

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

Development Application for _____

Name of Applicant

Address of site 121 Pacific Road, Palm Beach

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

I, _____ Woodie Theunissen _____ on behalf of JK Geotechnics Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 28 October 2021 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least \$2million.
we/I have:

Please mark appropriate box

- ☐ 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
- ☒ I- Are/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
- ☒ We 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. We 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 are/am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my/our report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.
- ☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Assessment

Report Date: 28 October 2021

Report Ref No: 34390YTrpt

Author: Arthur Billingham

Author's Company/Organisation: JK Geotechnics Pty Ltd

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

Drawings prepared by Daniel Boddam (Project No: 0267. Drawing No's: DA000, DA001, DA100 to DA105, DA200, DA201, DA300, DA301, DA400 to DA403, Rev 4, 1, 6, 6, 6, 4, 4, 2, 6, 6, 6, 4, 4, 4, 4 respectively, dated 21 October 2021)

I am We are 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 confirming 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, as discussed in the Report.

Signature



NameWoodie Theunissen.....

Chartered Professional Status.....CPEng.....

Membership No.889807.....

Company: JK Geotechnics Pty Ltd.

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for _____

Name of Applicant

Address of site 121 Pacific Road, Palm beach, Palm Beach _____

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 Assessment
 Report Date: 28 October 2021
 Author: Arthur Billingham
 Author's Company/Organisation: JK Geotechnics Pty Ltd

Report Ref No: 34390YTrpt

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 1 September 2021 _____
 (date)
- ☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:250 (as appropriate)
- ☒ Subsurface investigation required
- ☒ No Justification ...Bedrock exposed across site and adjoining properties
☐ Yes Date conducted
- ☒ 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 *recommendations presented in the Report are adopted.*
- ☒ 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 We are aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring confirming 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 as discussed in the Report.

Signature ... 

Name ...Woodie Theunissen.....

Chartered Professional Status.....CPEng.....

Membership No.889807.....

Company JK Geotechnics Pty Ltd.



**REPORT TO
KEDE CARBONI**

**ON
GEOTECHNICAL ASSESSMENT
(In Accordance with Pittwater Council Risk
Management Policy)**

**FOR
PROPOSED RESIDENCE**

**AT
121 PACIFIC ROAD, PALM BEACH, NSW**

Date: 28 October 2021

Ref: 34390YTrpt

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801





Report prepared by:

Arthur Billingham
Senior Geotechnical Engineer



Report reviewed by:

Woodie Theunissen
Principal Associate | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
34390YTrpt	Final Report	28 October 2021

© Document copyright of JK Geotechnics

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:

- JKG's proposal in respect of the work covered by the Report;
- The limitations defined in the Client's brief to JKG;
- The terms of contract between JKG and the Client, including terms limiting the liability of JKG.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

At the Company's discretion, JKG may send a paper copy of this report for confirmation. In the event of any discrepancy between paper and electronic versions, the paper version is to take precedence. The USER shall ascertain the accuracy and the suitability of this information for the purpose intended; reasonable effort is made at the time of assembling this information to ensure its integrity. The recipient is not authorised to modify the content of the information supplied without the prior written consent of JKG.



Table of Contents

1	INTRODUCTION	1
2	ASSESSMENT METHODOLOGY	1
3	SUMMARY OF OBSERVATIONS	2
4	SUBSURFACE CONDITIONS	5
5	GEOTECHNICAL ASSESSMENT	6
5.1	Potential Landslide Hazards	6
5.2	Risk Analysis	6
5.3	Risk Assessment	7
6	COMMENTS AND RECOMMENDATIONS	8
6.1	Conditions Recommended to Establish the Design Parameters	8
6.2	Conditions Recommended to the Detailed Design to be Undertaken for the Construction Certificate	10
6.3	Conditions Recommended During the Construction Period	10
6.4	Conditions Recommended for Ongoing Management of the Site/Structures	11
7	GENERAL COMMENTS	12

ATTACHMENTS

Table A: Summary of Risk Assessment to Property

Table B: summary of Risk Assessment to Life

Figure 1: Site Location Plan

Figure 2: Geotechnical Site Plan

Figure 3: Section A-A Looking North North-West

Figure 4: Section B-B Looking North North-West

Figure 5: Section C-C Looking West

Figure 6: Section D-D Looking West

Figure 7: Geotechnical Mapping Symbols

Appendix A: Landslide Risk Management Terminology

Appendix B: Some Guidelines For Hillside Construction

Vibration Emission Design Goals

Report Explanation Notes

1 INTRODUCTION

This report presents the results of our geotechnical assessment of the site at 121 Pacific Road, Palm Beach, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by Mr Kede Carboni and was completed in accordance with our proposal (Ref P54832YTRev1) dated 24 August 2021. The site was inspected by our Senior Geotechnical Engineer, Mr Arthur Billingham on 1 September 2021, in order to assess the existing stability of the site and the potential effect on stability of the proposed development.

From reference to the drawings prepared by Daniel Boddam Architecture & Interior Design (Project No. 0267, Drawing Nos. DA100 to DA105, DA200, DA201, DA301, DA400 to DA403, Revisions 2, 4 and 6 and 5, dated 21 October 2021) it is understood that following demolition of the existing structures on site the development will comprise:

- Construction of a two-storey residence at the western end of the site that will have a lower ground floor finished level of RL73.35m which will require excavation to maximum depths of about 3.2m. the maximum excavation depth will be along the western edge building footprint and will reduce towards the east with the eastern half of the lower ground floor elevated above existing site levels.
- Extending out from the lower ground floor level at the rear of the residence is a proposed in-ground pool and cabana with the pool coping and floor levels at RL73.35m. Excavation for the pool is anticipated to maximum depths of about 2m below existing surface levels.
- From the pool and rear of the residence, levels step down to a studio and carport that have finished floor levels at RL70.25m and RL70.4m respectively. The studio and carport will generally be suspended above the existing slope, however excavation to a maximum depth of 3m will be required along the southern edge of the carport footprint for the driveway and proposed OSD tank
- Along the southern boundary a driveway is proposed to provide access from Pacific Road to the carport at the rear of the development area. Levels along the driveway have not been confirmed, however the overall relief between Pacific Road and the carport is 9m and therefore some excavation is anticipated to be required to achieve the design profile.
- From the eastern end of the development a footpath is proposed to extend towards the north-eastern corner of the site to join an existing path which provides pedestrian access to Florida Road.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 5 below. It is understood that the report will be submitted to Council as part of the DA documentation. Our report is preceded by the completed Council Forms 1 and 1a.

2 ASSESSMENT METHODOLOGY

This stability assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk

assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 6 following our geotechnical assessment.

The attached Figure 2 presents a geotechnical sketch plan showing the principal geotechnical features present at the site. Figure 2 is based on the survey plan prepared by Mepstead & Associates (Ref No. 4590, Revision C dated 4 October 2019). Additional features on Figure 2 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 3 presents a typical cross-section through the site based on the survey data augmented by our observations.

3 SUMMARY OF OBSERVATIONS

The following observations should be read with reference to the attached Figure 2.

The site is located on a ridgeline that runs in a north-south direction along Barrenjoey Peninsula and has been shaped by the down-cutting of the sandstone plateau during the formation of Pittwater. The site is located on the eastern side of the ridgeline. Site slopes down diagonally across the site from the crest and Pacific Road. The upper (western) half of the site is more gently sloping than the lower (eastern) half and drops down to the north-east at approximately 7°. The eastern half of the property drops down more steeply to the north and north-east through a sandstone cliff line with an average slope of about 30° to 35°.

The property comprises a large lot with dimensions of approximately 29.5m x 144m. The site can be divided into two portions, the developed upper or western portion of the site and the undeveloped lower or eastern portion of the site.

A one- and two-storey weatherboard house that is supported on sandstone block strip and pier footings is located in the developed western half of the site. Surrounding the house are lawns and garden beds with medium to large trees interspersed across the site. At the rear of the house a relatively level terrace has been formed by a sandstone boulder wall which ranges in height from about 1.2m to 1.9m. The wall is laidback at approximately 20° and appears to be showing no signs of distress.

Underneath and around the western and southern sides of the existing house sandstone bedrock is exposed. Below the single storey western portion of the existing house, sandstone bedrock is exposed and the house in this area is supported on sandstone piers founded on the surface of the bedrock (see Plate 1 below). The surface of the bedrock dips down to the north-east below the house and, where the house is two-storeys it appears to have been cut to form the lower ground floor level at the rear of the house. Sandstone bedrock is also exposed on the southern side of the house and steps down a maximum of about 2.2m to the lower ground floor level. The sandstone was massive except for a 0.05m to 0.2m wide sub-horizontal bedding

parting with extremely weathered infill situated towards the crest of the cut. The surface of the sandstone adjacent to the southern wall of the house dips down to the east. The sandstone was generally assessed to be of at least medium strength. Between the southern boundary and the house, dense vegetation covers the ground surface although isolated exposures of sandstone bedrock can be observed stepping down to the north.



Plate 1. Exposed sandstone shelf below western end of existing residence

Roughly halfway across the site, a metal chain link fence provides a delineation between the developed western portion and the steeper, undeveloped eastern portion of the site. The northern half of this fence is situated at the crest of a sandstone boulder wall which increases in height from 1m to 2.1m towards the northern boundary. The wall appears to be founded on the surface of sandstone bedrock which is exposed at the toe on the northern boundary. An overgrown path extends roughly east-west from a gate at the centre of the fence to the boundary with the neighbouring property to the south-east, No. 117 Pacific Road. The path is predominantly situated above exposed sandstone bedrock which appears to dip down to the north-east at approximately 25° overall before stepping down through a sub-vertical cliff line.



Plate 2. Sandstone cliff line exposed in eastern half of site

The cliff line (partially shown in Plate 2 above) extends from the south-eastern corner of the residence at No. 123 on the northern boundary and then strikes roughly south-east and then east adjacent to the southern site boundary. The cliff line generally has maximum sub-vertical faces that range in height up to about 4m to 5m. Defects within the cliff line generally comprise sub-horizontal bedding partings and sub-vertical joints. Weathering along the bedding partings has resulted in undercut portions of sandstone particularly towards the crest of the cliff line. The sandstone was assessed as being generally at least medium strength. At the toe of the cliff line surface levels slope down at between 20° and 30° to the north and north-east with numerous large sandstone boulders (up to 20m² in surface area) observed. The boulders are generally 'platy' in shape and appear to be embedded within the soils.

Adjacent to the north-western boundary of the site is an approximately 60m long pebblecrete driveway, which provides access to the neighbouring residences at Nos. 121, 121A and 123 Pacific Road. Surface levels across the boundary step down from the subject site through a couple of retaining walls which generally increase in height towards the east. The retaining walls comprise mortared sandstone block walls that have a maximum height of about 1.1m. The eastern end of the sandstone block wall has been constructed at the crest of a sub-vertical cut through sandstone bedrock that has a height of about 1.8m, resulting in a total drop of about 3.1m.

The driveway ends at No. 123 Pacific Road, which contains a one to three-storey residence located adjacent to the middle portion of the northern site boundary. This house is set back about 1.2m from the common boundary, is of steel frame construction and appears to be in good condition. The residence appears to be primarily founded on the exposed sandstone cliff line which slopes down to the north-east below the building footprint (see Plate 3 below). The land adjoining the eastern end of the northern boundary is heavily vegetated. Surface levels across the boundary follow the contours of the sandstone cliff line.

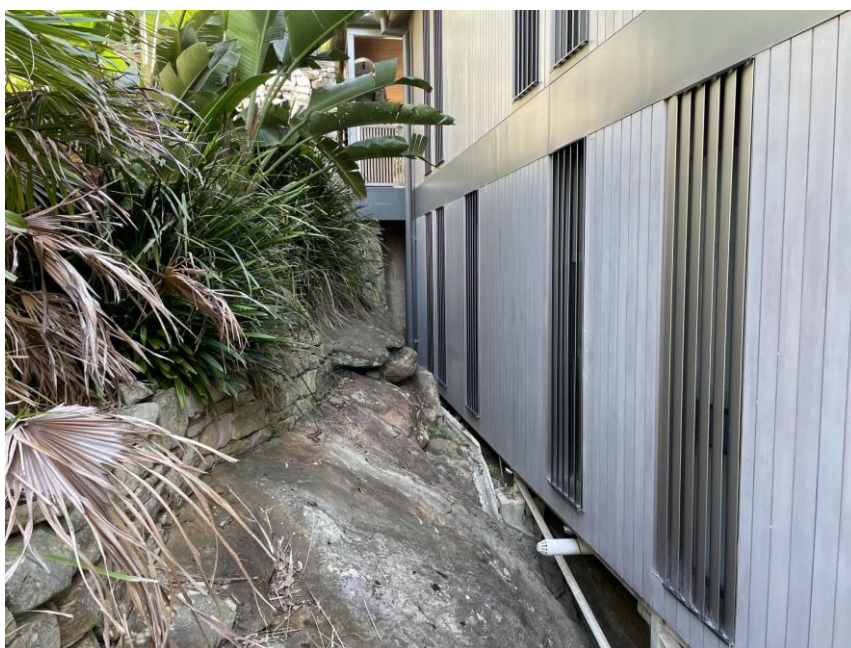


Plate 3. Southern side of residence at No. 123 Pacific Road north of site looking west

Adjoining the site to the south are two properties, Nos. 117 and 119 Pacific Road. No. 119 Pacific Road is the westernmost of these properties while No.117 Pacific Road is the easternmost.

No. 119 Pacific Road is occupied by two detached residences, both of which are two-storey and are set back between about 0.6m and 1.2m from the common boundary. The upper residence is timber clad while the lower residence is of sandstone block construction on the lower ground floor and weatherboard construction on the ground floor level. Both residences appear to be in good condition when viewed from within the site. Surface levels within adjoining property generally slope down to the east, although some modification of levels has been completed and is supported by sandstone block retaining walls varying in height up to about 1m. Along much of the common boundary is a sandstone block retaining wall which extends from adjacent to the northern edge of the upper residence to the eastern end of the lot. The wall ranges in height from about 0.5m to 3.2m above adjacent levels within the subject site. The wall generally increases in height towards the east from the northern corner of each of the residences with a step in the top of the wall of about 1m adjacent to the northern corner of the lower residence. The wall generally appears to be in good condition showing no signs of distress in the form of bulging, outward rotation or cracking.

Adjoining the site over the eastern portion of the common boundary is No. 117 Pacific Road. Towards the eastern end of the property is a two-storey weatherboard and sandstone block house which appears to be constructed at the crest of a continuation of the sandstone cliff line located within the eastern half of the site and steps down to the east and north. The house appears to be in good condition when viewed from within the site. Surface levels across the boundary are generally similar and follow the natural sandstone cliff line, except at the western end of the boundary where a timber crib wall that varies in height between 1.8m and 2.1m is present and retains the adjoining property.

East of the site surface levels slope down to the east and north-east at approximately 20° to 25°. Set back a minimum 2.5m from the north-eastern corner of the site is a two-storey weatherboard residence (No. 69 Florida Road) that is suspended above the hillside on concrete piers. This building appears to be in good condition. Another residence (No. 65 Florida Road) is set back approximately 40m from the eastern boundary, however inspection of this property was not possible from within the site.

4 SUBSURFACE CONDITIONS

The Sydney 1:100,000 Geological Series Sheet 9130 indicates that the site is underlain by Hawkesbury Sandstone. Sandstone was observed outcropping across the site, particularly in the steeper eastern half of the property. The surface of the sandstone steps and slopes down to the north-east with the surface of the rock shelf below the existing residence at approximately RL76m and the toe of the outcrop below No. 123 Pacific Road at approximately RL59m. We anticipate that, where not exposed, sandy and clayey colluvial and residual soils overlie the bedrock.

Groundwater seepage is expected to be present at the interface between the bedrock and soil with periodic flows expected during and following periods of rainfall. We do not believe that a groundwater table will be present within the proposed depth of excavation.

5 GEOTECHNICAL ASSESSMENT

5.1 Potential Landslide Hazards

We consider that the potential landslide hazards associated with the site to be the following:

- A Stability of hillside slope:
 - i. In the western half of the site,
 - ii. In the eastern half of the site,
- B Stability of the existing sandstone cliff lines:
 - i. Around and below the existing house,
 - ii. Along northern boundary adjacent to carport at No. 123 Pacific Road,
 - iii. Within eastern half of the site (beyond proposed development area),
- C Stability of the existing retaining walls on site:
 - i. Boulder retaining wall located at rear of the existing residence,
 - ii. Boulder retaining wall located roughly halfway along site.
- D Stability of the existing boundary retaining walls:
 - i. Sandstone block retaining wall along northern boundary,
 - ii. Sandstone block retaining wall along southern boundary with No. 119 Pacific Road,
 - iii. Timber crib retaining wall along southern boundary with No. 117 Pacific Road.
- E Stability of proposed retaining walls:
 - i. Supporting the excavation for the main house,
 - ii. Supporting excavation for the driveway along southern boundary.
- F Stability of proposed cuts through sandstone bedrock at western edge of proposed lower ground floor level of the main house.
- G Stability of embedded boulders.

These potential hazards are indicated on the attached Figures 2, 3 and 4.

5.2 Risk Analysis

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Use has been made of data in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A indicates that the assessed risk to property be “Very Low” to “Low”, which would be considered ‘acceptable’ in accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life for the person most at risk. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to

life for the person most at risk for the site in its existing state is 3×10^{-7} . This would be considered to be “acceptable” in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy for existing developments.

5.3 Risk Assessment

The Pittwater Risk Management Policy requires suitable measures ‘to remove risk’. It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site and/or development cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners are made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, where the policy requires that ‘reasonable and practical measures have been identified to remove risk’, it means that there has been an active process of reducing risk, but it does not require the geotechnical engineer to warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

Similarly, the Pittwater Risk Management Policy requires that the design project life be taken as 100 years unless otherwise justified by the applicant. This requirement provides the context within which the geotechnical risk assessment should be made. The required 100 years baseline broadly reflects the expectations of the community for the anticipated life of a residential structure and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 100-year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, the Policy does not seek the geotechnical engineer to warrant the development for a 100-year period, rather to provide a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.

In preparing our recommendations given below we have adopted the above interpretations of the Risk Management Policy requirements. We have also assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all Council’s buried services are, and will be regularly maintained to remain, in good condition.

We consider that our risk analysis has shown that the site in its existing state does not achieve the ‘Acceptable Risk Management’ the Pittwater Risk Management Policy. The recommendations in Section 6 form an integral part of the Landslide Risk Management Process.

6 COMMENTS AND RECOMMENDATIONS

We consider that the proposed development may proceed provided the following specific design, construction and maintenance recommendations are adopted to maintain and reduce the present risk of instability on the site and to control future risks. These recommendations address geotechnical issues only and other conditions may be required to address other aspects.

6.1 Conditions Recommended to Establish the Design Parameters

- 6.1.1 All proposed footings must be founded in sandstone bedrock. The footings should be designed for an allowable bearing pressure of 600kPa, subject to inspection by a geotechnical engineer prior to pouring. Where footings for structures are proposed to be located within a distance of $2H$ (where H is the retained height) from the crest of walls or sub-vertical cuts/cliff faces in rock then the bedrock in the face below will need to be inspected for suitability of the proposed loading. Alternatively, such footings may be designed to found below a line drawn up from the base of the cut/wall/cliff face at $1V:2H$.
- 6.1.2 The proposed excavation is expected to encounter sandstone bedrock and, where these are carried out using percussive equipment, such as hydraulic rock hammers, continuous vibration monitoring must be carried out during rock hammer use. The ground vibration measured as peak particle velocity must not exceed 5mm/sec at the northern and southern site boundaries.
- 6.1.3 We expect that excavation will encounter colluvial soils overlying sandstone bedrock at shallow depth. Any excavation within the soils will need to be battered at a suitable angle or retained by engineer designed retaining walls that are installed prior to the start of excavation.
- 6.1.4 Subject to inspection by a geotechnical engineer, temporary batters for the proposed excavation should be no steeper than 1 Vertical (V) in 1.5 Horizontal (H) within the soil profile and extremely weathered rock and vertical in competent rock, provided it is free from adverse defects. An allowance should be made for remedial measures should adverse defects be present. These measures will likely comprise shotcrete, mesh and bolts. All surcharge and footing loads must be kept well clear of the excavation perimeter.
- 6.1.5 We consider that temporary batters can be accommodated within much of the site due to the apparent presence of shallow good quality bedrock and the proposed setbacks. However, along the southern boundary where excavation for the proposed driveway extends up to the boundary care will be taken to ensure that excavation does not undermine the existing sandstone block wall along the boundary. Prior to the commencement of construction we recommend that a series of test pits be excavated at the base of this wall to assess the founding conditions of the wall. Where the wall is founded on soil or extremely to highly weathered sandstone bedrock the wall will need to be supported by a retention system or underpinned. The retention system may require anchoring to reduce deflections and damage to the existing wall.
- 6.1.6 Where anchors are to run below adjoining properties, permission will need to be sought from the owners of the adjoining properties before installation.
- 6.1.7 The proposed new retaining walls may provisionally be designed using the following parameters:

- For the design of cantilevered retaining walls a triangular earth pressure distribution may be adopted.
- For cantilever walls retaining no more than 3m, which is anticipated to be the case for this site, an 'active' earth pressure coefficient, K_a , of 0.35, may be adopted for the retained height. This assumes that some resulting ground movement is acceptable.
- Where cantilevered walls are restrained from some movement, such as by other structural elements in front of the wall, or where movements are to be reduced, an 'at rest' earth pressure coefficient, K_0 , of 0.55 should be used.
- The above coefficients assume a horizontal backfill surface. Where this is not the case the inclined backfill must either be taken as a surcharge load or a higher coefficient of lateral earth pressure adopted. Where a higher coefficient of lateral earth pressure is used further advice should be sought from this office. Should the height of retaining walls exceed 3m further advice should also be sought from this office.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.
- Any surcharge loads affecting the walls (e.g. traffic loading, live loading, compaction stresses, etc) should be allowed in the design.
- The retaining walls should be provided with complete and permanent drainage of the ground behind the walls. Where temporary batters are used for low height walls, the subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion. For soldier pile walls strip drains should be placed behind the shotcrete panels.
- Toe resistance of the wall may be achieved by keying the footing into bedrock below the base of the excavation. An allowable lateral stress of 200kPa for sandstone of at least low strength may be adopted for design.

6.1.8 It is difficult to successfully place engineered fill on a small scale and consequently, we recommend that all slabs be designed as fully suspended. Where it is decided not to suspend all slabs prior to the placement of engineered fill or slabs on grade we recommend that all topsoil and root affected soils be first stripped from site. The exposed subgrade should then be proof rolled with a minimum of six passes of five tonne (static weight) smooth drum roller. The final pass should be completed in the presence of an experienced geotechnical engineer or geotechnician. Where engineered fill is to be placed it should be free from organic or otherwise deleterious materials, should have a maximum particle size of no greater than 70mm and should ideally comprise a ripped sandstone or similar. Engineered fill should generally be placed in loose layers not exceeding 150mm thickness, although layer thickness may be varied provided the full layer thickness is compacted to within the required specification. Where engineered fill will be used to support structures, it must be compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD). Where it is used in areas of soft landscaping where some movement is acceptable this specification may be relaxed to greater than 95% to 102% of SMDD. In both instances it should be placed within 2% of Standard Optimum Moisture Content (SOMC).

6.1.9 The surface water discharging from the new roof and paved areas must be diverted to outlets for controlled discharge to the existing stormwater system which appears to drain to the road.

6.1.10 The guidelines for Hillside Construction given in Appendix B should also be adopted.

6.2 Conditions Recommended to the Detailed Design to be Undertaken for the Construction Certificate

- 6.2.1 All structural design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.
- 6.2.2 All hydraulic design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.
- 6.2.3 All landscape design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.
- 6.2.4 Dilapidation surveys must be carried out on the neighbouring buildings and structures to the north and south. A copy of the dilapidation report must be provided to the neighbours and Council or the Principle Certifying Authority.
- 6.2.5 An excavation/retention methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation techniques, the proposed excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types, contingency plans in case of exceedances.
- 6.2.6 The excavation/retention methodology must be reviewed and approved by the geotechnical engineer.

6.3 Conditions Recommended During the Construction Period

- 6.3.1 Continuous vibration monitoring must be carried out during rock excavations. The ground vibration measured as peak particle velocity must not exceed 5mm/sec at the northern and southern site boundaries.
- 6.3.2 The geotechnical engineer must inspect all footing excavations prior to placing reinforcement or pouring the concrete. Careful inspection of footings located close to the existing cliff must be carried out and removal of overhanging portions of the cliff or underpinning may be required at some locations.
- 6.3.3 The approved excavation/retention methodology must be followed.
- 6.3.4 Bulk excavations must be progressively inspected by the geotechnical engineer as excavation proceeds. We recommend inspections at 1.5m vertical depth intervals and on completion. Similarly, where existing sandstone cliffs are to remain these must be inspected by a geotechnical engineer during construction and any support required installed.
- 6.3.5 Proposed material to be used for backfilling behind retaining walls must be approved by the geotechnical engineer prior to placement.

- 6.3.6 Compaction density of the backfill material that will support structures must be checked by a NATA registered laboratory under Level 1 compaction control. Where the fill will support soft landscaped areas the level of compaction control may be reduced to Level 2. All testing must be completed in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer for review.
- 6.3.7 The geotechnical engineer must inspect all subsurface drains prior to backfilling.
- 6.3.8 If they are to be retained, the existing stormwater system, sewer and water mains must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer or architect, and repaired if found to be leaking.
- 6.3.9 An 'as-built' drawing of all buried services at the site must be prepared (including all pipe diameters, pipe depths, pipe types, inlet pits, inspection pits, etc).
- 6.3.10 The geotechnical engineer must confirm that the house has been constructed in accordance with the geotechnical reports.

We note that all above Conditions must be complied with. Where this has not been done, it may not be possible for Form 3, which is required for the Occupation Certificate to be signed.

6.4 Conditions Recommended for Ongoing Management of the Site/Structures

The following recommendations have been included so that the current and future owners of the subject property are aware of their responsibilities:

- 6.4.1 All proposed surface (including roof) and subsurface drains must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than ten yearly intervals; including provision of a written report confirming scope of work completed (with reference to the 'as-built' drawing) and identifying any required remedial measures.
- 6.4.2 No cut or fill in excess of 0.5m (e.g. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Pittwater Council.
- 6.4.3 Where the structural engineer has indicated a design life of less than 100 years then the structure and/or structural elements must be inspected by a structural engineer at the end of their design life; including a written report confirming scope of work completed and identifying the required remedial measures to extend the design life over the remaining 100-year period.

7 GENERAL COMMENTS

Provided the comments and recommendations above are followed, we consider that the site poses an acceptable risk to both life and property for the proposed house.

It is possible that the subsurface soil, rock or groundwater conditions encountered during construction may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

Reference 2: MacGregor, P, Walker, B, Fell, R, and Leventhal, A (2007) *'Assessment of Landslide Likelihood in the Pittwater Local Government Area'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp183-196.



TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A		B			C	
	i. Stability of Colluvial/Residual Slopes in Western Half of the Site	ii. Stability of Colluvial Slopes in the eastern half of the site	i. Stability of Existing Cliff Lines around and below the existing house	ii. Stability of Existing Cliff Line along northern boundary that extends to carport at No. 123 Pacific Road	iii. Stability of Existing Cliff Lines within eastern half of the site	i. Stability of Existing Boulder Retaining Wall at rear of residence	ii. Stability of Existing Boulder Retaining Wall roughly halfway along site
Assessed Likelihood	Unlikely	Likely	Unlikely	Unlikely	Unlikely	Possible	Possible
Assessed Consequence	Minor	Insignificant	Minor	Insignificant	Insignificant	Insignificant	Insignificant
Risk	Low	Low	Low	Very Low	Very Low	Very Low	Very Low
Comment	Assumes all footings founded within bedrock and suitable drainage provided to property.	Assumes no development occurs within eastern portion of site below toe of sandstone cliff lines		Assumes cliff line is inspected by a geotechnical engineer during construction and any stabilisation measures are constructed.	Assumes no development occurs within eastern portion of site and vibration monitoring during construction.		



POTENTIAL LANDSLIDE HAZARD	D			E and F		G
	i. Stability of Existing Sandstone Block Retaining Wall along northern boundary	ii. Stability of Existing Sandstone Block Retaining Wall along southern boundary with No. 119 Pacific Road	iii. Stability of Existing Timber Crib Retaining Wall along southern boundary with No. 117 Pacific Road	i. Stability of Proposed Retaining Walls Supporting Excavation for the main house	ii. Stability of Proposed Retaining Walls Supporting Excavation for the driveway along southern boundary.	Stability of Proposed Cuts through sandstone bedrock at western edge of proposed lower ground floor level of main house
Assessed Likelihood	Unlikely	Unlikely	Almost Certain	Barely Credible	Barely Credible	Unlikely
Assessed Consequence	Insignificant	Insignificant	Insignificant	Minor	Minor	Insignificant
Risk	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low
Comment	Assumes all footings founded within bedrock.	Assumes investigative test pits excavated to assess requirement for retention prior to excavation and recommendations implemented.	Assumes no development adjacent to toe of wall on site.	Assumes walls are properly designed and constructed in accordance with design and that all cut faces are inspected by Geotechnical Engineer.	Assumes walls are properly designed and constructed in accordance with design and that all cut faces are inspected by Geotechnical Engineer.	Assumes boulders runout only a short distance due to their platelike shape.

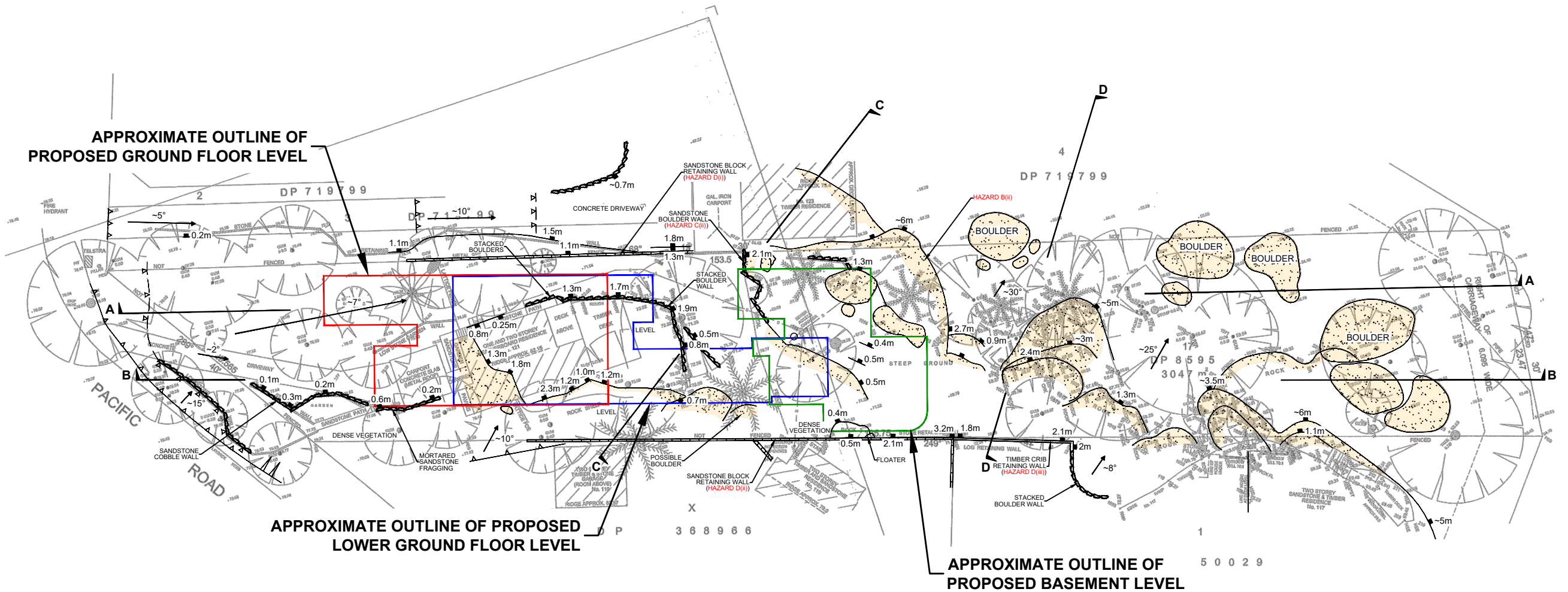
Estimated Value \$7.5M



TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A		B			C		D			E and F		G
	i. Stability of Colluvial/Residual Slopes in Western Half of the Site	ii. Stability of Colluvial Slopes in the eastern half of the site	i. Stability of Existing Cliff Lines around and below the existing house	ii. Stability of Existing Cliff Line along northern boundary that extends to carport at No. 123 Pacific Road	iii. Stability of Existing Cliff Lines within eastern half of the site	i. Stability of Existing Boulder Retaining Wall at rear of residence	ii. Stability of Existing Boulder Retaining Wall roughly halfway along site	i. Stability of Existing Sandstone Block Retaining Wall along northern boundary	ii. Stability of Existing Sandstone Block Retaining Wall along southern boundary with No. 119 Pacific Road	iii. Stability of Existing Timber Crib Retaining Wall along southern boundary with No. 117 Pacific Road	i. Stability of Proposed Retaining Walls Supporting Excavation for the main house	ii. Stability of Proposed Retaining Walls Supporting Excavation for the driveway along southern boundary.	Stability of embedded boulders in slope
Assessed Likelihood	Unlikely	Likely	Unlikely	Unlikely	Unlikely	Possible	Possible	Unlikely	Unlikely	Almost Certain	Barely Credible	Barely Credible	Unlikely
Indicative Annual Probability	1×10^{-4}	1×10^{-2}	1×10^{-4}	1×10^{-4}	1×10^{-4}	1×10^{-3}	1×10^{-3}	1×10^{-4}	1×10^{-4}	1×10^{-1}	1×10^{-6}	1×10^{-6}	1×10^{-6}
Duration of Use of area Affected (Temporal Probability)	Persons in garden around house: Say 10 minutes/day 3.47×10^{-3}	Person in rear yard of 121 and 123 Pacific Road Say 2 minute/day 6.94×10^{-4}	Person in plenum at between rear of house and base of existing cut Say 1 minute/week 9.92×10^{-5}	(i) Person at crest of cliff Say 5 minutes/week 1.19×10^{-2} (ii) Person at base of cliff Say 1 minute/day 6.94×10^{-4}	(i) At crest of cliff line Say 1 minute/day 6.94×10^{-4} (ii) At base of cliff line Say 1 minute/week 9.9×10^{-5}	(i) Person at crest of wall Say 2 minutes/day 1.39×10^{-3} (ii) Person at base of wall Say 1 minute/week 6.94×10^{-4}	(i) Person at crest of wall Say 2 minutes/day 1.39×10^{-3} (ii) Person at base of wall Say 1 minute/day 2.31×10^{-5}	(i) Person at crest of wall Say 5 minutes/month 1.16×10^{-4} (ii) Person at base of cliff Say 1 minute/day 6.94×10^{-4}	Person on driveway at base of wall Say 2 minutes/day 1.39×10^{-3}	Person in rear yard at base of wall Say 1 minute/month 2.31×10^{-5}	Person at base of retaining wall/cut in plant room, cellar, laundry and pool room Say 5 minutes/day 3.5×10^{-3}	Person on driveway at base of wall Say 2 minutes/day 1.39×10^{-3}	Person on slope Say 2 minutes/day 1.4×10^{-3}
Probability of not Evacuating Area Affected	(i) On slide 0.9 (ii) Below slide 0.1 Slide likely to be slowly moving and of low volume	(i) On slide 0.9 (ii) Below slide 0.5 (Some warning likely)	0.9	(i) On cliff line 1 (ii) Below cliff line 0.5	(i) 1 (ii) 0.5	(i) 0.9 (ii) 0.5	(i) 0.9 (ii) 0.5	(i) On top of wall 1 (ii) Below wall 0.1	0.9 Little warning likely	0.5	0.9	0.9	0.5
Spatial Probability	(i) Affected area $7\text{m} \times 4\text{m} / 430\text{m}^2 = 0.065$ (ii) Affected Area $3\text{m} \times 4\text{m} / 430\text{m}^2 = 0.028$	(i) Affected area $7\text{m} \times 4\text{m} / 1,080\text{m}^2 = 0.026$ (ii) Affected Area $7\text{m} \times 4\text{m} / 448\text{m}^2 = 0.0625$	$2 / 8.35\text{m} = 0.24$	(i) and (ii) Affected area $2\text{m} / 15\text{m} = 0.13$	(i) and (ii) Affected area $3\text{m} / 50\text{m} = 0.06$	(i) and (ii) Affected area $3\text{m} / 20\text{m} = 0.15$	(i) and (ii) Say $3\text{m} / 9\text{m}$ 0.33	(i) $1.5\text{m} / 30\text{m} = 0.05$ $1.5\text{m} / 10\text{m} = 0.15$	$2\text{m} / 40\text{m} = 0.05$	$2\text{m} / 13\text{m} = 0.15$	$3\text{m} / 46\text{m} = 0.065$	$3\text{m} / 40\text{m} = 0.075$	$6\text{m} / 80\text{m} = 0.075$
Vulnerability to Life if Failure Occurs Whilst Person Present	(i) 0.01 (Likely to ride slide down) (ii) 0.1 (Unlikely to be buried)	(i) 0.1 (Likely to ride slide down) (ii) 0.5 (Possibly buried)	0.9 (May be buried)	(i) 0.01 (Likely to ride collapse down) (ii) 0.1 (Unlikely to be killed)	(i) 0.1 (Possibly be buried under detached material) (ii) 1 (Death likely)	(i) 0.01 (Likely to ride slide down) (ii) 0.1 (Unlikely to be buried)	(i) 0.01 (likely to ride slide down) (ii) 0.1	(i) 0.01 Likely to ride slide down (ii) 0.1 (iii) Unlikely to be buried	1 Death likely	0.8 May be buried	0.8 May be buried	0.9 Death possible	0.9
Risk for Person most at Risk	(i) 4.1×10^{-10} (ii) 9.7×10^{-11}	(i) 1.6×10^{-8} (ii) 1.1×10^{-7}	2.1×10^{-9}	(i) 1.6×10^{-9} (ii) $4. \times 10^{-10}$	(i) 4.2×10^{-10} (ii) 3.0×10^{-10}	(i) 1.9×10^{-9} (ii) 5.2×10^{-9}	(i) 4.1×10^{-9} (ii) 3.8×10^{-10}	(i) 5.8×10^{-12} (ii) 1.0×10^{-10}	6.3×10^{-9}	1.4×10^{-7}	1.6×10^{-10}	8.4×10^{-11}	4.7×10^{-9}
Combined Total Risk for Person Most at Risk	2.9×10^{-7}												

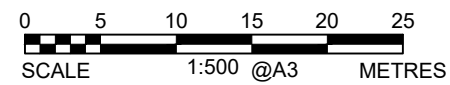
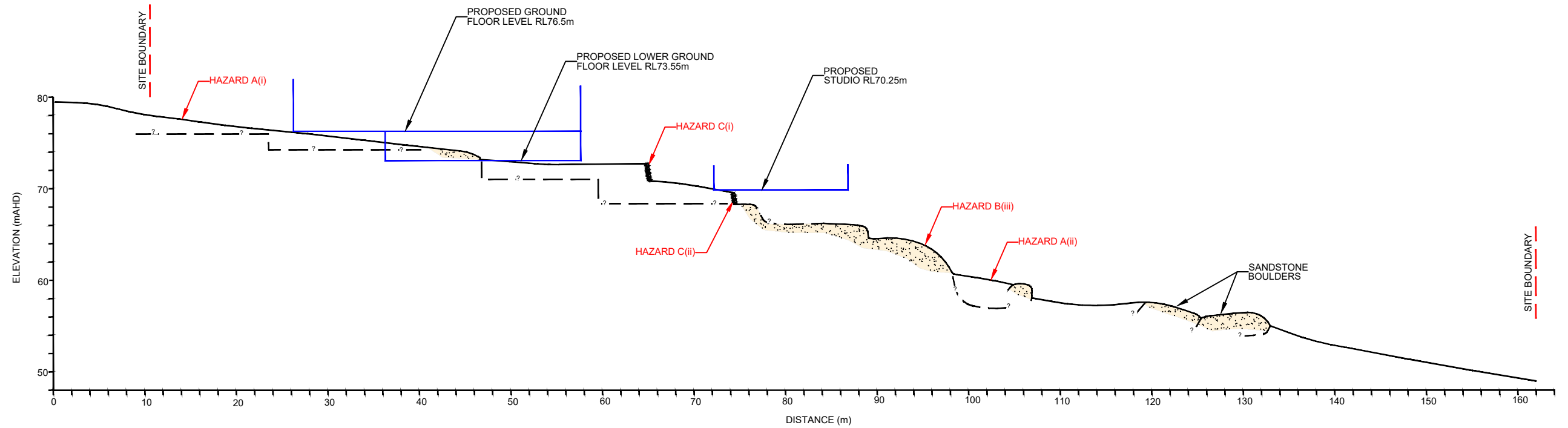
Note: From the summation of risk for person most at risk, the total risk for the person most at risk is



Title:		GEOTECHNICAL SKETCH PLAN	
Location:		121 PACIFIC ROAD, PALM BEACH, NSW	
Report No:		34390YT	Figure No: 2
JKGeotechnics			



PLOT DATE: 28/10/2021 2:57:10 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000\S\34390YT PALM BEACH\ICAD\34390YT.DWG

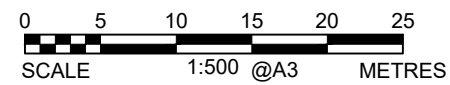
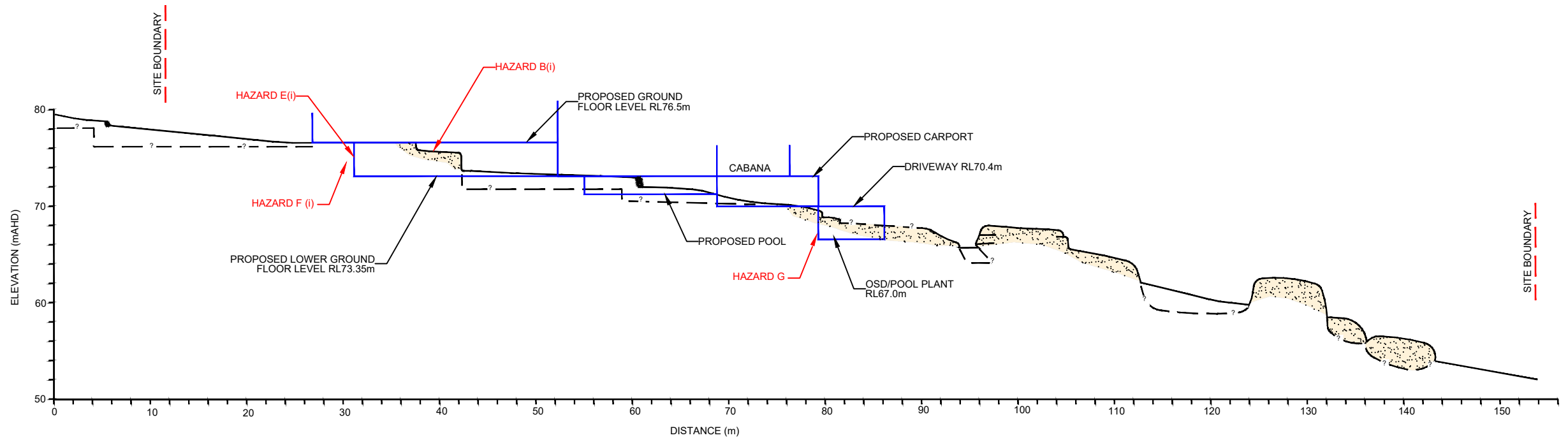


This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION A-A	
LOOKING NORTH NORTH WEST	
Location: 121 PACIFIC ROAD, PALM BEACH, NSW	
Report No: 34390YT	Figure No: 3
JKGeotechnics	



PLOT DATE: 28/10/2021 3:02:13 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000\S\34390YT PALM BEACH\ICAD\34390YT.DWG

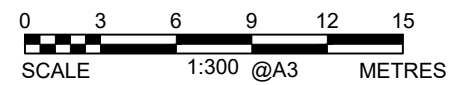
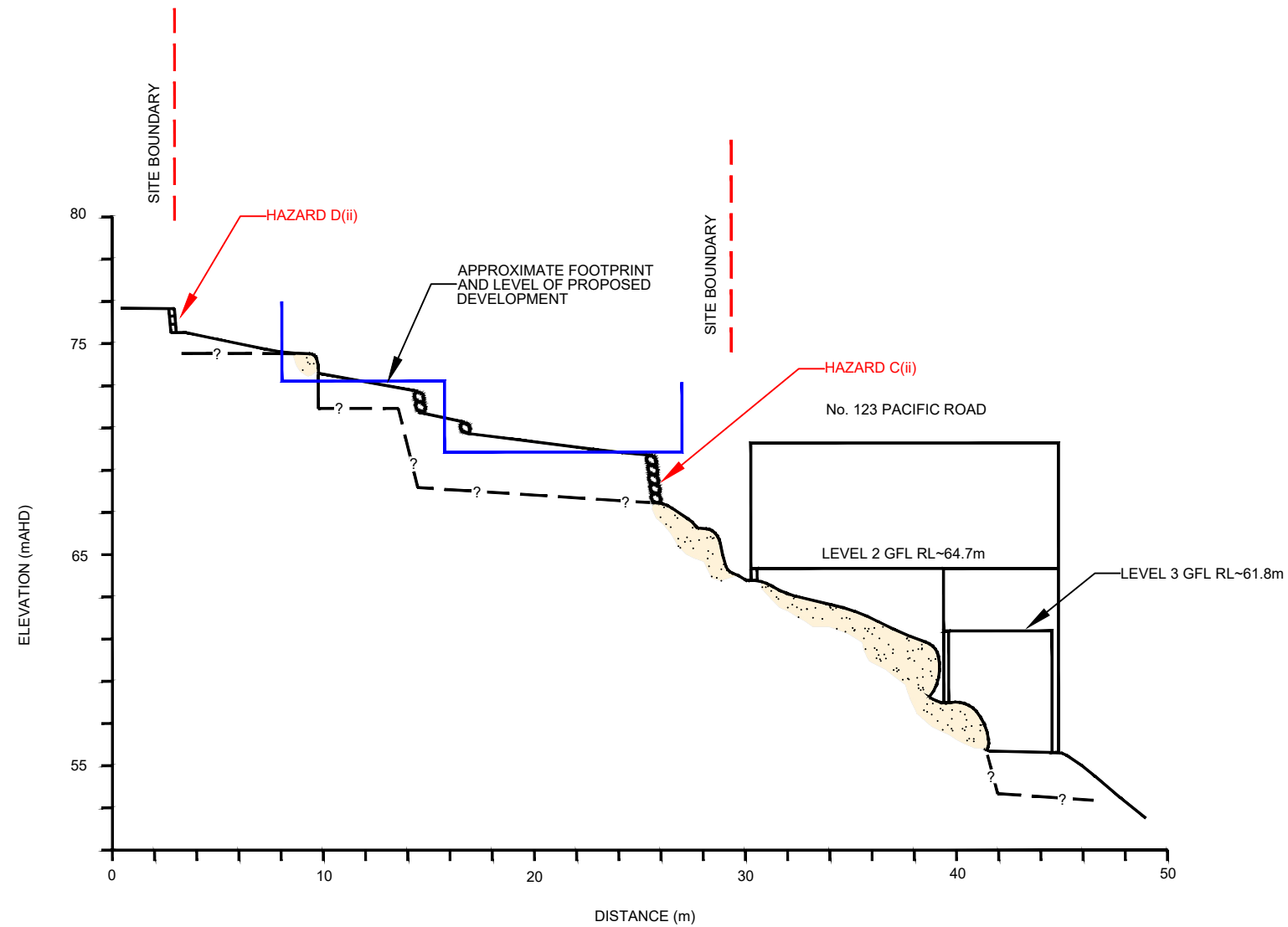


This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION B-B	
LOOKING NORTH NORTH WEST	
Location: 121 PACIFIC ROAD, PALM BEACH, NSW	
Report No: 34390YT	Figure No: 4
JKGeotechnics	



PLOT DATE: 28/10/2021 12:28:25 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000\34390YT PALM BEACH\CAD\34390YT.DWG

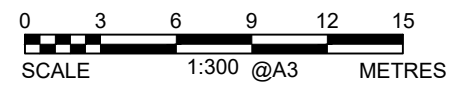
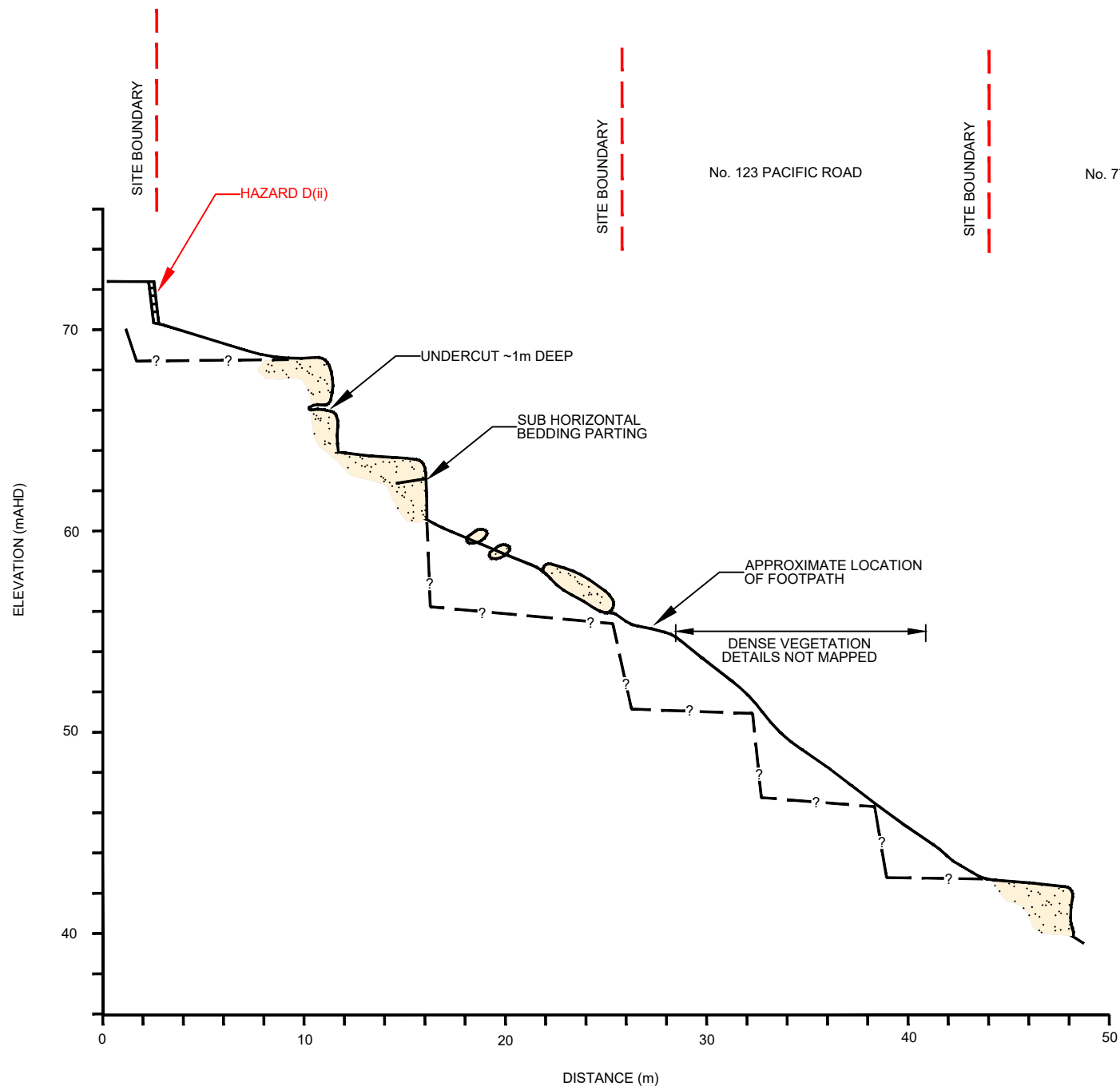


This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION C-C LOOKING WEST	
Location: 121 PACIFIC ROAD, PALM BEACH, NSW	
Report No: 34390YT	Figure No: 5
JKGeotechnics	



PLOT DATE: 28/10/2021 12:28:34 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\3400'S\34390YT PALM BEACH\CAD\34390YT.DWG



This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION D-D LOOKING WEST	
Location: 121 PACIFIC ROAD, PALM BEACH, NSW	
Report No: 34390YT	Figure No: 6
JKGeotechnics	



TOPOGRAPHY

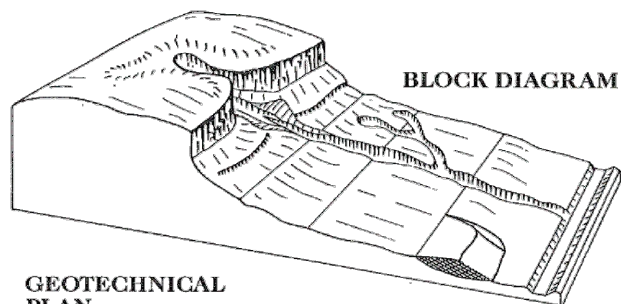
Symbol Ground Profile

		convex	}	well defined or angular break of slope
		concave		
		convex	}	poorly defined or smooth change of slope
		concave		
		breaks of slope	}	convex and concave too close together to allow the use of separate symbols
		changes of slope		
		sharp	}	ridge crest
		rounded		
		Cliff or escarpment or sharp break 40° or more (estimated height in metres)		
		Uniform Slope	}	Slope direction and angle (Degrees)
		Concave Slope		
		Convex Slope		
		Top	}	Cut or fill slope, arrows pointing down slope
		Bottom		
		Hummocky or irregular ground		

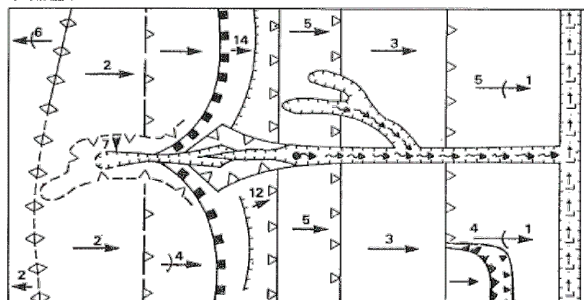
OTHER FEATURES

	Boulder
	Seepage/spring
	Swallow hole for runoff
	Natural water course
	Open drain, unlined
	Open drain, lined
	Fenceline
	Property boundary
	Dry Stone Wall
	Major joint in rock face (opening in millimetres)
	Tension crack (opening in millimetres)
	Masonry or concrete wall
	Ponding water
	Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

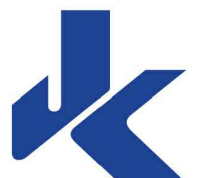


GEOTECHNICAL PLAN



(After Gardiner, V & Dackombe, R. V. (1983), Geomorphological Field Manual; George Allen & Unwin).

Title:	GEOTECHNICAL MAPPING SYMBOLS	
Location:	121 PACIFIC ROAD, PALM BEACH, NSW	
Report No:	34390YT	Figure No: 7
JKGeotechnics		



This plan should be read in conjunction with the JK Geotechnics report.



APPENDIX A

**LANDSLIDE RISK
MANAGEMENT
TERMINOLOGY**

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

Risk Terminology	Description
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.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
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.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
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.
Individual Risk to Life	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.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
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, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	<p>A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event.</p> <p>These are two main interpretations:</p> <ul style="list-style-type: none"> (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability (continued)	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
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.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
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 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 judgments 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 Management	The complete process of risk assessment and risk control (or risk treatment).
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.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
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.

NOTE: Reference should be made to Figure A1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

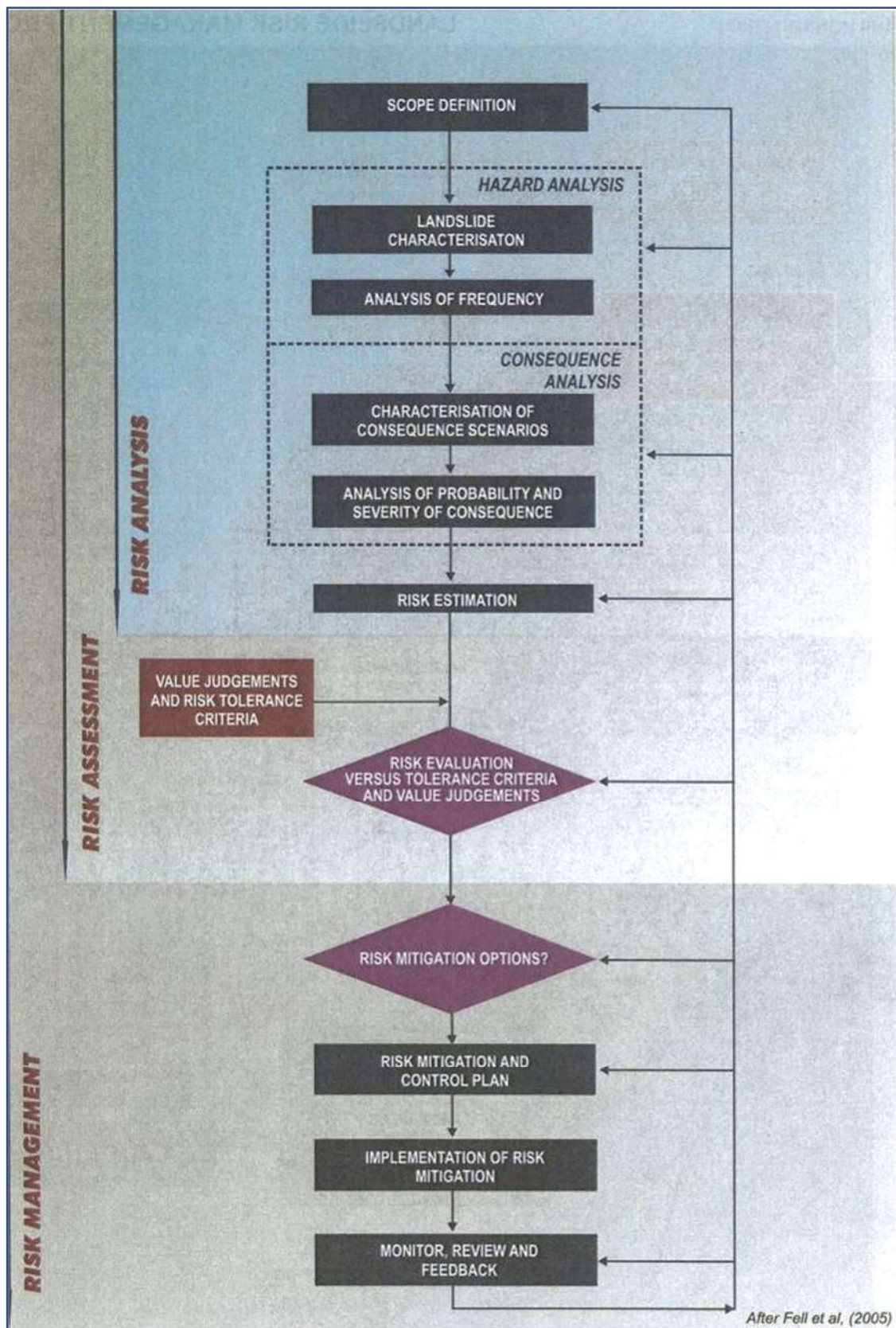


FIGURE A1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10 ⁻¹	5×10 ⁻²	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10 ⁻²		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10 ⁻³	5×10 ⁻³	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10 ⁻⁴	5×10 ⁻⁴	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10 ⁻⁵	5×10 ⁻⁵	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10 ⁻⁶	5×10 ⁻²	1,000,000 years	200,000 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%		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%	40%	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%	10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%	1%	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*.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
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) 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.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book “Guideline Document Landslide Hazards” published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board’s website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both “potential” and “actual” landslides must be taken very seriously. They present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with serious consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can “run-out” from above, “regress” from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else’s land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

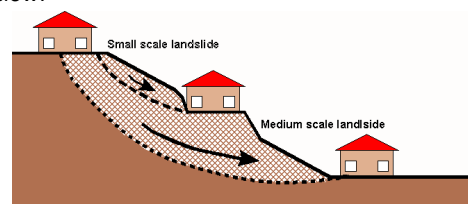


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

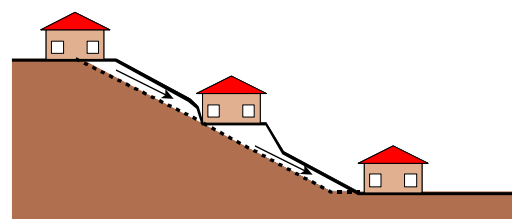


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

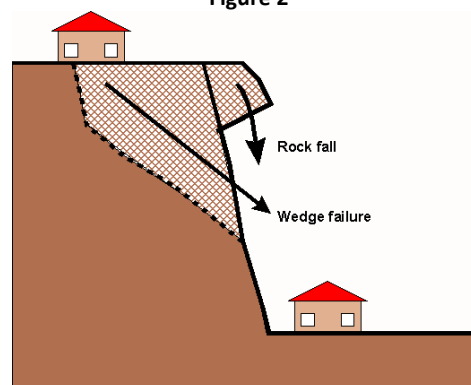


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

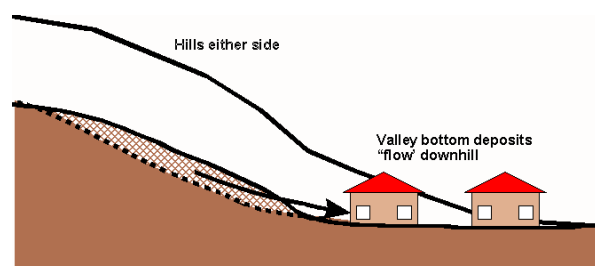


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the [Australian Geomechanics Society](#), a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific **"landslide hazard zones"**. Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment

for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of the repairs and temporary loss of use if the landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2 – LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable risk level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	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 the value of the property.
High	H	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	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 possible.
Low	L	Usually acceptable to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	Acceptable. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	Activity/Event Leading to Death (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding, ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.



APPENDIX B

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION



SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

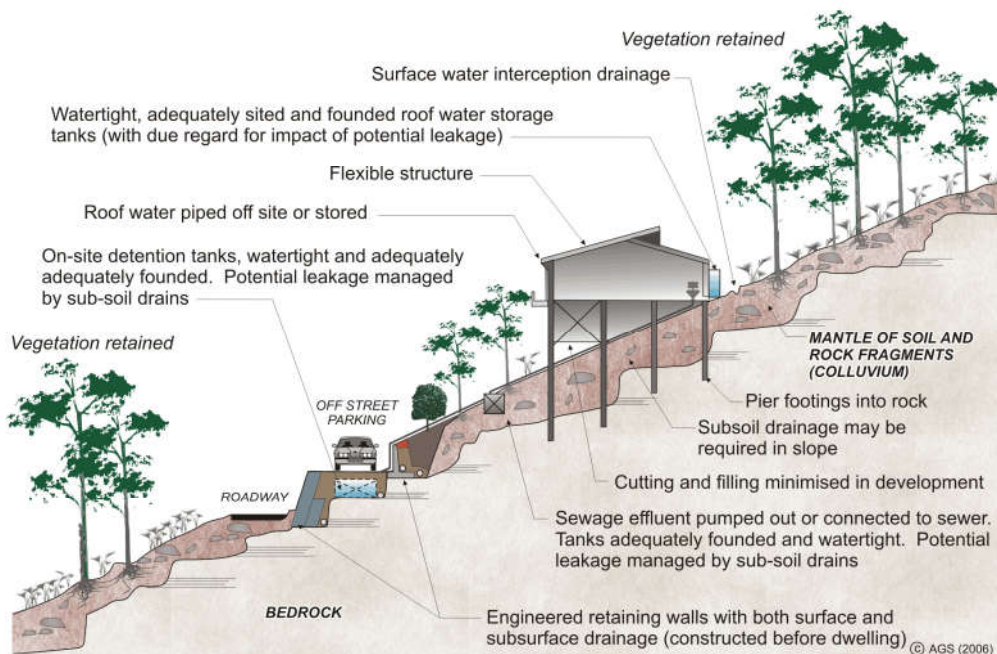
GOOD ENGINEERING PRACTICE		POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having 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.	Indiscriminant 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 properties 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 bedrock 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 bedrock 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 generous 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 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 of roof run-off 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 of 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 a 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 seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is extracted from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in *Australian Geomechanics*, Vol 42, No 1, March 2007 which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES FOR **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that due to level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfill the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

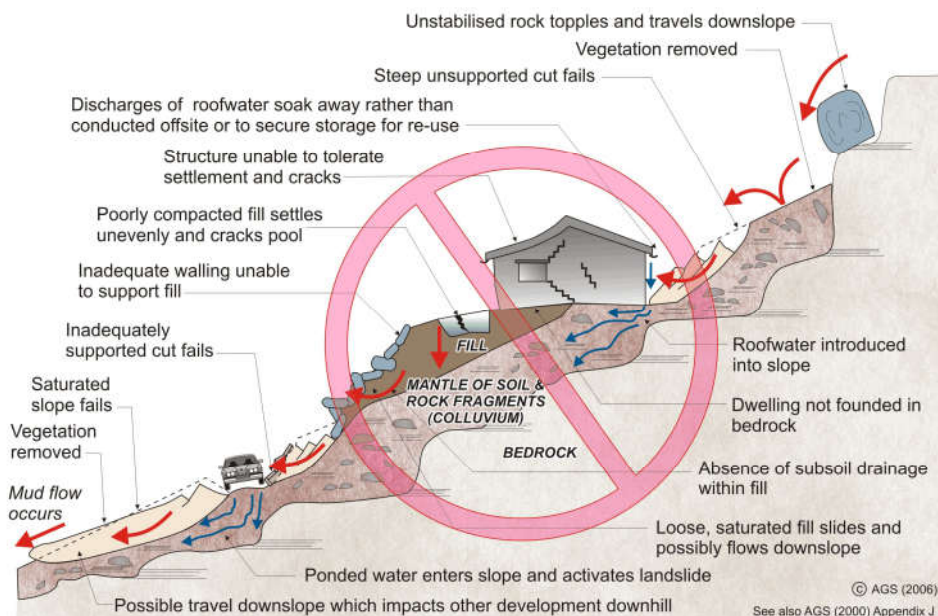
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soaks into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herringbone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- | | |
|-----------------------------------|--|
| • GeoGuide LR1 - Introduction | • GeoGuide LR7 - Landslide Risk |
| • GeoGuide LR3 - Soil Slopes | • GeoGuide LR8 - Hillside Construction |
| • GeoGuide LR4 - Rock Slopes | • GeoGuide LR9 - Effluent & Surface Water Disposal |
| • GeoGuide LR5 - Water & Drainage | • GeoGuide LR10 - Coastal Landslides |
| • GeoGuide LR6 - Retaining Walls | • GeoGuide LR11 - Record Keeping |

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the [Australian Geomechanics Society](#), a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite ‘safe’, depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are ‘safe limits’, up to which no damage due to vibration effects has been observed for the particular class of building. ‘Damage’ is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the ‘safe limits’, then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the ‘safe limits’ are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

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

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 'Geotechnical Site Investigations'. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and 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 by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually 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 assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg 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 blows, as

N = 13
4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

N > 30
15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation:

The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 as a percentage.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_0), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_v), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_0).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it 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 and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

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.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, 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 be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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 specially 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.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

- i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
- ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or
- iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL



FILL



TOPSOIL



CLAY (CL, CI, CH)



SILT (ML, MH)



SAND (SP, SW)



GRAVEL (GP, GW)



SANDY CLAY (CL, CI, CH)



SILTY CLAY (CL, CI, CH)



CLAYEY SAND (SC)



SILTY SAND (SM)



GRAVELLY CLAY (CL, CI, CH)



CLAYEY GRAVEL (GC)



SANDY SILT (ML, MH)



PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK



CONGLOMERATE



SANDSTONE



SHALE/MUDSTONE



SILTSTONE



CLAYSTONE



COAL



LAMINITE



LIMESTONE



PHYLLITE, SCHIST



TUFF



GRANITE, GABBRO



DOLERITE, DIORITE



BASALT, ANDESITE



QUARTZITE

OTHER MATERIALS



BRICKS OR PAVERS



CONCRETE



ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 60% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm)	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $C_u > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10} D_{60}}$$

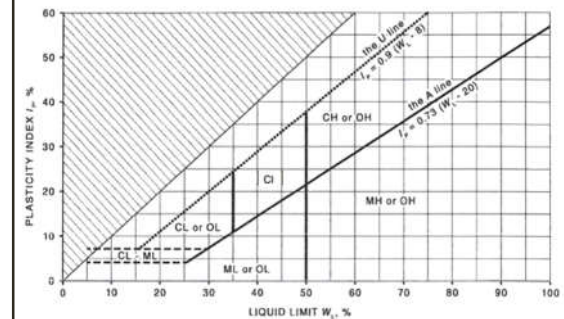
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

- For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
- Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.
- Clay soils with liquid limits $> 35\%$ and $\leq 50\%$ may be classified as being of medium plasticity.
- The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
fine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition																	
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.																	
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.																	
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.																	
Samples	ES	Sample taken over depth indicated, for environmental analysis.																	
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.																	
	DB	Bulk disturbed sample taken over depth indicated.																	
	DS	Small disturbed bag sample taken over depth indicated.																	
	ASB	Soil sample taken over depth indicated, for asbestos analysis.																	
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.																	
	SAL	Soil sample taken over depth indicated, for salinity analysis.																	
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.																	
	N _c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.																	
	VNS = 25	Vane shear reading in kPa of undrained shear strength.																	
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).																	
Moisture Condition (Fine Grained Soils) (Coarse Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.																	
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.																	
	w < PL	Moisture content estimated to be less than plastic limit.																	
	w ≈ LL	Moisture content estimated to be near liquid limit.																	
	w > LL	Moisture content estimated to be wet of liquid limit.																	
	D	DRY – runs freely through fingers.																	
	M	MOIST – does not run freely but no free water visible on soil surface.																	
	W	WET – free water visible on soil surface.																	
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.																	
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.																	
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.																	
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.																	
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.																	
	Hd	HARD – unconfined compressive strength > 400kPa.																	
	Fr	FRIABLE – strength not attainable, soil crumbles.																	
	()	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.																	
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE																	
	L	LOOSE																	
	MD	MEDIUM DENSE																	
	D	DENSE																	
	VD	VERY DENSE																	
	()	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.																	
		<table> <tr> <th></th><th>Density Index (I_D) Range (%)</th><th>SPT 'N' Value Range (Blows/300mm)</th></tr> <tr> <td>VERY LOOSE</td><td>≤ 15</td><td>0 – 4</td></tr> <tr> <td>LOOSE</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr> <tr> <td>MEDIUM DENSE</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr> <tr> <td>DENSE</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr> <tr> <td>VERY DENSE</td><td>> 85</td><td>> 50</td></tr> </table>		Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)	VERY LOOSE	≤ 15	0 – 4	LOOSE	> 15 and ≤ 35	4 – 10	MEDIUM DENSE	> 35 and ≤ 65	10 – 30	DENSE	> 65 and ≤ 85	30 – 50	VERY DENSE	> 85
	Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)																	
VERY LOOSE	≤ 15	0 – 4																	
LOOSE	> 15 and ≤ 35	4 – 10																	
MEDIUM DENSE	> 35 and ≤ 65	10 – 30																	
DENSE	> 65 and ≤ 85	30 – 50																	
VERY DENSE	> 85	> 50																	
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.																	



Log Column	Symbol	Definition
Remarks	'V' bit 'TC' bit T_{60} Soil Origin	Hardened steel 'V' shaped bit. Twin pronged tungsten carbide bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers. The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: 'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column	Symbol Abbreviation	Description
Point Load Strength Index	• 0.6	Axial point load strength index test result (MPa)
	x 0.6	Diametral point load strength index test result (MPa)
Defect Details – Type	Be	Parting – bedding or cleavage
	CS	Clay seam
	Cr	Crushed/sheared seam or zone
	J	Joint
	Jh	Healed joint
	Ji	Incipient joint
	XWS	Extremely weathered seam
	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	P	Planar
	C	Curved
	Un	Undulating
	St	Stepped
	Ir	Irregular
	Vr	Very rough
	R	Rough
	S	Smooth
	Po	Polished
	SI	Slickensided
	Ca	Calcite
	Cb	Carbonaceous
	Clay	Clay
	Fe	Iron
	Qz	Quartz
	Py	Pyrite
	Cn	Clean
	Sn	Stained – no visible coating, surface is discoloured
	Vn	Veneer – visible, too thin to measure, may be patchy
	Ct	Coating ≤ 1mm thick
	Filled	Coating > 1mm thick
	mm.t	Defect thickness measured in millimetres