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REPORT ON GEOTECHNICAL SITE INVESTIGATION AND LANDSLIP RISK

for

PROPOSED SECONDARY DWELLING

at

31A QUEENS AVENUE, AVALON

Prepared For

Mr. Boris Panov

Project: 2015-248

May, 2016

Document Revision Record

Issue No	Date	Details of Revisions
0	9th December 2015	Original Issue
1	31st May 2016	Updated Architectural Design

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

Development Application for ______________________________	
	Name of Applicant
Address of site ___ 31a Queens Avenue, Avalon_____	

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 __31st May 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:

☐ 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 Site Investigation and Risk Assessment Report for Proposed Secondary Dwelling	
Report Date: 31st May 2016	**Project No.:** 2015-248.1
Author: J. Butcher	
Author's Company/Organisation: Crozier Geotechnical Consultants	

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

Design Drawings – Ian Cubitts Classic Home Improvements, Job No.: 20743, Drawing No.: 20743 01 to
20743 07, Issue: E, Dated: 17/05/2016
Survey – Survcheck Surveyors, Reference: 4193D1, Dated: 11 May 2015

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
10197

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 __31a Queens Avenue, Avalon ____________

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 Site Investigation and Risk Assessment Report for Proposed Secondary Dwelling
Report Date: 31st May 2016 **Project No.:** 2015-248.1
Author: J. Butcher
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted __01/12/2015______ (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 ………01/12/2015……………………………………….
- ■ 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 ………………………………………………………

Name …**Troy Crozier**……………………………………

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

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

Company… **Crozier Geotechnical Consultants**

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Date: 31st May 2016

Project No.: 2015-248.1

GEOTECHNICAL SITE INVESTIGATION & LANDSLIP RISK ASSESSMENT FOR PROPOSED SECONDARY DWELLING 31A QUEENS AVENUE, AVALON, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for the construction of a proposed secondary dwelling at 31A Queens Avenue, Avalon, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Mr. Boris Panov.

It is understood that the proposed works involve construction of a raised secondary dwelling at the rear of the property. The new structure will be constructed above existing ground surface levels and will require only minor excavation for new footings.

The investigation was carried out to provide information for Development Application purposes. The site has been classified under the Pittwater Council Geotechnical Risk Management Policy 2009 as being within the H1 landslip hazard zone therefore the site requires a Geotechnical Landslip Risk Assessment to be conducted. The site is also classified as Acid Sulfate Soils Map ï Class 5 under the Council LEP 2013 which requires an assessment for Acid Sulfate Soils. The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent land by a Geotechnical Engineer and Principal Engineering Geologist including a photographic record of site conditions.
b) Review of Ortho Photomaps and Aerial Photography of the site.
c) Drilling of test boreholes to investigate subsurface geology and depth to bedrock across the site with dynamic penetrometer testing to assess soil strengths. The investigation was limited to hand equipment due to site access limitations.

The following diagrams were supplied by the structural engineers for the work;

- Architectural Drawings by Ian Cubittôs Classic Home Improvements, Job No. 20743, Drawing No.ôs: 20743 01 to 20743 07, Issue: E, Dated: 17/05/2016.
- Site Survey Plan by Survcheck Surveyors, Reference: 4193D1, Dated: 11 May 2015.

2. SITE FEATURES:

2.1. Description:

The site is located on the north-eastern face of a north-west plunging ridgeline that continues towards Pittwater. It contains a one and two storey brick dwelling in the middle of the block with a carport at the front and a gently sloping backyard at the rear. The site is mainly rectangular with rear, south-east boundary of 19.90m and south boundary of 38.86m as referenced from the provided survey plan. A narrow (1.905m) right of access extends approximately 50 metres from the site down to the roadway resulting in a ðbattle-axeô block. The site is adjoined by residential properties.

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is near a boundary between Newport Formation (Rnn) of the Upper Narrabeen Group and Hawkesbury Sandstone (Rh).

Hawkesbury Sandstone is of Triassic Age and the rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminate and forms a capping to the ridges in this area.

Newport Formation (Upper Narrabeen Group) is of middle Triassic Age and typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones. This unit underlies the Hawkesbury Sandstone and is expected below the site based on our previous experience within the area.

Narrabeen Group rocks are dominated by shales and thin siltstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise interbedded shale and siltstone beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints. The shale often forms deeply weathered silty clay soil profiles (medium to high plasticity) with thin silty colluvial cover.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 1st December 2015 by a Geotechnical Engineer which included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of slopes and existing structures. It also included the drilling of two auger boreholes (BH1 ï BH2) using a hand auger due to access limitations to investigate sub-surface geology.

Dynamic Cone Penetrometer (DCP) testing was carried out adjacent to and through the boreholes and at additional locations, in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone 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, Appendix 2.

3.2. Field Observations:

From Queens Avenue a shared concrete driveway provides access up to the site and several neighbouring properties. The driveway is steep (-20°) and ends approximately 50m from the roadway edge with a small cul-de-sac/turning circle. The site adjoins the turning circle at its eastern side.

The front of the site consists of a concrete paved car port with a low sandstone flagging retaining wall along its western side with cracks following along the mortar. The north east side boundary consists of a concrete strip driveway which ends near the site house and a moderately sloping (-11°) unsealed path that leads to the backyard beyond the driveway. The south west side boundary at the front consists of concrete paved stairs.

The residence itself consists of a one and two storey brick dwelling in good condition at the centre of the property on moderately south east sloping topography. Due to the natural ground surface slope of the site, the dwelling has a lower level towards the rear. The rear of the dwelling consists of a timber retaining wall up to 1.50m high in good condition. Past the retaining wall is a gently south east sloping lawn with a steeply south east sloping embankment covered in dense vegetation at the rear boundary.

The neighbouring property to the north (No. 31 Queens Avenue) consists of a two storey timber and weatherboard dwelling in good condition, situated at the front portion of the property on gentle to moderately south east sloping topography. The property is at a slightly lower ground level to the site along the boundary supported by rendered masonry and timber log retaining walls, with the dwelling located within 2.00m of the common boundary.

The neighbouring properties to the rear south east (No. 26 and 24 William Street) both consist of two storey dwellings located downslope to the site. Both of the dwellings are located within 11.00m of the common boundary. No. 26 contains a level lawn and paved terrace near the boundary, supported on the east side by a retaining wall above a pool terrace adjacent to the house.

The neighbouring property to the south west (No. 33 Queens Avenue) consists of a vacant block on moderately south east sloping (-17°) topography. The property is at a similar ground level to the site along the boundary. Where the shared driveway joins this property is a cutting into the hill slope with a dirt track that enters in to the property. On the upslope side of the track cutting weathered Narrabeen Group bedrock was visible from 0.50m depth below the adjoining ground surface. Crozier Geotechnical have previously carried out geotechnical investigation within this property which identified this property is underlain by shallow topsoil (Ò0.20m depth) underlain by firm to very stiff residual clay. Extremely low to very low strength bedrock was intersected from 0.55m to 1.0m depth.

3.3. Boreholes:

The boreholes (BH1 ï BH2) were drilled at the rear of the house towards the south west boundary (BH1) and at the north east boundary (BH2). The boreholes refused at shallow depths (0.35m) on ironstone banding in stiff clay of low plasticity.

A 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.

Dynamic Cone Penetrometer (DCP) tests were undertaken adjacent to the boreholes from the surface and at various locations at the rear. DCP1 and DCP3 were completed closest to the dwelling and refused at 1.25m and 1.35m depth respectively. It can be interpreted that the DCP refused on sandstone bedrock of at least very low strength. DCP2, DCP4 and 4a were completed closer to the rear boundary with DCP2 discontinued at 1.95m depth and DCP4a refusing at 1.80m, both on suspected sandstone bedrock of at least very low strength. DCP4 refused at 0.15m on a boulder.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified that the site is underlain by shallow layers of predominantly clayey sand topsoil and fill underlain by natural colluvium and residual clay. The investigation was undertaken with hand tools due to the steep nature of the site. The bores refused on ironstone bands in the fill and clay soil between 0.10m to 0.35m depth. It is interpreted that stiff to very stiff residual clay is present from 0.30m depth to near the rear of the site house. The results further down slope at BH2 and DCP4 indicate deeper fill to approximately 1.00m depth underlain by stiff to very stiff residual clay. The penetrometer results indicate weathered bedrock from between 1.00m to 1.80m depth. Bedrock was also visible in the driveway cutting at the front of the southern neighbouring property from about 0.50m depth. The groundwater table was not observed in the boreholes drilled during this geotechnical investigation and is not expected within the site works.

It is expected that the bedrock surface will be controlled on a smaller scale by variations in the type and weathering of the bedrock, which is expected to consist of numerous sandstone horizons and occasional thin siltstone/shale beds that will form a stepped profile down the slope towards the east to south-east. The profile will consist of small cliffs and ledges with the ledges infilled with colluvial soils. Detached sections/boulders will directly overlie the bedrock and also be entrained within the colluvium, therefore all new footings need to be inspected by a geotechnical professional to ensure they are supported off insitu bedrock. Some areas of the bedrock are extremely weathered and contain interbedded sandstone and siltstone with occasional shale bands. Narrabeen Group Rocks can weather to significant depth, therefore there is potential for areas to contain deeper soil profiles.

Due to the observed site geology and geomorphology there is no likelihood of intersecting Acid Sulfate Soils below the site, whilst the proposed works will have no impact on the water table external to the site. Therefore no further investigation or reporting into these soils is required.

The site extends across a moderate to steep slope. However, there were no signs of existing or previous, deep seated or large scale landslip instability identified within the site or adjacent properties. The two identified hazards are related to existing timber and stone flagging retaining walls. The timber retaining structure will not remain stable over the 100 year design life as required by the Council. The site has been assessed against Pittwater Councilǿs Geotechnical Risk Management Policy (2009) and Australian Geomechanics Society guidelines (2007).

The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser strength than interpreted. For confirmation of bedrock strength to below proposed footing level will need an investigation utilizing cored boreholes in the actual footing location however access for such equipment is very limited by site conditions and it is not considered necessary. As such bedrock strength at foundation level can be confirmed by geotechnical inspection during initial excavation/construction works.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site 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 hand drilling tools due to access limitations. 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 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. Slope Stability & Risk Assessment:

Based on our site mapping we have identified the following credible geological/geotechnical landslip hazards which need to be considered in relation to the existing site:

A. Collapse of existing timber retaining wall at the rear ($<2m^3$)
B. Debris slide due to collapse of stone flagging retaining wall at front next to driveway ($<2m^3$)

A qualitative assessment of risk to life and property related to this hazard is presented in Table 1A and 1B, Appendix: 3, and is based on methods outlined in Appendix C of the Australian Geomechanics Society Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

The frequency of failure was interpreted based on the methods of MacGregor et.al. (AGS 2007), due to the lack of evidence of previous instability within the site.

The risk assessment identified that Hazards A and B achieve a Risk to Life of Ò5.21 $x10^{-6}$ and a Risk to Property of ôLowô, in their current condition which is considered ôAcceptableô. The replacement of the existing retaining walls with a new engineered structure will reduce the likelihood of failure to ôRareô and therefore the risks to well within ôAcceptableô levels. The natural hill slope features can also be managed through proper implementation of surface stormwater control and sensible hill side construction and maintenance into the future to avoid additional landslip hazards developing, see Appendix: 5.

4.3. Design & Construction Recommendations:

4.3.1. New Footings:

The results of the subsurface investigation suggest that the site is underlain by relatively shallow (<1.00m) topsoil and fill overlying clayey colluvial soils. As inferred from the penetrometer testing the bedrock surface within the area of the proposed structure is present from approximately 1.0m to 1.80m below the existing ground surface. The bedrock underlying the site is of the Narrabeen Group Rocks and will weather to significant depths with medium to high strength bedrock potentially at Ó5.00m depth. It will contain inter-bedded sandstone and siltstone with shale bands.

It is expected that the bedrock surface will be controlled on a smaller scale by variations in the type and weathering of the bedrock, which is expected to consist of numerous sandstone horizons and occasional thin siltstone/shale beds that will form a stepped profile down slope towards the east. The profile will consist of small cliffs and ledges with the ledges in-filled with colluvial soils. Detached sections/boulders will directly overlie the bedrock and also be entrained within the colluvium. All new footings need to be inspected by a geotechnical professional to ensure they are supported off in-situ bedrock.

The new secondary dwelling will be raised above the ground surface and it is considered likely that new footing design will be based on bored pier footings. Pier footings are recommended within sloping areas to reduce the impact on slope stability. It is recommended that all new footings be founded within a minimum very low strength bedrock expected at about 2.0m depth to ensure stability within the steep topography. Piers should ideally be socketed at least 0.50m into insitu very low strength bedrock to give the piers lateral support in the event of an earthquake and resist lateral soil pressures.

Footings founded on very low strength sandstone bedrock should be designed for a maximum allowable bearing pressure of 800kPa whilst 1000kPa can be used for low strength bedrock.

New building footing trenches and piers must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and to confirm the insitu nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project (Pittwater Councils Form: 3).

For classification purposes the site would be considered Class óPô site as per the Australian Standard for Residential Slabs and Footings AS2870 ï 2011, due to the site being within an area identified as prone to landslip by Pittwater Council.

4.3.2. Drainage & Groundwater:

No groundwater table will be intersected in the proposed works however groundwater seepage can be expected at the soil rock interface and on geological defects within the bedrock. This seepage may be under slight artesian pressures due to water head from joints in the rock mass further upslope.

New down pipes and stormwater intercept trenches should be connected to an engineered stormwater system and discharged to the Councilôs stormwater system off site.

4.3.3 Site Exposure

The site is located less than 1km from a surf coast and is therefore within a severe marine environment as per Australian Standard 3700 (2001) ï Masonry Structures. It is important to take into consideration the given environment during the design and construction of new brickwork, to minimise the potential for salt attack. The most suitable mortar joints for aggressive environments are ironed or weather struck joints.

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 drawings and stormwater system plans for compliance with the recommendations of this report including signing off of the structural engineerôs assessment of the recent works and retaining walls,
2. 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,
3. Inspect the completed development to ensure all stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

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 can not 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 Councils Risk Management Policy to refer to structural elements designed to support the and secondary dwelling, the adjacent slope, control stormwater and maintain the risk of instability within óAcceptableô 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 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 1996 (50 years)). In order to attain a design life of 100 years as required by the Councils Risk Management Policy, 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. It should be noted that timber log/sleeper retaining walls will not remain stable for 100 years.

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: 2 and should also include the following guidelines.

- The conditions on the block donøt 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, and 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). Should the client have any concerns about the implementation of the recommended inspection and maintenance program they should contact the geotechnical engineer for clarification or to complete the component of the property. It is assumed that Pittwater 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 donđt leak and increase either the local groundwater level or landslide potential.

5. CONCLUSION:

The geotechnical investigation identified stiff to very stiff residual clay is present from 0.30m depth at the rear of the site house. The results further down slope near the eastern boundary at BH2 and DCP4 indicate deeper fill to approximately 1.0m depth underlain by stiff to very stiff residual clay. The penetrometer results indicate weathered bedrock from between 1.0m to 1.80m depth.

The proposed works involve construction of a raised secondary dwelling at the rear of the property. No bulk excavation works are proposed however it is expected that the new building will be founded off pier footings which will require minor excavation. The new building footings should be socketed into at least very low strength bedrock to provide lateral restraint in the steeply sloping area. There is also the potential for detached boulders within the slope and the new footings will require inspection to verify their bearing capacity and the insitu nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project (Pittwater Councils Form: 3).

There were no signs of previous or impending large scale slope instability within the site whilst the proposed works are not expected to create any new geotechnical hazards provided they are undertaken as per the recommendations of this report. The entire site and surrounding slopes has been assessed as per the Pittwater Council Geotechnical Risk Management Policy 2009 and the identified landslip hazards were assessed as having óAcceptableô risk levels.

Due to the observed site geology and geomorphology there is no likelihood of intersecting Acid Sulfate Soils below the site, whilst the proposed works will have no impact on the water table external to the site. Therefore no further investigation or reporting into these soils is required.

Based on our investigation, the site is suitable for the proposed works which can be carried out without adverse impact to the site or neighbouring areas provided proper engineering design and construction methods are implemented, including but not limited to the recommendations of this report. As such the site and proposed development can achieve the ñAcceptableò risk management required by Councils Policy

Prepared by:
James Butcher
Senior Geotechnical Engineer

Reviewed by:
Troy Crozier
Principal Engineering Geologist

6.0. REFERENCES:

1. Walker et al. ñGeotechnical Risk Associated with Hillside Developmentò Australian Geomechanics Society, Number10, December 1985.
2. Walker et. al. May 2002, Titled ñLandslide Risk Management Concepts and Guidelinesò in the Journal and News of the Australian Geomechanics Society, Volume 37, No 2.
3. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol 42, No 1, March 2007.
4. Geotechnical Risk Management Policy for Pittwater, 20th July 2009.
5. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall.

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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.

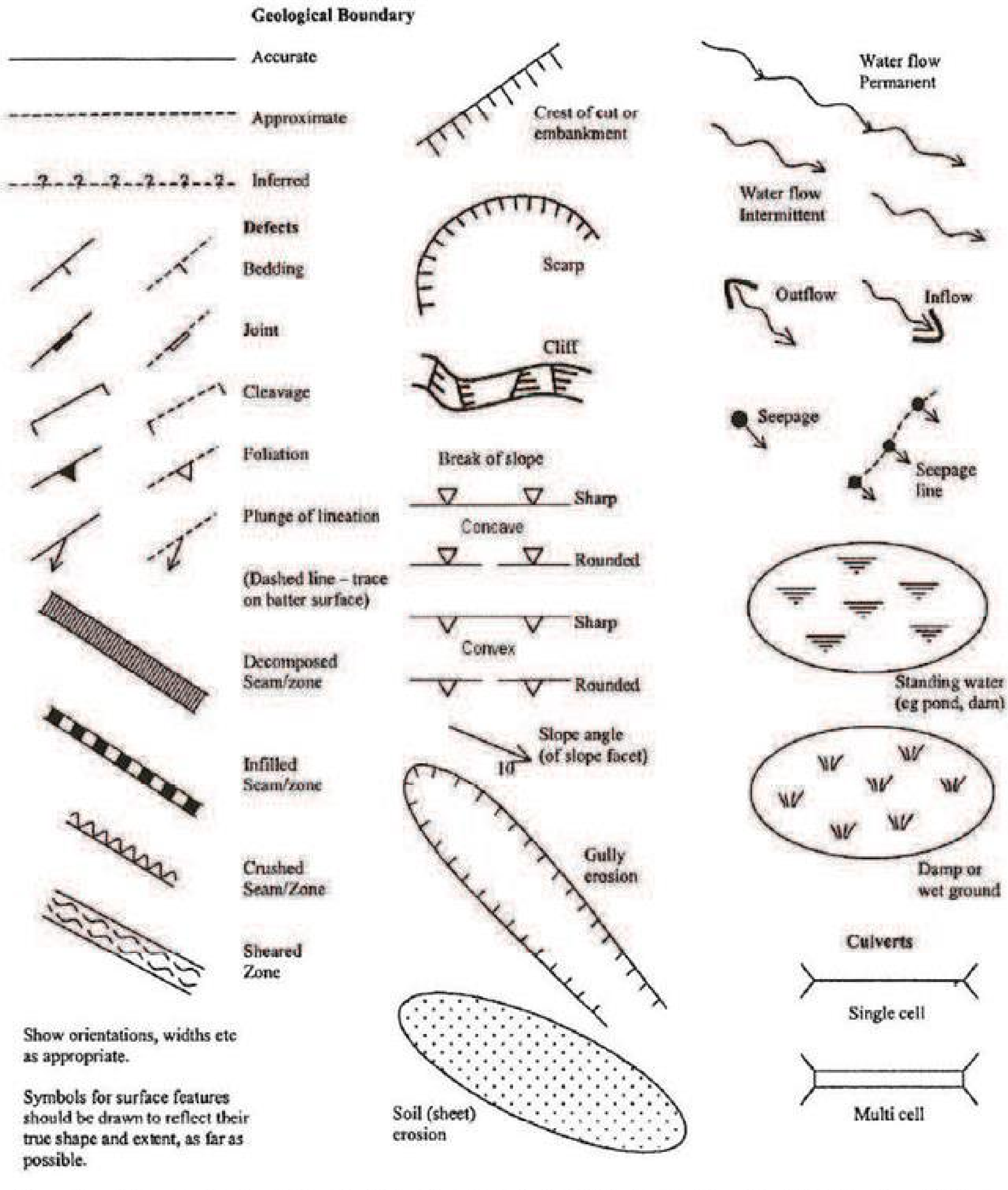
Reproduction of Information for Contractual Purposes

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.

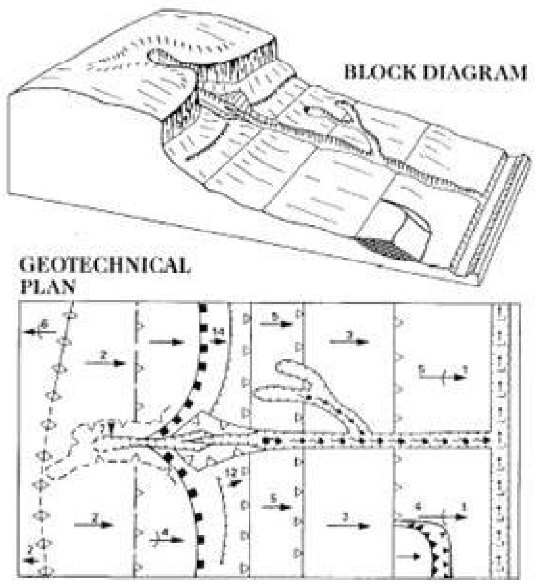
Site Inspection

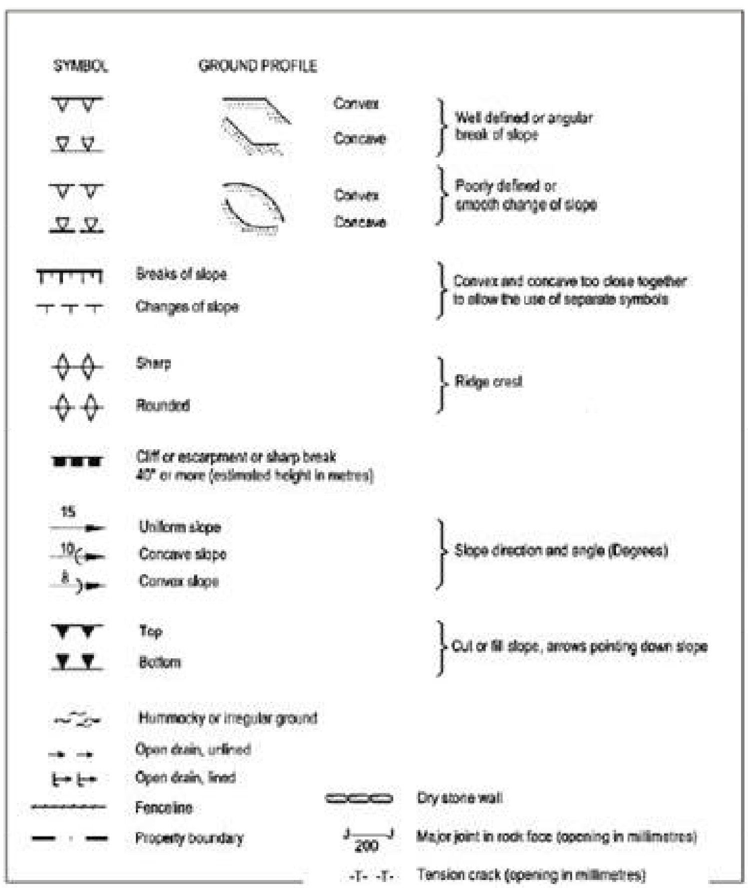
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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).

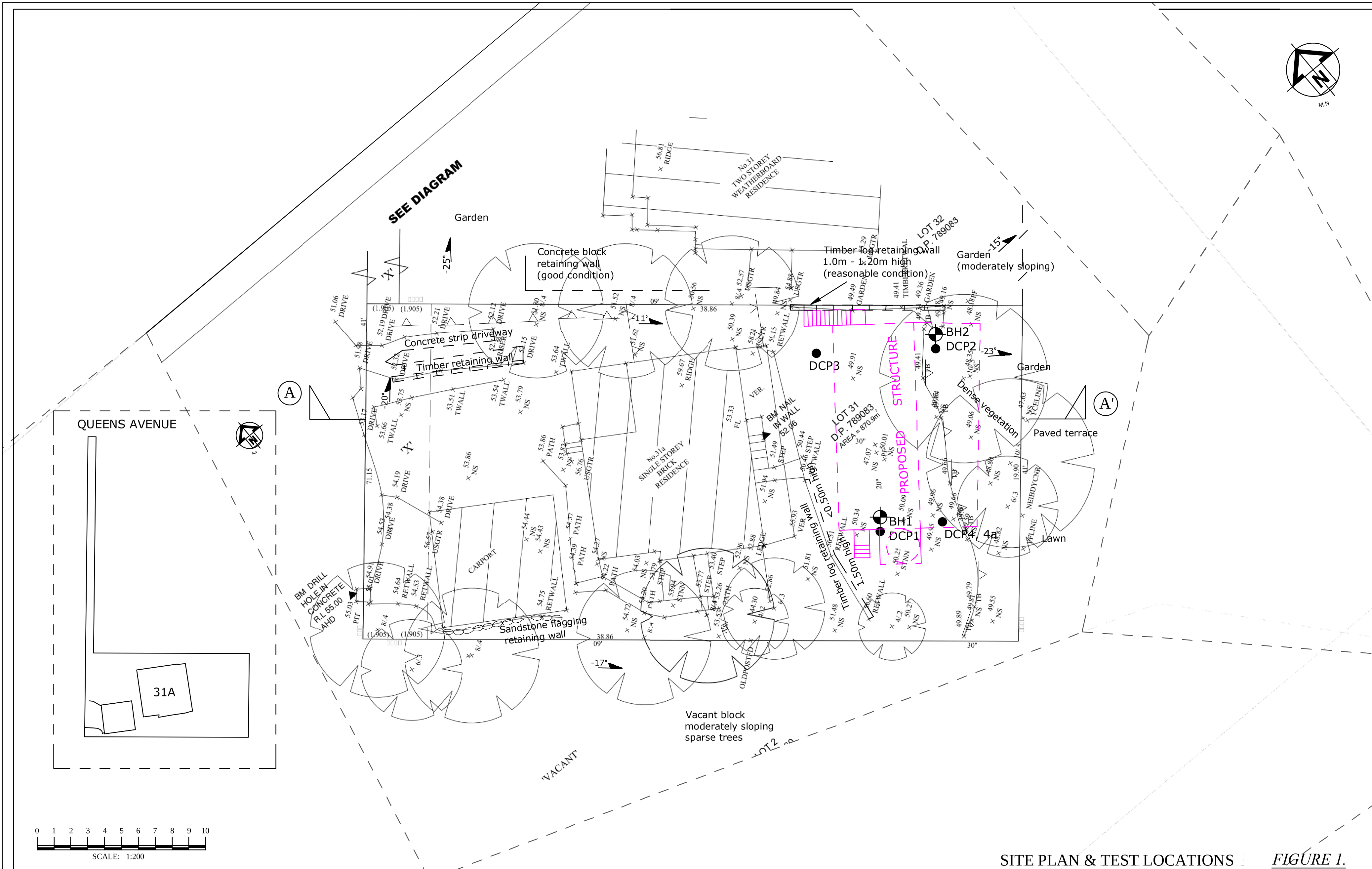




Example of Mapping Symbols

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

Appendix 2



SITE PLAN & TEST LOCATIONS

FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

- TOP OF SLOPE
- BASE OF SLOPE
- AUGER LOCATIONS
- DYNAMIC PENETROMETER TEST
- SLOPE ANGLE
- SURFACE WATER FLOW

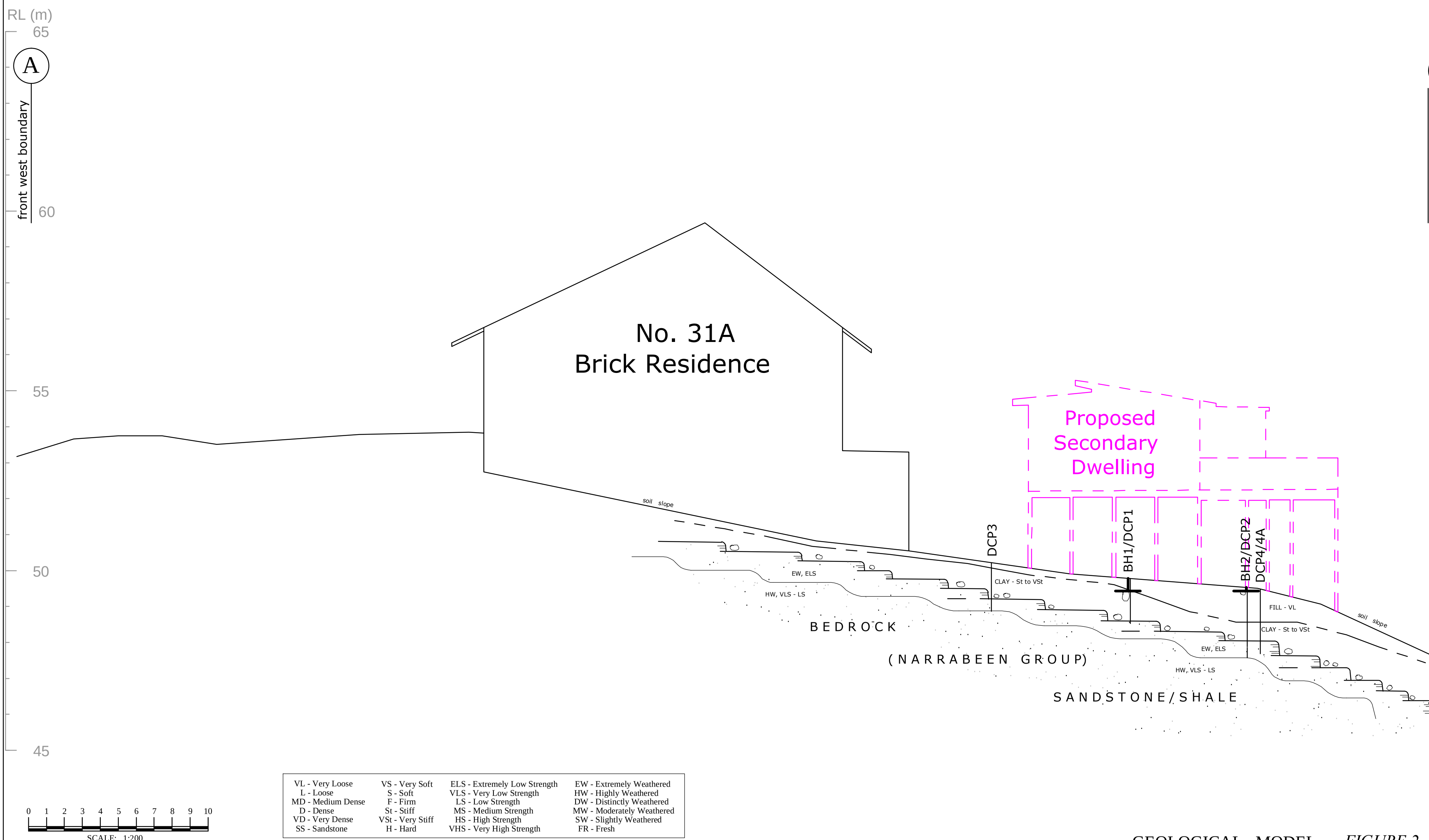
SCALE: 1:200 @ A3
DRAWING: FIG. 1
DRAWN: JB
DATE: DEC 2015

APPROVED BY: TMC

PROJECT: 2015-248

PREPARED FOR:
MR. BORIS PANOV

SITE ADDRESS:
31A QUEENS AVENUE
AVALON, NSW



VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered
SS - Sandstone	H - Hard	VHS - Very High Strength	FR - Fresh

GEOLOGICAL MODEL *FIGURE 2.*

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

SURFACE WATER FLOW
TEST LOCATIONS
WATER TABLE

SCALE: 1:200 @ A3
DRAWING: FIG. 2
DRAWN: JB
DATE: DEC 2015
APPROVED BY: TMC
PROJECT: 2015-248

PREPARED FOR:
MR. BORIS PANOV

SITE ADDRESS:
31A QUEENS AVENUE
AVALON, NSW

TEST BORE REPORT

CLIENT: Mr. Boris Panov **DATE:** 1/12/2015 **BORE No.:** 1

PROJECT: Proposed Secondary Dwelling **PROJECT No.:** 2015-248 **SHEET:** 1 of 1

LOCATION: 31A Queens Avenue, Avalon **SURFACE LEVEL:** RL å 50.34m

Depth (m)	Description of Strata	Sampling		In Situ Testing	
	PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Type	Depth (m)	Type	Results
0.00					
	GRASS				
	TOPSOIL - Medium dense, dark brown, fine grained, dry clayey sand topsoil with some organic material and ironstone gravels	D	0.20		
0.30	CLAY - Stiff, brown, low plasticity, moist clay with a trace of sand and some ironstone gravels	D	0.35		
	* 0.35m some charcoal				
0.35	HAND AUGER REFUSAL on ironstone gravels in clay				
1.00					
2.00					

RIG: None DRILLER: KB LOGGED: ER

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS: CHECKED:

Crozier Geotechnical Consultants

TEST BORE REPORT

CLIENT: Mr. Boris Panov **DATE:** 1/12/2015 **BORE No.:** 2

PROJECT: Proposed Secondary Dwelling **PROJECT No.:** 2015-248 **SHEET:** 1 of 1

LOCATION: 31A Queens Avenue, Avalon **SURFACE LEVEL:** RL å 49.18m

Depth (m)	Description of Strata	Sampling		In Situ Testing		
	PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Type	Depth (m)	Type	Results	
0.00						
	GRASS					
0.10	TOPSOIL - Medium dense, brown, fine grained, dry sand topsoil					
	HAND AUGER REFUSAL at 0.10m on sand fill					
1.00						
2.00						

RIG: None DRILLER: KB LOGGED: ER

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS: CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Mr. Boris Panov **DATE:** 1/12/2015

PROJECT: Proposed Secondary Dwelling **PROJECT No.:** 2015-248

LOCATION: 31A Queens Avenue, Avalon **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP4a			
0.00 - 0.15	6	9	6	2 (B)	2			
0.15 - 0.30	10	5	6		2			
0.30 - 0.45	6	3	7		3			
0.45 - 0.60	5	0	5		1			
0.60 - 0.75	7	0	5		2			
0.75 - 0.90	9	2	7		7			
0.90 - 1.05	11	4	9		6			
1.05 - 1.20	14	8	22		5			
1.20 - 1.35	10 (B)	6	8 (B)		5			
1.35 - 1.50	Refusal at 1.25m	7	Refusal at 1.35		7			
1.50 - 1.65		11			13			
1.65 - 1.80		19			6 (B)			
1.80 - 1.95		26			Refusal at 1.80m			
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								

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

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood	Spatial Impact	Occupancy	Evacuation	Vulnerability	Risk to Life
A	Collapse of existing timber retaining wall at the rear (<2m3)		Timber log retaining wall to approximately 1.50m height, wall is in good condition but will not remain stable over 100 year design life.	a) Failure would impact small area near base of wall in grassed terrace b) Failure would undermine small area near crest of wall	a) Person in grass terrace area approximate 1hr/day b) Persons near wall crest approx 10mins/week	a) Unlikely to not evacuate b) Unlikely to not evacuate	a) Person in open space and not buried b) Person in open space and not buried	
		a) Failure onto level grass area	Likely 0.01	0.05	0.04	0.25	0.10	**5.21E-07**
		b) Undermining area adjoining upslope	Likely 0.01	0.10	0.001	0.25	0.10	**2.48E-08**
B	Debris slide due to collapse of stone flagging retaining wall at the front (<2m3)		Old stone flagging retaining wall to approximately 1.50m height, wall is in average condition and will not remain stable over 100 year design life.	a) Failure would impact driveway edge	a) Person using driveway approximate 1hr/day	a) Possible to not evacuate	a) Person in vehicle and vehicle is damaged only	
		a) Failure onto driveway	Likely 0.01	0.05	0.04	0.50	0.30	**3.13E-06**

* likelihood of occurrence for design life of house (considered 100years)
* considered for person most at risk
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01)
* 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	Collapse of existing timber retaining wall at the rear (<2m3)	a) Failure onto level grass area	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Low
		b) Undermining area adjoining upslope	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Low
B	Debris slide due to collapse of stone flagging retaining wall at the front (<2m3)	a) Failure onto driveway	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	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%.

TABLE: 2

Recommended Maintenance and Inspection Program

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

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

CROZIER
GEOTECHNICAL CONSULTANTS

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.

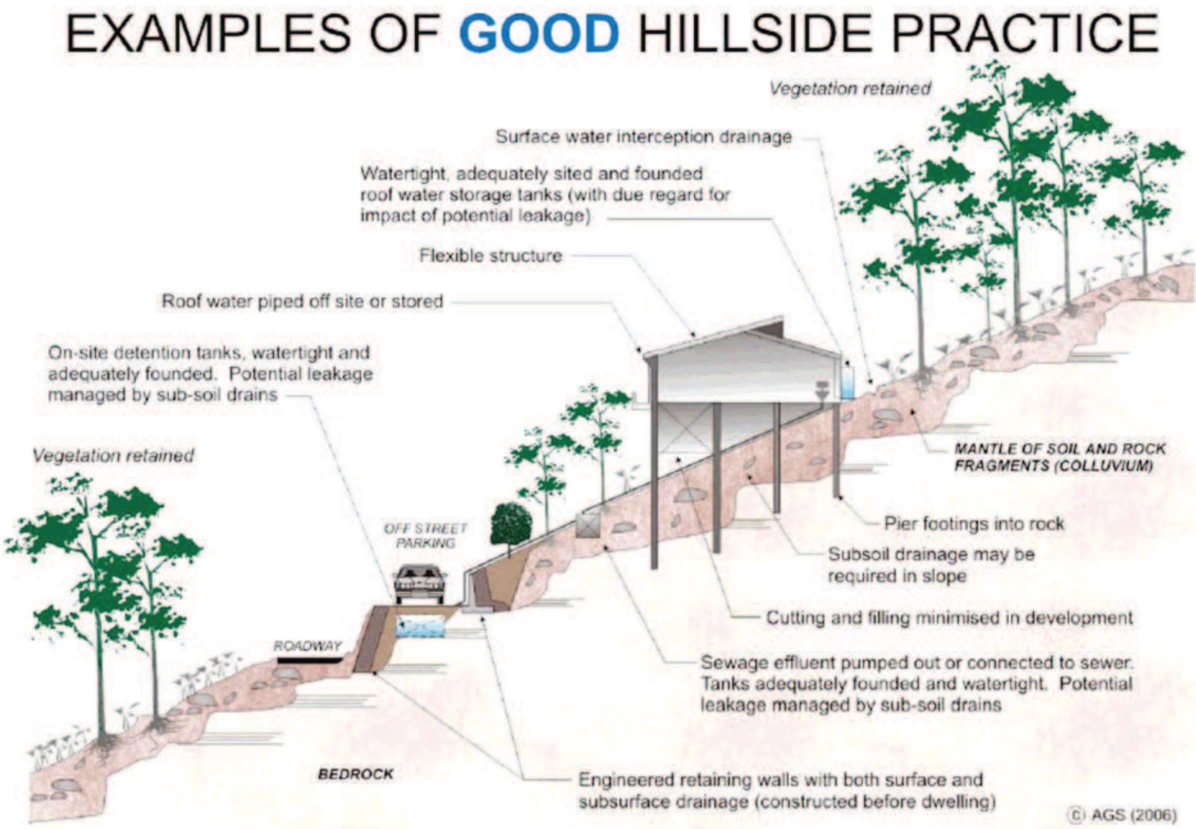
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

CROZIER
GEOTECHNICAL CONSULTANTS

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
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
OFF STREET PARKING
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
ROADWAY
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)
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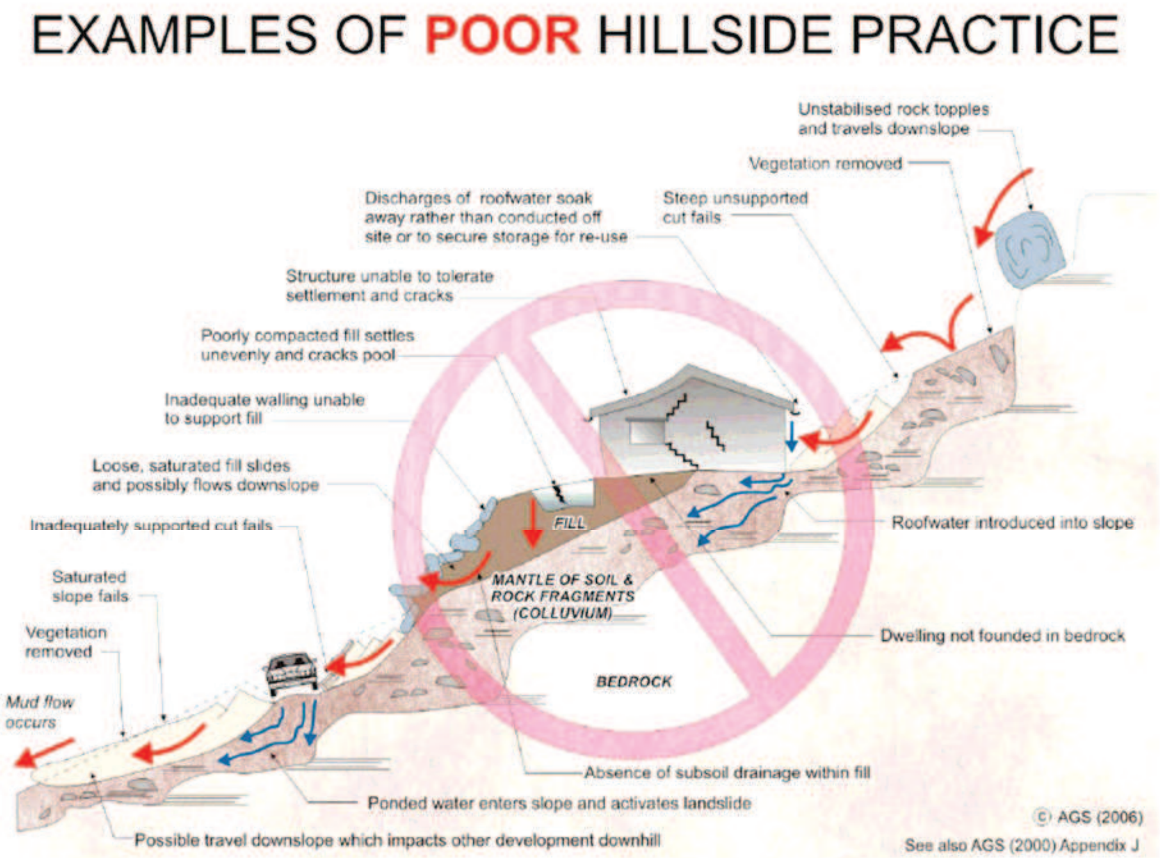
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles 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
BEDROCK
Mud flow occurs
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
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See also AGS (2000) Appendix J