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GEOTECHNICAL INVESTIGATION AND ASSESSMENT REPORT

FOR THE PROPOSED NEW DEVELOPMENT

AT

1749-1753 PITTWATER ROAD, MONA VALE, NSW

PREPARED FOR

BELLEVUE CO PTY LTD

JOB NO: 2025-136

July 2025

REVISION RECORD				
Issue No.	Date	Details of Revisions	Author(s)	Checker(s)
00	08/08/2025	Original Issue	JW	TMC

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for____________Bellevue Co Pty Ltd _____________________________

Name of Applicant

Address of site 1749-1753 Pittwater Road, Mona Vale, 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 the __05/08/2025__ certify that I am a engineering geologist as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above company to issue this document and to certify that the 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 Investigation and Assessment Report for Proposed New Development

Report Date: 08 August 2025 **Project No.:** 2025-136

Author: J. Watts and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural: Gartner Trovato Architects, Project No.: 2401, Drawing No.: A00 – A21, Dated: 25/07/2025

Geotechnical: Geotechnical Report – Crozier Geotechnical Consultants, Project No.: 2018-083, Dated: May 2018.

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___________ Bellevue Co Pty Ltd _________________
Name of Applicant
Address of site ___1749-1753 Pittwater Road, Mona Vale, 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 Investigation and Assessment Report for Proposed New Development
Report Date: 08 August 2025 **Project No.:** 2025-136
Author: J. Watts and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

■ Comprehensive site mapping conducted ___01/08/2025_________ (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 ......Preliminary conducted 20/05/2018.................
■ 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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APPENDICES

1. INTRODUCTION

In May 2018, Crozier Geotechnical Consultants undertook a Geotechnical Site Investigation (Ref: 2018-083) for the proposed development at 1753 Pittwater Road, Mona Vale. The site has since been enlarged to incorporate 1749–1753 Pittwater Road. In November 2024, EI Australia completed an additional geotechnical desktop report for DA purposes. On submission the following comments were received:

> ***Geotech***
> *Site is not mapped on Geotech Hazard area. Excavations greater than 6m are proposed for the basement car parking. Preliminary Geotech Assessment by EI Australia, E26557.G01, Dated 25th Nov 2024 is provided. In accordance with Pittwater DCP Clause B8.1, a detailed geotechnical assessment with form 1 and 1a is required and is to align with Council's Geotechnical Risk Management Policy for Pittwater.*
>
> ***Environmental Health Referral Response – Acid Sulfate Soils***
> *The site is classified as acid sulphate soils class 4 and 5.*
> *Findings by a specialist consultant have determined that the likelihood of the presence of acid sulphate soils is low, that it is acknowledged that further intrusive soil investigation would need to be conducted.*
>
> *Clause 7.1 of the Pittwater Local Environmental Plan 2014 provides that development consent must not be granted for the carrying out of the proposed works unless an Acid Sulfate Management Plan has been prepared for the proposed works in accordance with the Acid Sulfate Soils Manual and has been provided to the consent authority, and no such plan has been provided to the consent authority.*

Therefore, this assessment addresses the above-mentioned clauses and includes:

- A site walkover inspection to confirm current site conditions.
- Geotechnical Assessment (summarising previous results and new data).
- Geotechnical Risk Analysis as per Councils DA requirements
- Design and construction recommendations
- Form 1 and 1a for Council DA submission.
- Acid Sulfate Soils Management Plan (ASSMP) in accordance with the *Acid Sulfate Soils Manual* and Council requirements.

The following plans and drawings were supplied and/ or relied upon for the proposal, investigation and reporting:

- Geotechnical Report – Crozier Geotechnical Consultants, Project No.: 2018-083, Dated: May 2018.
- Geotechnical Report - EI Australia, Project No.: E26557.G01, Dated: 25/11/2024
- Architectural Drawings – Gartner Trovato Architects, Project No.: 2401, Drawing No.: A00 – A21, Dated: 25/07/2025
- Council Letter – Northern Beaches Council, Development Application No.: DA2025/0143, Dated: 30/05/2025

2. SUMMARY OF PREVIOUS INVESTIGATIONS/ DOCUMENTS

2.1. Crozier Geotechnical Consultants - Project No.: 2018-083, Dated: May 2018

A preliminary geotechnical investigation was undertaken by Crozier Geotechnical Consultants in May 2018 for a proposed four-storey shop-top residential development with basement carparking at 1753 Pittwater Road (development plans at this stage), Mona Vale. The investigation comprised one borehole using a restricted access drill rig and DCP testing, targeting assessment of subsurface conditions, depth to rock, and groundwater presence.

Subsurface conditions encountered comprised very dense granular fill overlying residual clayey to sandy clay soils to approximately 4.5 m depth, underlain by extremely weathered, extremely low strength bedrock. This transitioned to low strength siltstone at 6.6 m depth. Groundwater was not encountered to the maximum depth investigated (6.6 m; RL 1.80).

No signs of instability were noted on-site or within adjacent properties. The proposed basement excavation of up to 6.2 m depth was expected to extend to both side boundaries and near existing structures. The report concluded that suitable excavation support, such as a bored pile wall with anchors, bracing, or cantilevering, would be required to protect surrounding assets.

It was determined that the ground conditions at the base of the excavation were suitable for traditional strip/ pad or slab footings with an allowable bearing capacity of 800kPa, where higher bearing capacities were required, a piled foundation solution would be required.

While the geotechnical risks were considered ‘Unacceptable’ without proper support measures, they were assessed as reducible to ‘Acceptable’ levels with appropriately designed and constructed systems. Additional investigation, comprising at least three further locations, was recommended to better define soil and rock strength parameters for final structural design.

2.2. EI Australia, Project No.: E26557.G01, Dated: 25/11/2024

A Preliminary Geotechnical Assessment was undertaken by EI Australia in November 2024 for a proposed five-storey shop top housing development with two basement levels at 1749–1753 Pittwater Road, Mona Vale. Excavation depths of 5.3 m to 8.8 m below existing ground levels were anticipated, with the basement extending to the site boundaries.

The assessment was desktop-based and did not include any intrusive investigation. Subsurface conditions were inferred from regional data and EI’s in-house database, indicating fill and residual soils overlying weathered sandstone at variable depths (4 m–6 m). Groundwater was inferred at 3–5 m depth. The site lies within Class 4 and Class 5 acid sulfate soil risk zones.

The report recommended:

- Full-depth excavation support systems due to minimal setbacks and weak bedrock,
- Dilapidation surveys for adjacent structures within the excavation's zone of influence,
- Instrumentation and monitoring for ground movements and vibration,
- Further geotechnical investigation including at least four cored boreholes and three groundwater monitoring wells to confirm subsurface conditions,
- Preliminary allowable bearing capacity of 600 kPa on weathered sandstone, subject to confirmation.

The report provided conceptual guidance only and explicitly stated that it should not be relied upon for detailed design without further investigation.

3. THE SITE

3.1. Proposed Development

The enlarged development now comprises three (3no.) five to six storey residential and shop top residential buildings, with either two or three basement car parking levels (depending on the location of the residential building). The bulidings are separated to each other by a garden area and open space. The car parks are interconnected across the block.

The development requires excavations ranging from approximately 6.0 m on the southeastern side to 8.5 m on the northwestern side. Excavations are proposed to extend to about 0.30 m from the northwest, northeast, and southwestern boundaries. Along the southeastern boundary, excavation depths of approximately 1.60 m are anticipated directly at the site boundary, with further excavations of up to 6.0 m extending between 1.0 m and 3.0 m from this boundary. Access to Level 1 on the southwestern edge of the site will be provided via the adjoining council car park.

3.2. Site Description

The site is roughly rectangular in shape and comprises of both 1749 and 1753 Pittwater Road, Mona Vale. The site is gently (≤ 5°) southeasterly dipping and extends from the road reserve of Bungan Lane down to the road reserve of Pittwater Road.

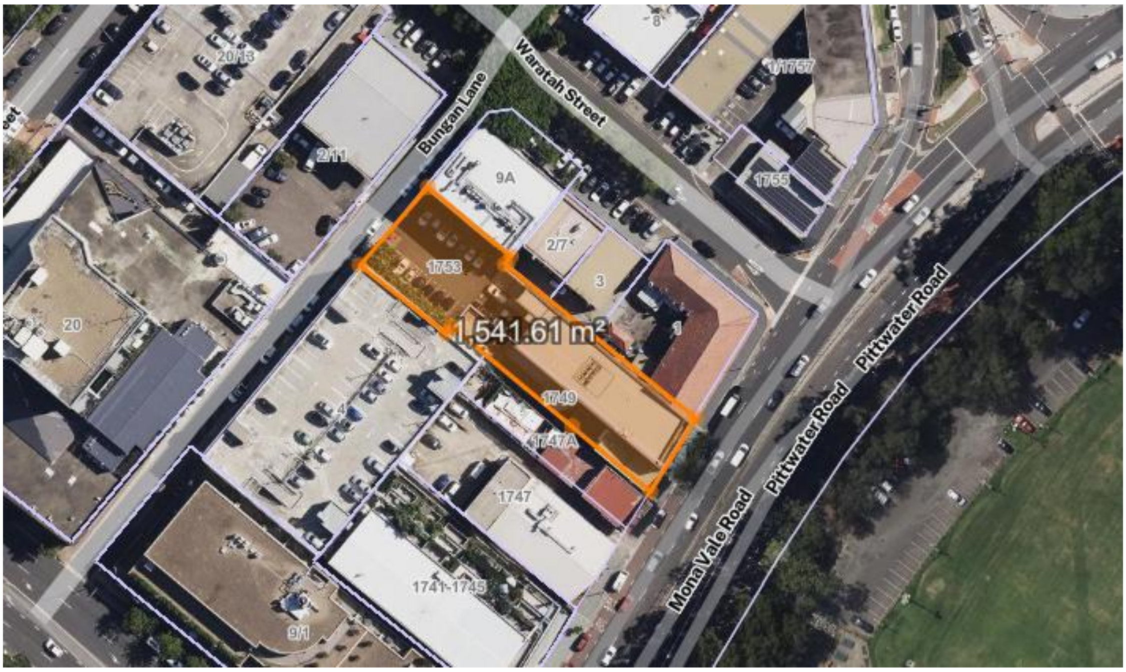

Figure 1. Site view (NSW Gov't Six Map Spatial Data).

1749 Pittwater Road is developed with a two-storey commercial building, with a concrete-covered yard located at the northern end of the lot. This area is at a similar elevation to both the adjoining Council-owned multistorey car park and the asphalt car park associated with 1753 Pittwater Road. While these surfaces are at similar levels, the concrete yard at 1749 is accessible only via the multistorey car park, with no direct vehicle access observed from Pittwater Road. The rear yard of the property was not accessible during the walkover survey; however, photographs taken over the metal fence line suggest it comprises a mixture of grass and vegetable patches. Four concrete piers, approximately 0.8 m in height, were observed protruding from the ground in this area, though their purpose is unclear. Along the southwestern side of the site, a grassed pedestrian walkway connects Pittwater Road to the concrete yard and the adjacent multistorey car park. The property (1747A) to the southwest comprises a derelict office building with an overgrown rear yard containing various metal items, resembling a salvage yard.

1753 Pittwater Road, located immediately northeast of 1749, is developed with a two-storey commercial building and an asphalt-surfaced car park in the northwestern portion of the site. The building was not accessed during the investigation but appears to be stepped slightly into the natural slope, with the rear entry approximately 1.5 m higher than the front. Although comprising two storeys, the building features tall ceiling heights, such that the top of the ground floor aligns approximately with the first floor of the adjoining building at 1749 Pittwater Road.

The eastern boundary of the site is partially adjoined by several commercial buildings, including 6/1 Waratah Street, and is separated from the rear yards of 2/7 and 3 Waratah Street by a masonry wall approximately 4 m high. The northwestern asphalt car park adjoins a three-storey office development at 9A Waratah Street and connects to the multistorey Council car park along its southwestern edge, with all surfaces formed at generally similar levels.

3.3. Geology

Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is underlain by Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones. A zone of Quaternary Alluvium (Qhf) comprising fluvially deposited quartz lithic sand, silt, gravel and clay is also shown to be approximately 80m to the southeast of the site, and due to mapping errors could extend onto the site.

Based upon our experience of the area, the Narrabeen Group bedrock has a tendency to weather to significant depths resulting in thick profiles of clayey sand to sandy clay to clay soils, which are likely to have implications on the development.

3.4. Acid Sulfate and Landslip Risk Maps (LEP 2014)

Reference to Northern Beaches (Pittwater Council's Local Environment Plan (LEP) 2014) including Landslip Risk Map (GTH_018) and Acid Sulfate Soils Map (Sheet ASS_018), the property is not categorised as subject to landslip however it is classified as both Class 4 and Class 5 Acid Sulfate Soils Hazard. The Class 4 designation requires an assessment for works more than 2 m below the natural ground surface and works by which the water table is likely to be lowered more than 2 m below the natural ground surface.

Based on the proposed development, the proposed excavation has the potential to induce landslip instability to adjacent properties therefore as per the Geotechnical Risk Management Policy for Pittwater – 2009 a detailed Geotechnical Report which meets the criteria of Paragraph 6.5 of that policy is required for the Development Application. An assessment of the water table and the potential for lowering within adjacent Acid Sulfate Soils Class 1 to 4, as well as for ASSS as per Class 4 definitions is required.

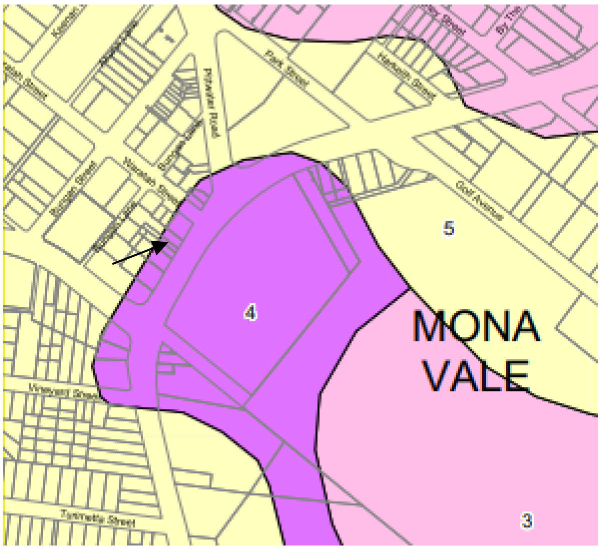

Figure 2. Acid Sulphate Soils (ASS) Map (Sheet ASS_018) of Pittwater Local Environmental Plan 2014. Black arrow indicating location of the site (Class 4 area).

4. FIELD WORK:

4.1. Methods:

A field investigation was carried out on 1 August 2025 by a Senior Engineering Geologist and comprised a walkover survey and geological mapping of the site and surrounding properties. The investigation included a photographic record of site conditions, geomorphological observations, examination of existing structures on-site, and a limited visual inspection of adjacent buildings.

The previous geotechnical investigation was undertaken on 20 May 2018, involving the drilling of one auger borehole (BH1) using a restricted access drill rig equipped with solid stem spiral flight augers and a tungsten carbide bit. This investigation was conducted by a Principal Engineering Geologist to characterise the subsurface geological conditions of the site and for feasibility/ initial design purposes.

Dynamic Cone Penetrometer testing was carried out from ground surface adjacent to the borehole and through the base of the borehole when it had progressed to 1.00m depth, in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer” to estimate near surface soil conditions.

4.2. Architectural Drawings/ Building Layout

The proposed development comprises three (3no.) five to six storey residential and shop top residential buildings, with either two or three basement car parking levels (depending on the location of the residential building). The buildings are separated to each other by a garden area and open space. The car parks are interconnected across the block.

In order to accommodate the basement car parking levels, excavations of between approximately 6.0 m (southeastern side) and 8.5 m (northwestern side) are required. The excavations are proposed to extend to approximately 0.3 m off the site boundary (in northwest, northeast and southwest) and between approximately 1.0 m to 3.0 m, away from the southeastern boundary (Pittwater Road). In order to accommodate the SW Filter Tank on the proposed basement 1 level excavations of approximately 1.6 m will be required.

Level 1 (ground floor) will be at a similar elevation to that of the surrounding site and will be connected to the adjoining southwestern multi-storey carpark.

4.3. Field Testing:

In May 2018, one (1no.) borehole (BH 1) was drilled in the lower north-east corner of the carpark where access was available. The borehole was progressed to 6.60m depth (R.L. 1.80) below surface where auger refusal was encountered on interpreted low strength siltstone bedrock. The Dynamic Cone Penetrometer (DCP) tests indicated very dense fill with the stiff clay to sandy clay becoming hard below 2.70m depth.

For a detailed profile of geological conditions, the Bore and DCP log sheets should be consulted within the previous report (Appendix 2). A summary of the geological conditions expected at the site, based on the investigation results and previous experience in the area is detailed below:

- **FILL –** bitumen pavement then very dense, gravel road base, controlled fill to 0.30m,
- **FILL –** very dense, clayey sand to silty sand fill to 0.40m,
- **Silty SAND –** very dense, dark grey, silty sand to 0.90m,
- **Sandy CLAY –** stiff to hard, yellow-brown to light grey, clayey sand to sandy clay to 4.50m,
- **Hard Sandy CLAY** – Due to updated standards, this material is classified as a hard sandy clay (though previously logged as an extremely weathered, extremely low strength, light grey sandstone). This material extended to proven depths of 6.60m,
- **SILTSTONE –** highly weathered, low strength with extremely low to medium strength bands, anticipated to encountered from 6.60m onwards.

A free standing ground water table or significant signs of water seepage were not identified within the borehole to 6.60m depth.

5. DISCUSSION:

5.1. Geotechnical Assessment:

5.1.1. General

A geotechnical investigation conducted in May 2018 identified a very dense granular fill overlying residual clay to sandy clay soils, extending to approximately 4.5 m depth. Beneath this, hard sandy clay, was encountered to 6.6 m depth, underlain by low strength siltstone of the Newport Formation. No groundwater was encountered during the investigation. However, the investigation was limited to the rear (northwest) of the site and further investigation is required to allow for design.

The site investigation and subsequent walkover survey identified no signs of previous or impending instability within the site and there were no hazards identified within the neighbouring properties that could impact the site.

5.1.2. Excavation Support

Due to spatial constraints, open-cut methods are not feasible at this site. Therefore, a piled retaining wall (contiguous to secant piled wall) is recommended to retain the encountered soil profile, potentially incorporating internal bracing. Anchored support may be limited due to space restrictions, and sheet piling is not recommended due to potential vibration-induced damage in surrounding dense urban areas.

It is assumed that the piled retaining wall will extend below the base of the excavation to provide adequate lateral support to the retained soils. Subject to confirmation through additional cored boreholes and geotechnical inspection. In areas where the piled wall does not extend below the final excavation level, the bedrock may be excavated near-vertically and remain unsupported in the short term to facilitate installation of retaining structures.

If shallow footings are proposed for the SW Filter Tank, these may impose loads on the internal retaining wall. To mitigate this, either the wall must be designed to accommodate these additional loads, or the tank footings must extend below the base of the excavation or be supported by the boundary wall system. It may be preferable to adopt a simplified design and increase the excavation size to avoid different walls influencing each other. In addition to the above, it is assumed that adjacent buildings are supported on shallow footings, therefore the retaining walls will need to be designed to resist additional surcharge loading from these structures and potential dynamic loads from passing traffic.

Excavation is expected to be undertaken with medium-sized excavators through the soil profile, it is envisaged that ripping through the bedrock siltstone bedrock will be possible. The need for full-time vibration monitoring and rock hammers and grinders/saws or should be determined via additional boreholes to confirm the strength of the underlying bedrock.

5.1.3. Foundation Recommendations

Variable ground conditions are expected at the base of the excavation, comprising both hard sandy clay and potentially low strength siltstone. Footings should be placed upon similar material and therefore will need to penetrate the stiff clay and bear on the underlying siltstone to avoid differential settlement. In absence of any geotechnical testing a conservative allowable bearing pressure of 800 kPa is considered appropriate for footings placed on low strength siltstone bedrock.

If the depth to siltstone is excessive, either deep trench footings or a piled solution may be required. Piled systems will also enable higher bearing capacities. Further cored investigation should be undertaken to confirm founding depths and provide parameters for pile design.

5.1.4. Groundwater

No groundwater was encountered to 6.6 m during the 2018 investigation, however investigation in the lower southeastern end of the site may determine a shallower groundwater table. If groundwater is not encountered during the additional investigation, seepage is still expected along bedrock discontinuities. Gravity drainage during excavation is likely to be sufficient; however, waterproofing measures (e.g. tanking) may be required to prevent long-term seepage into the structure.

As per NSW DPE Minimum Requirements for Building Site Groundwater Investigations, a formal groundwater assessment is required prior to final design, including further groundwater monitoring well installations to greater depths.

5.1.5. Acid Sulfate Soils (ASS)

According to the Pittwater LEP 2014 Acid Sulfate Soils Map (Sheet ASS_018), the site is classified as:

- Class 5 ASS (northwest)
- Class 4 ASS (southeast)

No acid sulfate soil (ASS) testing was undertaken during the 2018 investigation. In absence of such testing, an Acid Sulfate Soil Management Plan (ASSMP) is required. Details are provided in Section 6 of this report.

5.1.6. Further Work and Recommendations

The current findings are based on a single borehole and DCP testing conducted near the rear of the site due to access restraints. As such, they provide an indicative understanding of subsurface conditions only. Given the scale of the development, further subsurface investigation is required prior to construction, including:

- Additional boreholes to verify stratigraphy and subsurface variability

- Cored drilling to confirm bedrock depth and bedrock strength for pile design
- Laboratory testing for ASS characterisation
- Groundwater monitoring and assessment per DPE guidelines

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.

Nevertheless, the results of the investigation provide a reasonable basis for the Development Application analysis with further investigation recommended prior to design and construction.

5.2. Site Specific Risk Assessment:

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

A. Landslip (earth slide >20m³) from excavation sides.
B. Rockslide (<20m³) from excavation - would then cause failure of retaining wall and/or soil above and result in hazard A occurring.

A qualitative assessment of risk to life and property related to these hazards is presented in **Tables 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. Condition A relates to insufficient retention, whilst Condition B refers to engineer designed and implemented support.

The Risk to Life from Hazard A was estimated to be up to **7.50×10^{-2}** for any person in the main dwelling, while the Risk to Property was considered to be '**Very High**'. Hazard B was estimated to have a Risk to Life and Risk to Property of the same value, as the failure of the underlying bedrock would result in the failure in the overlying soil. The assessments were based on excavations with no support or planning, using ground conditions anticipated in adjacent properties. Provided the recommendations of this report are implemented, including further investigation, detailed geotechnical monitoring and the installation of engineered support systems, the likelihood of any failure becomes 'Unlikely' and as such the consequences reduce with risk becoming within 'Acceptable' levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

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5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings on bedrock at base of excavation or off bedrock surface
Maximum Allowable Bearing Capacity for Shallow Footings – though depending on the results from the additional investigation, these may be deep trench.	**Stiff Clay: 150kPa VLS Siltstone: 600kPa LS Siltstone: 800kPa
Piled Footings	To be confirmed through additional ground investigation works
Site sub-soil classification as per Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia	C_e – 'Shallow' soil site (based upon the guidance provided)
Remarks: Subject to confirmation by geotechnical professional including further investigation. Where possible, all footings should be founded off material of similar strength to prevent differential settlement. ** It is recommended that footings are extended down deeper than this stratum to avoid adversely loading the proposed 'internal' retaining wall/ prevent differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

5.3.2. Excavation:		
Depth of Excavation	Up to 8.00m depth	
Type of Material to be Excavated	Up to 4.50m	Fill and natural soils
	Between 4.50m and 6.60m	Residual soils – hard sandy clay
	From 6.60m	Siltstone bedrock – LS

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

Material	Safe Batter Slope (H:V or degree)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	45 degrees	30 degrees
Residual soil – hard sandy clay	45 degrees	36 degrees
Low to Medium strength (MS), defect free bedrock	Vertical*	Vertical*

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<table>
<tr><td colspan="3">*Subject to assessment by engineering geologist.
Remarks:
Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes or rock cuts and invoke the need to implement additional support measures.
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.</td></tr>
<tr><td rowspan="4">Equipment for Excavation</td><td>Fill/natural soils</td><td>Bucket</td></tr>
<tr><td>Residual soil – hard sandy clay</td><td>Bucket and ripper</td></tr>
<tr><td>LS</td><td>Ripping</td></tr>
<tr><td>MS-HS bedrock</td><td>Rock hammer and rock saw</td></tr>
<tr><td colspan="3">VLS – very low strength, LS – low strength, MS – medium strength, HS – high strength</td></tr>
<tr><td colspan="3">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 at least low strength silstone bedrock, to achieve a low level of vibration (5mm/s PPV) the below hammer weights and buffer distances are generally required:
<table>
<tr><td>Maximum Hammer Weight</td><td>Required Buffer Distance from Structure</td></tr>
<tr><td>300kg</td><td>2.00m</td></tr>
<tr><td>400kg</td><td>3.00m</td></tr>
<tr><td>600kg</td><td>6.00m</td></tr>
<tr><td>≥1 tonne</td><td>Up to 20.00m</td></tr>
</table>
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.</td></tr>
<tr><td>Recommended Vibration Limits
(Maximum Peak Particle Velocity (PPV))</td><td colspan="2">Neighbouring residential dwellings = 5mm/s
Services = 3mm/s,</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td colspan="2">If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed.</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td colspan="2">Pending proposed excavation equipment and vibration calibration testing results, if required.</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td colspan="2">Yes, recommended that these inspections be undertaken as per below mentioned sequence:</td></tr>
</table>

	• During the installation of excavation support systems. • At 1.50m depth intervals of unsupported excavation (if piles do not extend past the excavation depth). • At completion of the excavation. Where ground conditions are exposed that differ to those expected.
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

5.3.3. Retaining Structures:	
Required	Yes, as per pre-excavation support section.
Types	Contiguous to secant piled wall, subject to groundwater and geological condition findings.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:*	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *	Cu (kPa)	Modulus (MPa)
			Active (Ka)	At Rest (K_0)			
Fill	18	ϕ' = 28°	0.35	0.52	N/A	N/A	N/A
Natural soils	18	ϕ' = 30°	0.35	0.52	N/A	50	5
Residual soils – hard sandy clay	19	ϕ' = 34°	0.33	0.50	N/A	200	40
LS Siltstone bedrock	23	ϕ' = 38°	0.10	0.15	300kPa	N/A	200

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 back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.
Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilise active earth pressure coefficients (Ka).

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Not encountered, though minor seepage along bedding defects expected.
Excavation likely to intersect	Water Table	Unconfirmed for the southeast of the site and at full excavation depths.
	Seepage	Minor, on defects and possibly at soil/rock interface/ along defects.
Site Location and Topography		Low lying, gently sloping the southeast.
Impact of development on local hydrogeology		Appears negligible, pending further investigation.
Onsite Stormwater Disposal		Not possible via absorption.
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, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site.		

5.4. Conditions Relating to Design and Construction Monitoring:

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

A. Undertake additional ground investigation.
B. Review the structural drawings, including the retaining structure/batter slope design and construction methodology, and stormwater system plans for compliance with the recommendations of this report, this will be required for CC to meet condition 11.
C. Conduct excavation inspections as per the recommendations of Section 4.3.2 in this report
D. Inspect all new footings 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,
E. Inspect the completed development to ensure all retention and 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 requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot complete the certification if it has not been called to site to undertake the required inspections.

6. ACID SULFATE MANAGEMENT PLAN

In absence of any Acid Sulfate Laboratory Testing, a worst-case scenario has been adopted, and a management plan has been created below (utilizing values from a nearby investigation). It should be noted that based upon the previous investigation, it is assumed that no groundwater is present to the base of the excavation.

An Acid Sulfate Soils Management Plan has been prepared and is included in this letter report which we consider to meet the Acid Sulfate Soils Manual 1998, published by Acid Sulfate Soil Management Advisory Committee.

6.1. Soil Sampling and Laboratory Testing Plan

The purpose is to confirm whether Potential Acid Sulfate Soils (PASS) or actual Acid Sulfate Soils (ASS) are present within the zones of proposed excavations, to provide laboratory data to classify soils with respect to ASS risk and to determine appropriate on-site management or treatment measures (per ASSMAC / NSW guidelines).

The preliminary sampling/ testing plan is provided below:

- Boreholes – Drill a series of boreholes across the site with a minimum of one in each corner and one at the centre, using a drilling rig.
- Sampling – Collect small, disturbed samples at 0.5 m intervals from all exploratory holes. Samples will be stored in glass jars and placed in an ice-filled container to maintain temperature.
- In-situ Testing – Conduct field tests to determine pH, pHFox, and assess reaction rate to hydrogen peroxide solution, recording all results and observations.
- Chromium Reducible Sulfur (CRS) Testing – Submit selected samples to a NATA-accredited laboratory for analysis using the Chromium method, in accordance with the Acid Sulphate Soils Laboratory Methods Guidelines (Version 2.1, June 2004) and the National Acid Sulphate Soils Guidance (June 2018).
- Groundwater Monitoring – If the water table is encountered within the planned disturbance depth, install a groundwater monitoring well and collect samples for laboratory analysis.

6.2. Soil Neutralisation:

Where the disturbance of the ASS is unavoidable, neutralisation of the excavated soils with Calcium Carbonate (CaCO3) in the form of finely crushed limestone or 'Aglime' is required. The volume of lime required is calculated based on the acidity of the soil and its total oxidisable sulphur content along with the neutralising value (NV) of the agent and volume of soil disturbed. (Tabled 5.1, 6.1 and 6.2 in ASS Manual 1998). Neutralising material should be applied to counteract the ASS and PASS at a 'safety factor' of 1.5 to 2.0.

A staged treatment plan is provided below for use on all ASS soils excavated on this site. It is recommended that experienced ASS contractors be engaged to undertake all management of ASS on this site.

1. A bunded area of sufficient size to hold and treat all excavated soil to be treated will be required. This area needs to be lined with two layers of plastic sheeting to ensure no leakage at overlaps. Hay bales should be provided around the bunded area with the plastic extended over the hay bales to create a sealed containment zone. An alternative could be a sealed skip bin or similar with plastic sheet lining to ensure no escape of seepage waters. A low point should be created to one side of the bunded area for collection of seepage water that drains from the soils. This water will also require treatment therefore it will need to be retained. Plastic sheeting should also be used to cover the treatment area following placement of the soils to ensure no additional water enters during rainfall events.

2. The soils should then be treated with natural lime via mechanical mixing at regular intervals during excavation. Based on the results of a nearby investigation, it is considered that a value of 2 kg of lime per tonne of soil to be treated will be required. This value will be confirmed during the subsequent ground investigation and laboratory testing. If during pile drilling excavation the mixing of the non-acid sulphate soils then ASS soils occurs then this may result in a lower value of lime being suitable. However, this would need to be confirmed via onsite testing during the excavation and pier drilling process.

3. Testing of several samples of the mixed and treated soils, along with the separate drainage water (if significate seepage encountered), must be undertaken at approximately 3 day intervals after excavation to assess the treatment effectiveness. This will determine if the treatment is working and any required modifications to the plan. The field testing must continue until the treated soils can be determined as neutral (pH ≥ 6 and ≤ 8) at which time they may be classified as General Solid Waste and used as fill onsite or disposed off site. Additional lime should be stored on site with access to appropriate application equipment that can be engaged for immediate use where monitoring results determine failure to achieve soil neutralization. Where further increasing the liming value fails, the project should cease to operate and contingency plans implemented.

7. CONCLUSION:

The proposed development comprises three five- to six-storey shop-top housing buildings over two to three basement levels. Excavation is expected to extend approximately 6.0 m (southeast) to 8.5 m (northwest) below existing ground levels, with minimal setbacks of approximately 0.3 m along the northwest, northeast, and southwest boundaries, and 1.0 m to 3.0 m along the southeastern boundary. Additional excavation of approximately 1.6 m depth is also required along the southeastern boundary to facilitate installation of the stormwater (SW) Filter Tank.

A previous geotechnical investigation (May 2018) identified a very dense granular fill overlying natural clay to sandy clay soils to depths of approximately 4.5 m. These materials overlie extremely weathered, extremely low strength bedrock, recovered as clayey sand or sandy clay, to approximately 6.6 m depth. For the purposes of this report, and based on updated classification standards, this material is described as a hard sandy clay. Beneath this, low strength siltstone of the Newport Formation was encountered.

Due to the excavation depth and proximity to site boundaries and adjacent infrastructure, engineered excavation support will be required. A bored contiguous/secant pile wall is considered appropriate and will likely require lateral restraint (e.g. anchors or temporary bracing). Where the siltstone bedrock is of at least low strength, temporary vertical unsupported excavation may be feasible (subject to inspection); however, to adequately retain overlying soil materials, it is anticipated that the piled walls will extend into the siltstone bedrock and therefore will be supported.

Depending on the strength of the siltstone, removal of this material via ripping may be possible, where stronger bedrock is encountered appropriate vibration control measures will be required during rock excavation, including the use of low-energy plant and real-time vibration monitoring, to reduce risk of cosmetic or structural damage and address stakeholder concerns. Geotechnical supervision during the installation of excavation support and during the excavations is required to verify in-situ conditions and guide adjustments to excavation and support methods.

Variable ground conditions are expected at the excavation base, these are anticipated to be hard sandy clay and low strength siltstone. Building footings should be found upon the same material and therefore will be required to penetrate the hard clays and bear directly on the underlying siltstone to mitigate risks of differential settlement. An allowable bearing pressure of 800 kPa is recommended for footings bearing on the low strength siltstone, though further ground investigation will confirm the suitability of this value.

If the depth to suitable siltstone is excessive, deep trench footings or a piled foundation solution may be required. Piles can also provide higher allowable capacities where needed. Further cored boreholes are recommended to confirm bedrock depth and strength for detailed pile design.

The following additional geotechnical investigations should be undertaken prior to detailed design and construction:

- Boreholes to confirm soil and rock stratigraphy and assess spatial variability;
- Cored drilling to define depth and strength of siltstone bedrock;
- Laboratory testing for acid sulfate soil (ASS) classification;
- Groundwater monitoring and assessment in accordance with current DPE guidelines.

If appropriate support measures are not implemented, the development poses Unacceptable risk to surrounding structures and services. However, with properly designed and constructed geotechnical and structural support systems, these risks can be mitigated and reduced to Acceptable levels for a design life of 100 years.

Given that similar deep excavations have been successfully undertaken in the surrounding area under comparable geological conditions, the proposed development is considered geotechnically feasible and suitable for the site, subject to the implementation of the recommendations contained within this report.

Prepared By:	Reviewed By:
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	MAIG, PRGeo – Geotechnical and Engineering
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APPENDIX 1. NOTES RELATING TO THIS REPORT

CROZIER
GEOTECHNICAL CONSULTANTS

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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: -

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.

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.

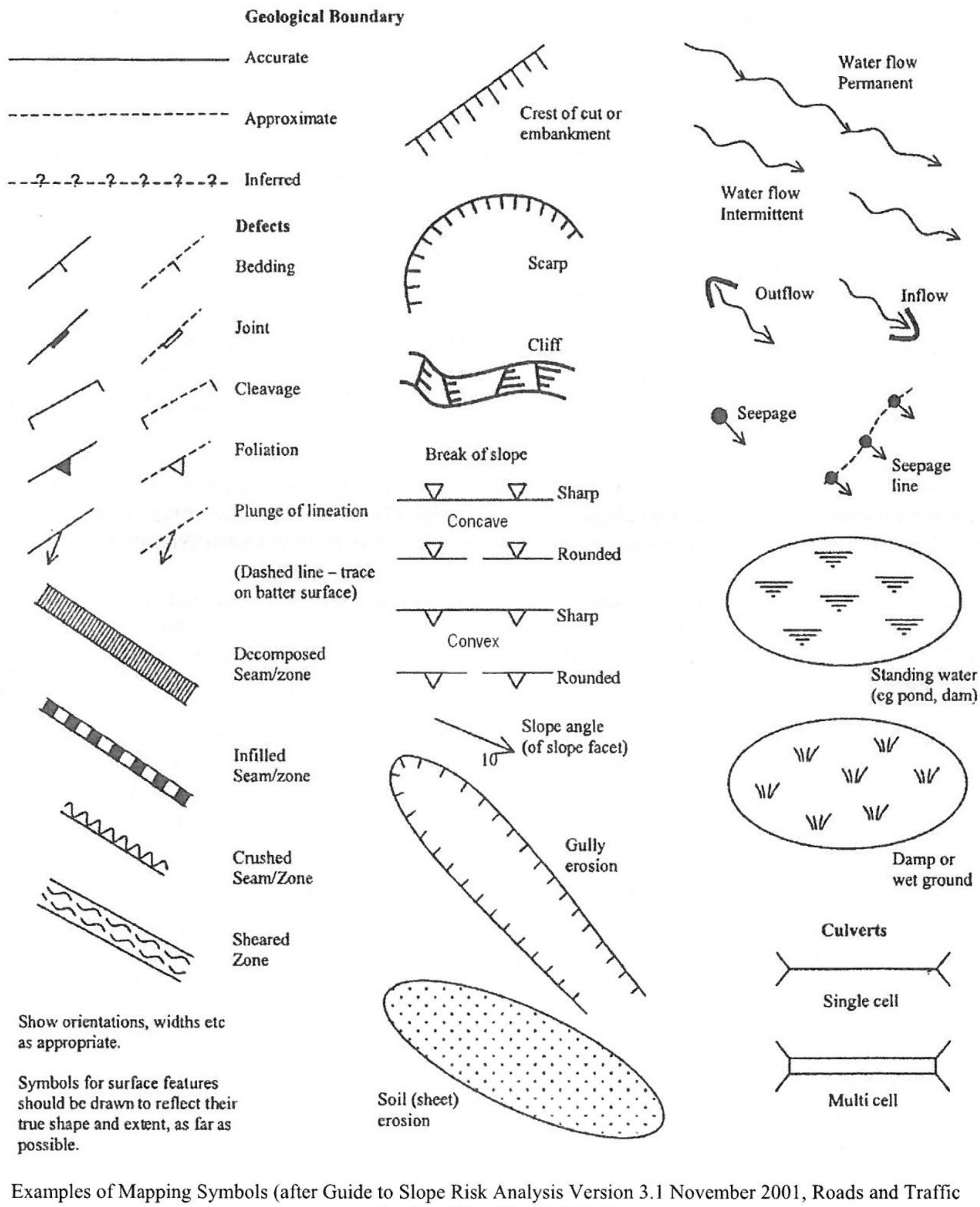
Reproduction of Information for Contractual Purposes

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.

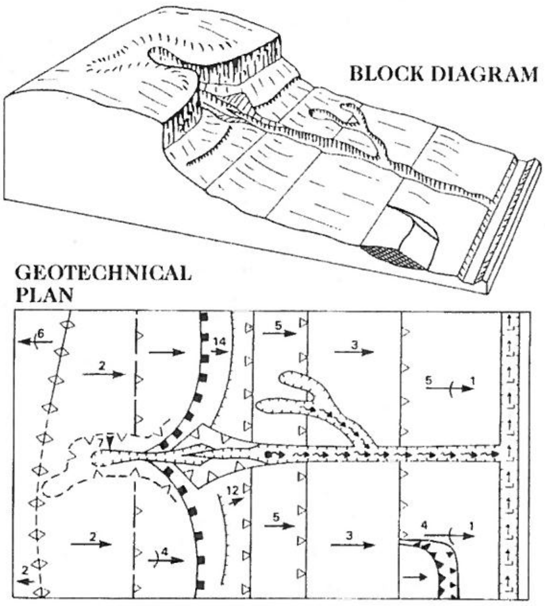
Site Inspection

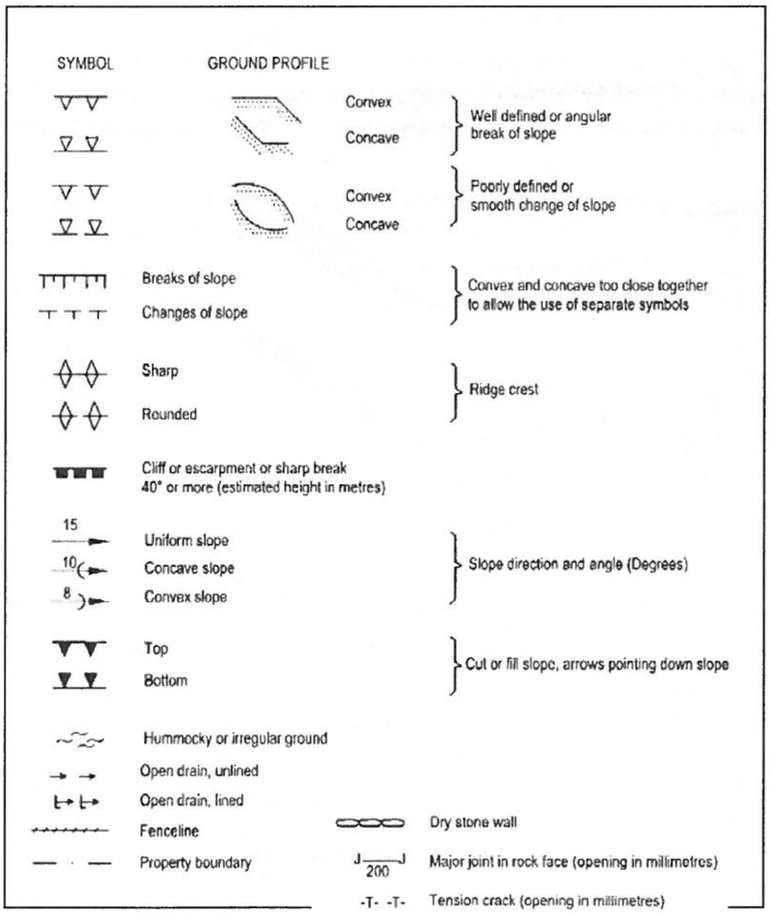
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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. RISK TABLES

TABLE : A - Part 1

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 (earth slide >20m³) from excavation sides		Fill and then residual clayey soils to excavation base **Condition A:** Insufficient retention - Almost Certain **Condition B:** Engineer designed and implemented suport - Rare		a) Pavement within 1.50m of excavation, impact full width b) footpath within 0.30m of excavation, impact full width c) building within 0.30m of excavation, impact 75% structure d) building within 0.30m of excavation, impact 30% of structure e) building within 0.30m of excavation, impact 100% of building. f) Pavement within 5m of excavation, impact 10% of width g) Footpath within 1.2m of excavation, impact full width h) Building within 0.30m of excavation, impact 40% i) Yard within 0.30m excavation, impact 60% j) Building within approximately 5m of excavation, impact 10 % k) Yard within 0.30m of excavation, impact 100% l) Building within approximately 5m of excavation, impact 10 % m) Yard within 0.30m of excavation, impact 100% n) Site structures, 100%, up to 8m excavations		a) 1000 vehicles per day, 20km/hr, . b) Persons on footpath 12hrs per day. c) Person in building 8 hrs per day avge. d) Person in carpark, 12 hrs per day. e) Person in building 8 hrs per day avge f) 10000 vehicles per day, 30km/hr, avg g) Persons on footpath 12hrs per day h) Person in building 8hrs/day average i) Person in yard 8hrs/ day avg. j) Person in building 8hrs/day average k) Person in yard 8hrs/ day avg. l) Person in building 8hrs/day average m) Person in yard 8hrs/ day avg. n) Person in buildings 24hrs per day.	a) Likely to not evacuate b) Unlikely to not evacuate c) Likely to not evacuate d) Likely to not evacuate e) Likely to not evacuate f) Unlikely to not evacuate g) Likely to not evacuate h) Likely to not evacuate i) Unlikely to not evacuate j) Likely to not evacuate k) Unlikely to not evacuate l) Likely to not evacuate m) Unlikely to not evacuate n) Likely to not evacuate	a) Person in vehicle, potentially buried b) Person in open space,very unlikley to be buried. c) Person in building, potentially buried d) Person in building, potentially buried e) Person in bulding, potentially buried f) Person in vehicle unlikely to be buried g) Person in open space,very unlikley to be buried. h) Person in bulding, potentially buried i) Person in open space,very unlikley to be buried. j) Person in building, damage only k) Person in open space,very unlikley to be buried. l) Person in building, damage only m) Person in open space,very unlikley to be buried.	Condition A	Condition B
			Almost Certain	**Rare**	**Prob. of Impact**	**Impacted**					
		a) Bungan Lane Pavement (northwest)	0.1	0.00001	1.00	1.00	0.010	0.75	1.0	**7.81E-04**	**7.81E-08**
		b) Bungan Lane Footpath	0.1	0.00001	1.00	1.00	0.500	0.25	1.0	**1.25E-02**	**1.25E-06**
		c) Building No. 9A Waratah	0.1	0.00001	1.00	0.75	0.333	0.75	1.0	**1.88E-02**	**1.88E-06**
		d) Council carpark building	0.1	0.00001	1.00	0.30	0.500	0.75	1.0	**1.13E-02**	**1.13E-06**
		e) 1747A Pittwater Road Building/ Storage yard	0.1	0.00001	0.75	1.00	0.333	0.75	1.0	**1.88E-02**	**1.88E-06**
		f) Pittwater Road Pavement	0.1	0.00001	0.75	0.10	0.069	0.25	0.9	**1.17E-04**	**1.17E-08**
		g) Pittwater Road Footpath	0.1	0.00001	1.00	1.00	0.500	0.75	1.0	**3.75E-02**	**3.75E-06**
		h) 6/1 Waratah Road building	0.1	0.00001	1.00	0.40	0.333	0.75	1.0	**1.00E-02**	**1.00E-06**
		i) 6/1 Waratah Road yard	0.1	0.00001	1.00	0.60	0.333	0.25	1.0	**5.00E-03**	**5.00E-07**
		j) 3 Waratah Road building	0.1	0.00001	0.50	0.10	0.333	0.75	1.0	**1.25E-03**	**1.25E-07**
		k) 3 Waratah Road yard	0.1	0.00001	1.00	1.00	0.333	0.25	1.0	**8.33E-03**	**8.33E-07**
		l) 7 Waratah Road building	0.1	0.00001	0.50	0.10	0.333	0.75	1.0	**1.25E-03**	**1.25E-07**
		m) 7 Waratah Road yard	0.1	0.00001	1.00	1.00	0.333	0.25	1.0	**8.33E-03**	**8.33E-07**
		n) The development	0.1	0.00001	1.00	1.00	1.000	0.75	1.0	**7.50E-02**	**7.50E-06**

* hazards considered in for unsuitable/insufficient excavation support measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* considered for 1 person only, where multiple persons occupy locatoin at any one time risk levels increase accordingly
* where vehicles/persons travel past site, considered for slide impact during travel
* considered for adjacent premises/buildings founded via 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 knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : A - Part 2

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide		Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life	
B	Rockslide (<20m³) from excavation - would then cause failure of retaining wall and/or soil above and result in hazard A occuring.		Fill and then residual clayey soils to excavation base **Condition A:** Insufficient retention - Almost Certain **Condition B:** Engineer designed and implemented suport - Rare		a) Pavement within 1.50m of excavation, impact full width b) footpath within 0.30m of excavation, impact full width c) building within 0.30m of excavation, impact 75% structure d) building within 0.30m of excavation, impact 30% of structure e) building within 0.30m of excavation, impact 100% of building. f) Pavement within 5m of excavation, impact 10% of width g) Footpath within 1.2m of excavation, impact full width h) Building within 0.30m of excavation, impact 40% i) Yard within 0.30m excavation, impact 60% j) Building within approximately 5m of excavation, impact 10 % k) Yard within 0.30m of excavation, impact 100% l) Building within approximately 5m of excavation, impact 10 % m) Yard within 0.30m of excavation, impact 100% n) Site structures, 100%, up to 8m excavations		a) 1000 vehicles per day, 20km/hr, . b) Persons on footpath 12hrs per day. c) Person in building 8 hrs per day avge. d) Person in carpark, 12 hrs per day. e) Person in building 8 hrs per day avge f) 10000 vehicles per day, 30km/hr, avg g) Persons on footpath 12hrs per day h) Person in building 8hrs/day average i) Person in yard 8hrs/ day avg. j) Person in building 8hrs/day average k) Person in yard 8hrs/ day avg. l) Person in building 8hrs/day average m) Person in yard 8hrs/ day avg. n) Person in buildings 24hrs per day.	a) Likely to not evacuate b) Unlikely to not evacuate c) Likely to not evacuate d) Likely to not evacuate e) Likely to not evacuate f) Unlikely to not evacuate g) Likely to not evacuate h) Likely to not evacuate i) Unlikely to not evacuate j) Likely to not evacuate k) Unlikely to not evacuate l) Likely to not evacuate m) Unlikely to not evacuate n) Likely to not evacuate	a) Person in vehicle, potentially buried b) Person in open space,very unlikley to be buried. c) Person in building, potentially buried d) Person in building, potentially buried e) Person in bulding, potentially buried f) Person in vehicle unlikely to be buried g) Person in open space,very unlikley to be buried. h) Person in bulding, potentially buried i) Person in open space,very unlikley to be buried. j) Person in building, damage only k) Person in open space,very unlikley to be buried. l) Person in building, damage only m) Person in open space,very unlikley to be buried.	Condition A	Condition B
			Almost Certain	**Rare**	**Prob. of Impact**	**Impacted**					
		a) Bungan Lane Pavement (northwest)	0.001	0.00001	1.00	1.00	0.010	0.75	1.0	**7.81E-06**	**7.81E-08**
		b) Bungan Lane Footpath	0.001	0.00001	1.00	1.00	0.500	0.25	1.0	**1.25E-04**	**1.25E-06**
		c) Building No. 9A Waratah	0.001	0.00001	1.00	0.75	0.333	0.75	1.0	**1.88E-04**	**1.88E-06**
		d) Council carpark building	0.001	0.00001	1.00	0.30	0.500	0.75	1.0	**1.13E-04**	**1.13E-06**
		e) 1747A Pittwater Road Building/ Storage yard	0.001	0.00001	0.75	1.00	0.333	0.75	1.0	**1.88E-04**	**1.88E-06**
		f) Pittwater Road Pavement	0.001	0.00001	0.75	0.10	0.069	0.25	0.9	**1.17E-06**	**1.17E-08**
		g) Pittwater Road Footpath	0.001	0.00001	1.00	1.00	0.500	0.75	1.0	**3.75E-04**	**3.75E-06**
		h) 6/1 Waratah Road building	0.001	0.00001	1.00	0.40	0.333	0.75	1.0	**1.00E-04**	**1.00E-06**
		i) 6/1 Waratah Road yard	0.001	0.00001	1.00	0.60	0.333	0.25	1.0	**5.00E-05**	**5.00E-07**
		j) 3 Waratah Road building	0.001	0.00001	0.50	0.10	0.333	0.75	1.0	**1.25E-05**	**1.25E-07**
		k) 3 Waratah Road yard	0.001	0.00001	1.00	1.00	0.333	0.25	1.0	**8.33E-05**	**8.33E-07**
		l) 7 Waratah Road building	0.001	0.00001	0.50	0.10	0.333	0.75	1.0	**1.25E-05**	**1.25E-07**
		m) 7 Waratah Road yard	0.001	0.00001	1.00	1.00	0.333	0.25	1.0	**8.33E-05**	**8.33E-07**
		n) The development	0.001	0.00001	1.00	1.00	1.000	0.75	1.0	**7.50E-04**	**7.50E-06**

* hazards considered in for unsuitable/insufficient excavation support measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* considered for 1 person only, where multiple persons occupy locatoin at any one time risk levels increase accordingly
* where vehicles/persons travel past site, considered for slide impact during travel
* considered for adjacent premises/buildings founded via 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 knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A & B	Landslip (earth slide >20m³) from excavation & Rockslide (<20m³) from excavation - would then cause failure of retaining wall and/or soil above and result in hazard A occuring	a) Bungan Lane Pavement (northwest)	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		b) Bungan Lane Footpath	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		c) Building No. 9A Waratah	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		d) Council carpark building	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		e) 1747A Pittwater Road Building/ Storage yard	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		f) Pittwater Road Pavement	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Very High
		g) Pittwater Road Footpath	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		h) 6/1 Waratah Road building	Almost Certain	Event is expected to occur over design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Very High
		i) 6/1 Waratah Road yard	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		j) 3 Waratah Road building	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Very High
		k) 3 Waratah Road yard	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		l) 7 Waratah Road building	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Very High
		m) 7 Waratah Road yard	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		n) The development	Almost Certain	Event is expected to occur over design life.	Catastrophic	Site structures completely destroyed, significant stabilising **or** MAJOR damage to neighbouring property.	Very High

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

APPENDIX 3. AGS TERMS AND DESCRIPTIONS

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}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

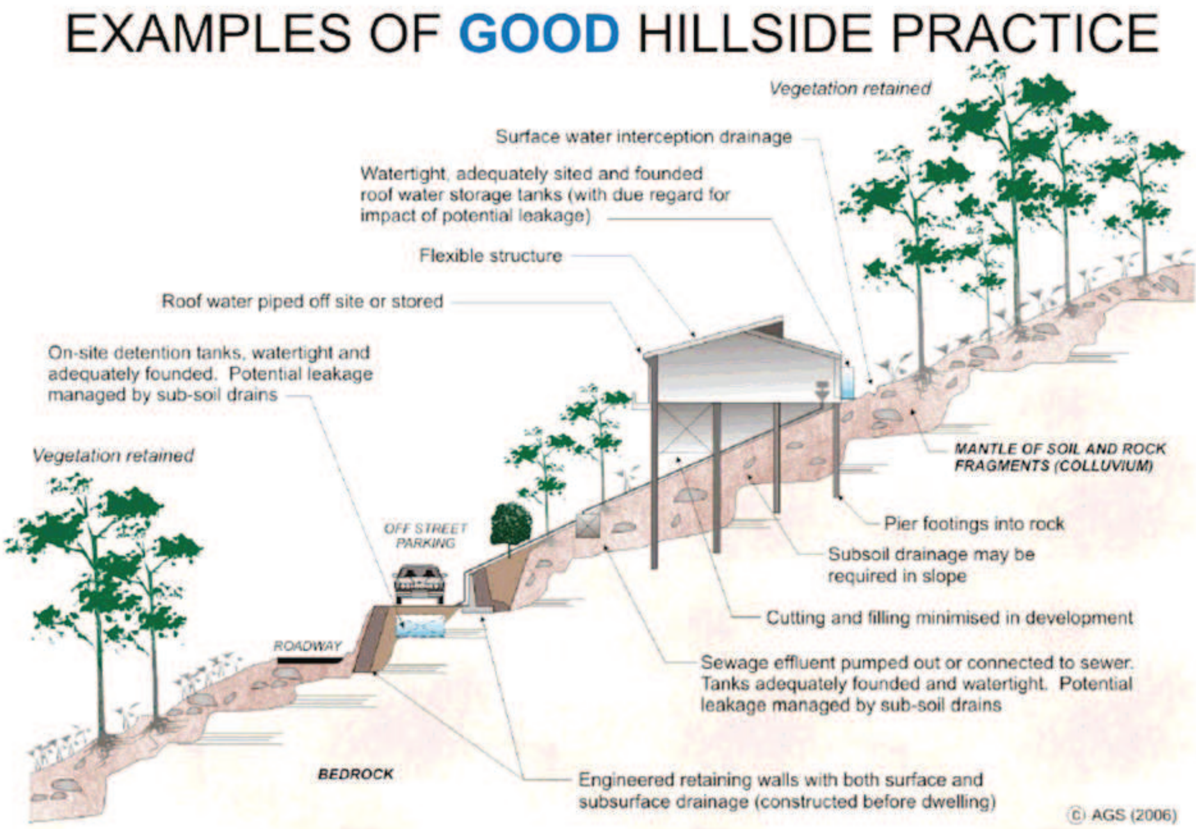
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

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

APPENDIX 4. HILLSIDE CONSTRUCTION DETAILS

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
ROADWAY
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)
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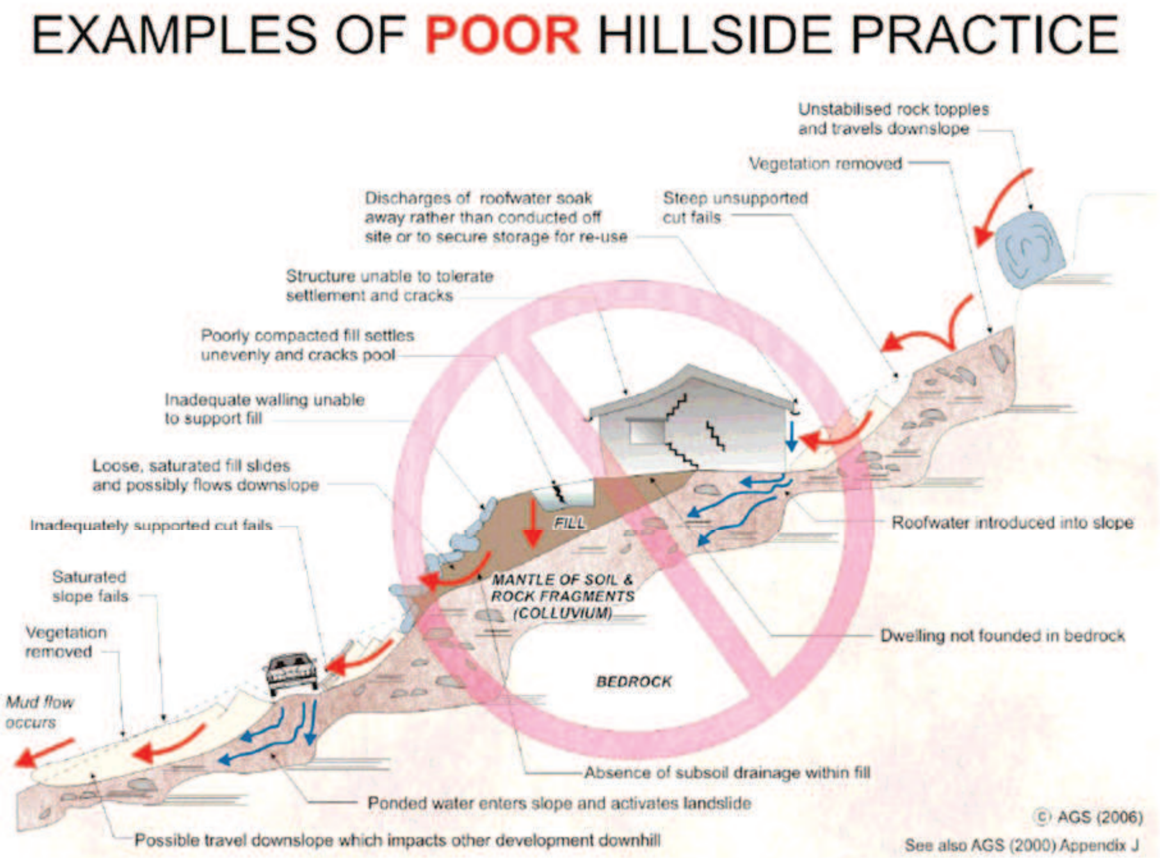
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles 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
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
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See also AGS (2000) Appendix J