Form 1 – Declaration and certification made by geotechnical engineer or engineering geologist in a geotechnical report.

DA Number: ____________________

To be submitted with a development application

You can use Form 1 to verify that the author of a geotechnical report is a geotechnical engineer or engineering geologist as defined by the Department of Planning & Environment (DP&E) Geotechnical Policy. Alternatively, where a geotechnical report has been prepared by a professional person not recognised by DP&E Geotechnical Policy, then Form 1 may be used as technical verification of the geotechnical report if signed by a geotechnical engineer or engineering geologist as defined by the DP&E Geotechnical Policy.

Please contact the Alpine Resorts Team in Jindabyne for further information - phone 02 6456 1733.

To complete this form, please place a cross in the appropriate boxes ☐ and complete all sections.

1. Declaration made by geotechnical engineer or engineering geologist as part of a geotechnical report

I,

Mr ☐ Ms ☐ Mrs ☐ Dr ☐ Other ____

First Name: Paul D.
Family Name: Roberts

OF

Company/organisation: JK Geotechnics

on this the 28th day of July 20 21

certify that I am a geotechnical engineer or engineering geologist as defined by the "Policy" and I (tick appropriate box)

☐ prepared the geotechnical report referenced below in accordance with the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

☐ am willing to technically verify that the Geotechnical Report referenced below has been prepared in accordance the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

2. Geotechnical Report Details

Report Title: Geotechnical Assessment

Author: Adrian Hulskamp

Dated: 28 July 2021

DA Site Address: 32 Bobucks Lane, Thredbo

DA Applicant:

I am aware that the Geotechnical Report I have either prepared or am technically verifying, (referenced above) is to be submitted in support of a development application for the proposed development site (referenced above), and it's findings will be relied upon by the Consent Authority in determining the development application.

3. Checklist of essential requirements to be contained in a geotechnical risk assessment report to be submitted with a development application

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Report. This checklist is to accompany the report.

Please tick appropriate box

☒ Risk assessment of all identifiable geotechnical hazards in accordance with AGS 2000, as per 6.1 (a) of the policy.

☒ Site plans with key hazards identified and other information as per 6.1 (b)

☒ Details of site investigation and inspections as per 6.1 (c)

☒ Photographs and/or drawings of the site as per 6.1 (d)

☒ Presentation of geotechnical model as per 6.1 (e)

☒ A specific conclusion as to whether the site is suitable for the development proposed on the above site, if applicable, subject to the following conditions;

- ☒ Conditions to be provided to establish design parameters,
- ☒ Conditions to be incorporated into the detailed design to be submitted for the construction certificate,
- ☒ Conditions applying to the construction phase,
- ☒ Conditions relating to ongoing management of the site/structure.

4. Signatures

Signature	Chartered professional status
Paul Roberts	MIE Aust, CP Eng 2307698
Name	**Date**
Paul D. Roberts	20 December 2022

5. Contact details

Department of Planning & Environment
Alpine Resorts Team
Shop 5A, 19 Snowy River Avenue
PO Box 36, JINDABYNE 2627
Telephone: 02 6456 1733
Facsimile: 02 6456 1736
Email: alpineresorts@planning.nsw.gov.au

REPORT TO

THE R.A.N. SKI CLUB LIMITED

ON

GEOTECHNICAL ASSESSMENT

(In Accordance with Kosciuszko Alpine Resorts Geotechnical Policy)

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

RAN SKI LODGE, 32 BOBUCK LANE,

THREDBO, NSW

Date: 28 July 2021

Ref: 31441RHrpt Rev1

JKGeotechnics

www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801

Report prepared by:

Adrian Hulskamp
Senior Associate l Geotechnical Engineer
NSW Fair Trading RPE No. PRE0000739

Report reviewed by:

Paul Roberts
Principal Associate | Engineering Geologist
NSW Fair Trading RPE No. PRE0000766

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

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
31441RHrpt	Final Report	3 July 2018
31441RHrpt Rev1	Updated Architectural Drawings	28 July 2021

Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical assessment for the proposed alterations and additions to the RAN Ski Lodge at 32 Bobuck Lane, Thredbo, NSW. The location of the site is shown in Figure 1.

In 2018, JK Geotechnics investigated the site for a similar proposed development and the results were presented in our report, Ref. 31441RHrpt, dated 3 July 2018. The original development details have since been revised. We have used the results of our previous investigation in the preparation of this report.

Based on the supplied architectural drawings by Maco Project Designs (Drawing Nos. A000, A001, A002, A101 to A105, A201, A202, A301, A302, A401 and A402, dated June 2021), we understand that following demolition of the western portion of the existing building, a three and four storey extension is proposed. The proposed basement level will have a finished floor level (FFL) at RL1365.0m, requiring excavation back into the hillside to a maximum depth of about 3.7m. The proposed ground floor level will then step up the hillside beyond the basement footprint with a proposed FFL at RL1368.6m. Along the rear of the proposed ground floor level will be a services cavity and then a retaining wall is proposed along the high side, requiring excavation further back into the hillside to a maximum depth of about 4.1m. A lift is proposed and the lift shaft base will be at RL1364.0m, requiring excavation to a maximum depth of about 0.8m below the basement bulk excavation level. The outlines of the proposed basement and ground floor levels are shown on the attached Figure 2. Structural loads typical for a three and four storey building have been assumed.

The purpose of the assessment was to carry out a walkover inspection of the site and to assess the subsurface conditions at two borehole locations, as a basis for comments and recommendations on excavation, drainage, shoring design, footing design, soil aggression and the basement and ground floor slabs.

This report has been prepared in accordance with the requirements of the Geotechnical Policy for Kosciuszko Alpine Resorts (2003) and the Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*.

2 ASSESSMENT PROCEDURE

2.1 Walkover Inspection

A walkover inspection was carried out by our Senior Engineering Geologist on 8 May 2018, and was based upon an 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 sites within Thredbo 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 flow chart illustrating the Risk Management Process, based on the guidelines given in the Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*.

A summary of our observations is presented in Section 3 below. Our geotechnical risk assessment is presented in Section 6. Our specific recommendations regarding the proposed development are discussed in Section 7.

The attached Figure 2 presents a geotechnical site plan and shows the principle geotechnical features observed at the site. Figure 2 is based on the provided survey plan by Peter W. Burns Pty Limited (Job Ref. 5061, Cad File. 5061_CD_01_A, dated 26 April 2018). The slope angles shown on Figure 2 were measured by hand held clinometer and hence are approximate. The heights of retaining walls were recorded using a tape measured. Should any of the features shown on Figure 2 be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. The geotechnical mapping symbols shown on Figure 2 are presented on the attached Figure 3.

A follow up site inspection was also carried out by our Senior Associate Geotechnical Engineer on 20 November 2019.

2.2 Subsurface Investigation

A subsurface investigation was carried out concurrently with the walkover inspection. The investigation included the auger drilling of two boreholes (BH1 and BH2) to depths of 5.0m and 5.05m respectively, using a Scout drilling rig. Both boreholes were extended to final depths of 8.45m (BH1) and 8.63m (BH2) using NMLC rotary diamond coring techniques with water flush. A Dynamic Cone Penetration (DCP) test was completed at each borehole location to refusal depths of 3.0m (DCP1) and 1.85m (DCP2).

The borehole locations were set out using tape measurements off the existing building and are shown on the attached Figure 2. The surface RL at each borehole location was estimated by interpolation between spot level heights and ground contour lines shown on the provided survey plan and are therefore approximate. The survey datum is the Australian Height Datum (AHD).

The relative density of the subsoil profile was assessed by interpretation of the DCP test results. In the augered portion of the boreholes, the strength of the bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings. The assessment of rock strength in this way is approximate, and variations of about one order of magnitude should not be unexpected. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests.

Groundwater observations were made during and on completion of auger drilling and on completion of core drilling. We note that water is used as part of the coring process, and therefore water levels at the completion of coring may not have stabilised in the short time period after drilling.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our senior engineering geologist was present on a full-time basis during the fieldwork to set out the borehole locations, nominate in-situ testing and sampling and prepare the attached borehole logs and DCP test results sheet. The Report Explanation Notes define the logging terms and symbols used.

Selected soil samples were returned to a NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, sulphate content, chloride content and resistivity testing. The results are presented in the attached Envirolab Services Certificate of Analysis Ref. 191912.

The recovered rock cores were photographed and returned to Soil Test Services Pty Ltd for Point Load Strength Index testing. The colour core photographs are enclosed with the relevant cored borehole log. The Point Load Strength Index test results are plotted on the borehole logs and are also summarised in the attached STS Table A. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in STS Table A.

Testing of the site soils and groundwater for possible contamination was outside the agreed scope of this investigation.

3 SUMMARY OF OBSERVATIONS

The site is located towards the toe of a moderately to steeply sloping north-west facing hillside, which generally grades at approximately 20° to 30°. There are locally steeper slopes in areas of cut, which grade at approximately 36° to 50°. Bobuck Lane bounds the site to the north.

At the time of the fieldwork, the site contained a three storey brick and weatherboard lodge building with a ground floor level estimated to be between 1m to 2m higher than Bobuck Lane. The building appeared to be in good external condition, based on a cursory inspection from within the site. A garden was located on the northern side of the building adjacent to its western portion and was supported by a mortared granite block retaining wall to a height of approximately 2m above Bobuck Lane. A narrow parking lane was located along the toe of the retaining wall. A staircase was located on the western side of the building. Refer to Plate 1 below. The rear yard was covered with grass, scattered medium to tall trees and occasional granite cobbles and boulders, up to 0.7m size. A granite block retaining wall supported the lower northern portion of the rear yard to heights between approximately 1.4m to 1.8m. The retaining walls on site appeared to be in good condition.

The site appeared to be well drained.

Plate 1: Looking upslope to the south-east showing The R.A.N. Ski Club building.

The neighbouring three storey brick lodge buildings to the east (30 Bobuck Lane) and west (34 Bobuck Lane) of the site were respectively set back approximately 3.8m and 2.1m from the building on site. The neighbouring two storey lodge building uphill to the south of the site (6 Bobuck Lane) was set back at least 8m from the proposed extension footprint. These neighbouring buildings all appeared to be in generally good external condition, based on a cursory inspection from within the site.

We did not observe any obvious sign of deep seated hillside instability, such as slumping, tension cracks, etc, at, or in the immediate vicinity of, the site. However, the basal portion of some tree trunks had a slight basal curvature, indicating possible hillside creep.

4 SUBSURFACE CONDITIONS

- The 1:250,000 geological map of Tallangatta (SJ 55-3) indicates the site is underlain by Undifferentiated Lower Devonian Volcanics, comprising 'G*ranite, granodiorite and tanalite'*. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. A summary of the encountered subsurface conditions is presented below:Topsoil comprising silty sand was encountered at the surface of both boreholes and extended down to depths of 0.3m (BH1) and 0.2m (BH2).
- Residual soils comprising silty sand, clayey sand and sand were encountered below the topsoil in both boreholes and extended down to depths of 2.5m (BH1) and 2.0m (BH2). The relative density of the residual soils ranged from very loose to dense/very dense. The relative density increased with depth. In DCP2, refusal is inferred to have occurred within the very dense soil profile soil.
- Granite bedrock was encountered in both boreholes at depths of 2.5m (BH1) and 2.0m (BH2) and extended down to the borehole termination depths. The granite bedrock was of poor quality being initially extremely weathered and comprising a dense sand, improving to highly weathered and of very low to low strength with depth. The cored bedrock contained defects including inclined joints and

extremely weathered seams. 'No core' zones were encountered at the commencement of coring in both boreholes and were 1.32m (BH1) and 1.11m (BH2) thick. 'No core' zones are usually the result of 'weaker' rock being washed out by the drill flush water. The full recovery of drill flush water during coring indicates the granite bedrock was relatively impermeable.

- Both boreholes were 'dry' during and on completion of auger drilling. On completion of rock coring, groundwater was measured at a depth of 1.0m in both boreholes, however these levels were most likely influenced by the introduced drill flush water. No long term groundwater level monitoring was carried out.

5 LABORATORY TEST RESULTS

The results of the axial Point Load Strength Index tests carried out on the recovered rock cores from BH1 & BH2 correlated well with our field assessment of bedrock strength. The estimated UCS's, based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (ie. UCS = 20 x $I_{S(50)}$), ranged from 1MPa to 6MPa.

A summary of the soil chemistry test results is provided in the table below:

Borehole	Sample Depth (m)	Description	pH Units	Sulphate (mg/kg)	Chloride (mg/kg)
1	4.5 – 4.7	Extremely Weathered granite	6.0	<10	<10
2	0.2 – 0.7	Residual Silty SAND	6.3	<10	<10
2	1.6 – 2.0	Residual SAND	6.0	<10	<10

6 GEOTECHNICAL RISK ASSESSMENT

The results of our walkover inspection and subsurface investigation have been used to prepare our geotechnical risk assessment for the proposed development. The methodology adopted is in accordance with the Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, risk classification system.

6.1 Potential Landslide Hazards

We consider that the following potential landslide hazards are associated with the site and the proposed development:

A Instability of the hillside slope (shallow earth slide).

B Instability of the hillside slope (slow creep movement).

C Instability of cobbles/boulders on the slope surface (rock fall/topple).

D Instability of existing retaining walls.

E Instability of temporary excavation batter slopes.

F Instability of proposed retaining walls.

We have considered the potential landslide hazard of a deep seated failure through the hillside. However, as granite bedrock is present at relatively shallow depth, the likelihood of such failure from occurring has been assessed to be 'Barely Credible' ie. the event is' inconceivable' or 'fanciful' over the design life, and therefore has not been considered any further in this report.

We note that potential landslide hazard D will no longer be present following demolition, as the retaining wall along the front of the site and at the north-western corner of the site will be demolished.

6.2 Risk Analysis

The attached Tables A and B provide details in relation to the potential landslide hazards A to F and our assessment of factors relevant in the assessment of the risk to both property and life.

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the potential landslide hazard occur. 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 the assessed risk to property for the existing and proposed conditions, with the risk being Very Low and Low, which would be considered to be 'Acceptable' in accordance with the Australian Geomechanics Society (2007c) risk classification system.

We have also used the indicative probabilities associated with the assessed likelihood to calculate the risk to life for the person most at risk for each of the potential landslide hazards following construction. The results of our assessment are presented in Table B. Included in Table B are our assessed temporal, spatial, evacuation and vulnerability factors that have been used for the risk calculation.

The resulting risk for the person most at risk is less than 10^{-6} which would be considered to be 'Acceptable' in accordance with the Australian Geomechanics Society (2007c) risk classification system.

6.3 Risk Assessment

It should be 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.

Our assessment of the probability of failure of existing structural elements such as retaining walls is based upon a visual appraisal of their type and condition at the time of our walkover inspection.

In preparing our recommendations, 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 buried services within and surrounding the site are, and will be, regularly maintained to remain in good condition.

It is important to be mindful that soil slumps, rock falls, etc can occur at anytime and it would be difficult to impossible to predict when the identified potential geotechnical hazards will occur. Also, we cannot predict when an extreme or unusual event may occur (such as an earthquake or 1 in 100 year rainfall event etc) and what impact it would have on the stability of the identified potential geotechnical hazards.

7 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 current risk of instability of the site and to control future risks.

The recommendations which follow address geotechnical issues only and other conditions may be required to address other aspects of the proposed development.

7.1 Conditions to be Provided to Establish the Design Parameters

7.1.1 New footings to be founded in the underlying granite bedrock. Pad and strip footings are appropriate where the depth of founding is less than about 1m. Piled footings, such as bored piles but with an allowance for temporary or permanent liners would be required where the depth to the bedrock is more than about 1m. An allowance should be made for localised excavation and removal of possible corestones within the residual soil profile during pile drilling.

7.1.2 Pad and strip footings founded at least 0.4m into extremely weathered granite bedrock should be designed for a maximum allowable bearing pressure of 600kPa. If the footings are founded in at least very low strength bedrock, the maximum allowable bearing pressure can be increased to 800kPa. Bored piles can be designed using similar allowable end bearing pressures of 600kPa and 800kPa, as described above.

7.1.3 All footings must be founded below a 45° line inclined up from the toe of an adjacent retaining wall.

7.1.4 The major consideration in the selection of earth pressures for the design of retaining walls, piled shoring walls and/or soil nail walls is the need to limit deformations occurring outside the excavations. The following characteristic earth pressure coefficients and subsoil parameters may be adopted. Note our comments also in Section 7.2.1 below.

- For free-standing cantilever walls which are retaining areas where minor movements can be tolerated (i.e. landscape walls), a triangular lateral earth pressure distribution may be adopted with an 'active' earth pressure coefficient, K_a, of 0.35, for the soil and extremely weathered granite bedrock profiles, assuming a horizontal backfill surface.

- For cantilever walls where the tops are restrained by the permanent structure or which retain areas where movements are to be reduced or for propped walls, a triangular lateral earth pressure distribution should be adopted with an 'at rest' earth pressure coefficient, K_o, of 0.55, for the soil and extremely weathered granite bedrock profiles, assuming a horizontal backfill surface.
- Propped or anchored contiguous pile walls may be designed based on a rectangular lateral earth pressure distribution of 6H kPa for the retained profile, where 'H' is the retained height of soil and weathered rock in metres. For the western and eastern sides of the basement excavation immediately adjacent to the existing building and neighbouring lodge to the west, it may be necessary to increase the lateral earth pressure distribution to 8H kPa for the retained profile, to reduce the wall movements.
- A bulk unit weight of $21kN/m^3$ should be adopted for the soil and extremely weathered granite bedrock profiles.
- Any surcharge affecting the walls (eg. traffic loading, construction loads, footing loads, inclined backfill surface, etc) should be taken into account in the wall design using the appropriate earth pressure coefficient from above.
- The retaining walls should be designed as drained and measures taken to provide complete and permanent drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric (eg. Bidim A34) to act as a filter against subsoil erosion. For contiguous pile walls, the drainage should comprise weepholes made up of, say, 50mm PVC pipes which are grouted into gaps or holes between adjacent piles at say, 1.2m horizontal spacing and located just above the proposed basement floor level. The embedded end of the weepholes must be covered by a non-woven geotextile filter fabric, such as Bidim A34, to act as a filter against subsoil erosion.
- Lateral toe restraint may be achieved by suitably embedding the retaining wall footing to sufficient depth. The embedment depth design should be based on a triangular lateral earth pressure distribution and a 'passive' earth pressure coefficient, K_p, of 3, assuming horizontal ground in front of the wall. We note that significant movement is required in order to mobilise the full passive pressure of a soil, and therefore a factor of safety of at least 2 should be adopted to reduce such movements. Any localised excavations, such as for buried services, in front of the walls should be taken into account in the embedment design. Alternatively, lateral toe restraint may be achieved by keying the retaining wall into granite bedrock. An allowable lateral stress of 150kPa may be adopted for key design.
- For contiguous piles embedded into bedrock below bulk excavation level, an allowable lateral toe resistance of 150kPa may be adopted. Piles should be socketed at least 1m below the base of the proposed excavation, including local excavations for buried services, footings, trenches, the lift pit, etc. A greater pile embedment may be required for stability of the wall particularly for piles located directly behind the basement retaining walls which provide lateral support for the ground floor level and the proposed wall at the rear of the site.
- Temporary anchors bonded into weathered granite bedrock may be designed for an allowable bond stress of 100kPa. The anchors should have a bond length of at least 3m and a free length of at least 4m, with the bond located beyond a 45° line inclined up from bulk excavation level. All anchors should be proof-loaded to at least 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor, with anchors 'locked off' at about 85% of the design working load. Lift-off tests should be carried out on at least 15% of the anchors

3-4 days following locking off to confirm that the anchors are holding their load. We recommend that only experienced contractors be considered for the anchor installation and stressing.

- Temporary anchors are expected to extend below at least the neighbouring property to the west and as such permission must be sought from the respective property owners, prior to installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping.
- We assume that permanent lateral support of the shoring walls will be provided by the proposed structure
- For a soil nail wall design, the following soil parameters should be adopted:

Material	Effective Cohesion, c' (kPa)	Effective Friction Angle, ϕ'	Bulk Unit Weight, γ (kN/m^3)
Soil Profile	0	30˚	18
Extremely Weathered (or better) Granite	10	40˚	20

- The estimated ultimate pull out (bond) resistance for each soil nail will be a function of the depth of embedment, bulk unit weight, effective cohesion and the effective friction angle of the soil/weathered rock. The pull out resistance will need to be confirmed by pull out testing.
- Permanent soil nails will need to be designed with due regard for long term corrosion protection, i.e. fully grouted, hot dipped galvanised and provided with a sacrificial thickness or using stainless steel bars.

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

7.1.6 Based on the Envirolab Services test results, a 'non-aggressive' exposure classification is applicable in accordance with AS2159-2009 for concrete piles.

7.2 Conditions to be Incorporated into the Detail Design to be Submitted for the CC

7.2.1 As the proposed excavation will extend close to the western boundary and the western portion of the section of the building that will remain, temporary batters as outlined in Section 7.3.6 below will not be possible and therefore retaining walls will be required to support the excavation cuts. Temporary batters are not recommended for the relatively deep cuts along the southern side of the excavation, as this may initiate slope instability. The retaining walls will need to be engineer designed. Based on our experience in Thredbo for a similar development on a steep site (ie. Mittabah Lodge), we recommended that the excavation cuts be supported by a reinforced shotcrete wall supported by permanent soil nails/rock bolts i.e. a soil nail wall. The wall will need to be excavated and constructed in stages, say maximum 1.5m high 'lifts'; to be confirmed by a numerical soil nail wall analysis and design. Design parameters for a soil nail wall are presented in Section 7.1.4 above. Further, design of the soil nail wall will need to be carried out by a geotechnical engineer, based on a numerical analysis of the proposed excavation.

7.2.2 An alternative retaining wall design, includes construction of contiguous pile retaining walls. We forewarn however that contiguous piles may be difficult to drill to the required depths, particularly if large granite corestones are encountered in the weathered rock profile. The steep site and relatively limited site access even after demolition will probably limit the size of a piling rig that can be brought to site and achieving the required embedments of the shoring piles may not be feasible. Bored piles, if selected, will need to include an allowance for sacrificial liners.

7.2.3 If contiguous pile walls are to be considered, we strongly recommend that a full copy of this report be provided to the prospective piling contractors so that appropriate pile drilling equipment is brought to site, including an allowance to drill through high or very high strength granite corestones, if encountered. We recommend that all pile holes in excess of 3m depth be tremie poured to reduce the potential for concrete segregation.

7.2.4 All structural drawings must be reviewed by a geotechnical engineer who should endorse that the recommendations contained within this report, and any subsequent soil nail wall design (or other retaining wall design) report, have been adopted. This will be part of the Form 2 requirements.

7.3 Conditions Applying to the Construction Phase

7.3.1 Prior to the commencement of demolition and excavation, we recommend that dilapidation surveys be completed on the neighbouring lodge buildings to the west, east and south. The dilapidation surveys should include a detailed internal and external inspection of the buildings, where all defects including defect location, type, length and width are rigorously described and photographed. The respective owner should be asked to confirm that the dilapidation survey report present a fair record of existing conditions. The dilapidation survey report may then be used as a benchmark against which to assess possible future claims for damage.

7.3.2 During excavation, we recommend that the excavation not extend below the base of any adjacent footings supporting the remaining portion of the lodge building, without approval from a geotechnical engineer. On this basis, test pits must be excavated when access is possible (say during or immediately following demolition) in the presence of a geotechnical engineer, to attempt to assess the existing footing details and foundation materials and to provide advice on underpinning or other temporary support, if required. We expect some underpinning or allowance in the shoring design will be required, and so the project program should allow for this rather than the alternative of assuming excavation will immediately follow demolition.

7.3.3 Excavation will be required over the footprint of the proposed extension to achieve design levels. Excavation through the soil, extremely low strength granite and very low strength granite can be completed using a 'digging' bucket fitted to a large hydraulic excavator, but with assistance of a ripping attachment. Excavation of the very low to low strength (or higher) granite bedrock would be most effectively completed using hydraulic rock hammers. The rock hammers would also be required for detailed rock excavations such as for footings, trenches, the lift pit, etc.

7.3.4 Excavation using hydraulic rock hammers will need to be strictly controlled as there will probably be direct transmission of ground vibrations to the neighbouring buildings and buried services. We recommend that continuous quantitative vibration monitoring be carried out whenever rock hammers are used, as a precaution against possible vibration induced damage. By referencing the German Standard DIN4150-3:1999-02, the vibrations on the neighbouring lodge buildings to the west, east and south should be limited to a peak particle velocity (PPV) of 5mm/s, subject to review of the dilapidation survey reports by the geotechnical and structural engineers. It should be noted that if this vibration limit is exceeded, the vibrations should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable depending on the associated vibration frequency. However, these vibrations may still result in perceived discomfort or concern to occupants of the neighbouring buildings. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to change to a smaller rock hammer. Appropriate alternative rock excavation techniques to reduce vibrations and therefore reduce vibration monitoring requirements to periodic, include using a rock grinder on the excavator, or a large excavator mounted rock saw to grid saw the bedrock into blocks that could then be removed using a ripping tyne attachment to the excavator. At the commencement of rock excavation using these alternative techniques, we recommend that a short period of quantitative vibration monitoring be undertaken. If excessive vibrations are confirmed, it will be necessary to use lower energy techniques and/or equipment such as increasing the number of rock saw cuts or hand held rock splitting methods. If vibrations are close to but below the threshold levels, and the excavation techniques are not modified, full time quantitative vibration monitoring would be warranted'

7.3.5 Groundwater inflows into the excavation are expected as localised seepage flows at the soil/rock interface and/or through joints within the bedrock profile, particularly after heavy or prolonged rain. Seepage volumes into the excavation, if any, are expected to be small and controllable by gravity drainage down to Bobuck Lane. Notwithstanding, groundwater seepage monitoring should be carried out during excavation by the builder and geotechnical engineer so that any unexpected conditions can be addressed.

7.3.6 Where feasible within the site geometry and for excavation cuts less than 1.5m deep (where no neighbouring footings are in close proximity, excavations through soil and granite bedrock may be temporarily battered to slopes no steeper than 1 Vertical (V) in 1.5 Horizontal (H), provided surcharge loads are kept well clear of the crest of the temporary batters. Steeper batters may be possible subject to geotechnical inspection. Retaining walls can then be constructed along the toe of the temporary batters and subsequently backfilled.

7.3.7 Based on the investigation results, we expect that the basement and ground floor slabs will directly overlie granite bedrock. We therefore recommend that underfloor drainage be provided. The under-floor drainage should comprise perimeter drains comprising a geofabric wrapped perforated drainage pipes in a gravel filter. Consideration may be given to an additional drain below the central section of the floor slab (orientated east-west). The under-floor drainage must connect to the stormwater system for controlled disposal. Joints in the basement and ground floor concrete floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints. The building floor slab over the bedrock subgrade should also be provided with at least a 20mm to 30mm thick 'clean' sand bedding layer to provide a de-bonding layer to limit the potential for shrinkage cracking or curling of the slabs.

7.3.8 Pad and strip footing excavations and drilling of piles must be inspected by a geotechnical engineer/engineering geologist to confirm that a satisfactory bearing stratum has been achieved. This inspection will form part of the Form 3 requirements.

7.3.9 The geotechnical engineer must inspect all subsurface drains, including those behind retaining walls, prior to backfilling.

7.3.10 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 builder, and repaired if found to be leaking.

7.4 Conditions Relating to On-Going Management of the Site/Structure

7.4.1 Any surface and subsurface drains around the lodge should be checked and cleared of all debris on a regular basis. We recommend this forms part of an annual maintenance plan to be carried out by the owners.

8 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between and beyond the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

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.

TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	EXISTING CONDITIONS				DURING CONSTRUCTION AND AFTER COMPLETION OF THE PROPOSED DEVELOPMENT AND IMPLEMENTATION OF RECOMMENDATIONS AS OUTLINED IN REPORT				
	A: Instability of the hillside slope (Shallow Earth Slide)	B: Instability of the hillside slope (Slow Creep Movement)	C: Instability of cobble/boulder on surface (Rock Fall/ Topple)	D: Instability of existing retaining walls	A: Instability of the hillside slope (Shallow earth slide)	B: Instability of the hillside slope (Slow creep movement of the soils)	C: Instability of cobble/boulder on surface (rock fall/ topple)	E: Instability of temporary excavation batter slopes	F: Instability of proposed retaining walls
Assessed Likelihood	Unlikely[1]	Almost Certain	Unlikely	Unlikely[2]	Unlikely	Almost Certain	Unlikely	Unlikley[3]	Rare[4]
Assessed Consequences	Minor	Insignificant	Minor	Minor	Minor	Insignificant	Minor	Insignificant	Medium
Risk	Low	Low	Low	Low	Low	Low	Low	Very Low	Low
Comments	1 No obvious signs of near surface or deep seated instability observed. Granite bedrock is present at relatively shallow depth. 2 The existing retaining walls were observed to be in good condition and assumed to be engineer designed. 3 Assumes the recommended temporary batter slopes will be adopted in accordance with our advice in Section 6. 4 Assumes the retaining walls will be engineer designed in accordance with our advice in Section 6.								

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE
AFTER COMPLETION OF CONSTRUCTION

Potential Landslide Hazard	**A: Instability of the hillside slope (Shallow Earth Slide)**	**B: Instability of the hillside slope (Slow Creep Movement)**	**C: Instability of cobble/boulder on surface (Rock Fall/ Topple)**	**E: Instability of temporary excavation batter slopes**	**F: Instability of proposed retaining walls**
Assessed Likelihood	Unlikely	Almost Certain	Unlikely	Unlikely	Rare
Indicative Annual Probability	10^{-4}	10^{-1}	10^{-4}	10^{-4}	10^{-5}
Persons at Risk	Occupant inside building			Construction worker at crest or toe of batter	Occupant inside building
Number of Persons Considered	1				
Duration of Use of Area Affected (Temporal Probability)	12 hours per day for 4 months of the year (ski season) = 0.17			8 hours per day for 3 months during construction = 0.08	12 hours per day for 4 months of the year (ski season) = 0.17
Spatial Probability Taking Into Account Travel Distance and Travel Direction	0.2 Assume 1m wide slump over 5m wide section	1 Entire hillside	0.14 Assume maximum 0.7m wide boulder per 5m wide section	0.1 Assume 1m wide slump over 10m wide section of excavation.	0.3 Assume 3m wide failure over 10m wide section of retaining wall
Probability of not Evacuating Area Affected prior to Failure	0.1 Prior warning likely	0.001 Slow creep movement	0.1 Prior warning likely	0.4 Prior warning likely. Assumes recommended batter slopes will be adopted	0.4 Prior warning likely. Assumes retaining walls will be engineer designed
Vulnerability to Life if Failure occurs whilst Person is Present	0.01 Small volume expected, unlikely to be buried	0.01 Unlikely to be buried	0.5 Possible fatal consequence if impacted	0.1 Relatively shallow cuts expected where temporary batters are permitted and therefore unlikely to be buried	0.5 Unlikely to be buried
Risk for Person Most at Risk	3.4×10^{-9}	1.7×10^{-7}	1.2×10^{-7}	3.2×10^{-8}	1×10^{-7}

115 Wicks Road
Macquarie Park NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31441RH
Project:	Proposed Extension	**Report:**	A
Location:	Bobuck Lane, Thredbo, NSW	**Report Date:**	15/05/2018
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	6.37 - 6.41	0.03	1
	6.86 - 6.90	0.1	2
	7.19 - 7.22	0.3	6
	7.95 - 8.00	0.2	4
	8.41 - 8.45	0.09	2
2	6.29 - 6.33	0.03	1
	6.96 - 7.00	0.03	1
	7.40 - 7.44	0.07	1
	7.87 - 7.91	0.07	1
	8.58 - 8.63	0.03	1

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 191912

Client Details

Client	JK Geotechnics
Attention	A Frost
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31441RH, Thredbo**
Number of Samples	3 Soil
Date samples received	17/05/2018
Date completed instructions received	17/05/2018

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	24/05/2018
Date of Issue	22/05/2018

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil				
Our Reference		191912-1	191912-2	191912-3
Your Reference	UNITS	1	2	2
Depth		4.5-4.7	0.2-0.7	1.6-2.0
Date Sampled		08/05/2018	08/05/2018	08/05/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	18/05/2018	18/05/2018	18/05/2018
Date analysed	-	18/05/2018	18/05/2018	18/05/2018
pH 1:5 soil:water	pH Units	6.0	6.3	6.0
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	<10

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	191912-2
Date prepared	-			18/05/2018	1	18/05/2018	18/05/2018		18/05/2018	18/05/2018
Date analysed	-			18/05/2018	1	18/05/2018	18/05/2018		18/05/2018	18/05/2018
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	6.0	6.0	0	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	101	93
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	112	88

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1 / 2

BOREHOLE LOG

Client: THE R.A.N. SKI CLUB LIMITED
Project: PROPOSED EXTENSION
Location: 32 BOBUCK LANE, THREDBO, NSW

Job No.: 31441RH **Method:** SPIRAL AUGER **R.L. Surface:** ~1370.7 m
Date: 8/5/18 **Datum:** AHD
Plant Type: SCOUT **Logged/Checked By:** A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS					TOPSOIL: Silty sand, fine to medium grained, brown, trace of clay fines, fine grained quartz gravel and root fibres.	M			GRASS COVER
ON COMPLETION OF CORING						1370, 1369	1, 2		SC	Clayey SAND: fine to coarse grained, orange brown, trace of silt fines and fine grained igneous gravel.		(L) (MD)		RESIDUAL
						1368, 1367, 1366	3, 4		-	Extremely Weathered granite: clayey SAND, fine to coarse grained, orange brown, with igneous gravel.	XW	(D - VD)		UNDIFFERENTIATED LOWER DEVONIAN VOLCANICS VERY LOW 'TC' BIT RESISTANCE
						1365, 1364	5, 6			REFER TO CORED BOREHOLE LOG				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1

CORED BOREHOLE LOG

Client: THE R.A.N. SKI CLUB LIMITED
Project: PROPOSED EXTENSION
Location: 32 BOBUCK LANE, THREDBO, NSW

Job No.: 31441RH | **Core Size:** NMLC | **R.L. Surface:** ~1370.7 m
Date: 8/5/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: SCOUT | **Bearing:** N/A | **Logged/Checked By:** A.F./A.J.H.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness — Specific / General	Formation
		1366									
			5		START CORING AT 5.00m						
100% RETURN		1365			NO CORE 1.32m						UNDIFFERENTIATED LOWER DEVONIAN VOLCANICS
			6								
		1364			GRANITE: coarse grained, orange brown.	HW	VL - L			(6.42m) J, 10°, P, R, Fe Sn (6.54m) XWS, 10°, Cn (6.74m) J, 20°, P, R, Fe Sn (6.99m) XWS, 0°, Cn	
			7		as above, but orange brown and grey.					(7.18m) J, 20°, P, R, Cn (7.29m) J, 20°, P, R, Cn (7.39m) J, 10°, Un, R, Fe Sn (7.67m) J, 30°, P, R, Fe Sn (7.73m) J, 70°, P, R, Fe Sn	
		1363	8		as above, but orange brown.						
		1362			END OF BOREHOLE AT 8.45 m						
			9								
		1361	10								
		1360									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB NO. 31441RH BH1 START CORING AT 5.00m
5
CORE LOSS 1.32m
6
7
8
END OF BOREHOLE AT 8.45m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 2

1 / 2

Client: THE R.A.N. SKI CLUB LIMITED
Project: PROPOSED EXTENSION
Location: 32 BOBUCK LANE, THREDBO, NSW

Job No.: 31441RH **Method:** SPIRAL AUGER **R.L. Surface:** ~1372.0 m
Date: 9/5/18 **Datum:** AHD
Plant Type: SCOUT **Logged/Checked By:** A.F./A.J.H.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	REFER TO DCP TEST RESULTS				TOPSOIL: Silty sand, fine to medium grained, brown, trace of clay fines and root fibres.	M			GRASS COVER
				SM	Silty SAND: fine to medium grained, brown, trace of fine igneous gravel, and clay fines.		(VL)		RESIDUAL
ON COMPLETION OF CORING		1371	1	SC	Clayey SAND: fine to coarse grained, orange brown, trace of silt fines, and fine grained igneous gravel.		(MD) / (D)		
				SP	SAND: fine to coarse grained, orange brown, with fine to coarse grained igneous gravel, trace of clay and silt fines.				
		1370	2	-	GRANITE: fine to coarse grained, orange brown.	HW	(M)		UNDIFFERENTIATED LOWER DEVONIAN VOLCANICS
		1369	3	-	Extremely Weathered granite: SAND, fine to coarse grained, orange brown, with igneois gravel.	XW	(D - VD)		MODERATE TO HIGH 'TC' BIT RESISTANCE PROBABLE CORESTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
		1368	4						
		1367	5		REFER TO CORED BOREHOLE LOG				
		1366	6						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2

CORED BOREHOLE LOG

Client: THE R.A.N. SKI CLUB LIMITED
Project: PROPOSED EXTENSION
Location: 32 BOBUCK LANE, THREDBO, NSW

Job No.: 31441RH | **Core Size:** NMLC | **R.L. Surface:** ~1372.0 m
Date: 9/5/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: SCOUT | **Bearing:** N/A | **Logged/Checked By:** A.F./A.J.H.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS: SPACING (mm) 600 200 60 20	DEFECT DETAILS: DESCRIPTION Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness Specific General	Formation
		1367	5		START CORING AT 5.05m						
		1366	6		NO CORE 1.11m						UNDIFFERENTIATED LOWER DEVONIAN VOLCANICS
100% RETURN		1365 1364	7 8		GRANITE: coarse grained, orange brown.	HW	VL		600 200 60 20	(6.29m) J, 10°, P, R, Fe Sn (6.42m) J, 10°, P, R, Fe Sn (6.65m) J, 65°, P, R, Clay, 2 mm.t (6.72m) XWS, 0°, 50 mm.t (7.33m) J, 25°, P, R, Fe Sn (7.61m) J, 25°, P, R, Fe Sn (7.77m) J, 50°, P, S, Clay, 2 mm.t (8.09m) J, 50°, Un, R, Clay, 2 mm.t	
		1363 1362	9 10		END OF BOREHOLE AT 8.63 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB NO. 31441RH BH2 START CORING AT 5.05m
5
CORE LOSS 1.11m
6
7
8
END OF BOREHOLE AT 8.63m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THE R.A.N. SKI CLUB LIMITED		
Project:	PROPOSED EXTENSION		
Location:	32 BOBUCK LANE, THREDBO, NSW		
Job No.	31441RH	Hammer Weight & Drop: 9kg/510mm	
Date:	8/5/18 & 9/5/18	Rod Diameter: 16mm	
Tested By:	A.F.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL≈1370.7m	RL≈1372.0m					
Depth (mm)	**1**	**2**					
0 - 100	1	1					
100 - 200	2	2					
200 - 300	3	2					
300 - 400	3	2					
400 - 500	3	2					
500 - 600	2	2					
600 - 700	3	2					
700 - 800	4	2					
800 - 900	3	2					
900 - 1000	4	3					
1000 - 1100	4	5					
1100 - 1200	4	4					
1200 - 1300	3	6					
1300 - 1400	3	12					
1400 - 1500	4	16					
1500 - 1600	3	18					
1600 - 1700	4	13					
1700 - 1800	5	8					
1800 - 1900	6	18/50mm					
1900 - 2000	8	REFUSAL					
2000 - 2100	8						
2100 - 2200	9						
2200 - 2300	6						
2300 - 2400	4						
2400 - 2500	4						
2500 - 2600	7						
2600 - 2700	15						
2700 - 2800	15						
2800 - 2900	20						
2900 - 3000	23 REF						

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD

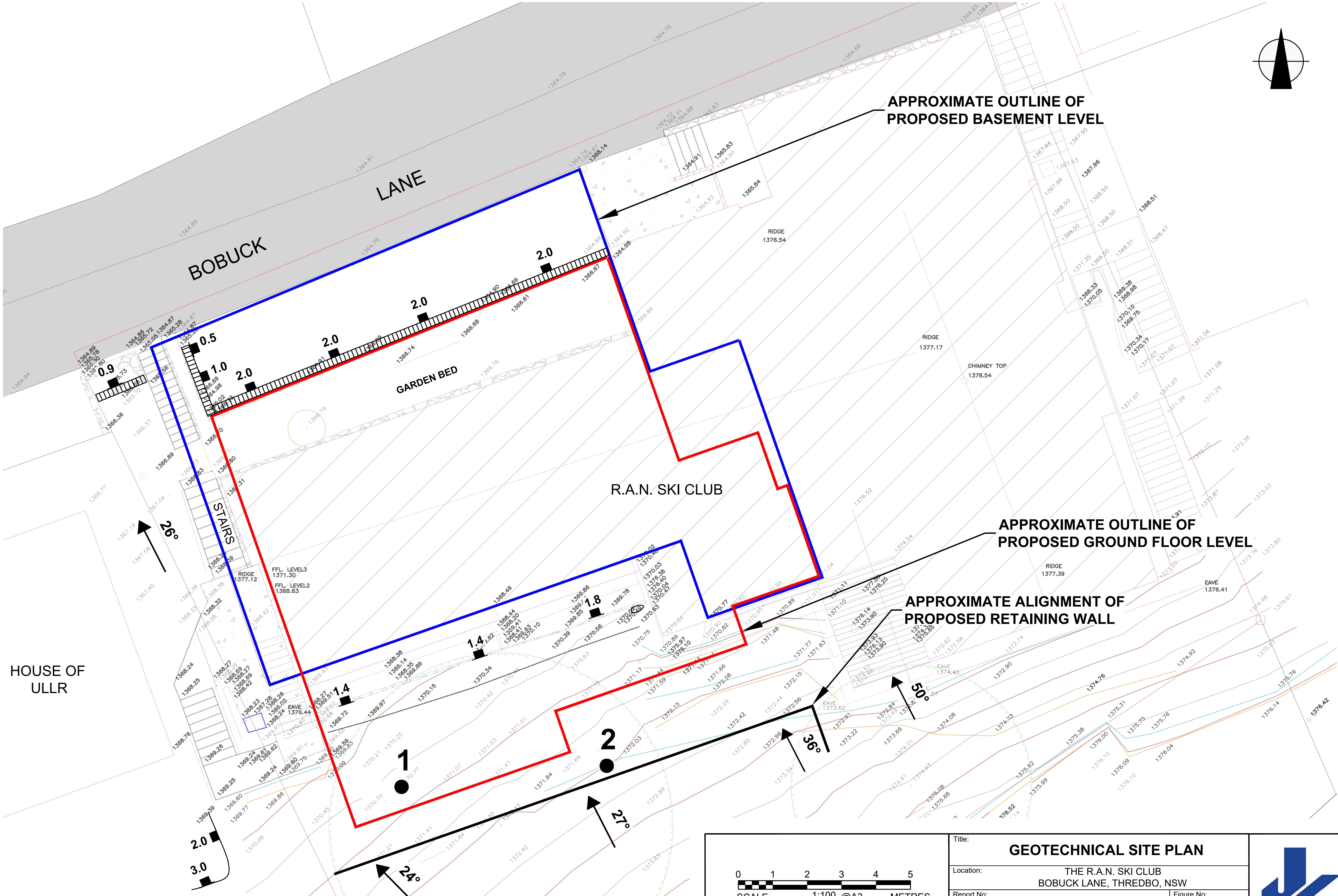
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	THE R.A.N. SKI CLUB BOBUCK LANE, THREDBO, NSW
Report No:	31441RH
Figure No:	1

JKGeotechnics

PLOT DATE: 14/07/2021 1:54:24 PM DWG FILE: Y:\31000's\31441RH THREDBO\CAD\31441RH.DWG



NOTE:
REFER TO FIGURE 3 FOR GEOTECHNICAL MAPPING SYMBOLS.

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title: **GEOTECHNICAL SITE PLAN**

Location: THE R.A.N. SKI CLUB
BOBUCK LANE, THREDBO, NSW

Report No: 31441RH

Figure No: 2

JKGeotechnics

PLOT DATE: 14/07/2021 2:35:59 PM DWG FILE: Y:\31000'S\31441RH THREDBO\CAD\31441RH.DWG

TOPOGRAPHY

Symbol *Ground Profile*

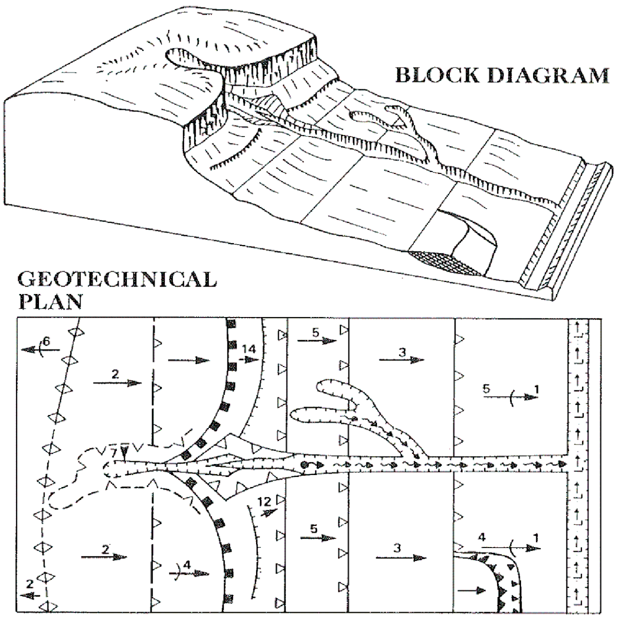
- convex / concave — well defined or angular break of slope
- convex / concave — poorly defined or smooth change of slope
- breaks of slope / changes of slope — convex and concave too close together to allow the use of separate symbols
- sharp / rounded — ridge crest
- 3 — Cliff or escarpment or sharp break 40° or more (estimated height in metres)
- 15 Uniform Slope / 10 Concave Slope / 8 Convex Slope — Slope direction and angle (Degrees)
- Top / Bottom — Cut or fill slope, arrows pointing down slope
- 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
- J —— J 200 — Major joint in rock face (opening in millimetres)
- – T – T – 10 — Tension crack (opening in millimetres)
- Masonry or concrete wall
- Ponding water
- Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

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



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

Title:	GEOTECHNICAL MAPPING SYMBOLS
Location:	THE R.A.N. SKI CLUB BOBUCK LANE, THREDBO, NSW
Report No: 31441RH	Figure No: 3

JKGeotechnics

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.

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	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. These are two main interpretations: (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.

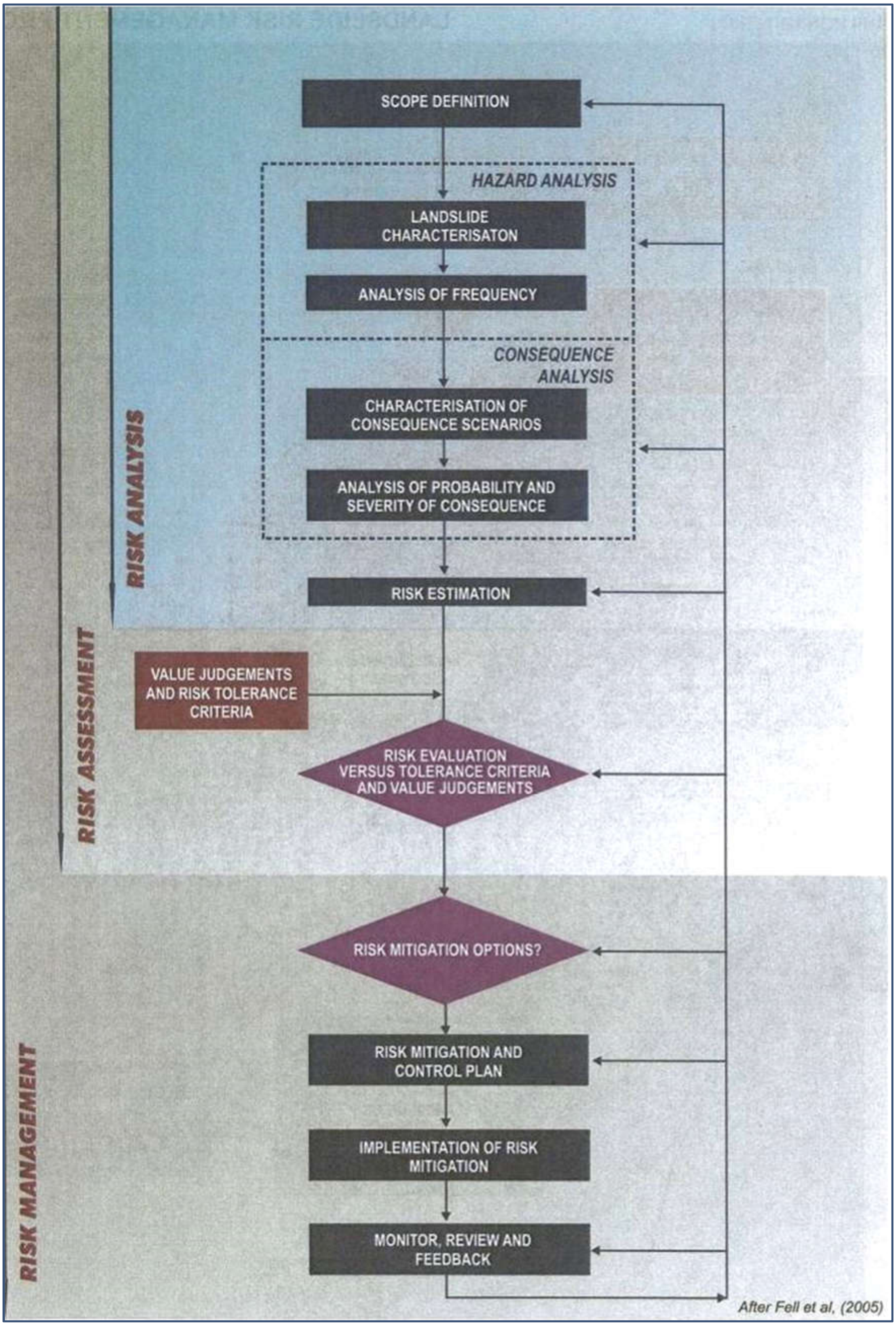


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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. The 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 series 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.

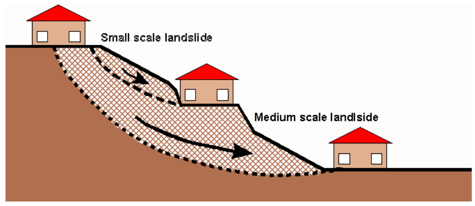


Figure 1

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.

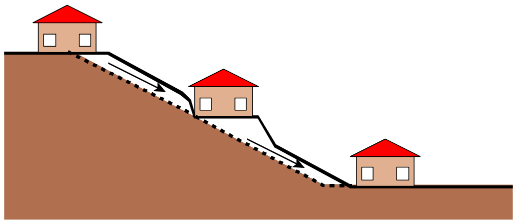


Figure 2

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.

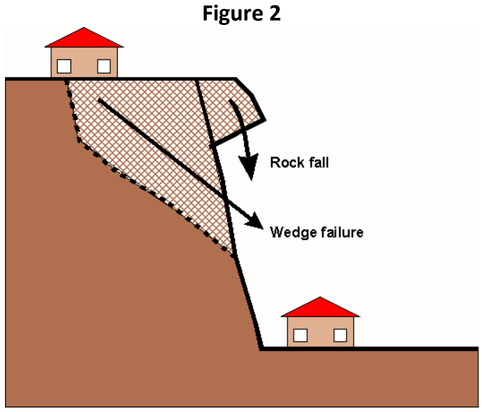


Figure 3

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.

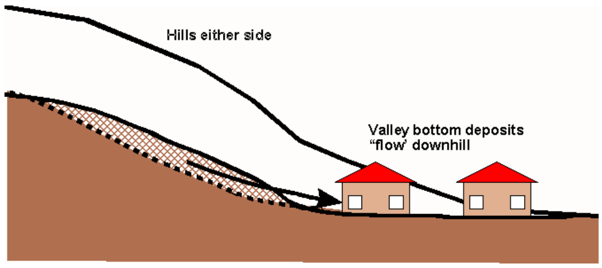


Figure 4

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

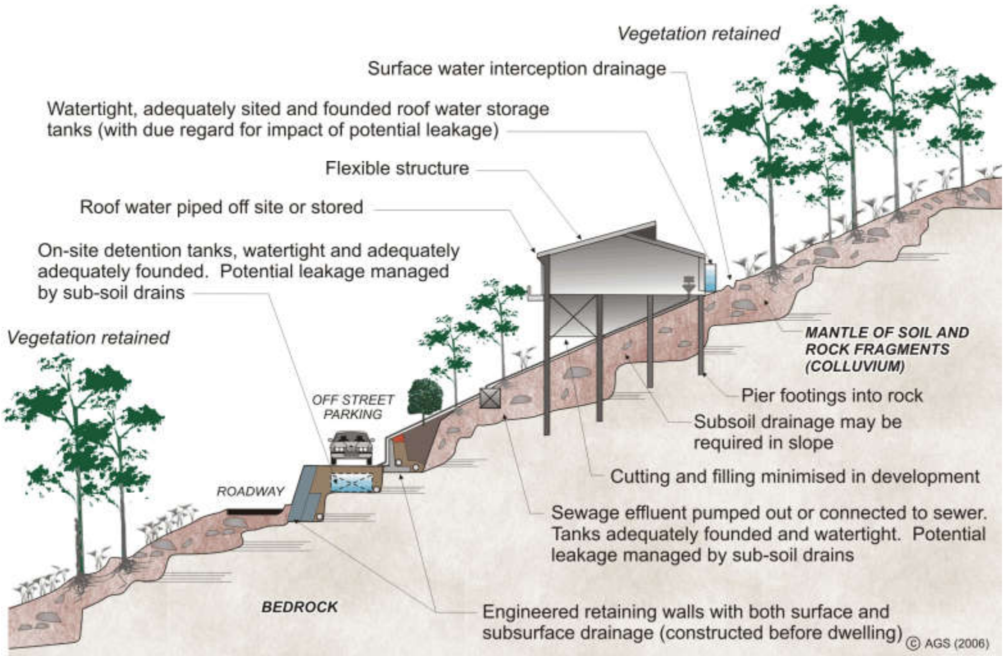
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having 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.

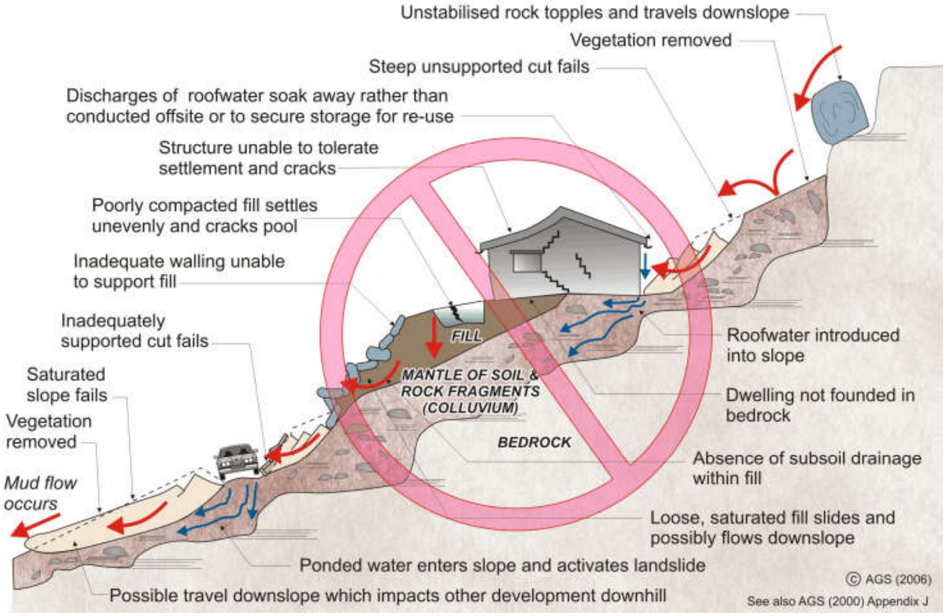
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

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 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.

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_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), 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_o).

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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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 65% 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	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine 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	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ 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:

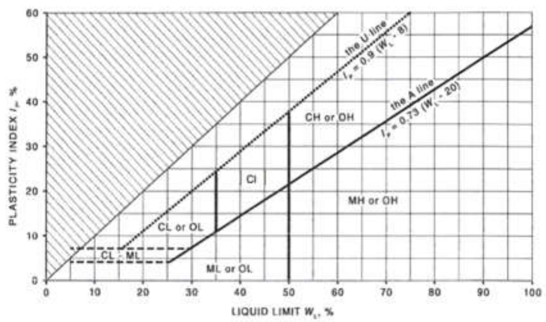
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 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.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	 C	Standing water level. Time delay following completion of drilling/excavation may be shown. Extent of borehole/test pit collapse shortly after drilling/excavation. Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. 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 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE — ≤ 15 — 0 – 4
	L	LOOSE — > 15 and ≤ 35 — 4 – 10
	MD	MEDIUM DENSE — > 35 and ≤ 65 — 10 – 30
	D	DENSE — > 65 and ≤ 85 — 30 – 50
	VD	VERY DENSE — > 85 — > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
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	Hardened steel 'V' shaped bit.	
	'TC' bit	Twin pronged tungsten carbide bit.	
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.	
	Soil Origin	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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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 $I_{s(50)}$ (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
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	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
	– Thickness	mm.t	Defect thickness measured in millimetres