Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

NEW DEVELOPMENT

at

321-331 CONDAMINE STREET, MANLY VALE

Prepared For

Manly Vale Developments No.2 Pty Ltd

Project No.: 2020-028

May, 2020

Document Revision Record

Issue No	Date	Details of Revisions
0	9th March 2020	Original submission
1	14th May 2020	Updated architectural drawings

CROZIER
GEOTECHNICAL CONSULTANTS

TABLE OF CONTENTS

1.0	INTRODUCTION		Page 1
2.0	PROPOSED DEVELOPMENT		Page 2
3.0	SITE FEATURES		
	3.1.	Description	Page 2
	3.2.	Geology	Page 4
4.0	FIELD WORK		
	4.1	Methods	Page 4
	4.2	Field Testing	Page 5
	4.3	Laboratory Testing	Page 6
5.0	COMMENTS		
	5.1	Geotechnical Assessment	Page 7
	5.2	Site Specific Risk Assessment	Page 9
	5.3	Design & Construction Recommendations	
		5.3.1 New Footings	Page 10
		5.3.2 Excavation	Page 11
		5.3.3 Retaining Structures	Page 13
		5.3.4 Drainage & Hydrogeology	Page 14
	5.4	Conditions Relating to Design and Construction Monitoring	Page 14
6.0	CONCLUSION		Page 15
7.0	REFERENCES		Page 16

APPENDICES

1 Notes Relating to this Report

2 Figure 1 ï Site Plan, Figure 2ï Interpreted Geological Model,
Core Log Sheets

3 Laboratory Testing

4 Risk Tables

5 AGS Terms and Descriptions

Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

GEOTECHNICAL INVESTIGATION FOR PROPOSED NEW DEVELOPMENT
321-331 CONDAMINE STREET, MANLY VALE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed new development at 321-331 Condamine Street, Manly Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Manly Vale Development No. 2 Pty Ltd.

Northern Beaches (Warringah) Councilôs 2011 LEP and DCP states that all building development applications must be accompanied by a geotechnical landslip assessment. This site is located within landslip risk Class óAô within the Geotechnical Risk Management Map, and based on the proposed development, a geotechnical report is required in support of the DA.

As detailed by Council (and the RMS), boreholes must be extended to 3.0m below the basement excavation level (å 10.5m depth) to determine ground conditions for support design and to monitor the presence and impact of groundwater to the development.

Therefore, this report includes a landslide risk assessment of the site and proposed works, plans, geological sections, geotechnical analysis and groundwater assessment and provides recommendations for construction. It is recommended that the client make themselves aware of the Policy and its requirements.

The investigation and reporting were undertaken as per the Tender P20-044, Dated: 10th February 2020.

The investigation comprised:

a) Onsite service location and clearing of borehole locations by accredited contractor
b) Cutting of pavement in two locations (200mm diameter core holes)
c) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
d) Drilling of two boreholes involving augering through soils and then core drilling of rock with core photography and sample collection.
e) Rock core sampling and testing by NATA accredited laboratories for rock strength analysis.
f) Installation of two groundwater wells to assess groundwater seepage rate
g) Infill boreholes with drilling spoil

The following plans and drawings were supplied by the Architect for the work;

- Architectural drawings by Gartner Trovato Architects, Project No.: 1511, Drawing No.: DA-02 to DA-23, Revision: P1 to P6, Dated 12/05/2020.
- Survey Plan by SDG Land Development Solutions, Reference No.: 7909, Survey Date: 19/08/2019, Issue: B.
- Prelodgement Advice ï Northern Beaches Council, Application No.: PLM2019/0190, Dated: 24/09/2019

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve demolition of the existing structures and construction of a new four storey mixed-use development along with a two level underground basement for parking. The works will require bulk excavation of up to 7.50m depth to achieve the designed basement Finished Floor Levels of RL12.20 with isolated deeper (1.0m) excavation for lift pits. The excavations will extend to the north, south and east boundaries and within 1.40m of the west boundary with neighbouring buildings and road reserves along the common boundaries.

3. SITE FEATURES:

3.1. Description:

The site is a rectangular shaped block located on the high west side of Condamine Street at the north west corner of the intersection between Condamine Street and Sunshine Street, within gentle southeast dipping topography with Somerville Place passing along the rear west boundary. The site has front east and rear west boundaries of 38.10m, side north and south boundaries of 33.53m as referenced from the supplied survey plan.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data, as Photograph 1.

The site is currently occupied by five single and two-storey mixed-use brick buildings located at the front of the properties with concrete carparks at the rear.

Condamine Street and Sunshine Street are gently south and east dipping respectively and are formed with asphalt pavements with concrete gutters and kerbs. Summerville Place, passing the rear of the site, is gently south dipping and is formed of concrete slabs. The rear of the site contains near level concrete carparks. There were no signs of excessive cracking or deformation within the road pavements and carparks to suggest any movement or underlying geotechnical issues.

Photograph: 1 – Aerial photo of site and surrounds.

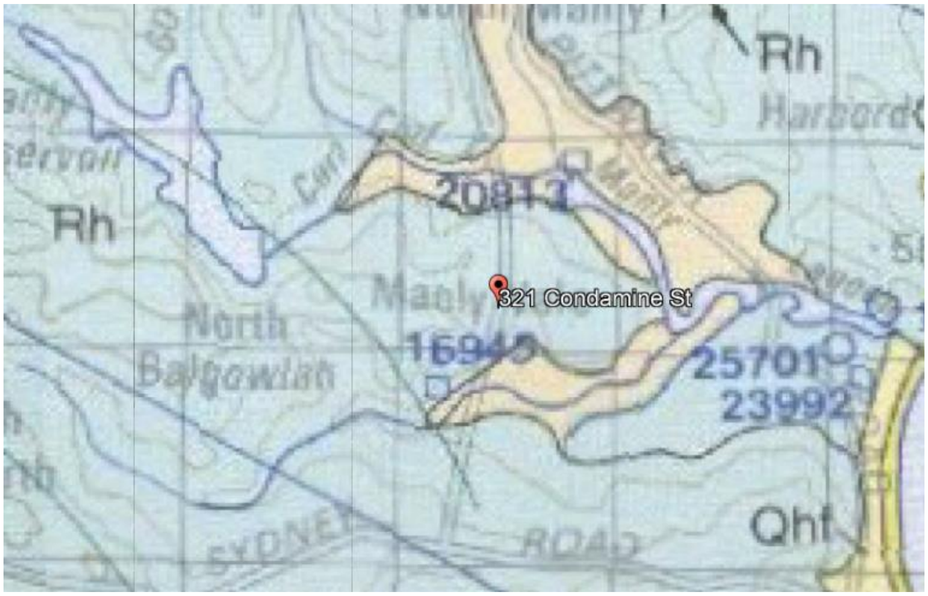
The existing buildings appear Ó40 years of age and appear in reasonable condition with no signs of significant cracking or settlement on the external walls.

The neighbouring property to the north (No. 333) contains a three-level mixed-use rendered building extending to all the boundaries. The structure is relatively new and appears in good condition with no signs of cracking or settlement on the external walls. The property has a similar ground floor level as the site along the common boundary.

The neighbouring buildings and properties were only inspected from within the site or from the road reserve however the visible aspects did not show any significant signs of large scale slope instability or other major geotechnical concerns which would impact the site or the proposed development.

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic age. The rock unit typically comprises of medium to coarse grained quartz sandstone with minor lenses of shale and laminite. This rock unit was identified in surface exposures and existing excavations within and adjacent to the site.



Extract of Sydney Geological Series Sheet

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and limited inspection of adjacent properties on the 21st February 2020 by a Geotechnical Engineer. It included a photographic record of site conditions with examination of existing structures and visible external arears of neighbouring properties.

It also included the drilling of two boreholes (BH1 and BH2) by the contractor BG Drilling Pty Ltd under the supervision of a geotechnical engineer from CGC.

Access to the site is limited by the existing structures. Therefore, the drilling of BH1 and BH2 was undertaken using a Dando Dual Terrier restricted access geotechnical drilling rig in the open, rear carpark. They were undertaken initially by solid stem, spiral flight augering through the near surface fill/soils prior to installing steel drilling casing. The boreholes were then extended utilising NMLC triple-tube techniques with water as a flush medium to recover samples to allow strata identification and for geotechnical rock strength testing. Following completion, ground water wells were installed in BH1 and BH2 and groundwater conditions were assessed.

Strata identification was undertaken on material recovered from the boreholes with samples collected as per ñAS1726: 2017 Geotechnical Site Investigationò for logging purposes and submission to NATA accredited laboratories.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed log sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

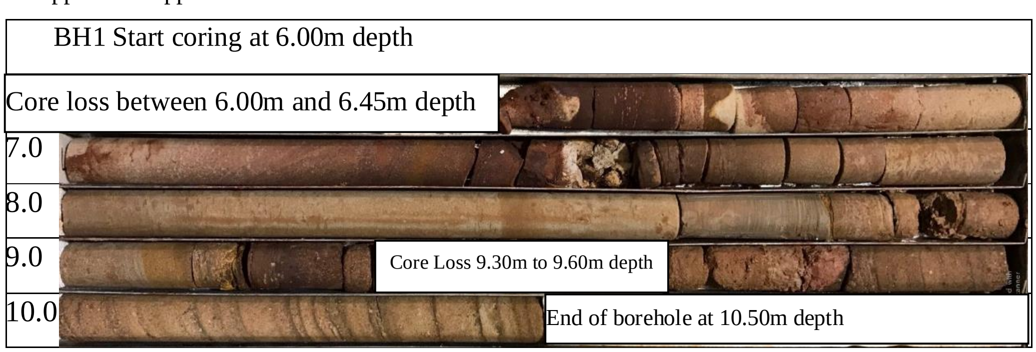
4.2. Field Testing:

BH1 was drilled adjacent to the north boundary at the centre of the site, whilst BH2 was drilled at the southwest corner of the site. Both boreholes were drilled to 10.50m depth below the existing ground surface level.

Based on the borehole logs and SPT test results, the sub-surface conditions at the project site can be classified as follows:

- **CONCRETE** – to 0.18m depth in both boreholes
- **FILL –** this layer was encountered in both boreholes below the concrete to 1.20m (BH1) and 1.30m (BH2) depth, respectively. It is classified as grey and brown, medium grained, moist sand/sand with gravel.
- **SAND –** this layer was encountered in BH1 below the fill to 4.50m depth. It is classified as very loose to loose, dark grey to grey, medium grained, moist sand.
- **Sandy CLAY** ï this layer was encountered in BH1 below the sand to 6.00m depth and in BH2 below the fill to 4.50m depth. It is classified as firm to very stiff, orange and pale grey, medium plasticity, moist sandy clay with bands of ironstone.
- **SANDSTONE** ï this layer was encountered in both boreholes below the sandy clay to the end of boreholes at 10.50m depth. It is classified as brown and grey, highly weathered to fresh, very low to medium strength, massive, coarse and fine to medium grained sandstone

Photographs of the recovered core are supplied below whilst detailed log sheets at each borehole location are supplied in Appendix: 2.



Core photograph – Borehole 1

BH2 Start coring at 4.50m depth, core loss between 4.50m and 5.00m depth

Core photograph – Borehole 2

Groundwater inflow was not encountered during the augered portion of BH1 drilling. Signs of water seepage inflow was encountered in BH2 at approximately 3.00m depth. Water was used as a flush medium during coring negating groundwater inflow observations.

Following completion of drilling, groundwater wells were installed in BH1 and BH2. Groundwater inflow was allowed to develop for over 24 hours prior to measurement on the 24th February and 4th March 2020. A groundwater table was not encountered within the depth of the borehole however seepage was encountered in both boreholes (<0.7L/min), with the inflow rate measured as shown in Table 1.

Table: 1 – Measured water inflow rate in boreholes

24th Feb Time	**Depth of water level(m)**		**Estimated inflow rate(L/min)**		**4th March Time**	**Depth of water level(m)**		**Estimated inflow rate(L/min)**	
	BH1	BH2	BH1	BH2		BH1	BH2	BH1	BH2
1:45pm	5.00	--	--	--	1:25pm	5.00	5.35	--	--
2:00pm	8.70	5.40	--	--	1:35pm	8.70	8.50	--	--
2:20pm	6.65	8.10	0.5	--	1:45pm	7.30	8.05	0.7	0.2
2:30pm	6.25	7.40	0.2	0.3	1:55pm	6.90	7.60	0.2	0.2
2:40pm	5.85	--	0.2	--	--	--	--	--	--

4.3. Laboratory Testing:

Directly following drilling, a series of samples were sealed and sent for testing at a NATA accredited laboratory (Macquarie Geotech) for confirmation of rock material strength via Point Load Strength Index (AS4133.4.1) method. The laboratory test reports are included in Appendix: 3.

The point load strength results ($I_{s\ 50}$) are detailed in Table: 1 below and were utilised to correlate rock material strength at other locations down the boreholes. It should be noted that the axial strength results are mostly suitable for classification due to horizontal bedding of the mudstone/interbedded bedrock.

Table: 2 – Laboratory Test Results for Rock Core

Borehole	Depth (m)	Rock Type	Moisture Content (%)	$I_{s\ 50}$ (MPa) Diametral	$I_{s\ 50}$ (MPa) Axial	Strength (AS1726-2017)
BH1	6.90-7.00	Sandstone	9.9	0.18	0.18	LS
BH1	7.85-8.00	Sandstone	8.6	0.31	0.34	MS
BH1	10.35-10.50	Sandstone	6.3	0.52	0.45	MS
BH2	5.05-5.20	Sandstone	7.6	0.75	0.88	MS
BH2	10.40-10.50	Sandstone	6.6	0.43	0.43	MS

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified the presence of concrete overlying a layer of fill to a maximum depth of 1.30m and then very loose to loose sand, firm to very stiff sandy clay to varying depths between 4.50m (RL14.69) and 6.00m (RL13.47), under which sandstone bedrock was encountered. The sandstone bedrock is of low to medium strength from the bedrock surface with a band of extremely low strength to very low strength encountered between 8.60m (RL10.87) and 9.80m (RL9.67) depth in BH1. Groundwater seepage inflow was encountered with an inflow rate of <0.7L/min determined at a level >5.00m depth.

The works require excavation of up to 7.50m depth to achieve the basement level with FFL12.20. The excavations will extend to the north, south and east boundaries and within 1.40m of the west boundary with a neighbouring building and road reserves along the common boundaries.

Based on the investigation results, the proposed excavation is anticipated to encounter fill, very loose to loose sand and firm to very stiff sandy clay to a maximum depth of 6.00m. Low to medium strength sandstone bedrock is expected from 4.50m depth with a zone/zones of extremely low strength to very low strength bedrock possibly at depth.

The significant depth of sand identified in BH1 is unusual for the site location and was not encountered in BH2 or to such significant depth in previous investigation in the adjacent site (No. 333). This unit will have a significant impact on the type/style of support required for the excavation. Therefore, it is recommended that a series of auger boreholes be undertaken prior to final engineering design and construction tendering to confirm the extent of this horizon. This will also need to include the eastern boundary of the site following demolition of existing structures, which are preventing access at present.

Considering the depth of excavation and distance to the boundaries, the recommended safe temporary batter slopes provided in Section 5.3.2 are not achievable along all the boundaries. Therefore, we recommend the installation of support measures prior to bulk excavation (i.e. bored pile wall or similar) to maintain stability. Driven concrete, steel (ie. sheet piles) or timber piles are not recommended on this site due to significant vibrations generated during installation of these structures.

Bored concrete piles could be utilized in the excavation support and/or for new footings. These structures will need to be installed as a contiguous structure in support systems due to the free running nature of the sandy soils or with a method that ensures the integrity of the foundation external to site boundaries is maintained and that over excavation of adjacent soils does not occur. Lateral support to the wall can be applied by braced or anchored systems, though anchors will require approval from adjacent property owners. For bored piles installed below groundwater, as per seepage identified below 5.00m depth, steel liners and tremie placement of concrete or CFA methods will be required to ensure maintenance of foundation and socket integrity, which should be allowed for in project costing/timing.

Careful consideration will have to be given to size of piling rig/machinery proposed to install any piled wall retention system especially, but not only, if a cantilever design is adopted due to the strength and quality of the bedrock encountered. It will be necessary to ensure that the method of installation selected is capable of drilling medium strength sandstone to avoid potentially significant (and costly) delays on site.

The medium strength bedrock that is expected to be encountered will be self-supporting with no indications from the boreholes of potential large scale instability or poor quality rock masses, except the extremely low strength to very low strength zones (could be supported post-excavation), requiring support systems. Where medium strength bedrock is excavated at steep (0.25H:1.0V) to vertical batter slopes, it will require geotechnical inspection at regular intervals to allow identification of any variations to the expected conditions and/or areas of poor quality bedrock and application of suitable support systems.

The excavation of low to medium strength rock will require the use of rock excavation equipment in combination with saw cutting. Rock excavation equipment can produce ground vibrations of a level which can potentially cause damage to neighbouring structures. Therefore selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use and the results of the vibration calibration testing. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use.

Based on the investigation results, a permanent water table or significant seepage flow was not encountered. As such there will be no requirement for significant dewatering and no lowering the water table. It is considered that there will be a negligible impact to the local hydrogeological conditions or adverse effect to neighbouring properties or the road reserve as a result of the proposed excavation and development due to any groundwater removal required. Therefore, tanking of the basement will not be required.

The recommendations and conclusions in this report are based on an investigation utilising a limited number of boreholes, site mapping and CGCõs extensive experience in excavations within similar geological and topographic setting. The boreholes provide small isolated data points across the entire site therefore some minor variations to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for assessment of the DA and preliminary design.

5.2. Site Specific Risk Assessment:

Based on our site investigation we have identified the following credible geological/geotechnical hazard which need to be considered in relation to the existing site and the proposed works. The hazards are:

A. Landslip of soils from basement excavation (<15m³)
B. Landslip of rock around perimeter of excavation for basement (<3m³)

A qualitative assessment of risk to life and property related to this hazard is presented in Table A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

Hazard A was estimated to have a **Risk to Life** of up to **2.34 x 10^{-3}** for a single person, whilst the **Risk to Property** was considered to be **'Very High'** in some situations.

Hazard B was estimated to have a **Risk to Life** of **1.56 x 10^{-5}** for a single person, whilst the **Risk to Property** was considered to be **'Moderate'**.

Although the óVery Highô Risk to Property for Hazard A and óModerateô Risk to Property for Hazard B is considered to be óUnacceptableô, the assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including regular detailed geotechnical mapping of the excavation and installation of determined support systems in timely manner the likelihood of any failure becomes óRareô and as such the consequences reduce and risk becomes within óAcceptableô levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAôsite for footings into bedrock at base of excavation
Type of Footing	Strip/Pad or Slab at base of excavation, Piles external to the excavation or for higher loads
Sub-grade material and Suggested Allowable End Bearing Capacity for bored piles*	- Weathered LS Sandstone: 1000kPa - MS Sandstone below RL9.50: 3500kPa*
Sub-grade material and Suggested Ultimate Shaft Adhesion for bored piles (compression)**	- ELS-VLS Sandstone: 150kPa - LS Sandstone: 400kPa - MS Sandstone: 800kPa
Basic geotechnical strength reduction factor as per Piling Design and installation AS 2159 - 2009	$\phi_{gb} = 0.52$
Site sub-soil classification as per Structural design actions AS1170.4 ï 2007, Part 4: Earthquake actions in Australia	B_e ï Rock Site Hazard factor (z) = 0.08

Remarks:

*End bearing pressure to cause settlement of <1% of minimum footing dimension

**Clean socket of roughness category R2 or better. (Pells, 1999)

All footings should be founded off material of similar strength/density to reduce the potential for differential settlement, unless allowance for variable movement is implemented in the design and construction.

Driven concrete, steel (i.e. sheet piles) or timber piles are not recommended on this site due to significant vibrations generated during installation of these structures.

Groundwater seepage was encountered at approximately 5.00m depth. Should a bored pile footing extend below the seepage inflow level then steel liners and tremie placement of concrete will be required to ensure maintenance of foundation integrity, which should be allowed for in project costings/timing. To avoid delays/variations, a CFA piling method could be considered.

Particular attention to be paid to the base cleaning of open bored pile footings otherwise end bearing capacity could be reduced significantly.

All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedôat the end of the project.

5.3.2. Excavation:

Depth of Excavation	Up to 7.50m depth
Distance of Excavation to Neighbouring Properties/structures	Condamine Street ï on boundary Sunshine Street ï on boundary Somerville Place ï 1.40m off boundary No. 333 ï on boundary, building on boundary
Type of Material to be Excavated	Granular fill up to 1.30m depth
	Very loose to loose sand up to 4.50m depth
	Sandy clay up to 6.00m depth
	LS-MS sandstone from 4.50m to 6.00m depth to below excavation level

Guidelines for unsurcharged batter slopes are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Granular fill/ very loose to loose sand	1.5:1	2.5:1
Firm to very stiff sandy clay	1:1	1.5:1
Very low strength bedrock	0.75:1*	1:1*
Low to medium strength, defect free bedrock	Vertical*	Vertical*

Remarks:

*Dependent on assessment by geotechnical engineer.

Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Soils / ELS	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS to MS bedrock	Rock hammer and saw

ELS ï extremely low strength, VLS ï very low strength, LS ï low strength, MS ï medium strength

Remarks:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Hawkesbury Sandstone bedrock, to maintain a 5mm/s PPV level of vibration the below hammer weights and buffer distances are required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
Ó1 tonne	20.00m

Onsite calibration of equipment and full time monitoring will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of proposed equipment and review of dilapidation surveys prior to excavation is required.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Residential buildings = 5mm/s (to maintain human comfort) Services = 3mm/s
Vibration Calibration Tests Required	Subject to proposed excavation machinery (if >250kg hammer)
Full time vibration Monitoring Required	Subject to Calibration test results and proper equipment
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • During installation of the excavation support system • Where unexpected ground conditions are identified or any other concerns are held • Where unsupported excavation in LS-MS sandstone occurs • At completion of the excavation • Following footing excavations to confirm founding material strength
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.

Remarks:

Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.

5.2.3. Retaining Structures:

Required	New retaining structures/excavation support wall will be required as part of the proposed development
Types	Pre-excavation piling or similar for basement. Designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.

Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m³)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill/loose sand/firm to stiff sandy clay	18	ϕ' = 28°	0.35	0.52	N/A
Silty clay (very stiff)	20	ϕ' = 32°	0.27	0.40	N/A
LS or fractured bedrock	23	ϕ' = 38°	0.10	0.15	300kPa
MS bedrock (defect free)	24	ϕ' = 42°	0.00	0.01	600kPa

Remarks:

In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided to allow release of groundwater seepage. If this is not done, then the walls should be designed to support full hydrostatic pressure. It is suggested that post excavation retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients and incorporate surcharge loading to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

Soil/Rock Anchors	Where extend across boundary will require permission from neighbouring property owners and should be based on a temporary system with permanent support implemented by the proposed development.
Ultimate Bond Stress	Very Stiff Clay = 30kPa Extremely Low Strength Rock = 150kPa Very Low Strength Rock = 200kPa Low Strength Rock = 350kPa

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Yes, below 5.00m depth (<0.7L/min) at soil/rock interface and on defects in bedrock
Excavation likely to intersect	Water Table	No
	Seepage	Yes
Site Location and Topography		High west side of the road, within gently southeast dipping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		No

Remarks:
As the excavation faces are expected to encounter seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Councilôs stormwater system off site.

5.4. Conditions Relating to Design and Construction Monitoring:

To allow CGC to provide post development certification, it will be necessary for Crozier Geotechnical Consultants to:

1. Complete additional investigation adjacent to road reserves for depth of sand horizon extents and condition including adjacent to Condamine Street where access is not currently available,
2. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
3. Inspection of site and works as per Section 5.3.2 of this report
4. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,

6. CONCLUSION:

The site investigation identified the presence of concrete overlying a layer of fill to a maximum depth of 1.30m and then very loose to loose sand, firm to very stiff sandy clay to varying depths between 4.50m (RL14.69) and 6.00m (RL13.47), under which low to medium strength sandstone bedrock with extremely low to very low strength bands was encountered. Groundwater seepage inflow was encountered at approximately 5.00m depth in both boreholes with a rate of <0.8L/min.

The works require excavation of up to 7.50m depth to achieve the basement level with FFL12.20. The excavations will extend to the north, south and east boundaries and within 1.40m of the west boundary with neighbour building and road reserve along the common boundaries.

It is recommended that support measures be installed prior to bulk excavation (i.e. bored pile wall or similar) to maintain stability.

The use of rock excavation equipment and dedicated piling rigs should be considered in project costing/timing for the identified low to medium strength sandstone. Should a bored pile footing installed below the seepage inflow level (approximately 5.00m depth) be required, then steel liners and tremie placement of concrete or CFA methods will be required to ensure maintenance of foundation integrity.

The risks associated with the proposed development are and can be maintained within óAcceptableô levels with negligible impact to neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared By:	Reviewed By:
Jun Yan	Troy Crozier
Geotechnical Engineer	Principal
	MEng, BSc, Dip. Civ. Eng
	MAIG, PRGeo ï Geotechnical and Engineering
	Registration No.: 10197

7. REFERENCES:

1. Geological Society Engineering Group Working Party 1972, ñThe preparation of maps and plans in terms of engineering geologyò Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
2. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
3. Australian Standard AS 3798 ï 2007, Guidelines on Earthworks for Commercial and Residential Developments.
4. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings ï Construction
5. Australian Standard AS1170.4 ï 2007, Part 4: Earthquake actions in Australia
6. Pells, PJN, Mostyn. G, Walker. BF; Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Society, Journal No. 33, 1998.

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

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

Description and classification Methods

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

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination.
The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT “N” Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

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

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

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

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

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7
  as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

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

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

$$Q_c = (12 \text{ to } 18) \, C_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Borehole Logs

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

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

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm " " " " "	C	Core		

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

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

Site Anomalies

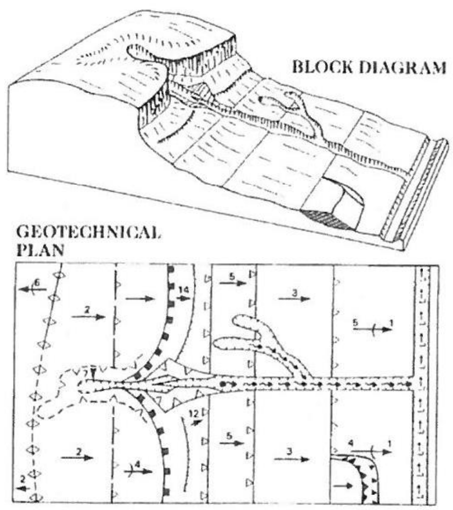
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

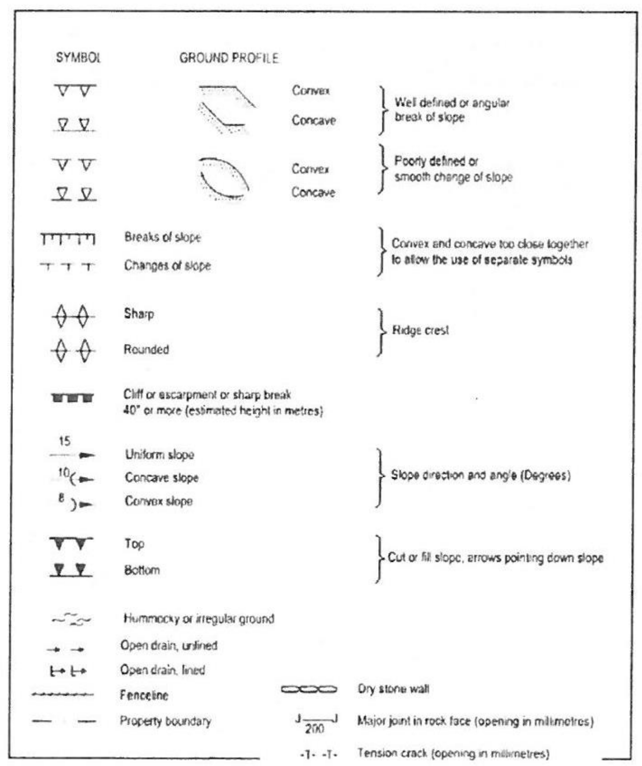
Reproduction of Information for Contractual Purposes

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Tender Documents”, published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

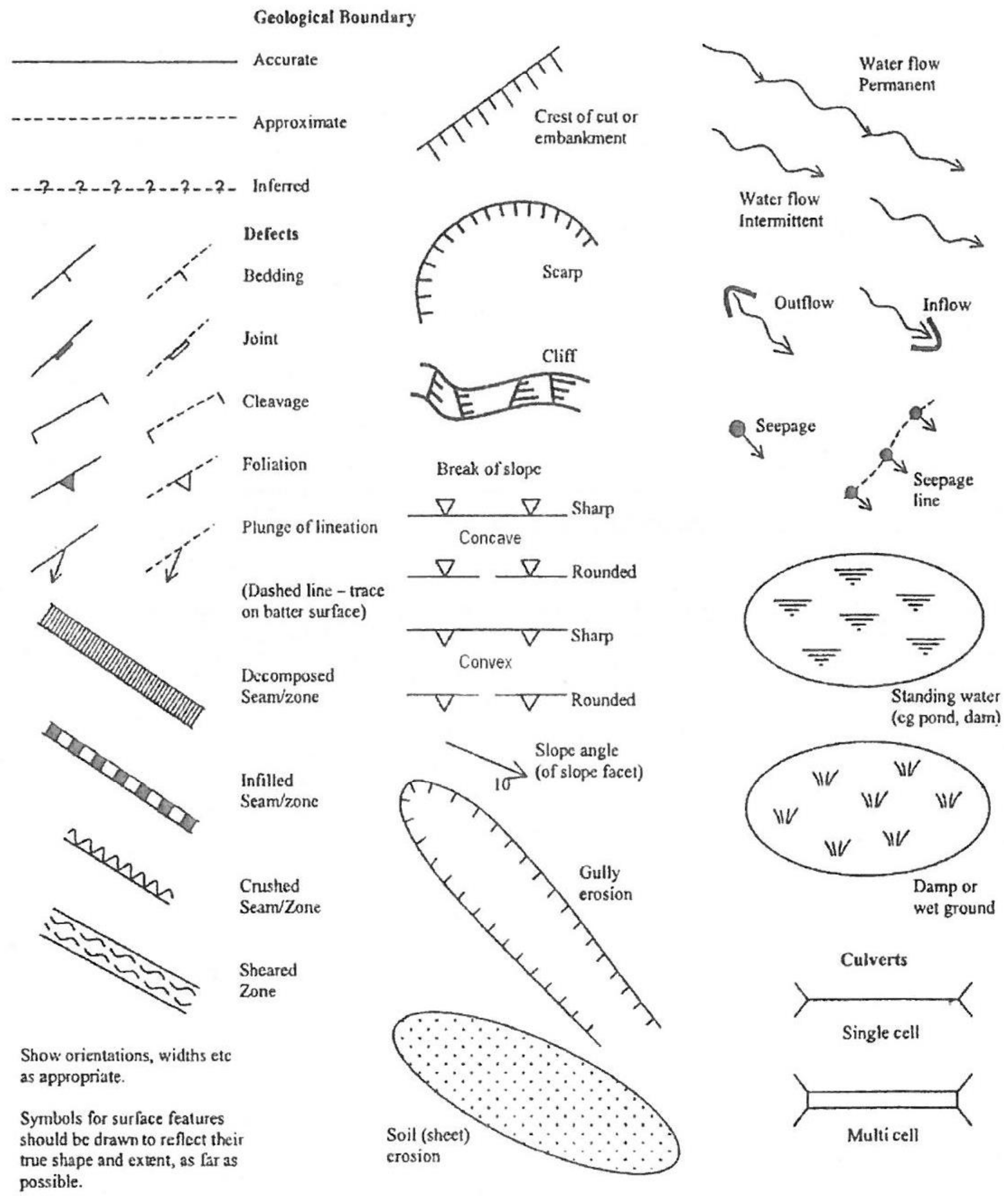
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





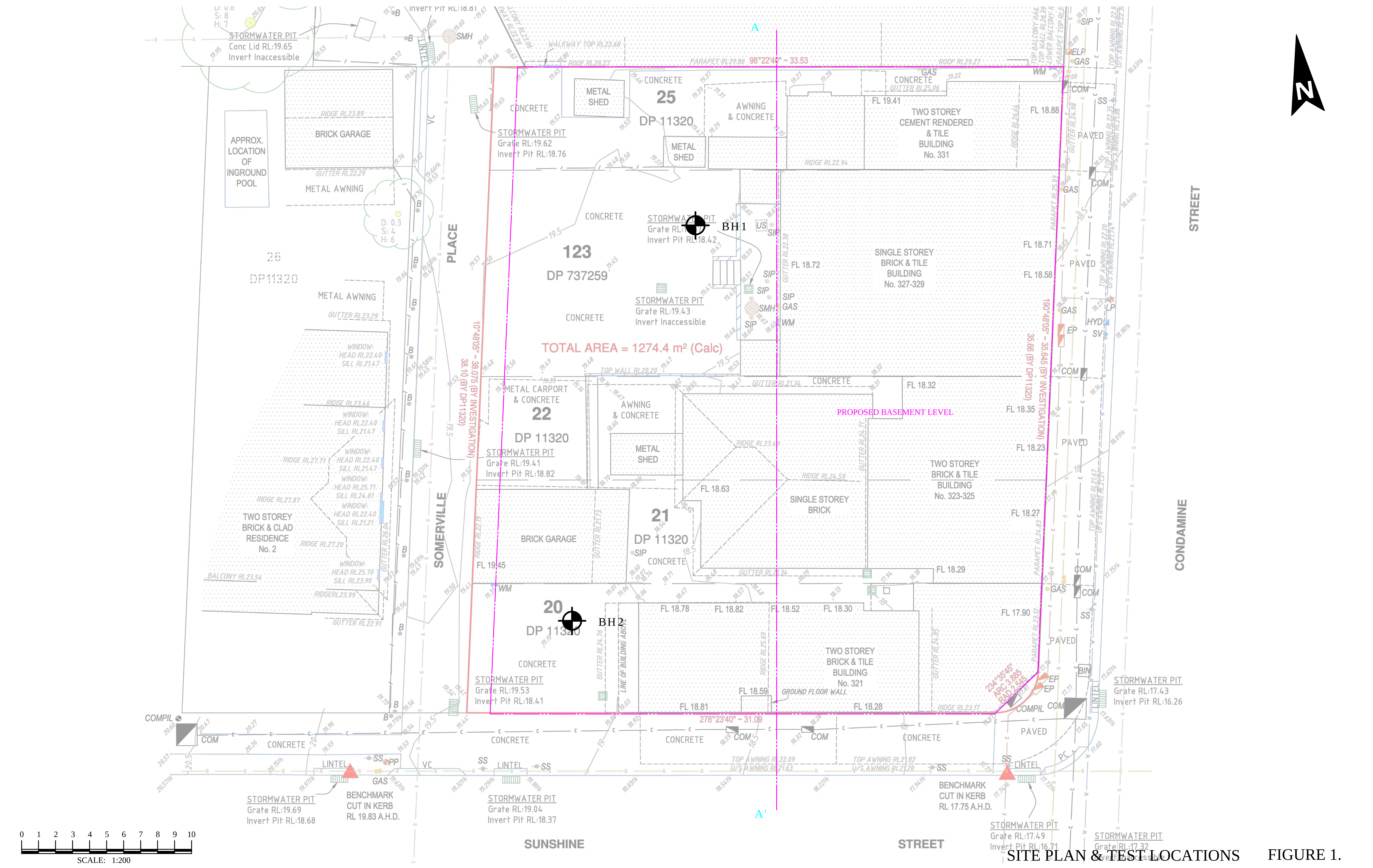
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

LEGEND

A — A' CROSS-SECTION REFERENCE LINE

PROPERTY BOUNDARY

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

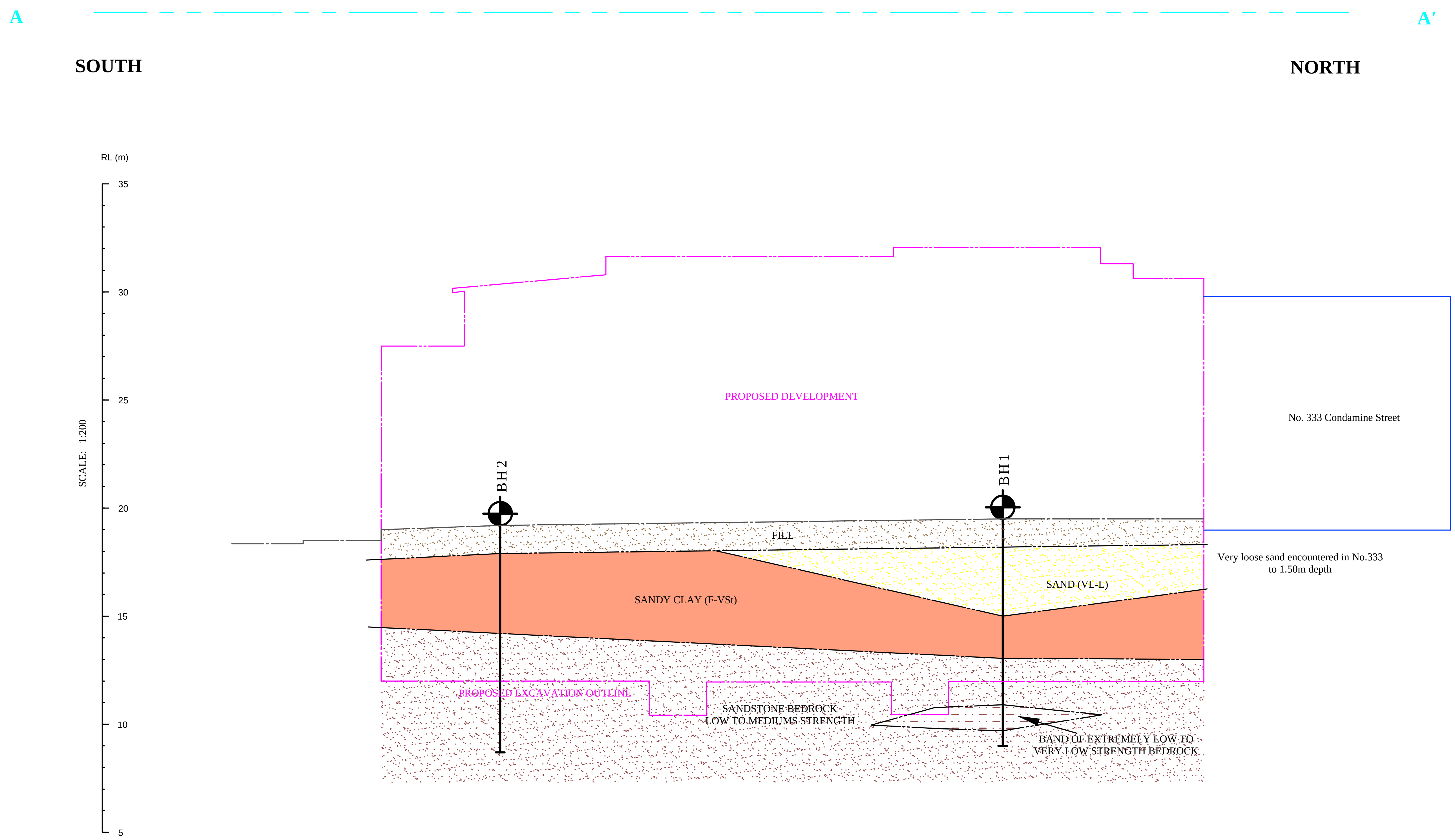
CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical ABN: 96 113 453 624
Unit 12, 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	03/03/2020
APPROVED BY:	TMC
DRAWN BY:	JY
PROJECT:	2020-028

PREPARED FOR:
Manly Vale Developments No. 2 Pty Ltd

ADDRESS:
321-331 Condamine Street, Manly Vale



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 2.

LEGEND

A — A' CROSS-SECTION REFERENCE LINE	BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION	SAND	Band of ELS to VLS bedrock
PROPERTY BOUNDARY	FILL	SANDSTONE BEDROCK	SANDY CLAY

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical ABN: 96 113 453 624
Unit 12, 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 09/03/2020

APPROVED BY: TMC
DRAWN BY: JY
PROJECT: 2020-028

PREPARED FOR:
Manly Vale Developments No. 2 Pty Ltd

ADDRESS:
321-331 Condamine Street, Manly Vale

CROZIER
GEOTECHNICAL CONSULTANTS

Client: Manly Vale Developments No.2 Pty Ltd **Date:** 21/02/2020 **Borehole:** 1

Project: New development **Project No.:** 2020-028

Location: 321-331 Condamine Street, Manly Vale **Surface Level:** RL19.47

Depth (m)	Description of Strata (Soil/rock name, grainsize, texture/fabric, colour)	Discontinuities	Weathering (Extremely, Highly, Moderately, Slightly, Fresh)	Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Defect Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Installation Details	Type	Depth (m)	Core Rec. %	RQD %	Test Results
	CONCRETE										
0.18	FILL: Brown, medium grained, moist sand with gravel										
1.00											
1.20	SAND: Very loose, dark grey, medium grained, moist sand							1.50			SPT 1, 1, 0 N = 1
2.00											
3.00	é becoming loose							3.00			SPT 3.00-3.45m 3, 3, 3 N = 6
4.00											
4.50	Sandy CLAY: Very stiff, pale grey, medium plasticity, moist sandy clay with gravel and band of ironstone							4.50			SPT 4.50-4.95 3, 7, 18 N = 25
5.00											
6.00	Start core at 6.00m depth										
	CORE LOSS BETWEEN 6.00m and 6.45m depth										
	SANDSTONE: Coarse grained, massive, brown								70%	35%	
7.00											6.90 - 7.00m: $I_{s(50)}$ = 0.18MPa (Diametral), $I_{s(50)}$ = 0.18MPa (Axial), Moisture Content = 9.9%
								7.50			
7.85	é becoming fine grained, pale brown, sandstone										7.85 - 8.00m: $I_{s(50)}$ = 0.31MPa (Diametral), $I_{s(50)}$ = 0.34MPa (Axial), Moisture Content = 8.6%
8.00											
8.60	é band of silty clay between 8.60m and 8.80m depth										
8.80	é becoming coarse grained, brown sandstone								83%	50%	
9.00											
9.30	CORE LOSS BETWEEN 9.30m and 9.60m depth										
9.60											
10.00											10.35 - 10.50m: $I_{s(50)}$ = 0.52MPa (Diametral), $I_{s(50)}$ = 0.45MPa (Axial), Moisture Content = 6.3%
10.50	End of borehole at 10.50m depth							10.50			
11.00											

Rig: Multi-Purpose CE180 **Driller:** BG

Type of Boring: Auger to 6.00m depth, then NMLC coring **Logged By:** JY

Water Observations: No freestanding groundwater encountered druing auger drilling **Casing:** 6.00m

Comments:

CROZIER GEOTECHNICAL CONSULTANTS

Client: Manly Vale Developments No.2 Pty Ltd **Date:** 21/02/2020 **Borehole:** 2

Project: New development **Project No.:** 2020-028

Location: 321-331 Condamine Street, Manly Vale **Surface Level:** RL19.19

Depth (m)	Description of Strata (Soil/rock name, grainsize, texture/fabric, colour)	Discontinuities	Weathering	Rock Strength	Defect Spacing	Installation Details	Type	Depth (m)	Core Rec. %	RQD %	Test Results
	CONCRETE										
0.18	FILL: Dark grey, medium grained, moist sand										
1.00											
1.30	Sandy CLAY: Firm to stiff, orange, low to meidum plasticity, moist sandy clay							1.50			SPT 3, 2, 2 N = 4
2.00											
3.00	é wet at retrieval of SPT rod below 3.00m depth é becoming pale grey below 3.00m depth							3.00			SPT 6, 6, 8 N = 14
4.00	é becoming red brown below 4.00m depth										
4.50	Start core at 4.50m depth CORE LOSS BETWEEN 4.50m and 5.00m depth										
5.00	SANDSTONE: Fine to medium grained, massive, red brown										5.05-5.20m: $I_{s(50)}$ = 0.75MPa (Diametral), $I_{s(50)}$ = 0.88MPa (Axial), Moisture Content = 7.6%
5.70	é becoming fine grained, pale brown										
6.00								6.00			
7.00											
8.00											
8.80	é band of shale										
9.00	é becoming coarse grained, brown sandstone							9.00			
10.00											
10.50	End of borehole at 10.50m depth										10.40-10.50m: $I_{s(50)}$ = 0.68MPa (Diametral), $I_{s(50)}$ = 0.43MPa (Axial), Moisture Content = 6.6%
11.00											

Weathering: Extremely, Highly, Moderately, Slightly, Fresh. Rock Strength: Ex. Low, Very Low, Low, Medium, High, Very High. Defect Spacing: < 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m. Sampling and In Situ Testing.

Rig: Multi-Purpose CE180 **Driller:** BG

Type of Boring: Auger to 4.50m depth, then NMLC coring **Logged By:** JY

Water Observations: Sign of water on retrieval of SPT rod at 3.00m depth **Casing:** 4.50m

Comments:

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

POINT LOAD STRENGTH INDEX REPORT

Client:	Crozier Geotech	**Moisture Content Condition:**	As received
Address:	Unit 12/ 42-46 Wattle Street Brookvale NSW 2100	**Storage History:**	Sealed
Project:	321-331 Condamine Street Manly Vale (2020-028)	**Report No:**	S58149-PL
Job No:	S20089-1	**Date Tested:**	27/02/2020

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

Sampling: Sampled by Client **Date Sampled:** 21/02/2020

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
S58149	BH1 6.90-7.0m	Sandstone	Diametral	-	45.0	0.39	0.19	0.18	1
			Axial	49.5	39.0	0.45	0.18	0.18	1
S58150	BH1 7.85-8.0m	Sandstone	Diametral	-	47.0	0.70	0.32	0.31	1
			Axial	49.8	33.0	0.73	0.35	0.34	1
S58151	BH1 10.35-10.50m	Sandstone	Diametral	-	48.0	1.21	0.53	0.52	1
			Axial	50.9	37.0	1.08	0.45	0.45	1
S58152	BH2 5.05-5.20m	Sandstone	Diametral	-	48.0	1.77	0.77	0.75	1
			Axial	50.7	37.0	2.13	0.89	0.88	1
S58153	BH2 10.40-10.50m	Sandstone	Diametral	-	48.0	1.60	0.69	0.68	1
			Axial	50.8	38.0	1.06	0.43	0.43	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

Accredited for compliance with ISO/IEC 17025 - Testing.

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

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

5/03/2020

Date

MACQUARIE GEOTECH

This report cancels and supersedes Report no. S58149-PL Issued 04/03/2020

Macquarie Geotechn
U7/8 10 Bradford Street
Alexandria NSW

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 4

<u>**TABLE : A**</u>

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide		Spatial Impact of Slide				Occupancy	Evacuation		Vulnerability	Risk to Life	
A	Landslip of soils from basement excavation (<15m³)		Excavation up to approximately 6.00m of fill, sand and sandy clay **Condition A**: Insufficient retention - **Condition B**: Engineer designed and implemented supoprt		a) excavation on boundary, footpath along boundary b) excavation on boundary, pavement 3.0m off boundary c) excavation on boundary, footpath along boundary d) excavation on boundary, pavement 3.0m off boundary e) excavation 1.40m off boundary, pavement on boundary f) excavation on boundary, neighbouring building on boundary				a) Person in footpath 12hrs/day avge b) Car in pavement 24hrs/day avge c) Person in footpath 12hrs/day avge d) Car in pavement 24hrs/day avge e) Car in pavement 8hr/day avge f) Person in building 24hr/day avge	Condition A a) Likely to not evacuate b) Likely to not evacuate c) Likely to not evacuate d) Likely to not evacuate e) Likely to not evacuate f) Likely to not evacuate	Condition B a) Unlikely to not evacuate b) Unlikely to not evacuate c) Unlikely to not evacuate d) Unlikely to not evacuate e) Unlikely to not evacuate f) Unlikely to not evacuate	a) Person in open space, buried b) Person in car, unlikely buried c) Person in open space, buried d) Person in car, partly buried e) Person in car, partly buried f) Person in building, minor damage	Condition A	Condition B
			Condition A	Condition B	Condition A		Condition B							
			Almost Certain	**Rare**	**Prob. of Impact**	**Impacted**	**Prob. of Impact**	**Impacted**						
		a) footpath on Condamine Street	0.1	0.00001	0.25	0.25	0.25	0.10	0.50	0.75	0.25	1.00	**2.34E-03**	**3.13E-08**
		b) pavement on Condamine Street	0.1	0.00001	0.25	0.05	0.10	0.01	1.00	0.75	0.25	0.25	**2.34E-04**	**6.25E-10**
		c) footpath on Sunshine Street	0.1	0.00001	0.25	0.25	0.25	0.10	0.50	0.75	0.25	1.00	**2.34E-03**	**3.13E-08**
		d) pavement on Sunshine Street	0.1	0.00001	0.25	0.05	0.10	0.01	1.00	0.75	0.25	0.25	**2.34E-04**	**6.25E-10**
		e) pavement on Somerville Place	0.1	0.00001	0.25	0.50	0.25	0.20	0.33	0.75	0.25	0.25	**7.73E-04**	**1.03E-08**
		f) No. 333	0.1	0.00001	0.25	0.10	0.25	0.05	1.00	0.75	0.25	0.10	**1.88E-04**	**3.13E-09**
B	Landslip of bedrock around perimeter of excavation for basement (<3m³)		Excavation up to approximately 3.00m into bedrock below soils		a) excavation on boundary, footpath along boundary b) excavation on boundary, pavement 3.0m off boundary c) excavation on boundary, footpath along boundary d) excavation on boundary, pavement 3.0m off boundary e) excavation 1.40m off boundary, pavement on boundary f) excavation on boundary, neighbouring building on boundary				a) Person in footpath 12hrs/day avge b) Car in pavement 24hrs/day avge c) Person in footpath 12hrs/day avge d) Car in pavement 24hrs/day avge e) Car in pavement 8hr/day avge f) Person in building 24hr/day avge	Condition A a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate d) Possible to not evacuate e) Possible to not evacuate f) Possible to not evacuate	Condition B a) Rare to not evacuate b) Rare to not evacuate c) Rare to not evacuate d) Rare to not evacuate e) Rare to not evacuate f) Rare to not evacuate	a) Person in open space, buried b) Person in car, unlikely buried c) Person in open space, buried d) Person in car, partly buried e) Person in car, partly buried f) Person in building, minor damage	Condition A	Condition B
			Condition A	Condition B	Condition A		Condition B							
			Possible	**Rare**	**Prob. of Impact**	**Impacted**	**Prob. of Impact**	**Impacted**						
		a) footpath on Condamine Street	0.001	0.00001	0.25	0.25	0.25	0.10	0.50	0.5	0.01	1.00	**1.56E-05**	**1.25E-09**
		b) pavement on Condamine Street	0.001	0.00001	0.25	0.05	0.10	0.01	1.00	0.5	0.01	0.25	**1.56E-06**	**2.50E-11**
		c) footpath on Sunshine Street	0.001	0.00001	0.25	0.25	0.25	0.10	0.50	0.5	0.01	1.00	**1.56E-05**	**1.25E-09**
		d) pavement on Sunshine Street	0.001	0.00001	0.25	0.05	0.10	0.01	1.00	0.5	0.01	0.25	**1.56E-06**	**2.50E-11**
		e) pavement on Somerville Place	0.001	0.00001	0.25	0.50	0.25	0.20	0.33	0.5	0.01	0.25	**5.16E-06**	**4.13E-10**
		f) No. 333	0.001	0.00001	0.25	0.10	0.25	0.05	1.00	0.5	0.01	0.10	**1.25E-06**	**1.25E-10**

* hazards considered in current condition and/or without remedial/stabilisation measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip of soils from basement excavation (<15m³)	a) footpath on Condamine Street	Almost Certain	Event is expected to occur over design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very High
		b) pavement on Condamine Street	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		c) footpath on Sunshine Street	Almost Certain	Event is expected to occur over design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very High
		d) pavement on Sunshine Street	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		e) pavement on Somerville Place	Almost Certain	Event is expected to occur over design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very High
		f) No. 333	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
B	Landslip of bedrock around perimeter of excavation for basement (<3m³)	a) footpath on Condamine Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		b) pavement on Condamine Street	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
		c) footpath on Sunshine Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		d) pavement on Sunshine Street	Possible	The event could occur under adverse conditions over the design life.	Medium	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		e) pavement on Somerville Place	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		f) No. 333	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event.
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace.	Every two years or following major rainfall event.
Large Trees on or adjacent to site	Arbourist to check condition of trees and remove as required. Where treee within steep slopes or adjacent to structures require geotechincal inspection prior to removal	Every five years
Slope Stability	Hydraulics (stormwater) & Geotechnical Consultants to check on site stability at same time and provide report.	One year after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with Councils Risk Management Policy.

Appendix 5

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-6}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	1: CATASTROPHIC 200%	2: MAJOR 60%	3: MEDIUM 20%	4: MINOR 5%	5: INSIGNIFICANT 0.5%
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.