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Centurion Project Management

Proposed Redevelopment
Abel Tasman Village, Chester Hill NSW

Geotechnical Investigation

Our ref: 6851-G1
20 May 2022

Your trusted engineering professionals

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For and on behalf of
AssetGeoEnviro

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1. Introduction

1.1 General

This report presents the results of a geotechnical investigation for a Proposed Redevelopment at Abel Tasman Village, Chester Hill NSW (the Site). The investigation was commissioned on 11 April 2022 by Nick Winberg of Centurion Project Management, on behalf of Abel Tasman Village board. The work was carried out in accordance with the proposal by AssetGeoEnviro (Asset) dated 17 March 2022, reference 6851-P1.

Drawings supplied to us for this investigation comprised Architectural Plans (prepared by: Boffa Robertson group; Job no: 2108; dwg: DA00 – DA39; dated: 14 July 2021).

Based on the supplied drawings, we understand that the project involves demolition of existing buildings, with associated aged care facilities and former service station and construction of several residential apartment buildings with a common basement of one level and up to five stories above, and others as standalone buildings with up to three levels above. Excavation of up to about 4m depth is anticipated for the basement construction. Bedrock is expected to be shallow within the site and surrounding area. No significant works beyond the existing building footprint are proposed.

The northern portion of the basement footprint is set back approximately 7m from the north and narrowing to 4.5m towards the eastern site boundary; 4.5m from the upper western and of the upper eastern boundary. On the south-eastern portion of the basement footprint, it is worth noting that no set back has been allowed from the northern, southern, and eastern boundaries at the intersection of Waldron Road and Campbell Hill Road.

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.
- Excavation conditions, methodology and monitoring.
- Subgrade preparation and earthworks.
- Site Classification as per AS2870 'Residential Slabs and Footings' (2011).
- Suitable foundation options.
- Allowable bearing pressure and shaft adhesion for piles.
- Settlement.
- Underpinning.
- Excavation support methodology and design parameters.
- Maximum allowable permanent and temporary batter slopes.
- Groundwater and dewatering.

The following scope of work was carried out to achieve the project objectives:

- 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 five locations to sample and assess the nature and consistency of subsurface soils and bedrock at accessible areas of the Site.
- Carrying out 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. 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 to the north of Waldron Road and at the intersection of Waldron Road and Campbell Hill Road in Chester Hill NSW, as shown in Figure 1. The site is irregular in shape. It has a street frontage of about 87.2m wide on Waldron Road and is about 54.2m wide on Campbell Hill Road. The Site is bounded to the west by open grassland, to the south by Waldron Road, to the East by Campbell Hill Road and to the north by existing residences and further beyond.

Topographically, the Site is in a region of gently undulating terrain in the south-east direction. The local topography in the site has a gentle slope of about 2° to 5° down to the south direction towards Waldron Road. The overall ground surface slopes in the region are about 6-7°.

At the time of the investigation, the Site was occupied by multiple one to two storey, tiled-roof brick-built residence structures within the existing aged care facility, a former service station and an adjacent vacant block to the north of the former service station. All dwellings and existing structures appeared to be at least 15 years old and are in good visual conditions with no obvious signs of ground settlement and cracking observed on external walls. The Site is generally paved with minimal landscape areas within the north-eastern portion of the Site. Access can be made via the paved driveway from Waldron Road along the southern site boundary into the shared parking lot. Based on cursory inspection, the bitumen driveway was also generally in good visual condition. No inspections were undertaken inside the residence units.

According to previous environmental investigations prepared by Environmental Strategies (ref: 16166 RAP (Rev1), dated 23 August 2016) and based on a site walkover observation, the former service station has an interconnected kiosk and automotive workshop building and appears to be over 25 years in age. It comprised of three disconnected fuel pump islands and four identified underground storage tanks (USTs), a former retail shop building and a vacant automotive workshop with outdoor canopy. The former service station is generally surrounded by a hardstand forecourt comprising sealed concrete pavement, generally sloped to the south at approximately 5°. No obvious signs of cracking or ground settlement were observed on the paved area.

Site drainage is observed to be collected by pit and pipe drainage system discharging to various stormwater easements and the municipal stormwater system or by overland flow to the south of the Site.

Soil Landscape type from eSPADE web site for the site is Blacktown Soil Type. This is typified by gently undulating rises on Wianamatta Group shales. The geotechnical hazards associated with this Soil Type include localised seasonal waterlogging, localised water erosion hazard, moderately reactive highly plastic subsoil with localised surface movement potential. Not all of them applies to this site.

3. Fieldwork & Laboratory Testing

3.1 Borehole Investigation

The fieldwork was undertaken from 4 May 2022 to 6 May 2022 under the full-time supervision of a Project Geotechnical Engineer from Asset and included invasive investigation at five locations (BH101 to 105). The test locations are shown in the attached Figure 2.1, 2.2, and 2.3, and were set out by our Geotechnical Engineer by measurements relative to existing site features.

Buried metallic services and utilities within the Site boundaries near the test locations were cleared by an accredited service location subcontractor and by referring to DBYD utility maps.

The invasive investigation included auger boreholes drilled by track-mounted drilling rig to depths of 1.3m to 2.55m below ground level (bgl), discontinued due to reaching practical refusal on less weathered bedrock. Four boreholes (BH101, 102, 103, 105) were continued by NMLC coring techniques to termination at about 7.5m to 8.2m depth. Selected soil samples and recovered rock core were retained for laboratory testing.

The subsurface conditions encountered were logged during drilling and testing. On completion of logging and sampling, a standpipe piezometer was installed in BH105 (construction details attached), and the remaining boreholes were backfilled with a mixture of the drilling spoil and 2mm sand. In the areas of former service station and paved driveway, bitumen and concrete were used to backfill within the top 150mm below the ground surface and trimmed neatly flush or slightly compacted to the adjacent ground surface.

Engineering logs are provided in Appendix B together with their explanatory notes.

3.2 Laboratory Testing

Soil and rock samples recovered during the fieldwork were delivered to a NATA registered laboratory. The following tests were carried out on selected samples:

- Soil aggressivity testing (chloride, sulfate, and pH).
- Point load strength index testing of rock core.

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 Penrith Geological Map indicates the Site is underlain by Bringelly Shale which includes shale, carbonaceous claystone, laminite, fine to medium-grained lithic sandstone and rare coal. These rocks typically weather to form residual clay soils of medium to high plasticity, anticipated to be encountered at about 2m to 3m depth.

4.2 Subsurface Conditions

A generalised geotechnical model for the Site has been developed is shown in Table 1. 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

Unit	Origin	Description	Depth to Top of Unit [1] (m)	Unit Thickness [1] (m)
1	Fill	Fill, Gravelly, cemented SAND/ Clayey SAND with some silt/ Sandy CLAY, fine to medium grained, low to medium plasticity, subangular gravels, dark brown/ brown/ dark grey. Appeared to be moderately to well compacted.	0.04 to 0.125	0.075 to 1.125
2	Residual	CLAY, medium to high plasticity, yellow brown/ brown/ orange brown, generally very stiff, becoming hard with depth.	0.15 to 1.2	0.25 to 1.45
3a	Bedrock[2]	SILTSTONE/ SHALE (remoulded as CLAY with some silt/ Shaley CLAY), extremely weathered, blocky, clayey matrix, remnant rock structure, very low strength, assessed as Class 5 Shale.	1 to 1.6	0.1 to 1.55
3b	Bedrock[2]	SHALE/ SILTSTONE, pale grey/ brown with dark grey lamination, ironstone staining, sub horizontal lamination, generally thinly to very thinly laminated, slightly to moderately fractured, very low to low strength, defect spacing <60mm, assessed as Class 4 Shale.	1.4 to 2.55	1.54 to 3.75
3c	Bedrock[2]	SHALE interbedded with SILTSTONE, dark brown/ brown/ pale grey with dark grey lamination, generally thinly laminated, horizontal lamination, slightly fractured, poorly to well-developed layering, defect spacing 60mm to 200mm, assessed as Class 3 Shale (only in BH101 & BH102).	1.3 to 2.0	3 to 4.93
3d	Bedrock[2]	SILTSTONE interbedded with SHALE, light grey with dark grey lamination, generally very thinly laminated, sub horizontal lamination, high strength, defect spacing >200mm, assessed as Class 2 Shale.	2.94 to 6.3	>1.2 to >5.1

Notes:

1. The depths and unit thicknesses are based on the information from the test locations only and do not necessarily represent the maximum and minimum values across the Site.
2. Rock classification to Pells, P.J.N., Mostyn, G. & Walker, B.F., Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

4.3 Groundwater

Groundwater was not observed in the boreholes during auger drilling to depths of 1.3m to 2.55m bgl. Due to the introduction of water whilst coring, observation of groundwater inflow/levels below auger termination depth was not possible.

Groundwater levels were measured on 17 May 2022, approximately 10 days after installation of the groundwater monitoring well. The result is included on the piezometer details in Appendix B and is summarised in Table 2, together with the groundwater readings from our environmental partner, Reditus for the purpose of environmental contamination assessment.

Table 2 – Groundwater Monitoring Results

Borehole	Collar Level (m AHD)	Date of Reading	Depth to Groundwater (m bgl)	Groundwater Level (m AHD)
MW1 (2014)	8.036	24 January 2014	1.195	6.841
MW1(2022)	8.036	17 May 2022	1.145	6.891
MW2 (2014)	8.0645	24 January 2014	1.270	6.795
MW2(2022)	8.0645	17 May 2022	1.17	6.895
MW3 (2014)	8.31	24 January 2014	1.45	6.860
MW3 (2022)	8.31	17 May 2022	1.345	6.965
BH105 (2022)	8.6	17 May 2022	1.15	7.45

The groundwater readings indicate the groundwater level gently falls to the south-east at a hydraulic gradient of about 1 vertical to 13 horizontal.

It is noted that the groundwater observation may have been made before water levels had stabilised. No long-term groundwater monitoring was carried out.

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. Results of lab testing of recovered rock core are also included in Appendix C and are shown on the engineering borehole logs.

Table 3 – Laboratory Test Results: Aggressivity Assessment

Test Location & Depth (m)	Chloride (mg/kg)	pH	Resistivity (Ω.cm)	Sulfate (mg/kg)	Soil Condition (A* or B†)	Exposure Classification (Concrete) AS 2159-2009	Exposure Classification (Steel) AS 2159-2009
BH101 (1.3-1.5m)	12	5.9	19,000	<10	B	Non-aggressive	Non-aggressive
BH103 (0.5-0.7m)	130	5.3	5,000	56	B	Mild	Non-aggressive
BH104 (1.2-1.4m)	11	8.8	8,900	27	B	Non-aggressive	Non-aggressive
BH105 (1.0-1.2m)	<10	5.7	20,000	38	B	Non-aggressive	Non-aggressive

Notes:

1 Ω.m x 100 = Ω.cm

* Soil conditions A – high permeability soils (e.g., sands and gravels) that are in groundwater

† Soil conditions B – low permeability soils (e.g., silts and clays) or all soils above groundwater

5. Discussions & Recommendations

Based on a lower basement finished floor level of RL 30.24m AHD (up to about 4m below ground level), and from the results of this investigation, it is assessed that the basement level would intercept groundwater within the fractured bedrock or at the soil / rock interface, and most of the basement excavation would be in weathered siltstone ranging from assessed Class 5 Shale to Class 2 Shale at the base of the excavation.

Key geotechnical constraints to the development include excavation condition, groundwater control, temporary shoring, permanent retaining, and foundation conditions. Recommendations for design and construction of the development are provided in the following sections.

5.1 Temporary Shoring

It is understood that permanent batter slopes are not proposed for the development. Temporary batters could likely be accommodated for the north-western majority part of the basement excavation. The proposed depth of excavation and the lack of clearance between the basement and boundary in the south-eastern portion of the site would preclude temporary batters, and therefore temporary shoring will be required for this part of the site. Depending on the design of the shoring, it could also be incorporated into the permanent foundation and retaining works.

Several 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 shotcrete infill panels	Relatively low cost. Strip drains behind shotcrete panels can be used to control groundwater seepage. Can maximise site building space as no temporary wall is required.	Forms a poor seal against groundwater. Greater amount of dewatering required. Potential drawdown of groundwater levels outside of the Site with possible adverse effects on adjacent structures.
2a or 2b	Contiguous or Secant bored piles	Can form part of the permanent structure. Can maximise site building space as no temporary wall is required. Permanent waterproofing 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. Contiguous piles may require additional waterproofing where close contact not achieved.

Based on the advantages and disadvantages listed in Table 4, we recommend Option 1. The spacing of the piles would need to consider the potential for soils and rock wedge failure in-between soldier piles which would likely require a relatively close spacing, say not more than 1.5m centres. A contiguous (Option 2a) or secant (Option 2b) pile wall retention system could also be constructed for the basement excavation but would be more expensive.

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 5 shale/siltstone bedrock was encountered at about 1m to 1.6m depth and assessed Class 4 shale/siltstone was encountered below about 2m depth and improved to Class 2 at about 3m to 6m depth. Practically, it may be difficult to achieve a substantial socket within the assessed Class 3 or better shale. However, adequate overturning resistance is likely to be achieved within the assessed Class 5 to 4 shale rock and the overlying very stiff to hard clays. Control of lateral deflections will also need to be considered (e.g., along the southern and eastern boundaries), where temporary rock anchors may be required.

Design of temporary shoring for carrying vertical loading should be in accordance with Section 5.5, and for lateral pressures, it should be in accordance with Section 5.7.

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.

5.2 Earthworks

5.2.1 Excavation

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

Excavation within the less weathered bedrock will likely require the 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.

5.2.2 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 the 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, per 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 900kg 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 the 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 consider acceptable noise limits as per the "Interim Construction Noise Guideline" (NSW EPA).

5.2.3 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 the 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 to design subgrade level, 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.

5.2.4 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 the 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 lightweight equipment (e.g., hand-operated plate compactor or ride-on compactor not more than 3 tonnes static weight) to limit compaction-induced lateral pressures.

Any soils to be imported onto the Site for backfilling and reinstatement 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. Asset can provide further advice on this matter if required.

5.2.5 Batter Slopes

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

Table 7 – Recommended Maximum Dry Batter Slopes

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

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

5.3 Site Classification for Ancillary Structures

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". This requires that footings be designed from first principles, rather than adopting prescriptive designs as per AS2870-2011. Where footings are founded on the underlying natural shale bedrock, then footings may be designed and constructed in accordance with the requirements in AS2870-2011 for a Class A site.

Footings should also be designed as per the recommendations in Section 5.5.

The classification and footing recommendations given above and in Section 5.5 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.

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

5.4 Aggressivity

5.4.1 Aggressivity to Concrete

The laboratory test results indicate that the soils are classified as "mild" with respect to concrete piles in soil (as per AS2159-2009 Piling-Design and Installation).

In accordance with AS 2159-2009 Section 6.4 Design for Durability of Concrete Piles (broadly applicable to buried concrete structures), "For the range of chemical conditions in the soil surrounding the piles, the condition leading to the most severe aggressive conditions shall be allowed for."

Therefore, for cast-in-place piles and structures with a 50-year design life, a minimum concrete strength of 32MPa and a minimum cover to reinforcement of 60mm is recommended for a "mild" environment in AS2159-2009 for concrete piles and buried structures. The cover should be increased to 75mm for a 100-year design life.

Similarly, for pre-cast piles & structures with a 50-year design life, a minimum concrete strength of 50MPa and a minimum cover to reinforcement of 20mm is recommended for a "mild" environment in AS2159-2009 for concrete piles and buried structures. The cover should be increased to 30mm for a 100-year design life.

The concrete strength and cover requirements are a minimum and should be reviewed by the designer / structural engineer to take other design considerations into account.

5.4.2 Aggressivity to Steel

The laboratory test results indicate that the soils are classified as "non-aggressive" with respect to steel piles (as per AS2159-2009 Piling-Design and Installation).

Corrosion allowance, coating protection systems, and cathodic protection should be adopted as per AS2159-2009 for a "non-saline" exposure classification, with a uniform corrosion allowance of <0.01mm/year.

5.5 Footings

Suitable footings might comprise a slab on ground for the basement area and pier and beam footings supporting the upper building loads. Where some footings are taken to bedrock, it is recommended that all footings are founded on bedrock to reduce the risk of differential settlement due to variable founding conditions.

Edge beams for slabs, 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*	Typical E_{field} MPa
Class 5 Shale	700	70	35	2,100	100	50	50-300
Class 4 Shale	1,000	100	50	3,000	150	75	100-500
Class 3 Shale	3,000	300	150	9,000	500	250	200-1,200
Class 2 Shale	6,000	600	300	18,000	800	400	700-2,000

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.

Clean socket of roughness category R2 or better is assumed.

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 considers 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 may require liners to support the overburden soils and highly fractured rock, although some over break and minor fretting should be allowed for. Groundwater may be expected within bored pile holes and dewatering by a 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 the Site. CFA piles are considered non-displacement piles as defined in AS2159.

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.

5.6 Groundwater Control

Limited groundwater observations made for this investigation are described in Section 4.3. The observations indicate that groundwater will need to be considered for the proposed development. Good practice should be followed to cater for potential groundwater, such as incorporating suitable temporary groundwater control during construction and designing retaining walls with adequate subsoil drainage. Further geotechnical advice must be sought if significant groundwater is encountered during construction.

5.6.1 Design Groundwater Level

Limited groundwater observations made for this investigation are described in Section 4.3. If groundwater levels are critical to the design, construction, and long-term operation of the basement, it is recommended that an appropriate design groundwater level is confirmed during detail design. This would normally require long-term monitoring of groundwater level fluctuations covering periods of statistically significant climatic conditions, and consideration of potential 'damming' effects of the development on groundwater levels. In the absence of such further information, it is suggested that a design groundwater level nominally 0.5m above the water levels observed for this investigation (i.e., depths of 0.7m to 1.0m below ground level) could be used for preliminary design purposes.

5.6.2 Potential Impacts of Dewatering

Temporary lowering of the groundwater level (e.g., for construction purposes) can cause settlement of the soil profile due to a change in the stress regime. The magnitude of settlement depends on the soil type and condition, draw-down depth and duration, and historical water levels.

The groundwater appears to be present within rock fractures and may also be perched within the soils above bedrock. Dewatering is unlikely to cause significant settlement that would adversely affect adjoining development and infrastructure.

Temporary dewatering during construction will likely be required. Permanent dewatering could be considered, subject to acceptance by regulatory authorities.

5.6.3 Estimated Dewatering Rates

The quantity of seepage expected to flow into the excavation during construction is unknown. 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 residual clays and referring to empirical charts, we anticipate that the permeability of the 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 spear points would likely be sufficient to permit excavation works. Piezometers external to the basement excavation and near the Site boundaries should be provided to monitor groundwater levels during dewatering. Only experienced dewatering subcontractors with appropriate monitoring systems should be considered. We recommend further involvement of an experienced Geotechnical Engineer and Hydrogeologist during the design, construction, and operation/monitoring of groundwater control systems.

If seepage of groundwater or perched water occurs through the fractured bedrock during excavation, we consider that dewatering via conventional sump-and-pump methods would be achievable. Further geotechnical advice should be obtained if higher inflows are encountered which cannot be controlled via this method.

5.6.4 Regulatory Authority Requirements

The proposed basement will intercept the groundwater, which constitutes ‘Aquifer Interference’ as defined in the Water Management Act 2000. A more detailed assessment of the potential impacts is therefore required. Analysis of potential drawdown and groundwater volume take is required, along with presentation of baseline groundwater quality data. This assessment must be submitted to Water NSW.

If it can be demonstrated through desk-top analysis that the potential impacts do not exceed the 'Minimal Impact' considerations in the NSW Aquifer Interference Policy (AIP), then the impacts will be considered as acceptable to Water NSW.

If the potential impacts exceed the 'Minimal Impact' considerations, then additional studies including more groundwater monitoring and testing, and more rigorous groundwater modelling will be required to further assess the potential impacts. If this further assessment demonstrates that the predicted impacts do not prevent the long-term viability of the dependent ecosystem, significant site, or affected water supply works, then the impacts will be considered as acceptable to Water NSW.

Where the potential impacts are acceptable to Water NSW, it is expected that an application to be submitted for "Approval for Water Supply Works and/or Water Use" will need to be prepared and submitted to Water NSW, via Council Consent Conditions. A Dewatering Management Plan may also be required to be submitted, which 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.

The application to Water NSW would also need to reference the permanent groundwater control proposed for the development. It is noted that Water NSW generally does not prefer permanent dewatering but leaves that to the local Council to determine. Further advice should be sought if it is proposed to adopt a permanently dewatered basement.

5.7 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.

5.7.1 Excavation Support Construction Methodology

Where temporary or permanent batter slopes as per Section 5.2.5 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 basement 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.

5.7.2 Excavation Support Design Parameters

Support system design may be based on the parameters given in Table 9. Cantilever walls or walls with 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

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	18.0	0.35	0.5	N/A
Class 5 Shale (3)	20.0	0.2	0.5	4
Class 4 Shale (3)	22.0	0.1	0.5	10
Class 3 Shale (3)	23.0	0.0	0.4	20

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 consider strain incompatibility between active and passive pressure conditions. Parameters assume horizontal backfill and no back of wall friction.
3. 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 the 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 $[0.65 \times \gamma \times \mathrm{H} \times K_a]$ where γ = unit weight of the retained material, H = height of the 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.

5.7.3 Surcharge

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

5.7.4 Hydrostatic Pressure

If tanked retaining walls are to be adopted, they should be designed for a hydrostatic pressure based on an appropriate design groundwater level (refer to Section 5.6).

If a permanently dewatered basement is to be adopted, then there may still be some hydrostatic pressure acting on the wall depending on the drainage system design capacity and the local variations in groundwater seepage rates. In the absence of further information, it is suggested that allowance for a water pressure of half of the wall height be adopted for preliminary design.

5.7.5 Underpinning

Where excavations (e.g., for new footings and underground storage area) extend below the 'zone of influence' of existing footings, then underpinning will be required. The 'zone of influence' is defined as a line extending downwards and outwards from the toe of the existing footing at an angle which is dependent on the nature and condition of the foundation soils. For the rock anticipated beneath the existing footings, an angle of 45° may be adopted. Further investigation of existing footing depths is recommended by carrying out inspection at the commencement of construction. The timing/programme of geotechnical inspections for further assessment of footings adjacent to proposed excavation should be nominated by the Geotechnical Engineer prior to the 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. Requirements for rock support must be nominated or approved by the Geotechnical Engineer during construction.

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

5.8 Potential Impacts on Adjacent Developments

Potential geotechnical risks of construction on adjoining developments could include; vibration effects due to rock excavation, settlement/deflection of adjacent footings due to the basement excavation, and induced settlement due to groundwater drawdown. 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 effects are anticipated to have negligible impact and be within acceptable limits.

5.9 Site Classification – Earthquake Actions

In accordance with the earthquake loading standard, AS1170.4 (2007), this site has a site sub-soil Class Be – Rock site, as not more than 3 m depth of soil or highly weathered rock with an Unconfined Compressive Strength (UCS) not more than 1MPa is present, and the underlying rocks have a UCS between 1 and 50MPa.

A Hazard Factor, z, of 0.08 for Sydney region is recommended.

6. Geotechnical & Hydrogeological Monitoring Program

6.1 Acceptable Vibration & Deflection Limits

The contractor shall carry out excavation and construction activities so that the limits in Table 10 are not exceeded:

Table 10 – 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

6.2 Monitoring System

6.2.1 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, and adjoining property or movement-sensitive buried services are present within the depth-of-influence of the excavation. The depth-of-influence is defined as a line extending upwards and outwards at 45° above horizontal from the base of the excavation.

Survey points shall be installed at a spacing of not more 5m. Survey measurements shall be taken:

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

6.2.2 Vibration

Where excavation is carried out in accordance with Section 5.2.1, adopting a methodology for a maximum peak particle velocity of 5 mm/s, a permanent vibration monitoring system should not be required during the excavation works. However, we recommend that vibration levels at critical adjoining developments be measured at the commencement of rock excavation to confirm that vibrations being generated are below the target values and to provide guidance on modifying excavation techniques if the target values are exceeded.

6.3 Hold Points

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

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

6.4 Contingency Plan

If 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.

7. 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. 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 only, which could range from a few days (for temporary batter slopes) up to six months (for temporary shoring). This 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 (e.g., Council) or are responsible for services that may be within or adjacent to the Site (e.g., Sydney Water), for their review.

Asset accepts no liability where our recommendations are not followed or are only partially followed. 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.

SOURCE: SIX MAPS

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N

issue	date	description
A	20.5.22	INITIAL ISSUE

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Proposed Redevelopment
Abel Tasman Village
Chester Hill NSW
for
Centurion Project Management

Site Locality

drawn: JW
date: 20.5.2022
checked: AT
scale: 1:4,000 A4

job no.: 6851
fig: 1
issue: A

LEGEND ⌖ CURRENT GEOTECHNICAL BOREHOLE

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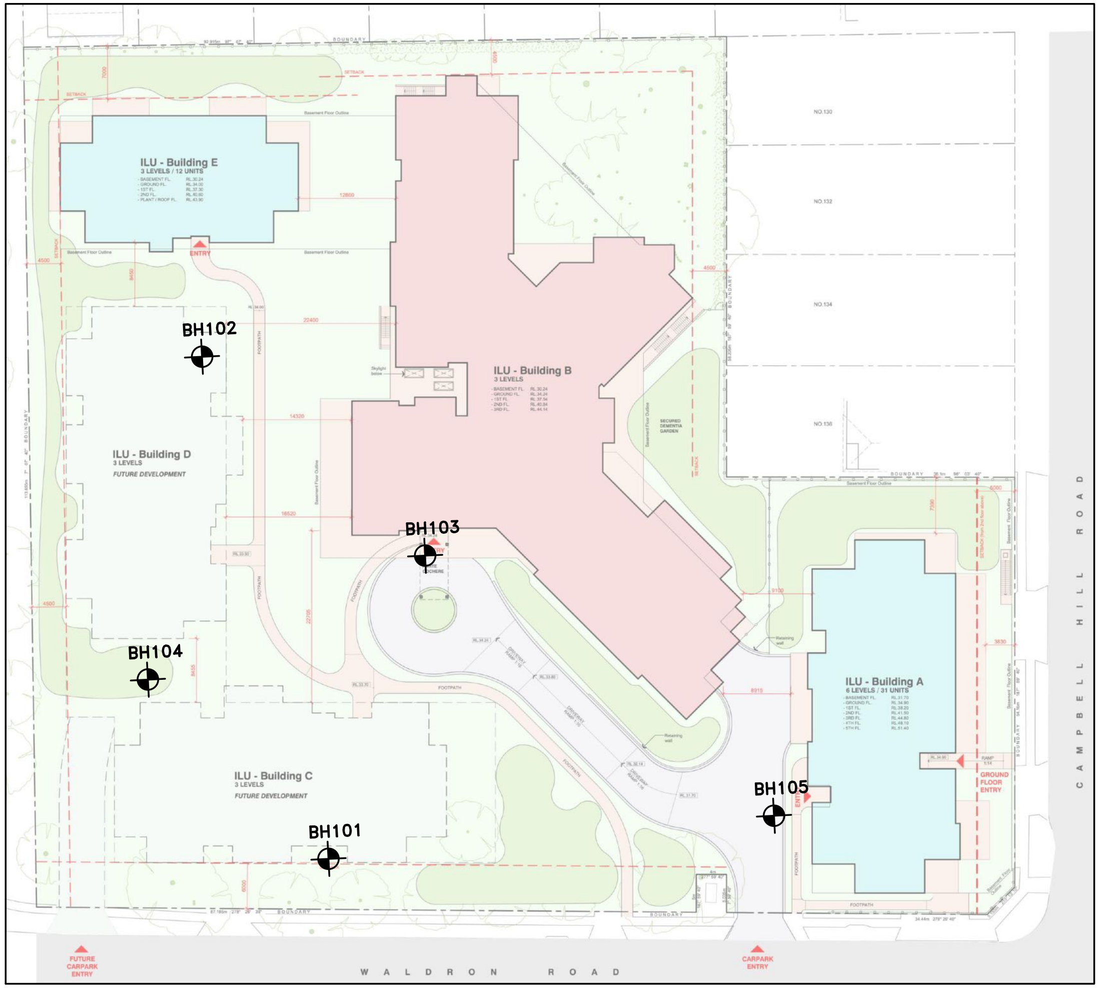
Test Locations – Air Photo

drawn: JW
date: 20.5.2022
checked: AT
scale: 1:1000 A4

job no.: 6851
fig: 2.1
issue: A



LEGEND ⌖ CURRENT GEOTECHNICAL BOREHOLE

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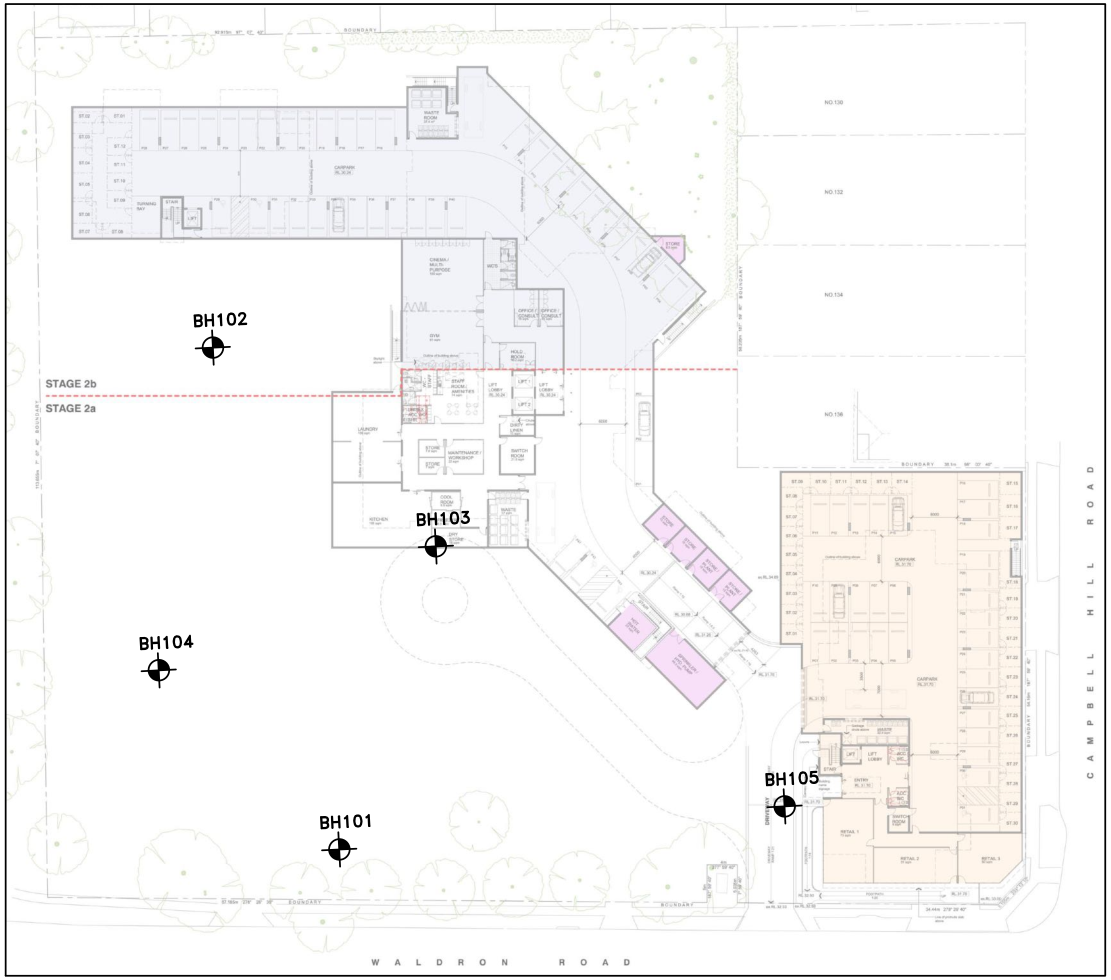
Chester Hill NSW
for
Centurion Project Management

Test Locations – Site Plan

drawn: MAB	**job no.:** 6851
date: 20.5.2022	
checked: AT	**fig:** 2.2
scale: 1:1000 A4	**issue:** A



LEGEND ⊕ CURRENT GEOTECHNICAL BOREHOLE

0 1:1000 @ A4 50m

APPROXIMATE ONLY – SUBJECT TO DETAIL SURVEY.
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Test Locations – Basement 1 Plan

drawn: MAB
date: 20.5.2022
checked: AT
scale: 1:1000 A4
job no.: 6851
fig: 2.3
issue: A

Appendix A

Important Information about your Geotechnical Report
CSIRO BTF 18

Important Information about your Geotechnical Report

assetgeoenviro

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

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).

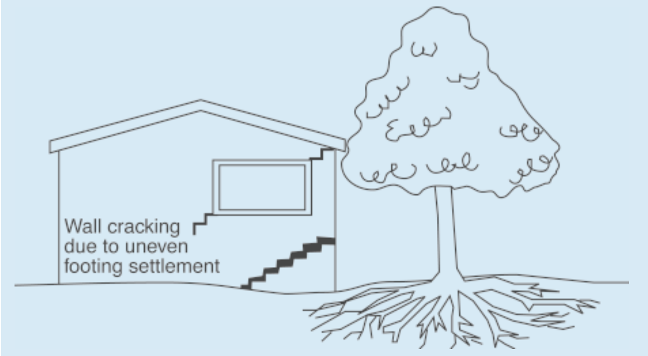
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

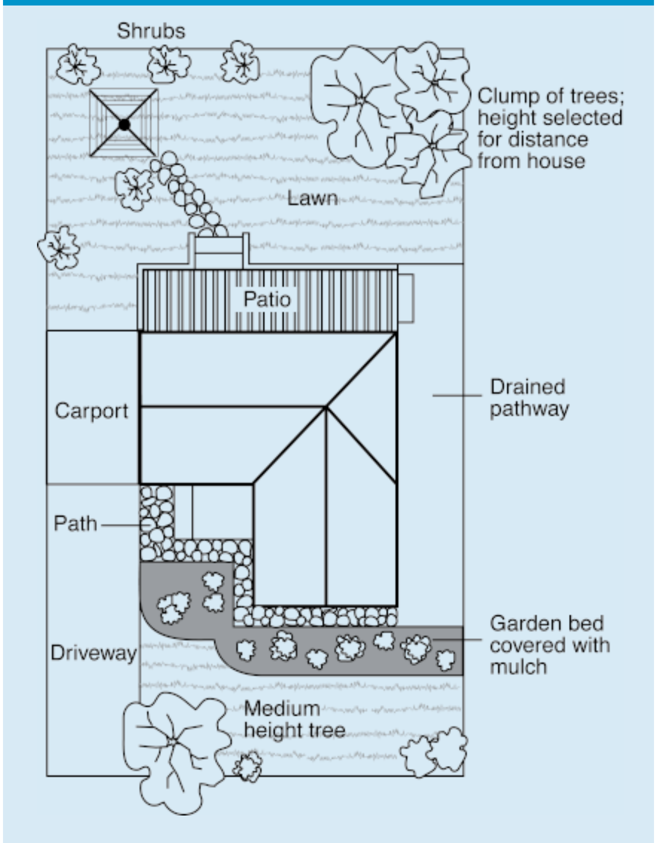
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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
Core Photographs
Piezometer Details

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Log Abbreviations & 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	Gravel and gravel-sand mixtures, little or no fines.
GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels
GM	Gravel-silt mixtures and gravel-sand-silt mixtures.
GC	Gravel-clay mixtures and gravel-sand-clay mixtures.
SW	Sand and gravel-sand mixtures, little or no fines.
SP	Sand and gravel sand mixtures, little or no fines.
SM	Sand-silt mixtures.
SC	Sand-clay mixtures.
ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity.
CL, CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays.
OL	Organic silts
MH	Inorganic silts
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity, organic silt
PT	Peat, highly organic soils.

MOISTURE CONDITION

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

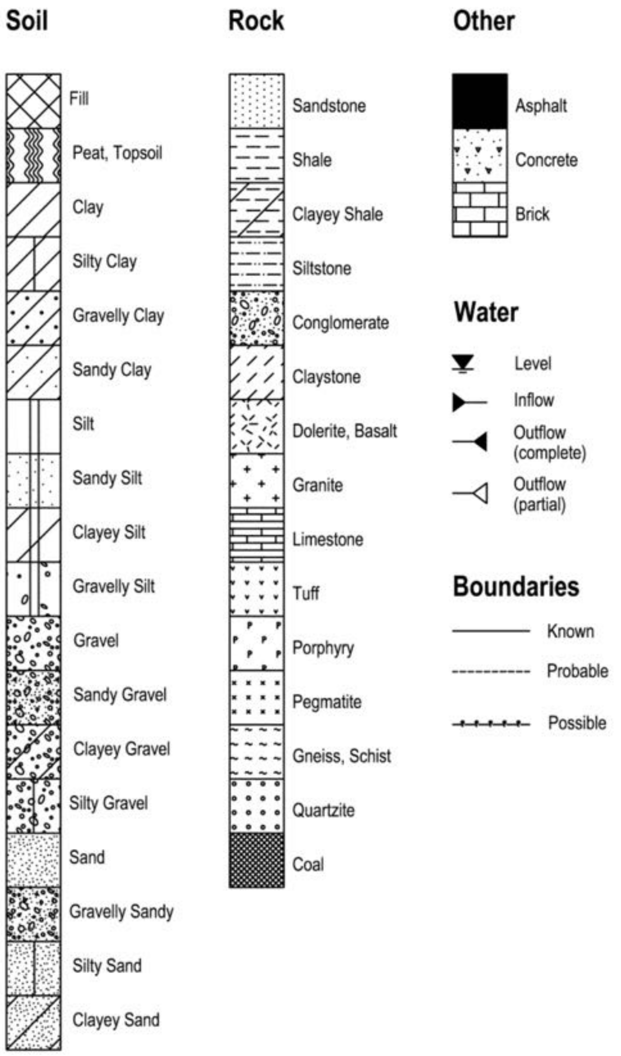
CONSISTENCY

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

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

RQD (%)

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

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)

AS1726-2017

Soils and rock are described in the following terms, which are broadly in accordance with AS1726-2017.

Soil

MOISTURE CONDITION

Term	Description
Dry	Looks and feels dry. Fine grained and cemented soils are hard, friable or powdery. Uncemented coarse grained soils run freely through hand.
Moist	Soil feels cool and darkened in colour. Fine grained soils can be moulded. Coarse soils tend to cohere.
Wet	As for moist, but with free water forming on hand.

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 FINE-GRAINED 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		

RELATIVE DENSITY OF COARSE-GRAINED 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	19 – 63
	medium	6.7 – 19
	fine	2.36 – 6.7
Sand	coarse	0.6 – 2.36
	medium	0.21 – 0.6
	fine	0.075 – 0.21
Silt & Clay		< 0.075

MINOR COMPONENTS

Term	Proportion by Mass: *coarse grained*	*fine grained*
Trace	≤ 15%	≤ 5%
With	>15% – ≤30%	>5% – ≤12%

SOIL ZONING

Layers	Continuous across exposures or sample.
Lenses	Discontinuous, lenticular shaped zones.
Pockets	Irregular shape zones of different material.

SOIL CEMENTING

Weakly	Easily broken up by hand pressure in water or air.
Moderately	Effort is required to break up by hand in water or in air.

USCS SYMBOLS

Symbol	Description
GW	Gravel and gravel-sand mixtures, little or no fines.
GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels.
GM	Gravel-silt mixtures and gravel-sand-silt mixtures.
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OL	Organic silts
MH	Inorganic silts
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity, organic silt
PT	Peat, 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	Material is weathered to an extent that it has soil properties. Rock structures are no longer visible, but the soil has not been significantly transported.
Extremely …..	Material is weathered to the extent that it has soil properties. Mass structures, material texture & fabric of original rock is still visible.
Highly …..	Rock strength is significantly changed by weathering; rock is discolored, usually by iron staining or bleaching. Some primary minerals have weathered to clay minerals.
Moderately …..	Rock strength shows little or no change of strength from fresh rock; rock may be discolored.
Slightly …..	Rock is partially discolored 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.

Borehole Log

assetgeoenviro

BH no:	**BH101**
sheet:	1 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	4.5.2022
principal:		**finished:**	4.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	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 penetrometer kPa (100 200 300 400)	structure and additional observations
ADT	N	None observed during augering						Bitumen.	--	--		BITUMEN.
					0.075		GPS	FILL, gravelly, cemented SAND, fine to medium grained, subangular gravels, brown/ dark grey.	D	MD?		FILL. Appeared to be well-compacted.
					0.15		CL-CH	CLAY, medium to high plasticity, yellow brown with traces of pale grey.	<Wp	VSt	× 120 (at 1.0)	RESIDUAL.
			D (1.3–1.5)									
					1.6			SHALE/SILTSTONE, HW-XW, clayey, silty matrix, light grey, very low strength, assessed as Class 5 Shale/Siltstone.				BEDROCK. High TC-bit resistance.
					2.0			Borehole No: BH101 continued as cored hole from 2m				Auger TC-bit refusal @ 2.0m on less weathered Shale.

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

6851 BOREHOLE LOGS.GPJ 21/5/22

A: 2.06 / 56 Delhi Road, North Ryde NSW 2113 P: 02 9878 6005 W: assetgeoenviro.com.au

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Cored Borehole Log

BH no:	**BH101**
sheet:	2 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	4.5.2022
principal:		**finished:**	4.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	
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	estimated strength (MPa: EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	$Is_{(50)}$ MPa (× D=diametral, o A=axial)	RQD %	defect spacing mm (20, 60, 200, 600, 2000)	defect description (type, inclination, thickness, shape, roughness, coating; specific / general)
				1.5, 1.6, 2.0		Continued from non-cored borehole from 2m						
NMLC				2		SHALE, brown/ pale brown with dark grey lamination, generally thinly laminated, subhorizontal lamination, slightly fractured.	SW					
				2.18		CLAY INFILL.						
				2.22		SHALE, dark grey with brown layering, generally thinly laminatedm subhorizontal lamination, slightly fractured.	MW					
				2.5, 2.67		CLAY INFILL.						JT, 87°, sm, cl, pl.
				2.71, 3.0, 3.5		SHALE, dark grey/ brown/ dark brown with some pale grey, generally thinly laminated, subhorizontal lamination, slightly fractured.	MW			81		FZ
				3.9, 4.0, 4.5, 5.0		SHALE, dark grey/ brown/ pale grey, generally thinly laminated, subhorizontal lamination, moderately fractured, ironstained, well-developed layering.	MW - HW		D=0.02 A=0.05	88		PT, 5°, PT, un, ro, filled, ct. FZ JT, 68°, un, ro, filled, clay. PT, 0°, un, ro, closed. JT, 65°, un, ro, filled. PT, 0-3°, un, ro, filled.
				5.4, 5.5, 6.0		SHALE/ SILTSTONE, pale grey with dark grey lamination, subhorizontal to horizontal lamination, generally thinly to very thinly laminated.	SW - FR					PT, 0-3°, un, ro, filled, ct.

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Cored Borehole Log

assetgeoenviro

BH no:	BH101
sheet:	3 of 3
job no.:	6851

client:	Centurion Project Management	started:	4.5.2022
principal:		finished:	4.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	
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	estimated strength (MPa: EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	$Is_{(50)}$ MPa (× D=diametral, o A=axial)	RQD %	defect spacing mm (20 60 200 600 2000)	defect description (type, inclination, thickness, shape, roughness, coating; specific / general)
NMLC				6.5, 7.0		SHALE/ SILTSTONE, pale grey with dark grey lamination, subhorizontal to horizontal lamination, generally thinly to very thinly laminated. *(continued)*	SW - FR		D=0.37 A=0.43	88 95		PT, 0°, un, ro, tight. PT, 0°, un, ro, tight. XW, SM, 20mmt. PT, 3-5°, un, ro, cl. VJ, 87°, pl, sm, cl. XW, Sm, 20mmt. PT, 0°, un, ro, filled. PT, 5-8°, un, ro, filled.
				7.3 7.5, 8.0		Sandstone interbedded with SILTSTONE, light grey with dark grey lamination, generally very thinly laminated, subhorizontal lamination.	FR		D=1.06 A=5.12			
				8.12 8.5, 9.0, 9.5, 10.0, 10.5, 11.0		NMLC terminated @ 8.12m, target depth. BH101 terminated at 8.12m						

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A	11/5/22	INITIAL ISSUE

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PROPOSED REDEVELOPMENT,
Abel Tasman Village
Chester Hill NSW
for
Centurion Project Management

CORE PHOTO – BH101

drawn: JW
date: 11.5.22
checked: AT
scale: 1:4 @ A4

job no.: 6851
fig: –
issue: A

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Borehole Log

BH no:	**BH102**
sheet:	1 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	4.5.2022
principal:		**finished:**	4.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	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	N	None observed during auger drilling			0.04	■		PAVERS.	--	--		PAVER
						▩	SP	FILL, SAND with some clay/ Sandy CLAY, dark brown/ brown, fine to medium grained,15-20% gravels, subangular gravels, low to medium plasticity.	D	D?		FILL. (SUBBASE + SUBGRADE). Appeared to be moderately compacted.
					0.5							
					1.0							
					1	▨	CL-CH	CLAY, medium to high plasticity, red brown/ pale grey/ pale brown.	<Wp	VSt?		RESIDUAL.
								Borehole No: BH102 continued as cored hole from 1.3m				Auger TC-bit refusal @ 1.3m on less weathered Shale
					1.5							
					2.0							
					2.5							
					3.0							
					3.5							
					4.0							
					4.5							
					5.0							

6851 BOREHOLE LOGS.GPJ 21/5/22

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Cored Borehole Log

BH no:	**BH102**
sheet:	2 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	4.5.2022
principal:		**finished:**	4.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	
diameter:	125mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

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

method	depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	$Is_{(50)}$ MPa (D=diametral ×, A=axial o)	RQD %
	1 – 1.3	Continued from non-cored borehole from 1.3m			
NMLC	1.3	SILTSTONE, brown/ pale grey, clay bands, thinly to medium bedded, slightly fractured.	MW	D=0.69, A=1.17	93
	2	SILTSTONE, pale grey/ brown with orange brown/ red brown layering, slightly fractured, ironstained.	SW - MW		
	2.8	SHALE interbedded with SILTSTONE, dark grey/ dark brown/ pale grey with dark grey lamination, generally thinly laminated, subhorizontal lamination, slightly fractured, ironstained.	MW	D=0.01, A=0.14	92
	4.18	SHALE interbedded with SILTSTONE, dark brown/ brown/ pale grey with dark grey lamination, generally thinly laminated, horizontal lamination, slighly fractured, poorly to well-developed layering.		D=0.03, A=0.52	95

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

- PT, stepped, filled, vr.
- clay infill.
- PT, 3-5°, un, ro, filled, clay.
- PT, 3-5°, un, ro, cl.
- PT, 0-3°, un, ro, filled, clay.
- FZ
- XW, SM, 20mmt.
- PT, 0-3°, un, ro, cl.
- JT, 10°, un, ro, cl.
- JT, curved, un, ro, cl.
- PT, 0-3°, un, ro, cl.
- XW, SM, 30mmt.
- PT, 0-3°, un, ro, closed.
- XW, SM, 30mmt.
- curved, un, ro, cl.
- JT, 10°, stepped, ro, st.

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Cored Borehole Log

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BH no:	BH102
sheet:	3 of 3
job no.:	6851

client:	Centurion Project Management	started:	4.5.2022
principal:		finished:	4.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	
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	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							MW			95		JT, 5-7°, un, ro, cl. JT, 55°, un, ro, cl.
				6.23 6.5 7.0		SHALE interbedded with SILTSTONE, dark grey/ brown grey with pale grey lamination, generally thinly laminated, horizontal lamination, slighly fractured.	SW - MW			96		FZ XW, SM, 30mmt. VJ, 89°, un, ro, filled.
				7.5 7.44 8.0		SILTSTONE interbedded with SHALE, pale grey/ dark grey, generally thinly to very thinly laminated, horizontal lamination.	SW - FR		D=0.22 A=1.24			
				8.19 8.5 9.0 9.5 10.0 10.5 11.0		NMLC terminated @ 8.19m, target depth. BH102 terminated at 8.19m						

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Chester Hill NSW
for
Centurion Project Management

CORE PHOTO – BH102

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date: 11.5.22
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job no.: 6851

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issue: A

Borehole Log

BH no:	BH103
sheet:	1 of 3
job no.:	6851

client:	Centurion Project Management	started:	4.5.2022
principal:		finished:	5.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	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	N	None Observed during auguring			0.0		CL	TOPSOIL, CLAY with some silt, medium to high plasticity, dark brown, rootlets.	<Wp	St-VSt?		TOPSOIL.
			D		0.3		CH	CLAY, medium to high plasticity, brown/ orange brown.				RESIDUAL.
					1.3			SHALE/ SILTSTONE, brown, grading to pale grey, XW-HW (remoulded to CLAY with some silt), very low strength.				BEDROCK.
					1.4			Borehole No: BH103 continued as cored hole from 1.4m				Auger TC-bit refusal @ 1.4m on less weathered bedrock.

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

6851 BOREHOLE LOGS.GPJ 21/5/22

Cored Borehole Log

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BH no:	BH103
sheet:	2 of 3
job no.:	6851

client:	Centurion Project Management	started:	4.5.2022
principal:		finished:	5.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	
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	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 o)	RQD %	defect spacing mm (20, 60, 200, 600, 2000)	defect description (type, inclination, thickness, shape, roughness, coating; specific / general)
				1.3		Continued from non-cored borehole from 1.4m						
NMLC				1.4		SHALE interbedded with SILTSTONE, generally thinly laminated, 5mm spacing, slightly to moderately fractured, dark grey/ dark brown/ orange brown, iron staining, remnant rock structure.	MW - HW		D=0.01 A=0.23	55		PT, 0°, pl, sm, cl. FZ PT, 0°, un, ro, cl. JT, 45°, un, ro, filled. PT, 0-3°, un, ro, cl.
				2.94		SILTSTONE interbedded with SHALE, brown/ dark brown with dark grey laminaiton, subhorizontal lamination, 5mm spacing, generally thinly laminated, ironstone staining.	SW			95		PT, 4°, un, ro, st. PT, 5-7°, un, ro, cl.
				4.1		SHALE, dark grey/ dark brown with brown layering and dark grey lamination, subhorizontal lamination, 3-5mm spacing, generally thinly laminated, ironstone staining, slightly fractured.			D=0.28 A=0.3	99		PT, 3-5°, un, ro, cl. PT, 3-5°, un, ro, cl. XW, Sm, 40mmt. VJ, 87°, un, ro, cl.

Depth scale: 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

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client:	Centurion Project Management	started:	4.5.2022
principal:		finished:	5.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	
diameter:	125mm inclination: -90° bearing: --- E: N:	datum:	AHD

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

depth metres	rock substance description (rock type; grain characteristics, colour, structure, minor components)	weathering	estimated strength	Is(50) MPa (D=diametral, A=axial)	RQD %	defect description (type, inclination, thickness, shape, roughness, coating)
to 6.14		SW	M		99	PT, un, ro, 7°, filled.
6.14	SHALE, darkk grey, clayey, blocky matrix, 80% Shale; 20% clay, remnant rock structure.	MW - HW	VL			JT, 48°, un, ro, filled, ct. XW, SM, 30mmt.
6.45	SILTSTONE, pale grey with dark grey lamination, horizontal lamination, generally very thinly laminated.	SW - FR	H	D=0.44 A=0.94 D=0.55 A=0.96	85	PT, 3°, un, ro, cl. JT, 8°, un, ro, filled, ct.
8.09	NMLC terminated @ 8.09m, target depth. BH103 terminated at 8.09m					

Depth scale: 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0

issue	date	description
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Abel Tasman Village
Chester Hill NSW
for
Centurion Project Management

CORE PHOTO – BH103

drawn: JW
date: 11.5.22
checked: AT
scale: 1:4 @ A4

job no.: 6851
fig: –
issue: A

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Borehole Log

BH no:	**BH104**
sheet:	1 of 1
job no.:	6851

client:	Centurion Project Management	**started:**	5.5.2022
principal:		**finished:**	5.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	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 penetrometer kPa (100 200 300 400)	structure and additional observations
ADT	N	None observed during auger drilling						Bitumen.	--	--		BITUMEN.
					0.075		CL	FILL, CLAY with some silt, brown, red brown, pale grey with traces of yellow brown, medium to high plasticity, trace of fines.	M	F-St?		FILL. Appeared to be moderately compacted.
			D		1.2		CL-CH	CLAY, red brown/ orange brown/ yellow brown, medium plasticity.	,Wp	St-VSt?		RESIDUAL.
					1.45			SILTSTONE/ SHALE, XW, very low to low strength, clayey, blocky matrix, remnant rock structure.				BEDROCK.
					1.82			Borehole No: BH104 terminated at 1.82m				Auger TC bit refusal on less weathered bedrock

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

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Borehole Log

BH no:	**BH105**
sheet:	1 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	5.5.2022
principal:		**finished:**	6.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	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 penetrometer kPa (100 200 300 400)	structure and additional observations
DT	N	None observed during augering						PAVERS.	--	--		PAVERS.
ADT					0.125		CL	CLAY with some silt and trace of sand, some gravels, 10-15% gravels, subangular gravels, dark brown/ brown/ yellow brown/ pale grey.	Wp	F-St?		FILL.
					0.35		CH	CLAY, brown, high plasticity.		St?		RESIDUAL.
			D		1		CL	SILTSTONE/ SHALE, XW, very low strength, (remoulded as CLAY with some silt with traces of fines), brown grey, low to medium plasticity, assessed as Class 5 Shale/Siltstone.				BEDROCK.
					2.1		SHALEY CLAY	SHALEY CLAY, XW SHALE, dark grey, low plasticity, blocky, clayey matrix, remnant rock structure, very low strength, assessed as Class 5 Shale.				
								Borehole No: BH105 continued as cored hole from 2.55m				Auger TC-bit refusal @ 2.55m on less weathered bedrock

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

6851 BOREHOLE LOGS.GPJ 21/5/22

Cored Borehole Log

BH no:	**BH105**
sheet:	2 of 3
job no.:	6851

client:	Centurion Project Management	**started:**	5.5.2022
principal:		**finished:**	6.5.2022
project:	Proposed Redevelopment	**logged:**	AT
location:	Abel Tasman Village	**checked:**	MAB
equipment:	Geo205 Tracked Rig	**RL surface:**	
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	estimated strength (MPa: EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH)	$Is_{(50)}$ MPa (× D=diametral, o A=axial)	RQD %	defect spacing mm (20 60 200 600 2000)	defect description (type, inclination, thickness, shape, roughness, coating; specific / general)
				2.1 – 2.55		Continued from non-cored borehole from 2.55m						
NMLC				2.55		SILTSTONE, light grey with dark grey lamination, generally very thinly laminated, horizontal lamination.	SW - FR		D=1.76			PT, 0-3°, un, ro, ct.
				2.7		SILTSTONE interbedded with SHALE, brown with dark grey lamination, orange brown layering, generally very thinly laminated, subhorizontal lamination, clay bands.	MW		A=2.27; D=0.07; A=0.13	75		FZ, 20mmt.; VJ, 85-87°, un, ro, cl.; FZ, 20mmt. Clay Infill; PT, 0°, un, ro, cl.
				3.5		SHALE/ SILTSTONE, pale grey/ dark grey with dark grey lamination, generally very thinly laminated, subhorizontal lamination.	SW - MW					JT, 10°, un, ro, st.
				3.95		SHALE, clayey/blocky matrix, dark grey/ dark brown, remnant rock structure.	XW			88		HFZ
				4.09		SHALE interbedded with SILTSTONE, pale grey/ brown with dark grey lamination, ironstone staining, subhorizontal lamination, generally thinly to very thinly laminated.	MW		D=0.49; A=0.06; D=0.34; A=0.45			PT, 0°, pl, sm, st.; PT, 0°, un, ro, st.; JT, 67°, un, ro, st.; JT, 67°, un, ro, st.; XW, SM, 20mmt.; PT, 0-3°, un, ro, filled, clay.; JT, 20°, un, ro, filled.
				5.4		SHALE/SILTSTONE, brown grading to dark grey, clayey, blocky matrix, remnant rock structure.	MW - HW			94		JT, 72°, un, ro, cl.
				6.3 – 7.0		SILTSTONE interbedded with SHALE, light grey with dark grey lamination, generally very thinly laminated, subhorizontal lamination.	FR		D=0.4; A=1.61			Pt, 0-3°, un, ro, cl.; Pt, 0-3°, un, ro, cl.; Pt, 0-3°, un, ro, cl.; Pt, 0-3°, un, ro, st.

Depth scale: 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0

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Cored Borehole Log

BH no:	BH105
sheet:	3 of 3
job no.:	6851

client:	Centurion Project Management	started:	5.5.2022
principal:		finished:	6.5.2022
project:	Proposed Redevelopment	logged:	AT
location:	Abel Tasman Village	checked:	MAB
equipment:	Geo205 Tracked Rig	RL surface:	
diameter:	125mm inclination: -90° bearing: --- E: N:	datum:	AHD

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, o A=axial)	RQD %	defect spacing mm (20 60 200 600 2000)	defect description (type, inclination, thickness, shape, roughness, coating; specific / general)
NMLC				7.5		SILTSTONE interbedded with SHALE, light grey with dark grey lamination, generally very thinly laminated, subhorizontal lamination. *(continued)*	FR			94		
				7.5		NMLC terminated @ 7.5m, target depth. BH105 terminated at 7.5m						
				8.0								
				8.5								
				9.0								
				9.5								
				10.0								
				10.5								
				11.0								
				11.5								
				12.0								

drilling information / material information / rock mass defects

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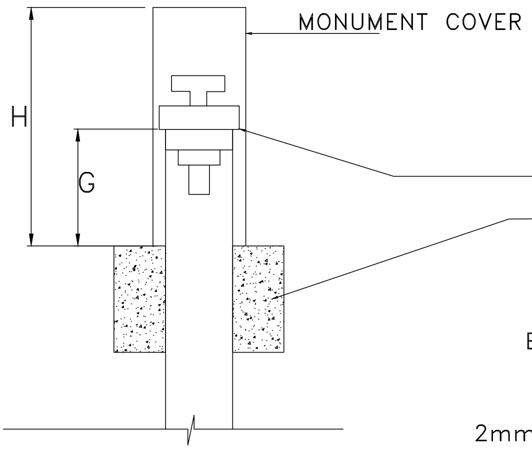
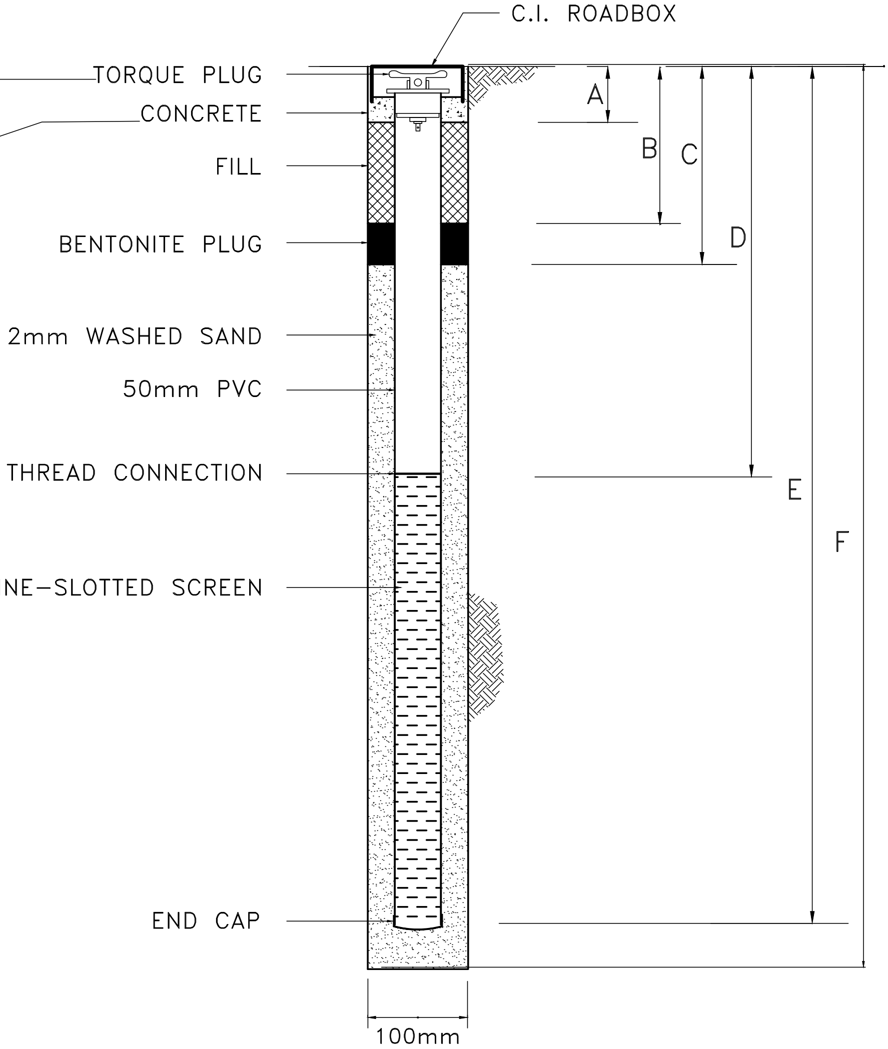
Chester Hill NSW
for
Centurion Project Management

CORE PHOTO – BH105

drawn: JW
date: 11.5.22
checked: AT
scale: 1:4 @ A4

job no.: 6851
fig: —
issue: A





PIEZOMETER DIMENSIONS

	BH105
A	–
B	3.5
C	4.0
D	4.5
E	7.5
F	7.5
G	–
H	–

GROUNDWATER READINGS

DATE:	BOREHOLE	DEPTH (m)	COMMENTS
6/5/22	105	–	Well installed.

issue	date	description
A	11.5.22	INITIAL ISSUE

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for
Centurion Project Management

Piezometer Details

drawn: JW	**job no.:** 6851	
date: 11.5.22		
checked: AT	**fig:** –	**issue:** A
scale: NTS		

Appendix C

Laboratory Test Results

POINT LOAD STRENGTH INDEX REPORT

Client	AssetGeoEnviro	**Moisture Content Condition**	As received
Address	Suite 2.06, 56 Delhi Road, North Ryde, NSW 2113	**Storage History**	Core boxes
Project	Proposed Residential Development (6851)	**Report #**	S77180-PL
Job #	S21620-1	**Test Date**	18/05/2022

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

Sampling Sampled by Client - results apply to the sample as received **Date Sampled** 04-05/05/2022

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)	Failure Mode
S77180	BH101 4.65-4.73m	Siltstone	Diametral	-	49.0	0.06	0.02	0.02	1
			Axial	53.8	31.0	0.12	0.06	0.05	1
S77181	BH101 6.67-6.77m	Siltstone	Diametral	-	50.0	0.92	0.37	0.37	1
			Axial	51.5	36.0	1.04	0.44	0.43	1
S77182	BH101 8.0-8.12m	Sandstone	Diametral	-	50.0	2.66	1.06	1.06	1
			Axial	51.7	35.0	12.01	5.21	5.12	1
S77183	BH102 1.90-2.0m	Sandstone	Diametral	-	49.0	1.68	0.70	0.69	1
			Axial	51.3	35.0	2.78	1.22	1.19	1
S77184	BH102 3.44-3.50m	Siltstone	Diametral	-	50.0	0.02	0.01	0.01	1
			Axial	51.4	35.0	0.33	0.14	0.14	4
S77185	BH102 5.62-5.70m	Siltstone	Diametral	-	49.0	0.08	0.03	0.03	1
			Axial	51.3	40.0	1.34	0.51	0.52	1
S77186	BH102 7.65-7.75m	Siltstone	Diametral	-	50.0	0.55	0.22	0.22	1
			Axial	51.6	40.0	3.22	1.23	1.24	1
S77187	BH103 2.60-2.68m	Siltstone	Diametral	-	49.0	0.02	0.01	0.01	1
			Axial	51.5	34.0	0.53	0.24	0.23	1
S77188	BH103 4.55-4.65m	Siltstone	Diametral	-	50.0	0.69	0.28	0.28	1
			Axial	51.7	36.0	0.72	0.30	0.30	1
S77189	BH103 6.88-7.0m	Siltstone	Diametral	-	50.0	1.11	0.44	0.44	1
			Axial	51.5	37.0	2.30	0.95	0.94	4

Failure Modes

1 - Fracture through fabric of specimen oblique to bedding, not influenced by weak planes.

2 - Fracture along bedding.

3 - Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.

4 - Chip or partial fracture.

Notes

NATA

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards. This document shall not be reproduced, except in full. Results relate only to the samples tested.

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

19/05/2022
Date

MACQUARIE GEOTECH

Macquarie Geotechnical
14 Carter St
Lidcombe NSW 2141

POINT LOAD STRENGTH INDEX REPORT

Client	AssetGeoEnviro	**Moisture Content Condition**	As received
Address	Suite 2.06, 56 Delhi Road, North Ryde, NSW 2113	**Storage History**	Core boxes
Project	Proposed Residential Development (6851)	**Report #**	S77190-PL
Job #	S21620-1	**Test Date**	18/05/2022

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

Sampling Sampled by Client - results apply to the sample as received **Date Sampled** 04-05/05/2022

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)	Failure Mode
S77190	BH103 8.0-8.09m	Siltstone	Diametral	-	50.0	1.38	0.55	0.55	1
			Axial	51.9	35.0	2.25	0.97	0.96	1
S77191	BH105 2.60-2.70m	Sandstone	Diametral	-	49.0	4.26	1.77	1.76	1
			Axial	51.7	40.0	5.90	2.24	2.27	1
S77192	BH105 3.20-3.28m	Siltstone	Diametral	-	50.0	0.18	0.07	0.07	1
			Axial	51.4	40.0	0.33	0.13	0.13	1
S77193	BH105 4.28-4.36m	Siltstone	Diametral	-	49.0	1.19	0.50	0.49	1
			Axial	51.6	39.0	0.15	0.06	0.06	3
S77194	BH105 5.17-5.27m	Siltstone	Diametral	-	48.0	0.79	0.34	0.34	1
			Axial	51.6	40.0	1.18	0.45	0.45	1
S77195	BH105 6.75-6.86m	Sandstone	Diametral	-	50.0	0.99	0.40	0.40	1
			Axial	51.7	35.0	3.77	1.64	1.61	1

Failure Modes

1 - Fracture through fabric of specimen oblique to bedding, not influenced by weak planes.

2 - Fracture along bedding.

3 - Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.

4 - Chip or partial fracture.

Notes

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Appendix D

Site Photos

Photo 1

Borehole investigation at BH101 location

Photo 2

Overview of site conditions inside the aged care facility

Photo 3

General overview of former service station

Photo 4

Continuation of Photo 3.