Innovate Architects

Proposed Mixed-Use Development
138 Cronulla Street, Cronulla NSW

Geotechnical Investigation

Our ref: 5533-G1
6 May 2019

DOCUMENT AUTHORISATION

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Geotechnical Investigation

Prepared for Innovate Architects

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6 May 2019

For and on behalf of
Asset Geotechnical Engineering Pty Ltd

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1. INTRODUCTION

1.1 General

This report presents the results of a geotechnical investigation for the above project. The investigation was commissioned on by Allen Sammut of Innovate Architects. The work was carried out in accordance with the email proposal by Asset Geotechnical Engineering Pty Ltd (AssetGeo) dated 23 April 2019, reference 5533-P1.

Drawings supplied to us for this investigation comprised:
- Architectural plans (prepared by: Innovate Architects; dated: April 2019).

Based on the supplied drawings, we understand that the project involves the construction of seven storey mixed-use building with three level basements. There is a reported unlined rock sewer tunnel across the site, and it is located at depth about 9.5m below ground level (bgl). The sewer is approximately 1.8m high and 1.2m wide. It is believed to have been completed in 1956. No further details have been provided.

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.
- Excavation support methodology and design parameters.
- Maximum allowable permanent and temporary batter slopes.
- Groundwater conditions.
- Potential impact on the existing sewer.

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 two locations to sample and assess the nature and consistency of subsurface soils and bedrock at accessible areas of the site.
- Laboratory tests on the recovered 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 on the eastern side of Cronulla Street, as shown in Figure 1. The site is bounded to the west by Cronulla Street, to the east by Surf Lane, to the north by commercial building and to the south by Beach Park Avenue.

Topographically, the site is located on flat to gently sloping terrain. The overall ground surface slopes in the region are about 2º.

At the time of the investigation, the site was occupied by a single-storey brick and brick-rendered commercial buildings with car park. There were cracks observed on car park asphalt and concrete paved area and multiple cracks observed on retaining wall at south-west corner of car park, but the buildings appeared otherwise to be in good visual condition.

3. FIELDWORK & LABORATORY TESTING

3.1 Borehole Investigation

The fieldwork was undertaken on 26 April 2019 under the full-time supervision of a Geotechnical Engineer from AssetGeo and included invasive investigation at two locations.

The test locations are shown in the attached Figure 2 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 drilling of machine-drilled boreholes at two locations. The boreholes were auger drilled to refusal at depths of 4.5m to 5.1m below ground level (bgl), and then continued using NMLC coring techniques to termination at 9m to 9.7m depth.

Recovered rock core was retained for laboratory testing for point load strength index.

The subsurface conditions encountered were logged during drilling and testing. On completion of logging and sampling, the boreholes were backfilled with the drilling spoil.

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

3.2 Laboratory Testing

Rock samples recovered during the fieldwork were delivered to a NATA registered laboratory and tested for point load strength index.

Test results are attached and are included on the engineering logs. 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 Wollongong – Port Hacking 1:100,000 Geological Map indicates that the site is underlain by Hawkesbury Sandstone.

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	Asphalt and concrete pavement.	Ground surface	0.05 - 0.3
		Gravelly Sandy CLAY/ Sandy CLAY, dark brown/dark red brown mottled dark grey, low to medium plasticity; fine to medium grained, gravel, subangular; trace ironstone fragments.	0.05	0.65
2	Residual	Sandy CLAY/CLAY, brown/pale grey mottled orange/red, low to medium plasticity, stiff to very stiff; trace sandstone fragments.	0.3 - 0.7	3.8 - 4.8
3a	Bedrock[2]	SANDSTONE, pale grey/grey/red mottled orange/purple/dark grey, fine to medium grained, BP0° to 30°, poorly developed layering, thinly laminated to laminated, highly to moderately weathered, low to medium strength, assessed to be Class 5 and 4 Sandstone.	4.5 - 5.1	1.6 - 2.1
3b	Bedrock[2]	SANDSTONE, pale grey mottled grey, fine to medium grained, BP0° to 20°, poorly to well-developed layering, thinly laminated to laminated, slightly weathered, medium strength becoming better with depth, assessed to be Class 3 Sandstone or better.	6.7 – 6.8	Not proven

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 4.5m to 5.1m bgl. Due to the introduction of water whilst coring, observation of groundwater inflow/levels below auger termination depths was not possible.

It is noted that the groundwater observation may have been made before water levels had stabilised. No long-term groundwater monitoring was carried out. Groundwater is likely to be confined to open joints in the sandstone and will locally be under-drained by the reportedly unlined sewer tunnel.

5. DISCUSSIONS & RECOMMENDATIONS

5.1 Key Geotechnical Site Constraints

Based on the provided sketch design, and from the results of this investigation, it is assessed that the lowest basement level will be about 9.5m to 10m below ground level and would be predominantly within sandstone bedrock.

Key geotechnical constraints to the development include excavation conditions, groundwater control (during construction and long-term), temporary shoring, permanent retaining, foundation conditions, and stressrelief impacts on the existing sewer. Recommendations for design and construction of the development are provided in the following sections.

5.2 Construction Sequence

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

1. Demolish existing buildings.
2. Remove existing pavements / concrete slabs.
3. Install temporary shoring around the basement perimeter.
4. Install temporary dewatering system (internal to the basement)
5. Excavate to bulk excavation level.
6. Install pile footings for internal column loads.
7. Carry out detail excavations (e.g. for lift pits) – additional localised dewatering may be required.
8. Construct the lower basement ground floor.
9. Pour lower basement roof and continue up to existing ground surface level to provide permanent support to the excavation.
10. Decommission temporary dewatering system.

5.3 Temporary Shoring

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

Several possible shoring systems could be considered for the site. These are summarised in Table 2 together with a brief description of the advantages and disadvantages of each.

Table 2 – Summary of Shoring Options

Option	Method	Advantages	Disadvantages
1	Conventional shoring with soldier piles and steel walers, or soldier piles and shotcrete infill panels	Relatively low cost	Risk of instability and loss of ground unless adequate external dewatering is provided. Forms a poor seal against groundwater. Greater amount of dewatering required. Potential drawdown of groundwater levels outside of the site with possible adverse effects on adjacent structures.
2	Steel sheet pile (driven or hydraulically installed)	Rapid installation. Lower cost than Option 3. Low permeability water barrier. Amenable to joint caulking.	Vibration may not be acceptable for adjoining developments. Permanent wall required. Will require soil anchors.
3a **or** **3b**	Contiguous or Secant bored piles	Can form part of the permanent structure. Minimum noise and vibration. Can maximise site building space as no temporary wall is required. Permanent 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. May require soil anchors along boundaries where high-level footings are located. Contiguous piles may require additional waterproofing where close contact not achieved.
4	Cutter Soil Mix (CSM) or Diaphragm wall	Practically impervious. Can be used as a permanent wall. Minimise settlement and ground disturbance of adjacent ground and properties.	Expensive. Close supervision of contractors required. May require soil anchors along boundaries where high-level footings are located.

Based on the advantages and disadvantages listed in Table 2, we recommend Option 1. Option 2 is not likely to be suitable due to the depth of excavation support and effect on adjacent structures. Option 3a / 3b could be considered if adopted as part of the permeant retaining system. Option 4 may be too expensive for the scale of the project, however, could still be considered.

The founding depth of the retaining wall piles is a function of: -

- the required socket depth to achieve adequate embedment to resist overturning, and
- the required load carrying capacity if the piles are to be incorporated into the permanent works,

Assessed Class 5 to 4 sandstone bedrock was encountered at about 4.5m to 5.1m depth and assessed Class 3 sandstone was encountered below about 6.7m to 6.8m depth. Practically, it may be difficult to achieve a substantial socket within Class 2 or better sandstone. However, adequate overturning resistance is likely to be achieved within the assessed Class 5 to 4 sandstone rock and the overlying very stiff to hard clays. Control of lateral deflections will also need to be considered (e.g. around open rock sewer tunnel), where temporary rock anchors may be required.

From the point of view of groundwater control, penetration into the sandstone bedrock would be preferred. Discussion and recommendations for groundwater control are provided in Section 5.7.

Design of temporary shoring for carrying vertical loading should be in accordance with Section 5.6, and for lateral pressures, it should be in accordance with Section 5.8.

Detailed construction supervision, monitoring and inspections will be required during the piling and subsequent bulk excavation to ensure an adequate standard of workmanship and to minimise potential problems.

5.4 Earthworks

5.4.1 Excavation

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

Excavation within the deeper, less weathered bedrock will likely require 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.

5.4.2 Vibration Management

Australian Standard AS 2187: Part 2-2006 recommends the frequency dependent guideline values and assessment methods given in BS 7385 Part 2-1993 “Evaluation and measurement for vibration in buildings Part 2” as they “are applicable to Australian conditions”. The standard sets guide values for building vibration based on the lowest vibration levels above which damage has been credibly demonstrated. These levels are judged to give a minimum risk of vibration-induced damage, where 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.

Due to proximity to the existing sewer, excavation methods should be adopted which limit ground vibrations at the adjoining developments to not more than 5mm/sec. Vibration monitoring is recommended to verify that this is achieved.

The limits of 5mm/sec are expected to be achievable if rock breaker equipment or other excavation methods are restricted as indicated in Table 3.

Table 3 – 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).

5.4.3 Subgrade Preparation

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

- Strip existing fill and topsoil. Remove unsuitable materials from 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, stockpiling for re-use as engineered fill or remove to spoil. Rock could be stockpiled separately from clayey soils, for select use beneath pavements.
- Where rock is exposed in bulk excavation level beneath pavements, rip a further 150mm.
- Where rock is exposed at footing invert level, it should be free of loose, “drummy" and softened material before concrete is poured.
- Where soil is exposed at bulk excavation level, compact the upper 150mm depth to a dry density ratio (AS1289.5.4.1–2007) not less than 100% Standard.

- Areas which show visible heave under compaction equipment should be over-excavated a further 0.3m and replaced with approved fill compacted to a dry density ratio not less than 100%.

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

5.4.4 Filling

Where filing is required, place in horizontal layers over prepared subgrade and compact as per Table 4.

Table 4 – 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 back-filling 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. Further advice should be sought from a specialist environmental consultant if required.

5.4.5 Batter Slopes

Recommended maximum slopes for permanent and temporary batters are presented in Table 5.

Table 5 – Recommended Maximum Dry Batter Slopes

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

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

5.5 Site Classification 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 sandstone 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 5.6.

The classification and footing recommendations given above and in Section 5.6 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.

5.6 Footings

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

Footings over the existing sewer should be designed to span across the sewer without placing additional vertical loading on to the rock mass above the tunnel overt.

Edge beams for slabs, pad footings, and rock-socketed piles may be designed for the parameters in Table 6.

Table 6 – 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 Sandstone	1,000	100	50	3,000	300	150	75
Class 4 or better Sandstone	3,500	350	175	10,500	1,050	525	300

Note:

* 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 6.

Options for piles include:

> **Bored Piles.** It assessed that the construction of sockets would require the use of a truck-mounted drilling rig. It is also assessed that the bored pile holes would not require liners to support the overburden soils, although some over break and minor fretting should be allowed for. Groundwater may be expected within bored pile holes, particularly after heavy rain towards the soil – rock interface 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.

5.7 Groundwater Control

Limited groundwater observations made for this investigation are described in Section 4.3. The observations indicate that groundwater is unlikely to be a constraint to the proposed development. However, good practice should be followed to cater for potential groundwater, such as designing retaining walls with adequate subsoil drainage. Further geotechnical advice must be sought if significant groundwater is encountered during construction.

5.8 Excavation Support

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

5.8.1 Excavation Support Construction Methodology

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

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

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

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

If bottom-up construction is considered, we recommend the use of temporary anchored walls where the retained height is 3.5m or more, and cantilever walls where the retained height is less than 3.5m.

5.8.2 Excavation Support Design Parameters

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

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

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

EARTH PRESSURE APPROACH

Support systems designed using the Earth Pressure Approach may be based on the parameters given in Table 7.

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

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

Table 7 – Excavation Support Design Parameters (Earth Pressure Approach)

Material	Moist Unit Weight (γ_m) kN/m^3	'Active' Lateral Earth Pressure Coefficient (1) (K_a)	'At Rest' Coefficient (1) (K_o)	'Passive' Coefficient (2) (K_p)
Residual Clay	19.0	0.35	0.5	N/A
Class 5 Sandstone (3)	21.0	0.2	0.4	6
Class 4 Sandstone (3)	22.0	0.1	0.3	15
Class 3 Sandstone (3)	24.0	0.0	0.0	30

Notes to table:

1. These values assume that some wall movement and relaxation of horizontal stress will occur due to the excavation. Actual in-situ K_0 values may be higher, particularly in the rock units.
2. Includes a reduction factor to the ultimate value of K_p to take into account strain incompatibility between active and passive pressure conditions. 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.

NUMERICAL MODELLING APPROACH

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

The values in Table 8 provide typical parameters that can be adopted for design. Review and refinement of these parameters may be necessary as part of carrying out more advanced numerical modelling (e.g. consideration of advanced soil models, use of elasto-plastic parameters).

Table 8 – Excavation Support Design Parameters (Numerical Modelling Approach)

Material	Moist Unit Weight (γ_m) kN/m^3	‘At Rest’ Coefficient $^{(1)}$ (K_o)	Effective Cohesion (c’) kPa	Effective Friction Angle (ϕ’) deg	Elastic Modulus (E) MPa
Residual Clay	18.0	0.5	2	26	40
Class 5 Sandstone $^{(2)}$	24.0	0.4	5	28	100
Class 4/3 Sandstone $^{(2)}$	24.0	0.3	100	35	400

Notes to table:

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

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.

5.8.3 Ground Anchors

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

Anchors could be inclined up to a maximum of 30° below horizontal if required to intercept bedrock / higher strength bedrock. Design of excavation support must be carried out by a suitably experienced and qualified structural/civil engineer. Requirements for rock support must be nominated or approved by the Geotechnical Engineer during excavation. Rock bolts may be designed for the parameters in Table 9.

Table 9 – Rock Bolting Preliminary Design Parameters

Layer	Ultimate Bond Stress (without Factor of Safety)
Residual Soil	25 kPa
Class 5 Sandstone	300 kPa
Class 4 Sandstone	600 kPa
Class 3 Sandstone	1,500 kPa

The following should be noted during anchor design and construction:

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

5.9 Potential Impacts on Open Rock Sewer Tunnel

The sewer tunnel is understood to have been formed by mining in the sandstone and be unlined. A 4m to 5m excavation exclusion zone around the sewer should be put in place.

Potential geotechnical risks of construction on adjoining open rock sewer tunnel could include; vibration effects due to rock excavation and deflection (horizontal stress release) of adjacent sewer tunnel due to the basement excavation. In order to confirm the stress changes around the sewer tunnel due to the basement excavation and the corresponding displacements in the tunnel and surrounding rock mass, numerical modelling is recommended before construction. If accessible, a condition survey of the tunnel should be undertaken before commencing any excavation on site. Sydney Water may also have additional monitoring requirements and further discussion with them is recommended. They are likely to require seeing the results of the numerical modelling. It should be noted that both Sydney Trains and RMS may also require to see evidence that the proposed basement excavation will not adversely affect their assets.

6. 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.

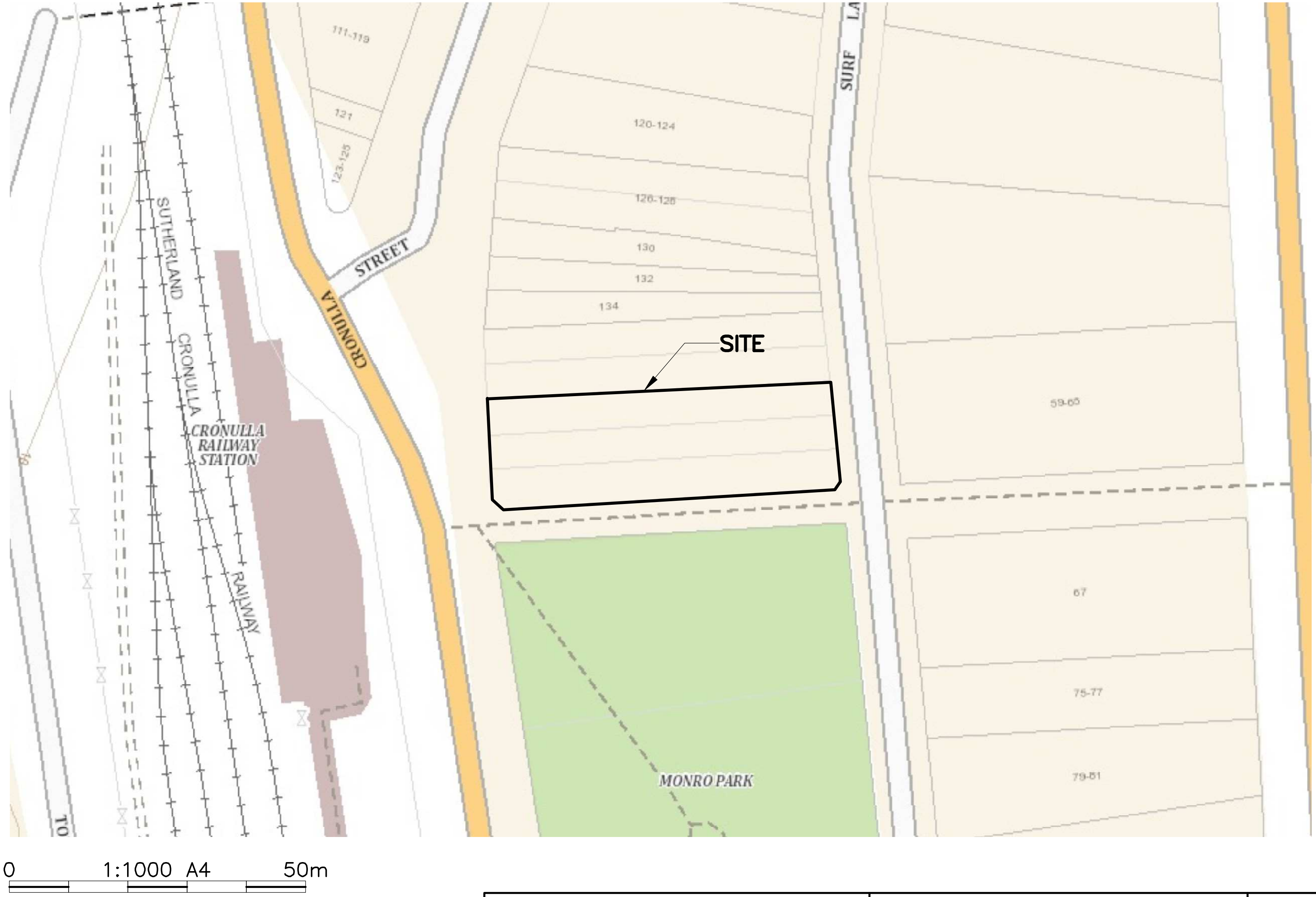
It is recommended that a qualified and experienced Geotechnical Engineer be engaged to provide further input and review during the design development; including site visits during construction to verify the site conditions and provide advice where conditions vary from those assumed in this report. Development 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, Sydney Trains, and Roads & Maritime Services), 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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issue	date	description
A	29.4.19	INITIAL ISSUE

assetgeo

Asset Geotechnical Engineering Pty Ltd
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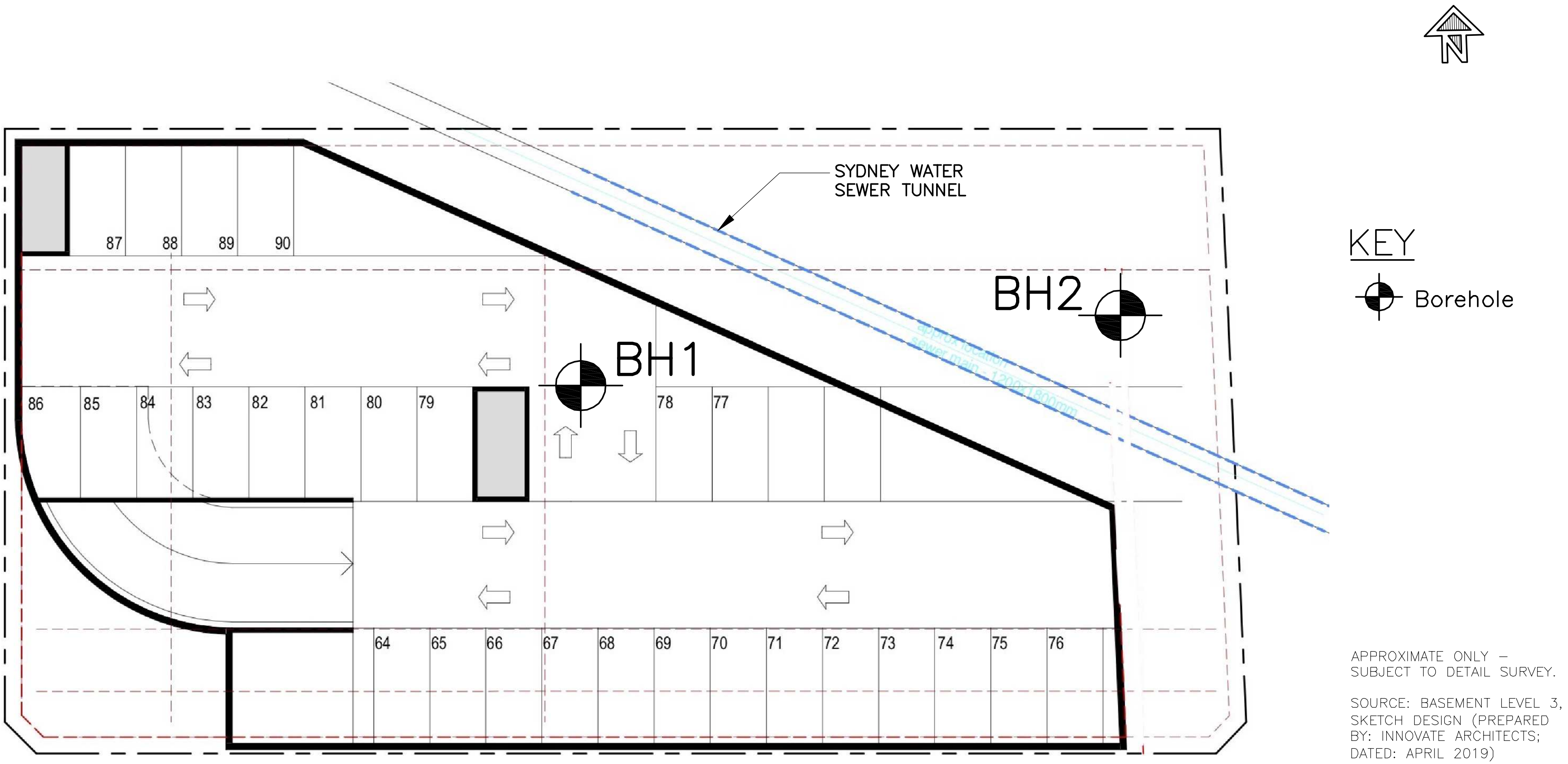
PROPOSED MIXED–USE DEVELOPMENT
138 CRONULLA STREET, CRONULLA NSW
for
INNOVATE ARCHITECTS

SITE LOCALITY

drawn: YG
date: 29.4.19
checked: MAG
scale: 1:1000 A4

job no.: 5533
fig: 1
issue: A



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TEST LOCATIONS

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date: 29.4.19

checked: MAG

scale: 1:300 A4

job no.: 5533

fig: 2

issue: A

APPENDIX A

Important Information about your Geotechnical Report
CSIRO BTF 18

Important Information about your Geotechnical Report

SCOPE OF SERVICES

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

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

RELIANCE ON DATA

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

GEOTECHNICAL ENGINEERING

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

LIMITATIONS OF SITE INVESTIGATION

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

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

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

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

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

VERIFICATION OF SITE CONDITIONS

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

REPRODUCTION OF REPORTS

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

REPORT FOR BENEFIT OF CLIENT

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

DATA MUST NOT BE SEPARATED FROM THE REPORT

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

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

PARTIAL USE OF REPORT

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

OTHER LIMITATIONS

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

Foundation Maintenance and Footing Performance:
A Homeowner's Guide

CSIRO

BTF 18
replaces
Information
Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

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

GENERAL DEFINITIONS OF SITE CLASSES

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

Tree root growth

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

Erosion and saturation

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

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

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

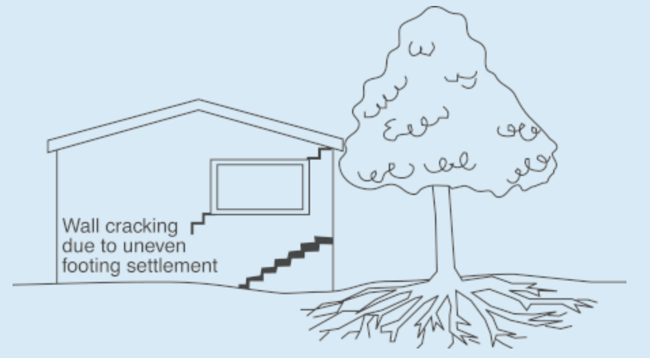
Seasonal swelling/shrinkage in clay

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

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

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

Trees can cause shrinkage and damage



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

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

Movement caused by tree roots

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

Complications caused by the structure itself

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

Effects on full masonry structures

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

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

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

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

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

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

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

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

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

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

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

Prevention/Cure

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

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

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

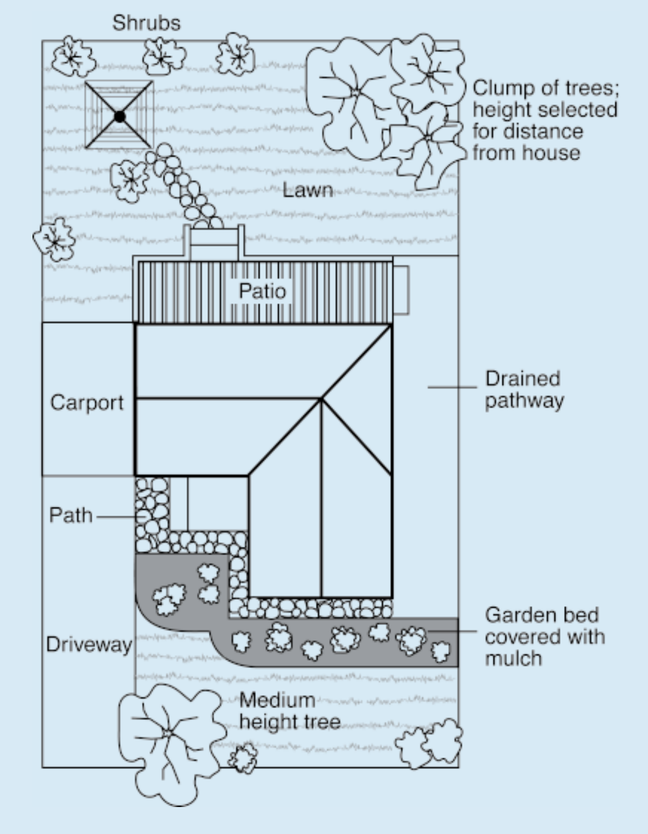
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

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APPENDIX B

Soil & Rock Explanation Sheets
Borehole Logs
Core Logs
Core Photographs

Soil and Rock Explanation Sheets (1 of 2)

LOG ABBREVIATIONS AND NOTES

METHOD

borehole logs		***excavation logs***	
AS	auger screw *	NE	natural excavation
AD	auger drill *	HE	hand excavation
RR	roller / tricone	BH	backhoe bucket
W	washbore	EX	excavator bucket
CT	cable tool	DZ	dozer blade
HA	hand auger	R	ripper tooth
D	diatube		
B	blade / blank bit		
V	V-bit		
T	TC-bit		

* bit shown by suffix e.g. ADV

coring

NMLC, NQ, PQ, HQ

SUPPORT

borehole logs		***excavation logs***	
N	nil	N	nil
M	mud	S	shoring
C	casing	B	benched
NQ	NQ rods		

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

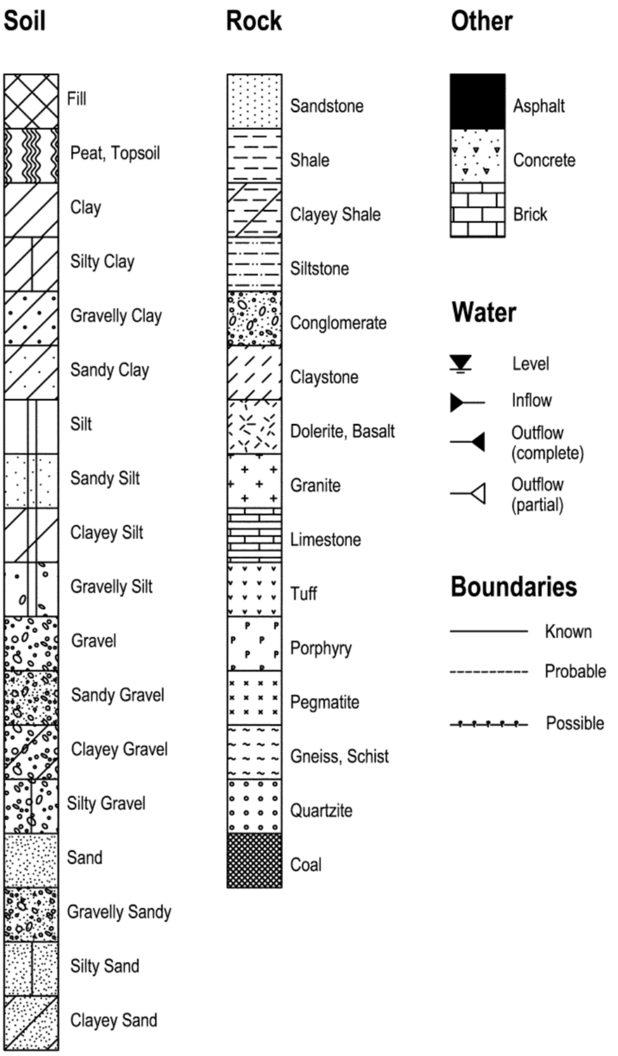
GW	Gravel and gravel-sand mixtures, little or no fines.
GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels
GM	Gravel-silt mixtures and gravel-sand-silt mixtures.
GC	Gravel-clay mixtures and gravel-sand-clay mixtures.
SW	Sand and gravel-sand mixtures, little or no fines.
SP	Sand and 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		**DENSITY INDEX**	
VS	very soft	VL	very loose
S	soft	L	loose
F	firm	MD	medium dense
St	stiff	D	dense
VSt	very stiff	VD	very dense
H	hard		
Fb	friable		

GRAPHIC LOG



WEATHERING		**STRENGTH**	
XW	extremely weathered	VL	very low
HW	highly weathered	L	low
MW	moderately weathered	M	medium
SW	slightly weathered	H	high
FR	fresh	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		***coating***	
JT	joint	cl	clean
PT	parting	st	stained
SZ	shear zone	ve	veneer
SM	seam	co	coating

shape		***roughness***	
pl	planar	po	polished
cu	curved	sl	slickensided
un	undulating	sm	smooth
st	stepped	ro	rough
ir	irregular	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 COURSE 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.
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.

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.

assetgeo

Borehole Log

BH no:	**BH1**
sheet:	1 of 4
job no.:	5533

client:	Innovate Architects	**started:**	26.4.2019
principal:		**finished:**	26.4.2019
project:	Proposed Mixed-Use Development	**logged:**	YG
location:	138 Cronulla Street, Cronulla NSW	**checked:**	MAB
equipment:	Ute-Mounted Drilling Rig	**RL surface:**	approx.
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information						**material information**						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetrometer kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed					-	Asphalt pavement (50mm)	-	-		Fill (asphalt pavement)
					.05		CL	FILL, Gravelly Sandy CLAY, dark brown mottled dark grey, low to medium plasticity; fine to medium grained gravel, subangular.	<Wp	S		Fill
					0.5 .5		CL	FILL, Sandy CLAY, dark red brown, low to medium plasticity; with gravel, fine to medium grained, subangular to angular; trace ironstone fragments.	~Wp	S-F		Fill
					.7		CL	CLAY, brown mottled orange/red, low to medium plasticity.	~Wp	St	× 70	Residual
					1.0							
					1.2		CL	Sandy CLAY, pale grey mottled red, low plasticity; trace sandstone fragments.	~Wp	St-VSt		Residual
					1.5						× 120	
					2.0							
					2.5							
					3.0							

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Borehole Log - Revision 10

Borehole Log

BH no:	**BH1**
sheet:	2 of 4
job no.:	5533

client:	Innovate Architects	**started:**	26.4.2019
principal:		**finished:**	26.4.2019
project:	Proposed Mixed-Use Development	**logged:**	YG
location:	138 Cronulla Street, Cronulla NSW	**checked:**	MAB
equipment:	Ute-Mounted Drilling Rig	**RL surface:**	approx.
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information						**material information**						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed			3	▨	CL	Sandy CLAY, pale grey mottled pale brown/red, low to medium plasticity; with sandstone fragments.	<Wp	St-VSt		Residual
					3.5	▨						
					4.0	▨						
					4.5	▨		ADT terminated at 4.4m, refusal on Sandstone bedrock.				
								Borehole No: BH1 continued as cored hole from 4.5m				
					5.0							
					5.5							
					6.0							

5533 BH LOGS.GPJ 30/4/19

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Cored Borehole Log

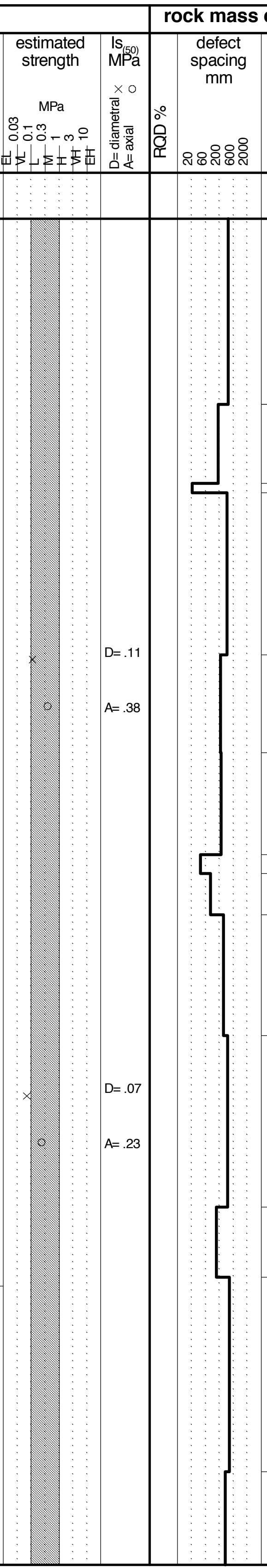
BH no: **BH1**
sheet: 3 of 4
job no.: 5533

client: Innovate Architects
principal:
project: Proposed Mixed-Use Development
location: 138 Cronulla Street, Cronulla NSW
equipment: Ute-Mounted Drilling Rig
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**

started: 26.4.2019
finished: 26.4.2019
logged: YG
checked: MAB
RL surface:
datum: AHD

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

method	support & core-lift	water	RL	depth metres	graphic log core recovery	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D= diametral × A= axial ○
		None observed		4.5		Continued from non-cored borehole from 4.5m		
NMLC				4.5		SANDSTONE with Ironstone layering, pale grey/red mottled orange, fine to medium grained, subrounded, BP0°-30°, poorly developed layering, laminated.	HW - MW	
				5.0				
				5.5				D= .11 A= .38
				6.0				
				6.5				D= .07 A= .23
				6.8		SANDSTONE, pale grey mottled grey, fine to medium grained, subrounded, BP0°-20°, poorly to well developed layering, thinly laminated to laminated.	SW	
		None observed		7.0				



defect description
type, inclination, thickness, shape, roughness, coating
specific general

- Core loss (400mm)
- PT0° pl ro st
- SM 20mm, XW
- PT30° stp ro st
- PT0° pl ro st
- Fractured
- PT20° pl ro st
- PT20° pl ro st
- PT20° pl ro st
- PT0° pl ro st
- PT0° ir ro st

5533 BH LOGS.GPJ 30/4/19

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

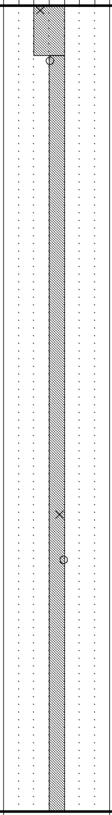
Cored Borehole Log - Revision 9

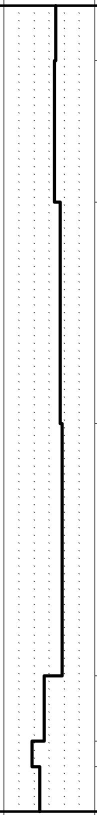
Cored Borehole Log

BH no: BH1
sheet: 4 of 4
job no.: 5533

client: Innovate Architects
principal:
project: Proposed Mixed-Use Development
location: 138 Cronulla Street, Cronulla NSW
equipment: Ute-Mounted Drilling Rig
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**

started: 26.4.2019
finished: 26.4.2019
logged: YG
checked: MAB
RL surface:
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				7.5		SANDSTONE, pale grey mottled grey, fine to medium grained, subrounded, BP0°-20°, poorly to well developed layering, thinly laminated to laminated. *(continued)*	SW		D= .16			
									A= .33			PT0° ir ro st
												PT5° pl ro st
				8.0								
												PT5° pl ro cl
									D= .67			
				8.5					A= .92			
												PT0° pl ro cl
												PT0° pl ro cl
												PT10° pl ro cl
				9.0								
				9		NMLC terminated at 9m, reaching target depth. BH1 terminated at 9m						
				9.5								
				10.0								

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Cored Borehole Log - Revision 9

A	30.4.19	INITIAL ISSUE
issue	date	description

assetgeo

Asset Geotechnical Engineering Pty Ltd
2.05/56 Delhi Rd, North Ryde NSW 2113
t: 02 9878 6005
e: info@assetgeo.com.au

PROPOSED MIXED–USE DEVELOPMENT
138 CRONULLA STREET, CRONULLA NSW
for
INNOVATE ARCHITECTS

BH1 CORE PHOTOS

drawn: YG

date: 30.4.19

checked: MAB

scale: 1:4 A4

job no.: 5533

fig: —

issue: A

Borehole Log

BH no:	**BH2**
sheet:	1 of 4
job no.:	5533

client: Innovate Architects
principal:
project: Proposed Mixed-Use Development
location: 138 Cronulla Street, Cronulla NSW
started: 26.4.2019
finished: 26.4.2019
logged: YG
checked: MAB
equipment: Ute-Mounted Drilling Rig
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**
RL surface: approx.
datum: AHD

drilling information						material information						
method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	**material description** soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	**structure and additional observations**
ADT	C	None observed					-	Concrete slab (150mm) and asphalt layer.	-	-		Fill (concrete and asphalt)
					.3		CL	Sandy CLAY, brown/orange, low to medium plasticity; trace ironstone fragments.	~Wp	S		Residual
					1.2		CL	Sandy CLAY, pale grey mottled red, medium plasticity.	~Wp	S-F		Residual
					2		CL	Sandy CLAY, pale grey mottled red/orange, medium plasticity; trace sandstone fragments.	<Wp	St		Residual
					2.7		CL	Sandy CLAY, pale grey/pale brown, low to medium plasticity.	<Wp	St-VSt		Residual

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Borehole Log

BH no:	**BH2**
sheet:	2 of 4
job no.:	5533

client:	Innovate Architects	**started:**	26.4.2019
principal:		**finished:**	26.4.2019
project:	Proposed Mixed-Use Development	**logged:**	YG
location:	138 Cronulla Street, Cronulla NSW	**checked:**	MAB
equipment:	Ute-Mounted Drilling Rig	**RL surface:**	approx.
diameter:	100mm **inclination:** -90° **bearing:** --- **E:** **N:**	**datum:**	AHD

drilling information | **material information**

method	support	water	notes samples, tests, etc	RL	depth metres	graphic log	USCS symbol	material description soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/ density index	hand penetro-meter kPa 100 200 300 400	structure and additional observations
ADT	C	None observed					CL	Sandy CLAY, pale grey/pale brown, low to medium plasticity. *(continued)*	<Wp	St-VSt		
					3.5							
					4.0							
					4.5							
					5.0							
	N							ADT terminated at 5.1m, refusal on Sandstone bedrock.				
								Borehole No: BH2 continued as cored hole from 5.1m				
					5.5							
					6.0							

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Borehole Log - Revision 10

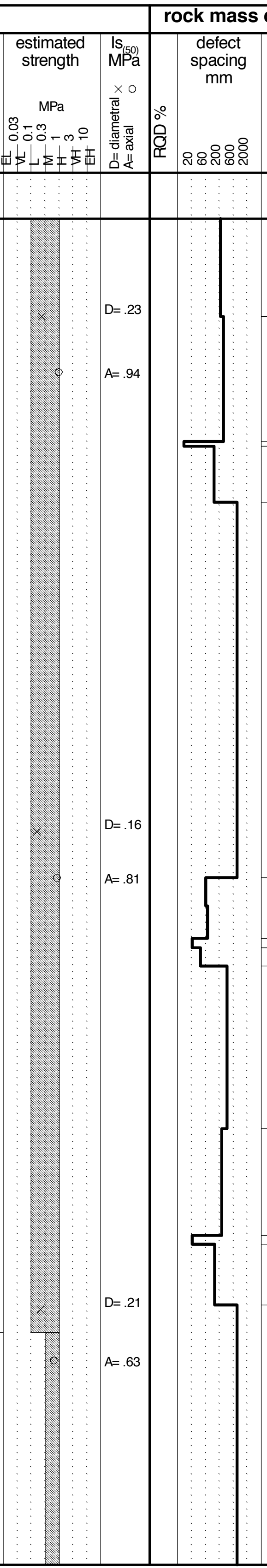
Cored Borehole Log

BH no: BH2
sheet: 3 of 4
job no.: 5533

client: Innovate Architects
principal:
project: Proposed Mixed-Use Development
location: 138 Cronulla Street, Cronulla NSW
equipment: Ute-Mounted Drilling Rig
diameter: 100mm **inclination:** -90° **bearing:** --- **E:** **N:**

started: 26.4.2019
finished: 26.4.2019
logged: YG
checked: MAB
RL surface:
datum: AHD

drilling information					material information					rock mass defects		
method	support & core-lift	water	RL	depth metres	graphic log core recovery	rock substance description rock type; grain characteristics, colour, structure, minor components	weathering	Is(50) MPa D= diametral × A= axial ○	RQD %	defect description type, inclination, thickness, shape, roughness, coating specific	general
		None observed				Continued from non-cored borehole from 5.1m					
NMLC				5.1		SANDSTONE, grey mottled red/orange/dark grey/purple, fine to medium grained, subrounded, BP0°-20°, poorly developed layering, thinly laminated to laminated.	HW - MW				
								D= .23		PT0° pl ro st	
								A= .94			
				5.5							
										SM 10mm XW	
										PT0° cu ro st	
				6.0							
								D= .16			
				6.5				A= .81		PT20° pl ro st	
										Fractured	
										PT10° ir ro st	
				7.0						PT5° pl ro cl	
										Fractured	
								D= .21		PT0° ir ro st	
				7.5		SANDSTONE, pale grey mottled grey, fine to medium grained, subrounded, BP0°-20°, poorly to well developed layering, thinly laminated to laminated.	SW	A= .63			
		None observed		8.0							



REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

Cored Borehole Log

BH no: **BH2**
sheet: 4 of 4
job no.: 5533

client: Innovate Architects
principal:
project: Proposed Mixed-Use Development
location: 138 Cronulla Street, Cronulla NSW
started: 26.4.2019
finished: 26.4.2019
logged: YG
checked: MAB
equipment: Ute-Mounted Drilling Rig
RL surface:
diameter: 100mm **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						SANDSTONE, pale grey mottled grey, fine to medium grained, subrounded, BP0°-20°, poorly to well developed layering, thinly laminated to laminated. *(continued)*	SW						
												PT5° cu ro cl	
								×	D= .73			PT20° pl ro cl	
								○	A= .64				
				8.5									
												PT5° pl ro cl	
				9.0									
								×	D= .32			PT5° cu ro cl	
								○	A= .67				
				9.5									
				9.7		NMLC terminated at 9.7m, reaching target depth. BH2 terminated at 9.7m							
				10.0									
				10.5									
				11.0									

5533 BH LOGS.GPJ 30/4/19

REFER TO EXPLANATION SHEETS FOR DESCRIPTION OF TERMS AND SYMBOLS USED

A	30.4.19	INITIAL ISSUE
issue	date	description

assetgeo

Asset Geotechnical Engineering Pty Ltd
2.05/56 Delhi Rd, North Ryde NSW 2113
t: 02 9878 6005
e: info@assetgeo.com.au

PROPOSED MIXED-USE DEVELOPMENT
138 CRONULLA STREET, CRONULLA NSW
for
INNOVATE ARCHITECTS

BH2 CORE PHOTOS

drawn: YG

date: 30.4.19

checked: MAB

scale: 1:4 A4

job no.: 5533

fig: —

issue: A

APPENDIX C

Laboratory Test Results

POINT LOAD STRENGTH INDEX REPORT

Client:	Asset Geotechnical	**Moisture Content Condition:**	As received
Address:	Suite 2.05, 56 Delhi Road, North Ryde, NSW 2113	**Storage History:**	Core boxes
Project:	138 Cronulla St Cronulla (5533)	**Report No:**	S47732-PL
Job No:	S19191	**Date Tested:**	29/04/2019

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

Sampling: Sampled by Client **Date Sampled:** 26/04/2019

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
S47732	BH1 5.45 - 5.55m	Sandstone	Diametral	-	49.0	0.27	0.11	0.11	1
			Axial	51.4	31.0	0.81	0.40	0.38	1
S47733	BH1 6.39 - 6.49m	Sandstone	Diametral	-	49.0	0.17	0.07	0.07	1
			Axial	51.8	43.0	0.63	0.22	0.23	1
S47734	BH1 7.41 - 7.51m	Sandstone	Diametral	-	49.0	0.38	0.16	0.16	1
			Axial	51.9	33.0	0.74	0.34	0.33	1
S47735	BH1 8.41 - 8.50m	Sandstone	Diametral	-	50.0	1.68	0.67	0.67	1
			Axial	51.8	37.0	2.27	0.93	0.92	1
S47736	BH2 5.31 - 5.43m	Sandstone	Diametral	-	50.0	0.58	0.23	0.23	1
			Axial	51.8	29.0	1.91	1.00	0.94	1
S47737	BH2 6.42 - 6.52m	Sandstone	Diametral	-	48.0	0.38	0.16	0.16	1
			Axial	51.9	37.0	2.00	0.82	0.81	1
S47738	BH2 7.45 - 7.56m	Sandstone	Diametral	-	48.0	0.49	0.21	0.21	1
			Axial	51.8	37.0	1.56	0.64	0.63	1
S47739	BH2 8.35 - 8.46m	Sandstone	Diametral	-	49.0	1.78	0.74	0.73	1
			Axial	51.8	30.0	1.35	0.68	0.64	1
S47740	BH2 9.32 - 9.43m	Sandstone	Diametral	-	49.0	0.77	0.32	0.32	1
			Axial	51.8	34.0	1.55	0.69	0.67	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.

NATA

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29/04/2019

Date

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