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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

6 -7 KARA CRESCENT, BAYVIEW, NSW

Prepared For

Matthew and Louise Baxter

Project No.: 2023-081

May, 2023

Document Revision Record

Issue No	Date	Details of Revisions
0	15th May 2023	Original issue

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

Development Application for ______________________________	
Name of Applicant	
Address of site	6-7 Kara Crescent, Bayview, NSW

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 15th May 2023_ 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:

☐ 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: Geotechnical Report for Proposed Alterations and Additions

Report Date: 15/05/2023 **Project No.:** 2023-081

Author: J. Dee and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural drawings by Campbell Architecture, Job No.: 2111, Drawing No.: DA00 – DA11, Issue: 1, Dated: 12/05/2023
Survey Plan by CMS Surveyors Pty. Ltd., Drawing Name: 20359Cdetail, Sheet: 1-6, Issue: 1, Dated: 09/03/2023

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 __6-7 Kara Crescent, Bayview, NSW ____________

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: Geotechnical Report for Proposed Alterations and Additions
Report Date: 15/05/2023 **Project No.:** 2023-081
Author: J. Dee and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___12th April 2023________
- ■ 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 12th April 2023…………
- ■ 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 …………50 years…………………………………..
    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 ………………………………………………………

Name …**Troy Crozier**…………………………………………

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

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

Company… **Crozier Geotechnical Consultants**

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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS 6-7 KARA CRESCENT, BAYVIEW, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 6-7 Kara Crescent, Bayview, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Campbell Architecture on behalf of the clients Matthew and Louise Baxter.

It is understood that the proposed works involve alterations and additions to the existing site structures including the extension of the existing dwelling as well as the construction of a new pool. Excavation below existing ground levels will be required in multiple locations and will extend to a maximum anticipated depth of 3.00m.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009 Sheet GTH_017). For Development Application purposes, to meet the Councils Policy requirements for land classified as H1 a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted. This report must include a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provide recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years.

The site is also classified under Northern Beaches Council's Local Environmental Plan (LEP) 2012 as being within 'Class 5' ASS hazard zones and is located within 500m of 'Class 3' land. As such, a preliminary assessment is required as part of the Development Application to determine if the proposed works, where within 500m of adjacent Class 1 to 4 land, are likely to lower the water table below 1 meter AHD on the adjacent Class 1 to 4.

This report is provided for DA submission and includes a description of site and sub-surface conditions including groundwater, soil logs and in-site test results, a geotechnical assessment of the proposed works, assessment of landslide hazards, site plan and recommendations for the design of works.

The investigation and reporting were undertaken as per Proposal No.: P23-133, Dated: 30th March 2023.

The investigation comprised:

a) Dial Before You Dig (DBYD) plan review for service mains;
b) Detailed geotechnical inspection and mapping of the site and adjacent properties with a photographic record and identification of geotechnical conditions and hazards related to the existing site and proposed works;
c) Drilling of seven boreholes using hand auger techniques along with seven Dynamic Cone Penetrometer (DCP) tests across the site

The following plans and drawings were supplied for the proposal, investigation and reporting:

- Architectural Drawings – Campbell Architecture, Job No.: 2111, Drawing No.: DA00 – DA11, Issue: 1, Dated: 12/05/2023
- Survey Drawing – CMS Surveyors Pty. Ltd., Drawing Name: 20359Cdetail, Sheet: 1-6, Issue: 1, Dated: 09/03/2023

1.1 Proposed Development

It is understood that the proposed works involve alterations and additions to the existing site structures. These will include extension of Level 0, Level 1 and Level 2 of the existing dwelling as well as the construction of a new in ground pool. Excavation below existing ground levels will be required for Level 0, Level 1 and the in ground pool and will extend to approximate depths of 3.00m, 2.40m and 3.00m respectively. The proposed excavations will be setback from neighbouring boundaries by a minimum of 10.00m.

2. SITE FEATURES:

2.1. Description:

The site is an irregular shaped block situated at upper slope level on the high eastern side of the road with the majority of the block situated within gentle east dipping topography and the rear eastern portion of the block characterized by steep to extreme east dipping topography. Site surface levels reduce from approximately RL90.00m near the roadway to a low of RL65.00m in the north east corner.

The site comprises the two separate properties of No. 6 and No. 7 Kara Crescent. The former being a vacant block understood to have previously contained a residential dwelling and the latter containing a two and three storey residential dwelling with adjacent pool.

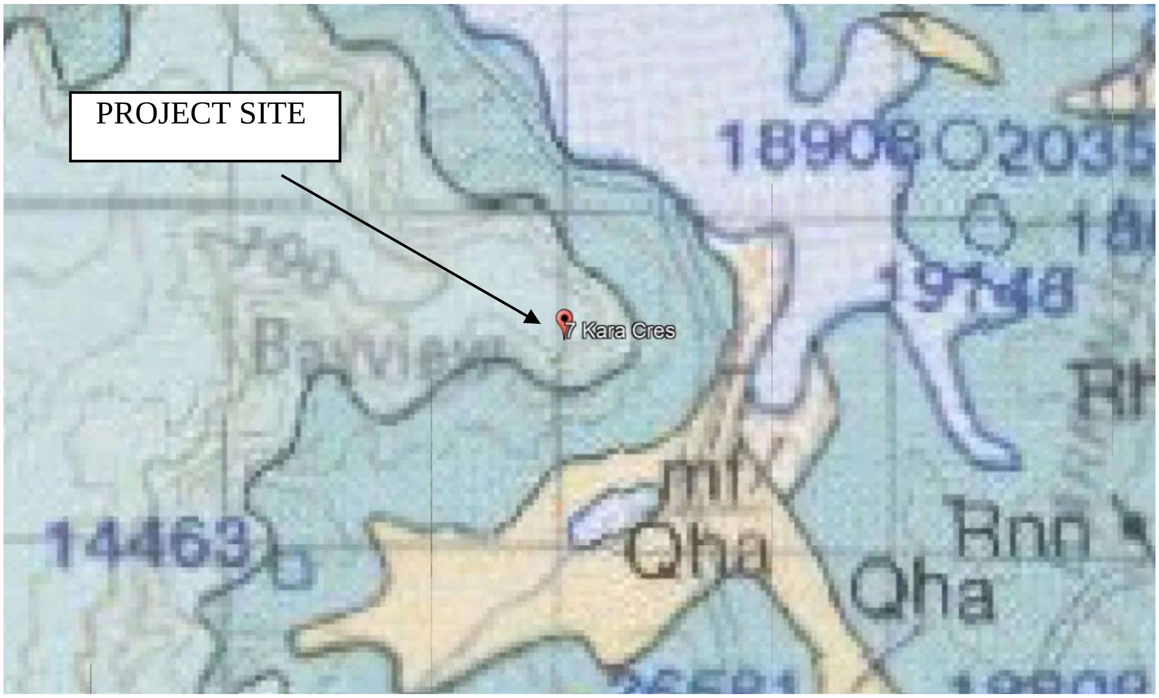
An aerial photograph of the site and its surrounds with boundary designations is provided below (Photograph 1), as sourced from NSW Government Six Map spatial data system.

Photograph 1: Aerial photo of site and surrounds

2.2. Geology:

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

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defect orientations being south-east and north-east. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders. An extract of the relevant Geology Series Sheet is provided as Extract 1 with the site indicated.



Extract 1: Sydney (9130 Geology Series Map): 1: 100000 – Geology underlying the site

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on 12th April 2023 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 soil slopes, existing structures and neighbouring properties. It also included the drilling of seven boreholes (BH1-BH7) using hand tools due to access limitations to investigate subsurface geology.

Geotechnical logging of the subsurface conditions was undertaken by a Geotechnical Engineer by inspection of disturbed soil recovered from the augers. Logging was undertaken in accordance with AS1726:2017 'Geotechnical Site Investigations'.

DCP testing was carried out from ground surface adjacent to the boreholes in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer" to estimate near surface ground conditions.

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

3.2. Field Observations:

The site is situated on the low eastern side of Kara Crescent within primarily gentle east dipping topography. Kara Crescent comprises a gently south dipping bituminous sealed pavement with no engineered drainage system adjacent to the road reserve. The road reserve did not exhibit any signs of significant cracking or settlement to indicate any impending geotechnical concern.

Two separate driveways extend from the road reserve and provide access to the respective sites of No. 6 and No. 7 Kara Crescent. The driveway for No. 7 extends sub-parallel to the southern site boundary and provides access to the existing site dwelling as well as a tennis court, situated within the southern corner of the site.

The existing site dwelling within No. 7 Kara Crescent comprises a two and three storey masonry and timber structure of anticipated construction age <40 years. The structure appears to be in good condition with no signs of excessive settlement or cracking to indicate any underlying geotechnical concern.

To the south and east of the existing dwelling and adjacent pool area, the topography transitions to a steep to extreme south east dipping slope and contains a large sandstone outcrop featuring both bedrock and detached boulders. Near the base of this outcrop, a significant overhang was observed where the rock was undercut up to 2.00m. The rock mass did not appear to feature any significant subvertical joint sets within the overhang and the overhang was considered relatively stable. Photographs 2 – Photograph 4 below provide views of this exposed rock.

Photograph 2: View of the sandstone outcrop, looking broadly south-east from the dwelling

Photograph 3: View of the sandstone outcrop, looking broadly south

Photograph 4: View of the undercut portion of the sandstone outcrop adjacent to the eastern site boundary

No. 6 Kara Crescent is currently a vacant site, understood to have previously contained a residential dwelling. Sandstone was observed outcropping both as bedrock and detached boulders in multiple locations within the property. Where exposed, all outcropping bedrock was preliminarily assessed as at least low strength and all boulders were considered stable. Photograph 5 below provides a view of some of the exposed bedrock within No. 6 Kara Crescent.

Photograph 5: View of the sandstone bedrock outcrop within No. 6, looking broadly south from the northern corner of site

The neighbouring property to the east (No. 27c Alexandra Crescent) comprises a three storey masonry dwelling setback from the shared boundary by approximately 15.00m. Boundary setbacks and elevation differentials precluded an extensive assessment of the structure however it is understood to be of relatively recent construction and in good condition. The property is generally situated at levels up to 10m lower than the site with the shared boundary comprising a very steep to extreme slope with outcropping sandstone bedrock.

The neighbouring property to the north (No. 5 Kara Crescent) comprises a two and three storey masonry dwelling of anticipated construction age >40 years. The ground levels along the shared boundary are relatively similar to the site, the structure is setback from the shared boundary by a minimum of 2.50m and appeared to be in relatively good condition with no signs of significant cracking or settlement on the visible aspects to indicate any underlying geotechnical concern.

The neighbouring property to the west (No. 8b Kara Crescent) comprises An vacant block, previously understood to have contained a two storey masonry residence. Boundary structures and vegetation precluded an extensive assessment of the property however the ground levels appeared to be relatively similar to the site along the shared boundary.

The neighbouring properties and structures were inspected from the site or road reserves, however visible aspects showed no indications of geotechnical hazard that may impact the site.

3.3. Ground Conditions:

The boreholes (BH1 – BH7) were drilled at locations across site predesignated by the architect, broadly within the envelope of proposed works. BH1, BH5 and BH6 all extended through a relatively shallow (<0.40m) depth of topsoil/fill prior to intersection of residual sandy/clayey soils, with all subsequently encountering refusal atop interpreted sandstone bedrock. BH2-BH4 and BH7 all encountered effective refusal atop sandstone boulders/cobbles or drainage aggregate at depths ranging between 0.50m and 0.70m.

DCP tests were carried out from the ground surface adjacent to the boreholes with refusal encountered atop interpreted LS Sandstone bedrock at depths varying from 0.90m (DCP5) and 1.40m (DCP7).

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

- **TOPSOIL/FILL –** Topsoil/Fill was encountered from ground surface in all boreholes and is anticipated to extend to a maximum depth of 0.70m (DCP7) and a minimum depth of 0.30m (BH1). The fill predominantly comprises a very loose to loose, brown silty sand with roots and sandstone gravels and also featured sandstone cobbles, boulders and drainage aggregate.
- **RESIDUAL SOILS –** Natural soils interpreted as residual were encountered underlying the topsoil/fill across site. This material typically initially comprised a yellow/brown clayey sand, grading with depth to a sandy clay and subsequently a yellow with red and red mottle sandy clay interpreted as weathered bedrock.
- **SANDSTONE BEDROCK –** Sandstone bedrock of at least low strength was both observed as outcropping within and adjacent to the site as well as being intersected within all test locations via borehole or DCP refusal. It was encountered at a maximum depth below existing ground levels of 1.40m (DCP7).

Whilst a freestanding groundwater table was not identified within the investigation, minor seepage was observed above the bedrock surface.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of a relatively thin layer of topsoil/fill overlying residual soils and subsequently sandstone bedrock, encountered from ground surface in multiple locations and intersected at a maximum depth of 1.40m. Minor seepage was observed overlying the bedrock surface however a freestanding groundwater table or significant seepage was not encountered in the investigation and will not be intersected within the envelope of proposed works.

It is understood that the proposed works will involve alterations and additions to the existing site dwelling which will include the northerly extension of the structure across all three existing levels as well as the construction of a new in-ground pool. Excavation will be required for the extension of the lower two levels as well as the construction of the new pool and will extend to maximum depths of 3.00m, 2.40m and 1.85m respectively. All proposed works will be setback from site boundaries by a minimum of 10.00m.

Based on the results of the investigation, all excavations will extend through surficial and residual soils prior to intersection of sandstone bedrock which will extend to the Base Excavation Level (BEL).

Due to the relatively large setbacks of the excavations to site boundaries as well as the relatively shallow bedrock encountered across site, safe batter slopes will be feasible along all excavation edges with respect to neighbouring properties.

Additionally, the excavation for the proposed lift shaft within the existing Level 0 is situated within the footprint of the existing dwelling structure. The founding condition and depth of the existing footing system was not assessed as part of the investigation. It is understood based off anecdotal evidence and correspondence with the architect, that the dwelling is founded atop competent bedrock in the vicinity of the excavation. Regardless, the excavation presents a risk of undermining the dwelling due to the potential of encountering unfavorable defects within the rock mass. Therefore, caution must be exercised throughout the excavation process and regular geotechnical inspection should be undertaken to reconfirm conditions during excavation.

All new footings are recommended to bear atop competent bedrock to avoid the risk of differential settlement. As previously stated, investigation was not undertaken into footing style, depth and founding condition of the existing dwelling structure in any additional locations. The new dwelling extension should have a matching foundation as the existing dwelling unless differential settlement is factored into structural design. Let it be noted that the existing structure has likely completed all settlement under the existing load regime. Therefore, founding new structures to bedrock may be used to reduce settlement of the new structures.

Fill, natural soils and extremely to very low strength bedrock can be excavated using conventional earthmoving equipment (e.g. buckets and rippers), however low up to high strength rock requires the use of rock excavation equipment which can produce ground vibrations of a level which can potentially cause damage to nearby structures. Therefore, selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use however this is only expected to be required to protect the existing site structures. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use. It is recommended that a rock saw and small (≤250kg) rock hammers be proposed for use at this site to avoid the need for full time monitoring. Larger rock hammers will likely be preferred and as such, further assessment and potentially full time monitoring will be necessary.

At the time of the investigation the boulders within the site and immediate surrounds were all considered to be stable with no risk of impending instability identified. However, the bulk excavation proposed will impose risk of inducing instability within such boulders. Therefore, careful excavation methodology and frequent geotechnical inspections by CGC to ensure boulder stability will be required.

Cliff overhang stability calculations as per Young & Young 1992 were utilized, based on visible bedrock conditions, to assess the stability of the overhang adjacent to the southern site boundary and returned a factor of safety in excess of 3, it was therefore not considered as a hazard.

It is anticipated that due to the nature of the excavation that the majority of boulders within the vicinity footprint of proposed works site will be removed. Unstable boulders within a 1.0V:1.5H influence zone of an excavation base are recommended to be removed or secured by way of rock bolts and/or underpinning prior to or during any excavation on site. This will help to prevent potential movement both during and after excavation works. Boulders that are determined to be retained during the excavation and construction phase are subject to further geotechnical inspection by CGC.

The groundwater table was not intersected during investigation and is not expected within the site works based on site location/topography.

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 augered boreholes. This test equipment provides limited data from small, isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted subsurface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent initial design of the proposed works.

4.2. Site Specific Risk Assessment:

Based on our site investigation and the proposed works, it is considered that the only stability hazard associated with the proposed works is limited to the existing site dwelling. The hazard is:

A. Rockslide/topple ($<2m^3$) of bedrock around perimeter of excavation due to poorly oriented defects.

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 to be up to **2.81×10^{-7},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be '**Acceptable**' when assessed against the criteria of the AGS 2007 and the Geotechnical Risk Management Policy.

The above risk to life and property from Hazard A has been assessed assuming insufficient stabilizing measures/retention systems are constructed within the site. Where appropriate systems are installed the anticipated risks are expected to reduce further within "Acceptable" risk management criteria of the Council's policy. As such, the works are considered suitable for the site.

4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	- Class 'A' for footings in excavation base within bedrock
Type of Footing	Strip/Pad, Slab or piers
Sub-grade material and Maximum Allowable Bearing Capacity	- Stiff residual soils – 150kPa - Very Stiff residual soils – 200kPa - Hard residual soils – 400kPa - Weathered, VLS Bedrock: 800kPa - Weathered LS Bedrock: 1000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – Rock Site
Remarks: These values are subject to confirmation by geotechnical inspection/testing during construction. All permanent structure footings should be founded off bedrock similar strength to reduce the potential for differential settlement unless designed for by the structural engineer. 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 'certified' at the end of the project.	

4.3.2. Excavation:	
Depth of Excavation	Up to 3.00m for excavation within Level 0 Up to 2.40m for excavation within Level 1 Up to 3.00m for excavation for pool
Type of Material to be Excavated	Loose topsoil/fill with cobbles and boulders to potential maximum of 0.40m depth
	Residual soils to potential maximum of 1.40m
	Sandstone bedrock – VLS – MS, potentially HS from minimum of surface level

Guidelines for un-surcharged batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	1.5:1	2.0:1
Very Low (VLS) strength or fractured bedrock	0.75:1	0.5:1*
Medium strength (MS), defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by engineering geologist.

Remarks:

Seepage at the bedrock surface or along defects in the rock can also reduce the stability of batter slopes or rock cuts 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 permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval.

Should further detail on rock strengths or conditions for excavation costing be required, then cored boreholes and laboratory testing will be required.

Equipment for Excavation	Fill/natural soils	Bucket
	VLS bedrock	Bucket and ripper
	LS – MS/HS bedrock	Rock hammer and rock saw

VLS – very low strength, LS – low strength, MS – medium strength, HS – high strength

Remarks:

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

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

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
≥1 tonne	Up to 20.00m

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

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Neighbouring residential dwellings = 5mm/s Services = 3mm/s
Vibration Calibration Tests Required	If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed

Full time vibration Monitoring Required	Pending proposed excavation equipment and vibration calibration testing results, if required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Upon demolition and initial clearing of soils and structures adjacent to the proposed lift shaft • At 1.50m depth intervals of excavation • At completion of the excavation • Where ground conditions are exposed that differ to those expected
Dilapidation Surveys Requirement	Not necessary
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

4.3.3. Retaining Structures:

Required	New retaining structures are proposed as part of proposed works
Types	Steel reinforced concrete/concrete block post excavation designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.

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

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill and Residual Clay Soils/ EW bedrock	20	$\phi' = 30°$	0.33	0.47	3.25
VLS -LS bedrock	22	$\phi' = 38°$	0.10	0.20	200kPa

Remarks:

In suggesting these parameters, it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. 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 backfilled 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).

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Seepage identified at the and anticipated above bedrock surface
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.50L/min), within soil interface and at bedrock surface.
Site Location and Topography		Low southern side of the road within gentle to extreme east dipping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Due to the shallow bedrock the property is not suitable for onsite absorption disposal system, however a dispersion system may be possible in accordance with council requirements
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, downpipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer.		

4.4. Conditions Relating to Design and Construction Monitoring:

To comply with Councils conditions and to enable us to complete Forms: 2b and 3 required as part of construction, building and post-construction certificate requirements of the Councils Geotechnical Risk Management Policy 2009, it will be necessary for Crozier Geotechnical Consultants to:

1. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspection of site and works as per Section 4.3 of this report
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
4. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

The client and builder should make themselves familiar with the Councils Geotechnical Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot sign Form: 3 of the Policy if it has not been called to site to undertake the required inspections.

4.5. Design Life of Structure:

We have interpreted the design life requirements specified within Council's Risk Management Policy to refer to structural elements designed to support the existing structures, 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:

- stormwater and subsoil drainage systems,
- retaining walls 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 – 2011 (50 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 don't 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 "Landslide Risk Management" Volume 42, March 2007.
  c) AS 2870 – 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 don’t leak and increase either the local groundwater level or landslide potential.

5. CONCLUSION:

The site investigation identified the presence of sandstone bedrock at relatively shallow depths across site, overlain by a relatively thin layer of residual soils and topsoil/fill featuring sandstone cobbles and boulders. The residual soils were generally intersected as at least stiff and graded to hard with depth prior to intersection of sandstone bedrock interpreted as at least low strength. A freestanding water table or signs of significant seepage were not observed within the investigation range and are considered unlikely within the envelope of proposed works.

The proposed works involve alterations and additions to the existing site structures including the northerly extension of the dwelling across all three existing levels. The proposed works will require excavation in multiple locations, extending to a maximum anticipated depth below existing ground levels of 3.00m.

Setbacks to neighbouring boundaries limit ground vibration concerns and allow for the implementation of safe batter slopes however one of the excavations will be directly underneath the existing dwelling structure.

The founding condition of the site dwelling should be assessed prior to bulk excavation to avoid the potential of undermining the structure adjacent to the excavation as well as to ensure the completed structure does not have a varied founding condition.

Careful consideration should be taken during the demolition and excavation phases to ensure that all boulders across the site remain in a stable condition. More boulders may be intersected during the excavation phase, stabilisation or removal of these boulders will be critical to ensure safe working conditions. Inspections by CGC as per Section 4.3 will be required during the excavation and construction works.

There is expected to be negligible impact to the local hydrogeology with the water table not intersected or expected with the depth of proposed works. As such, no ASS management plan will be required as part of the proposed works.

The risks associated with the proposed development can be maintained within 'Acceptable' risk management levels with negligible impact to neighbouring or site 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:
James Dee
Geotechnical Engineer

Reviewed by:
Troy Crozier
Principal Engineering Geologist
MIEAust., MAIG, RPGeo
Registration No.: 10197

6. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
4. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
5. Australian Standard AS 2870 – 2011, Residential Slabs and Footings – Construction
6. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia
7. Australian Standard AS 1726 – 2017, Geotechnical Site Investigations

Appendix 1

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

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

Q_c (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: -

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

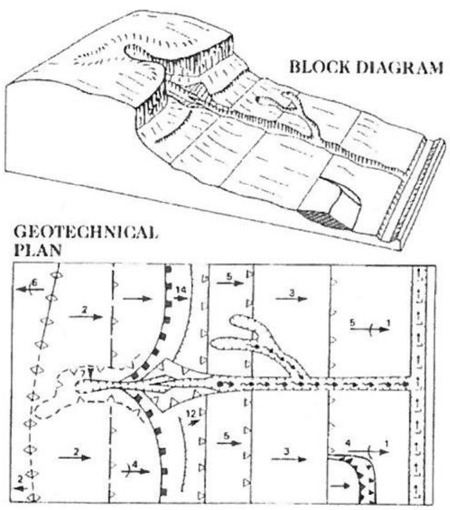
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

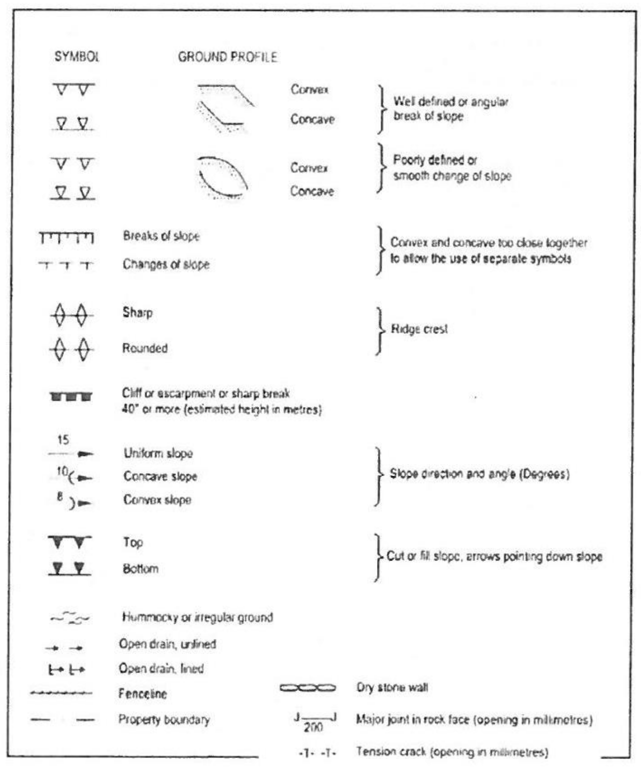
Reproduction of Information for Contractual Purposes

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

Site Inspection

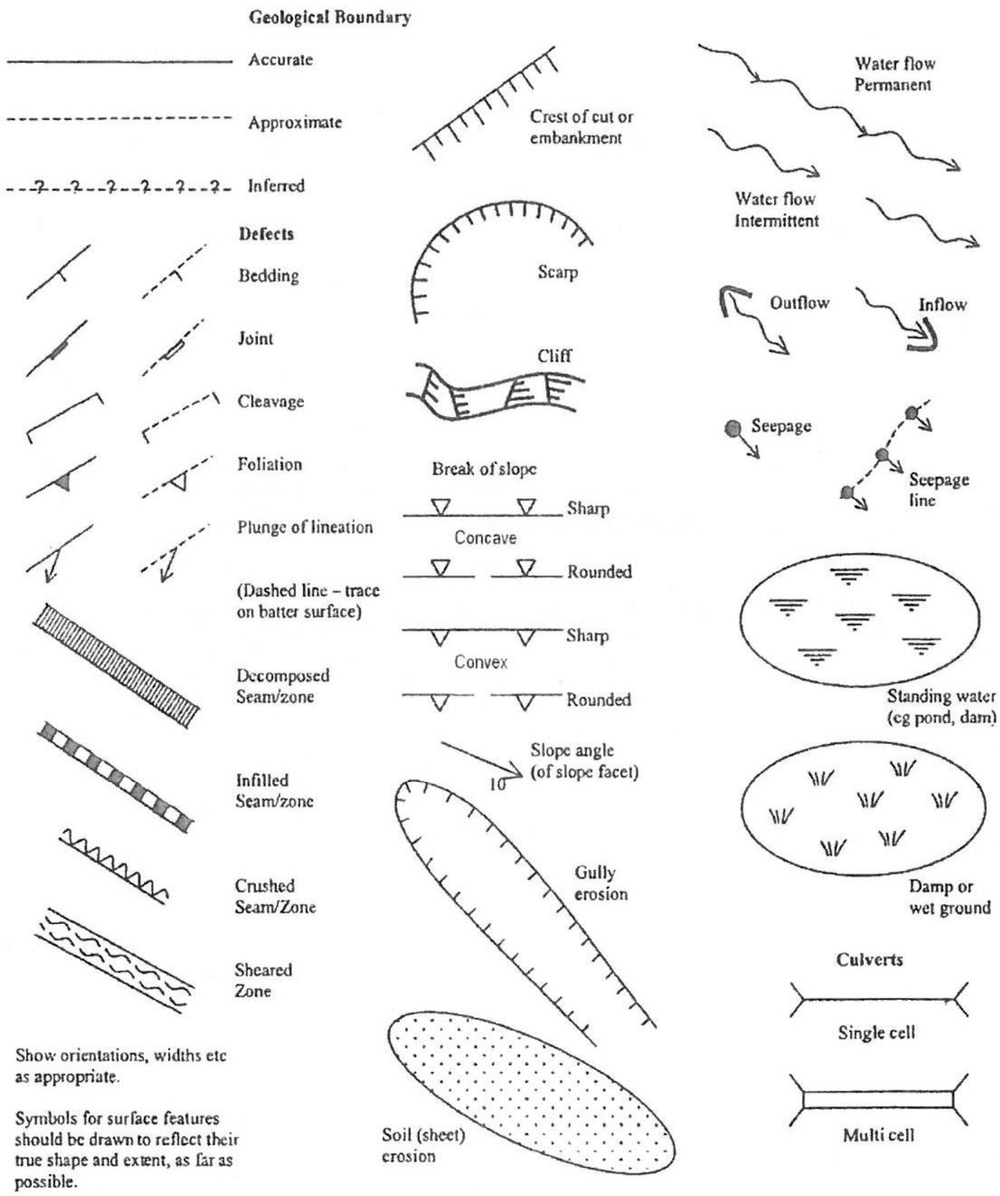
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





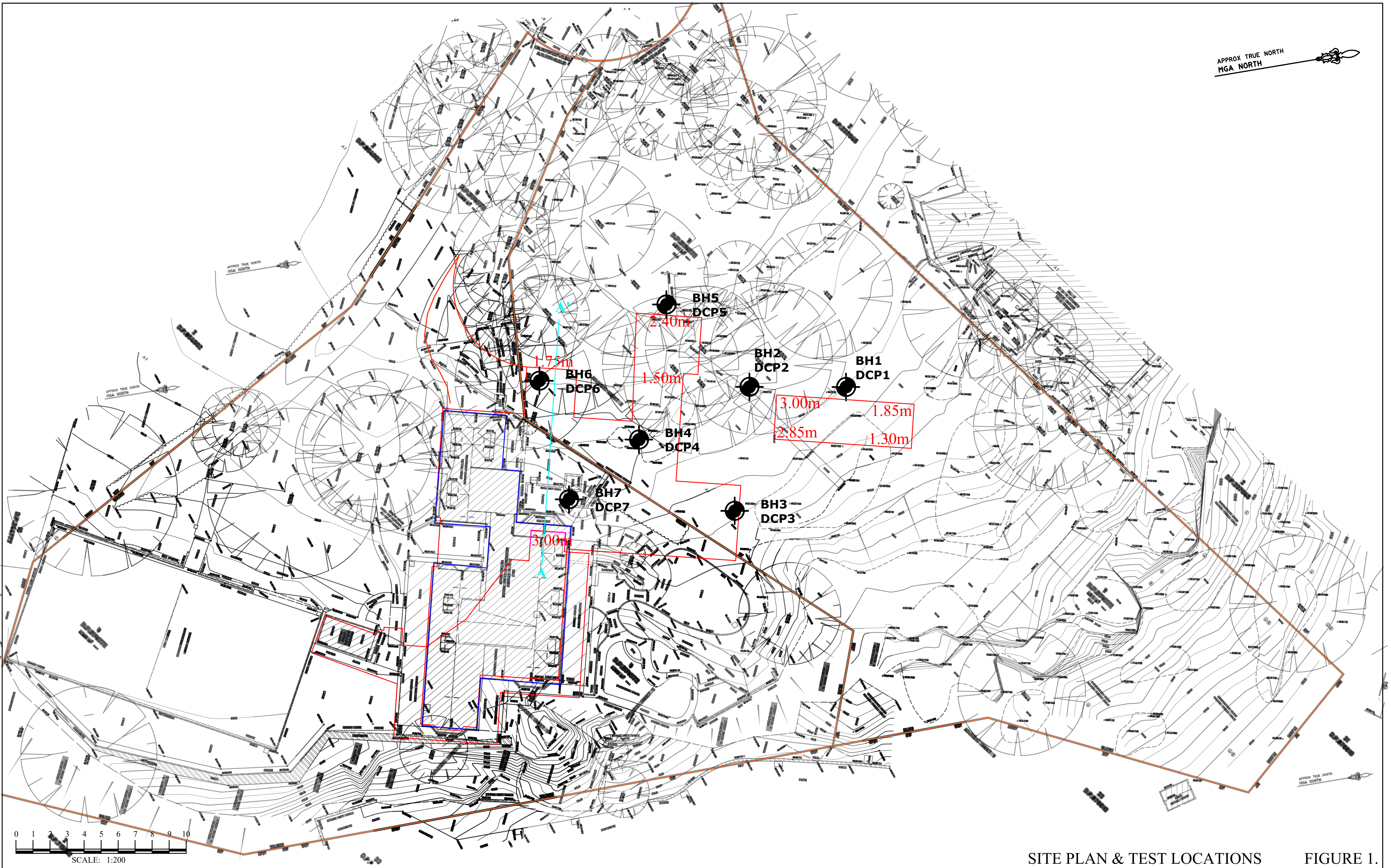
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

LEGEND

EXISTING STRUCTURES	(blue line)
PROPERTY BOUNDARY	(orange line)
PROPOSED WORKS OUTLINES WITH APPROXIMATE EXCAVATION DEPTHS	(red line)
AUGER / DYNAMIC CONE PENETROMETER LOCATION	BH DCP
CROSS-SECTION REFERENCE LINE	A — A'

CROZIER GEOTECHNICAL CONSULTANTS

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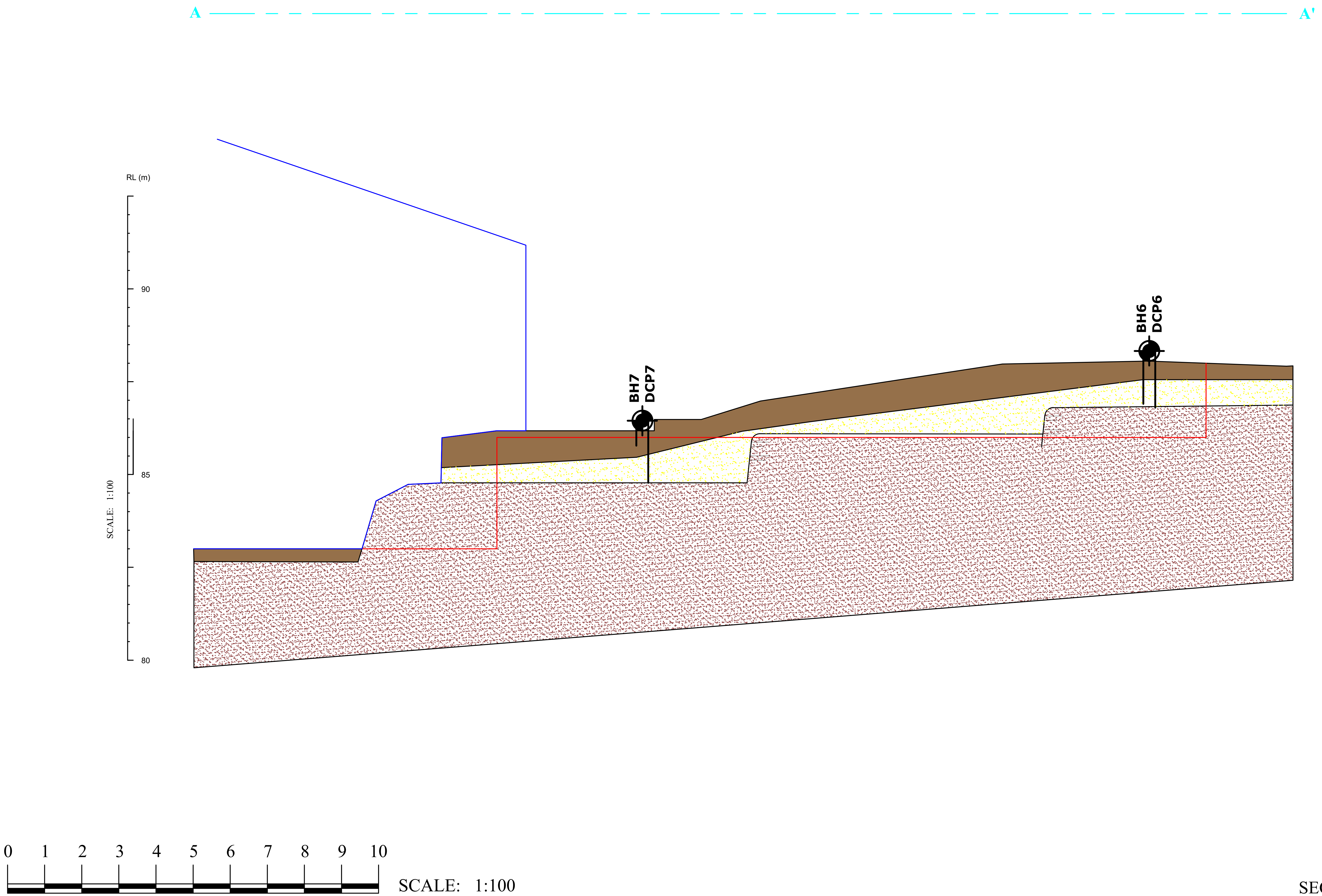
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE:	1:400 @ A3
DRAWING:	FIGURE 1
DATE:	05 /2023
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2023-081

PREPARED FOR:
Matthew and Louise Baxter

ADDRESS:
6-7 Kara Crescent, Bayview



SECTION A: FIGURE 2

CROZIER GEOTECHNICAL CONSULTANTS		SCALE: 1:100 @ A3	PREPARED FOR: Matthew and Louise Baxter
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**: EXISTING STRUCTURES; PROPOSED EXCAVATION; PROPERTY BOUNDARY; BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION; A — A' SECTION LINE	DRAWING: FIGURE 2 DATE: 04/2023	
		APPROVED BY: TMC DRAWN BY: JD PROJECT: 2023-081	ADDRESS: 6-7 Kara Crescent, Bayview

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 1

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 86.00m

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.30	SM	SILTY SAND: medium dense, yellow/brown, fine to medium grained, moist/dry				
0.50		… moist, with some clay zones				
0.80	SC/CL	SANDY CLAY: hard, yellow/brown with red and pale grey mottle, medium plasticity moist	D	0.90 1.10		
1.10		Hand auger refusal @ 1.10m atop interpreted LS sandstone bedrock				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter
DATE: 12/04/2023
BORE No.: 2/2a

PROJECT: Alterations and Additions
PROJECT No.: 2023-081
SHEET: 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview
SURFACE LEVEL: Rl 86.70m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.40	SM	SILTY SAND: dense, yellow/brown, fine to medium grained, moist/dry				
0.50		Hand auger refusal @ 0.50m atop sandstone cobbles, DCP extended to 1.10m				

RIG: Not applicable
DRILLER: JD

METHOD: Hand Auger
LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:
CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 3/3a

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 85.20m

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.40		Hand auger refusal @ 0.40m on sandstone boulder, DCP extended to 1.20m				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 4

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 87.20m

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.20		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
		Hand auger refusal @ 0.20m atop sandstone boulder, DCP extened to 1.35m				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 5

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 88.60m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.40	SC/CL	SANDY CLAY: stiff, yellow/brown, medium plasticity, moist				
0.50	CL	… with silt/ silty clay				
0.80		… with pale grey and red mottle, powdery (potentially extremely weathered bedrock)	D	0.80 0.90		
1.10		Hand auger refusal @ 1.10m atop interpreted LS sandstone bedrock				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 6

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 86.00m

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.30		… with sandstone cobbles				
0.50	SC/CL	SANDY CLAY: firm, yellow/brown, medium palsticity, moist				
0.80	CL	… stiff, with some silt/ silty clay				
0.90		… with pale grey and red mottle				
1.15		Hand auger refusal @ 1.15m depth atop interpreted LS sandstone bedrock				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Matthew and Louise Baxter **DATE:** 12/04/2023 **BORE No.:** 7

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081 **SHEET:** 1 of 1

LOCATION: 6 - 7 Kara Crescent Bayview **SURFACE LEVEL:** Rl 86.07m

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil/fill: loose, dark brown, moist/dry silty sand with roots and sandstone gravels				
0.30		… with drainage gravels (Blue metal)				
0.40						
		Hand auger refusal @ 0.40m within gravel band, DCP extended to 1.40m				

RIG: Not applicable DRILLER: JD

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Mtthew and Louise Baxter **DATE:** 12/04/2023

PROJECT: Alterations and Additions **PROJECT No.:** 2023-081

LOCATION: 6 - 7 Kara Crescent, Bayview **SHEET:** 1 of 1

	Test Location									
Depth (m)	1	2	3	4	5	6	7			
0.00 - 0.10	4	0	0	-	0	0	1			
0.10 - 0.20	6	3	1	-	0	2	2			
0.20 - 0.30	4	2	2	-	1	5	2			
0.30 - 0.40	6	3	1	-	3	7	2			
0.40 - 0.50	6	8	2	1	2	7	2			
0.50 - 0.60	5	5	2	0	3	3	3			
0.60 - 0.70	5	5	2	1	6	3	3			
0.70 - 0.80	7	6	3	2	6	3	2			
0.80 - 0.90	11	4	5	8	9	4	4			
0.90 - 1.00	11	12	7	4	B@0.90	5	4			
1.00 - 1.10	13	20	10	2		6	11			
1.10 - 1.20	B@1.10	B@1.10	22	10		8	11			
1.20 - 1.30			B@1.20	8		7	12			
1.30 - 1.40				9		B@1.25	14			
1.40 - 1.50				B@1.35			B@1.40			
1.50 - 1.60										
1.60 - 1.70										
1.70 - 1.80										
1.80 - 1.90										
1.90 - 2.00										
2.00 - 2.10										
2.10 - 2.20										
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

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

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (rockslide/topple >2m³) of bedrock around perimeter of excavation due to poorly oriented defects		3.00m excavation primarily through sandstone	a) Excavation underneath existing dwelling, impact 5%		a) Person on dwelling 18 hrs/day avge.. .	a) Likely to not evacuate	a) Person in dwelling, minor damage only	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Existing site dwelling	0.0001	1.00	0.05	0.750	0.75	0.1	**2.81E-07**

* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
  Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)
* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* 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). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (rockslide/topple >2m³) of bedrock around perimeter of excavation due to poorly oriented defects	a) Existing site dwelling	Unlikely	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low

* 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%.
* Cost of site development estimated at $1,000,000

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

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

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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.

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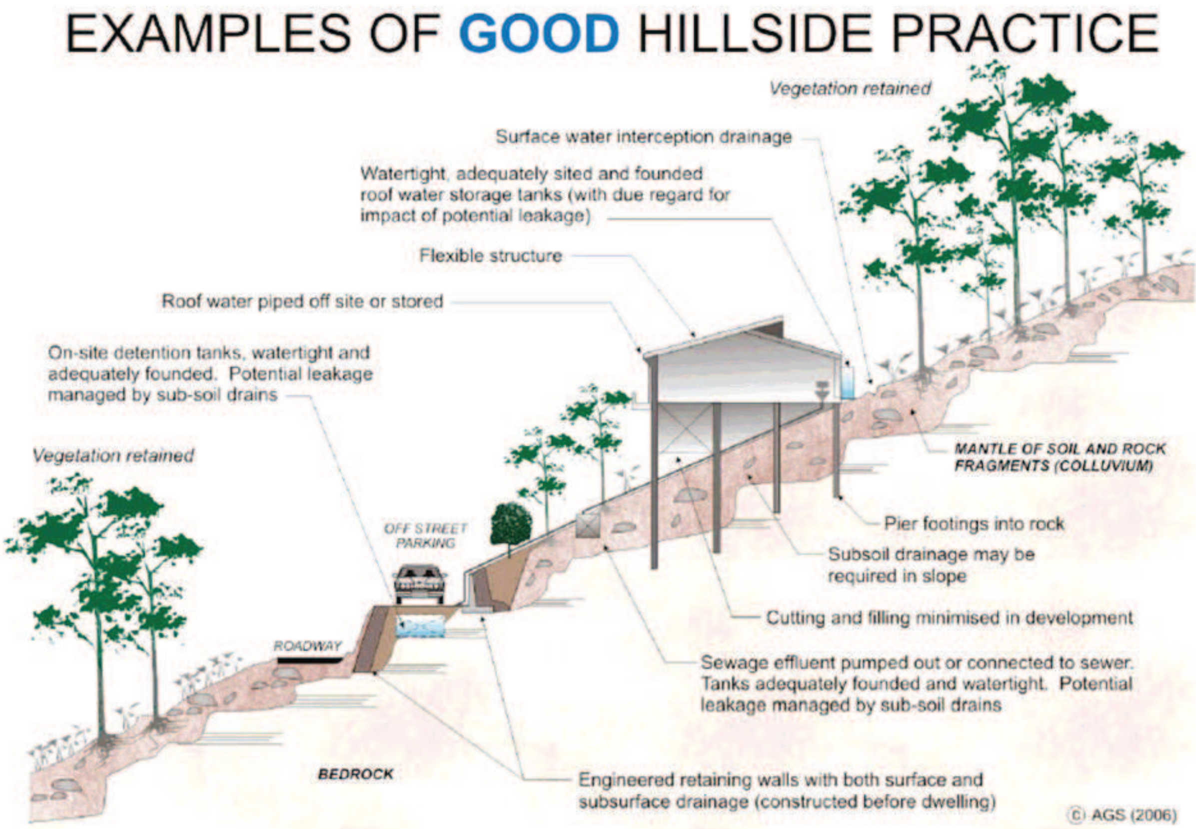
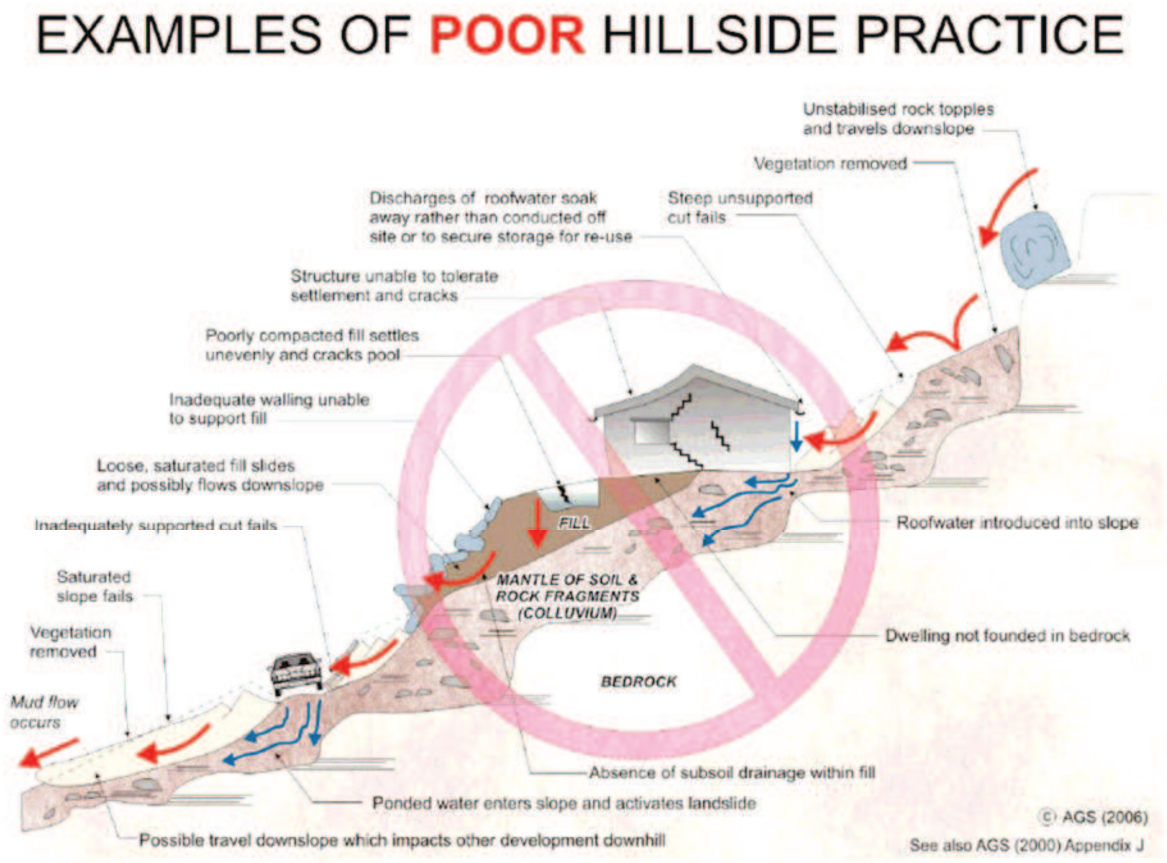
Appendix 5

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
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
© AGS (2006)
See also AGS (2000) Appendix J