SOIL SURVEYS

SLOPE STABILITY ASSESSMENT

PROJECT NO. 1-28192

SEPTEMBER, 2024

GIOVANNI D'ERCOLE

PROPOSED GARAGE STRUCTURE

12 BROWNELL DRIVE, BYRON BAY

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1.0 INTRODUCTION

1.1 General

This report presents the results of the investigation, analysis and slope stability assessment carried out by Soil Surveys Engineering Pty Limited (SSE) for the Proposed Garage Structure at 12 Brownell Drive, Byron Bay.

The objectives of this analysis were to carry out a slope stability risk assessment for the proposed garage structure, in accordance with the Scope of Services in Section 1.3, as detailed in our original cost proposal (refer 1-28192 PR 1.1) dated 29th August 2024.

1.2 Proposed Development

It is understood that the proposed development is to consist of the construction of a new subterranean garage structure, cut into the slope of the site, accessed off Julian Place at the northern corner of the lot (refer Figure 1).

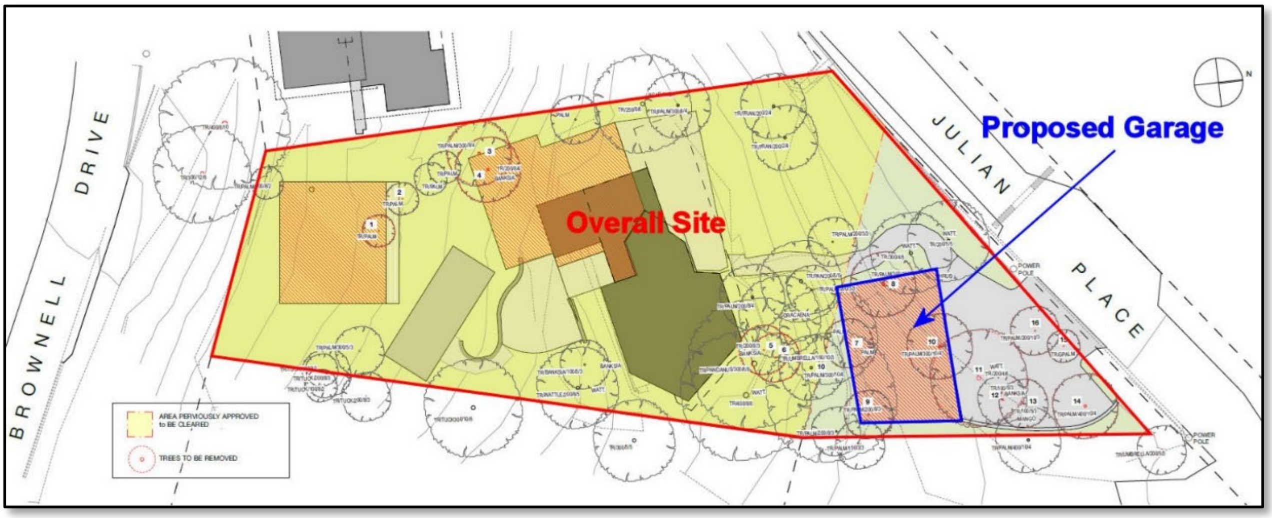

FIGURE 1 - PROPOSED DEVELOPMENT LAYOUT – DWG 04, REV A, 4/12/23

Based upon the provided drawings, excavations of up to approx. 3m are proposed to allow construction of the three-sided garage block walls.

Building loads have not been provided, however based on the provided drawings it has been assumed that they would be generally consistent with domestic type structural loads.

1.3 Scope of Services

We understand that a Request for Information (RFI) from Byron Shire Council (BSC) has been issued requesting the following in relation to stability of the site (refer Item 4 of RFI for DA 10.2024.203.1, dated 19th July, 2024):-

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>
> 4. The submitted geotechnical site investigation relates to an existing dwelling and is outdated. Include boreholes for the proposed development location and as such an updated geotechnical report from a professional Engineer experienced in Geotechnical Science is to be provided to demonstrate that the site is stable and will not be affected by landslip or subsidence at, above or below the site when the excavation is undertaken and buildings constructed.

The objective of this study would therefore be to carry out a review of the previous investigation and field data to assess the proposed garage development and provide recommendations for temporary and permanent stability. The following will be included:-

- Civil works recommendations
    - Excavatability
    - Batter recommendations
- Foundation recommendations
- Retaining wall design parameters
- Stability risk assessment, in accordance with National Disaster Mitigation Program (NDMP), Landslide Risk Management (LRM) Guidelines, Practice Notes and Geoguidelines as published in the "Australian Geomechanics Journal" Volume 42 No. 1, March, 2007 – To address Council Request (Item 4) noted in Section 1.2 above.
- Construction recommendations (where applicable)

1.4 Provided Information

The following information was provided to SSE and reviewed as part of the investigation and assessment:-

- Contour & Detail Survey by Ardill Payne & Partners, refer 8531, Dwg. S03, dated 26th October 2021.
- Proposed DA Site Drawings Set, Drawings 01 to 18, Revision A, dated 4th December 2023.
- Civil Engineering Drawings by SDS Civil Enterprises, refer:-
    - CIV – C1 to C4, Revision A, dated September 2024.
    - SWMP – 01 to 03, Revision A, dated September 2024.
- Engineering Driveway Access and Stormwater Management Report by SDS Civil Enterprises, refer 2104, Revision A, dated September 2024.

1.5 Previous Reports

SSE have carried out previous investigations and reports for this site as part of previous proposed developments. The most recent was a March 2007 report (refer 205-5614) which included a deep borehole at the northern end of the site, located within the approx. footprint of the proposed garage structure and will be utilised in this report.

2.0 SITE DETAILS

2.1 Location

The site is located at 12 Brownell Drive, Byron Bay (refer Figure 2), also known as Lot 1 in DP1208728. Access to the sites was via Julian Place along the northern site boundary (refer Figure 3).

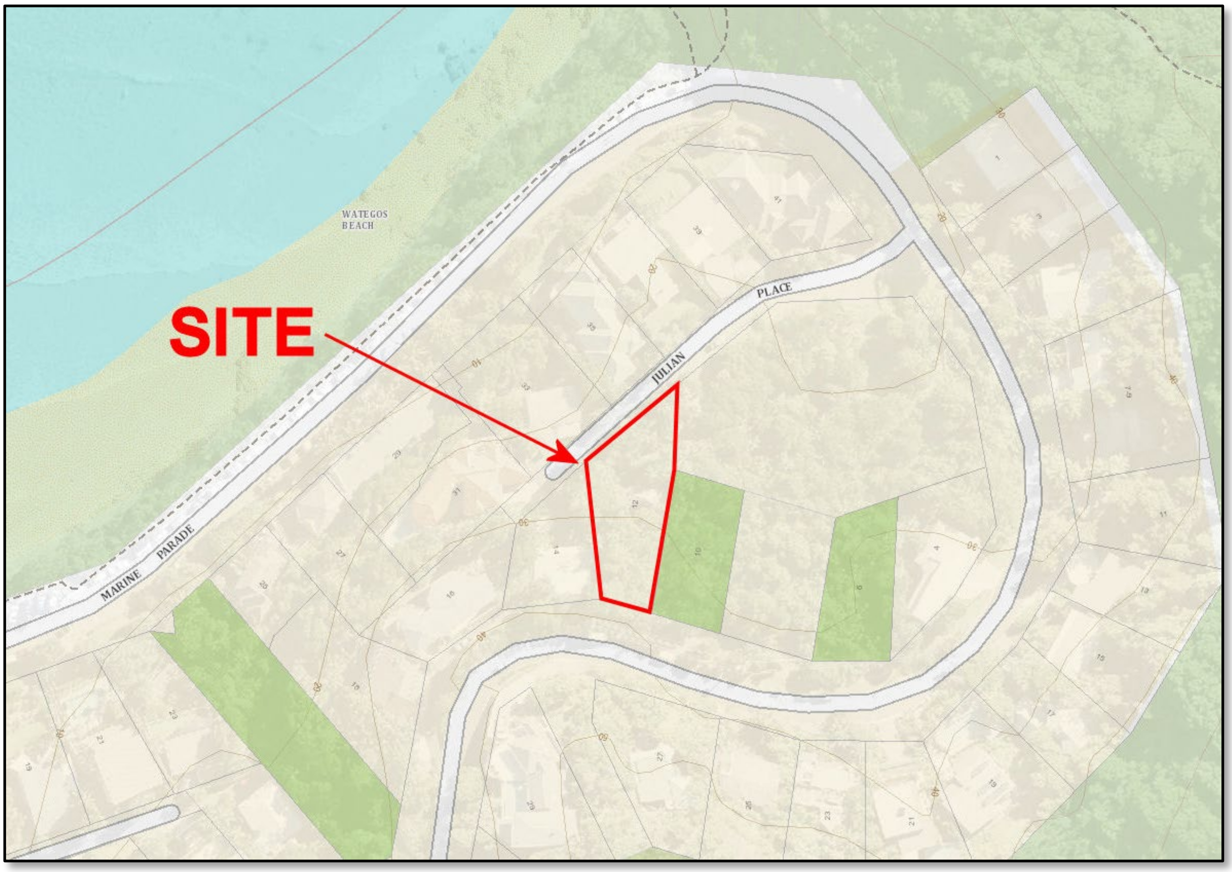

FIGURE 2 - SITE LOCATION (NSW SIXMAPS)

2.2 Site Factors

A recent aerial photograph of the site (refer Figure 3) provides an overview of the existing site conditions. The principal site features are:-

- The overall site comprises a roughly rhombohedral shape lot with narrow frontage onto Brownell Drive at the southern end and wide frontage to Julian Place (refer Photo C1, Appendix C) at the northeastern side (refer Figure 2).
- The overall site spans the western side of a broad ridgeline which runs roughly north south, with an overall slope of between approx. 10º to 18º towards the north northeast (based on Council Mapping).
- The subject site includes slopes of generally 10º to 14º, with low height (approx. 1.0m to 1.2m) stone and copper log walls forming small terraces down the slope (refer Photo C2, Appendix C), as well as along the northern boundary line with Julian Place.
- At the time of the inspection, the site was covered by well landscaped gardens and lawns, with numerous small to medium sized trees (refer Photo C3, Appendix C).
- Drainage has been assessed as fair to good due to the predominantly sloping surface.

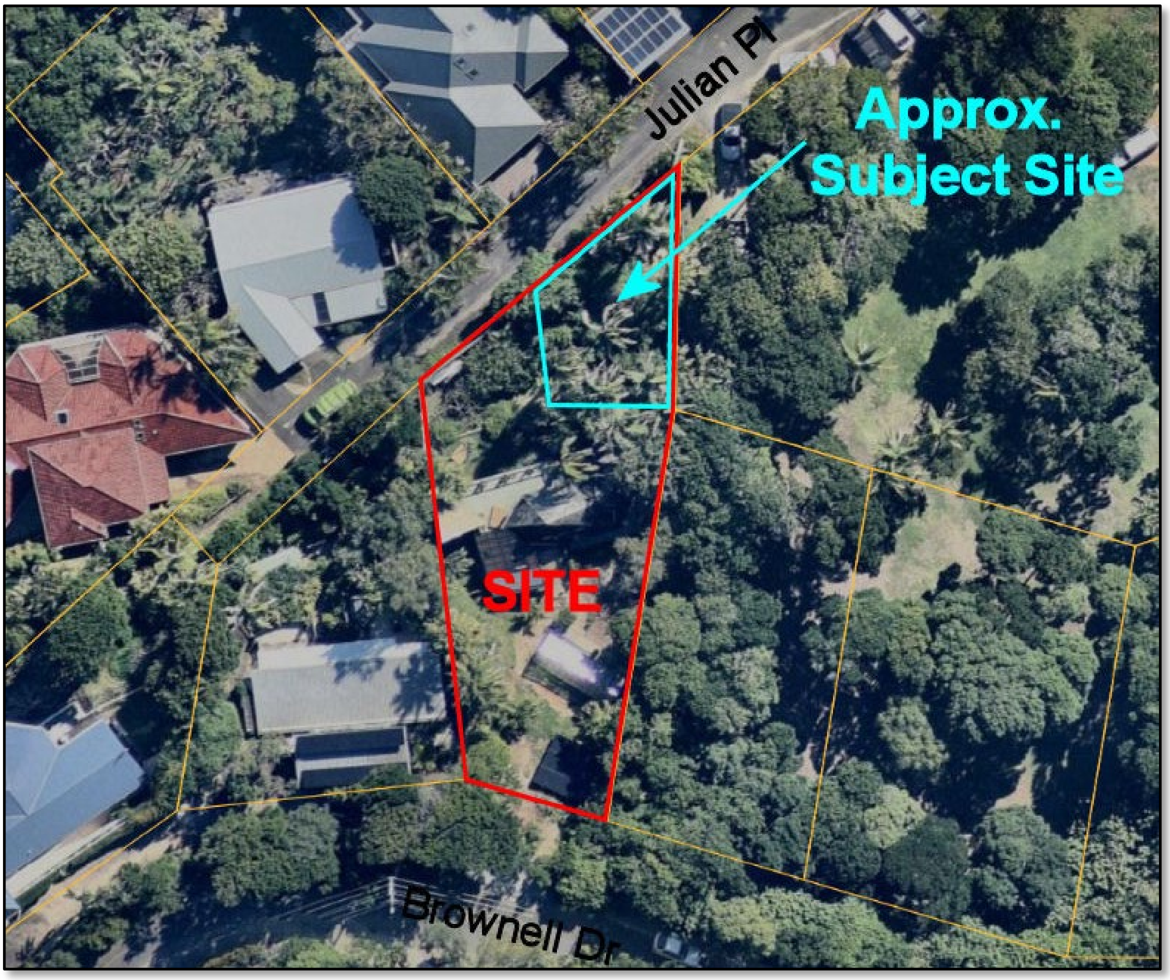

FIGURE 3 - SITE AERIAL (NEARMAP.COM, 15th JULY, 2024)

3.0 SITE ASSESSMENT AND DATA COLLECTION

3.1 Regional Geology

According to the Tweed Heads 1:250,000 Geology Map (Sheet SH 56-3) published by the Geological Survey of NSW (1972), the site is underlain by the weathering products of the Devonian to Carboniferous Age (~380-320Mya) Neranleigh-Fernvale beds.

These rocks generally consist of greywacke, slate, phyllite and quartzite (refer Figure B1, Appendix B).

3.2 Field Investigation

The site was inspected and mapped by a Senior Engineering Geologist, experienced in slope stability assessments on the 11th September 2024.

As part of a previous development on the site, subsurface conditions were investigated by drilling and sampling one borehole to a depth of 6.00m, using a 4WD truck mounted HP7 drilling rig in January 2007.

The borehole included Standard Penetrometer Tests (SPT's) and recovered 3.0m of NMLC rock core.

The soil classification descriptions, field and laboratory testing were carried out in general accordance with the following Australian Standards:-

- AS 1726-2017 'Geotechnical Site Investigations'
- AS 1289 'Methods of Testing Soils for Engineering Purposes'

A description of the investigation method, borehole records and site plan showing investigation locations are included in the Appendices.

3.3 Subsurface Profile

The subsurface profile intersected during the 2007 drilling program within the current subject site comprised generally Clayey Sandy Gravel overlying weathered metamorphic rock.

Very dense Clayey Sandy GRAVEL (GC) was encountered to a depth of 1.0m, overlying the weathered rock. The weathered rock generally comprised weak (now classified as very low to low strength) Extremely Weathered Metasandstone/Metasiltstone (XW), becoming Distinctly to Slightly Weathered (DW-SW) and strong (now classified as high strength) at a depth of 3.0m and continued to borehole termination depth of 6.0m.

Nearby rock outcrops in cuttings along Julian Close to the west of the site (in 2005) included Extremely to Distinctly Weathered, very low to low strength Metasiltstone (XW-DW), highly fractured with localised bands of siliceous (high silica content) Metasiltstone of low to medium strength.

Refer to the borehole records in Appendix D for detailed subsurface profile descriptions and site plan (Figure B2) in Appendix B showing the approximate 2007 borehole location.

3.4 Groundwater

Whilst groundwater was not encountered in the 2007 boreholes or noted on site at the time of the recent inspection, it should be noted that groundwater may be present; however, due to the relatively short time that observations are able to be made during drilling of the boreholes or time on site, and the possible effect of smear on the sides of the boreholes, groundwater was not identified.

However, seepage may be encountered, particularly within the upper layer Sandy Gravels and at the soil/rock interfaces. It should also be noted that groundwater levels can vary both seasonally and with prevailing weather conditions.

4.0 SLOPE STABILITY RISK ASSESSMENT

4.1 Introduction

This section of the report presents the results of the slope stability risk assessment carried out by Soil Surveys Engineering Pty Limited for the above subject site. The assessment has considered the existing site condition and proposed development during and following construction.

4.2 Controls and Procedures (Slope Stability)

The slope assessment was undertaken in accordance with the National Disaster Mitigation Program (NDMP), Landslide Risk Management (LRM) Guidelines, Practice Notes and Geoguidelines as published in the "Australian Geomechanics Journal" Volume 42 No. 1 March, 2007.

4.3 Assessment Methodology

The assessment of the stability of slopes at the site has utilised a Hazard and Risk Assessment approach. In this method, the potential stability hazards on the site are assessed using certain features/properties of the site (i.e., slope angle, ground water conditions, etc.). This method provides a ranking for each of the identified hazards.

Using this hazard ranking and based on assumed consequences, an assessment of risk can then be made using a risk matrix from National Disaster Mitigation Program (NDMP), "Landslide Risk Management (LRM) Guidelines" (refer Appendix E).

The general processes in assessment and management of risks associated with landslides are given in detail in the National Disaster Mitigation Program (NDMP), "Landslide Risk Management (LRM) Guidelines", Practice Notes and Geoguidelines as published in the Australian Geomechanics Journal Volume 42 No. 1, March 2007.

The effects of earthquake on slope stability have not been included in this study.

This study has identified hazard ratings using a classification system consistent with the procedures detailed in the paper entitled "A Method of Zoning Landslide Hazards", prepared by McGregor and Taylor. This method has been adopted on a wide range of projects and has proven to be robust.

4.4 Site Assessment and Data Collection

The subject site is described in detail in Section 2.2 with Site Geology in Section 3.1.

The searchable Geoscience Australia Landslide Database was reviewed. No landslide locations were recorded in the vicinity of the site.

The geological model for the site is based upon the site investigation detailed in Section 3.3.

4.5 Risk Assessment - Stability

4.5.1 General

For the purposes of this study, the assessment of Risk for the site has been based on risk to property and risk to persons, where Risk is the product of Likelihood and Consequence.

As noted above, this method is based on the method outlined in the “Practice Note Guidelines for Landslide Risk Management 2007” (PNGLRM), produced by the Australian Geomechanics Society, 2007.

The following assessment is also based on the premise that all works are undertaken in accordance with our recommendations.

4.5.2 Identification of Potential Hazards on Site

Based on the site walkover, a review of the published and site information and experience on similar sites, the following potential land stability hazards have been identified for assessment within the proposed development site:-

A. Shallow landslides on the existing natural slopes - applies to overall site.
B. Proposed cut batters - applies to upslope cut batters for the garage construction.
C. Surface scour on the site - applies to overall site.

4.5.3 Likelihood Estimation

Methods of Assessment

The site was assessed and zoned using the method as outlined in MacGregor and Taylor (2001). The method has been adopted as a slope stability assessment tool and used in both the Gold Coast and Redlands Shire regional stability assessments.

Evaluation of Likelihood Rating for Existing Overall Site

Once the relative frequency of slope instability for the site is calculated, MacGregor and Taylor (2001) suggest a likelihood rating as outlined in Table 1 below.

This rating has then been compared to the Likelihood Descriptor from Appendix C of “Practice Note Guidelines for Landslides Risk Management (PNGLRM) (2007c)”.

TABLE 1 LIKELIHOOD RATING

MacGregor and Taylor (2001)		Landslide Risk Management (2007c) Appendix C	
Relative Frequency	**Likelihood Rating**	**Likelihood Descriptor**	**Adopted Indicative Value of Annual Probability**
< 0.2	Very Low	Barely Credible	10^{-6}
0.2 to 0.6	Low	Rare to Unlikely	10^{-5} to 10^{-4}
0.6 to 2.0	Moderate	Unlikely to Possible	10^{-4} to 10^{-3}
2.0 to 6.0	High	Possible to Likely	10^{-3} to 10^{-2}
> 6.0	Very High	Almost Certain	10^{-1}

Notes:-
1. McGregor & Taylor (2001).
2. Likelihood adopted based on Landslide Risk Management (2007c), Appendix C.

Hazard A - Natural Shallow Landslides - Overall Site

The overall site was assessed using the parameters as described by MacGregor and Taylor (2001) and the site features identified on the supplied information, observations made whilst on site and where necessary worst-case parameters.

Therefore, based on the above, **natural shallow landslides** on the subject site in its current form has been given a likelihood hazard rating of **Very Low to Low (low end)**, which translates to a Likelihood Descriptor of **Barely Credible to Rare.**

Hazard B - Proposed Cut Batters

Based on the provided drawings, it is understood that upslope cut batters will be required around three sides of the proposed garage structure, with heights of between approx. 1.5m and 2.8m. Cuts are expected to be temporary during construction (using our recommendations) and understood to be permanently supported by engineer designed and certified concrete block retaining walls.

To assess these cuts, the method outlined by MacGregor & Taylor (2001) for the assessment of cut has been adopted, assuming either engineer designed and certified retaining structures or temporary cut batters as per our recommendations in Section 5.2.2 of this report.

Therefore, an assessment of the **proposed cuts** has been given the following:-

- Temporary Cut Batters - A likelihood hazard rating of **Low** which translates to a Likelihood Descriptor of **Rare to Unlikely.**
- Retained Cuts - A likelihood hazard rating of **Very Low** which translates to a Likelihood Descriptor of **Barely Credible.**

Hazard C - Scour of Exposed Surfaces - Overall Site

As noted above, the methods as adopted for Hazards A and B do not have a corresponding method for the analysis of scour (Hazard C). The following comments will allow the hazard to be assessed:-

- The proposed development will require the clearing of the majority of the proposed subject site.
- Exposed materials would likely be residual clay soils and weathered rock.
- Experience with these types of materials would suggest that the exposed material would scour easily if left exposed and water is allowed to run over the slopes.

Based on the above comments and observations on site, the likelihood rating for the scour of exposed surfaces would be **Moderate to High**, which translates to a Likelihood Descriptor of **Possible.**

However, where scour and erosion are managed in accordance with the recommendations outlined in Section 5.2.3 of this report, they could be reassessed as **Very Low to Low** which translates to a Likelihood Descriptor of **Barely Credible to Unlikely**.

4.5.4 Consequence Analysis

General

As part of the assessment of the risk of slope instability, the analysis of the consequence of any failure must also be considered. This analysis is based on the effect on the following for each type of hazard:-

i) Consequence to Property (proposed future structure)

ii) Consequence to Persons within and around the proposed structure

Site Factors

The following has been based on the design assumptions and will be used to assume consequence:-

- The site works will include the construction of a new subterranean garage structure within the subject site.
- Cuts will be undertaken, temporarily battered using our recommendations during construction, then permanently supported using engineer designed and certified retaining structures.
- The proposed structure will not be used as a residence and only for parkin of vehicles and storage.
- All works will be undertaken in accordance with the recommendations of this report.

Consequence to Property

For the purposes of this study, a semi-qualitative measure of consequence to property will be adopted, as outlined in Appendix C of PNGLRM 2007c. If slope instability were to occur, the aspect of property most at risk would be the proposed new garage structure.

Based on Appendix A of PNGLRM 2007c the structures at risk would be classified as Importance Level 2.

Considering the above information, the consequence to property for each of the hazards taking into account the proposed development on the site is as follows:-

- Hazard A - **Minor**
- Hazard B - **Minor**
- Hazard C - **Insignificant**

Consequence to Persons

For the purposes of this study, the methodology as outlined in Section 7 of PNGLRM 2007, will adopt the following assumptions with respect to persons within the subject site:-

- The affected site will contain a new garage structure.
- The probability of spatial impact ($P_{(S:H)}$) - this has been based on the total width of the site exposed to possible instability compared to the total width of the proposed structures within that length. Assessed as 40%
- Proportion of time structures occupied:-
    - Within Structure = 10%.
    - Surrounding structure = 5%
- Probability of not being able to escape:-
    - Within Structure = 20%.

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    - Surrounding structure = 10%
- Temporal Spatial Probability ($P_{(T:S)}$) - Proportion of time structures occupied multiplied by the Probability of not being able to escape:-
    - Within Structure = 2%.
    - Surrounding structure = 0.5%
- Vulnerability ($V_{(D:T)}$), (Appendix F - PNGLRM 2007):-
    - Within Structure = 30%.
    - Surrounding structure = 10%

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4.5.5 Risk Determination and Evaluation - Property

Appendix C of "PNGLRM (2007)" outlines a method of assessing Risk to Property. The calculation of likelihood is based on the estimate outlined in Section 4.5.3 of this report.

The consequence is based on the proposed development and the effects on the property of the specific small sized hazards from Section 4.5.2.

The risk is then assessed based on the Risk Matrix shown in Appendix E. This is summarised in Table 2.

TABLE 2 SUMMARY OF RISK TO PROPERTY

Hazard	Area of Site	Hazard Rating	Assessed Likelihood	Assessed Consequence	Assessed Risk to Property
A	Overall Site	Very Low to Low	Barely Credible to Rare	Minor	Very Low
B	Temporary Cut Batters(3)	Low	Rare to Unlikely	Minor	Very Low to Low
	Retained Cuts(3)	Very Low	Barely Credible	Minor	Very Low
C	Overall Subject Site(2)	Moderate to High	Possible	Insignificant	Very Low
	Overall Subject Site(3)	Very Low to Low	Barely Credible to Unlikely	Insignificant	Very Low

Notes:
1. Refer Appendix B of "Practice Note Guidelines for Landslides Risk Management (2007)".
2. Where recommendations have not been put in place
3. Assuming recommendations outlined in this report are adopted and adhered to.

A review of the table above indicates that the risk of slope instability occurring on the site has been assessed as **Very Low to Low** based upon the current site condition and proposed development.

If the recommendations of this report are not followed, including cut batters, retaining structures and drainage above the batters/walls, then the long-term Risk could increase.

4.5.6 Risk Determination and Evaluation - Persons

Evaluation of the risk to life must be carried out on a quantitative basis to be consistent and for comparison with acceptance criteria. To do this, the indicative probabilities given in Appendix C of PNGLRM (2007) may be adopted as there are no other frequency/probability data.

The assessment results are shown in Table 3. The risk to persons relates to the risk for someone within the subject site, subject to each particular hazard.

The analysis is based on formula 1 in Section 7 of PNGLRM (2007), i.e.:-

$$R_{(LoL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times V_{(D:T)}$$

Where: $R_{(LoL)}$ = The risk for Loss of Life
$P_{(H)}$ = Annual probability of the Landslide - based on Section 4.5.3
The other factors are outlined in Section 4.5.4

The acceptance criteria (Table 4) adopted for this assessment is outlined in Section 8 of PNGLRM (2007) which suggests the following Risk to Life as outlined in Table 3.

TABLE 3 SUMMARY OF RISK TO LIFE CALCULATIONS

Hazard	Area of Site	Hazard Rating	Assessed Likelihood	$P_{(H)}$	$P_{(S:H)}$	$P_{(T:S)}$	$V_{(D:T)}$	$R_{(LoL)}$	Assessment
A	Overall Site	Very Low to Low	Barely Credible to Rare	10^{-6} to 10^{-5}	0.4	0.02	0.3	2×10^{-9} to 2×10^{-8}	Acceptable
B	Temporary Cut Batters(3)	Low	Rare to Unlikely	10^{-5} to 10^{-4}	0.4	0.02	0.3	2×10^{-8} to 2×10^{-7}	Acceptable
	Retained Cuts(3)	Very Low	Barely Credible	10^{-6}	0.4	0.02	0.3	2×10^{-9}	Acceptable
C	Overall Subject Site(3)	Very Low to Low	Barely Credible to Unlikely	10^{-6} to 10^{-4}	1.0	0.005	0.1	5×10^{-10} to 5×10^{-8}	Acceptable

Notes:
1. Refer Appendix B of "Practice Note Guidelines for Landslides Risk Management (2007)".
2. Where recommendations have not been put in place
3. Assuming recommendations outlined in this report are adopted and adhered to.

Based on the assessment above, the risk to life for the three hazards has been assessed to be **Acceptable,** based upon our noted assumptions and provided the recommendations of this report are followed.

If the recommendations are not followed, then the Risk could increase to Tolerable to Unacceptable.

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TABLE 4 RISK CRITERIA

Situation	Suggested Tolerable Loss of Life Risk limit for the person most at risk	Suggested Acceptable Loss of Life Risk limit for the person most at risk
Existing Slope	10^{-4} /annum	10^{-5} /annum
Existing Development	10^{-4} /annum	10^{-5} /annum
New Constructed Slope	10^{-5} /annum	10^{-6} /annum
New Development	10^{-5} /annum	10^{-6} /annum
Existing Landslide	10^{-5} /annum	10^{-6} /annum

Notes:
1. “Existing Slopes” in this context are slopes that are not part of a recognizable landslide and have demonstrated non-failure performance over at least several seasons or events of extended adverse weather, usually being a period of at least 10 to 20 years.
2. “Existing Development” includes existing structures, and slopes that have been modified by cut and fill, that are not located on or part of a recognizable landslide and have demonstrated non-failure performance over at least several seasons or events of extended adverse weather, usually being a period of at least 10 to 20 years.
3. “New Constructed Slope” includes any change to existing slopes by cut or fill or changes to existing slopes by new stabilisation works (including replacement of existing retaining walls or replacement of existing stabilisation measures, such as rock bolts or catch fences).
4. “New Development” includes any new structure or change to an existing slope or structure. Where changes to an existing structure or slope result in any cut or fill of less than 1.0m vertical height from the toe to the crest and this change does not increase the risk, then the Existing Slope / Existing Structure criterion may be adopted. Where changes to an existing structure do not increase the building footprint or do not result in an overall change in footing loads, then the Existing Development criterion may be adopted.
5. “Existing Landslides” have been considered likely to require remedial works and hence would become a New Constructed Slope and require the lower risk. Even where remedial works are not required per se, it would be reasonable expectation of the public for a known landslide to be assessed to the lower risk category as a matter of “public safety”.
6. Acceptable risks are usually considered to be one order of magnitude lower than the Tolerable Risks.
7. It is important to distinguish between “acceptable risks” and “tolerable risks”.
8. Tolerable Risks are risks within a range that society can live with so as to secure certain benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if practicable.
9. Acceptable Risks are risks which everyone affected is prepared to accept. Action to further reduce such risk is usually not required unless reasonably practicable measures are available at low cost in terms of money, time and effort.

4.5.7 Risk Summary

A summary of the results of the risk assessment as detailed in Sections 4.5.5 and 4.5.6 are as follows:-

- Risk to Property - This is outlined in Table 2 and given the assessed existing and proposed site conditions, has been assessed as **Very Low to Low Risk,** provided construction is in accordance with the recommendations of this report, including cut batters, engineer designed and certified retaining structures and drainage above and below the batters and walls.
- Risk to Persons - This is outlined in Table 3 and based on that assessment the risk to persons from the identified hazards was assessed as **Acceptable.** As above, construction will need to be in accordance with the recommendations of this report, including cut batters, engineer designed and certified retaining structures and drainage above and below the batters and walls.

5.0 ENGINEERING RECOMMENDATIONS

5.1 General Recommendations with Respect to Construction

Continued long term stability of the site is subject to development of the site taking place in accordance with the guidelines of this report and relevant Australian Standards and good building practices.

Based on the field observations, provided data, published hazard mapping and observed subsurface conditions, it is concluded that **provided the development of the site is undertaken using our recommendations, it would not adversely affect the slope stability conditions at this site or on neighbouring sites**.

This conclusion is qualified by the following provisions:-

- Any earthworks should be designed and constructed in accordance with sound and proper engineering principles.
- Likewise, the future construction should be planned designed and constructed in accordance with sound and proper engineering principles, and more specifically in accordance with good hillside construction practices (refer Appendix F).
- All construction works to be carried out in accordance with the recommendations of this report.
- The access and drainage infrastructure must be properly and effectively maintained, to ensure that all stormwater is intercepted and controlled. This requirement extends to all owners and occupiers being aware that ongoing maintenance of the site drainage is essential for continued site stability. **If the site is not properly maintained, the long-term stability of the site may become compromised.**

The following recommendations should be adopted:-

- All engineering works should follow the appropriate codes i.e.:-
    - Footings for buildings - AS 2870 'Residential Slabs and Footings'.
    - Retaining structures - AS 4678 ‘Earth Retaining Structures’.
- Vegetation - where possible, the prompt re-establishment of ground cover on the completed slopes or cut or timed batters should be undertaken to reduce the risk of surface scour during and following rainfall. Where not possible, other forms of surface protection should be adopted.

5.2 Civil Works

5.2.1 Excavatability Comments

Based on geotechnical knowledge of excavations/earthworks on projects in the local area and the findings of the investigation, the following comments can be made on excavation characteristics:-

- Bulk Works
  - Excavation in the soil and upper 1m, or so, of the weathered rock is expected to be possible using a bucket on a medium size excavator.
  - Excavation further into the weathered rock may be possible using a single tyne ripper on an excavator.
  - Below these levels and possibly to final bulk excavation level, a medium to large excavator using hydraulic rock hammers may be required.
  - In areas of shallower, medium to high strength rock, specialised tools, e.g., rock breakers may be required.
- Trenching
  - Trench excavations in the soils and upper 1.0m, or so, of the weathered rock should be within the capacity of a medium size backhoe or small excavator.
  - Below these levels a larger excavator would be required for excavation further into the weathered rock.
  - In areas of shallower, stronger rock, specialised tools, e.g. rock breakers, may be required.

5.2.2 Batters

General

Based on the provided drawings, temporary cut batters are expected to be required around three sides of the proposed garage structure, supported by engineer designed and certified retaining structures in the long term. Batters along the eastern side may extent to or near the boundary line. Where batters may not pass this boundary line, other options such as a bored pier wall may need to be considered. Additional design details can be provided for this option if required.

Design Batter Angles

Maximum batter angles for different material types are outlined in Table 5 for the un-surcharged cut batters less than 3m high on the site.

TABLE 5 DESIGN MAXIMUM BATTER ANGLES (Slopes up to 3m high)

Material	Short Term	Long Term[1]
Clayey Gravel Soils	45 degrees (1V:1H)	26 degrees (1V:2H)
Weathered Rock XW[2]	45 degrees (1V:1H)	26 degrees (1V:2H)
Weathered Rock DW or better[2]	60 degrees (2V:1H)	45 degrees (1V:1H)

Notes:
1. These values assume no seepage. If seepage is present the recommended angles would need to be significantly reduced, or the use of dewatering considered.
2. Subject to inspection by experienced geotechnical engineer/engineering geologist.

Note values above are maximum values in the case of soils. Flatter angles may be considered for ease of maintenance or if required by Council.

Where surcharges (e.g. footings, live loads, etc.) are located within H (height of batter) of the top of the batter, then some reduction in design angle **will need to** occur.

5.2.3 Scour Protection and Erosion Control

It is essential that any permanent batters be suitably protected from erosion and scour by appropriate drainage and the establishment of ground cover and shrubs, etc. Lot/pavement/driveway runoff should not be allowed to discharge directly across the batters without suitable scour protection.

If runoff velocities are sufficiently high, following cutting and construction of batters, erosion of the exposed soil and fractured very low to low strength rock could occur.

Landscaping, top soiling and seeding or hydromulching on cut areas steeper than 1:5 (11 degrees) is suggested so that protection could be provided until such time as the grass cover is established.

Drain outlets would also need to consider the effects of scouring runoff water. These will need to be concrete lined near each entry and exit, with grassing further out.

The seed selection for the hydromulch will depend on the season when placed, but in any case, should include a rapid growing seasonal variety that could rapidly establish itself until such time as the regular grass variety is established.

5.3 Foundation Recommendations

5.3.1 High Level Footings

Based upon the provided drawings and site investigation, high level footings are expected to be utilised for the proposed garage structure. This includes retaining wall footings and pads/strip footings for slab on ground construction. The expected subgrade over the majority of the garage bulk excavation is very dense clayey gravel and extremely weathered rock.

Footings should penetrate any fill and residual soils and found a minimum of 300mm into the weathered rock. All footings should found in the same material types.

An allowable bearing capacity of 400kPa is available for high level footings founded in the extremely weathered rock profile.

Where necessary, deeper footings may be made up with mass concrete poured to the underside of the footings, or alternatively, footings may be constructed over mass concrete filled, backhoe excavated pedestals.

5.3.2 Design Considerations

<u>Articulation</u>

It is recommended that any masonry walls be articulated. This articulation may be achieved by the use of full height (footings to eaves) openings or vertical construction joints at regular intervals. Guidelines on articulation are contained in the Cement and Concrete Association's Technical Note 61, 'Articulated Walling'.

Footings Adjacent to Excavations

Temporary Excavations

As a guide where footings are located adjacent to temporary excavations (e.g., services trenches), the footings should extend to base a minimum of 200mm below the trench/wall base level for 1.0m out from the trench/wall. Beyond 1.0m the footings should be taken a minimum of 200mm below an imaginary line drawn up at 45° from the trench/wall base level. Figure 4 refers.

Note the long-term effect of the footing on the services should be considered. These requirements do not override minimum footing levels.

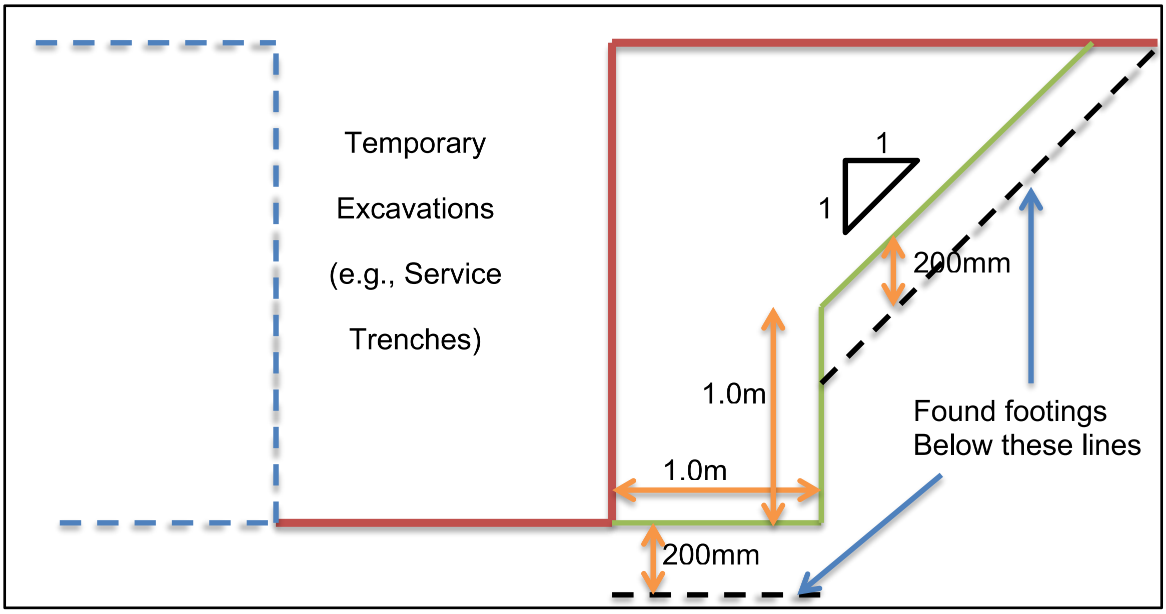

FIGURE 4

Other Excavations/Retaining Walls Not Designed for the Footing Surcharge

As a guide, where footings are located adjacent to any excavation not designed for the footing surcharge (e.g., retaining walls or services sensitive to additional surcharges) the footings should extend to base a minimum of 200mm below the trench/wall base level for 1.0m out from the trench/wall. Beyond 1.0m the footings should be taken a minimum of 200mm below an imaginary line drawn up at 26° from the base level of the excavation. Figure 5 refers.

These requirements do not override minimum footing levels. Note in theory there will still be some lateral load from the surcharge but this is likely to be approximately 10%-15% of the vertical load. If there is any doubt a detailed assessment should be undertaken.

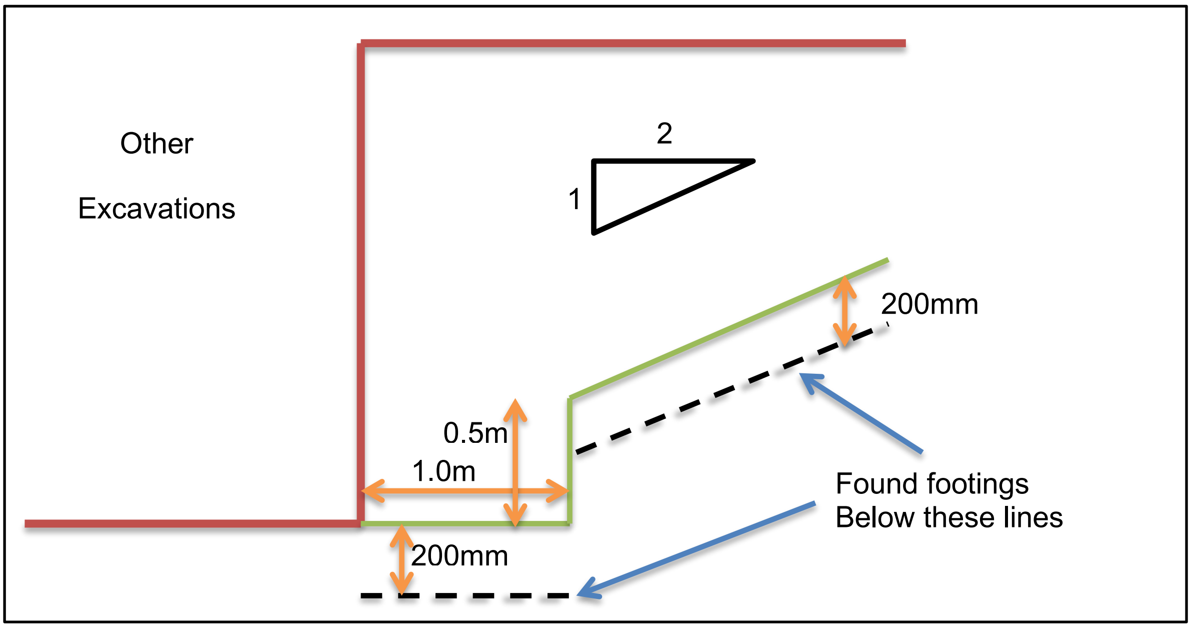

FIGURE 51

5.3.3 Construction Considerations

It is recommended that inspections be undertaken by an experienced and qualified geotechnical engineer or engineering geologist following footing excavations and during pier excavations to confirm the adequacy of the founding material. **Inspections should be carried out prior to placement of reinforcing steel and ordering of concrete.**

5.4 Retaining Walls

5.4.1 General

It is understood that retaining walls will be required for the proposed development. It is expected that the maximum retained height will be up to 3.0 metres. The lateral earth pressure distribution that affects the retaining walls on the site will depend upon the following parameters:-

- In-situ and backfill material properties.
- Design water regime at the rear of the wall.
- Wall and cut geometry.
- Surcharges affecting the wall.
- Wall type.

5.4.2 Design Requirements

It is recommended that any retaining structure is designed in accordance with AS 4678-2002 'Earth Retaining Structures'. Section 3 of this code outlines the design requirements for these retaining structures which specifically includes both Ultimate and Serviceability Limit Modes, it is recommended that the retaining structures be assessed for each of these modes.

Note the following sections provide general recommendations with respect to some of these limit modes however it doesn't provide a detailed assessment in full accordance with the above-mentioned modes for AS 4678 as this will require additional information. A detailed assessment can be undertaken of the geotechnical aspects of the Limit Modes for the proposed retaining structures once details of the wall have been provided. **Of particular importance is the assessment for Limit Mode U5 Global Failure for the proposed walls.**

5.4.3 Pressure Distribution

The following situations should be considered:-

- For cantilever walls, which allow some movement at the top, (i.e., at least 0.005H in clays) the active case (K_a) applies with a triangular distribution in both the short- and long-term situations.
- For cantilever walls which cannot tolerate this movement, the at rest case (K_o) applies with a triangular distribution in both the short- and long-term situations.
- For structurally braced walls the wall design should be checked for both a trapezoidal (clay soils - short term conditions) and triangular distribution with K_o values (long term conditions).

The pressure distributions as referred to above are shown in Figure 6. The parameters selected for use in the figure are dependent on the preconstruction geometry of the face being retained. Where the material has been cut using the recommendations as outlined in Section 5.2.2, the backfill parameters will control. Where a steeper angle is used the earth parameters will control.

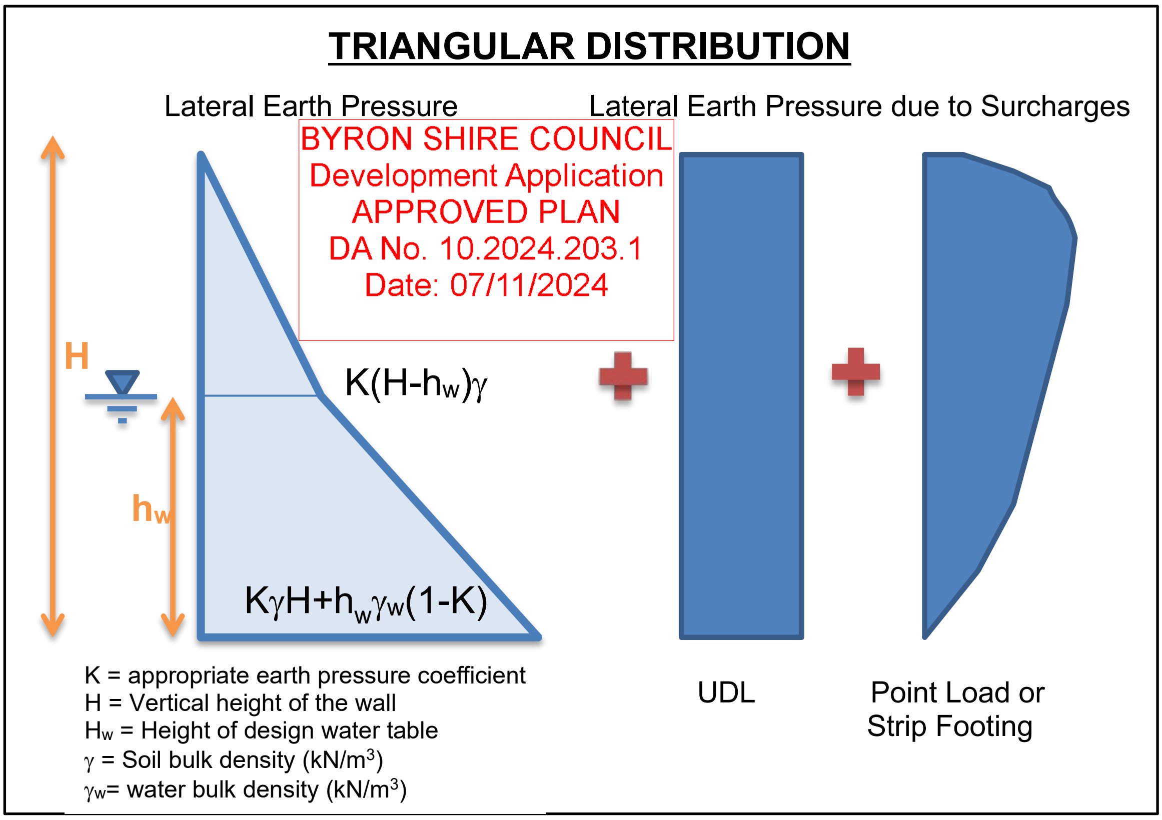

FIGURE 6

The lateral pressure distributions shown in Figure 6 include hydrostatic pressure and show typical pressure distributions due to surcharge loadings. It is recommended that where the retaining walls are expected to be surcharged (e.g., by footings, traffic loads, sloping ground surface, etc.)

Soil Surveys Engineering should be contacted to provide a recommended lateral earth pressure distribution.

Parameters (unfactored) for assessment of lateral earth pressures are outlined in Table 6.

TABLE 6 RETAINING WALL DESIGN PARAMETERS (UNFACTORED)

Material	Density (kN/m^3)	Earth Pressure Coefficient Vertical Wall			Long Term Drained ϕ (°)
		K_a	K_o	K_p	
Compacted Fill	19	0.36	0.53	2.8	28
Gravel Back Fill[2]	20	0.27	0.43	NR	35
Dense Gravel	19	0.25	0.41	3.9	36
Weathered Rock	21-23	0.27	0.43	3.7	35

Notes:-
1. NC = Not Considered, NR = Not recommended.
2. Drainage gravel.

The above recommend values are based on a drained model but has assumed a cohesion value of 0kPa for all materials. Whilst testing will indicate a value more than 0kPa there is some concern that in the long term a very low cohesion would be more appropriate for design. Should an undrained model be more appropriate, values can be provided.

5.5 Wastewater Disposal

Appropriate drainage provisions are essential in any development. Adequate subsoil and surface drainage should also be incorporated in any batters and access driveway/platform area construction, as well as any retaining wall construction.

Wastewater (sewer and/or stormwater) from future structures should be piped to storage tanks or an approved Council discharge location (we understand that the site will be fully sewered). Discharge onto the allotments is not recommended.

Other usual treatment options associated with Good Hillside Practice, as outlined in Appendix F of this report should also be adopted. Some of these are implicit or assumed by the analysis. For example, provision of surface water drainage measures (**such as a lined drain across the top of retaining walls and batters**), properly engineered retaining walls and structure foundations. Such treatment options should be clearly stated as part of the risk management requirements.

5.6 Continued Maintenance

As with all developments, continued maintenance of the proposed site works is essential to maintain the assessed risks outlined in this report. Maintenance should include but not be limited to the following:-

- Maintaining clear drainage paths for surface water flows.
- Maintaining and promoting revegetation of exposed batter areas.
- Regular maintenance checks to ensure any damage is repaired in a timely manner.

6.0 CONSTRUCTION INSPECTIONS

It is recommended that any retention system installation, footing excavations and cut batter slopes should be inspected by Soil Surveys Engineering Pty. Limited or a duly qualified Geotechnical Engineer.

Should subsurface conditions other than those described in this report be encountered, Soil Surveys Engineering Pty. Limited should be consulted immediately, and appropriate modifications developed and implemented if necessary.

7.0 LIMITATIONS

We have prepared this report for the use of **Giovanni D'ercole and appointed Consultants**, for design purposes in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has not been prepared for use by parties other than **Giovanni D'ercole and appointed Consultants**; it may not contain sufficient information for purposes of other parties or for other uses. Please note that any third party relying on the information contained in this report for any purpose whatsoever does so entirely at its own risk, and any duty of care to that third party is excluded.

Any interpretation or recommendation given by Soil Surveys Engineering shall be understood to be based on judgement and experience and not on greater knowledge of the facts than the reported investigations would imply. The interpretation and recommendations are therefore opinions provided for our Client's sole use in accordance with the specific brief. As such they do not necessarily address all aspects of ground behaviour on the subject site. Information provided by others has been taken in good faith, but no liability can be accepted for information provided by others.

Your attention is drawn to 'Appendix A', 'Notes Relating to this Report'. Interpretation of factual data given in this report is based on judgement, not a greater knowledge of facts other than those reported.

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

In the event that conditions encountered on site during construction appear to vary from those expected from the information contained in the report, the Company strongly recommends that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, after the event. Should Soil Surveys Engineering not be notified or if this notification is delayed, then Soil Surveys can not be held responsible for the effect that any variation has on any aspect of the development.

Soil Surveys Engineering consider that a documentation review service (during the design phase and prior to construction) to verify that the intent of geotechnical recommendations is properly reflected in the design, along with construction inspections, forms a very important component of the geotechnical engineering design service/process.

The geotechnical review ensures geotechnical risks to our Client and their project are minimised at the design and tender stage of the project. Further, with Soil Surveys Engineering being commissioned to carry out geotechnical construction inspections, an opportunity at the time of construction to confirm any assumptions made in the preparation of the report and allow the effect of any normally occurring variation in ground conditions to be assessed with respect to construction becomes available.

The above statements are not intended to reduce the level of responsibility accepted by Soil Surveys Engineering in accordance with our commission, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in doing so and the risks they accept should they decline to have Soil Surveys Engineering carry out a geotechnical documentation review and geotechnical construction inspections.

It is highly recommended that the Client avail themselves of these review and inspection services; our standard rates will apply.

G. BURKITT
SENIOR ENGINEERING GEOLOGIST

C.P. JOHNSON (RPEQ 7052)
PRINCIPAL GEOTECHNICAL ENGINEER

For and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

BYRON SHIRE COUNCIL
Development Application
APPROVED PLAN
DA No. 10.2024.203.1
Date: 07/11/2024

APPENDICES

APPENDIX A

NOTES RELATING TO THIS REPORT

INTRODUCTION

These notes are provided by Soil Surveys Engineering Pty Limited (the Company) to complement the geotechnical report in regard to classification methods and field procedures. 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 information 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 information 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 at the time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

Soils - The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726-2017 (Geotechnical Site Investigations), where appropriate. 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 dominant particle size and behaviour as set out in AS 1726-2017.

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, shear vane, laboratory testing or engineering examination. The strength terms are defined in AS 1726-2017 Table 11.

Non-cohesive soils are classified on the basis of relative density usually based on insitu testing or engineering examination (see AS 1726-2017 Table 12).

Rocks - Rock types are classified by their geological names (AS 1726-2017 Tables 15 to 18), together with descriptive terms regarding weathering (AS 1726-2017 Table 20), strength (AS 1726-2017 Table 19), defects (AS 1726-2017 Table 22), etc.

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 sample disturbance, (information on strength and structure).

Undisturbed samples are taken by pushing a thin walled sample tube, usually 50mm diameter (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 shear strength, volume change potential 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.

SAMPLE STORAGE – SOIL, ROCK AND WATER SAMPLES

Soil samples (not subject to testing) are not stored beyond a period of 90 days of taking or receiving said soil sample. Rock core (not subject to testing) is not stored beyond a period of six months of taking or receiving said rock core.

Should any party require that soil samples (not subject to testing) be stored beyond 90 days, or rock core (not subject to testing) be stored beyond six months, please contact Soil Surveys Engineering.

Water samples (not subject to testing) are not stored beyond a period of seven days of taking or receiving water samples.

TEST LOCATIONS

Test locations (e.g. boreholes, CPT's, test pits etc.) were based on available access at the time of testing. Test locations may have been shifted if access was not suitable.

Unless noted otherwise, accuracy of test locations are to the accuracy of hand held GPS equipment.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application.

Test Pits - These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to approximately 3.0m for a backhoe and up to 6.0m for an 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 augers can occur on a variety of materials such as hard clay, gravel or rock fragments and does not necessarily indicate rock level.

Continuous Spiral Flight Augers - The borehole is advanced using 75mm to 300 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the augers. Information from the drilling (as distinct from specific sampling) is of relatively lower reliability due to remoulding, inclusion of cuttings from above or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table has a lower reliability than augering above the water table. Various drill bits are attached to the base of the augers during the drilling. The depth of refusal of the different bit types can provide information as to the strength of the material encountered. Generally the 'TC' bit (a tungsten carbide tipped screw type bit) is used.

Wash Boring - The borehole is usually advanced by a rotary bit with water or fluid pumped down the hollow drill rods and returned up in the space between the rods and the soil or casing, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration. More accurate information on soil strata is gained by regular testing and sampling using the Standard Penetration Test (SPT) and undisturbed thin walled tube samples (U50).

Mud Stabilized Drilling - Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilize the borehole. The term "mud" encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from regular intact sampling (e.g. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling - A continuous core sample is obtained using a diamond or tungsten carbide tipped core barrel. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable method of investigation. In rocks, NMLC coring (nominal 52 mm diameter) 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 CORE LOSS. The location of losses is determined on site by the supervisor. If the location of the loss is uncertain, it is placed at the top end of the run, when the core is placed in a storage tray and recorded on the log.

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. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" - Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm, the upper 150 mm being neglected due to possible disturbance from the drilling method. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued at a reduced penetration.

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

4, 6, 7 N = 13

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

15, 30/40mm

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

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, it is noted on the borehole logs.

A modification to the SPT test is where the same driving system is used with a solid 600 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 SPT are shown as "N_c" on the borehole logs, together with the number of blows per 150 mm penetration.

Cone Penetration Tests - Test Method - Cone Penetration Tests (CPT) are carried out in accordance with AS 1289 Test 6.5.1-1999, using an electrical friction-cone penetrometer.

The test essentially comprises the measurement of resistance to penetration of a cone of 35.7 mm diameter pushed into the soil at a rate of 10-20 mm per second by hydraulic force. The resistance to penetration is recorded in terms of pressure on the end area of the cone (cone resistance, q_c, in MPa) and friction on the side of the 135 mm long sleeve immediately above the top of the cone (friction resistance, f_s, in kPa). These forces are measured by electrical transducers (strain gauges) within the cone device. The ratio between friction resistance and cone resistance is also calculated as a percentage, i.e.-

$$\text{Friction Ratio (FR)} = \frac{\text{Friction Resistance, } f_s \text{ (kPa) x 100}}{\text{cone resistance, } q_c \text{ (kPa)}}$$

The friction ratio, FR, is generally low in sands (less than 1% or 2%) and generally higher in clays (say 3% or more). The interpretation of sandy clays, clayey sands and material with a high silt content is more difficult, but intermediate values (between 1% and 3%) would be expected. Highly organic clays and peats generally have a friction ratio in excess of 5%.

Static cone data is recorded in the field on disc for later presentation using computer aided drafting.

The equipment can be operated from any conventional drill rig. A total applied load in the range of 4 to 10 tonnes is required for practical purposes, although lighter loads may be used. The cone penetrometers are available with various capacities of cone resistance ranging up to 100 MPa for general purpose investigations, while a range of 0 to 10 MPa can be used where more sensitive investigations of soft clay are required.

The cone resistance value provides a continuous measure of soil strength or density, and together with the friction ratio, provide very useful indications of the presence of narrow bands of geotechnically significant layers such as thin, soft clay layers or lenses of sand which might otherwise be missed using conventional drilling methods.

The lithology of the encountered soils is interpreted from static cone data and is generally presented on the static cone log sheets.

It is important to note that the lithology is interpreted information and is based on research by Schmertmann (1970), Sanglerat (1972), Robinson and Campinalli (1986), modified to suit local conditions as indicated by borehole information and laboratory testing.

As soils generally change gradually it is sometimes difficult to accurately describe depths of strata changes, although greater accuracy is obtained with the static cone compared with conventional drilling. In addition, friction ratios decrease in accuracy with low cone resistance values, and in desiccated soils. As a result, some overlap and minor discrepancies may exist between static cone and nearby borehole information.

Portable Dynamic Cone Penetrometers - Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 100mm increments of penetration.

The DCP comprises a Cone of 20 mm diameter with 30 degree taper attached to steel rods of smaller section.

The cone end is driven with a 9 kg hammer falling 510 mm (AS 1289 Test 6.3.2). The test was developed initially for pavement subgrade investigations, and empirical correlations of the test results with California Bearing Ratio have been published by various Road Authorities. The Company has developed their own correlations with Standard Penetration tests and Density Index tests in sands.

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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 lower permeability soils, it may enter the hole slowly or perhaps not at all during the time the hole is open.
- A localized 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 bailed out of the bore and mud must be washed out of the hole or "reverted" if water observations are to be made.

More reliable measurements can be made by use of standpipes which are read after stabilizing at periods 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 (e.g. 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 determine 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 important 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 Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms and the attached explanatory notes summarize important aspects of the Laboratory Test Procedures adopted.

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. The information provided in Soil Surveys Engineering reports is opinion and interpretation and not factual. The client/contractor increases their risk by not retaining the person who authored the geotechnical report, to carry out site inspection and review (overseeing role) during construction, to confirm opinion and interpretation expressed in the report is accurate. Where the report has been prepared for a specific design proposal the information and interpretation may not be relevant if the design proposal is changed. If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. Since the test sites in any exploration represent a very small proportion of the total site and since the exploration only identifies actual ground conditions at the test sites, even under the best circumstances actual conditions may vary from those inferred to exist. No responsibility is taken for:-

- Unexpected variations in ground and/or groundwater conditions.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of other persons.
- Any work where the company is not given the opportunity to supervise the construction using the Companies designs/recommendations.

If differences 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 expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, well after the event.

Extreme events including but not limited to the results of climate change, e.g. flood levels above previously identified levels, beach scour or erosion beyond normal expectations (as identified by local authorities) extreme rainfall events, war, espionage, sabotage may result in different conditions between time of investigation and time of construction.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Construction Contracts (1987)”, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances, where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a 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.

REVIEW OF DESIGN

Where major civil or structural developments are propose <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer. We would be happy to assist in this regard as an extension of our investigation commission. Construction drawings should be reviewed by Soil Surveys Engineering, with sufficient time to allow changes if required, prior to inspections. Otherwise Soil Surveys Engineering reserves the right to refuse to carry out inspections.

SITE INSPECTION

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

i. Site visits during construction to confirm reported ground conditions
ii. Site visits to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, the stability of a filled or excavated slope; or
iii. Full-time engineering presence on site.

In the vast majority of cases it is advantageous to the principal for the geotechnical engineer who wrote the investigation report to be involved in the construction stage of the project.

The geotechnical engineer cannot take responsibility for variations in encountered conditions, where he is not given the opportunity to review plans for the proposed development with sufficient time to allow review and make changes to the proposed development if required, and where he is not given the opportunity to inspect the site and oversee construction methods with regard to site conditions with sufficient time to observe all relevant site conditions and operations.

RESPONSIBLE USE OF GEOTECHNICAL INFORMATION

Recommendations in our report are for design purposes only and provided on the basis that inspections are carried out to allow finalisation of opinions and recommendations contained in our report.

The geotechnical investigation consisting of field and laboratory testing has been carried out to indicate typical conditions by indicating conditions and parameters at the specific locations of boreholes/test pits. Subsurface conditions are indicated at these locations only and the inference of conditions between or away from these locations (interpolation and extrapolation) involves a certain degree of risk. Persons inferring such conditions or carrying out such inferences should do so with a degree of caution and conservatism which is commensurate with the consequences of the risk of error.

Estimates of volumes based on our findings require interpolation and extrapolation between test locations and as such may be significantly different from actual volumes.

APPENDIX B

SITE INVESTIGATION MAPS

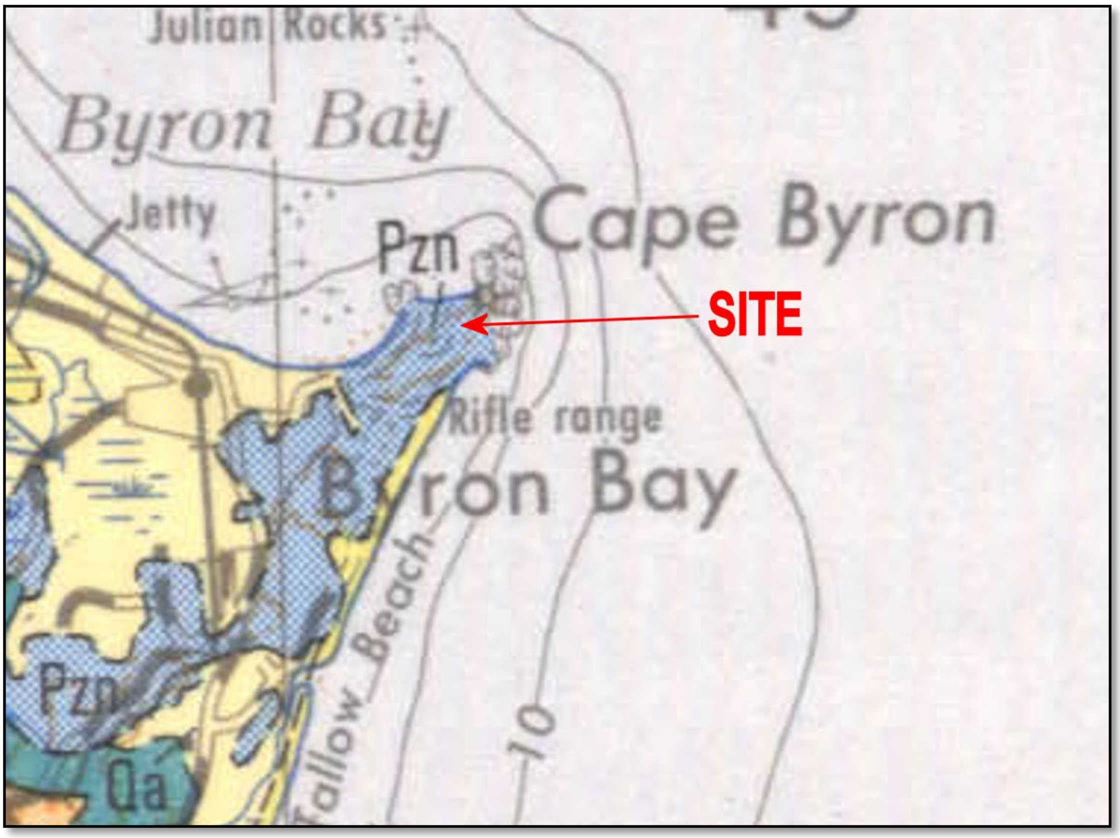

FIGURE B1 - REGIONAL GEOLOGY, TWEED HEADS GEOLOGICAL MAP

Qa – River gravels, alluvium, sand, clay

Pzn - Neranleigh-Fernvale Beds: Greywacke, slate, phyllite, quartzite

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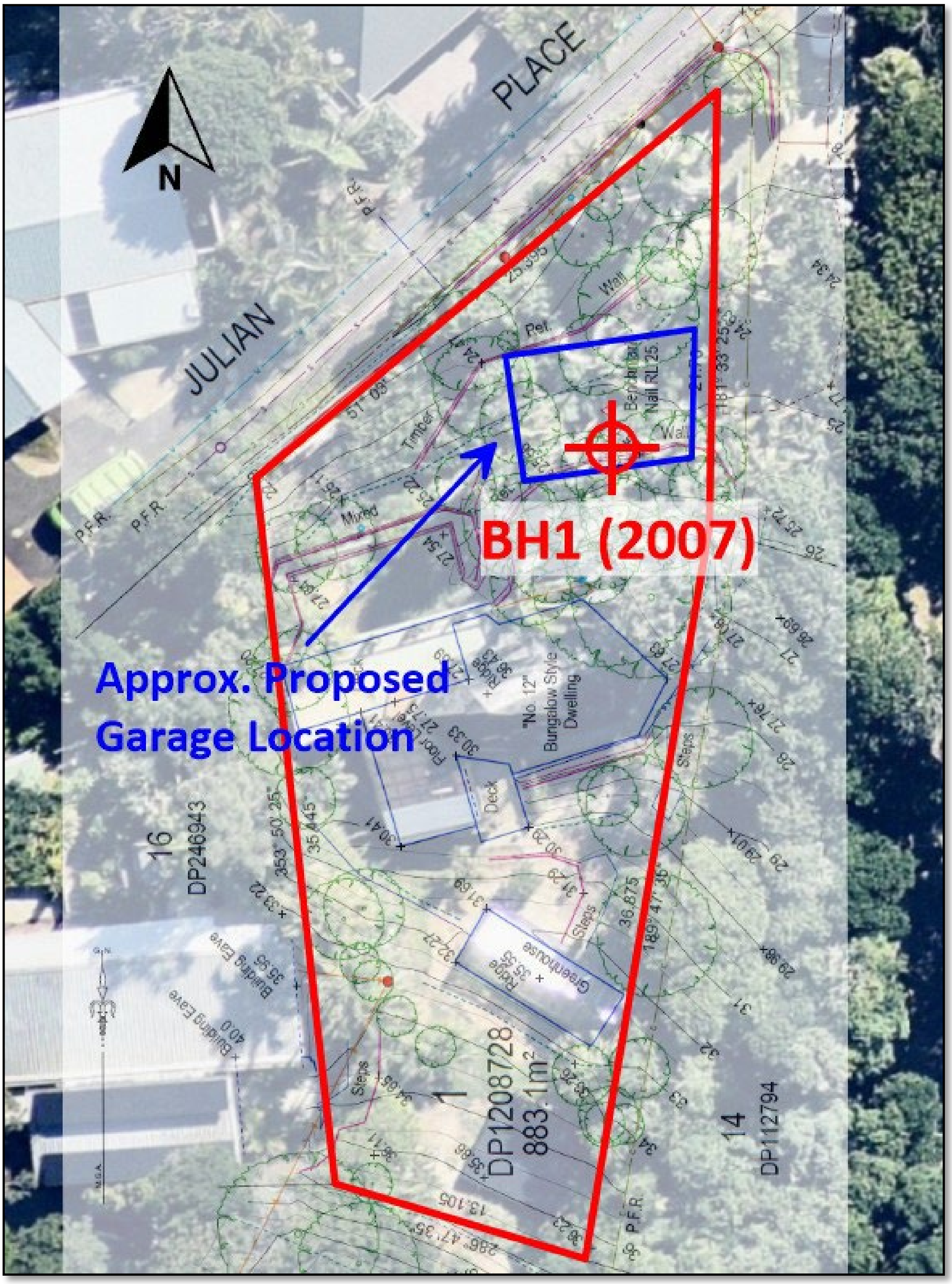

FIGURE B2 – SITE PLAN - APPROXIMATE BOREHOLE LOCATION FROM 2007 INVESTIGATION

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APPENDIX C

PHOTOGRAPHS

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PHOTO C1 - SITE LOOKING SOUTH SOUTHWEST LONG NORTHERN FRONTAGE OF JULIAN PLACE

PHOTO C2 - SITE LOOKING SOUTHWEST ACROSS SUBJECT SITE

PHOTO C3 – SUBJECT SITE LOOKING NORTHEAST TOWARDS EASTERN BOUNDARY

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APPENDIX D

BOREHOLE LOG

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Consulting Geotechnical engineers RPECQ No. 195
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Buddina Q 4578
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BOREHOLE RECORD SHEET

Borehole Number : 1

Project Number : 205-5614

Project Name : Proposed Multi level Dwelling

Location : 12 Brownell Dr., Byron Bay

Client : Kavanaugh Consulting Engineers

Date : 22/01/2007

Page : 1

Easting : Northing : RL :

Logger : BM Driller : md Drilling Rig : Gemco HP7

Drilling Method (V, TC, WB, RR, NMLC, Casing)	Depth	Graphic	Description	Weathering	Strength Estimated (VW W MS S VS)	DEFECT SPACING (mm) (20 60 200 600)	Rec (%)	RQD	Samples and Remarks
TC	0.15		Clayey Sandy GRAVEL (GC) Very dense, fine to coarse size, grey brown, fine to coarse grained sand, low plasticity fines, moist.						
TC	1.00		Clayey Sandy GRAVEL (GC) Very dense, fine to coarse size, orange brown, fine to coarse grained sand, low plasticity fines, moist.						
TC	1.20		SANDSTONE (XW) Weak, orange brown mottled light grey, fine to coarse grained sand.						
TC	3.00		SILTSTONE (XW) Weak, orange brown mottled grey, moist.						SPT 23,30/110mm SPT 30/30mm
NMLC	6.00		METASILTSTONE (DW-SW) Strong, dark grey stained orange brown on defects, trace of fine sand, with very close to closely spaced 20-40deg. inclined rough weathered defects.	DW SW DW-SW SW DW SW			100 99	33 36	3.06m;J20;u,v,o,l 3.1m;J40;s,r,o,l 3.15m;J75;s,r,o,l 3.2m;J70;c,r,o,l 3.33m;J40;s,r,o,l 3.43m;J40;s,v,o,w 3.53m;J20;s,v,o,w 3.72m;J45;p,r,o,l 3.8m;J25;s,r,o,l 3.91m;J60;s,r,o,l 4.03m;J50;c,r,o,l 4.14m;J30;s,v,o,w 4.25m;J40;p,r,o,l 4.32m;J60;s,r,o,l 4.4m;J40;s,r,o,z 4.43m;J45;p,r,o,l 4.49m;J25;u,v,o,z 4.55m;J70;c,r,o,l 4.62m;J80;s,r,o,l 4.67m;J70;s,r,o,l 4.73m;J40;s,r,o,z 4.84m;J35;u,v,o,z 5.0m;RJ75;u,r,c,w 5.06m;J70;u,v,o,z 5.09m;J45;p,r,o,w 5.2m;J40;p,r,o,w 5.25m;J20;u,v,o,z 5.39m;J25;u,v,o,z 5.62m;J5;u,v,o,z 5.85m;J30;u,v,o,z 5.95m;J50;s,v,o,z 5.99m;J30;u,v,o,z
			Borehole Terminated 6.00m						

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0

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COMMENTS
1) Groundwater not observed.

Approved : [illegible]
Date : 12 / 3 / 07

ISSUE No. 1.1 08/10/97 RS008A

Borehole 1, 3.00m to 6.00m

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APPENDIX E

LRM2007 – PRACTICE NOTE GUIDELINES – RISK MATRIX

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

RISK LEVEL IMPLICATIONS

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

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

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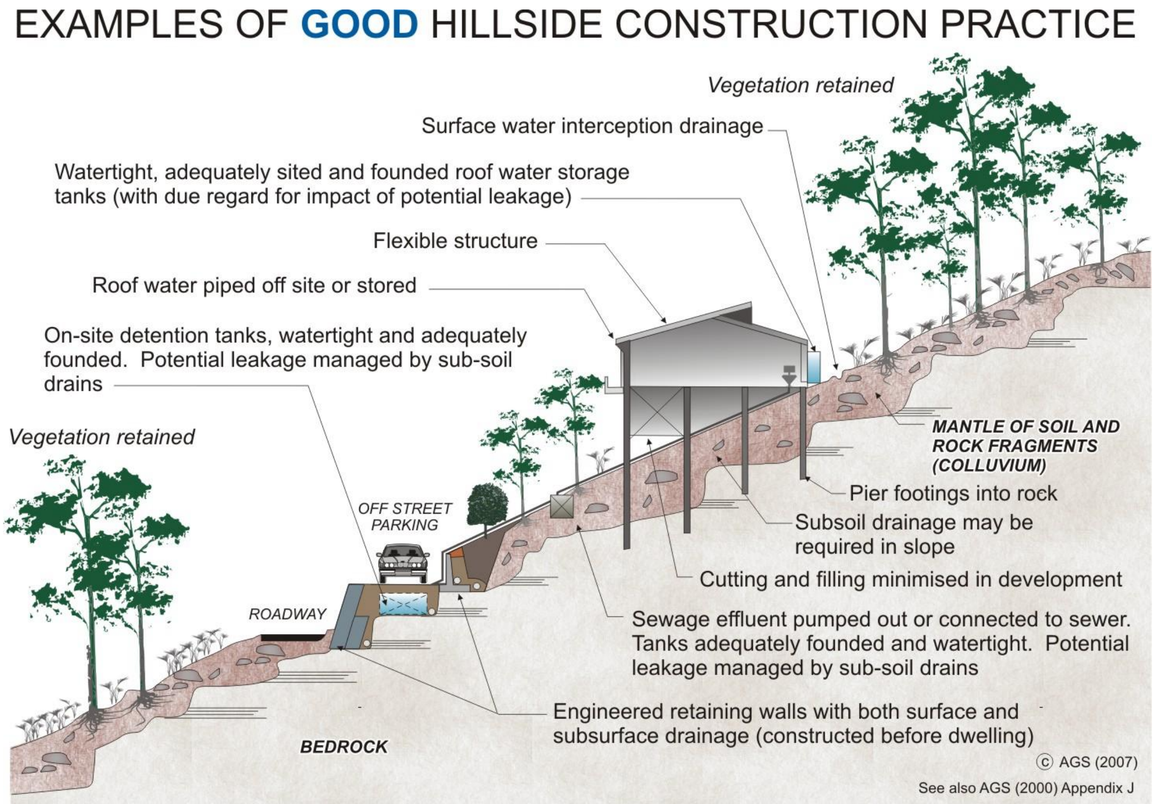
APPENDIX F

HILLSIDE CONSTRUCTION GUIDELINES

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.



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 in 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 fulfil 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.

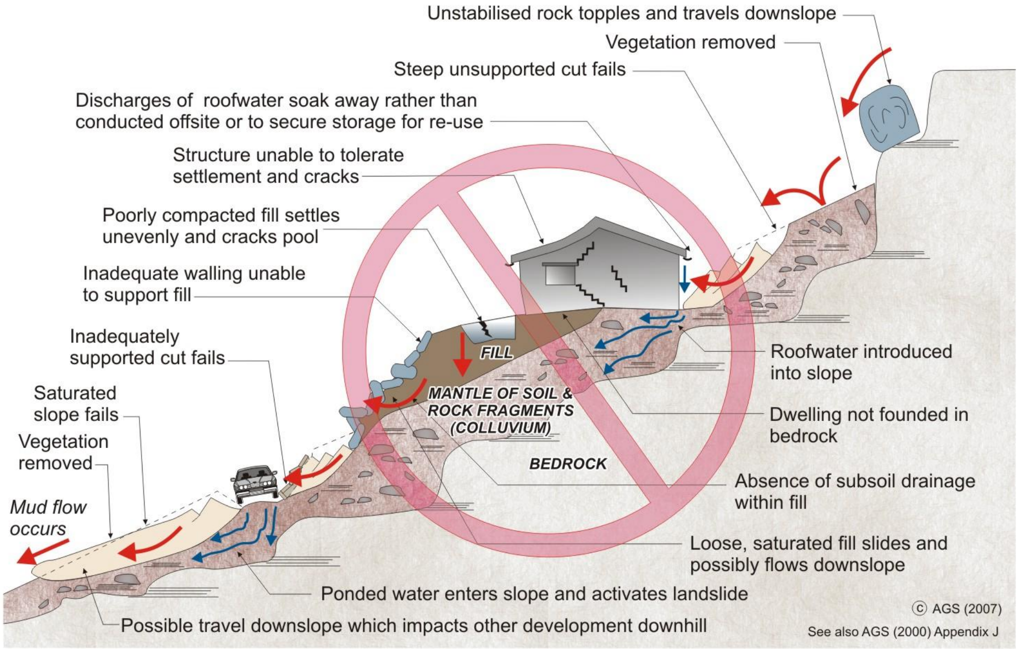
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing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

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF **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 soak 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 herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

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

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

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

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

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- 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.