

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

Development Application for Meraki Developments Pty Ltd

Address of site 96-104 Cabarita Road, Avalon Beach, NSW

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

I, Daniel Bliss on behalf of JK Geotechnics
(Insert Name) (Trading or Company Name)

on this the 30 November 2018 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least \$2million.
we/I have:

Please mark appropriate box

- ☐ Prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ We/I ~~are~~am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ Have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. We/I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ☐ Have examined the site and the proposed development/alteration in detail and ~~are~~am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my/our report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.
- ☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Assessment for Proposed Subdivision at 96-104 Cabarita Road, Avalon Beach, NSW

Report Date: 30 November 2018

Report Ref No: 31842BMrpt Rev2

Author: Matthew Pearce

Author's Company/Organisation: JK Geotechnics

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

See text of report:

Mark Hurcum Design Practice- Architects drawings Proj No. 1801 Various Drawing No.s, Issued 26 October 2018, and as finalised (with no changes affecting our assessment) by Rev A dated 2 November 2018, including: A008, {A1. to A9}301; and A002 A/P5 & A005 A dated November 2018

I am We are aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring confirming that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk, as discussed in the Report.

Signature



Name

Daniel Bliss

Chartered Professional Status

MIEAust; CPEng

Membership No.

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Company:

JK Geotechnics

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

Development Application for Meraki Developments Pty Ltd
Name of Applicant

Address of site 96-104 Cabarita Road, Avalon Beach, NSW

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Assessment for Proposed Subdivision at 96-104 Cabarita Road, Avalon Beach, NSW
Report Date: 30 November 2018 Report Ref No: 31842BMrpt Rev2
Author: Matthew Pearce
Author's Company/Organisation: JK Geotechnics

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 3 October 2018
(date)
- ☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (1:600 deemed appropriate in this instance)
- ☐ Subsurface investigation required
- ☐ No Justification
- ☒ Yes Date conducted 14 September 2018
- ☒ Geotechnical model developed and reported as an inferred subsurface type-section
- ☒ Geotechnical hazards identified
- ☒ Above the site
- ☒ On the site
- ☒ Below the site
- ☒ Beside the site
- ☒ Geotechnical hazards described and reported
- ☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Consequence analysis
- ☒ Frequency analysis
- ☒ Risk calculation
- ☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved *recommendations presented in the Report are adopted.*
- ☒ Design Life Adopted:
- ☒ 100 years
- ☐ Other
specify
- ☒ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ☒ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

I am We are aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring confirming that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk as discussed in the Report.

Signature

Name

Chartered Professional Status

Membership No.

Company



Daniel Bliss

MIEAust CPEng

969495

JK Geotechnics



REPORT

TO

MERAKI DEVELOPMENTS PTY LTD

ON

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

FOR

PROPOSED SUBDIVISION

AT

96-104 CABARITA ROAD, AVALON BEACH, NSW

30 November 2018
Ref: 31842BMrpt



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Date: 30 November 2018
Report No: 31842BMrpt
Revision No: 2

Report prepared by:



Matthew Pearce
Associate | Geotechnical Engineer

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Principal | Geotechnical Engineer

For and on behalf of
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TABLE A: SUMMARY OF RISK ASSESSMENT TO PROPERTY

TABLE B: SUMMARY OF RISK ASSESSMENT TO LIFE

STS TABLE A: MOISTURE CONTENT, ATTERBERG LIMITS & LINEAR SHRINKAGE TEST REPORT

BOREHOLE LOGS 1 TO 4 INCLUSIVE

FIGURE 1: SITE LOCATION PLAN

FIGURE 2: BOREHOLE LOCATION PLAN

FIGURE 3: GEOTECHNICAL MAPPING PLAN

FIGURE 4: SITE PLAN SHOWING GEOTECHNICAL HAZARDS

FIGURE 5: SECTION A-A SHOWING POTENTIAL LANDSLIDE HAZARDS

FIGURE 6: SECTION B-B SHOWING POTENTIAL LANDSLIDE HAZARDS

FIGURE 7: GEOTECHNICAL MAPPING SYMBOLS

REPORT EXPLANATION NOTES

APPENDIX A: LANDSLIDE RISK MANAGEMENT TERMINOLOGY



APPENDIX B: SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

APPENDIX C: COPY OF PROPOSED SUBDIVISION PLAN (MHDP REF 1801 A005 A dated November 2018)



1 INTRODUCTION

This report presents the results of our geotechnical assessment for the proposed subdivision of 96 to 104 Cabarita Road, Avalon Beach, NSW. A site location plan is presented as Figure 1. The assessment was commissioned by Ms Katarina Bignasca of Meraki Developments Pty Ltd and was carried out in accordance with our proposal (Ref P47210Z) dated 21 May 2018.

Details of the proposed development are presented in Section 5 below. In summary however, following is proposed:

- subdivision of the land into 10 Community Title lots (9 residential, Lots 2 to 10, and one community lot, Lot 1) and 1 torrens title lot, Lot 11 (first development application),
- construction of 9 individual dwelling houses within the proposed Community Title Lots 2 to 10 (9 development applications concurrent with the application above),
- construction of a common driveway, utilities and services, within the torrens title lot (proposed Lot 1) and,
- the existing house, boat shed and pontoon will be retained within the proposed Lot 11. No works are proposed within this lot or below Mean High Water Mark.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 6 below. It is understood that the report will be submitted to Council as part of the DA documentation. Our report is preceded by the completed Council Forms 1 and 1a.

2 ASSESSMENT METHODOLOGY

2.1 Walkover Survey

This stability assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs, carried out by our Associate geotechnical engineer, Mr Matthew Pearce, on 3 October 2018. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).



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

To supplement our surface observations, a subsurface investigation was also carried out as detailed in Section 2.2 below. Figure 2 shows the location of the boreholes drilled for the subsurface investigation.

Figure 3 presents a geotechnical mapping plan showing the principal geotechnical features present at the site. Figure 4 shows the geotechnical landslip hazards based on our assessment. Figures 2 to 4 are based on the survey plan prepared by Byrne & Associates Pty Ltd (Ref 10937D3 Sheets 1 to 10, all Rev B, dated 22 August 2018). Additional features on Figure 3 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figures 5 and 6 present typical cross-sections through the site based on the survey data augmented by our mapping observations. These sections also demonstrate some of the geotechnical hazards. Note the sections extend beyond the site boundaries for consideration of whether geotechnical hazards above the slope may be affected by the proposed development and similarly such that development effects can be considered on properties downslope of the site.

2.2 Subsurface Investigation

The fieldwork comprised the auger drilling of four boreholes, BH1 to BH4, using our track mounted JK308 drilling rig. Boreholes were drilled to refusal at depths ranging from 2.4m to 5.6m below the existing ground surface.

The borehole locations, as shown on Figure 2, were determined by taped measurements from existing surface features shown on the above referenced survey plan. The surface reduced levels of the boreholes were estimated by interpolation between spot levels shown on the above referenced survey plan, and so should be considered to be approximate. The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and strength of the soils was assessed from the Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on clayey samples obtained from the SPT sampler. The strength of the weathered bedrock was assessed by



observation of the auger penetration resistance when using the Tungsten Carbide (TC) bit, together with examination of the recovered rock cuttings, and subsequent correlation with moisture content test results. It should be noted that assessment of strength using this technique is approximate and variations of up to one strength order should be expected. For more accurate strength assessment cored boreholes would be necessary but were beyond the scope of investigation.

Groundwater observations were made in all boreholes during drilling, on completion of drilling and a short time after completion of drilling. No longer-term groundwater monitoring was carried out.

The fieldwork was completed in the full-time presence of our geotechnical engineer who set out the boreholes, nominated the sampling and testing, and prepared the borehole logs. The logs are attached to this report, together with a glossary of the terms and symbols used in the logs.

Selected samples were returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for testing to determine moisture contents, Atterberg limits and linear shrinkages. The results of the laboratory testing are summarised in the STS Table A.

For further details of the investigation techniques adopted, reference should be made to the attached Report Explanation Notes.

Environmental and contamination testing of the soils was beyond the agreed scope of the works.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follow be read in conjunction with the attached Figures 1 to 6 which are based on a representative survey of conditions at the time of our investigation and walkover.

The site is located on a localised peninsular which extends northwards into Pittwater, forming Careel Bay to the east. The site spans from the middle reaches to the toe of a moderately steep hillside, which locally faces northwards and north-eastwards. The site is located on the southern side of a localised depression in the hillside formed by an intermittent creek.

As shown in the image below, the existing site comprises Nos 96, 100, 102 and 104 Cabarita Road. Collectively they form an irregular shape that extends about 100m from the road way to the shore, and about 110m to 130m parallel to the shore and road (north-west to south-eastwards). Surface



reduced levels (RLs) range from about RL30.4mAHD at the southern corner to RL1.0mAHD at the beach. Overall slopes are in the order of 15° (eastern side) to 20° (central and western sides) but localised slopes generally range from 10° to 23° with some flatter terraces, beach side flats and only two steeper slopes, described further below.



At the time of the site walkover, previous structures within No 96 and No 100 had been cleared except for a few small concrete slabs, numerous low height retaining walls and some stone paved paths. The majority of the trees shown on the survey remained interspersed across the entire site. A snaking vehicular access track had been formed where the house of No 96 once was present. Between the tracks were batter slopes of about 30° and up to about 2m in height, and minor unsupported cuttings to the south of this area. The cuttings exposed thick topsoil fill over gravelly sandy fill. The central portion of No 96 had remnants of a previous house.

Sloping down (from a bend in Cabarita Road) at about 12° was a concrete driveway which looped around (within No 100) but also provided access to Nos 104 and 96. Within the loop was a rounded hillock with a few sandstone exposures at the ground surface although it was not clear if these were outcrops or boulders. The lower, looping portion of the driveway was retained a stone block wall just two-to-three courses high but was in a state of disrepair. A couple of metres downslope was a properly constructed stone masonry retaining wall about 1m high on a concrete footing. Downslope of this wall was a grass slope.



At No. 104 a concrete driveway traversed the slope to the eastern side where there was a one-to-two storey house of weather board construction with a timber balcony and under-croft parking beneath the entire footprint of the house. There was a 2m deep concrete retaining wall supporting land immediately upslope of the house to form the under-croft. The wall appeared in reasonable condition.

The steepest slope within the site was located at the south-western end of No. 100 adjacent to Cabarita Road and sloped at about 45°, but only for a height of 2m. This steep section was about 8m wide, with the crest set back from the kerb by 1.5m to 2m.



As shown to the left, a longer wider slope of about 35° was located at the north-eastern end of Nos 102 and 104. The crest of this slope was about 5m downslope (northwards) of the existing house at No. 104. The slope was generally densely vegetated, but beside some timber steps there was a small exposure of hard residual clayey soil, about 1m height. The toe of the slope was supported by a retaining wall of about 1.6m in height. This slope and the retaining wall extended

about 35m from the bayside pavilion to the boatshed within No. 102. The retaining wall was inclined at about 70° and appeared to be in good condition. Below the retaining wall was a flat garden and patio retained by a sandstone sea wall of about 1m height. Refer to photograph below.





The boat house was a one-to-two storey building of timber clad construction on sandstone block foundation walls. It had a boat ramp and an adjacent jetty. To the north-west of the boat house was the natural toe of the hillslopes and flatter land beside the sandy beach.

There were two stormwater drainage easements within the site. The easement between No. 102 and No. 104 mostly comprised an open concrete culvert, but was piped across the lower bayside garden. The other easement was on the western side of No. 96, extending from No. 90 and comprised a much larger concrete culvert following the above described depression/intermittent creek alignment within the hillside.

The south-western boundary of No. 96 is set back about 2m from the kerb line of Cabarita Road. Along most of this boundary was an old stone wall of about 0.6m in height, which had soil at its crest forming a gently sloping roadside verge. Where it was still intact, the stone wall was significantly tilting. Further along the wall, the upper courses of stone had failed and/or the wall was not evident.



About 1m upslope of the old wall remnants, was a crib-lock wall of about 1m in height supporting a concrete footpath. The path and crib-lock wall was limited to a length of about 20m. The wall, path and asphaltic concrete road appeared to be in good condition with no signs of movement.

Along the southern side of Cabarita Road were residential properties with similar overall moderate to steep hillside slopes, but with cuttings exposing sandstone bedrock and various retaining walls to accommodate driveways and terracing for houses and balconies.

To the west of the site were gardens of residential properties with similar slopes and levels across the common boundaries.

To the east of the No. 100 was a concrete driveway providing access to numerous properties further eastwards but also to No. 106 Cabarita Road, which comprised a one to two storey brick building



set back about 1m from the common boundary with No. 104. Across the common boundaries, slopes and levels were generally similar to those of the subject site.

4 SUBSURFACE CONDITIONS AND LABORATORY TEST RESULTS

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site and surrounds to be underlain by Newport Formation of the Narrabeen Group, comprising interbedded laminite, shale and quartz to lithic quartz sandstone. In summary, the boreholes disclosed a subsurface profile of shallow topsoil and fill over residual sandy clay and silty clay, which graded into weathered sandstone bedrock at depths of 0.6m to 3.4m. No groundwater seepage was encountered within the boreholes. The pertinent details are described below and presented in the sections Figures 5 and 6, but for more detailed descriptions of the conditions encountered at specific locations, reference should be made to the borehole logs.

Topsoil/fill

Sandy silt topsoil was encountered in BH1 to a depth of 0.2m. Fill was present from surface in the other boreholes to depths ranging from 0.2m (BH2) to 0.6m (BH3) and comprised sandy clay and silty sand. The fill appeared to be poorly compacted.

Natural Soil

Natural soil was present in each borehole beneath the fill. Loose silty sand was present from 0.2m to 0.6m depth in BH1. In BH2 to BH4 residual silty sandy clay and sandy clay was encountered and extended to depths of 0.8m (BH2), 2.5m (BH3) and 1.5m (BH4). In BH3 residual silty clayey sand was present to 3.4m depth. The strength of the cohesive soil ranged from very stiff to hard, while the residual sand was assessed to be of medium dense relative density. The silty sandy clay was assessed to be of high plasticity while the sandy clays were assessed to be of low to medium plasticity.

Sandstone Bedrock

Sandstone bedrock was encountered in all the boreholes from the following depths levels:

	BH1	BH2	BH3	BH4
Depth to Rock (m)	0.6	0.8	3.4	1.3
Top of Rock RL (mAHD)	22.9	15.7	9.9	12.6

Other than in BH3, on first contact the sandstone was extremely weathered in the upper 0.3m to 0.7m, and was then distinctly weathered and of very low or low strength. In BH3 the rock was assessed to be distinctly weathered and of very low strength on first contact. The strength of the



sandstone increased with depth with refusal occurring at depths ranging from 2.4 (BH1) to 5.6m (BH2).

Groundwater

No groundwater was encountered during drilling or shortly after completion. It should be noted however, that sufficient time may not have occurred for seepage to ingress and the investigation was completed during a lengthy period of dry conditions.

Laboratory Test Results

The liquid limit of the residual silty sandy clay was 60% and the liquid limit 15% indicating a high potential for reactive soil movements with changes in moisture content. The moisture content was 13.5% which is dry compared with the plastic limit of 23%. Moisture content test results carried out on augered rock cuttings ranged between 3.1% and 7.6% which generally suggest slightly higher strength rock than our field assessment of rock strength. For further details refer to STS Table A.

5 PROPOSED DEVELOPMENT

We understand from the provided architectural drawings (Job No 1801, Drawing Nos A002A/P5, dated November 2018, A004A/P1 and SK201A/P1 dated September 2018, A005 A dated November 2018, and A008/P1, {A1 to A9} 301A/P1 dated October 2018,) prepared by Mark Hurcum Design Practice (MHDP), that the proposed development will comprise the following:

- Subdivision of the vacant No. 96, and No. 100 Cabarita Road into 9 new residential lots (proposed Lots 2 to 10).
- Construction of a new common access driveway (proposed Lot 1) extending approximately west to east from the existing concrete driveway (just south of the existing Lot 100). Although the proposed levels of the driveway are not known, we expect that it will follow the hillside and will require some cut on the southern side and fill on the northern side possibly to depths of about 2m;
- Relocating the stormwater drainage easement to the western extent of the property (No. 96)
- Construction of two to three level houses within each Lots 2 to 10. The houses will step up the hillside slope with stepped cuts typically not greater than 3m depth and horizontal distances between steps of greater than 3m and
- Merging the existing No. 102 and No. 104 Cabarita Road (Lots 8 and 9 of DP62964) into one residential lot (proposed Lot 11) retaining the existing house, lower driveway, pavilion,



boat shed and pontoon. No works are proposed within this lot or below Mean High Water Mark

The layout of the proposed subdivision development is shown on a copy of A005 A in Appendix C.

6 GEOTECHNICAL ASSESSMENT

The overall hillslopes are no greater than the likely angle of repose typical of the soils encountered and there is generally only shallow soil over rock which we expect will have a stepped profile, further reducing potential instability. There are no distinct outcrops, cliff lines, watercourses or depressions present. Generally the slopes are well vegetated with established trees. The steepest of the slopes are also densely covered with low lying vegetation. Some of the trees on the 'steep' 35° slope have minor bends which may indicate slope creep but the inclined retaining wall at the toe of this slope appears to be in good condition and there were no other signs of medium or large scale slope movements.

There are however, numerous low height retaining walls across the site. Some of these appear to have been properly constructed, albeit perhaps not to an engineered design, and appear to be functioning adequately, while other walls comprise simple stacked stone blocks, inclined sandstone flagging and are dilapidated or in some cases have already failed. There is little consequence to most of these low height walls failing as there is sufficient space for natural batters to form and no structures or cause for any persons to spend any significant time below them. Likewise, there were some minor cuttings in the vicinity of the former house No. 96 where the soils should be expected to slump to a stable batter in time without consequence.

The 0.6m to 1.0m high stone wall along the southern boundary of No. 96 is in a state of failure. We expect it was formerly a free standing 'fence' and soil, from road related works, has been pushed up against it. Notwithstanding, the survey shows this wall to be on the property boundary the embankment beyond is therefore council property. Failure of the wall (which it already has in some portions) will result in batters regressing towards the kerbside but they are unlikely to reach the roadway. Failure does not therefore present a road related hazard to the subject property, provided the road verge is maintained by council in due course. In this regard it would appear the crib-lock wall was constructed and has performed successfully on an adverse bend.

The geotechnical assessment provided herein is for the proposed subdivision and for construction of developments within the individual allotments created as per the proposed plans. Should the



details of the proposed cuts and/or filling for the developments on individual lots alter, we should be engaged to review the amendments and validity of our geotechnical advice.

6.1 Potential Landslide Hazards

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

- A. Overall slope failure, slopes typically 10° to 20°.
- B. Localised slope failure of slopes of less than or equal to 30°.
- C. Failure of the steep vegetated slopes of 35° located downslope of No. 104, and upslope of No. 100.
- D. Unsupported cuts in soil greater than or equal to 45° and less than 1.5m in height.
- E. Failure of the existing crib-lock wall at Cabarita Road and other properly constructed walls of less than 1.6m in height
- F. Failure of existing stone walls of less than 1.5m in height
- G. Proposed engineered retaining walls and cut & fill for the access driveway.
- H. Proposed engineered retaining walls for new houses
- I. Local slope failure beneath the new houses

These potential hazards are indicated in schematic form on the attached Figure 4; some of them are also on Figures 5 and 6.

6.2 Risk Analysis

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

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 10^{-6} . This would be considered to be 'acceptable' in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.



6.3 Risk Assessment

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

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

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

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



We consider that our risk analysis has shown that the site and existing and proposed development can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy provided that the recommendations given in Section 7 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

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 present risk of instability of the site and to control future risks. These recommendations address geotechnical issues only and other conditions may be required to address other aspects.

The recommendations provided herein are specifically related to the proposed subdivision works and the individual lots as shown in the supplied architectural drawings. Although some of the recommendations may also be applicable to amended development plans, specific advice should be obtained should any changes be made to the proposed development within the individual lots, in particular the cut or fill depths. For the purposes of this report we have assumed that the houses will require cuts of no more than 3m depth and minor fill as shown on the supplied drawings. Further geotechnical investigation may be required at individual lots to assist with detailed design.

7.1 Conditions Recommended to Establish the Design Parameters

- 7.1.1 The existing retaining wall along the road side (south-western) boundary with No. 96 is leaning over, and in some sections fallen. Even though this risk assessment has shown that the risk from the hazard is acceptable we recommend that it be replaced as part of the subdivisional works. Since this wall extends across several of the proposed allotments it would be much easier to reconstruct the wall as part of the subdivision rather than within each individual lot. Retaining wall design parameters are provided below. Stone block gravity, or reinforced block retaining walls will be suitable subject to further investigation of the depth to rock along the wall alignment. Gabion and crib-lock walls, and reinforced earth walls, are also feasible structures provided they are founded on rock with sufficient space to form temporary batters.
- 7.1.2 We expect the majority of other low height retaining walls within the site will be demolished during future works following subdivision, but they could be demolished as part of the subdivisional works and the site battered.



- 7.1.3 Subject to inspection by a geotechnical engineer excavations for the proposed driveway and the individual houses should be formed at temporary batters no steeper than 1 Vertical (V) in 1 Horizontal (H) within the soil profile and extremely weathered rock for heights of no more than 3m. All surcharge and footing loads must be kept well clear of the crest of any excavation batters. Any permanent batters of no more than 3m in height should be no steeper than 1V:3H. If excavations of more than 3m are proposed additional geotechnical advice on retention options should be obtained.
- 7.1.4 If rock excavation using hydraulic rock hammers is required close to existing structures, say within 20m, vibration monitoring must be carried out. The ground vibration measured as peak particle velocity must not exceed 5mm/sec at the existing structures.
- 7.1.5 All proposed footings, including for the proposed houses, must be founded in sandstone bedrock. The footings should be designed for an allowable bearing pressure of 1,000kPa on sandstone of at least very low strength, subject to inspection by a geotechnical engineer prior to pouring.
- 7.1.6 The proposed new retaining walls should be designed using the following parameters:
- For cantilever walls, adopt a triangular lateral earth pressure distribution and an ‘active’ earth pressure coefficient, K_a , of 0.3, for the retained height, assuming a horizontal backfill surface.
 - A bulk unit weight of 20kN/m³ should be adopted for the soil profile.
 - Any surcharge affecting the walls (eg. inclined backfill, traffic loading, live loading, compaction stresses, etc) should be allowed in the design.
 - The retaining walls should be provided with complete and permanent drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (eg. Bidim A34), to act as a filter against subsoil erosion.
 - Toe resistance of the wall may be achieved by keying the footing into bedrock. An allowable lateral stress of 200kPa may be adopted for design.
- 7.1.7 Prior to construction of the proposed subdivisional driveway and any individual driveways, all vegetation, loose and deleterious material should be stripped. The subgrade should be proof rolled with a minimum 8 passes of at least an 8 tonne roller, with the final pass carried out without vibration and witnessed by a geotechnical engineer to detect any unsuitable subgrade areas. Any unsuitable material detected should be replaced with engineered fill.
- 7.1.8 Engineered fill may comprise site won material including clayey soils but would preferably comprise inert granular material, such as crushed sandstone with a maximum particle size of no more than 75mm. Such material should be compacted in layers no greater than



200mm loose thickness to at least 98% Standard Maximum Dry Density (SMDD). Any clay fill should be compacted to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% Standard Optimum Moisture Content (SOMC).

- 7.1.9 The diverted stormwater alignment on the north-western edge of the site must be properly engineer designed to avoid uncontrolled concentrated flows and infiltration into the hillside slope. Preferably a piped system should be designed.
- 7.1.10 The surface water discharging from new paved areas must be diverted to outlets for controlled discharge to the existing/new stormwater system which appears to drain to the bay.
- 7.1.11 The guidelines for Hillside Construction given in Appendix B should also be adopted.

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

- 7.2.1 All structural design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle. This includes drawings prepared for the subdivisional works and for each individual lot.
- 7.2.2 All civil design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle. This includes drawings prepared for the subdivisional works and for each individual lot.
- 7.2.3 All hydraulic design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle. This includes drawings prepared for the subdivisional works and for each individual lot.
- 7.2.4 All landscape design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle. This includes drawings prepared for the subdivisional works and for each individual lot.

7.3 Conditions Recommended During the Construction Period

- 7.3.1 Bulk excavations for the proposed driveway and houses must be progressively inspected by the geotechnical engineer as excavation proceeds to confirm that appropriate excavation batters are being adopted. We recommend inspections at 1.5m vertical depth intervals and on completion.



- 7.3.2 The geotechnical engineer must inspect all footing excavations prior to placing reinforcement or pouring the concrete.
- 7.3.3 Proposed material to be used for filling for the proposed driveway and for backfilling behind retaining walls must be approved by the geotechnical engineer prior to placement.
- 7.3.4 Compaction density of any fill, including backfill behind walls must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer.
- 7.3.5 Where they are to be retained, the existing stormwater system, sewer and water mains must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer or architect, and repaired if found to be leaking.
- 7.3.6 The geotechnical engineer must inspect all subsurface drains prior to backfilling.
- 7.3.7 An 'as-built' drawing of all buried services at the site must be prepared (including all pipe diameters, pipe depths, pipe types, inlet pits, inspection pits, etc).
- 7.3.8 The geotechnical engineer must confirm that the proposed subdivisional works have been completed in accordance with the geotechnical reports.

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

7.4 Conditions Recommended for Ongoing Management of the Site/Structure(s)

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

- 7.4.1 All existing and proposed surface and subsurface drains must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than five yearly intervals; including provision of a written report confirming scope of work completed (with reference to the 'as-built' drawing) and identifying any required remedial measures.
- 7.4.2 The existing retaining walls within No 102 and 104 Cabarita Road must be inspected by a structural engineer at no more than five yearly intervals; including the provision of a written report confirming scope of work completed and identifying any required remedial measures.



- 7.4.3 No cut or fill in excess of 0.5m (eg. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Pittwater Council.
- 7.4.4 Where the structural engineer has indicated a design life of less than 100 years then the structure and/or structural elements must be inspected by a structural engineer at the end of their design life; including a written report confirming scope of work completed and identifying the required remedial measures to extend the design life over the remaining 100 year period.

8 OVERVIEW

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

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

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

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



TABLE A: SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL STABILITY HAZARD	A	B	C	D	E	F	G	H	I
	Overall slope failure, typically 10°-20°	Localised slope failure, typically ≤ 30°	Failure of steep vegetated slope (35°) downslope of No 104 and top of No 100	Unsupported cuts in soil ≥45°, <1.5m high	Failure of existing cribblock wall at Cabarita Rd and other properly constructed walls <1.5m height	Failure of existing stone walls <1.5m high	Proposed engineered retaining walls and cut & fill for access driveway.	Proposed engineered retaining wall for new houses	Local slope failure beneath the new houses
Assessed Likelihood	Rare	Unlikely	Unlikely	Almost certain	Unlikely	Almost certain	Rare	Rare	Rare
Assessed Consequences	Minor	Insignificant	Minor	Insignificant	Insignificant	Insignificant	Minor	Minor*	Minor*
Risk	Very Low	Very Low	Low	Low	Very Low	Low	Very Low	Very Low	Very Low
Comments	Slope not overly steep. Soil depth is shallow and reasonable surface drainage is evident	Few existing structures and slopes are relatively shallow so extent of failure will be minimal	Unlikely to be deep seated failure plains so unlikely to extend to house. Existing conditions held for >50years already. Vegetation well established.	Temporary cuts will be remediated during future works and if fail there is sufficient space to slump to angle of repose without adversely affecting structures.		All low height and will not affect other structures. Remediate by batters or rebuild.	Assumes walls and earthworks are properly designed and constructed	Assumes walls are properly designed and constructed	Assumes house footings are founded on rock and approved by a geotechnical engineer

Assumed property price: \$16,400,000 for all for four properties combined (source: onthefhouse.com.au) except where in respect of individual lots assumed value is \$2,000,000/lot

TABLE B: SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL STABILITY HAZARD	A	B	C	D	E		F	G	H	I
Assessed Likelihood	Rare	Unlikely	Unlikely	Almost certain	Unlikely		Almost certain	Rare	Rare	Rare
Indicative Annual Probability	10 ⁻⁵	10 ⁻⁴	10 ⁻⁴	10 ⁻¹	10 ⁻⁴		10 ⁻¹	10 ⁻⁵	10 ⁻⁵	10 ⁻⁵
Duration of Use of Area Affected (Temporal Probability)	Landscapers maintaining vegetation, say 1, 8hr day per fortnight	Landscapers maintaining vegetation, say 1 8hr day per fortnight	Patio user, say 3hrs twice a week	Construction worker building retaining wall, 4hrs/day, 4 weeks/yr	Pedestrian road user, say 1minute twice/day	Patio user adjacent to wall, say 1hr/week	Landscapers maintaining vegetation, mostly away from paths, say 10mins per shift per fortnight	Subdivision workers using future driveway, say 1hr per working day, 5 days per week	Persons in rooms of houses cut into hillside Say average of 8hrs per day	Persons in house, say average 12hrs per day
	2.4 x 10 ⁻²	2.4 x 10 ⁻²	3.6 x 10 ⁻²	1.8 x 10 ⁻³	1.4 x 10 ⁻³	6.0 x 10 ⁻³	4.9 x 10 ⁻⁴	4.8 x 10 ⁻⁴	0.33	0.5
Probability of Not Evacuating Area Affected	1.0	0.5	1.0	1.0	0.5	0.5	0.1	0.1	0.5	1
Spatial Probability	0.3	0.1	0.3	0.1	0.2	0.2	0.05	0.2	0.5	0.2
Vulnerability to Life if Failure Occurs Whilst Person Present	0.8 Likely to be buried, probably killed	0.5 May be buried, possibly killed	0.5 May be buried, possibly killed	0.01 Not buried, high chance of survival	0.01 Not buried high chance of survival	0.5 may be buried possibly killed	0.01 Not buried, high chance of survival	0.01 Not buried, high chance of survival	0.5 May be buried, possibly killed	0.01 Not buried, high chance of survival provided early signs of settlement failure are detected
Risk for Person Most at Risk	5.8 x 10 ⁻⁸	1.2 x 10 ⁻⁷	5.4 x 10 ⁻⁷	1.8 x 10 ⁻⁷	1.4 x 10 ⁻¹⁰	3 x 10 ⁻⁸	2.5 x 10 ⁻⁹	9.6 x 10 ⁻¹³	8.25 x 10 ⁻⁷	1.0 x 10 ⁻⁸
Total Risk For Person Most at Risk	1.3 x 10 ⁻⁶									

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND
LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Subdivision
Location: 96-104 Cabarita Road, Avalon Beach, NSW

Ref No: 31842AH
Report: A
Report Date: 27/09/2018
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	1.40-1.60	3.9				
1	2.00-2.40	3.1				
2	0.50-0.80	13.5	60	23	37	15.0
2	3.10-3.40	5.3				
2	4.50-4.70	5.3				
2	5.50-5.60	3.4				
4	1.90-1.95	7.6				
4	3.90-4.10	4.2				

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 12/9/18

BOREHOLE LOG

Borehole No.

1

1/1

Client: MERAKI DEVELOPMENTS PTY LTD
Project: PROPOSED SUBDIVISION
Location: 96-104 CABARITA ROAD, AVALON BEACH, NSW

Job No. 31842BM **Method:** SPIRAL AUGER
Date: 14/09/2018 **JK308**
R.L. Surface: ≈ 23.5m
Datum: AHD
Logged/Checked by: C.A./M.P.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPL- ETION						0		-	TOPSOIL: Sandy silt, low plasticity, dark brown, fine to medium grained sand.	w<PL			GRASS COVER
								SM	Silty SAND: fine to medium grained, brown.	w<PL	L		
					N > 12 4,4,8			-	Extremely Weathered sandstone: Clayey SAND, fine to medium grained, orange brown and light grey trace of root fibres.	XW	D		NEWPORT FORMATION
					REFUSAL	1			SANDSTONE: fine to coarse grained, light grey and orange brown.	DW	L		LOW 'TC' BIT RESISTANCE
									as above, but light grey mottled orange brown.		M		MODERATE RESISTANCE
						2			as above, but orange brown, with iron indurated bands.		H		
									END OF BOREHOLE AT 2.4m				'TC' BIT REFUSAL
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.

2

1/1

Client: MERAKI DEVELOPMENTS PTY LTD
Project: PROPOSED SUBDIVISION
Location: 96-104 CABARITA ROAD, AVALON BEACH, NSW

Job No. 31842BM **Method:** SPIRAL AUGER JK308 **R.L. Surface:** ≈ 16.5m
Date: 14/09/2018 **Datum:** AHD
Logged/Checked by: C.A./M.P.

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION					0		CH	FILL: Silty sandy clay, low to medium plasticity, dark grey and brown, with fine to coarse grained sand.	w>PL			RESIDUAL
								Silty sandy CLAY: high plasticity, orange brown mottled grey, fine to coarse grained sand.	w≈PL	Hd		
				N = 17 5,10,7			-	Extremely Weathered sandstone: Clayey SAND, fine to coarse grained, light grey mottled orange brown.	XW	D	>600 >600 >600	NEWPORT FORMATION
					1							
								SANDSTONE: fine to medium grained, light grey mottled orange brown, with iron indurated bands.	DW	VL		LOW TO MODERATE 'TC' BIT RESISTANCE WITH VERY LOW BANDS
					2					VL-L		
												LOW RESISTANCE WITH VERY LOW BANDS
					3			SANDSTONE: fine to coarse grained, orange brown, with clay bands.		L		
												MODERATE RESISTANCE
					4			as above, but without clay bands.		M		
					5							'TC' BIT REFUSAL
								END OF BOREHOLE AT 5.6m		H		
					6							
					7							

BOREHOLE LOG

Borehole No.

3

1/1

Client: MERAKI DEVELOPMENTS PTY LTD
Project: PROPOSED SUBDIVISION
Location: 96-104 CABARITA ROAD, AVALON BEACH, NSW

Job No. 31842BM **Method:** SPIRAL AUGER JK308 **R.L. Surface:** ≈ 13.3m
Date: 14/09/2018 **Datum:** AHD
Logged/Checked by: C.A./M.P.

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION					0			FILL: Clayey sandy silt, low plasticity, dark brown.	w<PL			APPEARS POORLY COMPACTED
								FILL: Sandy clay, medium plasticity, brown, fine to coarse grained sand, trace of root fibres and fine to medium grained ironstone gravel.	w>PL			
				N = 6 2,2,4			CI	Sandy CLAY: medium plasticity, orange brown mottled red brown, fine to medium grained sand.	w≈PL	VSt	250 300 300	RESIDUAL
					1			as above, but light grey mottled orange and red brown.			230 330 430	
				N = 11 4,5,6								
					2							
				N = 15 5,6,9			SC	Silty clayey SAND: fine to coarse grained, light grey mottled red brown, with medium to coarse grained ironstone gravel with bands of sandy clay.	M	MD	(350) (480)	NEWPORT FORMATION
					3							
					4		-	SANDSTONE: fine to coarse grained, light grey.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
					5			as above, but with iron indurated bands.		L-M		LOW RESISTANCE WITH MODERATE BANDS
								END OF BOREHOLE AT 5.1m				'TC' BIT REFUSAL
					6							
					7							

BOREHOLE LOG

Borehole No.

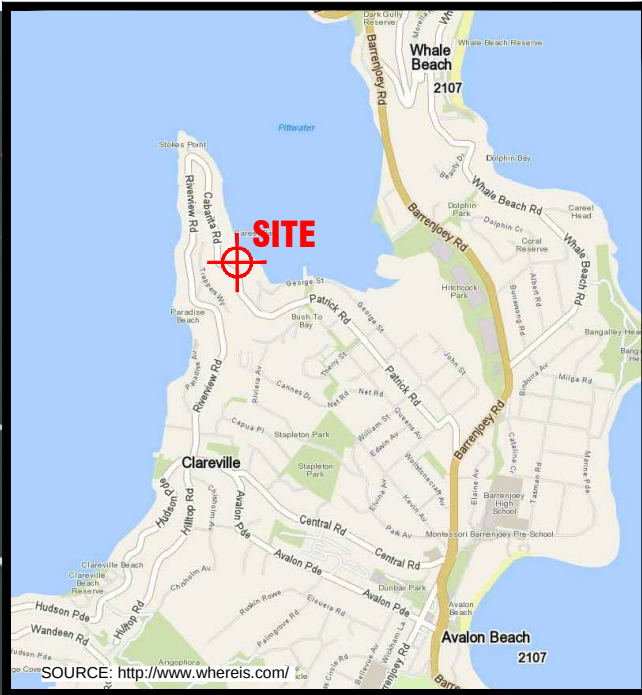
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1/1

Client: MERAKI DEVELOPMENTS PTY LTD
Project: PROPOSED SUBDIVISION
Location: 96-104 CABARITA ROAD, AVALON BEACH, NSW

Job No. 31842BM **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 13.9m
Date: 14/09/2018 **JK308** **Datum:** AHD
Logged/Checked by: C.A./M.P.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION						0			FILL: Silty SAND, fine to medium grained, brown, trace of fine to medium grained igneous gravel.	M			
								CL-CI	Sandy CLAY: low to medium plasticity, orange brown and grey, with root fibres, fine to medium grained sand.	w≈PL			RESIDUAL
					N = 10 3,3,7				as above, but orange brown.		Hd	>600 >600 >600	
						1							
								-	Extremely Weathered sandstone: Silty clayey SAND, fine to medium grained, light grey mottled red brown, with medium to coarse grained ironstone gravel.	XW	D		NEWPORT FORMATION
					N = 28 8,11,17				SANDSTONE: fine to medium grained, light grey and orange brown.	DW	VL		
						2							
						3			as above, but with iron indurated bands.				LOW 'TC' BIT RESISTANCE
						4			as above, but red brown.		H		HIGH RESISTANCE
									END OF BOREHOLE AT 4.10m				'TC' BIT REFUSAL POSSIBLY ON IRON INDURATED BAND



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

Title:

SITE LOCATION PLAN

Location:

96-104 CABARITA ROAD
AVALON BEACH, NSW

Report No:

31842BM

Figure No:

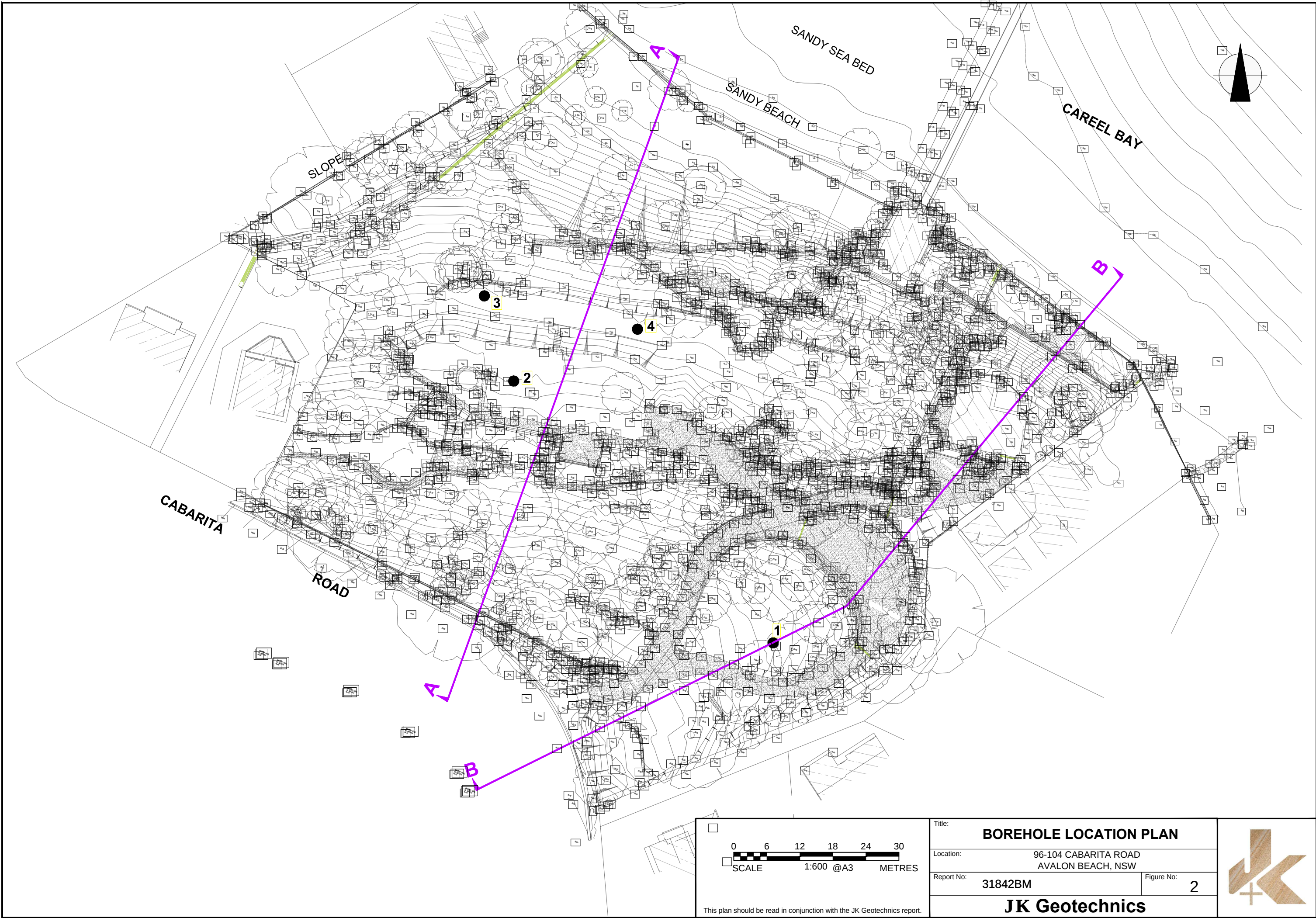
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JK Geotechnics



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

PLOT DATE: 30/11/2018 4:35:01 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL_JOBS\31000 S\31842BM AVALONICAD\31842BM.DWG



This plan should be read in conjunction with the JK Geotechnics report.	Title: BOREHOLE LOCATION PLAN		
	Location: 96-104 CABARITA ROAD AVALON BEACH, NSW		
	Report No: 31842BM	Figure No: 2	
	JK Geotechnics		

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© JK GEOTECHNICS

CABARITA

ROAD

SLOPE

SANDY BEACH

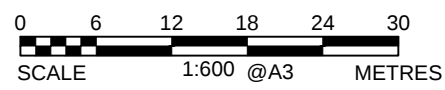
SANDY SEA BED

CAREEL BAY

TERRACED SLOPE

DENSE VEGETATION

STEEP SLOPE

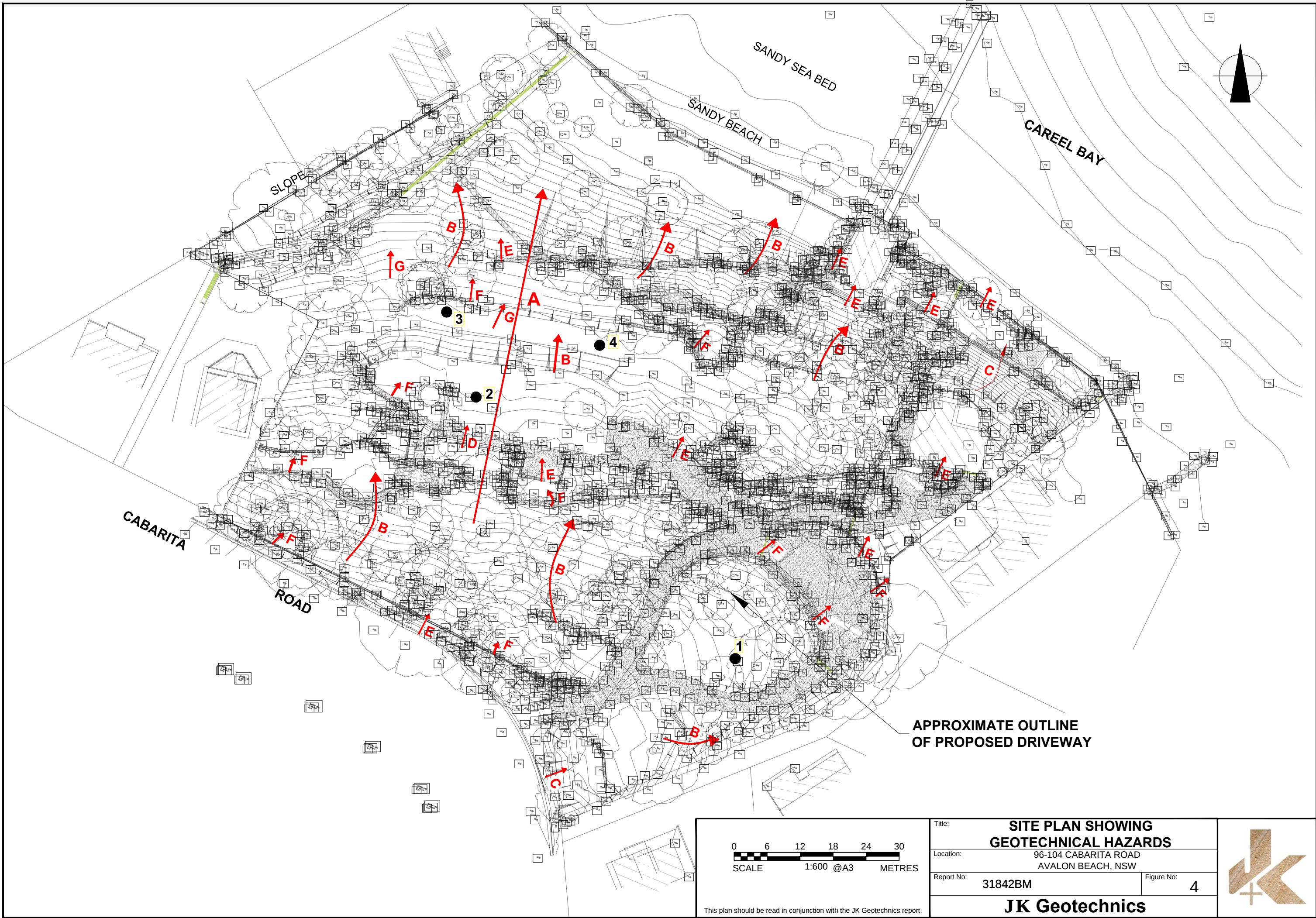


This plan should be read in conjunction with the JK Geotechnics report.

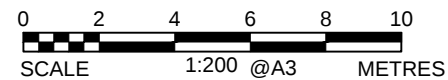
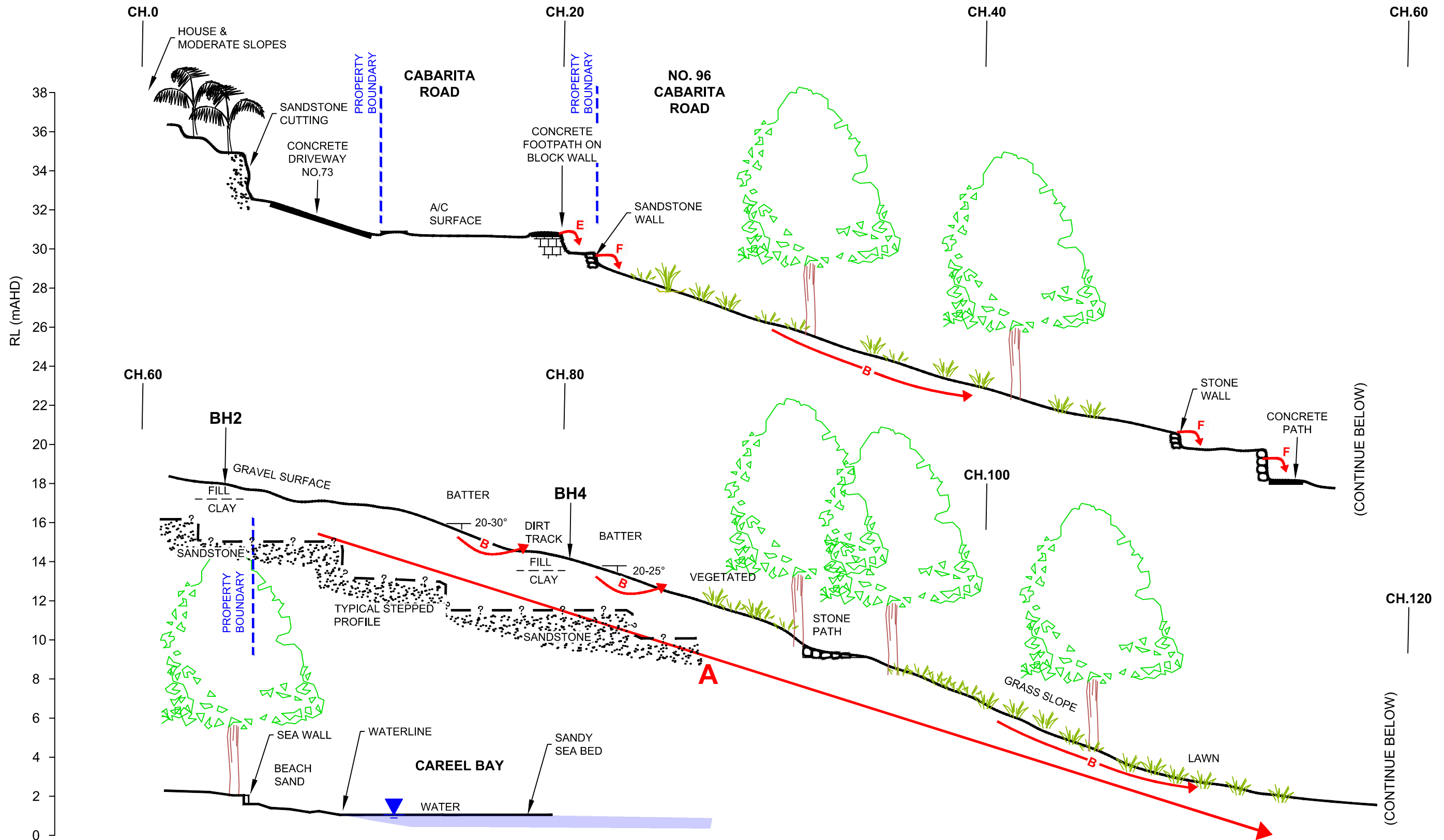
Title:		GEOTECHNICAL MAPPING PLAN	
Location:		96-104 CABARITA ROAD AVALON BEACH, NSW	
Report No:	31842BM	Figure No:	3
JK Geotechnics			



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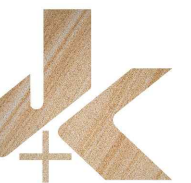


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This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION A-A (LOOKING NORTH-WESTWARDS)	
Location: 96-104 CABARITA ROAD AVALON BEACH, NSW	
Report No: 31842BM	Figure No: 5
JK Geotechnics	



GEOTECHNICAL MAPPING SYMBOLS

TOPOGRAPHY

Symbol Ground Profile

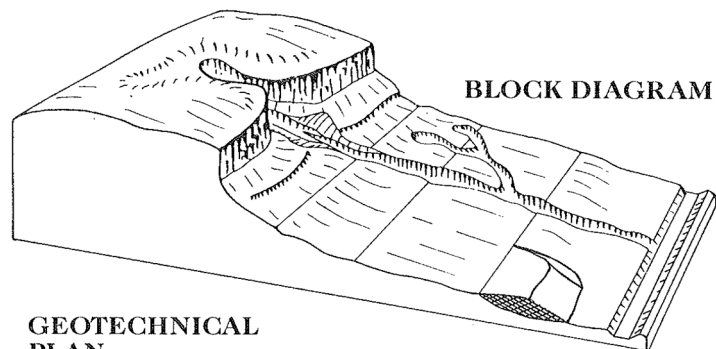
		convex	} well defined or angular break of slope
		concave	
		convex	} poorly defined or smooth change of slope
		concave	
		breaks of slope	} convex and concave too close together to allow the use of separate symbols
		changes of slope	
		sharp	} ridge crest
		rounded	
		Cliff or escarpment or sharp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

Hummocky or irregular ground

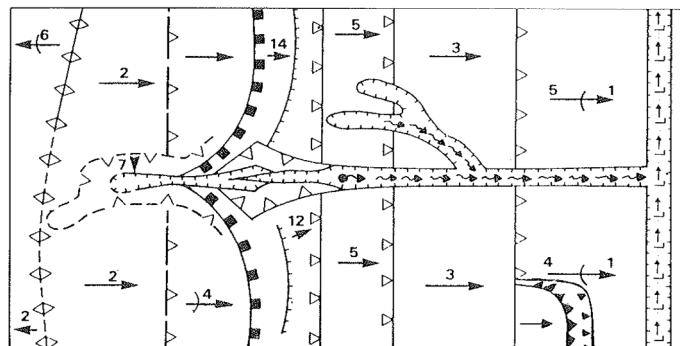
OTHER FEATURES

	Boulder
	Seepage/spring
	Swallow hole for runoff
	Natural water course
	Open drain, unlined
	Open drain, lined
	Fenceline
	Property boundary
	Dry Stone Wall
	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:



GEOTECHNICAL PLAN



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



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

$$N > 30 \\ 15, 30/40\text{mm}$$

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_b), horizontal stress index (K_b), and dilatometer modulus (E_b). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_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_0).

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

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

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



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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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



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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would

be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

SYMBOL LEGENDS

SOIL

	FILL
	TOPSOIL
	CLAY (CL, CI, CH)
	SILT (ML, MH)
	SAND (SP, SW)
	GRAVEL (GP, GW)
	SANDY CLAY (CL, CI, CH)
	SILTY CLAY (CL, CI, CH)
	CLAYEY SAND (SC)
	SILTY SAND (SM)
	GRAVELLY CLAY (CL, CI, CH)
	CLAYEY GRAVEL (GC)
	SANDY SILT (ML, MH)
	PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

	CONGLOMERATE
	SANDSTONE
	SHALE/MUDSTONE
	SILTSTONE
	CLAYSTONE
	COAL
	LAMINITE
	LIMESTONE
	PHYLLITE, SCHIST
	TUFF
	GRANITE, GABBRO
	DOLERITE, DIORITE
	BASALT, ANDESITE
	QUARTZITE

OTHER MATERIALS

	BRICKS OR PAVERS
	CONCRETE
	ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 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	

Laboratory Classification Criteria

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

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

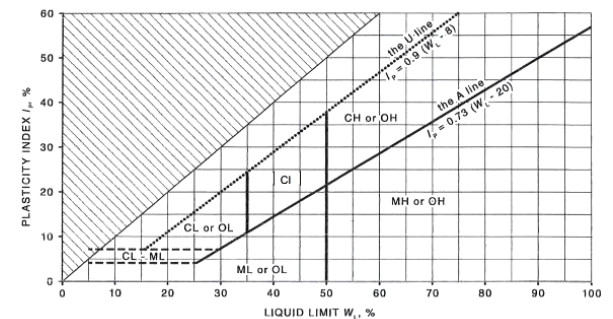
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

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

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

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





LOG SYMBOLS

Log Column	Symbol	Definition																		
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.																		
		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																		
	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 ≈ PL w < PL w ≈ 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)	VL L MD D VD ()	<table> <tr> <th></th><th>Density Index (I_D) Range (%)</th><th>SPT 'N' Value Range (Blows/300mm)</th></tr> <tr> <td>VERY LOOSE</td><td>≤ 15</td><td>0 – 4</td></tr> <tr> <td>LOOSE</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr> <tr> <td>MEDIUM DENSE</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr> <tr> <td>DENSE</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr> <tr> <td>VERY DENSE</td><td>> 85</td><td>> 50</td></tr> </table> Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)	VERY LOOSE	≤ 15	0 – 4	LOOSE	> 15 and ≤ 35	4 – 10	MEDIUM DENSE	> 35 and ≤ 65	10 – 30	DENSE	> 65 and ≤ 85	30 – 50	VERY DENSE	> 85	> 50
	Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)																		
VERY LOOSE	≤ 15	0 – 4																		
LOOSE	> 15 and ≤ 35	4 – 10																		
MEDIUM DENSE	> 35 and ≤ 65	10 – 30																		
DENSE	> 65 and ≤ 85	30 – 50																		
VERY DENSE	> 85	> 50																		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.																		



Log Symbols continued

Log Column	Symbol	Definition
Remarks	'V' bit 'TC' bit T ₆₀ Soil Origin	Hardened steel 'V' shaped bit. Twin pronged tungsten carbide bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers. The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits. LITTORAL – beach deposited soil.



Classification of Material Weathering

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

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

Rock Material Strength Classification

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



Abbreviations Used in Defect Description

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



APPENDIX A

LANDSLIDE RISK MANAGEMENT TERMINOLOGY



APPENDIX A

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 <i>(continued)</i>	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

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

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

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

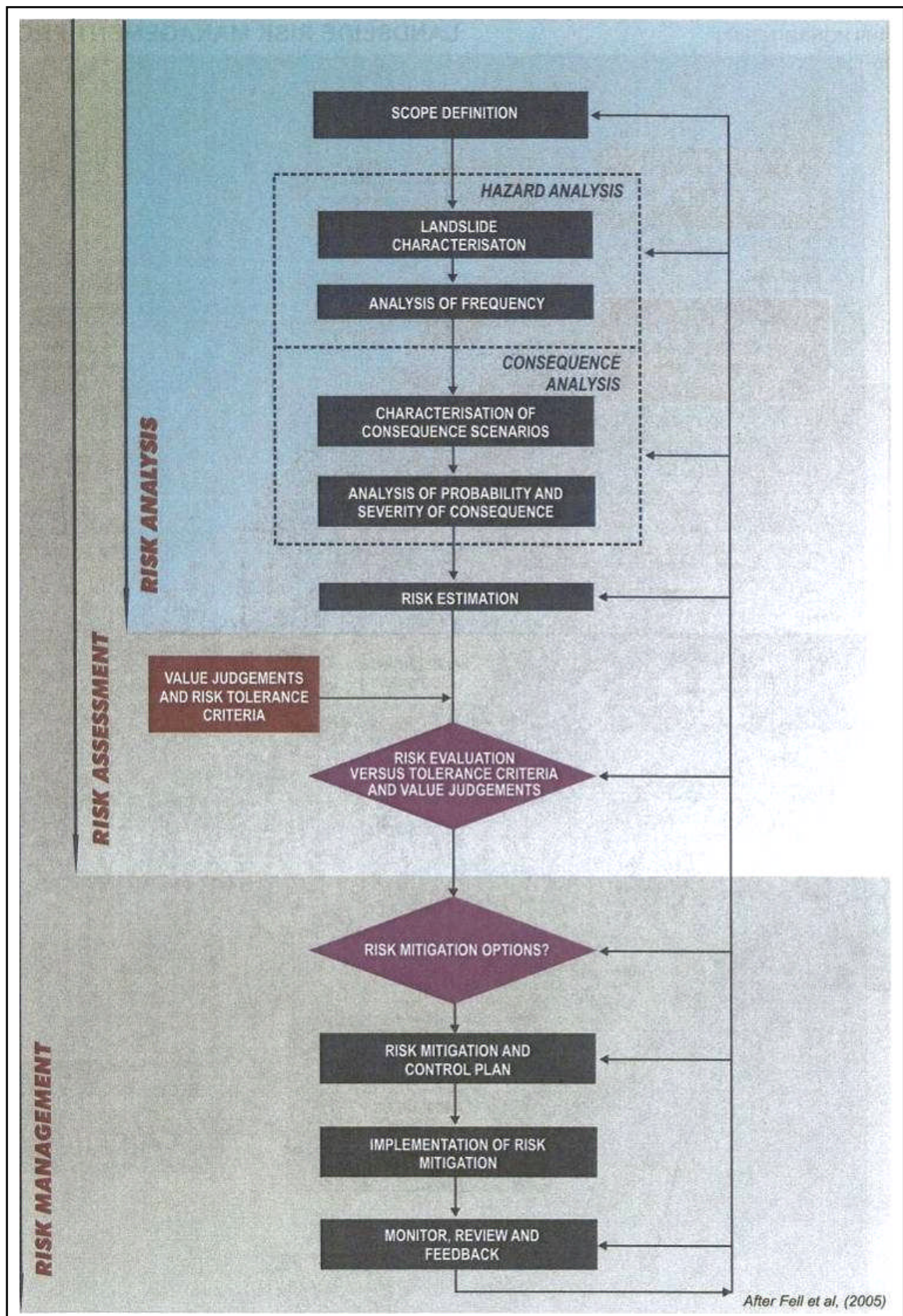


FIGURE A1: Flowchart for Landslide Risk Management.

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



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

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-3}	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-4}	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-5}	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}	5×10^{-6}	1,000,000 years	200,000 years	The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

- (3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.
- (4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not *vice versa*.

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



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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

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

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

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



AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

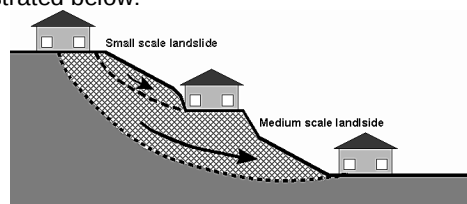


Figure 1

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.

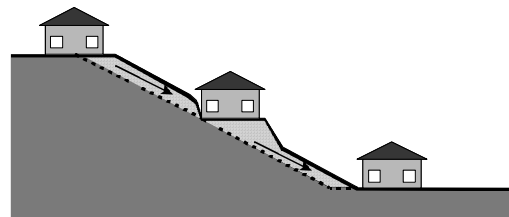


Figure 2

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.

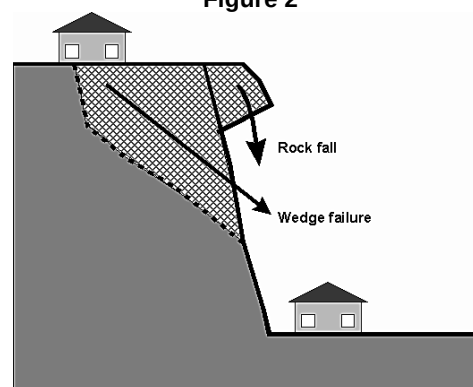


Figure 3

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.

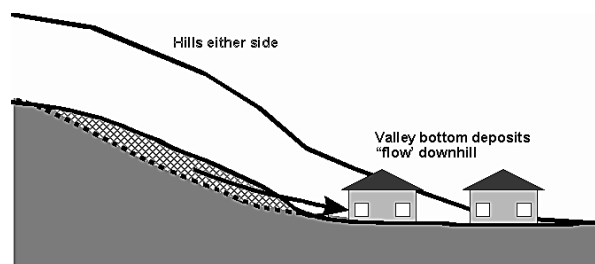


Figure 4

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

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

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



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. If you have any concern that you could be dealing with a landslide hazard that your local council is not aware of you should seek advice from a geotechnical practitioner.

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 inevitably lacks 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 perhaps temporary loss of use. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

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.

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", "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. This may be lower than you feel is reasonable for your block but it is, nonetheless, a pre-requisite for development. Reasons for this include the fact that a landslide on your block may pose a risk to neighbours and passers-by and that, should you sell, subsequent owners of the block may be more risk averse than you.



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 LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

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APPENDIX B

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION



APPENDIX B – SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

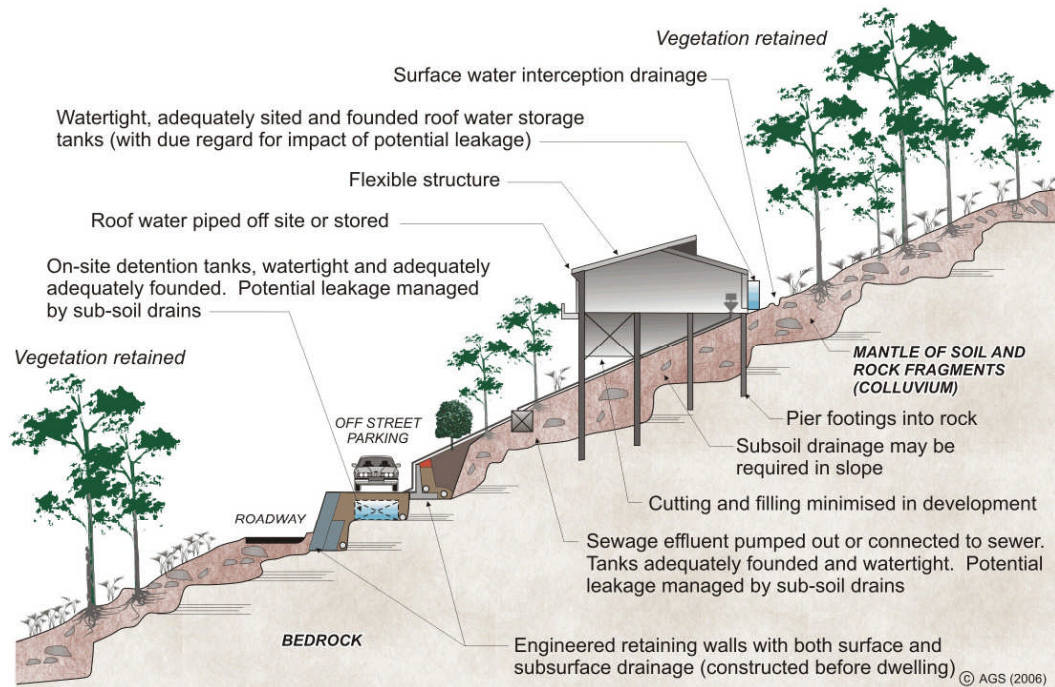
GOOD ENGINEERING PRACTICE		POOR ENGINEERING PRACTICE	
ADVICE			
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.	
PLANNING			
SITE PLANNING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

HILLSIDE 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.

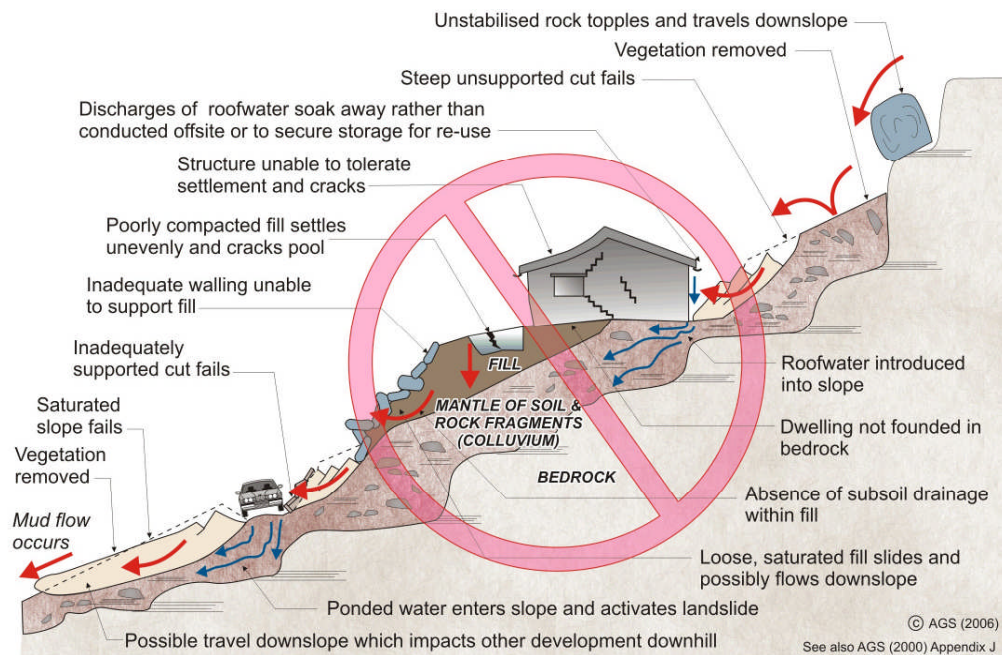
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing 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
Extract from Geoguide LR8 – Hillside Construction Practice



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 LR6 - Retaining Walls |
| • GeoGuide LR2 - Landslides | • GeoGuide LR7 - Landslide Risk |
| • GeoGuide LR3 - Landslides in Soil | • GeoGuide LR9 - Effluent & Surface Water Disposal |
| • GeoGuide LR4 - Landslides in Rock | • GeoGuide LR10 - Coastal Landslides |
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Extract from Geoguide LR8 – Hillside Construction Practice.

APPENDIX C

