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Crozier Geotechnical Consultants a division of PJC Geo-Engineering Pty Ltd

REPORT ON PRELIMINARY GEOTECHNICAL INVESTIGATION

for

PROPOSED MIXED USE DEVELOPMENT
at

1 – 3 NARRABEEN PARK PARADE, NORTH NARRABEEN

Prepared For

UNITY PTY. LTD.

Project: 2016-092.1

December, 2016

Document Revision Record

Issue No	Date	Details of Revisions
0	24th May 2016	Original Issue
1	16th December 2016	Additional Investigation for DA Issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for____	
	Name of Applicant
Address of site 1 – 3 Narrabeen Park Parade, North Narrabeen	

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, __***Troy Crozier***___________ *on behalf of* ___***Crozier Geotechnical Consultants*** ______

on this the _16th December 2016_______ certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million.

I:

- [x] have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- [ ] am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- [ ] have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- [ ] have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- [ ] have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- [ ] have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Proposed Mixed Use Development at 1 – 3 Narrabeen Park Parade, North Narrabeen	
Report Date: 16th December 2016	**Project No.:** 2016-092.1
Author: T. Crozier	
Author's Company/Organisation: Crozier Geotechnical Consultants	

Documentation which relate to or are relied upon in report preparation:

Site Survey by Usher and Company, Reference: 5551-DET, Dated: 08/04/2015
Design by 02 Architecture, Job No.: 1608, Drawing No.: A1.00 to A1.07, Dated: June 2016

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ....

Name ...**Troy Crozier**...

Chartered Professional Status...**RPGeo (AIG)**.........................

Membership No. ...**10197**..

Company... **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10,197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for______________________________________

Name of Applicant

Address of site ___1 – 3 Narrabeen Park Parade, North Narrabeen ___________________

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Proposed Mixed Use Development at 1 – 3 Narrabeen Park Parade, North Narrabeen
Report Date: 16th December 2016 **Project No.:** 2016-092.1
Author: T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___16/05/16 and 25/11/16___________________________ (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ……………………………………………………..
  - ■ Yes Date conducted ……16/05/16 and 25/11/16………………
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other ……………………………………………… specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……[signature]……………………………

Name …**Troy Crozier**………………………………

Chartered Professional Status…**RPGeo (AIG)**……… ………

Membership No. …**10197**……………………………………

Company… **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST AUSTRALIAN INSTITUTE OF GEOSCIENTISTS TROY CROZIER 10,197

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CROZIER
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GEOTECHNICAL REPORT FOR PROPOSED MIXED USE DEVELOPMENT
1-3 NARABEEN PARK PARADE, NORTH NARRABEEN, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed mixed use development at 1 ï 3 Narrabeen Park Parade, North Narrabeen, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client representative Mr. Peter Gurtner of Unity Australia.

The site is situated on the slightly higher, eastern side of the road and consists of two rectangular shaped properties. The southern property (No. 1) contains a single storey brick restaurant building at the western end with a separate garage at the rear, excavated into the hill slope. The northern property (No. 3) contains a three storey rendered residential dwelling with driveway at the front.

It is understood that the proposed works involve demolition of existing site structures and construction of a new four storey residential and commercial structure with basement car park. The new structure will contain residential unit dwellings at the upper levels with commercial premises at the ground floor. The basement level will extend to all side boundaries and is formed with a Finished Floor Level at R.L. 2.32. As such an excavation of up to 2.50m depth is proposed across most of the site with the excavation depth increasing towards the north-east corner due to the rise in ground surface levels.

A review of Pittwater Councils LEP/DCP 2014 identified that the site is located within the highest landslip hazard zone, H1 (GTH_019) and within Acid Sulphate Soils óClass 3 and 5ô (ASS_019). For works involving significant excavations or development works within land classified as H1 a detailed geotechnical assessment and report is required that meets the requirements of their Geotechnical Risk Management Policy 2009. For acid sulphate soils óClass 3ô zoning an assessment is required where works will extend beyond 2.0m depth below ground level and/or works that will result in lowering the natural water table beyond 2.0m below the natural surface. Class 5 land requires assessment where a water table will be lowered on adjacent land.

An assessment of the site is required to ensure the stability and structural integrity of adjacent properties is maintained during the construction phase. This report therefore includes an assessment of the site, plans, geological section, site risk assessment and provides recommendations for footing design and excavation support.

The investigation and reporting were taken as per Tender No. P16-128, Dated 11th March 2016 and subsequent correspondence.

The investigation was completed in two phases which comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) Drilling of two boreholes using hand tools
c) Drilling of three boreholes using a drill rig
d) Dynamic Penetrometer testing (DCP)
e) Collection of soil samples and laboratory analysis for Acid Sulphate characteristics.

The following plans and diagrams were supplied by the client for the work;

- Site survey plan by Usher & Company, Plan Reference: 5551-DET, Date of Survey: 08/04/2015.
- Architectural plans by O2 Architecture, Job No.: 1608, Drawing No.: A1.00 to A1.07, Dated: June 2016

2. SITE FEATURES:

2.1. Description:

The site consists of two properties (No. 1 and 3) which form a rectangular shaped site located on the slightly higher eastern side of Narrabeen Park Parade adjacent to a Council Reserve. It has a combined front east boundary of 19.59m, rear west boundary of 18.295m, side north boundary of 34.14m and side south boundary of 41.15m as referenced from the provided survey plan.

The front of No. 1 consists of a single storey brick commercial building with a driveway and then garage at the rear. The front of No. 3 consists of a driveway which provides access to a three storey brick rendered dwelling which extends to the rear of the block.

2.2. Geology:

The site is situated close to a contact boundary between Quaternary age deposits (Qha) and Newport Formation (Upper Narrabeen Group) bedrock (Rnn) which is of middle Triassic Age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones and has a tendency to weather to significant depth. This bedrock unit is outcropping to the east of the site within North Narrabeen headland. The Quaternary deposits infill the valley and typically consist of silty to peaty quartz sand, silt and clay with ferruginous and humic cementation in places and common shell layers.

3. FIELD WORK:

3.1. Methods:

The field investigation was completed in two phases and comprised a walk over inspection and mapping of the site and adjacent properties on the 16th May 2016 and the 25th November 2016 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing structures and adjacent slopes/conditions. Two hand auger boreholes were drilled on the 16th May 2016 with an additional three auger boreholes (BH101 ï BH103) drilled using a mini drill rig with solid stem spiral flight augers on the 25th November 2016.

Dynamic Penetrometer (DCP) testing was carried out from ground surface adjacent to the boreholes and at one location at the rear of the site in both phases of investigation in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò and in accordance with AS1289.6.3.3 ï 1997, ñDetermination of the penetration resistance of a soil ï Perth Sand Penetrometerò to estimate near surface soil conditions and confirm depths to bedrock.

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 and 3, Appendix: 2.

3.2. Field Observations:

The site is located on the slightly higher eastern side of Narrabeen Park Parade, generally within gently south west sloping topography at the base of a south-striking ridge line that forms the adjacent Narrabeen headland. Narrabeen Park Parade contains a bitumen pavement with a low concrete gutter, which is near level where it passes the site. There were no signs of excessive cracking or deformation within the road pavement to suggest any movement.

The site consists of two properties, No. 1 and No. 3 Narrabeen Park Parade.

<u>No. 1</u>

This property contains a single storey brick building that occupies the entire front western half of the block with a timber extension at the rear. Along the southern side of the building are some paved seating areas with large pine trees at the edge of the Council reserve to the south. At the rear of the site building is a pebblecrete driveway, accessed from the reserve, with a brick garage at the eastern end of the block extending across to the northern, southern and eastern boundaries. On top of the garage is a balcony area. The garage is excavated into the hillslope by up to 2.50m depth, with the slope moderately (-14°) east and south dipping within the grass covered reserve just to the south of the garage. The building structure at the front of the site appears at least 75 years old, however the garage appears approximately 20 to 30 years of age. All structures appear to be in good condition with no significant or obvious signs of cracking, deformation or settlement on external walls. The internal areas of the garage were not inspected.

<u>No. 3</u>

At the front of the property is a wide pebblecrete driveway which gently slopes (-2°) up from the street front, western boundary to a ground floor level open garage below the front of a residential house. Along the sides of the driveway are gardens with small palm trees. The house is situated across the entire eastern half of the site and consists of a three storey rendered dwelling with a pathway and steps along the northern side. The ground level at the rear of the house is approximately 6.0m in elevation above the driveway level. The house appears excavated into the hill slope at each level and as such steps up the slope.

Rendered block walls/retaining walls extend along the northern and eastern boundaries adjacent to the house. The northern wall had signs of remediated cracking and the wall was visibly bowed. The wall is up to approximately 2.50m in height above the pathway and steps along this side of the house and supports a raised level within the northern neighbouring property (No. 5). Similarly along the eastern boundary the land surface is approximately 2.60m above the site level. The dwelling appears to be approximately 40 years old and in good condition with no significant or obvious signs of cracking or settlement on its external walls.

The neighbouring property to the north and east (No. 5 Narrabeen Park Parade) consists of a ôbattle-axedô shape block that extends around the site along both boundaries. At the front of the block, adjacent to the entire northern boundary of the site, are driveways, gardens and two garage/carport structures which are partially excavated into the hillslope. Adjacent to the north-east corner of the site is a residential unit block which is understood to contain 4 dwellings. The estimated age and condition of the dwelling could not be confirmed during the investigation, however it appears <50 years of age. The structure is angled across the block and as such is within 3.0m of the north-east corner of the site and becomes > 11.0m at the southern end of the building. Sloping gardens and lawn extend across the southern end of the block with a narrow pathway extending along the eastern boundary of the site.

3.3. Field Testing:

The first phase of hand drilled boreholes (BH1 ï BH2A) were drilled through existing gardens, BH1 near the southern boundary of No. 1 and BH2 near the northern boundary of No. 3. Dynamic Penetrometer (DCP) tests were undertaken from the surface adjacent to the boreholes.

Boreholes (BH101 ï BH103) were drilled through the existing gardens and driveway with BH101 located on the southern boundary, BH102 located within the driveway at No. 1 and BH103 located within the driveway at No. 3. The drill rig refused between 3.00m and 3.60m depth (BH103 and BH101 respectively) on sandstone bedrock whilst BH102 refused at 1.55m depth on shale bedrock.

Dynamic Cone Penetrometer (DCP) tests were undertaken from the surface adjacent to the boreholes with test refusal encountered at similar levels to the boreholes. DCP103a was discontinued at 3.40m depth, and it is interpreted that this test penetrated into a sub-vertical defect within the sandstone bedrock. Additional testing at the south-east corner of the site was undertaken (DCP 104) due to limitations with access for drilling.

Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified as follows:

- **FILL** ï encountered within BH 1, BH2, BH101 and BH103 to a maximum depth of 0.80m. It consists of very loose to medium dense, fine to medium grained, dry to moist sand with some roots and concrete gravels;
- **SAND** ï encountered below the fill. It consists of loose to dense, medium grained, moist sand with some weakly cemented sand, clay, ironstone and sandstone gravels;
- **SANDY CLAY** ï this layer was encountered within BH102 below the concrete slab (0.15m thick) extending to 1.20m depth. It is classified as firm to hard, low plasticity and moist;

- **SHALEY CLAY** ï encountered below the sandy clay within BH102 from 1.20m to 1.55m depth. It is classified as very stiff to hard, low plasticity, moist with some fine grained sand and shale gravels;
- **SANDSTONE and SHALE BEDROCK** ï based on the results of DCP testing and refusal of the drill rig, it is interpreted that the south-west half of the site is underlain by sandstone bedrock of a minimum of very low strength from 3.00m to 3.60m depth. The rear east to north-east half of the site is interpreted to be underlain by a layer of interbedded shale and siltstone bedrock of a minimum of low strength from 1.55m depth below natural ground surface levels, overlying the sandstone bedrock.

A free standing ground water table or significant water seepage were not identified within any of the boreholes. No signs of ground water were observed after the retrieval of the DCP rods.

3.4. Laboratory Testing

Of the soil samples collected, two representative samples was supplied to a NATA accredited laboratory (Envirolabs) for testing via the sPOCAS method, based on the recommendations of the Acid Sulphate Soils Laboratory Methods Guidelines, Version: 2.1, June 2004. A summary of the test results is listed in Table: 1 below:

Table: 1 – sPOCAS Test Results

Borehole	Depth (m)	pH	pH (oxidized)	TPA moles H^+ / t	Spos % w / w	Liming Rate kg CaCO3 / t
103	1.50	9.2	6.9	<5	<0.005	<0.75
103	2.50	9.2	7.0	<5	<0.005	<0.75

The full set of laboratory results analysis sheets is included in Appendix: 3.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of sandy fill of shallow thickness (å 0.80m) at the front of the site underlain by loose to dense natural sand which overlies sandstone bedrock of at least very low strength for the front south-west half of the site. The bedrock depth was identified to vary between 3.00m and 3.60m across the width of the site and it is expected to drop further towards the south-west corner of the site, where access for investigation is not available, matching the local topography. Towards the rear, the site is underlain by sandy clay and shaley clay, overlying shale bedrock, which appears of at least very low strength from 1.55m depth. The very low strength bedrock is expected to grade quickly to low to medium strength bedrock though actual bedrock strengths are unconfirmed.

The site is extensively modified from its natural condition and as such the geological conditions across the site will vary, especially where previous excavation for the garage and neighbouring house development have occurred.

It is understood that the proposed works will involve demolition of existing site structures and construction of a mixed commercial and residential development with a basement level car park. The excavation for the basement level will be approximately 2.50m depth below the existing ground levels across the front and south-west corner of the site and will be up to 8.50m depth below the adjacent property in the north-east corner of the basement due to the rise in the hill slope.

It is expected that the basement excavation will extend through sandy soils at the front of the site whilst at the rear it will extend through clayey soils and shale bedrock. The sandy soils at the front of the site will not stand unsupported at slope angles >2.0H:1.0V, therefore contiguous excavation support will be required and installed prior to bulk excavation.

At the rear, the site is currently supported by existing retaining walls slightly below the adjacent neighbouring property levels. Based on the depth of excavation, ground conditions identified and proximity of the excavation to the side boundaries and existing structures an excavation support system will also need to be implemented either prior to bulk excavation or in stages as the excavation progresses down. A post excavation retaining wall system will not be suitable.

The existing garage structure, and any other existing boundary/retaining walls, are providing support to old excavations around the north-east half of the site. As such their removal has the potential to result in ground movement upslope to the east or north within neighbouring properties. Therefore, these walls should be investigated as part of the demolition works to confirm their footing style/construction. It may be suitable to incorporate these walls within the new excavation support walls through temporary anchoring into the ground to the east and north of the site, thus reducing potential instability issues.

No groundwater table was identified in the investigation to 3.60m depth, therefore dewatering is not expected for any excavation or post excavation development. However, the site is located at the base of a steep hill slope therefore groundwater seepage from the east and north should be expected and the excavation and development must incorporate control and disposal systems.

The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and/or of lesser or higher strength than interpreted, especially between borehole locations. For confirmation of bedrock strength to below proposed footing or excavation level will need an investigation utilizing cored boreholes in the actual locations. However, access is extremely limited by existing structures and ground conditions can vary over short distances. As such bedrock strength/condition can be confirmed by geotechnical inspection during excavation/construction works, especially where footings will be located. It is recommended that all footings be founded within bedrock of at least low strength, due to the rear edge of the excavation being within this material, to reduce the risk of differential settlement within the structure and ensure long term stability.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and several boreholes. This provides limited data from small isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. The results of the investigation provide a reasonable basis for the analysis and subsequent design of the proposed works.

4.2. Acid Sulfate Assessment:

The site investigation and laboratory test results indicate that Acid or Potential Acid Sulfate Soils are not present within the marine sands encountered towards the front of the site whilst a water table was not intersected above the bedrock surface to 3.60m depth. Due to the clayey nature of the subsurface at the rear of the site, the presence of acid generating soils is highly improbable in this location.

The test result did not trigger the Action Criteria for Equivalent Acidity or for Equivalent Sulphur as referenced from the NSW Acid Sulfate Soil Manual. Therefore no further assessment or an acid sulfate management plan is required for the proposed development.

4.3. Site Specific Risk Assessment:

Based on our site investigation we have identified the following geological/geotechnical landslip hazard which needs to be considered in relation to the existing site and the proposed works. This hazard is:

A. Earth/debris slide (<10m^3) due to improper excavation support

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.

The Risk to Life from Hazard A was estimated at up to **3.33 x 10^{-3}** whilst the Risk to Property was considered to be up to **‘Very High’** where poor design and construction is undertaken without suitable support measures. The hazard was therefore considered to be óUnacceptableô when assessed against the criteria of the AGS 2007 and Pittwater Councils Policy.

However through engineer designed retention systems and permanent retaining walls the potential for instability will reduce to óRareô and as such the proposed works can be achieved whilst maintaining óAcceptableô risk levels (< 10^{-6} / Low).

4.4. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.4.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAô due to sandy soils and when footings founded off bedrock.
Exposure Classification as per AS3700 ï 2011 for masonry structures	Severe Marine Environment
Type of Footing	Strip/pad or slab at base of excavation, may require piers towards western end due to increased bedrock depth
Remarks: As the bedrock depth in the south-west corner of the site is unconfirmed due to access restrictions it is recommended that a borehole be drilled in this location following demolition and prior to bulk excavation to allow finalization of footing design and excavation support and confirm suitable equipment.	
Sub-grade material and Maximum Allowable Bearing Capacity	- Very Low Strength bedrock: 800kPa - Low Strength bedrock: 1000kPa - Medium Strength bedrock: 2000kPa *higher footing pressures may be achieved but will require core drilling below footing locations
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: • All footings should be founded off consistent LS bedrock to prevent differential settlement. • 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 ócertifiedô at the end of the project.	

4.4.2. Excavation:	
Depth of Excavation	Between 2.50m and 6.00m based on existing site levels
Distance to Neighbouring Properties/Structures	Road /Council Reserve = 0.0m No. 5 - boundary 0.0m, garage/carport = 1.0m, residential building Ó3.0m

Type of Material to be Excavated	Very loose to medium dense sandy fill up to 0.80m depth
	Loose to dense sand up to 3.0m depth
	Firm to hard clay/shaley clay up to 1.55m depth at rear
	VLS ï LS sandstone/shale bedrock (undetermined)
	LS-MS shale bedrock (Ó1.55m depth)

Guidelines for batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill and natural soils	1:1	2:1
Low to medium strength bedrock, fractured	1 : 1	0.5:1.0*
Medium strength (MS), defect free bedrock	Vertical*	Vertical *

*Dependent on assessment by engineering geologist.

Remarks:

- 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.
- Based on the proposed design safe batter slopes will not be achievable around the basement perimeter and excavation support will need to be implemented prior to excavation where sandy soils are encountered and during excavation across the rear half where clay soils and existing retaining walls are located.

Equipment for Excavation	Fill, sand & clayey soils	Excavator with bucket
	ELS bedrock	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS ï 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:

- It is recommended that the hard rock excavation perimeter be saw cut prior to rock hammering, this will generally reduce the amount of rock support required, reduce deflection of rock across boundary and under neighbouring structures and will provide a slight buffer distance to ground vibrations for the use of rock hammers.
- It is recommended that a small (<500kg) rock hammer be used for rock excavation, where LS-MS bedrock is encountered. This will significantly reduce the probability of ground vibration damage to

the neighbouring properties. However this scale equipment will result in a relatively slow excavation progress. Whilst larger rock hammers will increase the speed of works the risks from vibration damage or dislodgement of rocks is significantly increased.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	No. 5 (Garage and Dwelling) = 5mm/s
	Road reserve service lines = 3mm/s
Vibration Calibration Tests Required	Yes if >500kg rock hammer proposed for use
Full time vibration Monitoring Required	Depending on proposed equipment and calibration test results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • After removal of fill/soil and existing structures • During installation of excavation support • At completion of the excavation.
Dilapidation Surveys Requirement	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:	

4.4.3. Retaining Structures:	
Required	New retaining structures will be required as part of the proposed development
Types	Concrete soldier piles across front of site where sandy soils exist. Either concrete soldier piles or reinforced shotcrete with anchors across rear half of site where clay soils and existing retaining structures support the slope. All walls must be 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)	
Sand (loose-medium dense)	18	$\phi' = 30°$	0.34	0.55	3.33

Clay soils (firm to hard) sloping surface above	20	$\phi' = 30°$	0.37	0.60	N/A
ELS bedrock	22	$\phi' = 35°$	0.27	0.43	3.69
LS bedrock (fractured)	23	$\phi' = 38°$	0.15	0.25	200kPa
MS bedrock	24	$\phi' = 42°$	0.00	0.05	1000kPa

Remarks:

- Based on the apparent bedrock strength and depth across the front of the site, a cantilever support design for retaining walls may be difficult to achieve. As such a specialist piling contractor should be consulted. A piled support wall through sand will need to be contiguous to prevent loss of sand from behind the wall.
- The use of driven style support is not recommended due to the potential for settlement in the road reserve and due to the limited toe support available below the excavation base due to the bedrock depth.
- Across the rear of the site, access and existing retaining structures are expected to prevent suitable installation of a piled support wall prior to excavation. As such it may be more practical to stabilize existing retaining walls in place and undertake a staged excavation and wall construction system (i.e. reinforced shotcrete with anchors).
- At all times, continuous support should be provided to all portions of the excavation perimeter.
- In suggesting the above retention parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the walls to prevent groundwater buildup. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the 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 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).

4.4.4. Temporary Anchors		
Sub-horizontal anchors can be utilized to provide lateral restraint to the retaining structures. As any anchor for the excavations will extend across property boundaries it is recommended that they be temporary with permanent support applied to the boundary retaining structures by the completed building structure. Anchors must extend greater than 2.50m below any neighbouring footing.		
Recommended Allowable Bond Stresses: (Grout/rock)	Very Stiff and Hard Clay	50kPa
	Extremely low strength bedrock	100kPa
	Low strength bedrock	300kPa

Remarks:

- The above parameters can be applied where the anchor holes are clean and thoroughly flushed, with grouting and installation procedures carried out sensibly and in accordance with correct anchoring practice. It is the contractorôs responsibility to ensure that the correct design values, according to site specific ground conditions, the anchor system and method of installation, are used and that the anchor holes are carefully cleaned out before grouting.
- It is recommended that a geotechnical engineer approve the proposed methods and supervise the anchor installation process. After anchors are installed, it is recommended that they be check stressed to above the working load. Checks will be required to ensure the anchors maintain their loads and creep movements do not occur until permanent structures are in place.
- Permission is required from the owners of neighbouring properties when anchoring is required across the site boundaries or into footpath/roadway reserves etc.

4.4.5. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (C̀2.0 L/min), on defects and at soil/rock interface
Site Location and Topography		On slightly higher eastern side of the road at the base of a hill slope and extending into the valley floor
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Potentially available in south-west corner of site

	through sandy soils below basement

Remarks:

- Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow. This can result in relaxation of excavation faces causing instability. Therefore excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional.
- 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 Council's stormwater system off site.

4.5. Conditions Relating to Design and Construction Monitoring:

To allow certification at the completion of the project it will be necessary for Crozier Geotechnical Consultants to:

1. Review and approve the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
3. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilisation and drainage measures are in place.

Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

4.6. Design Life of Structure:

We have interpreted the design life requirements specified within Councils Risk Management Policy to refer to structural elements designed to support the house etc, the adjacent slope, control stormwater and maintain the risk of instability within acceptable limits. Specific structures and features that may affect the maintenance and stability of the site in relation to the proposed and existing development are considered to comprise:

- storm water and subsoil drainage systems,
- retaining walls and soil slope erosion and instability,
- maintenance of trees/vegetation on this and adjacent properties.

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 ï 2011 (100 years)). It will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally the property owner should adopt and implement a maintenance and inspection program.

If this maintenance and inspection schedule are not maintained the design life of the property cannot be attained. A recommended program is given in Table: C in Appendix: 3 and should also include the following guidelines.

- The conditions on the block donqt change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics ñLandslide Risk Managementò Volume 42, March 2007.
  c) AS 2870 ï 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owner has any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component.

It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they donqt leak and increase either the local groundwater level or landslide potential.

Recommendations for construction within hill slopes are also provided in Appendix: 5.

5. CONCLUSION:

The site investigation identified the presence of sandy fill of shallow thickness (Ò0.80m) at the front of the site underlain by loose to dense sand which overlies sandstone bedrock and appears up to 3.60m depth below existing ground surface, however this may increase slightly in the south-west corner matching the local topography. Towards the rear, the site is underlain by sandy clay and shaley clay, overlying shale bedrock with the bedrock surface expected to rise towards the east and north-east.

It is understood that the proposed works will involve demolition of existing site structures and construction of a mixed unit development with a basement level car park. The basement will require an excavation of up to 2.50m depth across the south-west half of the site however the excavation will be up to 8.50m depth with respect to the site boundaries around the north-east half. The excavation will extend to property boundaries and as such will require the installation of support measures prior to bulk excavation. Where this is not possible across the eastern half of the site, then excavation and support installation must occur in a systematic manner that ensure the stability of all boundaries at all times is maintained to a sensible level.

The test results indicate that Acid or Potential Acid Sulfate Soils are not present within the marine sands encountered over the project site to the investigated depth whilst they will not be located across the north-east half where residual clay soils exist. Therefore a management plan for treatment of acid sulfate soils will not be required.

The existing site contains only one existing potential landslip hazard however it provides óTolerableô risk levels. The risks associated with the proposed development can be at óUnacceptableô levels where the works are undertaken in an uncontrolled manner without installation of suitable support systems. However the proposed works will remove the existing landslip hazard and can maintain 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:

Troy Crozier
Principal Engineering Geologist

6. REFERENCES:

1. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings ï Construction
2. Pells et. al. Design loadings for foundations on shale and sandstone in the Sydney region. Australian Geomechanics Society Journal, 1978.
3. 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.
4. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
5. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin.

Appendix 1

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.

Details of the type and method of sampling are given in the report.

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
  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

  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: -

Qc (MPa) = (0.4 to 0.6) N blows (blows per 300mm)

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

Qc = (12 to 18) Cu

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.

Hand Penetrometers

Hand 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 6.3.2). 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 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.

Bore 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.

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.

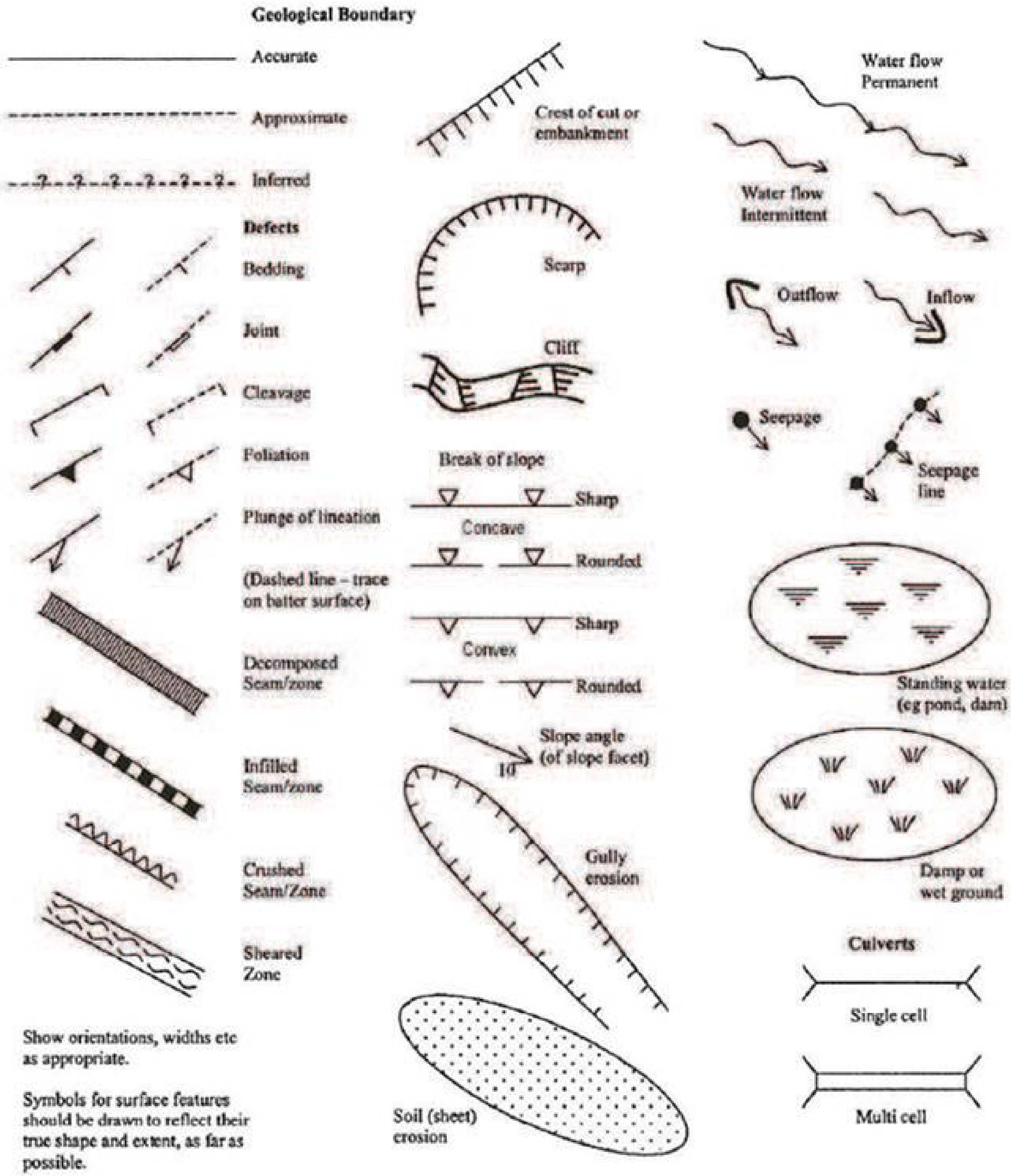
Reproduction of Information for Contractual Purposes

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.

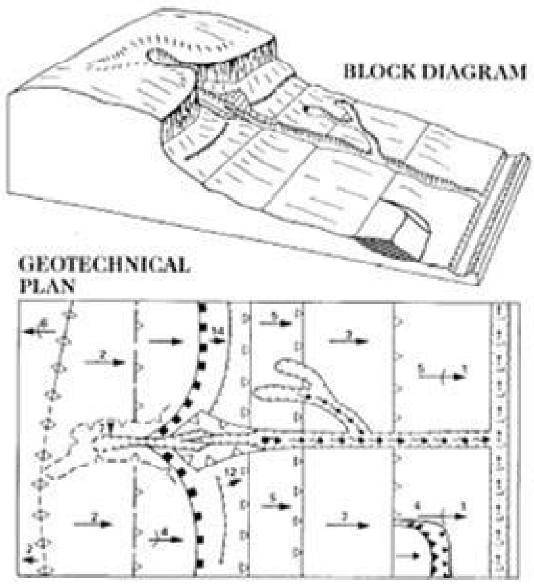
Site Inspection

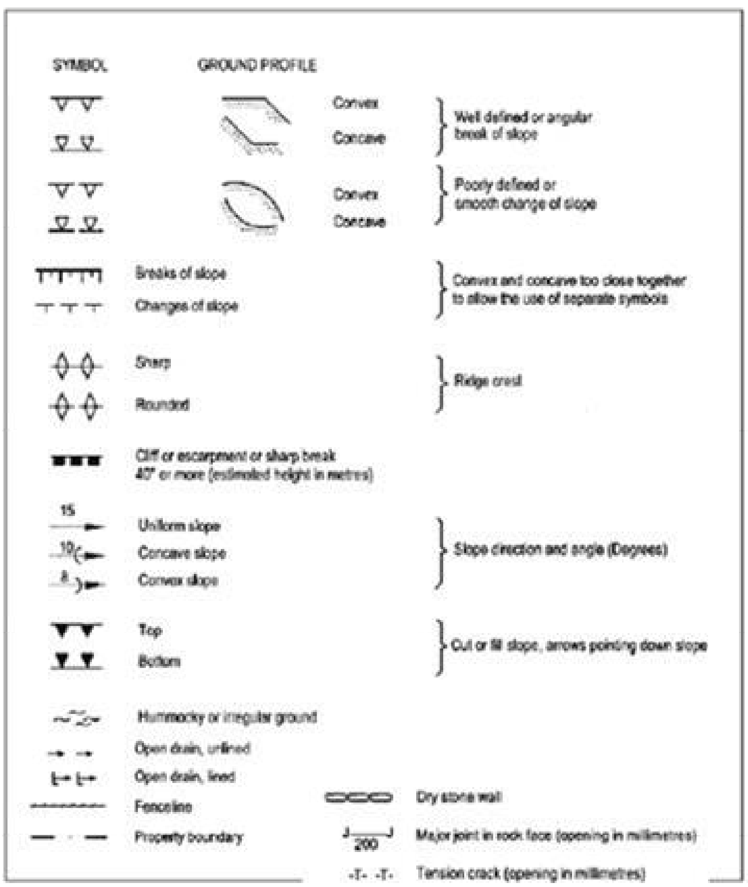
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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).

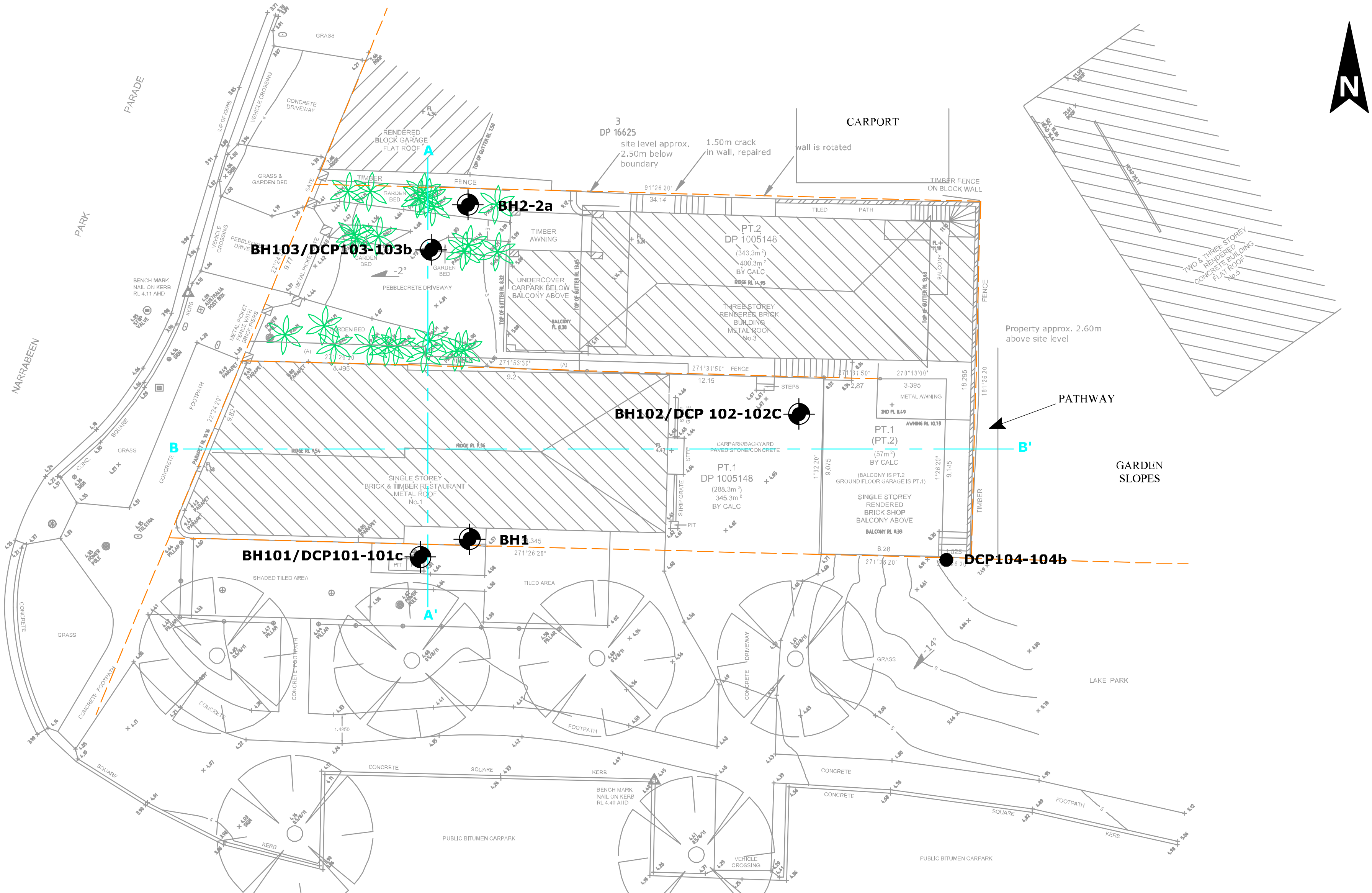




Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 2



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

0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

SITE PLAN & TEST LOCATIONS FIGURE 1.

LEGEND

BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION

DCP — DYNAMIC CONE PENETROMETER

PROPERTY BOUNDARY

A — A' CROSS-SECTION REFERENCE LINE

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100

ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883

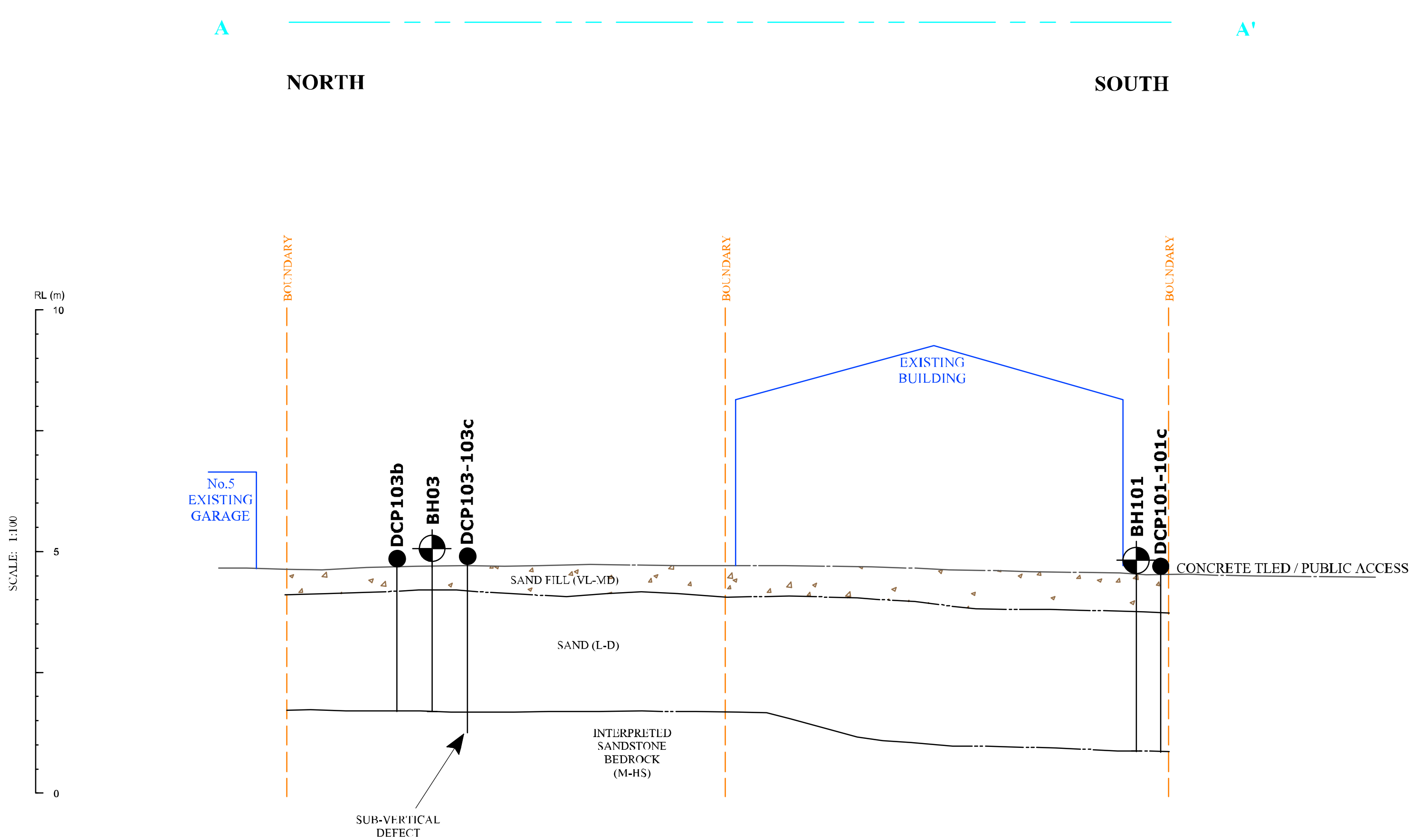
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE: 1:200 @ A3
DRAWING: FIGURE 1
DATE: 16/12/2016

APPROVED BY: TMC
DRAWN BY: KB
PROJECT: 2016-092.1

PREPARED FOR:
UNITY PTY LTD

ADDRESS:
1-3 NARRABEEN PARK PARADE
NORTH NARRABEEN



0 1 2 3 4 5
SCALE: 1:100

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

BH AUGER LOCATIONS

DCP DYNAMIC CONE PENETROMETER

A — A' CROSS-SECTION REFERENCE LINE

PROPERTY BOUNDARY

GEOLOGICAL BOUNDARY

SAND FILL

SAND

SANDSTONE BEDROCK

CROZIER
GEOTECHNICAL CONSULTANTS

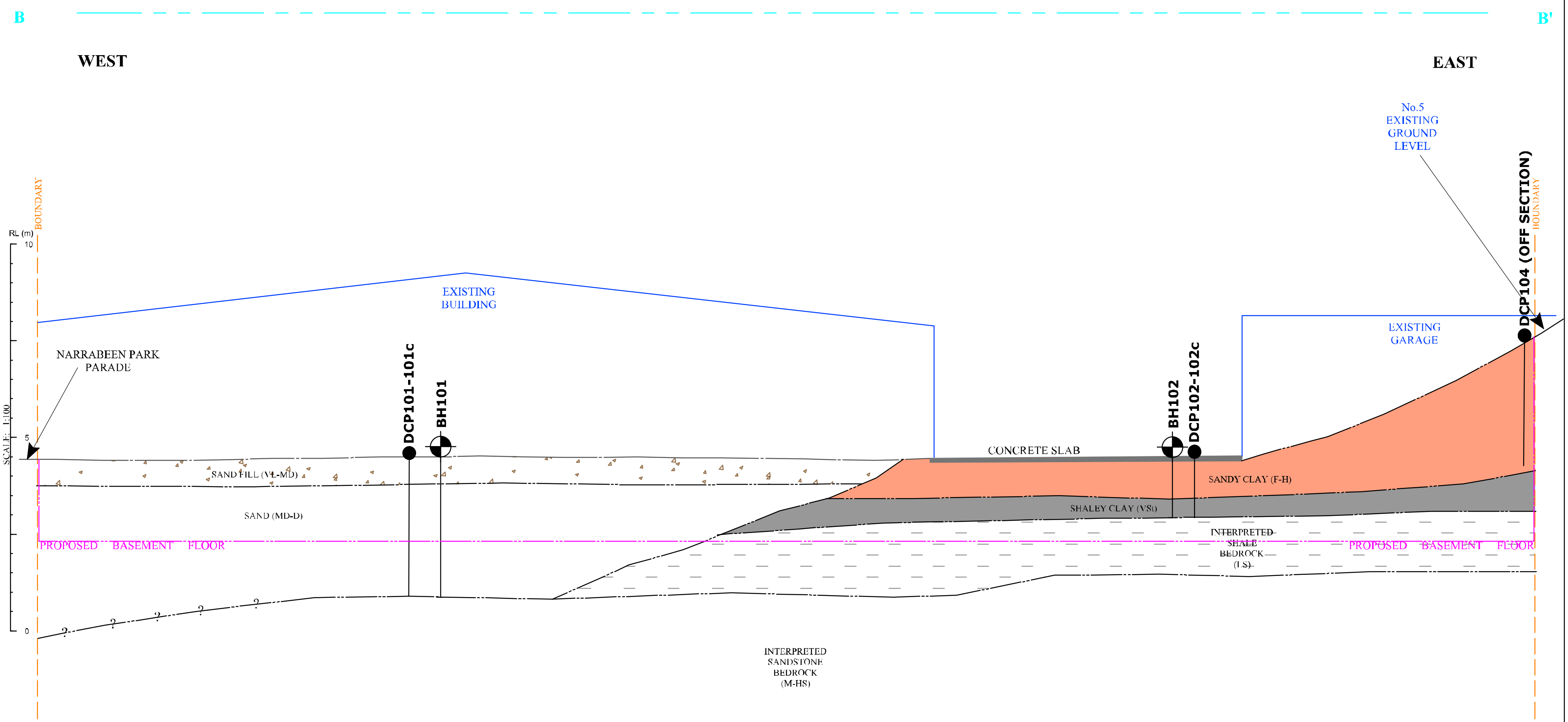
Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE: 1:100 @ A3
DRAWING: FIGURE 2
DATE: 6/12/2016

APPROVED BY: TMC
DRAWN BY: KB
PROJECT: 2016-092.1

PREPARED FOR:
UNITY PTY LTD

ADDRESS:
1-3 NARRABEEN PARK PARADE
NORTH NARRABEEN



SCALE: 1:100

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
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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 B-B', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

INTERPRETED GEOLOGICAL MODEL FIGURE 3.

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

BH AUGER LOCATIONS	PROPERTY BOUNDARY	SANDY CLAY	SAND FILL
DCP DYNAMIC CONE PENETROMETER	A — A' CROSS-SECTION REFERENCE LINE	SHALEY CLAY	SAND
	GEOLOGICAL BOUNDARY	SHALE BEDROCK	SANDSTONE BEDROCK

SCALE:	1:100 @ A3
DRAWING:	FIGURE 3
DATE:	16/12/2016
APPROVED BY:	TMC
DRAWN BY:	KB
PROJECT:	2016-092.1

PREPARED FOR:
UNITY PTY LTD

ADDRESS:
1-3 NARRABEEN PARK PARADE
NORTH NARRABEEN

TEST BORE REPORT

CLIENT: Unity Pty Ltd **DATE:** 25/11/2016 **BORE No.:** 101

PROJECT: New mixed use development **PROJECT No.:** 2016-092.1 **SHEET:** 1 of 1

LOCATION: 1-3 Narrabeen Park Pde, Narrabeen **SURFACE LEVEL:** RL å 4.58m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	In Situ Testing Results
0.00	FILL - Very loose, light brown and brown, medium grained, moist sand fill with some roots and concrete gravels * 0.15m loose * 0.45m medium dense * 0.60m some grey clay				
0.70	SAND - Medium dense, orange and light brown, medium grained, moist sand				
1.00					
		D	1.20		
	* 1.80m dense				
2.00					
		D	2.50		
	* 2.70m medium dense				
3.00					
	* 3.10m some sandstone gravels	D	3.10		
	* 3.30m orange and grey with some ironstone gravels and clay				
		D	3.55		
3.60	* 3.60m red				
	DINGO REFUSAL at 3.60m on sandstone bedrock of at least very low strength				
4.00					

RIG: Dingo Mini Drill Rig DRILLER: KB LOGGED: BL

METHOD: Continuous Flight Soild Stem Auger with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS: CHECKED:

TEST BORE REPORT

CLIENT: Unity Pty Ltd **DATE:** 25/11/2016 **BORE No.:** 102

PROJECT: New mixed use development **PROJECT No.:** 2016-092.1 **SHEET:** 1 of 1

LOCATION: 1-3 Narrabeen Park Pde, Narrabeen **SURFACE LEVEL:** RL å 4.67m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	Results	
0.00	CONCRETE SLAB					
0.15	SANDY CLAY - Firm, grey and light orange, low plasticity, moist sandy clay					
	* 0.30m stiff	D	0.30			
	* 0.45m very stiff					
		D	0.75			
1.00		D	1.00			
	* 1.05m hard					
1.20	SHALEY CLAY - Very stiff, light brown and grey, low plasticity, moist shaley clay with some fine grained sand and shale gravels					
	* 1.35m hard					
		D	1.50			
1.55	DINGO REFUSAL at 1.55m on low strength shale bedrock					
2.00						

RIG: Dingo Mini Drill Rig DRILLER: KB LOGGED: BL

METHOD: Continuous Flight Soild Stem Auger with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS: CHECKED:

TEST BORE REPORT

CLIENT: Unity Pty Ltd **DATE:** 25/11/2016 **BORE No.:** 103

PROJECT: New mixed use development **PROJECT No.:** 2016-092.1 **SHEET:** 1 of 1

LOCATION: 1-3 Narrabeen Park Pde, Narrabeen **SURFACE LEVEL:** RL å 4.73m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	Results	
0.00	FILL - Very loose, brown, fine grained, dry sand fill with some roots and concrete gravels					
0.50	SAND - Medium dense, light brown and orange, medium grained, moist sand	D	0.80			
1.00						
	* 1.20m black with a trace of clay					
	* 1.50m loose	D	1.50	sPOCAS		
2.00	* 1.95m medium dense					
	* 2.30m some cemented sand gravels					
		D	2.50	sPOCAS		
	* 2.95m orange-red with some sandstone gravels					
3.00		D	3.00			
	DINGO REFUSAL at 3.00m on sandstone bedrock of at least very low strength					
4.00						

RIG: Dingo Mini Drill Rig DRILLER: KB LOGGED: BL

METHOD: Continuous Flight Soild Stem Auger with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS: CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Unity Pty Ltd | **DATE:** 25/11/2016

PROJECT: New mixed use development | **PROJECT No.:** 2016-092.1

LOCATION: 1-3 Narrabeen Park Pde, Narrabeen | **SHEET:** 1 of 2

	Test Location							
Depth (m)	DCP101 (s)	DCP101a (s)	DCP101b (s)	DCP101c (s)	DCP102 (s)	DCP102a (c)	DCP102b (c)	DCP102c (c)
0.00 - 0.15	1	1	-	-	-	-	-	-
0.15 - 0.30	2	2	-	-	3	3	-	-
0.30 - 0.45	3	2	-	-	4	6	-	-
0.45 - 0.60	4 (B)	4	-	-	10	9	-	-
0.60 - 0.75	Refusal on a boulder	4	-	-	13	29	-	-
0.75 - 0.90		8	-	-			3	-
0.90 - 1.05		6	-	-			10	-
1.05 - 1.20		5	2	-			16	16
1.20 - 1.35			4	-				11
1.35 - 1.50			5	-				17
1.50 - 1.65			8	-				10 (B)
1.65 - 1.80			9	-				Refusal at 1.50m on rock
1.80 - 1.95			11	-				
1.95 - 2.10			7	-				
2.10 - 2.25			8	-				
2.25 - 2.40			11	-				
2.40 - 2.55			11	-				
2.55 - 2.70			11	3				
2.70 - 2.85			7	4				
2.85 - 3.00			9	5				
3.00 - 3.15			11	5				
3.15 - 3.30			11	6				
3.30 - 3.45			14	7				
3.45 - 3.60				17 (B)				
3.60 - 3.75				Refusal at 3.60m on rock				

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER - (c)
AS 1289. F3.3, PERTH SAND PENETROMETER - (s)

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Unity Pty Ltd **DATE:** 25/11/2016

PROJECT: New mixed use development **PROJECT No.:** 2016-092.1

LOCATION: 1-3 Narrabeen Park Pde, Narrabeen **SHEET:** 2 of 2

	Test Location							
Depth (m)	DCP103 (s)	DCP103a (s)	DCP103b (s)	DCP104 (c)	DCP104a (c)	DCP104b (c)		
0.00 - 0.15	1	-	-	9	6 (B)	5		
0.15 - 0.30	4	-	-	9	Refusal at 0.15m on a boulder	7		
0.30 - 0.45	6	-	-	Disct at 0.20m on a boulder		6		
0.45 - 0.60	7	-	-			7		
0.60 - 0.75	5	-	-			23		
0.75 - 0.90	3	-	-			13		
0.90 - 1.05	3	-	-			10		
1.05 - 1.20	4	2	-			10		
1.20 - 1.35		3	-			10		
1.35 - 1.50		4	-			7		
1.50 - 1.65		2	-			6		
1.65 - 1.80		2	-			5		
1.80 - 1.95		2	-			5		
1.95 - 2.10		3	-			5		
2.10 - 2.25		4	-			6		
2.25 - 2.40		5	-			14		
2.40 - 2.55		6	-			Disct at 2.40m		
2.55 - 2.70		4	-					
2.70 - 2.85		6	-					
2.85 - 3.00		8	10 (B)					
3.00 - 3.15		10	Refusal at 3.00m on rock					
3.15 - 3.30		14						
3.30 - 3.45		25						
3.45 - 3.60		Disct at 3.40m						
3.60 - 3.75								

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER - (c)
AS 1289. F3.3, PERTH SAND PENETROMETER - (s)

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood	Spatial Impact	Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide $<10m^3$) due to improper excavation support design and construction	Excavation works and neighbouring property	Sandy soils at front, clay hill slope with pre-existing excavations at rear of site	a) work within excavation, slide may impact 1/5 of site b) impact small portion of gardens/pathways adjacent to site c) impact half of garage/carport structures due to proximity and excavation depth d) impact north-west corner of building e) impact up to half of pathway adjacent to site	a) Person on site 8 hrs/day, b) Person in gardens/pathways 4hrs/day c) Person in garage/carport 2hrs/day d) person in bulding 20hrs/day e) Person on footpath past site 4hrs/day	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate d) Likely to not evacuate e) Possible to not evacuate	a) Person in open space, buried b) Person in open space, likely buried c) Person in open space/car, likely buried d) Person in building, builing impact only e) Person in open space, possible buried	
		a) excavation	0.1	0.20	0.33	0.50	1.00	**3.33E-03**
		b) No. 5 gardens and pathways	0.1	0.05	0.17	0.50	0.90	**3.75E-04**
		c) No. 5 garage/carport	0.1	0.50	0.08	0.50	0.90	**1.88E-03**
		d) No. 5 building	0.1	0.10	0.83	0.75	0.05	**3.13E-04**
		e) Public/Road Reserve	0.1	0.50	0.17	0.50	0.80	**3.33E-03**

* hazard considered in current condition and/or without suitable remedial/stabilisation measures
* likelihood of occurrence for design life ofproposed development (considered 100years)
* considered for person most at risk, where within buidling considered for person in bed and or without notification of slide movement
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01)
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007 and assessment of slide scale and location of impact (above slide considered less vulnerable than below slide)

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <10m3) due to improper excavation support design and construction	a) excavation	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, MINOR damage to neighbouring property.	Very High
		b) No. 5 gardens and pathways	Almost Certain	Event is expected to occur over design life.	Minor	Limited Damage to part of structure or site requires some stabilisation, INSIGNIFICANT damage to neighbouring properties.	High
		c) No. 5 garage/carport	Likely	Event will probably occur under adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works, MINOR damage to neighbouring property.	High
		d) No. 5 building	Likely	Event will probably occur under adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works, MINOR damage to neighbouring property.	High
		e) Public/Road Reserve	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, MINOR damage to neighbouring property.	Very High

* 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%.

Appendix 4

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.

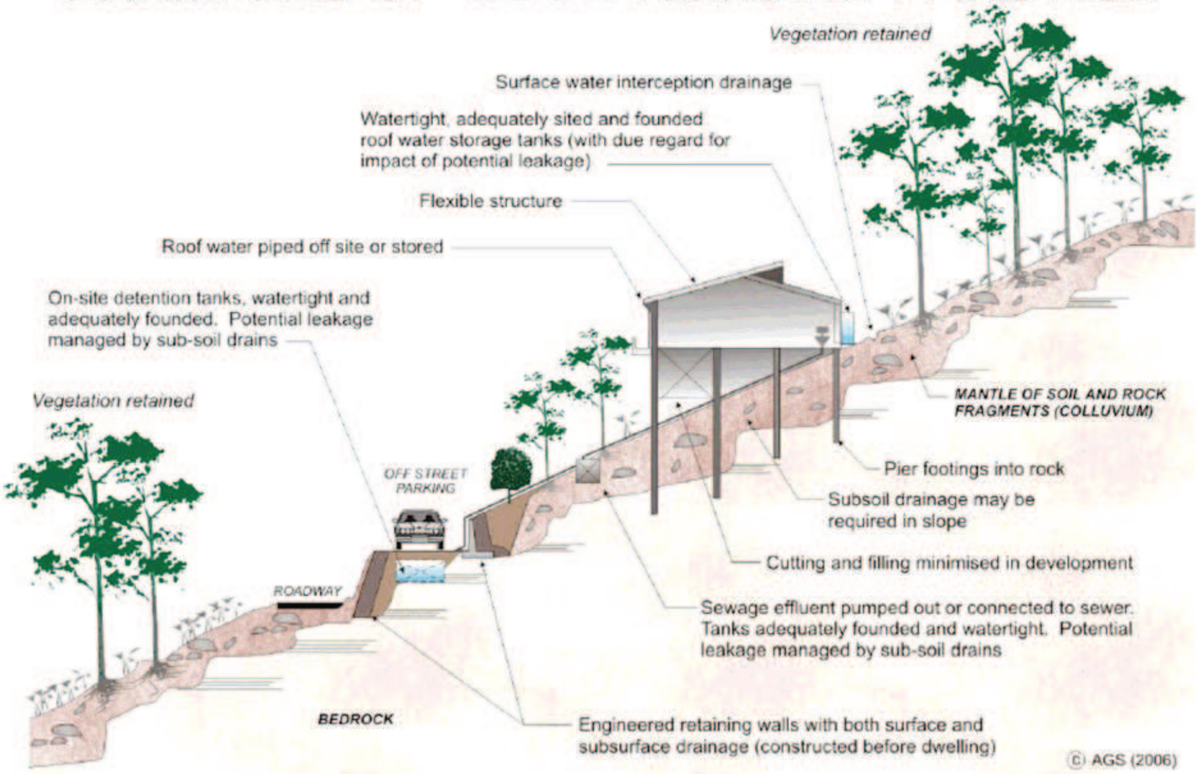
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE