REPORT TO

ANTHONY JEFFCOAT

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

22 HAY STREET, COLLAROY, NSW

Date: 20 August 2019

Ref: 31892Yrpt

JKGeotechnics

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For and on behalf of
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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
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Table of Contents

1 INTRODUCTION

This report represents the results of a geotechnical investigation and stability assessment for the proposed alterations and additions at 22 Hay Street, Collaroy, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Ms Vivianne Marston on behalf of Anthony Jeffcoat by signed 'Acceptance of Proposal' form dated 30 July 2019. The commission was on the basis of our fee proposal (Ref: P50007Y) dated 29 July 2019.

Based on the concept designs provided by Marston Architects, we understand that the proposed works will include:

- Construction of a new two car carport at the western corner of the site. The carport will be suspended and will have a finished floor level of RL48m. Access will be from the shared driveway that runs off Lancaster Crescent. A new access bridge will connect the carport to the house at first floor level.
- In addition to the proposed carport remodelling of the house is also proposed which will primarily comprise internal changes but will include construction of a new first floor bedroom at the southern corner of the house and a new first floor deck at the front of the house. All additions will be located wholly within the existing footprint of the building.

The purpose of the investigation was to assess the risk posed to both life and property as a result of potential slope instability and to obtain geotechnical information on subsurface conditions at the test locations. Based on the results of the subsurface investigation we have also provided comments and recommendations on excavation, retention, footings, slabs on grade and soil aggression.

2 INVESTIGATION PROCEDURE

The investigation comprised two parts which were:

- A site walkover to identify potential slope stability hazards and
- A subsurface investigation to determine the ground conditions present across the site.

The risk to both life and property was assessed based on a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs together with the results of the subsurface investigation. The site walkover was completed by our Principal Associate, Mr Woodie Theunissen, who compared the features on site with those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the existing 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 assessment of the risk of slope instability for the site in its existing condition is discussed in Section 4.

The attached Figure 2 presents a geotechnical sketch plan showing the principal geotechnical features present at the site. Figure 2 is based on the survey plan prepared by CMS Surveyors Pty Ltd (Drawing Name 17755detail, Issue 1, dated 9 May 2018). Additional features on Figure 2 have been measured by hand held clinometer and tape measure techniques and hence are approximate only. Should any of the features be critical, we recommend they be located more accurately using instrument survey techniques. Figure 3 presents a typical cross-section through the site based on the survey data augmented by our mapping observations.

The subsurface investigation was carried out on 26 September and the 9 October 2018 and included the drilling of four boreholes (BH1 to BH4) to depths ranging from 2.1m to 6.66m. The boreholes were all drilled to refusal using hand auger techniques. On hand auger refusal, BH1 and BH4 were then drilled using portable coring techniques to termination depths of 3.61m and 6.66m respectively. In addition, at each borehole location Dynamic Cone Penetration (DCP) tests were completed to depths ranging from 2.05m to 2.205m, at which depths DCP refusal occurred.

While the nature of the materials present was determined from the boreholes, the degree of compaction of the fill and the strength of the natural soils were interpreted from the DCP test results. In the natural soils, the DCP test assessment was augmented by hand penetrometer tests completed on remoulded samples recovered from the hand augered boreholes.

Where drilled using coring techniques, the strength of the underlying sandstone bedrock was determined from the point load strength index test (Is50) results. This testing was completed by a NATA registered laboratory, Soil Test Services (STS), which then calculated the unconfined compressive strength (UCS) from the Is50 test results using established correlations. The UCS results are shown on the cored portions of the borehole logs and are presented in STS's Table A Point Load Strength Index Test Report. Photographs of the core were also taken and are presented at the end of this report adjacent to the appropriate borehole logs.

Selected samples were also returned to Envirolab Services, also a NATA registered laboratory for pH, sulphide, chloride and resistivity testing. The results of these tests are presented in Envirolab Services Certificate of Analysis 202962.

Groundwater observations were made during and on the completion of borehole drilling. No longer term groundwater monitoring was completed.

The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The borehole locations were scanned for buried services by a specialist sub-contractor prior to drilling commencing such that all tests could be located clear of all services. The relative levels shown on the attached logs and DCP sheets were interpolated from spot levels shown on the survey plan referenced above and are thus only approximate.

The fieldwork was completed by our engineering geologist, Mr Bo Jonak, who set out the investigation locations, nominated the testing and sampling and prepared the attached borehole logs and recorded the

DCP test results. For more details of the investigation procedures, their limitations and the meanings of the symbols and terminology used, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located towards the lower slopes of Collaroy Plateau and is positioned above the Long Reef terrace. The site is north-east facing with a general slope of 10-13^{o}. At the time of the investigation the site was occupied by a two storey clad residence with metal roof over a lower ground carpark.

While the front of the site, although terraced, generally slopes down to the north-east at about 5^{o} to 8^{o}, at the driveways slopes increase up to about 10^{o} to 12^{o}. A sandstone block retaining wall runs along the front of the property and supports the site, which is up to 0.9m higher than the road reserve. This wall generally appears in good condition but does, in places show some signs of distress in the form of outward rotation. The front garden is typically grassed with manicured hedging and the occasional tree. Along the south-eastern corner of the site a paved sitting area is retained by a low height masonry wall no greater than 0.8m high that appeared in good condition.

From the rear to the front of the house site levels drop about 2m while from the rear of the backyard to the house site levels drop about 6m, from RL46.5m to RL40.35m, with original ground levels sloping down to the south-east. Most of the rear yard has been terraced and the terraces are supported by a mix of masonry retaining walls ranging from about 0.4m to 2.2m in height, all of which appeared in good condition. The rear yard has two main terraces, an astro turf covered terrace immediately behind the house, which is generally about 1.4m higher than the house and supported by a sandstone block retaining wall and the pool area, which is about 1.75m higher than the astro turf area.

The pool area has been cut into the hillside with a masonry retaining wall in good condition and a maximum height of about 2.2m supporting this cut. While the high side of the pool area has been cut into the hillside, on the low side it comprises a suspended deck. Consequently, the inground pool is cut into the hill on the high side and out of the ground on the low side. To the north of the pool the ground slopes down at about 15^{o} to the south-east, is currently used as a chicken run and is vegetated with a number of trees, none of which showed signs of basal curvature. At the toe of this portion of the site was a sandstone block retaining wall with a maximum height of about 0.8m, beyond which was a relatively level garden area comprising shrubs and trees that then drops down to the astro turf surfaced terrace through a sandstone block retaining all that has a maximum height of about 1.1m.

To the north is 1A Bedford Crescent, a two to three storey house of masonry and clad construction that appeared in good condition and is set back approximately 2.3m form the boundary. This property had a similar land form to that of the site. To the south is 20 Hay Street, a single storey house of masonry and clad construction that is set back about 0.7m from the boundary and appeared in good structural condition. This property had a similar land form to that of the site although immediately to the rear of this house levels were approximately 0.7m lower with the difference in height retained by a masonry wall that appeared in good condition.

To the east a grassed nature strip slopes down at up to about 10^{o} to Hay Street, an asphaltic concrete paved road. To the west, runs a concrete driveway that provides access to both 1 and 3 Lancaster Crescent. Lancaster Crescent is located further to the west and is also an asphaltic concrete paved road.

3.2 Subsurface Conditions

The 1:100,000 Geological sheet of Sydney (Geological Series Sheet 9130) indicates the site to be located close to the boundary between the Hawksbury Sandstone and the Newport Formation of the Narrabeen Group.

The boreholes generally disclosed a subsurface profile comprising a relatively thin layer of sandy fill overlying residual clays that in turn overlay sandstone bedrock. Based on our observation of the sandstone bedrock we consider that the site is underlain by the Newport Formation which consists of undifferentiated laminite, shale and quartz lithic-quartz sandstone.

A more detailed description of the materials encountered is presented below. For a description of the materials encountered at a particular borehole location reference should be made to the attached borehole logs.

Fill

A sandy fill that was assessed to be poorly compacted was encountered in all boreholes and extended to depths of 0.3m to 0.7m.

Residual Clays

With the exception of BH3, where residual silty clays extended to the underlying bedrock, in all other boreholes residual silty clays overlie residual sandy clays that in turn overlie sandstone bedrock. The residual clays extend to depths ranging from 2.06m to 2.2m and were assessed to be of stiff to hard strength.

Sandstone Bedrock

Highly weathered sandstone bedrock was encountered at depths of 2.06m (BH1) and 2.1m (BH4). When first encountered the bedrock was of very low to low strength and, in the case of BH1 did not increase in strength prior to the termination depth of the borehole. In BH4 an increase to medium strength was observed between 3m to 4.5m before rock decreased to low strength. Core loss was logged at the start of the coring in BH1, which typically is considered to represent clay or poor quality bedrock that has been washed away during the coring process.

Defects within the rock mass generally comprised bedding partings and jointing, both of which were characterised by infill comprising either clay or extremely weathered infill material. BH4 was particularly fractured and contained numerous closely spaced defects.

Where the bedrock was not core drilled (BH2 and BH3), the depth to bedrock was inferred from the refusal depth of the DCP tests at depths of 2.08m and 2.105m. While it is possible that the DCP test has prematurely refused on a harder layer within the soil profile we believe that this is unlikely to be the case.

Groundwater

Groundwater monitoring was completed during and on completion of drilling. No longer term groundwater monitoring was completed. Groundwater seepage was observed in BH3 at a depth of 0.4m, although this was inferred to represent seepage from the recent rain events rather than an indication of groundwater levels. All augered boreholes were dry on completion of drilling while the use of water during the coring process obscures groundwater levels and has not been reported.

3.3 Laboratory Test Results

The point load strength index test results indicated an estimated UCS of the rock core ranged from 1MPa to 14MPa, although results more typically fell between 2MPa and 4MPa. It should be noted that the high UCS readings (UCS = 10MPa to 14MPa) recorded in the upper portion of BH4 were tests completed on iron indurated bands within the very low to low strength sandstone bedrock and should not be considered to be representative of the strength of the bedrock over this portion of the borehole.

The soil pH values ranged from 4.9 to 7.0, indicating acidic soil conditions in BH4 (at the rear of the property) and neutral soil conditions in BH2 (located at the front of the property). The sulphate contents ranged from <10mg/kg to 50mg/kg, the chloride contents ranged from 29mg/kg to 51mg/kg and the resistivity was recorded in both cases as 12,000ohm.cm. Based on these results, the soils would be classified as 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'. For steel piles, the soils would be classified as 'non-aggressive' in accordance with Table 6.5.2(C) of AS2159-2009.

4 GEOTECHNICAL ASSESSMENT

4.1 Potential Landslide Hazards

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

A Stability of existing masonry retaining wall along western boundary

B Stability of existing low height retaining walls,
  (i) sandstone block
  (ii) masonry
  (iii) sandstone block wall behind house

C Stability of low height sandstone block retaining wall along front boundary.

D Stability of low height masonry retaining wall along southern boundary.

E Stability of steep slope in north-western corner of the site.

F Stability of slope below house.

G Stability of slope in front of house and

H Stability of slope in Hay Street nature reserve.

Some of these potential hazards are indicated in schematic form on the attached Figure 2.

4.2 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 very low to low, which would be considered acceptable in accordance with the criteria given in Reference 1. Provided our comments and recommendations provide below are followed, the risk to property for the proposed development will be less than the risk currently posed by the existing site.

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 for the site in its current state is about 10^{-6}. Provided our comments and recommendations provide below are followed, the risk to life for the person most at risk for the proposed development will be less than the risk currently posed by the existing site. This would be considered to be acceptable in relation to the criteria given in Reference 1.

4.3 Risk Assessment

It should be noted 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, while it is necessary to undertake an active process of reducing risk, the geotechnical engineer does not warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

For the purpose of this assessment the design life be taken as 100 years. 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 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 geotechnical engineer does not warrant the development for a 100 year period, rather we 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.

In preparing our recommendations given below we have 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 development achieves the 'Acceptable Risk Management' criteria set out in Reference 1. In addition, provided the comments and recommendations set out below are followed, we consider that the proposed development will also achieve the 'Acceptable Risk Management' criteria set out in Reference 1.

5 COMMENTS AND RECOMMENDATIONS

5.1 Retention

The design life of the existing retaining wall on the low side of the common driveway must be assessed by the structural engineer. This may require the excavation of a number of test pits to determine the materials on which the wall is founded. Should it be considered to not have an adequate design life (ie 100 years) a new wall must be constructed that will satisfy the required design life and should be designed in accordance with the recommendations provided below.

Due to the anticipated limited height requiring retention and that anticipated relatively shallow depth to bedrock we anticipate that a soldier pile wall with shotcrete/concrete infill panels will be adopted. This wall may be designed as a cantilevered retaining wall resisting a triangular earth pressure distribution. Due to the fractured nature of the bedrock this pressure distribution should be adopted over the full height of the wall. As movement sensitive structures are located in the zone of influence of the excavation, a coefficient of lateral earth pressure, k, of 0.55 should be adopted. A unit weight of $20kN/m^3$ should be used. Appropriate surcharge loads (ie construction loadings, stockpiles, footings, sloping backfill, traffic loadings etc) and hydrostatic pressures should be added to the above pressures. It should be noted that the above pressures allow only for a horizontal backfill. Where a sloping backfill is present the additional load applied may be accounted for by either adopting a higher k value or the sloping backfill may be treated as a surcharge load.

Where a soldier pile wall is adopted care must be taken that the piling contractor is able to penetrate the underlying bedrock to a depth sufficient to achieve the required socket to provide appropriate lateral restraint to the wall. In this regard bands of sandstone bedrock of up to at least medium strength should not be unexpected and will require the use of rock augers with tungsten carbide teeth and possibly coring buckets. Vertical strip drains should be installed behind the shotcrete and mesh or reinforced concrete panels at spacings of no greater than 1.5m and should discharge to the dish drain at the toe of the wall for controlled discharge to the stormwater system.

The soldier pile wall must be socketed into the underlying sandstone bedrock. Consequently, all lateral restraint for retaining walls will be achieved by socketing into the underlying bedrock. In this regard, where footings are socketed into sandstone of at least very low strength an allowable lateral pressure of 300kPa may be adopted for that part of the footing that extends below a nominal 0.5m socket.

5.2 Footings

All new structures, including the proposed driveway pavement running from the shared driveway to the carport must be designed as fully suspended and uniformly founded on the underlying sandstone bedrock. Footings founded on sandstone bedrock of at least very low strength may be designed for an allowable bearing pressure (ABP) of 1,000kPa.

Where the depth to bedrock is such that pad and strip footings cannot be accommodated piled footings will be required. Where piles are used, a shaft adhesion of 100kPa may be adopted for that part of the pile that extends below a nominal 0.3m socket and is founded in sandstone bedrock of at least very low strength.

Prior to pouring concrete, all footings should be free from all loose and softened materials and should be inspected by a geotechnical engineer to confirm that the design ABP's have been achieved. Due to the poor quality of the sandstone bedrock, where water ponds in the base of the footing it will soften the bedrock and no longer be suitable for an ABP of 1,000kPa. Where this occurs the footing will need to be re-excavated to remove all loose and softened materials. Where a delay in pouring is anticipated, a concrete blinding layer may be poured to protect the base of the footing following excavation, cleaning and inspection by a geotechnical engineer.

5.3 Soil Aggression

The soil aggression test results indicate that the soils and rock classify as a 'mild' exposure classification for concrete in contact with the ground in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'. For steel in contact with the ground the soils and rock classify as 'non-aggressive' in accordance with Table 6.5.2(C) of AS2159-2009.

5.4 Further Geotechnical Input

The following is a summary of further geotechnical input required and which has been detailed in the preceding sections:

- An assessment of the design of the existing retaining wall along the low side of shared driveway by the structural engineer and, if required test pits to confirm the materials on which the wall is founded, and
- Inspection of all footing excavations by a geotechnical engineer to confirm that the design ABP's have been achieved.

5 GENERAL COMMENTS

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A	B(i)	B(ii)	C	D	E	F	G	H
Assessed Likelihood	Barely Credible	Possible	Possible	Likely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely
Assessed Consequences	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant	Medium	Insignificant	Insignificant
Risk	Very Low	Very Low	Very Low	Low	Very Low	Very Low	Low	Very Low	Very Low
Comments	Assumed to be a properly engineered and constructed wall					.	Total destruction of house unlikely	Volume of soil mobilised likely to be small with short travel distances.	Volume of soil mobilised likely to be small and unlikely to extend far into the site.

* Property value assume to be $3.35 Million

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A**	**B(i)**	**B(ii)**	**B(iii)**	**C**	**D**	**E**	**F**	**G**	**H**
Assessed Likelihood	Barely Credible	Possible	Possible	Possible	Likely	Unlikely	Unlikely	Rare	Unlikely	Unlikely
Indicative Annual Probability	1×10^{-6}	1×10^{-3}	1×10^{-3}	1×10^{-3}	1×10^{-2}	1×10^{-4}	1×10^{-4}	1×10^{-5}	1×10^{-4}	1×10^{-4}
Duration of Use of Area Affected (Temporal Probability)	(i) At crest of Wall 5 minutes/week 4.96×10^{-4} (ii) At toe of wall 15 minutes/week 1.49×10^{-3}	(i) and (ii) At crest and toe of wall 5 minutes/week 4.96×10^{-4}	(i) and (ii) At crest and toe of wall 1 minute/week 9.92×10^{-5}	(i) At crest of Wall 1 minute/week 9.92×10^{-5} (ii) At toe of wall 15 minutes/week 1.49×10^{-5}	(i) and (ii) At crest and toe of wall 30 seconds/week 4.96×10^{-5}	(i)) At crest of Wall 30 seconds/week 4.96×10^{-5} (ii) 5 minutes/week 4.96×10^{-4}	(i) On the Slope 5 minutes/week 4.96×10^{-4} (ii) Below the Slope 1 minute/week 9.92×10^{-5}	10 hours/day 0.417	(i) On the Slope 1 minute/week 9.92×10^{-5} (ii) Below the Slope 30 seconds /week 4.96×10^{-5}	Above the slope 30 seconds/week 4.96×10^{-5}
Probability of Not Evacuating Area Affected	(i) 1 (ii) 0.9	(i) and (ii) 0.1	(i) and (ii) 0.1	(i) 0.5 (ii) 0.9	(i) and (ii) 0.1	(i) and (ii) 0.1	(i) 1 (ii) 0.1	1	(i) 1 (ii) 0.1	0.1
Vulnerability to Life if Failure Occurs Whilst Person Present	(i) 0.01 Likely to ride wall failure down (ii) 1 Likely to be buried	(i) and (ii) 0.01 Unlikely to be buried	(i) and (ii) 0.01 Unlikely to be buried	(i) 0.01 Likely to ride failure down (ii) 0.8 Possibly crushed	(i) and (ii) 0.01 Unlikely to be buried	(i) and (ii) 0.01 Unlikely to be buried	(i) and (ii) 0.01 Unlikely to be buried	0.1 House unlikely to collapse	(i) and (ii) 0.01 Unlikely to be buried	0.01 Unlikely to be buried
Risk for Person Most at Risk	(i) 4.96×10^{-12} (ii) 1.34×10^{-9}	(i) 4.96×10^{-10} (ii) 4.96×10^{-10}	(i) 9.92×10^{-11} (ii) 9.92×10^{-11}	(i) 4.96×10^{-10} (ii) 1.07×10^{-6}	(i) 4.96×10^{-10} (ii) 4.96×10^{-10}	(i) 2.46×10^{-11} (ii) 2.46×10^{-10}	(i) 4.96×10^{-10} (ii) 9.92×10^{-12}	4.17×10^{-7}	(i) 9.92×10^{-11} (ii) 4.96×10^{-12}	4.96×10^{-12}
Combined Risk for Person Most at Risk	1.49×10^{-6}									

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Macquarie Park NSW 2113
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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31892LY
Project:	Proposed Subdivision	**Report:**	A
Location:	22 Hay Street, Collaroy, NSW	**Report Date:**	12/10/2018
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	2.63 - 2.67	0.08	2
	2.91 - 2.95	0.1	2
	3.20 - 3.23	0.07	1
	3.33 - 3.36	0.1	2
4	3.24 - 3.28	0.5	10
	3.91 - 3.94	0.7	14
	4.28 - 4.31	0.5	10
	4.77 - 4.81	0.2	4
	5.41 - 5.45	0.2	4
	5.86 - 5.89	0.2	4
	6.23 - 6.26	0.2	4
	6.63 - 6.67	0.2	4

NOTES:

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

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS 202962

Client Details

Client	JK Geotechnics
Attention	B Jonak, L Speechley
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31892LY, Collaroy**
Number of Samples	2 Soil
Date samples received	12/10/2018
Date completed instructions received	12/10/2018

Analysis Details

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

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

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

Report Details

Date results requested by	19/10/2018
Date of Issue	19/10/2018

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil			
Our Reference		202962-1	202962-2
Your Reference	UNITS	BH2	BH4
Depth		0.7-0.8	1.5-1.7
Date Sampled		26/09/2018	09/10/2018
Type of sample		Soil	Soil
Date prepared	-	16/10/2018	16/10/2018
Date analysed	-	16/10/2018	16/10/2018
pH 1:5 soil:water	pH Units	7.0	4.9
Sulphate, SO4 1:5 soil:water	mg/kg	<10	50
Chloride, Cl 1:5 soil:water	mg/kg	51	29
Resistivity in soil*	ohm m	120	120

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			16/10/2018	[NT]	[NT]	[NT]	[NT]	16/10/2018	[NT]
Date analysed	-			16/10/2018	[NT]	[NT]	[NT]	[NT]	16/10/2018	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	92	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1 / 2

BOREHOLE LOG

Client: ANTHONY JEFFCOAT
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY **Method:** HAND AUGER **R.L. Surface:** ~37.6 m
Date: 26/9/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** J.B.J./W.T.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to medium grained, with brick and terracotta fragments.	M			GRASS COVER APPEARS POORLY COMPACTED
						37	1		CH	Silty CLAY: high plasticity, light brown, trace of sandy bands.	w>PL	St - VSt		RESIDUAL
										Silty CLAY: high plasticity, light grey mottled red brown.		Hd		
						36	2		CI-CH	Sandy CLAY: medium to high plasticity, light grey mottled orange brown.				
						35, 34, 33, 32, 31	3, 4, 5, 6			REFER TO CORED BOREHOLE LOG				

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31892SY COLLAROY.GPJ <<DrawingFile>> 07/11/2018 11:31 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 1

CORED BOREHOLE LOG

Client: ANTHONY JEFFCOAT
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY | **Core Size:** NMLC | **R.L. Surface:** ~37.6 m
Date: 26/9/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** J.B.J./W.T.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific / General	Formation
		36	2		START CORING AT 2.06m						
100% RETURN					NO CORE 0.25m					~~(2.31m) XWS, 0°, 160 mm.t~~	
		35	3		SANDSTONE: fine to medium grained, light grey mottled orange brown.	HW	VL - L			(2.56m) Be, 10°, P, R, XWS 10mm.t (3.36m) Be, 10°, Ir, R, Clay Vn (3.45m) J, 45°, Ir, R, Cn (3.53m) Be, 10°, Ir, R, XWS 5mm.t	Narrabeen Group
		34			END OF BOREHOLE AT 3.61 m						
		33	4								
		32	5								
		31	6								
		30	7								

JK 9.01.2 LIB.GLB Log JK CORED BOREHOLE - MASTER 31892SY COLLAROY.GPJ <<DrawingFile>> 07/11/2018 11:31 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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JOB N° 31892LY
BH1
START CORING AT: 2.06m
2
CORE LOSS .25m
3
END OF CORE 3.61m

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
1 / 1

BOREHOLE LOG

Client: ANTHONY JEFFCOAT

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY **Method:** HAND AUGER **R.L. Surface:** ~37.7 m

Date: 26/9/18 **Datum:** AHD

Plant Type: **Logged/Checked By:** J.B.J./W.T.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to medium grained, dark grey, with brick and terracotta pipe fragments, trace of root fibres.	M			APPEARS POORLY COMPACTED
						37			CH	Silty CLAY: high plasticity, light brown.	w>PL	St - VSt		RESIDUAL
							1							
						36								
							2		CI-CH	Sandy CLAY: medium to high plasticity, light grey mottled yellow brown, trace of extremely weathered sandstone bands.		Hd		
										END OF BOREHOLE AT 2.10 m				
						35								
							3							
						34								
							4							
						33								
							5							
						32								
							6							
						31								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3
1 / 1

BOREHOLE LOG

Client: ANTHONY JEFFCOAT
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY **Method:** HAND AUGER **R.L. Surface:** ~41.0 m
Date: 10/10/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** J.B.J./W.T.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS					FILL: Silty gravelly sand, fine grained, dark brown, with fine to medium grained sandstone gravel.	M			APPEARS POORLY COMPACTED
										FILL: Silty clayey sand, fine grained, dark brown.	W			
						40	1		CH	Silty CLAY: high plasticity, light brown.	w>PL	St	160 170 150	HAND PENETROMETER TESTS COMPLETED ON REMOULDED SAMPLES
						39	2			Silty CLAY: high plasticity, light grey mottled red brown, trace of ironstone gravel.		VSt	240 250 230	
						38	3			END OF BOREHOLE AT 2.20 m				REFUSAL IN INFERRED SANDSTONE BEDROCK
						37	4							
						36	5							
						35	6							

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JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4

BOREHOLE LOG

Client: ANTHONY JEFFCOAT
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY **Method:** HAND AUGER **R.L. Surface:** ~42.0 m
Date: 10/10/18 **Datum:** AHD
Plant Type: **Logged/Checked By:** J.B.J./W.T.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to medium grained, dark brown, trace of igneous gravel.	M			APPEARS POORLY COMPACTED
										FILL: Silty sandy clay, high plasticity, orange brown, trace of sandstone gravel.				
									CH	Silty CLAY: high plasticity, light brown, trace of ironstone gravel and roots.	w>PL	St	180 170 180	RESIDUAL HAND PENETROMETER TESTS COMPLETED ON REMOULDED SAMPLES
						41	1							
										Silty CLAY: high plasticity, light grey mottled red brown, trace of ironstone gravel.		VSt	250 280 230	
						40	2		CI	Sandy CLAY: medium plasticity, light grey.				
										REFER TO CORED BOREHOLE LOG				
						39	3							
						38	4							
						37	5							
						36	6							

JK 9.01.2 LIB.GLB Log JK AUGERHOLE - MASTER 31892SY COLLAROY.GPJ <<DrawingFile>> 07/11/2018 11:31 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.01.2 2018-04-02 Prj: JK 9.01.0 2018-03-20

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
4
2 / 2

Client: ANTHONY JEFFCOAT
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22 HAY STREET, COLLAROY, NSW

Job No.: 31892SY | **Core Size:** NMLC | **R.L. Surface:** ~42.0 m
Date: 10/10/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** J.B.J./W.T.

Water Loss\Level	RL (m AHD)	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	DEFECT DETAILS – DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) – Specific / General	Formation
	40	2	START CORING AT 2.10m				
100% RETURN	39	3	SANDSTONE: fine to medium grained, light grey mauve, with medium strength iron indurated bands and extremely weathered bands.	HW	VL - L	(2.20m) J, 80°, Ir, XWS 5mm.t (2.36m) XWS, 0°, 120 mm.t (2.52m) J, 90°, Ir, XWS 5mm.t (2.54m) J, 90°, Ir, Fe Ct, 1 mm.t (2.57m) XWS, 0°, 120 mm.t (2.85m) XWS, 0°, 37 mm.t (2.86m) J, 75°, Ir, Fe Sn (2.87m) XWS, 0°, 370 mm.t (3.38m) J, 85°, XWS 5mm.t	Narrabeen Group
			NO CORE 0.16m				
	38	4	SANDSTONE: fine to medium grained, light grey mauve, with medium strength iron indurated bands and extremely weathered bands.			(3.64m) J, 90°, P, Fe Ct, 1 mm.t (3.74m) XWS, 0°, 160 mm.t (3.97m) J, 90°, Ir, XWS SANDSTONE 5mm.t (4.10m) J, 75°, St, XWS SANDSTONE 5mm.t (4.20m) XWS, 0°, 50 mm.t	Narrabeen Group
	37 36	5 6	SANDSTONE: fine to medium grained, light grey with orange staining.		L	(4.62m) J, 75°, St, Cn (5.05m) J, 75°, St, Fe Vn (6.12m) J, 45°, P, Fe, 10 mm.t (6.52m) CS, 0°, 40 mm.t	
	35	7	END OF BOREHOLE AT 6.66 m				

POINT LOAD STRENGTH INDEX $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH

SPACING (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB Nº 31892
BH4
START CORING AT 2.13m
2
3
C/L 0.16m
4
5
6
END OF HOLE 6.66m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ANTHONY JEFFCOAT	
Project:	PROPOSED RESIDENTIAL DEVELOPMENT	
Location:	22 HAY STREET, COLLAROY, NSW	
Job No.	31892SY	Hammer Weight & Drop: 9kg/510mm
Date:	26-9-18	Rod Diameter: 16mm
Tested By:	J.B.J.	Point Diameter: 20mm

Test Location	1	2	3	4			
Surface RL	≈37.6m	≈37.7m	≈41.0m	≈42.0m			
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	4	4	3	1			
100 - 200	4	3	1	1			
200 - 300	5	3	2	2			
300 - 400	14	4	4	3			
400 - 500	2	8	4	2			
500 - 600	4	3	2	3			
600 - 700	5	3	2	2			
700 - 800	6	3	3	2			
800 - 900	6	4	2	3			
900 - 1000	8	4	3	3			
1000 - 1100	6	6	4	4			
1100 - 1200	6	4	6	6			
1200 - 1300	6	5	10	8			
1300 - 1400	9	6	12	10			
1400 - 1500	15	6	16	12			
1500 - 1600	15	5	20	15			
1600 - 1700	14	6	23	18			
1700 - 1800	15	6	20	22			
1800 - 1900	18	6	23	25			
1900 - 2000	28	13	21	30			
2000 - 2100	30/50mm	30/80mm	35	35/80mm			
2100 - 2200	REFUSAL	REFUSAL	6/5mm	REFUSAL			
2200 - 2300			REFUSAL				
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

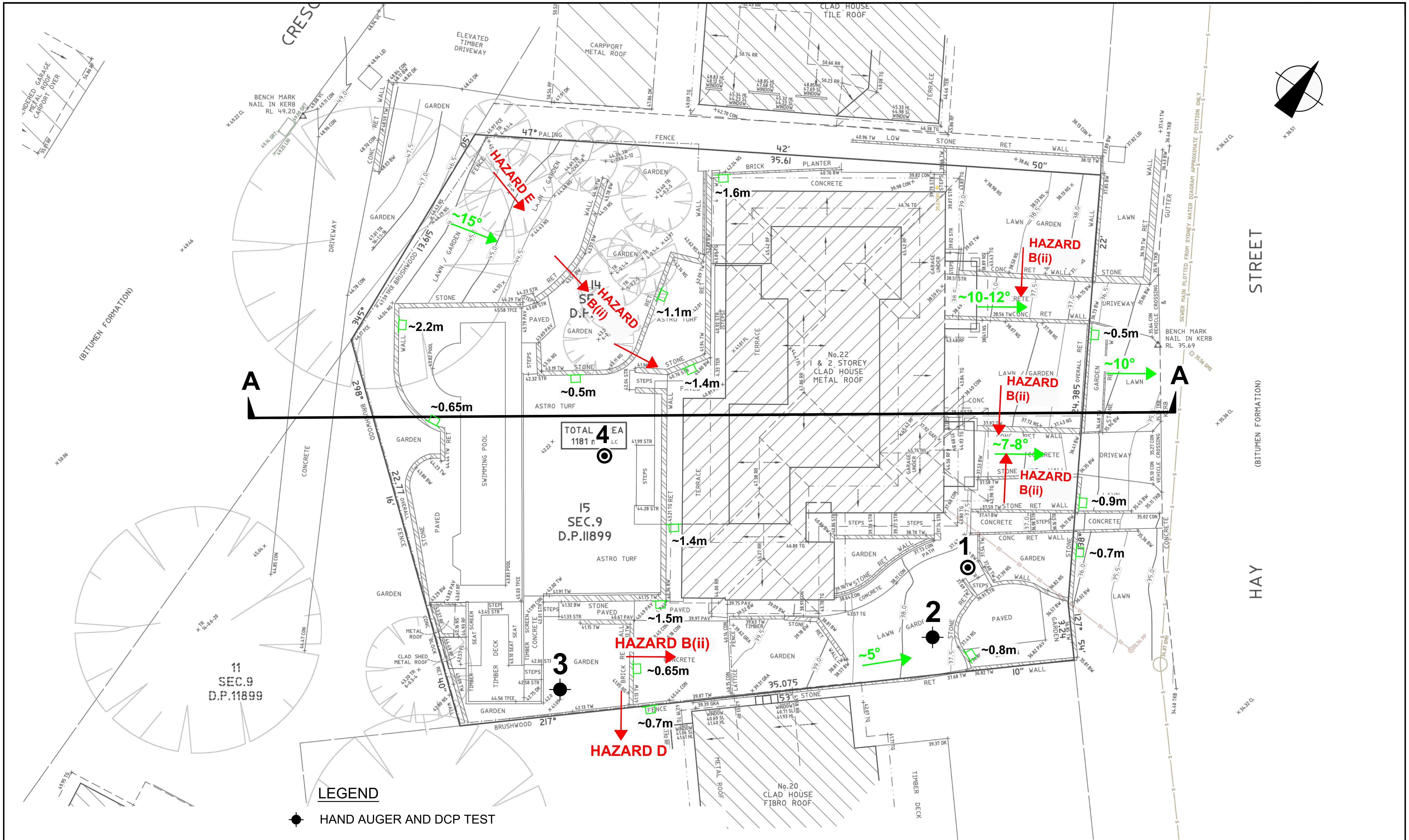
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	22 HAY STREET COLLAROY, NSW
Report No: 31892LY	Figure No: 1

JK **Geotechnics**

PLOT DATE: 17/10/2018 12:27:38 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31892LY COLLAROY\CAD\31892LY.DWG



LEGEND

- HAND AUGER AND DCP TEST
- HAND AUGER, DCP & CORED BOREHOLE

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title: **TEST LOCATION & TOPOGRAPHICAL MAPPING PLAN**

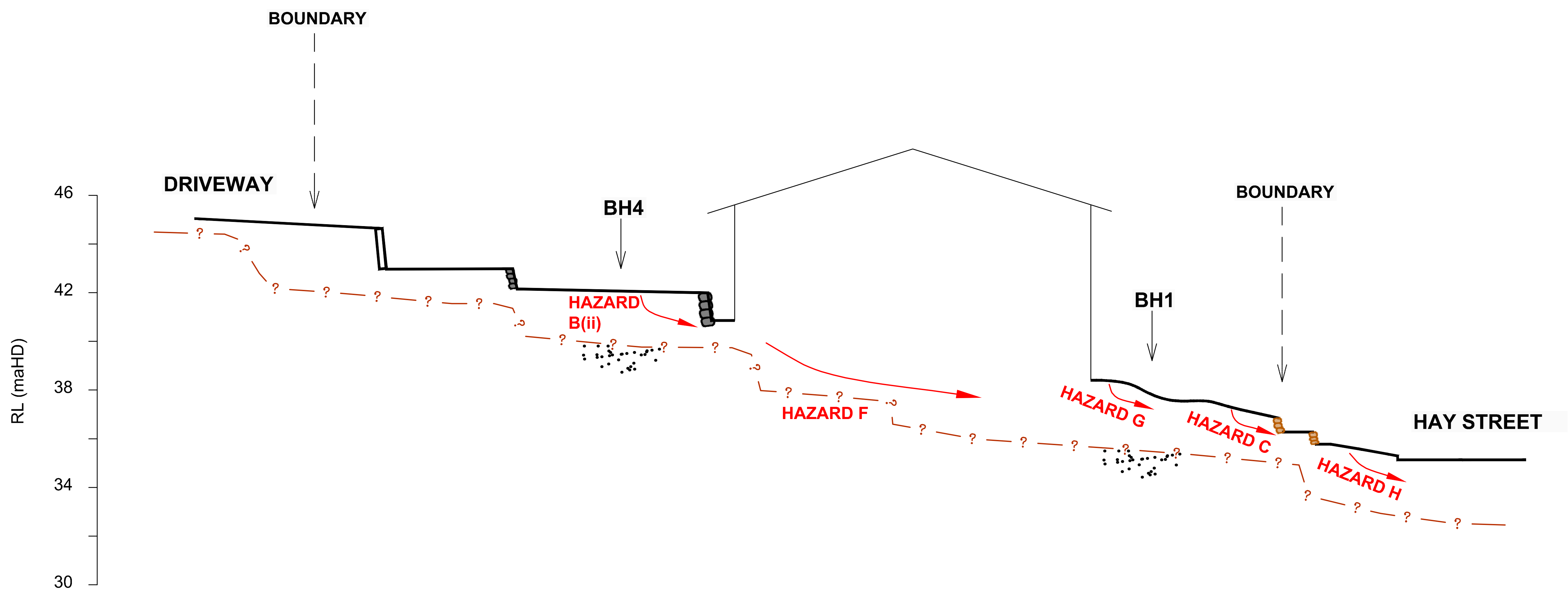
Location: 22 HAY STREET COLLAROY, NSW

Report No: 31892LYRev1

Figure No: 2

JK Geotechnics

PLOT DATE: 21/08/2019 11:49:48 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31892LY COLLAROY\CAD\31892LYREV2.DWG



0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title: **SECTION A-A**

Location: 22 HAY STREET COLLAROY, NSW

Report No: 31892LY

Figure No: 3

JK **Geotechnics**

PLOT DATE: 8/11/2018 3:48:43 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31892LY COLLAROY\CAD\31892LY.DWG

GEOTECHNICAL MAPPING SYMBOLS

TOPOGRAPHY

Symbol *Ground Profile*

Symbol	Ground Profile		
		convex	well defined or angular break of slope
		concave	
		convex	poorly defined or smooth change of slope
		concave	

breaks of slope / changes of slope — convex and concave too close together to allow the use of separate symbols

sharp / rounded — ridge crest

Cliff or escarpment or sharp break 40° or more (estimated height in metres) — 3

15 Uniform Slope / 10 Concave Slope / 8 Convex Slope — Slope direction and angle (Degrees)

Top / Bottom — Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

OTHER FEATURES

Boulder

Seepage/spring

Swallow hole for runoff

Natural water course

Open drain, unlined

Open drain, lined

Fenceline

Property boundary

Dry Stone Wall

J —— J 200 Major joint in rock face (opening in millimetres)

– T – T – 10 Tension crack (opening in millimetres)

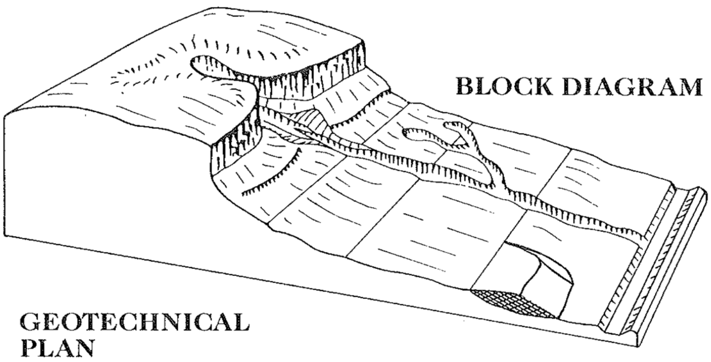
Masonry or concrete wall

Ponding water

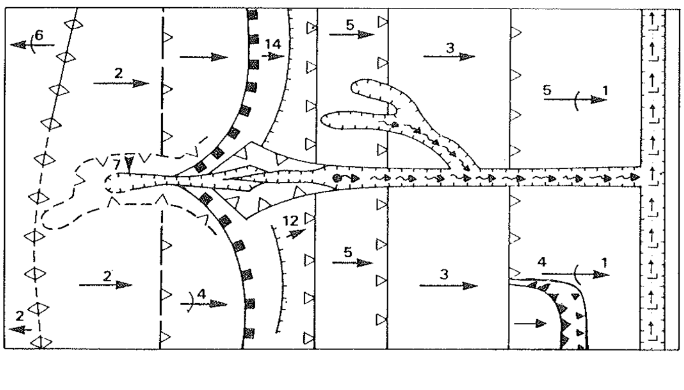
Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

BLOCK DIAGRAM



GEOTECHNICAL PLAN



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

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GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 31892LYrpt Figure No 4

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

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

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

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

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed 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	

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

Laboratory Classification Criteria

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

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

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

NOTES:

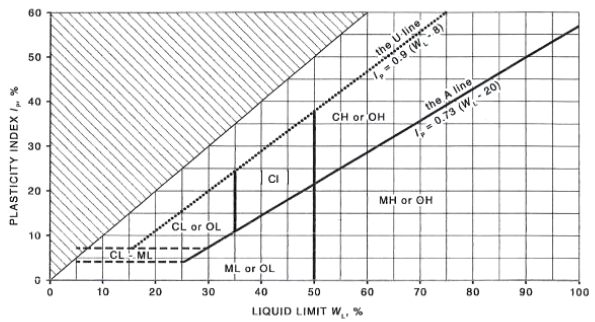
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

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

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

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



LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

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

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (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
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

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

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

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

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

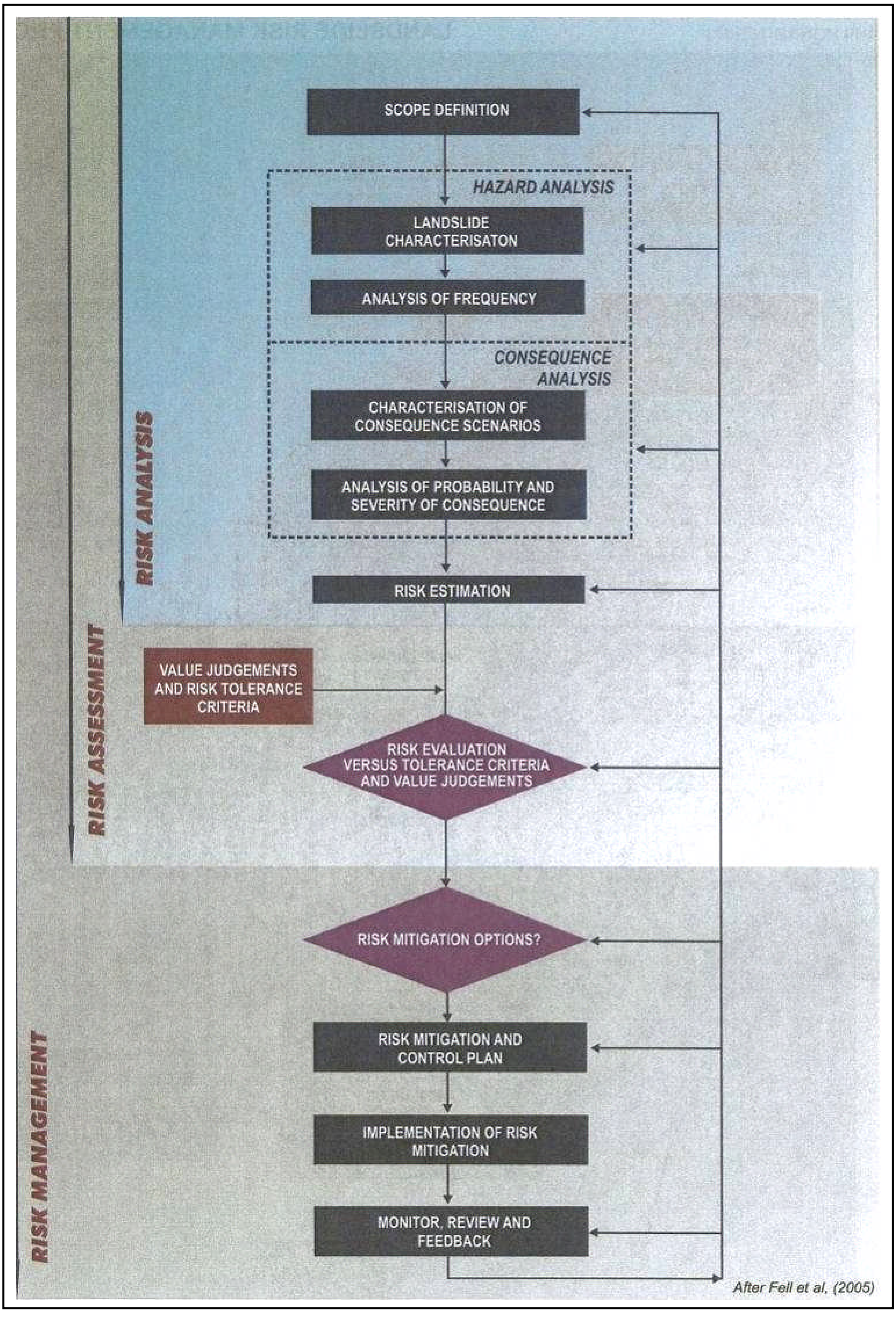


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

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

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

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

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

What Causes a Landslide?

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

Does a Landslide Affect You?

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

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

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

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

TABLE 1 – Slope Descriptions

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

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

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

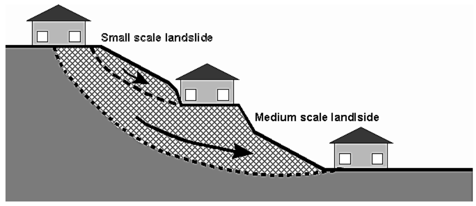


Figure 1

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

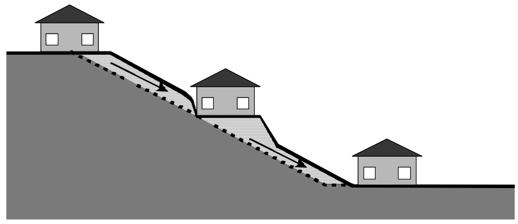


Figure 2

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

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

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

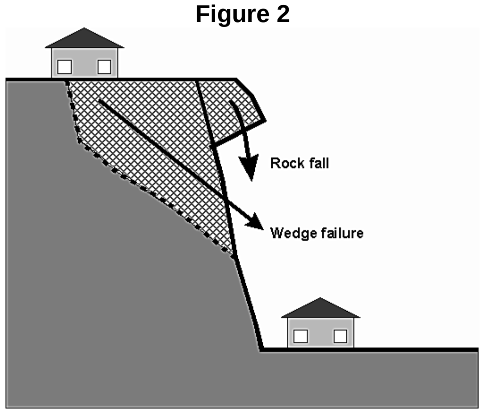


Figure 3

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

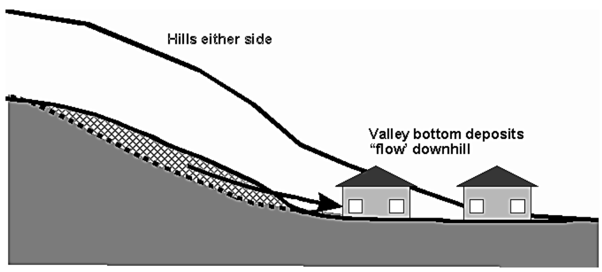


Figure 4

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

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

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

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

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

Landslide Risk Assessment

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

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- 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.	

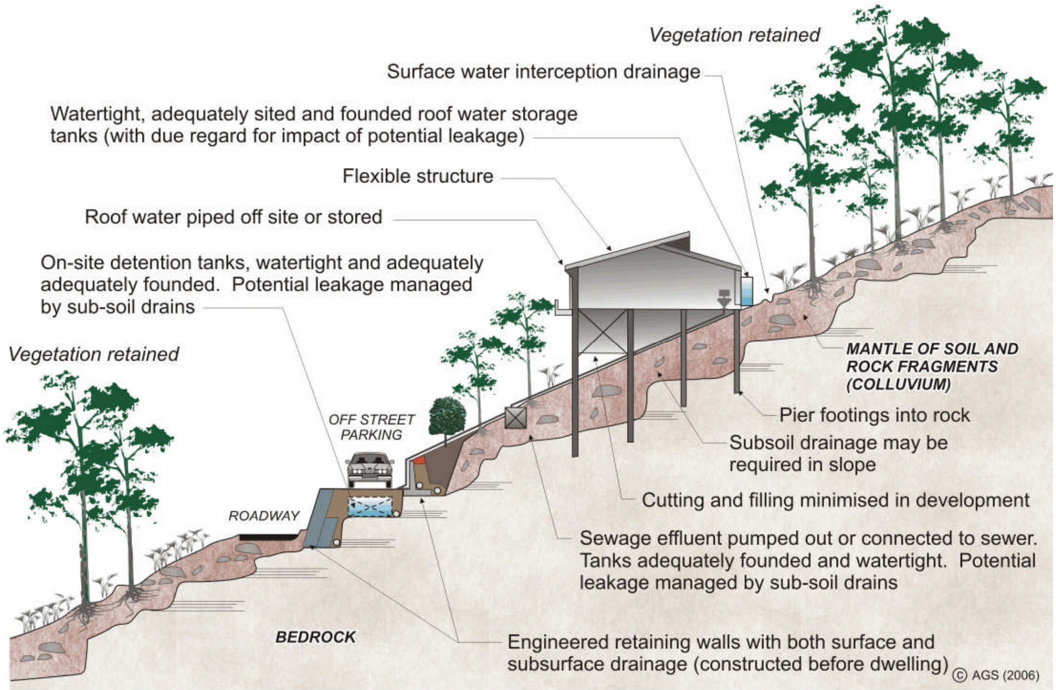
This table is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in *Australian Geomechanics*, Vol 42, No 1, March 2007 which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

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.

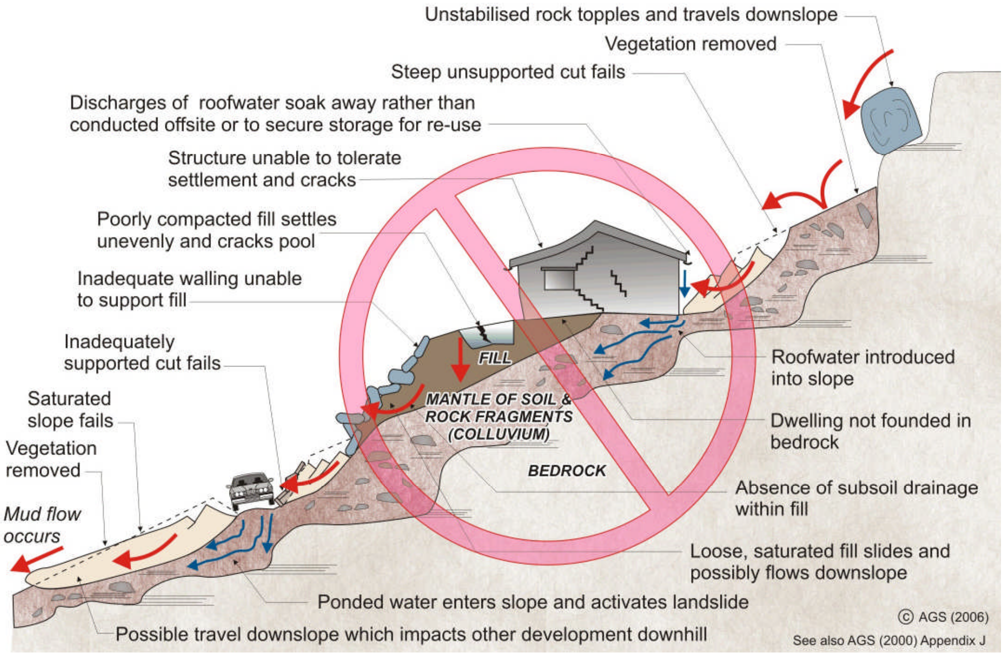
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

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

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Extract from Geoguide LR8 – Hillside Construction Practice.