in tension loading should also be checked for inverted cone pull out mechanism.

In accordance with AS2159-2009 “Piling–Design and Installation”, for limit state design, the ultimate geotechnical pile capacity shall be multiplied by a geotechnical reduction factor (Φg). This factor is derived from an Average Risk Rating (ARR) which takes into account geotechnical uncertainties, redundancy of the foundation system, construction supervision, and the quantity and type of pile testing (if any). Where testing is undertaken, or more comprehensive ground investigation is carried out, it may be possible to adopt a larger Φg value that results in a more economical pile design. Further geotechnical advice will be required in consultation with the pile designer and piling contractor, to develop an appropriate Φg value.

Settlements for footings on rock are anticipated to be about 1% of the minimum footing dimension, based on serviceability parameters as per Table 8.

Options for piles include:

Bored Piles. It assessed that the construction of sockets would require the use of a truck mounted drilling rig. It is also assessed that the bored pile holes would not require liners to support the overburden soils, although some over break and minor fretting should be allowed for. Groundwater may be expected within bored pile holes and dewatering by down-hole pump may be required to limit softening of the bases prior to concreting.

Continuous Flight Auger (CFA) Piles. CFA piles are constructed by drilling a hollow stemmed continuous flight auger to the required founding depth. Concrete is then injected under pressure through the auger stem as the auger is extracted from the soil. The reinforcing cage is then inserted upon completion of the concreting process. Pile diameters vary from 300mm to 1200mm. Drilled spoil is produced during CFA piling, and must subsequently be removed from site. CFA piles are considered non-displacement piles as defined in AS2159. Examples of CFA piles are Frankipile “Atlas” type piles, or Vibropile “Omega” type piles. It is considered that CFA may not be able to penetrate through harder bands within Class 4/3 Sandstone to reach the Class 2 Sandstone.

Steel Screw Piles and Driven Piles. These are not considered appropriate to this development.

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

5.8 Excavation Support

Excavation of soil and rock results in stress changes in the remaining material, and some ground movement is inevitable. The magnitude and extent of lateral and vertical ground movements will depend on the design and construction of the excavation support system. Experience and published data suggest that lateral movements of an adequately designed and installed retention system in soil and weathered rock will typically be in the range of 0.2% to 0.5% of the retained height. The extent of the horizontal movement behind the excavation face typically varies from 1.5 to 3 times the excavated height.

5.8.1 Excavation Support Construction Methodology

Where temporary or permanent batter slopes as per Section 5.4.5 cannot be accommodated in the development or are not desired, temporary shoring and/or permanent retaining will be required.

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

In the long–term, the ground floor slab will provide bracing at the top of the wall and the garage floor slab will provide bracing at the bottom of the wall. Therefore, basement retaining walls should be designed as braced walls for the long–term loading condition.

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

Top–down construction typically involves:
- constructing the perimeter walls as either contiguous bored piles or cast-insitu wall (e.g. Geocast) and internal columns as bored piles;
- pouring the ground floor slab;
- excavating to subgrade level; and
- pouring the basement floor slab.

Bottom–up construction typically involves:
- constructing the perimeter wall as either contiguous bored piles, cast–insitu wall (e.g. Geocast), or conventional soldiers installed in concreted pile sockets;

- options for wall design include cantilever, anchored (“deadman”, soil, or rock anchors), and propped (internal props);
- excavating to basement subgrade level (installing horizontal walers and timber lagging if solider pile wall construction is adopted);
- pouring the ground floor slab and proceeding upwards.

If bottom-up construction is considered, we recommend the use of temporary anchored walls where the retained height is 3m or more, and cantilever walls where the retained height is less than 3m.

5.8.2 Excavation Support Design Parameters

Excavation support design can be relatively complex as it involves soil-structure interaction. Also, the pressures acting on the support will depend on a range of factors including the stiffness of the support, the construction sequence, external forces (e.g. surcharge loading), and varying groundwater conditions.

For relatively simple support systems (e.g. cantilever walls or anchored / propped walls with only one row of anchors / props, design may be based on an Earth Pressure Approach and using closed-form solutions or simple analytical programs such as WALLAP.

For more complex support systems (e.g. multiple anchors / props), or where it is desired to optimise the design, more advanced numerical analysis tools are recommended (e.g. 2D Finite Element Method), which include more complex soil models that allow for stress re-adjustment to occur with wall movements. The use of 3D FEM software may also be appropriate depending on the excavation geometry and potential cost-savings by optimising the support design.

EARTH PRESSURE APPROACH

Support systems designed using the Earth Pressure Approach may be based on the parameters given in Table 9.

Cantilever walls or walls within only a single row of anchors / props may be designed for a triangular earth pressure distribution with the lateral pressure being determined as follows:

$$\sigma_z = K_{o,a,p}\ z\ \gamma$$

where σ_z = lateral earth pressure (kPa) at depth z
$K_{o,a,p}$ = earth pressure coefficient
o = ‘at rest’, a = ‘active’, p = ‘passive’
z = depth (m)
γ = unit weight of soil / rock (kN/m^3)

Table 9 – Excavation Support Design Parameters (Earth Pressure Approach)

Material	Moist Unit Weight (γ_m) kN/m^3	'Active' Lateral Earth Pressure Coefficient (1) (K_a)	'At Rest' Coefficient (1) (K_o)	'Passive' Coefficient (2) (K_p)
Residual Clay	19.0	0.35	0.5	N/A
Class 5 Shale (3)	22.0	0.2	0.5	4
Class 4/3 Sandstone/Shale (3)	23.0	0.0	0.4	10
Class 2 Sandstone/Shale (3)	24.0	0.0	0.2	30

Notes to table:

1. These values assume that some wall movement and relaxation of horizontal stress will occur due to the excavation. Actual in-situ K_0 values may be higher, particularly in the rock units.
2. Includes a reduction factor to the ultimate value of K_p to take into account strain incompatibility between active and passive pressure conditions.
3. Parameters assume horizontal backfill and no wall friction.
4. The values for rock assume no adversely dipping joints or other defects are present in the bedrock. All excavation rock faces should be inspected regularly by an experienced Geotechnical Engineer / Engineering Geologist as excavation proceeds.

The parameters for the 'at rest' condition (K_o) should be used for design of lateral earth pressures where adjacent footings/structures are located within the 'zone of influence' of the wall. The 'zone of influence' may be taken as a line extending upwards and outwards at 45° above horizontal from the base of the wall. Piles for cantilever walls should be socketed below bulk excavation level by a depth at least equal to the retained height. For assessment of passive restraint embedded below excavation level, we recommend a triangular pressure distribution.

Walls supported by multiple rows of anchors / props may be designed for a uniform lateral earth pressure of $0.65\ \gamma\ H\ K_a$ where γ = unit weight of retained material, H = height of wall, and K_a = earth pressure coefficient (Table 9). Piles for braced walls should be socketed at least 0.75m below basement subgrade level to provide toe "kick-in" resistance until the slab can be poured.

NUMERICAL MODELLING APPROACH

More complex excavation support may also be designed using strength and stiffness parameters for soil and rock stratum, with 2D numerical analysis software such as Phase^2 from Rocscience, or WALLAP (for preliminary design).

The values in Table 10 provide typical parameters that can be adopted for design. Review and refinement of these parameters may be necessary as part of carrying out more advanced numerical modelling (e.g. consideration of advanced soil models, use of elasto-plastic parameters).

Table 10 – Excavation Support Design Parameters (Numerical Modelling Approach)

Material	Moist Unit Weight (γ_m) kN/m^3	'At Rest' Coefficient (1) (K_o)	Effective Cohesion (c') kPa	Effective Friction Angle (ϕ') deg	Elastic Modulus (E') MPa
Stiff Residual Clay	19.0	0.5	15	25	10
Class 5 Shale (2)	22.0	0.5	90	21	300
Class 4/3 Sandstone/Shale (2)	23.0	0.4	250	31	500
Class 2 Sandstone/Shale (2)	24.0	0.2	2000	36	750

Notes to table:

1. Actual in-situ K_0 values may be higher, particularly in the rock units. Consideration should be given to the locked-in horizontal stress which may be present within the rock units.
2. The values for rock assume no adversely dipping joints or other defects are present in the bedrock. All excavation rock faces should be inspected regularly by an experienced Geotechnical Engineer / Engineering Geologist as excavation proceeds.

5.8.3 Surcharge

Allowance must also be made for surcharge loadings and footing loads from adjacent structures.

5.8.4 Drainage and Groundwater Control

Where adequate subsoil drainage is provided behind walls, no allowance for groundwater is considered necessary. Appropriate surcharge loading at the finished surface level should also be adopted for design of the wall. Control of groundwater seepage through the basement wall should also be allowed for, unless a waterproof basement is designed and constructed.

5.8.5 Underpinning

The timing / programme of geotechnical inspections for further assessment of rock conditions and footings supporting the adjacent buildings and surrounding roads should be nominated by the Geotechnical Engineer prior to commencement of bulk excavation.

The assessment of adjacent footings should include assessment of soil or filling depths along the site boundaries that could require support during construction.

The requirements for rock support must be nominated or approved by the Geotechnical Engineer.

The design of underpinning measures and/or excavation support must be carried out by a suitably experienced and qualified structural/civil engineer.

5.8.6 Ground Anchors

Prestressed anchoring of shoring / retaining walls can be adopted for the development, subject to obtaining permission from adjacent property owners / authorities where anchors extend outside the site boundaries.

Anchors could be inclined up to a maximum of 30° below horizontal if required to intercept bedrock / higher strength bedrock. Design of excavation support must be carried out by a suitably experienced and qualified structural/civil engineer. Requirements for rock support must be nominated or approved by the Geotechnical Engineer during excavation. Rock bolts may be designed for the parameters in Table 11.

Table 11 – Rock Bolting Design Parameters

Layer	Ultimate Bond Stress (without Factor of Safety)
Residual Soil	25 kPa
Class 5 Sandstone /Shale	90 kPa
Class 4 Sandstone /Shale	150 kPa
Class 3 Sandstone /Shale	600 kPa
Class 2 Sandstone	2000 kPa
Class 2 Shale	900 kPa

The following should be noted during anchor design and construction:

- The contractor should adopt design values including an appropriate factor of safety relevant to the installation methodology and anchor type adopted.
- Anchor holes must be clean prior to grouting
- Anchors should be check stressed to 125% of the nominal working load and then locked off at 60% to 80% of the working load.

5.9 Potential Impacts on Adjacent Developments

Potential geotechnical risks of construction on adjoining developments could include; vibration effects due to rock excavation and settlement / deflection of adjacent footings due to the basement excavation. These risks have been discussed in the relevant sections of this report. We assess that if the development is designed and constructed in accordance with the recommendations given in this report, these affects are anticipated to have negligible impact and be within acceptable limits.

6. PAVEMENT CBR VALUES

Based on the laboratory California Bearing Ratio (CBR) test results that varied from 2.5% to 6.0%, it is recommended that a lower bound CBR value of 2.5% is used for design purposes. This should be verified during construction by insitu testing.

7. GEOTECHNICAL & HYDROGEOLOGICAL MONITORING PROGRAM

7.1 Acceptable Vibration and Deflection Limits

The contractor shall carry out excavation and construction activities so that the limits in Table 12 are not exceeded:

Table 12 – Vibration and Deflection Limits

Parameter	Limit
vertical settlement of ground surface at adjoining boundaries	5 mm
lateral deflection of temporary or permanent retaining works (measured at the top or any point of the retaining works)	5 mm
peak particle velocity at any sensitive adjoining structure	5 mm/sec

7.2 Monitoring System

7.2.1 Deflections / Settlement

Monitoring of deflections and settlements shall be carried out by a registered surveyor. Survey points shall be established along the site boundaries where excavation is proposed, at a spacing of not more 10m. Survey measurements shall be taken:

- prior to the commencement of excavation
- immediately after bulk excavation
- immediately after installation of temporary retaining works
- immediately after construction of permanent retaining works
- immediately after backfilling of retaining works

The inclusion of inclinometers to measure deflection in the shoring may be necessary close to high risk neighbouring structures. Additional groundwater monitoring wells outside of the basement structure may be required to provide long term monitoring of the effect of construction dewatering on the surrounding developments and groundwater table.

7.2.2 Vibration

Where excavation is carried out in accordance with Section 5.4.1, adopting methodology for a maximum peak particle velocity of 5 mm/s, a vibration monitoring system will not be required. Tactile assessment of vibration shall be carried out by a Geotechnical Engineer during commencement of rock excavation, and where construction equipment varies from that used at the commencement of rock excavation.

7.3 Hold Points

Hold points shall be provided at the following stages to allow for inspection by a Geotechnical Engineer:

- At commencement of shoring / pile installation.
- At the commencement of rock anchor installation.
- At the commencement of rock excavation.
- At commencement of dewatering (if required)
- At the completion of bulk excavation.
- At completion of detail footing excavation.

7.4 Contingency Plan

In the event that the above listed acceptable limits are exceeded, the following works shall be carried out:

- The Project Geotechnical Engineer shall be notified immediately.
- Excavations adjacent to areas that have settled shall be backfilled with spoil or other suitable material.
- Additional bracing shall be installed adjacent to areas of temporary or permanent shoring.
- Excavation equipment shall cease work immediately, and vibration monitoring equipment shall be installed at locations selected by the Geotechnical Engineer to measure vibrations. If the vibration limit exceeds 10 mm/second, alternative equipment and/or methodology shall be used.

8. CONSTRUCTION METHODOLOGY

Construction should be carried out in accordance with the following suggested methodology:

1. Install survey monitoring points and inclinometers if not installed directly in shoring piles.
2. Demolish existing structures.

3. Install temporary shoring walls as recommended.
4. Carry out bulk excavation through rock. Rock breaking equipment and operation must be in accordance with Section 5.4.1.
5. Survey monitoring points and inclinometers to check for movement.
6. Geotechnical Engineer to inspect excavation sides and base.
7. Install sump-and-pump drainage control measures as bulk dig progresses.
8. Carry out detail footing excavation.
9. Geotechnical Engineer to inspect footing excavations.
10. Construct footings, basement walls and subsoil drainage behind.
11. Survey monitoring points to check for movement.
12. Construct basement slabs.
13. Backfill behind basement walls when authorised by the Structural Engineer.
14. Survey monitoring points to check for movement.

9. LIMITATIONS

In addition to the limitations inherent in site investigations (refer to the attached Information Sheets), it must be pointed out that the recommendations in this report are based on assessed subsurface conditions from limited investigations. In order to confirm the assessed soil and rock properties in this report, further investigation would be required such as coring and strength testing of rock, and should be carried out if the scale of the development warrants, or if any of the properties are critical to the design, construction or performance of the development.

It is recommended that a qualified and experienced Geotechnical Engineer be engaged to provide further input and review during the design development; including site visits during construction to verify the site conditions and provide advice where conditions vary from those assumed in this report. Development of an appropriate inspection and testing plan should be carried out in consultation with the Geotechnical Engineer.

This report may have included geotechnical recommendations for design and construction of temporary works (e.g. temporary batter slopes or temporary shoring of excavations). Such temporary works are expected to perform adequately for a relatively short period of time only, which could range from a few days (for temporary batter slopes) up to six months (for temporary shoring). This time period depends on a range of factors including but not limited to: site geology; groundwater conditions; weather conditions; design criteria; and level of care taken during construction. If there are factors which prevent temporary works from being completed and/or which require temporary works to function for periods longer than originally designed, further advice must be sought from the Geotechnical Engineer and Structural Engineer.

This report and details for the proposed development should be submitted to relevant regulatory authorities that have an interest in the property or the adjoining properties (e.g. Council) or are responsible for services that may be within or adjacent to the site (e.g. Sydney Water, Sydney Trains, Roads and Maritime Services), for their review and approval.

The document “Important Information about your Geotechnical Report” in Appendix A provides additional information about the uses and limitations of this report.

FIGURES

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APPROXIMATE ONLY – SUBJECT TO DETAIL SURVEY.

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issue	date	description
A	12.10.16	INITIAL ISSUE

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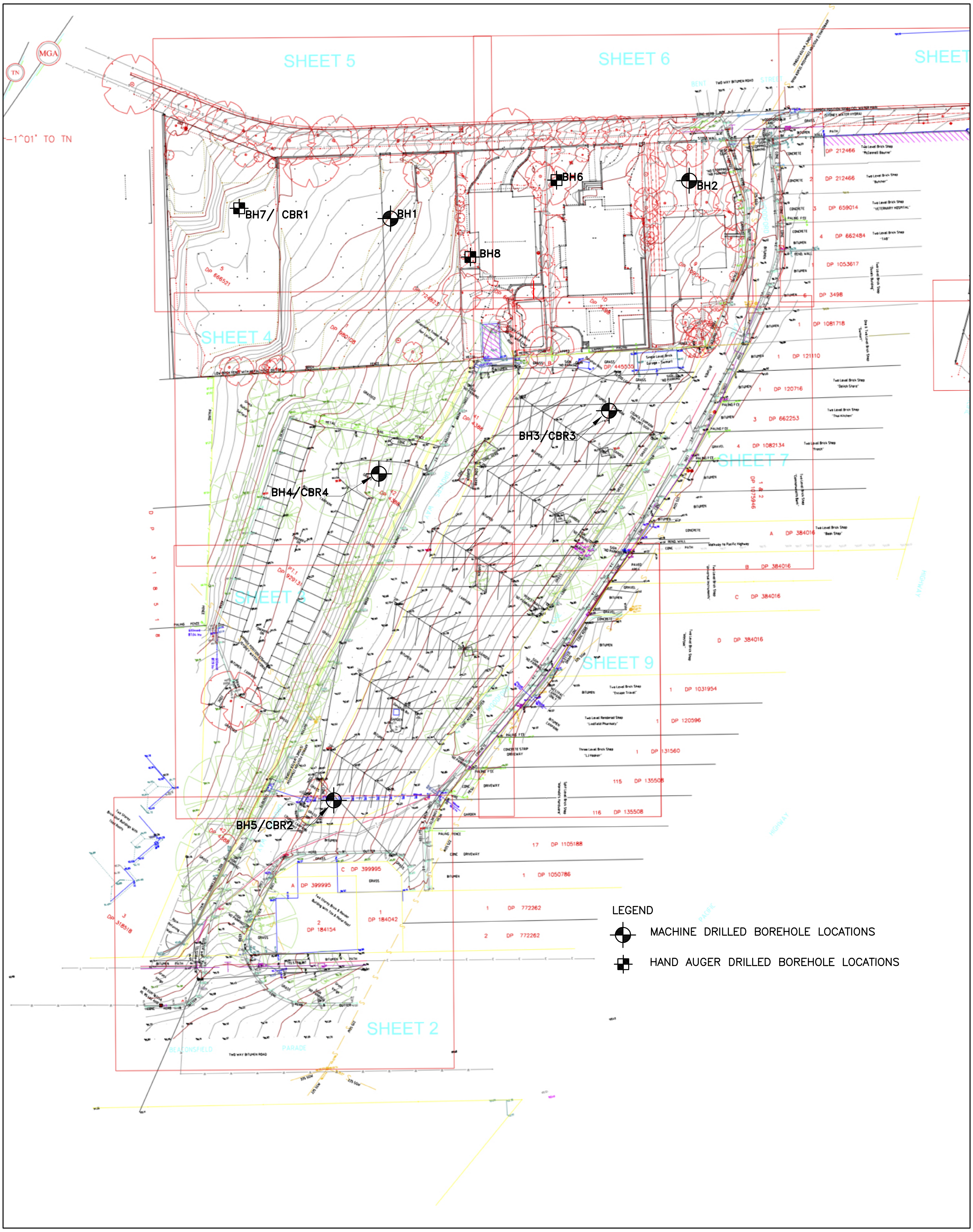
PROPOSED MIXED USE DEVELOPMENT
LINDFIELD COMMUNITY HUB PROJECT,
LINDFIELD, NSW
For
KU-RING-GAI COUNCIL

SITE LOCALITY

drawn: DJ
date: 12.10.16
checked: MAG
scale: 1:2500 A4

job no.: 3914
fig: 1
issue: A



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SOURCE: "SURVEY PLAN BY WILLIAM L. BACKHOUSE, SURVEYORS, PLANNERS & DEVELOPMENT CONSULTANTS"

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checked: MAG
scale: 1:800 A3

job no.: 3914
fig: 2
issue: A

APPENDIX A

Important Information about your Geotechnical Report
CSIRO BTF 18

Important Information about your Geotechnical Report

SCOPE OF SERVICES

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

The report should not be used if there have been changes to the project, without first consulting with Asset to assess if the report's recommendations are still valid. Asset does not accept responsibility for problems that occur due to project changes if they are not consulted.

RELIANCE ON DATA

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

GEOTECHNICAL ENGINEERING

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

LIMITATIONS OF SITE INVESTIGATION

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

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

Therefore, the recommendations in the report can only be regarded as preliminary. Asset should be retained during the project implementation to assess if the report's recommendations are valid and whether or not changes should be considered as the project proceeds.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

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

VERIFICATION OF SITE CONDITIONS

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

REPRODUCTION OF REPORTS

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

REPORT FOR BENEFIT OF CLIENT

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

DATA MUST NOT BE SEPARATED FROM THE REPORT

The report as a whole presents the site assessment, and must not be copied in part or altered in any way.

Logs, figures, drawings, test results etc. included in our reports are developed by professionals based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These data should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

PARTIAL USE OF REPORT

Where the recommendations of the report are only partially followed, there may be significant implications for the project and could lead to problems. Consult Asset if you are not intending to follow all of the report recommendations, to assess what the implications could be. Asset does not accept responsibility for problems that develop where the report recommendations have only been partially followed if they have not been consulted.

OTHER LIMITATIONS

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

Foundation Maintenance and Footing Performance:

A Homeowner's Guide

CSIRO

BTF 18
replaces
Information
Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth

Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation

Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

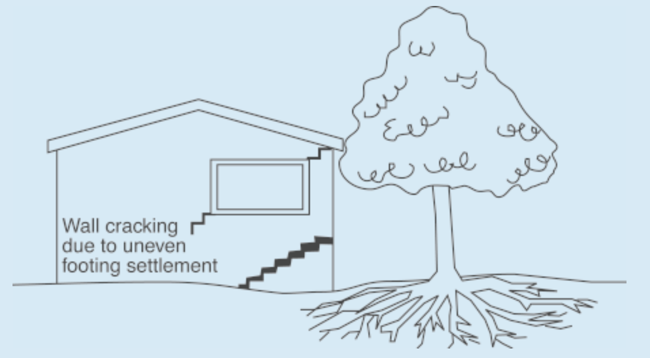
Seasonal swelling/shrinkage in clay

Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots

In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself

Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures

Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

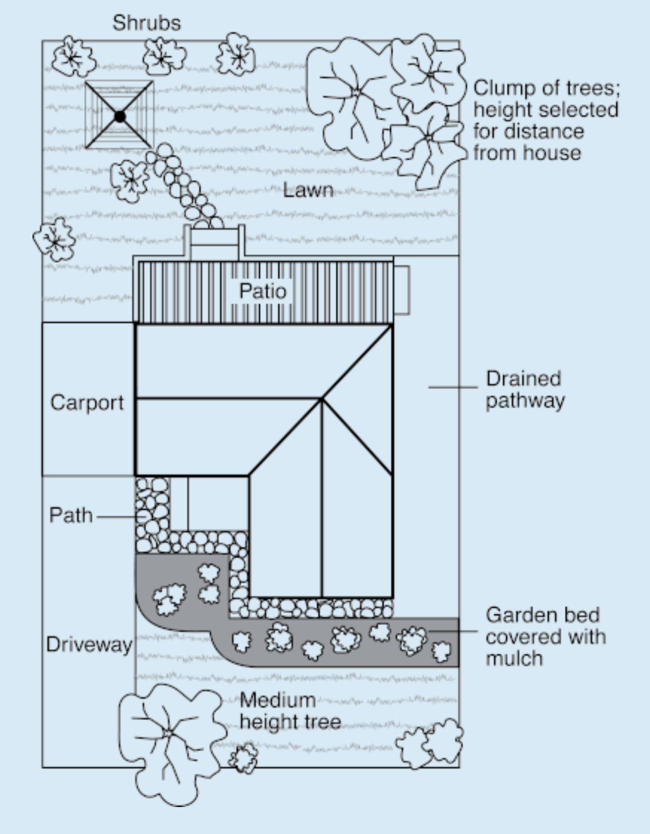
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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APPENDIX B

Soil & Rock Explanation Sheets
Borehole Logs
Core Logs
DCP Logs
Core Photographs

Soil and Rock Explanation Sheets (1 of 2)

LOG ABBREVIATIONS AND NOTES

METHOD

borehole logs

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

* bit shown by suffix e.g. ADV

excavation logs

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

coring

NMLC, NQ, PQ, HQ

SUPPORT

borehole logs

N	nil
M	mud
C	casing
NQ	NQ rods

excavation logs

N	nil
S	shoring
B	benched

CORE—LIFT

‖	casing installed
├┤	barrel withdrawn

NOTES, SAMPLES, TESTS

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

USCS SYMBOLS

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

MOISTURE CONDITION

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

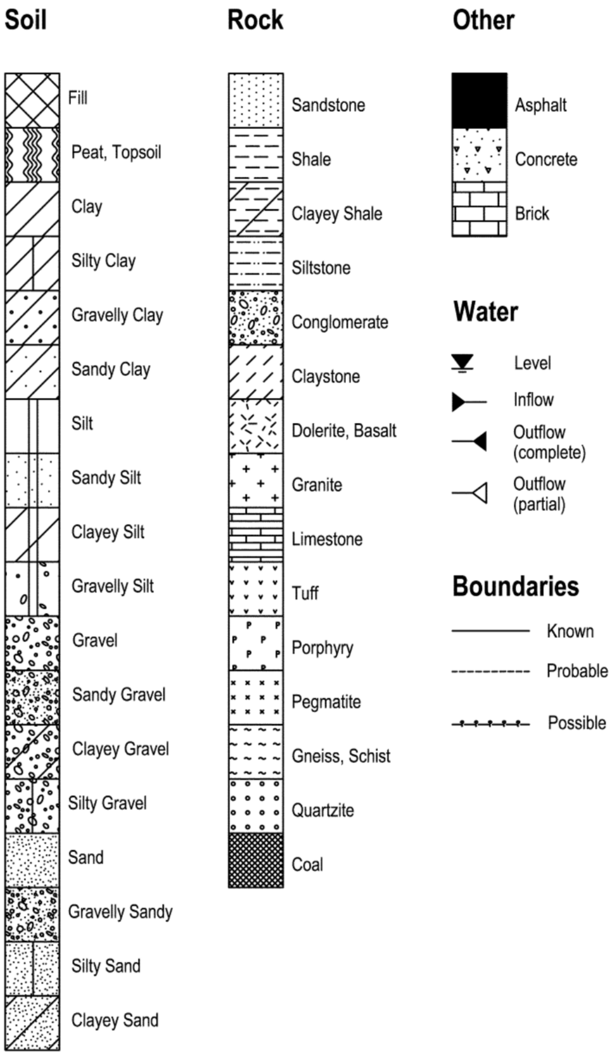
CONSISTENCY

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

DENSITY INDEX

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

GRAPHIC LOG



WEATHERING

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

STRENGTH

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

RQD (%)

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

DEFECTS:

type

JT	joint
PT	parting
SZ	shear zone
SM	seam

coating

cl	clean
st	stained
ve	veneer
co	coating

shape

pl	planar
cu	curved
un	undulating
st	stepped
ir	irregular

roughness

po	polished
sl	slickensided
sm	smooth
ro	rough
vr	very rough

inclination

measured above axis and perpendicular to core

Soil and Rock Explanation Sheets (2 of 2)

AS1726-1993

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

SOIL

MOISTURE CONDITION

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

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

CONSISTENCY OF COHESIVE SOILS

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

DENSITY OF GRANULAR SOILS

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

PARTICLE SIZE

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

MINOR COMPONENTS

Term	**Proportion by Mass:**	
	coarse grained	***fine grained***
Trace	= 5%	= 15%
Some	5 – 2%	15 – 30%

SOIL ZONING

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

SOIL CEMENTING

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

USCS SYMBOLS

Symbol	Description
GW	Well graded gravels and gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
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MH	Inorganic silts of high plasticity.
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity.
PT	Peat muck and other highly organic soils.

ROCK

SEDIMENTARY ROCK TYPE DEFINITIONS

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

LAYERING

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

STRUCTURE

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

STRENGTH (NOTE: Is50 = Point Load Strength Index)

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

WEATHERING

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

DEFECT DESCRIPTION

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

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

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

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

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Borehole Log

BH no: BH1
sheet: 1 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 21.9.2016
finished: 22.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 88 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed					CL	CLAY with some sand and traces of gravel, low plasticity, brown to dark brown, sand is fine to medium grained, gravel is subangular, some building rubble (brick fragments) and root fibres observed.	M	F-St		Topsoil, brick fragments observed are part of building debris.
					0.3		CL	As above, brown, brick fragments and root fibres considerably lesser.				FILL
				87.5	0.5							
				87.0	1.0							
			SPT 5,10,18 N*=28									
					1.2		CH	CLAY with trace sand and trace gravel, high plasticity, orange brown mottled grey and red, sand is fine to medium grained, traces of ironstone inclusions, gravel is subangular.		H		RESIDUAL
				86.5	1.5						×	450
					1.8			SHALE, logged as clay, fine grained, brown, extremely weathered, extremely low strength, poorly developed.				
				86.0	2.0							
				85.5	2.5							
				85.0	3.0							
				84.5	3.5							
				84.0	4.0							
					4.2			As above, some sandstone fragments observed.				
				83.5	4.5							
				83.0	5.0			Borehole No: BH1 continued as cored hole from 4.9m.				

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Cored Borehole Log

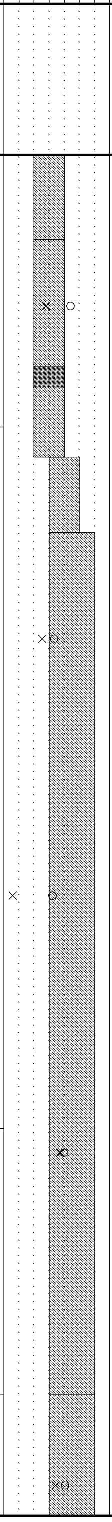
BH no: BH1
sheet: 2 of 4
job no.: 3914

client:	Ku-ring-gai Council	**started:**	21.9.2016
principal:		**finished:**	22.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	88 m
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

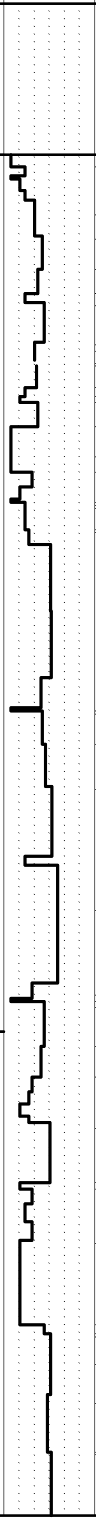
drilling information — method: NMLC; support & core-lift; water: None observed

method	water	RL	depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	$Is_{(50)}$ MPa (D= diametral ×, A= axial ○)
	None observed	83.0	5.0	Continued from non-cored borehole from 4.9m		
NMLC			5	SHALE, fine grained, dark grey with orange bands in some bedding planes, poorly developed, bedding planes at 0°-5°.	XW - HW	
			5.28	SANDSTONE, fine to medium grained, orange to orange grey with ironstains observed in bedding layers, poorly developed, bedding planes at 0°-5°.		D= 0.24 A= 1.55
		82.5	5.5			
			5.77	SHALE, fine to medium grained, dark grey, poorly developed, bedding planes at 0°-5°.		
			5.9	SANDSTONE, fine to medium grained, orange-grey, poorly developed.		
		82.0	6.0			
			6	SHALE, fine grained, dark grey, poorly developed, bedding planes at 0°-5°.		
			6.25	Interbedded LAMINITE and SILTSTONE, dark grey to grey with some orange bands, poorly developed.		D= 0.18 A= 0.45
		81.5	6.5			
			6.85	SANDSTONE, fine to medium grained, grey to orange, well developed bedding layers between 7.50m-7.80m, bedding planes at 0°-10°.	HW	D= 0.02 A= 0.4
		81.0	7.0			
		80.5	7.5			
		80.0	8.0			
			8.05	Interbedded LAMINITE and SILTSTONE, fine grained, grey, poorly developed, bedding planes at 0°-5°.		
			8.22	SANDSTONE, fine to medium grained, grey, highly fractured.	HW - MW	D= 0.72 A= 0.95
		79.5	8.5			
		79.0	9.0			
			9.1	SILTSTONE, fine grained, grey, poorly developed.	MW	
			9.3	SANDSTONE, fine to medium grained, grey, poorly developed.		D= 0.5 A= 1.02
	None observed	78.5	9.5			

estimated strength MPa: EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH



rock mass defects — RQD %; defect spacing mm: 20 60 200 600 2000



defect description (type, inclination, thickness, shape, roughness, coating) — specific / general

- Crushed seam.
- PT@0°-5°,pl,sm,cg.
- JT@0°-5°,pl,ro,st.
- JT@0°-5°,un,ro,st.
- JT@0°-10°,un,ro,st.
- PT@0°-5°,ir,ro,cg.
- JT@0°-10°,ir,ro,cl.
- JT@0°-5°,ir,ro,cg.
- Possible clay infill.
- JT@0°-10°,ir,ro,cg.
- Crushed seam.
- JT@0°-10°,ir,ro,cg.
- JT@0°-5°,pl,ro,cg.
- Highly fractured seam.
- JT@0°-10°,pl,ro,cg.
- PT@0°-5°,pl,ro,cg.
- PT@0°-10°,pl,ro,cl.
- Possible clay infill.
- JT@0°-5°,pl,ro,st.
- JT@0°-10°,pl,ro,cl.
- Manganese oxide stains in bedding planes.
- Crushed seam.
- JT@0°-5°,pl,ro,cl.
- JT@0°-10°,pl,ro,cg.
- JT@0°-5°,pl,ro,cl.
- Highly fractured rock.
- Possible clay infill.
- Vertical joint.
- PT@0°-5°,pl,ro,cl.
- PT@0°-5°,pl,ro,cl.

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

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Cored Borehole Log

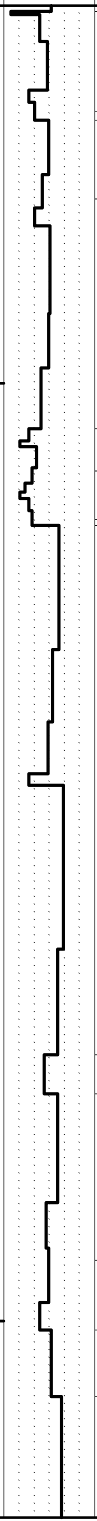
BH no: BH1
sheet: 3 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 21.9.2016
finished: 22.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 88 m
datum: AHD

drilling information – method: NMLC; water: None observed

material information / **rock mass defects**

depth metres	RL	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D= diametral × A= axial ○	defect description type, inclination, thickness, shape, roughness, coating (specific)
9.52		Interbedded LAMINITE and SILTSTONE/MUDSTONE, fine grained, grey, well developed bedding planes at 0°-5°.	MW - SW		
					JT@0°-5°,pl,ro,cl.
10.0	78.0				
					Vertical joint.
10.5	77.5				
					PT@0°-5°,pl,sm,cl.
				D= 0.08 A= 0.82	
11.0	77.0				JT@0°-10°,pl,ro,cl.
					JT@0°-5°,pl,ro,st.
					JT@0°-5°,pl,ro,cl.
11.22		SANDSTONE, fine to coarse grained, grey, poorly developed.			
11.5	76.5			D= 0.46 A= 0.63	
12.0	76.0				
					JT@0°-10°,pl,sm/ro,cl.
12.5	75.5				
				D= 0.4 A= 1.06	
12.97		SHALE, fine grained, grey, poorly developed.	SW		
13.0	75.0				PT@0°-5°,pl,sm,cl.
13.1		SANDSTONE, fine to medium grained, grey, poorly developed bedding planes.	SW - FR		
					JT@0°-10°,pl,ro,cl.
13.5	74.5			D= 0.5 A= 0.96	
					JT@0°-5°,pl,ro,cl.
14.0	74.0				
14.5	73.5			D= 0.49	



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Cored Borehole Log

BH no: BH1
sheet: 4 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 21.9.2016
finished: 22.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 88 m
datum: AHD

drilling information					material information					rock mass defects		
method	support & core-lift	water	RL	depth metres	graphic log core recovery	**rock substance description** rock type; grain characteristics, colour, structure, minor components	weathering	estimated strength MPa EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH	$Is_{(50)}$ MPa × D= diametral ○ A= axial	RQD %	defect spacing mm 20 60 200 600 2000	**defect description** type, inclination, thickness, shape, roughness, coating specific general
NMLC			73.0	15.0		SANDSTONE, fine to medium grained, grey, poorly developed bedding planes. *(continued)*	SW - FR		A 1.32			PT@0°-5°,pl,sm,cl.
				15		BH1 terminated at 15m						
			72.5	15.5								
			72.0	16.0								
			71.5	16.5								
			71.0	17.0								
			70.5	17.5								
			70.0	18.0								
			69.5	18.5								
			69.0	19.0								
			68.5	19.5								

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PROPOSED MIXED-USE DEVELOPMENT
8-12 BENT ST, LINDFIELD, NSW
for
KU-RING-GAI COUNCIL

CORE PHOTOS

drawn: TT
date: 28.09.16
checked: MAG
scale: NTS

job no.: 3914
fig: BH1
issue: A

issue	date	description
A	28.09.16	INITIAL ISSUE

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Borehole Log

BH no: **BH2**
sheet: 1 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 21.9.2016
finished: 21.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 95.31 m approx.
datum: AHD

drilling information						**material information**						
method	support	water	notes, samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetrometer kPa 100 200 300 400	**structure and additional observations**
ADT	C	6.4					CL	CLAY with some sand, low plasticity, dark brown, sand is fine to medium grained, traces of root fibres, <5% gravel, subangular.	M	S-F		TOPSOIL
					0.2		CL	CLAY with traces of sand, dark brown to grey, low to medium plasticity, sand is fine to medium grained.		F-St		COLLUVIUM/RESIDUAL? (No fill observed)
				95.0	0.5							
				94.5	0.8		CH	As above, high plasticity, grey.				
										H		RESIDUAL
					1.0							
			SPT 4,7,12 N*=19	94.0							×	450
					1.4			SHALE logged as clay, grey, extremely weathered, extremely low strength, very thinly laminated.				RESIDUAL/CLASS 5 SHALE
					1.5							
				93.5								
					2.0							
				93.0								
					2.5							
					2.5			As above, grey-brown, extremely low strength to very low strength.				
				92.5								
					3.0							
				92.0								
					3.5							
				91.5								
					4.0							
					4			As above, dark grey to light grey, very thinly bedded.				
				91.0								
					4.5							
				90.5								
					5.0							

3914 LOGS.GPJ 11/10/16

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Borehole Log

BH no:	**BH2**
sheet:	2 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	21.9.2016
principal:		**finished:**	21.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	95.31 m approx.
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information						**material information**						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	6.4 Measured on 29/09/16 ▼ Auger cuttings moist ▽		90.0 89.5 89.0	5.5 6.0			As above, dark grey to light grey, very thinly bedded. *(continued)*	M			
				88.5 88.0	6.2 6.5 7.0 7.5			SHALE logged as clay, brown, presence of ironstains on rock fragments.				
				87.5 87.0 86.5 86.0 85.5	8.0 8.5 9.0 9.5 10.0			Borehole No: BH2 continued as cored hole from 7.6m				

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Cored Borehole Log

BH no:	**BH2**
sheet:	3 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	21.9.2016
principal:		**finished:**	21.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	95.31 m
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

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

method	support & core-lift	water	RL	depth metres	graphic log core recovery	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	$Is_{(50)}$ MPa (D= diametral, A= axial)
		6.4	88.0	7.5		Continued from non-cored borehole from 7.6m		
NMLC			87.5	7.6		SHALE, fine grained, grey-orange mottled red, poorly developed, very thinly bedded, ironstains visible in bedding, bedding planes at 0°-5°.	MW	D= 0.76 A= 0.94
			87.0	8.0 8		SHALE, fine grained, dark grey to grey, poorly developed to well developed, very thinly bedded, bedding planes @ 0°-5°.	MW	D= 0.26 A= 0.76
			86.5 86.0	8.5 9.0				
				9.4 9.5		As above, highly fractured with possible clay infill.	RS - XW	
			85.5 85.0	9.6 10.0		SANDSTONE, fine to medium grained, grey, poorly developed, very thinly bedded.	HW - MW	D= 0.13 A= 0.35 D= 0.55 A= 0.73
			84.5	10.4 10.5		Interbedded LAMINITE and SILTSTONE, fine grained, dark grey, well developed, very thinly bedded, bedding planes at 0°-5°.		
			84.0	10.84 11.0		SANDSTONE, fine to medium grained, grey, poorly developed.	MW	
			83.5	11.33 11.5		Interbedded LAMINITE and SILTSTONE, fine grained, grey, poorly developed bedding planes @ 0°-5°.		D= 0.28 A= 0.38
		6.4		11.88 12.0		Interbedded SHALE and SANDSTONE, grey to dark grey, poorly developed layering.		

estimated strength MPa: EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH

defect spacing mm: 20, 60, 200, 600, 2000

defect description (type, inclination, thickness, shape, roughness, coating; specific / general)

- JT @ 0°-5°, pl, sm, cg.
- SM@0°-5°, pl, sm, cg.
- PT @ 0°-5°, pl, sm, cg.
- PT @ 0°-5°, pl, sm, cl.
- JT @ 0°-5°, un, sm, cl.
- Possible clay infill.
- JT @ 0°-10°, pl, sm, cl.
- PT @ 0°-5°, pl, sm, cl.
- PT @ 0°-10°, pl, ro, cl.
- JT @ 0°-5°, pl, sm, cl.
- PT @ 0°-5°, pl, sm, cl.
- JT @ 0°-10°, un, sm, cl.
- JT @ 0°-5°, pl, sm, cl.
- PT @ 0°-5°, pl, sm, cl.
- JT @ 0°-10°, un, ro, cg.
- PT @ 0°-5°, pl, sm, cg.
- JT @ 0°-5°, ir, ro, cl.
- PT @ 0°-5°, pl, ro/sm, cl.
- JT @ 0°-5°, pl, sm, cl.

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Cored Borehole Log

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BH no: BH2
sheet: 4 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**

started: 21.9.2016
finished: 21.9.2016
logged: DJ
checked: MAG
RL surface: 95.31 m
datum: AHD

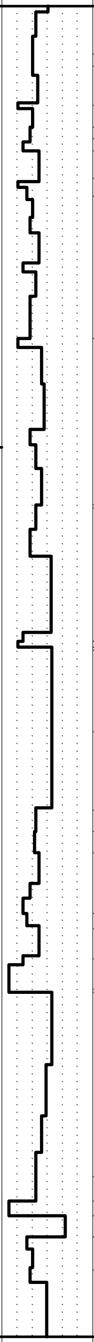
drilling information / material information / rock mass defects

Method: NMLC. Weathering: MW.

depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	Is(50) MPa (D= diametral × A= axial ○)
	Interbedded SHALE and SANDSTONE, grey to dark grey, poorly developed layering. *(continued)*	
12.4	SANDSTONE, fine to medium grained, grey, very thinly bedded, well developed bedding planes at 0°-5°.	D= 0.19 A= 1.16 D= 0.32 A= 0.85
13.65	SHALE, fine grained, dark grey, very thinly bedded, well developed, bedding planes @0°-5°.	D= 0.02 A= 0.41
14.1	SANDSTONE, fine to medium grained, grey, poorly developed, vertical joint between 14.10m - 14.95m, interbedded layers of shale and sandstone between 14.80m - 15.20m, highly fractured zone between 15.20m - 15.25m.	
15.25	Interbedded LAMINITE and SILTSTONE, fine grained, grey, very thinly bedded, well developed bedding planes @ 0°-5°, vertical joint between 14.10m - 16.40m.	D= 0.03 A= 0.06 D= 0.02 A= 0.24
16.4	BH2 terminated at 16.4m	

RL: 83.0, 82.5, 82.0, 81.5, 81.0, 80.5, 80.0, 79.5, 79.0, 78.5
Depth: 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 15.5, 16.0, 16.5, 17.0

estimated strength (MPa): EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH



defect description (type, inclination, thickness, shape, roughness, coating; specific / general)

- (vertical joint observed).
- PT @ 0°-5°, pl, sm, cl.
- JT @ 0°-5°, pl, ro, cl.
- Crushed seam/ clay infill?
- PT @ 0°-5°, pl, ro, cl.
- JT @ 0°-10°, ir, ro, cl.
- JT @ 0°-5°, pl, ro/sm, cl.
- PT @ 0°-10°, pl, ro, st (Manganese stains?).
- PT @ 0°-10°, pl, ro/sm, cl.
- JT @ 0°-5°, pl, sm, cl.
- PT @ 0°-5°, pl, ro, cl.
- JT @ 0°-5°, ir, ro, cl.
- PT @ 0°-5°, pl, ro, cl.
- JT @ 0°-5°, un, sm, cl.
- JT @ 0°-5°, pl, sm, cg.
- Crushed seam, intensely fractured.
- PT @ 0°-5°, pl, sm, cl.
- Possible induced joint.
- PT @ 0°-5°, pl,ro, cl.
- PT @ 0°-5°, pl, sm, cl. Verical joint observed.

A	28.09.16	INITIAL ISSUE
issue	date	description

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PROPOSED MIXED-USE DEVELOPMENT
2 BENT ST, LINDFIELD, NSW
for
KU-RING-GAI COUNCIL

CORE PHOTOS

drawn: TT
date: 28.09.16
checked: MAG
scale: NTS
job no.: 3914
fig: BH2
issue: A

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Borehole Log

BH no: BH3
sheet: 1 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 22.9.2016
finished: 22.9.2016
logged: DJ
checked: MAG
equipment: Edson100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 95 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
AST	C	None observed				■	--	ASPHALT with sub-base	--	--		ASPHALT
					0.1	▨	CH	CLAY, traces of sand and traces of gravel, high plasticity, sand is fine to medium grained, <5% subangular, orange-brown mottled red and grey	M	F - St	150	COLLUVIUM / RESIDUAL?, no fill observed
				94.5	0.5							
				94.0	1.0							
			SPT 5,10,13 N*=23	93.5	1.5	▨	CH	As above, traces of ironstone inclusions, orange mottled red to grey		H	450	RESIDUAL
					1.9	▤		SHALE / Shaley CLAY, extremely weathered, extremely low strength, poorly developed layering, logged as CLAY, brown to red-brown, very thinly bedded				RESIDUAL
				93.0	2.0							
				92.5	2.5							
				92.0	3.0							
				91.5	3.5	▤		As above, but brown				
				91.0	4.0	▤		As above, extremely weathered, extremely low strength, brown to dark brown, logged as CLAY				
				90.5	4.5							
				90.0	5.0							

3914 LOGS.GPJ 11/10/16

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Borehole Log

BH no:	**BH3**
sheet:	2 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	22.9.2016
principal:		**finished:**	22.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson100 Truck mounted	**RL surface:**	95 m approx.
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information						**material information**						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
AST	C	None observed						As above, extremely weathered, extremely low strength, brown to dark brown, logged as CLAY *(continued)*				RESIDUAL
				89.5	5.5							
				89.0	6.0			Borehole No: BH3 continued as cored hole from 6m				
				88.5	6.5							
				88.0	7.0							
				87.5	7.5							
				87.0	8.0							
				86.5	8.5							
				86.0	9.0							
				85.5	9.5							
				85.0	10.0							

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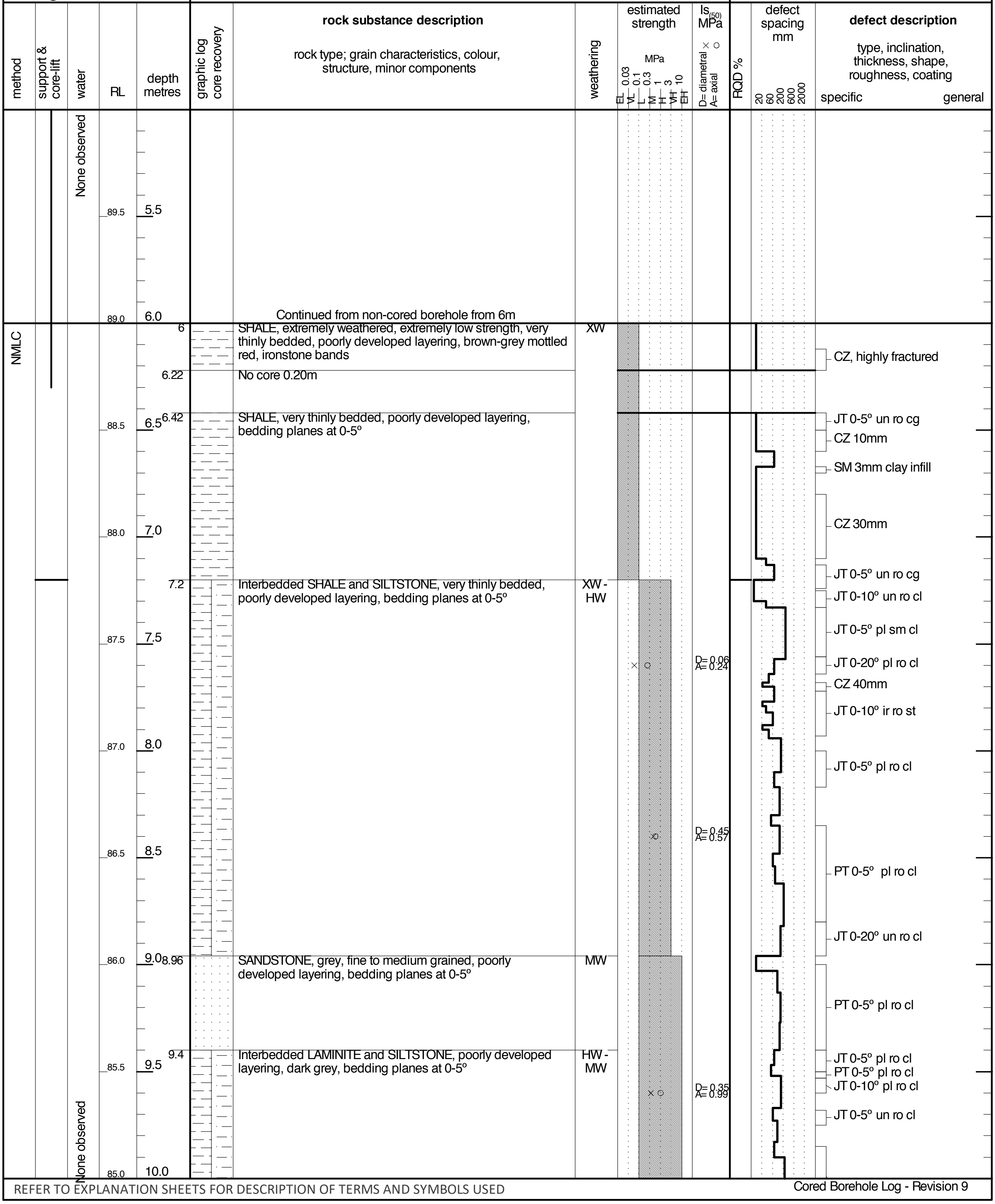
Cored Borehole Log

BH no:	**BH3**
sheet:	3 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	22.9.2016
principal:		**finished:**	22.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson100 Truck mounted	**RL surface:**	95 m
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

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



depth (m)	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	Is(50) MPa
	Continued from non-cored borehole from 6m		
6	SHALE, extremely weathered, extremely low strength, very thinly bedded, poorly developed layering, brown-grey mottled red, ironstone bands	XW	
6.22	No core 0.20m		
6.42	SHALE, very thinly bedded, poorly developed layering, bedding planes at 0-5°		
7.2	Interbedded SHALE and SILTSTONE, very thinly bedded, poorly developed layering, bedding planes at 0-5°	XW - HW	D= 0.06 A= 0.24 D= 0.45 A= 0.57
8.96	SANDSTONE, grey, fine to medium grained, poorly developed layering, bedding planes at 0-5°	MW	
9.4	Interbedded LAMINITE and SILTSTONE, poorly developed layering, dark grey, bedding planes at 0-5°	HW - MW	D= 0.35 A= 0.99

defect description (type, inclination, thickness, shape, roughness, coating; specific, general)

- CZ, highly fractured
- JT 0-5° un ro cg
- CZ 10mm
- SM 3mm clay infill
- CZ 30mm
- JT 0-5° un ro cg
- JT 0-10° un ro cl
- JT 0-5° pl sm cl
- JT 0-20° pl ro cl
- CZ 40mm
- JT 0-10° ir ro st
- JT 0-5° pl ro cl
- PT 0-5° pl ro cl
- JT 0-20° un ro cl
- PT 0-5° pl ro cl
- JT 0-5° pl ro cl
- PT 0-5° pl ro cl
- JT 0-10° pl ro cl
- JT 0-5° un ro cl

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Cored Borehole Log

BH no:	**BH3**
sheet:	4 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	22.9.2016
principal:		**finished:**	22.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson100 Truck mounted	**RL surface:**	95 m
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

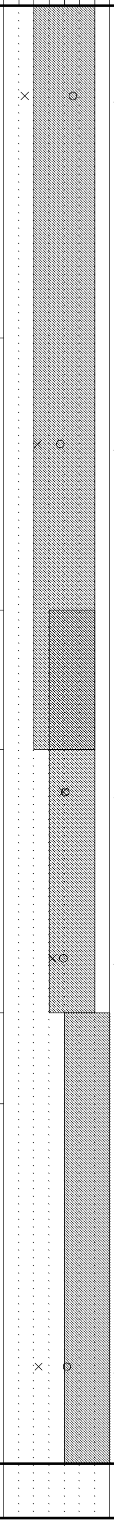
drilling information — method: NMLC

RL	depth metres
84.5	10.5
84.0	11.0
83.5	11.5
83.0	12.0
82.5	12.5
82.0	13.0
81.5	13.5
81.0	14.0
80.5	14.5
80.0	15.0

material information

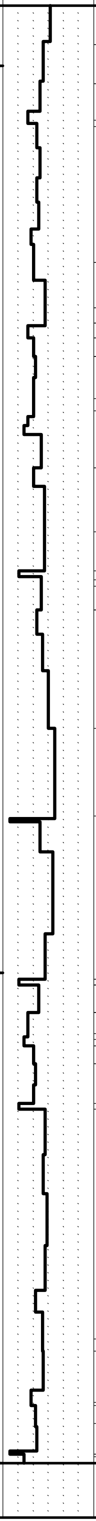
depth	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D= diametral × A= axial ○
	Interbedded LAMINITE and SILTSTONE, poorly developed layering, dark grey, bedding planes at 0-5° *(continued)*	HW - MW	D= 0.05 A= 1.85
11.1	SANDSTONE, grey, fine to medium grained	MW - SW	D= 0.13 A= 0.71
12	Interbedded LAMINITE and SILTSTONE, dark grey, poorly developed layering, bedding planes at 0-5°, thinly bedded	MW	
12.46	SANDSTONE, fine to medium grained, grey, poorly develop layering, manganese bands observed in bedding planes	MW - SW	D= 0.89 A= 1.05 D= 0.4 A= 0.9
13.33	Interbedded LAMINITE and SILTSTONE, poorly developed layering, bedding planes at 0-5°, grey	MW	
13.5	SANDSTONE, fine to coarse grained, poorly developed layering, bedding planes at 0-5°, grey		
13.63	Interbedded LAMINITE and SILTSTONE, grey, poorly developed layering, bedding planes at 0-5°	MW - SW	
14.46	Interbedded SILTSTONE and SANDSTONE, fine to medium grained, grey, poorly developed layering, bedding planes at 0-5°		D= 0.14 A= 1.17
14.82	BH3 terminated at 14.82m		

estimated strength MPa



rock mass defects — RQD %

defect spacing mm



defect description — type, inclination, thickness, shape, roughness, coating (specific / general)

- PT 0-5° pl sm cl
- JT 0-5° pl ro cl
- 0-10° un ro cl
- PT 0-5° pl ro cl
- JT 0-5° pl ro/sm cl
- vertical joint
- CZ 50mm
- vertical joint
- JT 0-5° pl sm cl
- PT 0-5° pl sm cl
- JT 0-5° pl sm cl
- PT 0-5° pl sl sm
- PT 0-5° pl sm cg
- PT 0-5° pl sm cl
- JT 0-5° pl ro cl
- drilling induced joint
- PT 0-5° pl sm st (transition to shale at bedding planes
- PT 0-5° pl sm cl
- induced joint
- JT 0-20° pl ro cl
- JT 0-10° pl ro cl
- PT 0-5° pl sm cl
- PT 0-5° pl sm cl
- JT, clay infill
- JT 0-5° pl sm cg
- PT 0-5° pl sm cl

3914 LOGS.GPJ 12/10/16

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PROPOSED MIXED–USE DEVELOPMENT
DROVERS WAY CARPARK (NORTH),
LINDFIELD, NSW
for
KU–RING–GAI COUNCIL

CORE PHOTOS

drawn: TT
date: 28.09.16
checked: MAG
scale: NTS

job no.: 3914
fig: BH3
issue: A

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Borehole Log

BH no: BH4
sheet: 1 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 92 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed					CL-ML	Silty CLAY, low plasticity, Dark brown, <5% subangular gravel, some root fibres.	-	-		TOPSOIL
					0.3		CH	CLAY with traces of gravel, high plasticity, red brown mottled orange, gravdel is subangular, traces of root fibres.	D	Fb		RESIDUAL
				91.5	0.5							
				91.0	1.0							
					1		CH	As above, traces of ironstone inclusions, no root fibres.		St-VSt	× 200	
				90.5	1.5							
				90.0	2.0							
					2.3		CH	As above, red-grey.				
				89.5	2.5							
				89.0	3.0							
					3.2			SHALE/SHALEY CLAY, extremely weathered to highly weathered, extremely low to very low strength, logged as clay, brown, poorly developed layering, very thinly bedded.				RESIDUAL/CLASS 5 SHALE
				88.5	3.5							
				88.0	4.0							
				87.5	4.5							
					4.5			As above, traces of ironstones.				
				87.0	5.0							

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Borehole Log

BH no:	**BH4**
sheet:	2 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	26.9.2016
principal:		**finished:**	26.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	92 m approx.
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information | **material information**

method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed						As above, traces of ironstones. *(continued)*				
				86.5	5.5							
				86.0	6.0							
				85.5	6.5							
				85.0	7.0							
					7			As above, highly weathered, very low to low strength.				
				84.5	7.5							
								Borehole No: BH4 continued as cored hole from 7.5m				
				84.0	8.0							
				83.5	8.5							
				83.0	9.0							
				82.5	9.5							
				82.0	10.0							

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Cored Borehole Log

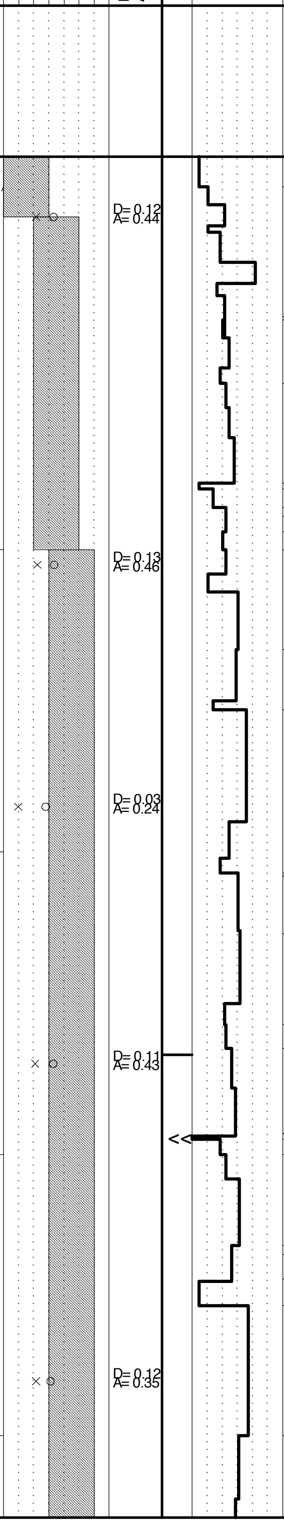
BH no:	**BH4**
sheet:	3 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	26.9.2016
principal:		**finished:**	26.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	92 m
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

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

method	support & core-lift	water	RL	depth metres
		None observed		7
			84.5	7.5
NMLC			84.0	8.0
			83.5	8.5
			83.0	9.0
			82.5	9.5
			82.0	10.0
			81.5	10.5
			81.0	11.0
			80.5	11.5
		None observed	80.0	12.0

depth	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering
	Continued from non-cored borehole from 7.5m	
7.5	Interbedded layers of shale and sandstone, fine grained, grey to dark grey mottled orange, very thinly bedded, poorly developed layering, bedding planes exhibit ironstaining.	RS - XW XW - HW
8.8	SANDSTONE, fine to medium grained, grey with orange in some bedding planes, poorly developed layering.	HW - MW
9.8	Interbedded LAMINITE and SILTSTONE, fine grained, grey to dark grey, well developed, bedding planes at 0°-5°.	HW - MW
10.8	SANDSTONE, fine to medium grained, grey, poorly developed layering.	HW - MW
11.73	Interbedded LAMINITE and SILTSTONE, fine grained, dark grey to black, poorly developed layering, bedding planes at 0°-5°.	HW - MW



defect description type, inclination, thickness, shape, roughness, coating specific general
CZ, highly fractured, residual/clay infill.
JT 0°-5° un ro st/cg. (ironstains in bedding layers).
CZ 10 mm.
JT 0°-10° pl ro st.
JT 0°-10° pl ro cg/st.
Possible clay infill.
Possible clay infill.
JT 0°-5° pl ro st/cg. Ironstains in bedding layers.
JT 0°-5° pl ro cl.
JT 0°-5° pl ro/sm cl.
PT 0°-5° pl sm cl.
CZ 10mm.
JT 0°-5° pl ro cg.
PT 0°-5° pl sm cl.
JT 0°-5° pl sm cl.
JT 0°-5° pl sm cl.
CZ intensely fractured.
JT 0°-5° pl sm cl.
Vertical fracture.
CZ/clay infill.
JT 0°-5° pl ro cg.
JT 0°-5° pl sm cl.

3914 LOGS.GPJ 12/10/16

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Cored Borehole Log

BH no: BH4
sheet: 4 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 125mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 92 m
datum: AHD

method	RL	depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	Is(50) MPa (D= diametral, A= axial)	defect description (type, inclination, thickness, shape, roughness, coating)
NMLC			Interbedded LAMINITE and SILTSTONE, fine grained, dark grey to black, poorly developed layering, bedding planes at 0°-5°. *(continued)*	HW - MW		JT 0°-5° pl sm cl.
	79.5	12.5 12.45	SANDSTONE, fine to medium grained, grey, poorly developed layering.	MW - SW	D= 1.35 A= 1.65	
	79.0	13.0				
	78.5	13.5			D= 1.47 A= 1.05	JT 0°-5° pl sm cl.
	78.0	14.0				
	77.5	14.5			D= 0.63 A= 2.39	
		14.85	Interbedded layers of SILTSTONE and SANDSTONE, fine grained, dark grey, poorly developed layering, bedding planes at 0°-10°.	MW - SW		
	77.0	15.0			D= 0.54 A= 1.03	PT 0°-5° pl sm cl.
		15.27	BH4 terminated at 15.27m			
	76.5	15.5				
	76.0	16.0				
	75.5	16.5				
	75.0	17.0				

estimated strength MPa: EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH

defect spacing mm: 20 60 200 600 2000

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PROPOSED MIXED–USE DEVELOPMENT
DROVERS WAY CARPARK (WEST),
LINDFIELD, NSW
for
KU–RING–GAI COUNCIL

CORE PHOTOS

drawn: TT
date: 28.09.16
checked: MAG
scale: NTS

job no.: 3914
fig: BH4
issue: A

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Borehole Log

BH no: BH5
sheet: 1 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: Edson 100 Truck mounted
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 93.42 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	structure and additional observations
ADT	C	None observed			0.0			Asphalt with crushed stone sub-base.				ASPHALT
					0.12			Silty CLAY, low plasticity, dark brown, traces of root fibres.	D	Fb		FILL?
					0.2		CH	CLAY with some silt and trace gravel, high plasticity, red brown, <5% subangular gravel, no root fibres.				
					1.4		CH	As above, red brown mottled grey.			150	
					2.0							RESIDUAL/COLLUVIUM?
					2.1		CH	As above, shale fragments recovered.				
					2.5		CH	As above, but grey brown.				RESIDUAL/CLASS 5 SHALE
		Measured on 29/9/16			3.3			SHALE, extremely weathered, extremely weak to very low strength, logged as clay, grey brown, poorly developed bedding layers, laminated to very thinly bedded, traces of ironstains in bedding planes.				

RL	depth
93.0	0.5
92.5	1.0
92.0	1.5
91.5	2.0
91.0	2.5
90.5	3.0
90.0	3.5
89.5	4.0
89.0	4.5
88.5	5.0

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Borehole Log

BH no:	**BH5**
sheet:	2 of 4
job no.:	3914

client:	Ku-ring-gai Council	**started:**	26.9.2016
principal:		**finished:**	26.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	93.42 m approx.
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information | **material information**

method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed				▨			D			
					5.2	▨		As above, but dark brown to dark grey. Sample is noticeably moister.	M			
				88.0	5.5	▨						
				87.5	6.0	▨						
				87.0	6.5	▨						
				86.5	7.0	▨						
				86.0	7.5	▨						
				85.5	8.0	▨						
				85.0	8.5	▨						
								Borehole No: BH5 continued as cored hole from 8.5m				
				84.5	9.0							
				84.0	9.5							
				83.5	10.0							

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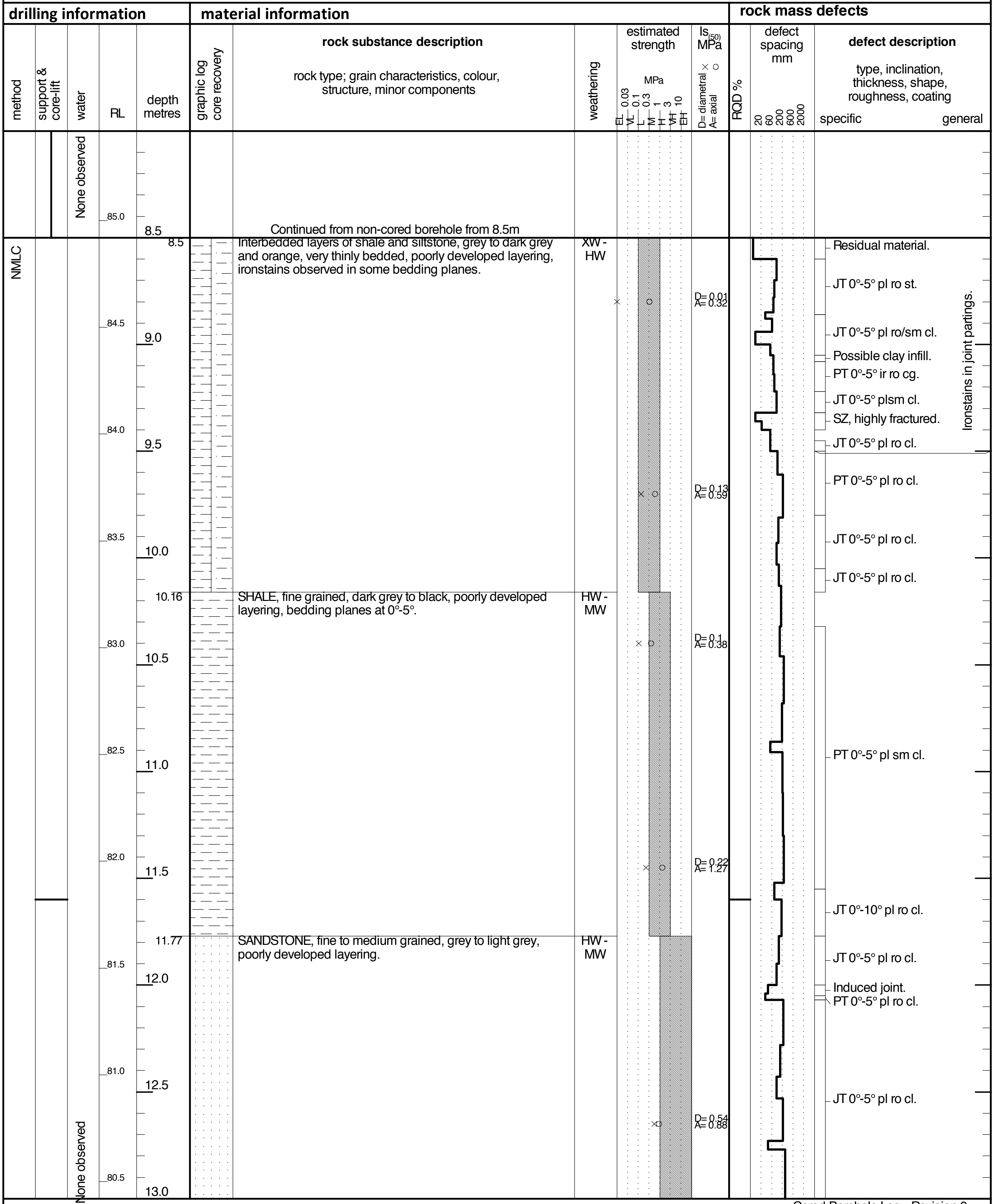
Cored Borehole Log

BH no: BH5
sheet: 3 of 4
job no.: 3914

client:	Ku-ring-gai Council	**started:**	26.9.2016
principal:		**finished:**	26.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	Edson 100 Truck mounted	**RL surface:**	93.42 m
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD



rock substance description — rock type; grain characteristics, colour, structure, minor components

defect description — type, inclination, thickness, shape, roughness, coating

depth	rock substance description	weathering
8.5	Continued from non-cored borehole from 8.5m	
8.5	Interbedded layers of shale and siltstone, grey to dark grey and orange, very thinly bedded, poorly developed layering, ironstains observed in some bedding planes.	XW - HW
10.16	SHALE, fine grained, dark grey to black, poorly developed layering, bedding planes at 0°-5°.	HW - MW
11.77	SANDSTONE, fine to medium grained, grey to light grey, poorly developed layering.	HW - MW

Is(50) MPa:
- D= 0.01 A= 0.32
- D= 0.13 A= 0.59
- D= 0.1 A= 0.38
- D= 0.22 A= 1.27
- D= 0.54 A= 0.88

Defect description (specific):
- Residual material.
- JT 0°-5° pl ro st.
- JT 0°-5° pl ro/sm cl.
- Possible clay infill.
- PT 0°-5° ir ro cg.
- JT 0°-5° plsm cl.
- SZ, highly fractured.
- JT 0°-5° pl ro cl.
- PT 0°-5° pl ro cl.
- JT 0°-5° pl ro cl.
- JT 0°-5° pl ro cl.
- PT 0°-5° pl sm cl.
- JT 0°-10° pl ro cl.
- JT 0°-5° pl ro cl.
- Induced joint.
- PT 0°-5° pl ro cl.
- JT 0°-5° pl ro cl.

Defect description (general): Ironstains in joint partings.

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Cored Borehole Log

BH no: BH5
sheet: 4 of 4
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
equipment: Edson 100 Truck mounted
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
RL surface: 93.42 m
datum: AHD

drilling information					material information				rock mass defects
method	support & core-lift	water	RL	depth metres	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D= diametral × A= axial ○	RQD %	defect description type, inclination, thickness, shape, roughness, coating
NMLC				13.05	Interbedded LAMINITE and SILTSTONE, fine grained, grey, well developed layering, bedding planes at 0°-5°, possible manganese oxide stains in bedding planes.	MW			PT 0°-5° pl ro cl.
			80.0	13.5					JT 0°-5° pl ro cg.
							D= 0.07 A= 1.37		
				13.74	SANDSTONE, fine to medium grained, grey, poorly developed layering.	MW - SW			JT 0°-5° pl ro cl.
			79.5	14.0					
									PT 0°-5° pl ro cl.
			79.0	14.5			D= 0.62 A= 0.89		
									JT 0°-5° pl ro cl.
				14.74	BH5 terminated at 14.74m				
			78.5	15.0					
			78.0	15.5					
			77.5	16.0					
			77.0	16.5					
			76.5	17.0					
			76.0	17.5					
			75.5	18.0					

estimated strength MPa: EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH

defect spacing mm: 20, 60, 200, 600, 2000

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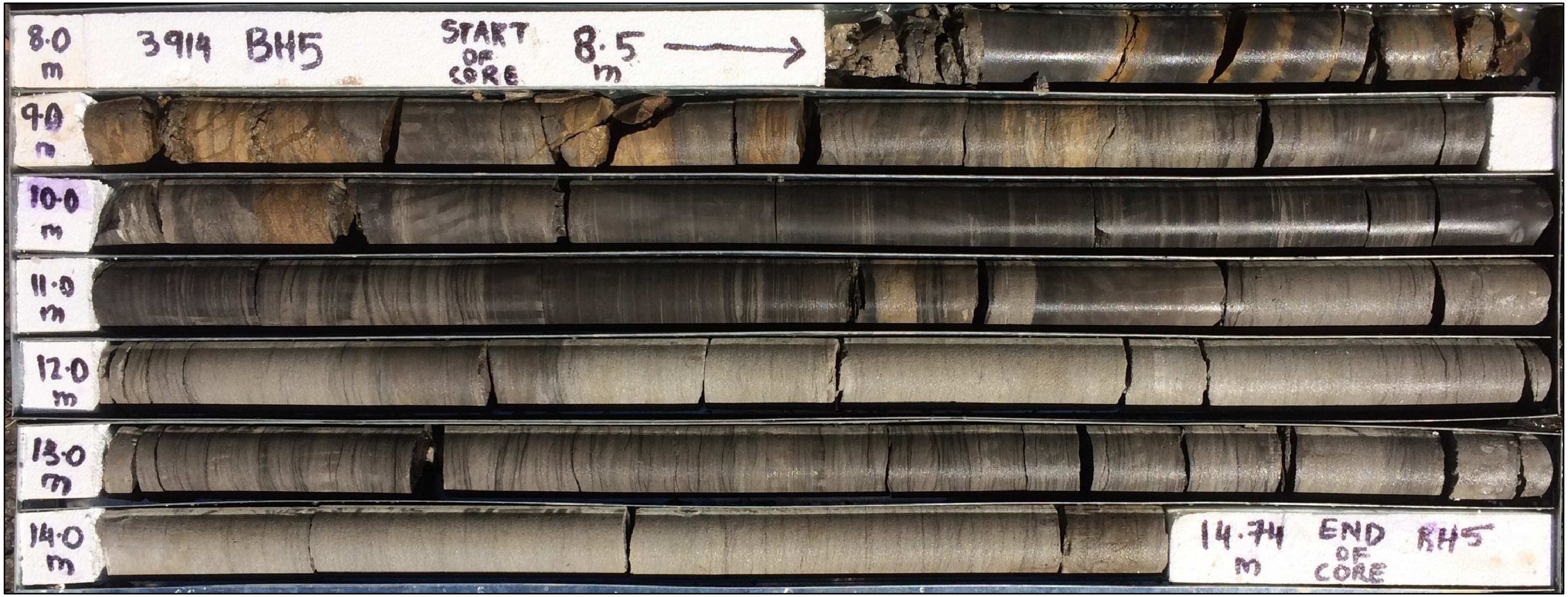

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PROPOSED MIXED–USE DEVELOPMENT
DROVERS WAY CARPARK (SOUTH),
LINDFIELD, NSW
for
KU–RING–GAI COUNCIL

CORE PHOTOS

drawn: TT
date: 28.09.16
checked: MAG
scale: NTS

job no.: 3914
fig: BH5
issue: A

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Borehole Log

BH no: BH6
sheet: 1 of 1
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: HA/DCP Nil
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 92 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes, samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	structure and additional observations
HA/DCP		None observed				░	CL	CLAY with traces of sand and gravel, low plasticity, dark brown, sand is fine grained, gravel is less than 5% and is angular/subangular, some root fibres.	M			Topsoil
				91.5	0.4 0.5	▨	CH	CLAY with traces of sand, medium to high plasticity, brown, sand is fine to medium grained, traces of ironstone inclusions.		S-VSt	× 150	Fill/residual (some sedimentary rock fragments retrieved).
				91.0 90.5	0.8 1.0 1.5	▨	CH	Inferred as above from DCP				Residual.
				90.0 89.5 89.0 88.5 88.0 87.5 87.0	1.8 2.0 2.5 3.0 3.5 4.0 4.5 5.0			Practical refusal on apparent bedrock. Borehole No: BH6 terminated at 1.8m				Apparent bedrock (Ashfield Shale).

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

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Fax: 02 8282 5011

Borehole Log

BH no:	**BH7**
sheet:	1 of 1
job no.:	3914

client:	Ku-ring-gai Council	**started:**	21.9.2016
principal:		**finished:**	21.9.2016
project:	Lindfield Community Hub	**logged:**	DJ
location:		**checked:**	MAG
equipment:	HA/DCP Nil	**RL surface:**	89 m approx.
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information | **material information**

method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	structure and additional observations
HA/DCP		None observed					SC	Sandy CLAY with traces of gravel, low plasticity, dark brown, sand is fine to medium grained, gravel is subangular, traces of brick fragments (building rubble), traces of roots.	M	St-H	× 200	Topsoil
					0.2		SC-SM	Silty sandy CLAY with traces of gravel, low to medium plasticity, dark brown to red brown mottled orange, sand is fine to medium grained, gravel is subangular, traces of ironstone inclusions, some rock fragments (shale).			× 200	Residual/borderline class 5 shale. Some sandstone fragments retrieved.
				88.5	0.5							
					0.9		SC-SM	As above, increased percentage of rock fragments.			× 200	Residual/class 5 shale.
				88.0	1.0							
					1.1		SC-SM	Inferred as above from DCP.				
					1.2		SC-SM	Inferred as above from DCP.				
				87.5	1.5							
				87.0	2.0							
					2			Solid refusal on apparent bedrock Borehole No: BH7 terminated at 2m				Apparent bedrock (Ashfield Shale)
				86.5	2.5							
				86.0	3.0							
				85.5	3.5							
				85.0	4.0							
				84.5	4.5							
				84.0	5.0							

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

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Borehole Log

BH no: BH8
sheet: 1 of 1
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: HA/DCP Nil
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 90 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
HA/DCP		None observed					CL	CLAy with traces of sand and gravel, low plasticity, dark brown, sand is fine to medium grained, less than 5% subangular gravel, some root fibres.	M			Topsoil
					0.3		CL	As above, traces of ironstone inclusions.		St		Residual
				89.5	0.5							
											× 150	
				89.0	1.0							
					1		CL	Inferred as above from DCP.		St-VSt		
				88.5	1.5							
				88.0	2.0							
					2		CL	Inferred as above from DCP.		H		
				87.5	2.5							
					2.5			Practical refusal on apparent bedrock. Borehole No: BH8 terminated at 2.5m				Apparent bedrock (Ashfield Shale)
				87.0	3.0							
				86.5	3.5							
				86.0	4.0							
				85.5	4.5							
				85.0	5.0							

3914 LOGS.GPJ 11/10/16

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Borehole Log

BH no: BH8
sheet: 1 of 1
job no.: 3914

client: Ku-ring-gai Council
principal:
project: Lindfield Community Hub
location:
started: 26.9.2016
finished: 26.9.2016
logged: DJ
checked: MAG
equipment: HA/DCP Nil
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: 90 m approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetrometer kPa (100 200 300 400)	structure and additional observations
HA/DCP		None observed					CL	CLAy with traces of sand and gravel, low plasticity, dark brown, sand is fine to medium grained, less than 5% subangular gravel, some root fibres.	M			Topsoil
					0.3		CL	As above, traces of ironstone inclusions.		St	150	Residual
				89.5	0.5							
				89.0	1.0							
					1		CL	Inferred as above from DCP.		St-VSt		
				88.5	1.5							
				88.0	2.0							
					2		CL	Inferred as above from DCP.		H		
				87.5	2.5							
					2.5			Practical refusal on apparent bedrock. Borehole No: BH8 terminated at 2.5m				Apparent bedrock (Ashfield Shale)
				87.0	3.0							
				86.5	3.5							
				86.0	4.0							
				85.5	4.5							
				85.0	5.0							

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

assetgeo

Dynamic Cone Penetrometer

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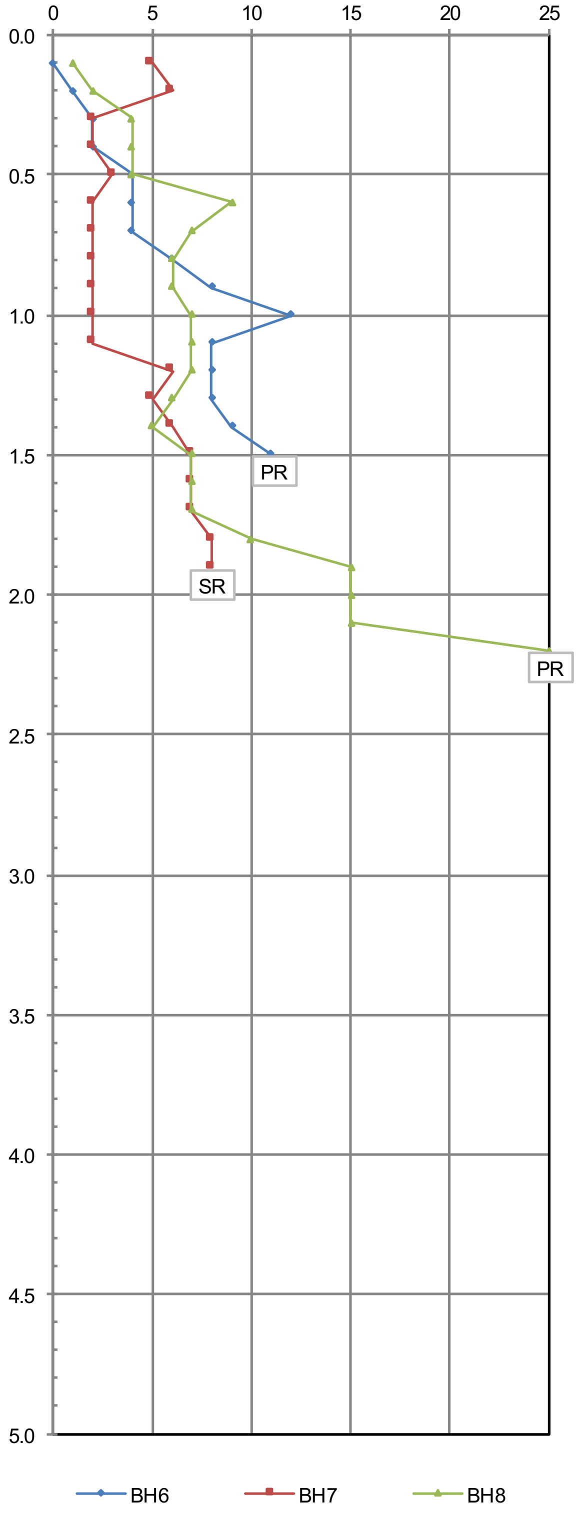
Sheet: 1 of 1

Job No: 3914

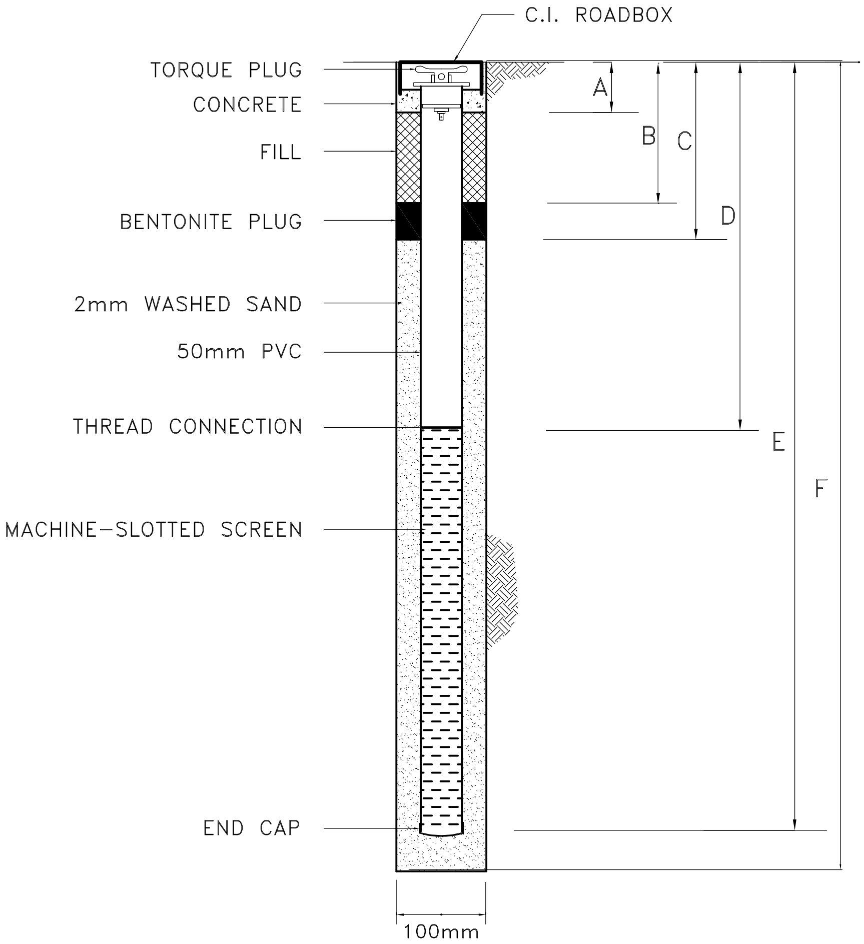
client:	Ku-ring-gai Council	**started:**	21/9/16
principal:		**finished:**	26/9/16
project:	Lindfield Community Hub	**logged:**	DJ
location:	4, 6, 12 Bent Street, Lindfield, NSW	**checked:**	MAG
equipment:	9kg hammer, 510mm drop, cone tip		
standard:	AS1289.6.3.2-1997		

Depth (m)	**Test Results (blows / 100mm)**				
	BH6	**BH7**	**BH8**		
	RL 92.00	RL 89.00	RL 90.00		
0.00 – 0.10	0	5	1		
0.10 – 0.20	1	6	2		
0.20 – 0.30	2	2	4		
0.30 – 0.40	2	2	4		
0.40 – 0.50	4	3	4		
0.50 – 0.60	4	2	9		
0.60 – 0.70	4	2	7		
0.70 – 0.80	6	2	6		
0.80 – 0.90	8	2	6		
0.90 – 1.00	12	2	7		
1.00 – 1.10	8	2	7		
1.10 – 1.20	8	6	7		
1.20 – 1.30	8	5	6		
1.30 – 1.40	9	6	5		
1.40 – 1.50	11	7	7		
1.50 – 1.60	PR	7	7		
1.60 – 1.70		7	7		
1.70 – 1.80		8	10		
1.80 – 1.90		8	15		
1.90 – 2.00		SR	15		
2.00 – 2.10			15		
2.10 – 2.20			25		
2.20 – 2.30			PR		
2.30 – 2.40					
2.40 – 2.50					
2.50 – 2.60					
2.60 – 2.70					
2.70 – 2.80					
2.80 – 2.90					
2.90 – 3.00					
3.00 – 3.10					
3.10 – 3.20					
3.20 – 3.30					
3.30 – 3.40					
3.40 – 3.50					
3.50 – 3.60					
3.60 – 3.70					
3.70 – 3.80					
3.80 – 3.90					
3.90 – 4.00					
4.00 – 4.10					
4.10 – 4.20					
4.20 – 4.30					
4.30 – 4.40					
4.40 – 4.50					
4.50 – 4.60					
4.60 – 4.70					
4.70 – 4.80					
4.80 – 4.90					
4.90 – 5.00					

Plot (blows / 100mm vs depth)



Notes:
RL = ground surface level (m) AHD
TD = target depth, PR = practical refusal (25+ blows per 100mm), SR = "solid" refusal (no further penetration and "solid" ringing sound from slide hammer)



PIEZOMETER DIMENSIONS

	BH2	BH5
A	0.4	0.3
B	5.1	4.8
C	6.5	5.8
D	7.2	11.3
E	16.2	14.3
F	16.5	14.7

issue	date	description
A	14.10.16	INITIAL ISSUE

assetgeo

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PROPOSED MIXED USE DEVELOPMENT
LINDFIELD COMMUNITY HUB
LINDFIELD, NSW
for
KU–RING–GAI COUNCIL

PIEZOMETER DETAILS

drawn: DJ
date: 14.10.16
checked: MAG
scale: NTS

job no.: 3914
fig: 4
issue: A

APPENDIX C

Laboratory Test Results

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17950-PL
Job No:	S16412	**Date Tested:**	28/09/2016

Test Procedure: ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Notes
S17950	BH1 5.50m	Sandstone	Diametral	-	49.0	0.57	0.24	0.24	
			Axial	49.0	43.0	4.10	1.53	1.55	
S17951	BH1 6.60m	Shale	Diametral	-	50.0	0.45	0.18	0.18	
			Axial	50.0	32.0	0.96	0.47	0.45	
S17952	BH1 7.45m	Sandstone	Diametral	-	50.0	0.04	0.02	0.02	
			Axial	50.0	46.0	1.13	0.39	0.40	
S17953	BH1 8.30m	Sandstone	Diametral	-	47.0	1.64	0.74	0.72	
			Axial	47.0	41.0	2.34	0.95	0.95	
S17954	BH1 9.40m	Sandstone	Diametral	-	49.0	1.22	0.51	0.50	
			Axial	49.0	42.0	2.64	1.01	1.02	
S17955	BH1 10.60m	Shale	Diametral	-	50.0	0.20	0.08	0.08	
			Axial	50.0	41.0	2.12	0.81	0.82	
S17956	BH1 11.45m	Sandstone	Diametral	-	48.0	1.09	0.47	0.46	
			Axial	48.0	31.0	1.28	0.68	0.63	
S17957	BH1 12.65m	Sandstone	Diametral	-	50.0	1.01	0.40	0.40	
			Axial	50.0	38.0	2.59	1.07	1.06	
S17958	BH1 13.55m	Sandstone	Diametral	-	50.0	1.24	0.50	0.50	
			Axial	50.0	33.0	2.10	1.00	0.96	
S17959	BH1 14.50m	Sandstone	Diametral	-	50.0	1.23	0.49	0.49	
			Axial	50.0	40.0	3.36	1.32	1.32	

Comments:

NATA

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NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

7/10/2016

Date:

MACQUARIE GEOTECH

Macquarie Geotechr
Unit 8/10
Bradford Street
Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17960-PL
Job No:	S16412	**Date Tested:**	28/09/2016

Test Procedure: ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Notes
S17982	BH2 7.90m	Sandstone	Diametral	-	48.0	1.79	0.78	0.76	
			Axial	48.0	45.0	2.54	0.92	0.94	
S17983	BH2 8.55m	Shale	Diametral	-	50.0	0.64	0.26	0.26	
			Axial	50.0	45.0	2.12	0.74	0.76	
S17984	BH2 9.65m	Sandstone	Diametral	-	50.0	0.32	0.13	0.13	
			Axial	50.0	45.0	0.96	0.34	0.35	
S17985	BH2 10.20m	Sandstone	Diametral	-	48.0	1.28	0.56	0.55	
			Axial	48.0	43.0	1.90	0.72	0.73	
S17986	BH2 11.80m	Shale	Diametral	-	50.0	0.70	0.28	0.28	
			Axial	50.0	35.0	0.88	0.39	0.38	
S17987	BH2 12.55m	Sandstone	Diametral	-	50.0	0.48	0.19	0.19	
			Axial	50.0	34.0	2.59	1.20	1.16	
S17988	BH2 13.35m	Sandstone	Diametral	-	50.0	0.81	0.32	0.32	
			Axial	50.0	46.0	2.39	0.82	0.85	
S17989	BH2 14.05m	Shale	Diametral	-	50.0	0.06	0.02	0.02	
			Axial	50.0	29.0	0.81	0.44	0.41	
S17990	BH2 15.55m	Shale	Diametral	-	50.0	0.08	0.03	0.03	
			Axial	50.0	38.0	0.15	0.06	0.06	
S17991	BH2 16.15m	Shale	Diametral	-	51.0	0.06	0.02	0.02	
			Axial	51.0	41.0	0.62	0.23	0.24	

Comments:

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Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17966-PL
Job No:	S16412	**Date Tested:**	28/09/2016

Test Procedure: ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Notes
S17960	BH3 7.60m	Sandstone	Diametral	-	49.0	0.15	0.06	0.06	
			Axial	49.0	35.0	0.55	0.25	0.24	
S17961	BH3 8.40m	Shale	Diametral	-	48.0	1.06	0.46	0.45	
			Axial	48.0	35.0	1.26	0.59	0.57	
S17962	BH3 9.60m	Sandstone	Diametral	-	50.0	0.87	0.35	0.35	
			Axial	50.0	30.0	2.00	1.05	0.99	
S17963	BH3 10.30m	Shale	Diametral	-	50.0	0.13	0.05	0.05	
			Axial	50.0	33.0	4.04	1.92	1.85	
S17964	BH3 11.45m	Sandstone	Diametral	-	48.0	0.30	0.13	0.13	
			Axial	48.0	40.0	1.74	0.71	0.71	
S17965	BH3 12.60m	Sandstone	Diametral	-	48.0	2.10	0.91	0.89	
			Axial	48.0	37.0	2.42	1.07	1.05	
S17966	BH3 13.15m	Sandstone	Diametral	-	50.0	1.00	0.40	0.40	
			Axial	50.0	35.0	2.05	0.92	0.90	
S17967	BH3 14.50m	Sandstone	Diametral	-	50.0	0.34	0.14	0.14	
			Axial	50.0	39.0	2.92	1.18	1.17	

Comments:

NATA

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Authorised Signatory:

Chris Lloyd

7/10/2016

Date:

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Unit 8/10
Bradford Street
Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17976-PL
Job No:	S16412	**Date Tested:**	28/09/2016

Test Procedure: ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index $Is_{(50)}$ (MPa)	Notes
S17968	BH4 7.70m	Shale	Diametral	-	50.0	0.30	0.12	0.12	
			Axial	50.0	30.0	0.90	0.47	0.44	
S17969	BH4 8.85m	Sandstone	Diametral	-	48.0	0.31	0.13	0.13	
			Axial	48.0	38.0	1.09	0.47	0.46	
S17970	BH4 9.65m	Sandstone	Diametral	-	50.0	0.08	0.03	0.03	
			Axial	50.0	38.0	0.58	0.24	0.24	
S17971	BH4 10.50m	Shale	Diametral	-	50.0	0.27	0.11	0.11	
			Axial	50.0	43.0	1.15	0.42	0.43	
S17972	BH4 11.55m	Sandstone	Diametral	-	50.0	0.30	0.12	0.12	
			Axial	50.0	42.0	0.91	0.34	0.35	
S17973	BH4 12.50m	Sandstone	Diametral	-	48.0	3.17	1.38	1.35	
			Axial	48.0	43.0	4.30	1.64	1.65	
S17974	BH4 13.50m	Sandstone	Diametral	-	46.0	3.24	1.53	1.47	
			Axial	46.0	45.0	2.74	1.04	1.05	
S17975	BH4 14.50m	Sandstone	Diametral	-	50.0	1.58	0.63	0.63	
			Axial	50.0	41.0	6.17	2.36	2.39	
S17976	BH4 15.10m	Shale	Diametral	-	50.0	1.34	0.54	0.54	
			Axial	50.0	40.0	2.60	1.02	1.03	

Comments:

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Authorised Signatory:

Chris Lloyd

Date: 7/10/2016

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Unit 8/10
Bradford Street
Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17982-PL
Job No:	S16412	**Date Tested:**	28/09/2016

Test Procedure: ☑ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index Is$_{(50)}$ (MPa)	Notes
S17977	BH5 8.80m	Shale	Diametral	-	50.0	0.03	0.01	0.01	
			Axial	50.0	26.0	0.58	0.35	0.32	
S17978	BH5 9.70m	Shale	Diametral	-	50.0	0.32	0.13	0.13	
			Axial	50.0	30.0	1.19	0.62	0.59	
S17979	BH5 10.40m	Shale	Diametral	-	50.0	0.24	0.10	0.10	
			Axial	50.0	41.0	0.97	0.37	0.38	
S17980	BH5 11.45m	Shale	Diametral	-	50.0	0.56	0.22	0.22	
			Axial	50.0	30.0	2.57	1.35	1.27	
S17981	BH5 12.65m	Sandstone	Diametral	-	49.0	1.31	0.55	0.54	
			Axial	49.0	40.0	2.19	0.88	0.88	
S17992	BH5 13.60m	Shale	Diametral	-	50.0	0.17	0.07	0.07	
			Axial	50.0	28.0	2.64	1.48	1.37	
S17993	BH5 14.40m	Sandstone	Diametral	-	49.0	1.50	0.62	0.62	
			Axial	49.0	40.0	2.23	0.89	0.89	

Comments:

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Authorised Signatory:

Chris Lloyd

Date: 7/10/2016

Macquarie Geotechr
Unit 8/10
Bradford Street
Alexandria NSW

CALIFORNIA BEARING RATIO REPORT

Client:	Asset Geotechnical	**Source:**	BH3 CBR3
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No.:**	S17995-CBR
Job No.:	S16412	**Lab No.:**	S17995

Test Procedure:

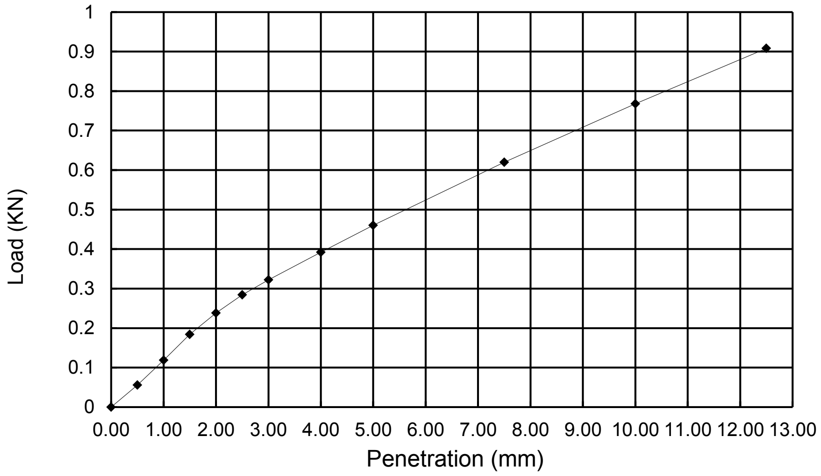
☑ AS1289 6.1.1 Soil strength and consolidation tests - Determination of the California Bearing Ratio of a soil - Standard laboratory method for a remoulded specimen

☑ AS1289 5.1.1 Soil compaction and density tests - Determination of the dry density/moisture content relationship of a soil using standard compactive effort

☑ AS1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (standard method

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16

Preparation: Prepared in accordance with the test method



Compaction and Placement Data

Compaction Used	Standard	Dry Density				
Maximum Dry Density t/m^3	1.69	At Compaction	1.65 t/m^3	98.0	% Comp.	
Optimum Moisture Content %	20.0	After Soaking	1.62 t/m^3	96.0	% Comp.	
No. of Layers	3	Moisture Content				Moisture Ratio (%)
Blows per Layer	53	At Compaction		%	20.3	101
Drop of Rammer mm	300	After Soaking		%	23.1	115
Mass of Rammer kg	2.7	After Penetration (Top 30mm)		%	28.3	141
Surcharge Used kg	4.5	After Penetration (Entire Depth)		%	22.2	111
% Ret. 19mm Sieve	0	Swell After 4 Days Soaking		%	1.8	

Note: material coarser than +19mm Sieve was discarded (as per test method)

California Bearing Ratio

CBR (4-day Soaked) = 2.5 % at 5.0 mm Penetration

Notes:

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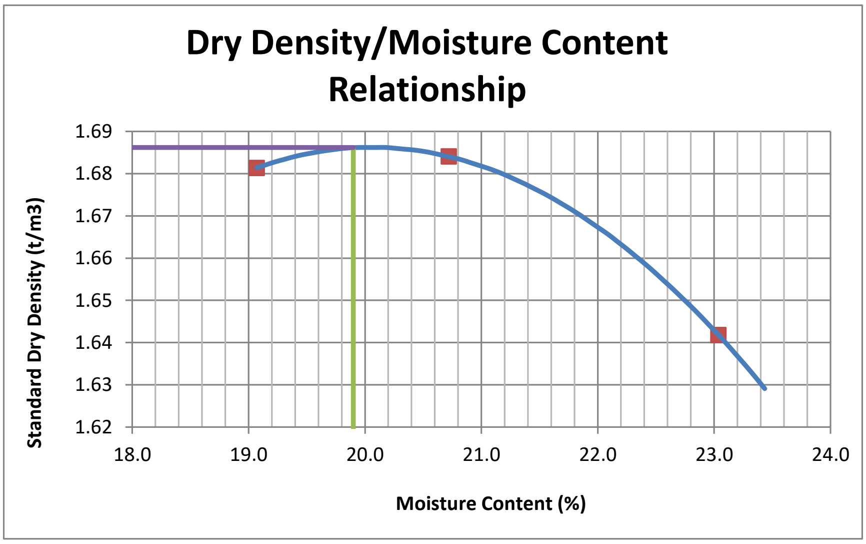
DRY DENSITY / OPTIMUM MOISTURE CONTENT REPORT

Client:	Asset Geotechnical	**Source:**	BH3 CBR3
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17995-MDD
Job No:	S16412	**Lab No:**	S17995

Test Procedure:
☑ AS1289.5.1.1 Determination of the dry density/moisture content relation of a soil using standard compactive effort
☑ AS1289.2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method)

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16
Preparation:	Prepared in accordance with the test method		



Maximum Dry Density (t/m^3)	1.686
Optimum Moisture Content (%)	20.0
Percentage Oversize on 19mm sieve (%)	0
Percentage Oversize on 37.5mm sieve (%)	0

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CALIFORNIA BEARING RATIO REPORT

Client:	Asset Geotechnical	**Source:**	BH4 CBR 4
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No.:**	S17996-CBR
Job No.:	S16412	**Lab No.:**	S17996

Test Procedure:

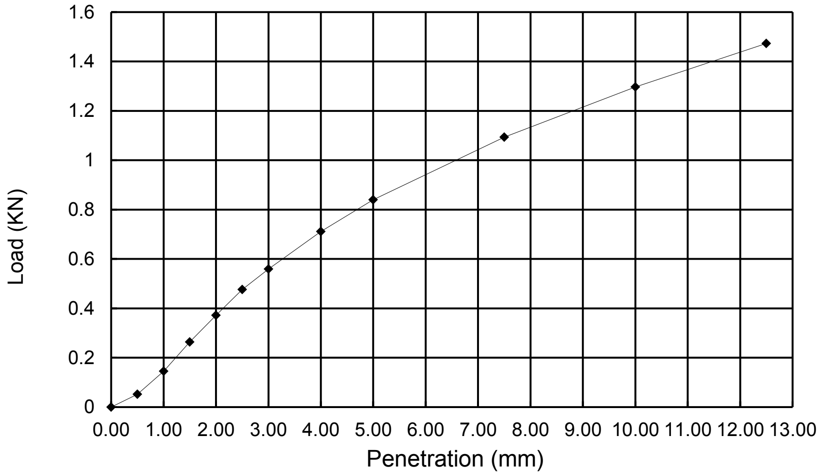
☑ AS1289 6.1.1 Soil strength and consolidation tests - Determination of the California Bearing Ratio of a soil - Standard laboratory method for a remoulded specimen

☑ AS1289 5.1.1 Soil compaction and density tests - Determination of the dry density/moisture content relationship of a soil using standard compactive effort

☑ AS1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (standard method

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method



Compaction and Placement Data

Compaction Used	Standard	Dry Density				
Maximum Dry Density t/m^3	1.68	At Compaction	1.64 t/m^3	98.0	% Comp.	
Optimum Moisture Content %	20.0	After Soaking	1.61 t/m^3	96.0	% Comp.	
No. of Layers	3	Moisture Content				Moisture Ratio (%)
Blows per Layer	53	At Compaction		%	19.9	100
Drop of Rammer mm	300	After Soaking		%	24.5	122
Mass of Rammer kg	2.7	After Penetration (Top 30mm)		%	27.3	136
Surcharge Used kg	4.5	After Penetration (Entire Depth)		%	23.3	117
% Ret. 19mm Sieve	0	Swell After 4 Days Soaking		%	2.4	

Note: material coarser than +19mm Sieve was discarded (as per test method)

California Bearing Ratio

CBR (4-day Soaked) = 4.5 % at 5.0 mm Penetration

Notes:

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DRY DENSITY / OPTIMUM MOISTURE CONTENT REPORT

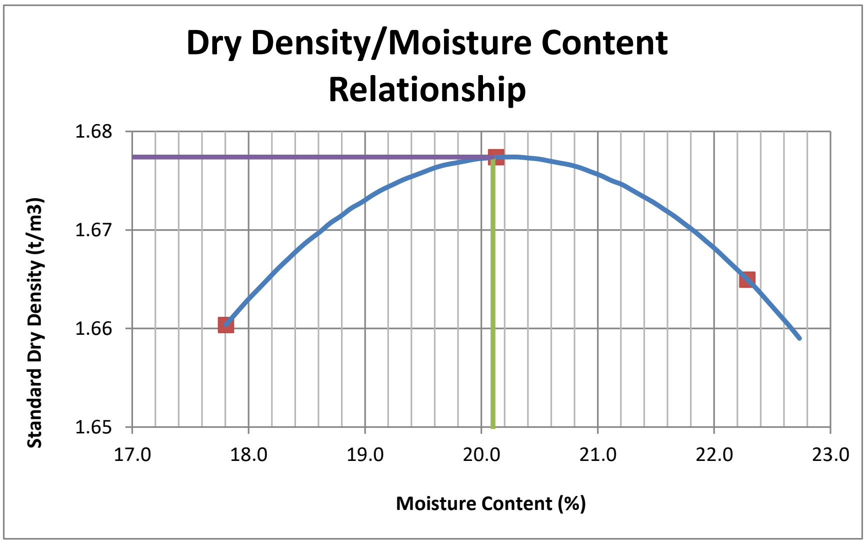
Client:	Asset Geotechnical	**Source:**	BH4 CBR 4
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17996-MDD
Job No:	S16412	**Lab No:**	S17996

Test Procedure:

☑ AS1289.5.1.1 Determination of the dry density/moisture content relation of a soil using standard compactive effort

☑ AS1289.2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method)

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16
Preparation:	Prepared in accordance with the test method		



Maximum Dry Density (t/m^3)	1.677
Optimum Moisture Content (%)	20.0
Percentage Oversize on 19mm sieve (%)	0
Percentage Oversize on 37.5mm sieve (%)	0

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CALIFORNIA BEARING RATIO REPORT

Client:	Asset Geotechnical	**Source:**	BH5 CBR2
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No.:**	S17997-CBR
Job No.:	S16412	**Lab No.:**	S17997

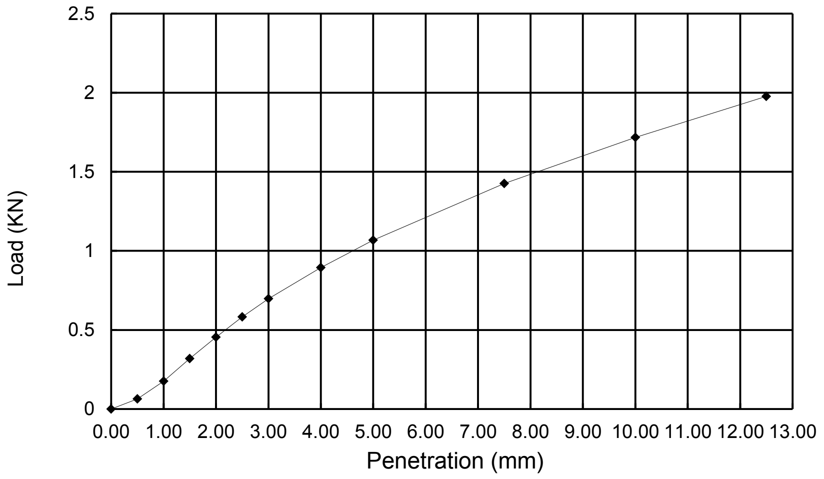
Test Procedure:

☑ AS1289 6.1.1 Soil strength and consolidation tests - Determination of the California Bearing Ratio of a soil - Standard laboratory method for a remoulded specimen

☑ AS1289 5.1.1 Soil compaction and density tests - Determination of the dry density/moisture content relationship of a soil using standard compactive effort

☑ AS1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (standard method

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16
Preparation:	Prepared in accordance with the test method		



Compaction and Placement Data

Compaction Used	Standard	Dry Density				
Maximum Dry Density t/m^3	1.72	At Compaction	1.68 t/m^3	98.0	% Comp.	
Optimum Moisture Content %	19.0	After Soaking	1.65 t/m^3	96.0	% Comp.	
No. of Layers	3	Moisture Content				Moisture Ratio (%)
Blows per Layer	53	At Compaction	%	19.0		100
Drop of Rammer mm	300	After Soaking	%	22.5		118
Mass of Rammer kg	2.7	After Penetration (Top 30mm)	%	27.4		144
Surcharge Used kg	4.5	After Penetration (Entire Depth)	%	21.5		113
% Ret. 19mm Sieve	0	Swell After 4 Days Soaking	%	1.9		

Note: material coarser than +19mm Sieve was discarded (as per test method)

California Bearing Ratio

CBR (4-day Soaked) = 6.0 % at 5.0 mm Penetration

Notes:

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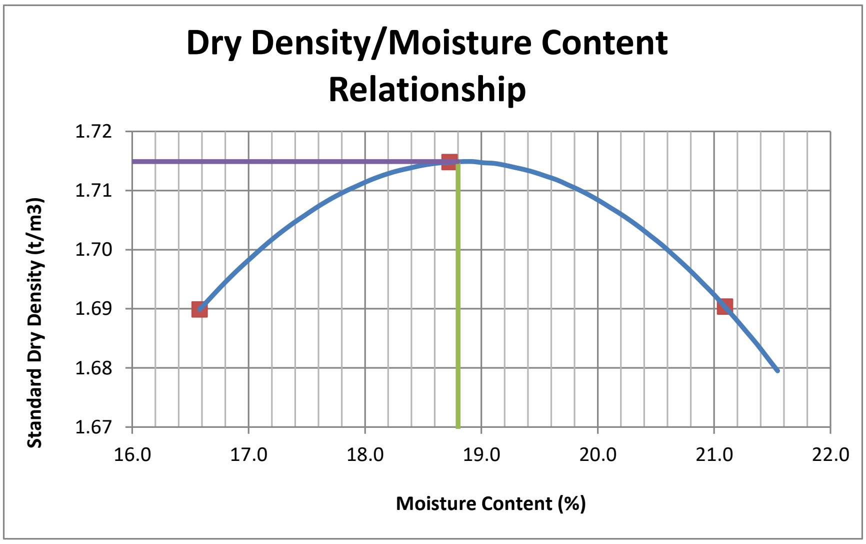
DRY DENSITY / OPTIMUM MOISTURE CONTENT REPORT

Client:	Asset Geotechnical	**Source:**	BH5 CBR2
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17997-MDD
Job No:	S16412	**Lab No:**	S17997

Test Procedure: ☑ AS1289.5.1.1 Determination of the dry density/moisture content relation of a soil using standard compactive effort

☑ AS1289.2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method)

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16
Preparation:	Prepared in accordance with the test method		



Maximum Dry Density (t/m^3)	1.715
Optimum Moisture Content (%)	19.0
Percentage Oversize on 19mm sieve (%)	0
Percentage Oversize on 37.5mm sieve (%)	0

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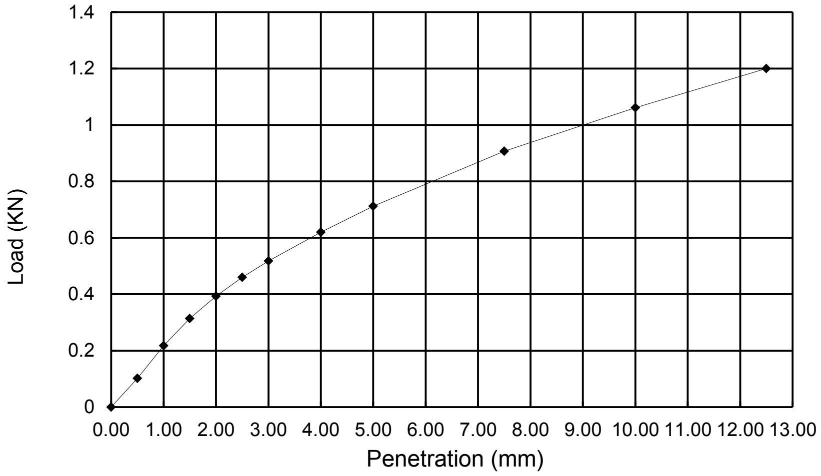
CALIFORNIA BEARING RATIO REPORT

Client:	Asset Geotechnical	**Source:**	BH7a CBR1
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No.:**	S17998-CBR
Job No.:	S16412	**Lab No.:**	S17998

Test Procedure:
☑ AS1289 6.1.1 Soil strength and consolidation tests - Determination of the California Bearing Ratio of a soil - Standard laboratory method for a remoulded specimen
☑ AS1289 5.1.1 Soil compaction and density tests - Determination of the dry density/moisture content relationship of a soil using standard compactive effort
☑ AS1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (standard method

Sampling: Sampled by Client **Date Sampled:** 21-26.09.16

Preparation: Prepared in accordance with the test method



Compaction and Placement Data

Compaction Used	Standard	Dry Density				
Maximum Dry Density t/m^3	1.62	At Compaction	1.59 t/m^3	98.0	% Comp.	
Optimum Moisture Content %	23.0	After Soaking	1.59 t/m^3	98.0	% Comp.	
No. of Layers	3	Moisture Content				Moisture Ratio (%)
Blows per Layer	53	At Compaction		%	21.8	95
Drop of Rammer mm	300	After Soaking		%	23.3	101
Mass of Rammer kg	2.7	After Penetration (Top 30mm)		%	28.4	124
Surcharge Used kg	4.5	After Penetration (Entire Depth)		%	23.4	102
% Ret. 19mm Sieve	0	Swell After 4 Days Soaking		%	0.5	

Note: material coarser than +19mm Sieve was discarded (as per test method)

California Bearing Ratio

CBR (4-day Soaked) = 3.5 % at 2.5 mm Penetration

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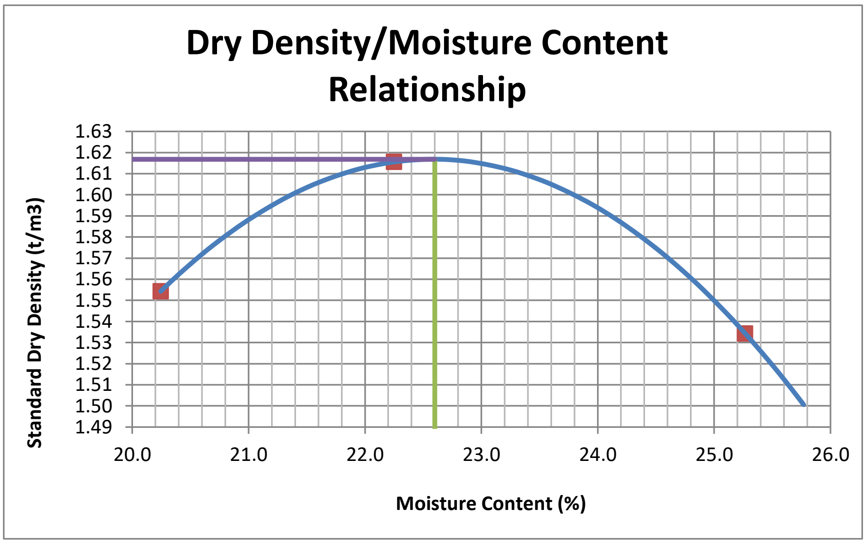
DRY DENSITY / OPTIMUM MOISTURE CONTENT REPORT

Client:	Asset Geotechnical	**Source:**	BH7a CBR1
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Sample Description:**	silty CLAY
Project:	Lindfield Community Hub, Kuringai Council (3914)	**Report No:**	S17998-MDD
Job No:	S16412	**Lab No:**	S17998

Test Procedure:
☑ AS1289.5.1.1 Determination of the dry density/moisture content relation of a soil using standard compactive effort
☑ AS1289.2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method)

Sampling:	Sampled by Client	**Date Sampled:**	21-26.09.16
Preparation:	Prepared in accordance with the test method		



Maximum Dry Density (t/m^3)	1.617
Optimum Moisture Content (%)	23.0
Percentage Oversize on 19mm sieve (%)	0
Percentage Oversize on 37.5mm sieve (%)	0

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